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Bambang Bakri
M. Asad Abdurrahman

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WELCOME MESSAGE

Dear Colleagues,

In this opportunity of the 2nd Makassar International Conference on Civil Engineering (MICCE 2015), on behalf of the organization committees, we would like to welcome all of you to attend this prestigious conference in Makassar, Indonesia.

The MICCE 2015 compiles 87 papers that have been submitted and accepted for presentation in plenary session and parallel technical session. The presentation is divided into several sessions, such as water resources, structural and material engineering, geotechnical and geo-environmental engineering, transportation and urban planning, and construction management. In addition, invited speaker from well known universities and government officials also present their technical and research papers.

We sincerely thanks to all member of the scientific committee who have contributed idea and time to support the organization of this conference. We would like also to thank all our colleagues in the organizing committee that having given hard efforts to prepare this conference.

Finally, we wish this conference will have a great success; all participants will deepen their knowledge and experience during the technical sessions and will widen their networking with the colleagues in the same fields of research. We do hope you will have a sweet memory in Makassar.

Yours Sincerely,

Dr. Rudy Djamaluddin
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EFFECT OF HORIZONTAL DRAIN ON SLOPE STABILITY

Muhammad Mukhlisin[1], and Nurul Aini Binti Abd Aziz[2]

ABSTRACT: Slope failure usually occurs when soil particles are unable to build a strong bond with each other and become loose and straggling because of the presence of water. Water pressure weakens the ties between the particles and they tend to slip. Therefore, this study focused on the use of horizontal drains to reduce water entry and control the ground water level as a method of slope stabilization. The main objective in this study was to analyze the effect of horizontal drains on slope stability. Information on slope condition during the landslides which happened at Precinct 9, Putrajaya, Malaysia was used for analytical simulation. Seep/W and Slope/W analyses were carried out with GeoStudio version 2007 software. Slopes with and without horizontal drains were then compared in terms of groundwater level and factor of safety (FOS) values. Scenarios were created for seven types of soil for case-by-case analysis: residual, clay, silt, loam, sandy loam, sandy clay loam, and silt clay loam. The effect of real-condition rainfall was studied. The results revealed that when a drain was installed on a slope, the groundwater level dropped immediately and the safety factor of the slope increased.

Keywords: *Horizontal Drain, Slope Stability, Groundwater Table, Factor of Safety (FOS).*

INTRODUCTION

A landslide is a serious geological hazard. It can cause damage to property of billions of dollars and is responsible for thousands of deaths and injuries every year. The development on hill slopes area has increased in occurrence of landslides in developed areas in Malaysia. Landslide is one of major natural disasters in mountainous areas in Malaysia. Landslides often occur particularly during the rainy season even in high-populated areas (Mukhlisin et al. 2010). Examples of such landslides include the landslide at Precinct 9 Putrajaya on March 22, 2007, the Bukit Antarabangsa tragedy at Ulu Klang on December 6, 2008 and that at Rumah Anak Yatim Al-Takwa, Batu 14, Hulu Langat, Selangor on May 21, 2011, when 16 of the 25 people buried by the landslide died.

Puncak Setiawangsa suffered a landslide on December 28, 2012 when a 50-meter-high retaining slope collapsed. Since 1973, nearly 600 deaths caused by landslides have been reported in Malaysia. The Malaysian Works Ministry reported that 20% of the 4700 registered slopes of all federal roads across the country face critical markers, including the risk of collapse, and need to be maintained as soon as possible (Ibrahim, 2010). In December 2010, the Public Works Department of Malaysia recorded about 58 residential areas in Kuala Lumpur and Selangor at risk of experiencing a landslide. All this housing is located in risky hilly areas. Rapid building development has affected the strength and stability of natural slopes, which have been replaced with man-made slopes requiring regular maintenance.

The construction of the drainage system on the slopes is the most important thing. Drainage systems can be combined with cover crops or the surface of the slope can be covered with concrete so that water runoff does not seep into the ground. Underground drainage systems are intended to reduce the entry of water and control the earth water level. The system can be built by deflecting water to flow into building systems. This method can be used when there is a possibility of groundwater causing slippage. However, this method is limited to deep flow of 10 meters (Khairul, 2005).

In addition, a drainage system can be built so that the drill drainage is inclined in the slope. It is also known as horizontal drain construction. Horizontal drains can be defined as holes drilled into a slope and cased with perforated metal or slotted plastic liner to drain out groundwater (Royster, 1980). The purpose of horizontal drains is to reduce groundwater pressures within slopes and behind retaining walls and to modify the seepage pattern to reduce its destabilizing effect. They are commonly used to ameliorate such problems. Drainage tunnels and vertical well systems have also been used to improve slope stability in the past (Craig & Gray, 1985). In a study conducted by Ahmed et al (2011), landslide that occurred on March 22, 2007 at 4:30 am near the staff quarters at Precinct 9, Putrajaya is probably due to the increase of groundwater level in the slope. There was 40 numbers of 24-meter-deep horizontal drains were installed at the foot of the slope with a length between 8 to 24 meters. Overall the study notes that the installed horizontal drains has affected the decline in water levels and stabilize groundwater levels despite the continuous rainy weather conditions. Inclinometer measurements generally show no additional creep movement on the hillside near the residence since the construction of drains in residential location situated in the ski Vail, Colorado, USA. As a result of

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the decline in the piezometer and monitoring results further, horizontal drains were found to enhance significantly the stability of slopes and slope creep movements are reduced (Pik, 2008).

There are two basic design approaches for drain installation in slopes which serve different purposes (Martin & Siu, 1996). First, designed drains are installed to reduce groundwater pressures so that a specified factor of safety or margin of stability is achieved. The drains can be used alone or in combination with other stabilization measures. Design calculations incorporating a model of how the drains will affect the groundwater regime are normally carried out. Conventional monitoring of designed drains involves measurement of groundwater levels and drain flows before and after installation of the drains. Second, prescriptive drains are installed to provide some additional but unquantified improvement of slope stability by reducing groundwater pressures. Such drains are not critical in terms of achieving a specified factor of safety or margin of stability. The effect of the drains is not normally included in design calculations.

Drains should function satisfactorily without clogging in the long term with the minimum of maintenance. In order to assess the effectiveness of horizontal drains, regular maintenance of drains is desirable (Pik, 2008). Drains should be regularly inspected for any blockage at the outlet, and any undesirable vegetation removed. Clogging of drains by soil ingress, plant roots, and mineral precipitates can be treated by flushing with a water jet. If drains contain removable inner liners, these can be withdrawn or replaced by water flushing.

Literature which provides reliable understanding of the effectiveness of horizontal drains on a variety of soil types is very limited. Most of the horizontal drain designs in Malaysia are based on the locally available machinery without much technical justification. Comparison of horizontal drain effectiveness for various soil types has not been performed previously. Therefore in this paper, the effectiveness of horizontal drains as a slope stabilization method is analyzed for seven types of soil in two cases of rainfall, as it is important to understand the influence of rainfall on seepage and slope stability. Comparison of the slope safety factor (FOS) and its relation to groundwater level gradient for several basic types of soil could be of relevance to the design of horizontal drain systems in the future. Effective horizontal drain systems could be a major factor in decisions regarding the type of slope stabilization built.

THEORY AND METHODS

GeoStudio 2007

GeoStudio 2007 is a software-based finite element for analyzing soil deformation and stability in geotechnical projects. Geotechnical applications require computerized simulation of nonlinear models and the behavior of soil properties over time. Since land has many phases, a suitable method is required to determine hydrostatic and non-hydrostatic water pressure. However, modeling of soil is an important issue for many projects, involving geotechnical modeling of its structure and the reactions of that structure. GeoStudio 2007 software comes with its own character and is divided into several parts representing complex geotechnical structures. In this study, modeling and analysis used its SEEP/W and SLOPE/W components. SEEP/W was used to analyze the pore-water pressure. SLOPE/W analysis was then used to obtain the factor of safety for the studied slopes (Geo-slope International, 2007).

The equation for the flow of water in the soil used in SEEP/W software to complete the analysis of two-dimensional transient and seepage is:

$$m_w{}^2 \gamma_w \frac{\partial h_w}{\partial t} = \frac{\partial}{\partial x}\left(-k_{wx}\frac{\partial h_w}{\partial x}\right) + \frac{\partial}{\partial y}\left(-kw_y\frac{\partial h_w}{\partial y}\right) + q \tag{1}$$

where :

$m_w{}^2$	=	Slope soil-water relationship characteristic curves
γ_w	=	Unit weight of water
h_w	=	Total head
k_{wx}	=	Soil permeability coefficient of water in the x-direction
k_{wy}	=	Soil permeability coefficient of water in the y-direction
q	=	Flux boundary
t	=	Time

Two important soil parameters used in the analysis of SEEP/W are the function of the coefficient of permeability of the soil and the soil characteristic curve-water contact (SWCC). A shear strength equation for unsaturated soil used in the slope stability analysis, SLOPE/W, is shown as Equation 2. This equation generally combines the shear strength of soils as a result of negative pore-water pressure and the matrix suction of unsaturated soil. To obtain the factor of safety (FOS) in the analysis of slope, SLOPE/W, the Morgenstern-Price method was used.

$$\tau = c' + (\sigma_n - u_w)\tan \emptyset' + (u_a - u_w)\tan \emptyset^b \quad (2)$$

where:

τ	=	Unsaturated soil shear strength
c'	=	Cohesion
σ_n	=	Total normal stress
u_a	=	Pore air pressure
$\emptyset$	=	Internal friction angle
u_w	=	Pore-water pressure
$(u_a - u_w)$	=	Matrix suction

SEEP/W and SLOPE/W

SEEP/W is a finite-element CAD software product for analyzing groundwater seepage and excess pore-water pressure dissipation problems within porous materials such as soil and rock. Its comprehensive formulation allows analyses of problems ranging from simple, saturated steady-state problems to sophisticated, saturated/unsaturated time-dependent problems. SEEP/W can be applied to the analysis and design of geotechnical, civil, hydrogeological, and mining engineering projects. It can model both saturated and unsaturated flow, a feature which greatly broadens the range of problems that can be analyzed. In addition to traditional steady-state saturated flow analysis, the saturated/unsaturated formulation of SEEP/W makes it possible to analyze seepage as a function of time and to consider such processes as the infiltration of precipitation.

The transient feature allows analysis of such problems as the migration of a wetting front and the dissipation of excess pore-water pressure. It can model almost any groundwater problem, including dissipation of excess pore pressure after reservoir drawdown, changes in pore-water pressure conditions within earth slopes owed to infiltration of precipitation, mounding of the groundwater table beneath water retention structures such as lagoons and tailing ponds, effect of subsurface drains and injection wells, drawdown of a water table because of pumping from an aquifer, and seepage flow quantities in excavations.

SLOPE/W is the leading slope stability CAD software product for computing the factor of safety of earth and rock slopes. This software can effectively analyze simple and complex problems for a variety of slip surface shapes, pore-water pressure conditions, soil properties, analysis methods, and loading conditions. Using limit equilibrium principle, SLOPE/W can model heterogeneous soil types, complex stratigraphic and slip surface geometry, and variable pore-water pressure conditions using a large selection of soil models. Slope stability analyses can be performed by using deterministic or probabilistic input parameters.

Stresses computed by finite element stress analysis can be used in addition to limit equilibrium computations to achieve the most complete slope stability analysis available. With a comprehensive range of features, it can be used to analyze almost any slope stability problem encountered in geotechnical, civil, and mining engineering projects. SLOPE/W can model almost any stability problem, including natural earth and rock slopes, sloping excavations, earth embankments, berms at the toe of a slope, surcharges at the top of a slope, tension cracks, and unsaturated soil behavior.

Rainfall (infiltration)

In this study, the actual infiltration for real-condition analysis were chosen to ascertain horizontal drain effect. Figure 1 shows the actual infiltration data. The latter was taken from a previous study from Ahmed et al. (2011). In the Figure 1, most rain fall is on day 8 (May 20, 2007). On May 22, a landslide occurred near a government apartment at Precinct 9 in Putrajaya. This incident was taken as a reference study and the soil data used for the analysis.

Table 1 shows the cases and scenarios used in analysis. Table 2 shows the input data assigned to the residual soil and Table 3 the soil parameters used for the unsaturated hydraulic function in Seep/W for the other six soils. Two cases were included in order to obtain the output finding on comparison effectiveness of horizontal drain. These were :

1. Case 1 : slope with rain but without horizontal drain
2. Case 2 : slope with installed horizontal drain on FOS<1 (and assumed other than that was installed at day 10)

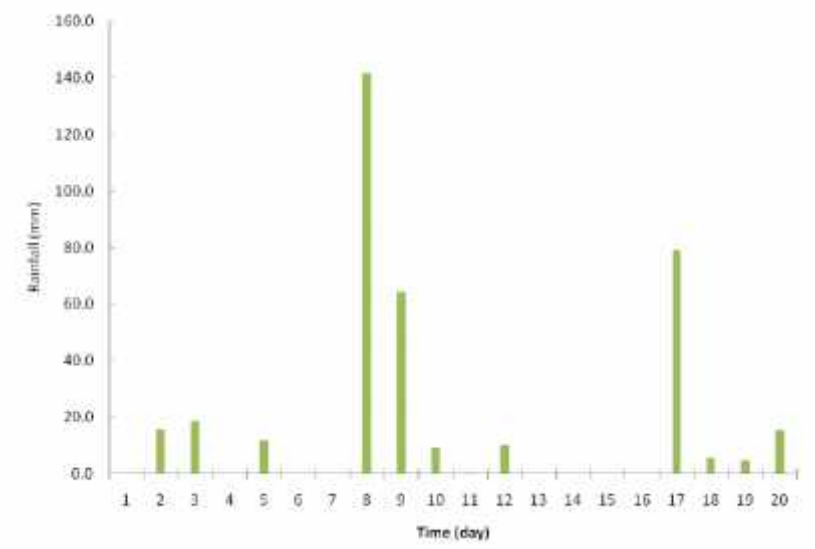

Figure 1 Actual infiltration

Table 1 Summary of cases and scenarios for analysis

Scenario	Soil Type	Actual Rainfall
1	Residual	Case 1, Case 2,
2	Clay (C)	
3	Silt (S)	
4	Loam (L)	
5	Sandy loam (sL)	
6	Sandy clay loam (scL)	
7	Silty clay loam (sicL)	

Table 2 Input data for 'Residual Soil'

Function Assigned	'Volume Water Content'		'Hydraulic Conductivity'	
Function Name	'Residual Soil'			
Type	'Data Point Function'			
Grain Size Data	LL	10% passing	60% passing	ɵs
	40%	0.006 mm	0.2 mm	0.6 m3/m3
Van Genuchten	ɵr		Ks	
	0.15 m3/m3		0.0864 m/days	

Source: Ahmed et al. (2011)

Table 3 Soil parameters for unsaturated hydraulic function

Soil Type	ɵs	ɵr	α	n	Ks (m/day)
Clay (C)	0.38	0.068	0.008	1.09	0.0480
Silt (S)	0.46	0.034	0.016	1.37	0.0600
Loam (L)	0.43	0.078	0.036	1.56	0.2496
Sandy loam (sL)	0.41	0.065	0.075	1.89	1.0608
Sandy clay loam (scL)	0.39	0.100	0.059	1.48	0.3144
Silty clay loam (sicL)	0.43	0.089	0.010	1.23	0.0168

Source: Carsel and Parrish (1988)

Analyses of the actual rainfall (real condition) with fluctuating flow patterns were conducted for Scenarios 1 to 7. Through case-by-case comparison of the effectiveness of horizontal drains for each scenario has been implemented based on initial groundwater levels obtained from the pre-analysis determination of groundwater levels.

Two layers of soil were used for this analysis, as shown in Figure 2. The first layer was assigned initially as residual soil and the second layer as schist. The first layer changed according to the type of soil selected (clay, silt, loam, sandy loam, sandy clay loam, and silt clay loam) but the second layer was fixed as saturated soil-schist.

RESULTS AND DISCUSSION

The results obtained from all the analyses were divided into two groups: (1) the effect of horizontal drains on steady rainfall and (2) the effect of horizontal drains on actual rainfall. All data were analyzed and comparisons performed through tables and graphs.

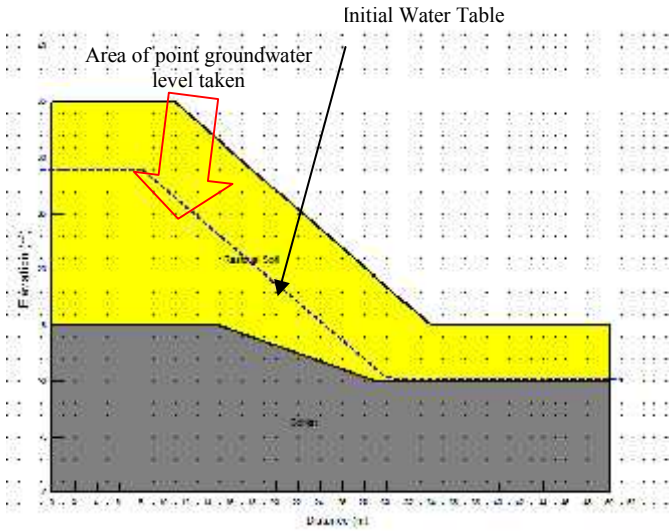

Figure 2 Diagram of slope layer

Figure 3 shows the current groundwater level used as initial condition, the initial groundwater table. Figure 4 shows the Van Genuchten soil water characteristic curve (SWCC) for the seven types of soil in the scenarios analyzed. The Van Genuchten function is often used to describe the soil water retention curve of unsaturated soil. This curve is also known as the soil water characteristic curve and it is defined as the relationship between water content and the suction of the soil. Figure 4 shows that suction from the sandy loam scenario curve was higher than for other soil types. Sandy loam seepage was the most rapid. Clay and silt clay loam exhibited the lowest suction.

In this case, the soil water content itself was the major factor responsible for the curve. Amount of water content within the pores of soil effected the capillary pressure is high. Once the soil high pressure, the soil at slope area will lose and tent to be collapse. However, if the metric suction of soil is high, the amount of water content within the pores can be controlled or drained to make sure that the groundwater level is lower. This in turn means that the soil in a slope area is more stable and the safety factor will give better protection from landslides.

A horizontal drain about 20 m long was used in this study to stabilize the slope by draining water to lower the groundwater table. The purpose of horizontal drains is to reduce groundwater pressure within slopes and behind retaining walls and also to modify the seepage pattern to reduce its destabilizing effect. It is commonly used to ease the problem. Underground drainage systems like horizontal drains are intended to reduce the entry of water and control the soil's water level.

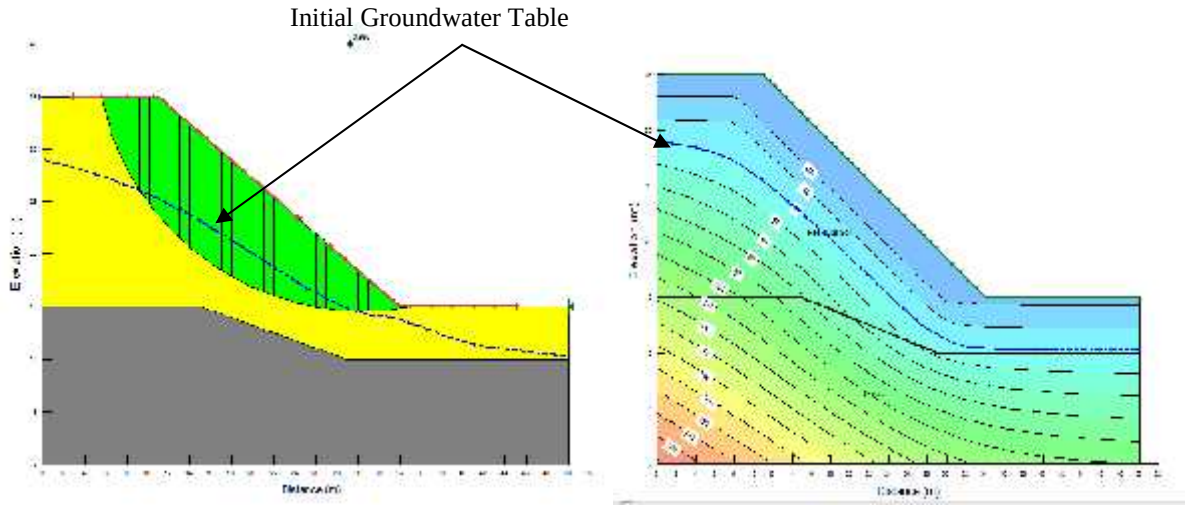

Figure 3 Current groundwater levels (initial condition)

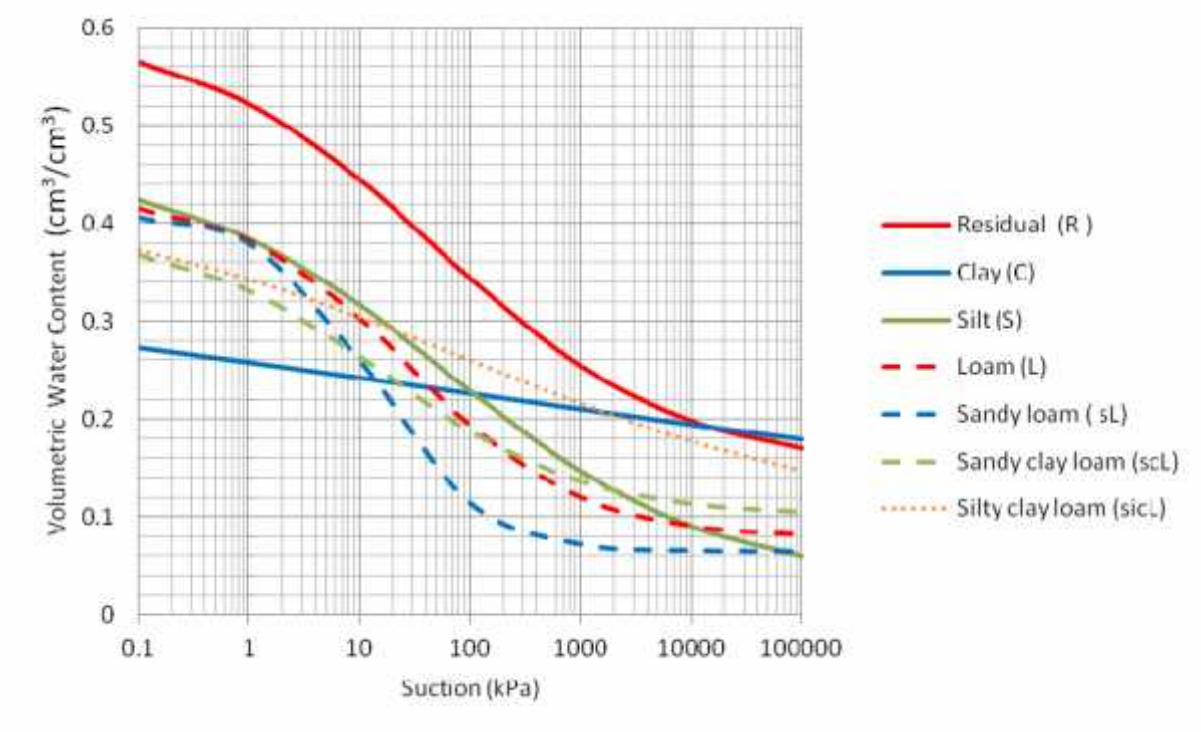

Figure 4 Soil water characteristic curves for seven types of soil

Effect of horizontal drains on actual rainfall (real condition)

The overall results for the effect of horizontal drains on actual rainfall (real condition) for seven scenarios are shown in Figure 5. In Figure 5a, the groundwater level of the whole scenario in Case 1 is similar to the groundwater level for the whole scenario in Case 2 for the analysis of steady rainfall. Groundwater level for scenario 5 (sandy loam) was the highest level recorded and the lowest groundwater level was found for scenario 7 (silt clay loam). Groundwater level for most scenarios was lower after one day's use of horizontal drains.

Figures 5b and 5d show the FOS for all scenarios in actual rainfall analysis. The FOS of slope in scenario 5 (sandy loam) decreased during extensive rainfall, but once a horizontal drain was installed, it increased and the slope did not collapse or fail. Figure 5d shows that silt had the highest FOS value, but before and after installation of the horizontal drain, the highest value was for sandy loam and the lowest for silty clay loam.

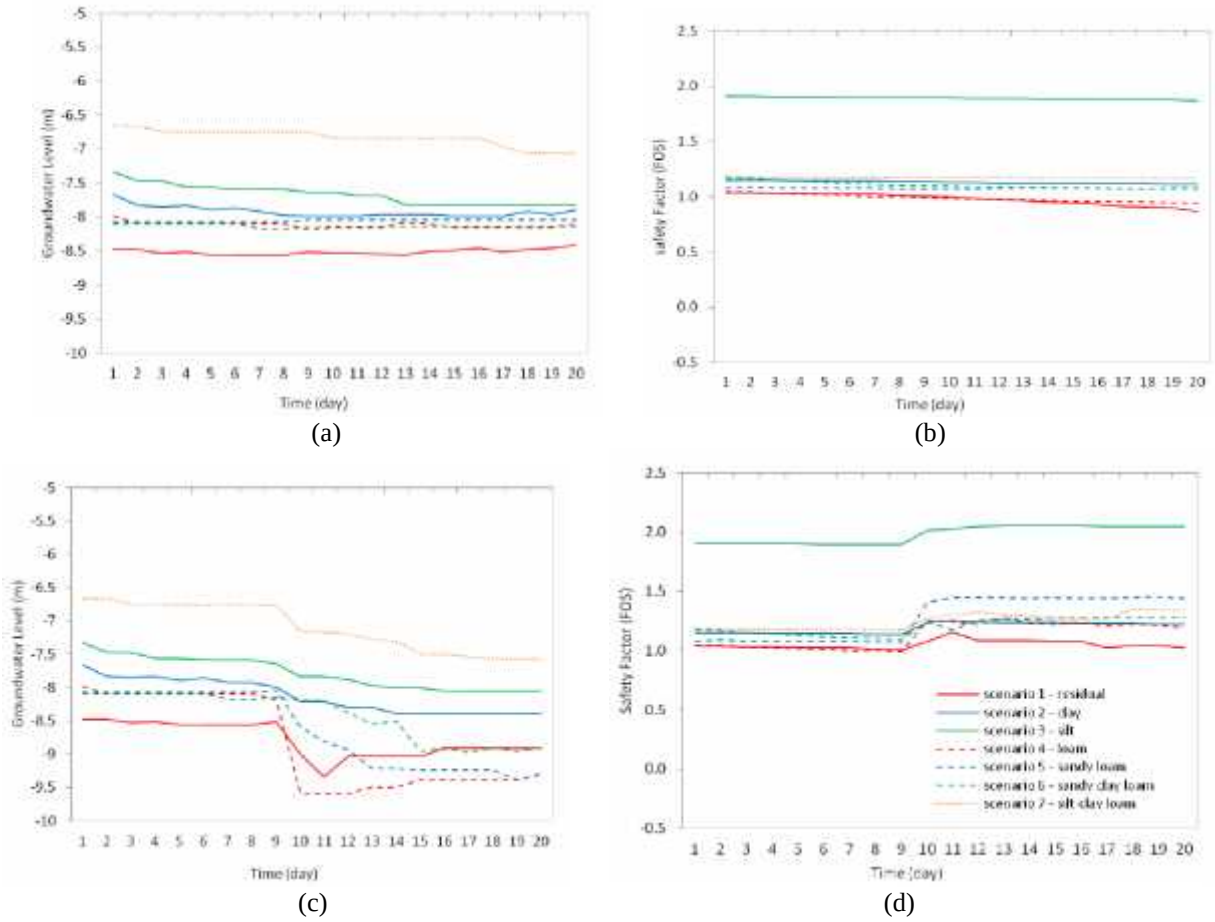

Figure 5 Actual rainfall: comparison of groundwater level (a) and factor of safety (b) for all scenarios in Case 1 and groundwater level (c) and factor of safety (d) for all scenarios in Case 2.

The use of horizontal drains in this analysis was intended to increase slope stability and reduce the risk of landslides. Horizontal drains influence the soil's capacity to drain excess water from earlier rainfall infiltration to ensure the bonds between soils are not loose in order to prevent instability. The water which causes seepage is associated with groundwater flow.

It is the most significant single factor which triggers landslide activity or slope instability. It is evident that horizontal drains have a salutary impact on the stability of a slope with increasing FOS. The decreased value of groundwater levels in Case 2 for the actual rainfall condition is shown in Table 4. In Table 4, available reaction groundwater level biggest decline was in sandy loam soil (SL) and the hardest bit is on silty clay loam soil type (sicL). Only loam (L) and silty clay loam (sicL) showed differences level significant impairment in day is up to the second day. In addition, a significant difference in groundwater level was seen until the third day.

Table 4 Decreased value of groundwater level in Case 2 – actual rainfall condition

Scenario	Soil Types	Groundwater Level (m) / day					
		Start	1	2	3	4	5
1	Residual	0	-0.49	-0.83	-0.52	-0.52	-0.52
2	Clay (C)	0	-0.22	-0.22	-0.31	-0.31	-0.40
3	Silt (S)	0	-0.19	-0.19	-0.24	-0.33	-0.36
4	Loam (L)	0	-0.55	-0.77	-1.17	-1.17	-1.17
5	Sandy loam (sL)	0	-1.40	-1.40	-1.40	-1.31	-1.31
6	Sandy clay loam (scL)	0	-0.06	-0.06	-0.24	-0.41	-0.36
7	Silty clay loam (sicL)	0	-0.04	-0.04	-0.04	-0.04	-0.04

The results obtained are closely related to the value of K, which was used in the analysis. The groundwater level of scenario 5, sandy loam, was the highest since the K of sandy loam is 106.08 cm / day. This parameter is higher than the K of silty clay loam of 1.68 cm / day. The movement of water into the ground is easy when K is high. Then the water flows in quickly and will affect the soil water capacity increase of existing and cause groundwater levels to be higher, and vice versa if the Ks is low. Therefore, groundwater level for scenario 5, sandy loam, showed a significant increase, higher than most other scenarios in Case 2.

Groundwater level in each scenario affected the safety factor of the slope. Some of the water created pressure diffusion in the soil and increased the weight of sliding mass. The formation of excess pore pressure then reduced the shear strength of the land. Analysis showed that when groundwater level rises, the safety factor of a slope will decline, and vice versa if the groundwater level is down. Both of these factors affect the stability of slopes. If the FOS of a slope is less than one, the slope is unstable and prone to the risk of slope failure which can cause landslides.

CONCLUSIONS

The study was carried out to find the effect of horizontal drains on slope stability. Use of the simulation software Seep/W and Slope/W from Geo-Slope 2007 allowed analysis of the impact of horizontal drains on the stability of a slope with seven different soils. Groundwater during the initial groundwater table slope collapse was obtained by using simulation software based on the data and parameters of the landslide which occurred on March 22, 2007. Comparison of the results of each case and scenario analyses were performed to determine the effect of horizontal drains on slope stability with a focus on groundwater level and the safety factor of the slope.

Analysis with Seep/W and Slope/W proved that horizontal drains lower the groundwater table and improve the slope FOS for all scenarios studied in a reasonable time period. The scenario 5 slope of sandy loam soil showed the greatest effect when a horizontal drain was used. Thus, it is evident that the horizontal drain is very effective for sandy loam soil slopes, offering good slope stabilization to prevent collapse during the rainy season. In terms of the impact of horizontal drains and rain from the highest scenario to the lowest scenario, scenario 5 (sandy loam) ranked first, followed by scenario1 (residual) and scenario 6 (sandy clay loam), scenario 3 (silt), scenario 2 (clay), scenario 4 (loam) and finally scenario 7 (silty clay loam).

From the results obtained, it can be concluded that horizontal drains can lower the water level in soil to drain excess water from slopes after high rainfall. The slope FOS increases from less than one, when instability can occur, to satisfactory figure promoting slope stability. Significant changes were detected when the horizontal drain was used in this research. Even in heavy daily rainfall, horizontal drains help to stabilize the slope. This study demonstrates that the presence of rain and horizontal drains strongly influence slope stability.

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OPTIMIZATION OF CALCITE PRECIPITATION AS A SOIL IMPROVEMENT TECHNIQUE

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ABSTRACT: A soil improvement technique that utilizes calcite precipitated as a cementing material is presented. Various combinations of chemicals and enzymes are mixed and the amount of precipitated calcite is evaluated to find the optimal result. Soil specimens are prepared in PVC mold and treated with optimally combined solution. The calcite distribution within the soil is evaluated by sampling the treated sand at different locations. Evolutions in the mechanical and the hydraulic properties of treated sand specimens are examined through unconfined compressive strength (UCS) and permeability tests, respectively. A precipitation ratio up to 70% can be obtained using a small amount of enzyme. The uniform distribution of the calcite precipitated is obtained. The mechanical test results show that even a small percentage of calcite precipitated within soils of interest bring about an appreciable improvement in the strength of the soils compared to that of untreated soils. The permeability of the improved specimens shows more than one order of magnitude smaller than that of the untreated specimens.

Keywords: *calcite, precipitation, distribution, permeability, soil improvement.*

INTRODUCTION

Grouting is used as a soil improvement technique in civil, geotechnical and mining engineering applications. The grouting solutions either displace the soil (i.e. compaction grouting), are mixed with the soil under high pressure (i.e. jet grouting), and/or permeate through the soil, which fills up the voids between the soil grains (i.e. chemical penetration or permeation grouting) (Karol 2003). Grouting methods have limitations such as they may be rather expensive, have limited injection distance (0.3 or 1.0 m for chemical grouting (Ivanov and Chu 2008)) or up to 2.5 m for super jet grouting, and have significant negative impact (Ivanov and Chu 2008).

In recent years, grout materials that produce cement materials mediated by microorganisms have been developed for soil permeability control and improvement (DeJong et al. 2010; Harkes et al. 2010; Whiffin, van Paassen, and Harkes 2007). Microbial Induced Calcite Precipitation, MICP, has been studied extensively for its possibility as a novel soil improvement technique. This technique utilizes the whole bacterial cells containing urease; e.g., *Sporosarcina pasteurii* to dissociate urea into ammonium and carbonate ions. The increase in compressive strength ranges from 0.2–12 MPa depending upon the amount of the calcite precipitated (van Paassen et al. 2010). In this MICP technique, the transport and the fixation of the bacteria of interest are significant issues for achieving a suitable level of improvement of the saturated porous media. Yasuhara et al. (2012) considers the reaction efficiency of the enzyme-reagent solution. However, the direct measurement of the amount of precipitated $CaCO_3$, and hence, an evaluation of the reaction efficiency, has not been carried out yet. They used the powdered urease enzyme and mixed it thoroughly with the sand prior to the sample preparation. The sample was treated with a solution of urea and $CaCl_2$. However, the grouting technique, in which the enzyme is mixed well within the soil samples prior to the injection of the solutions, is unsatisfactory because it may not be feasible for applications at real sites (Yasuhara et al. 2012). The enzymatic $CaCO_3$ precipitation technique was evaluated for in situ application in a larger scale. It was found that a noticeable precipitation ratio can be observed even with a 2 – 3 g/L of enzyme. The amount of precipitated calcite varied from 1.5 to 6.0% on the inner spherical portion, with a diameter of 30 cm, and the corresponding reduction in porosity varied from 2 to 7%. The amount of precipitated calcite may be enough to modify the mechanical properties of sandy soil (Neupane et al. 2013)

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In this work, the small amount of urease and reagent solution (urea + $CaCl_2$) is poured over the sand specimens from the top. The optimal combination of enzyme and reagent precipitation is utilized for improving small scale specimens. Distribution of calcite precipitated and improvement in the sand specimens are evaluated. The pouring technique approved for small scale tests are upgraded and applied to mid-scale tests. The calcite distribution within the specimen is evaluated by sampling the treated sand at several locations. Evolutions in the mechanical and the hydraulic properties of treated sand specimens are examined through unconfined compressive strength (UCS) and permeability tests, respectively.

EXPERIMENTAL METHODS AND RESULTS

Material

Urea, $CaCl_2$ and urease are the primarily materials in this work. Urea and $CaCl_2$, with claimed purity levels greater than 95.0%, were obtained from the Kanto Chemicals Co. Inc. Urease (020-83242, Kishida Chemical, Osaka, Japan), purified from the jack bean meal and with urease activity of 2,950U/g, is used in the bio catalytic dissociation of urea. Keisa #6 sand having e_{max}, e_{min}, and specific gravities of 0.899, 0.549, and 2.653, respectively, is used in this work. Expected reaction to obtain precipitated carbonates ions precipitates in presence of the calcium ion (Ca^{2+}) supplied in the solution is showed in **Eq. (1) – (3)**.

$$CO(NH_2)_2 + 2H_2O \rightarrow 2NH_4^+ + CO_3^{2-} \tag{1}$$

$$CaCl_2 \rightarrow Ca_2^+ + 2Cl^- \tag{2}$$

$$Ca_2^+ + CO_3^{2-} \rightarrow CaCO_3 \tag{3}$$

Precipitation Test

In this work, precipitation of calcite is evaluated directly in transparent test tubes. Varying combinations of urea-$CaCl_2$ and urease are mixed thoroughly and allowed to react in various curing times. 15 mL of a same molarity solution of urea-$CaCl_2$ with concentrations of 0.5, 1.0 and 1.5 mol/L, respectively are prepared separately. Various amounts of urease (1.0 – 5.0 g/L) are mixed in water for 2 minutes and filtered using filter paper (pore size of 11 μm) to remove the undissolved particles of urease. Urea-$CaCl_2$ solution and the filtered urease solution are mixed in total solution volume of 30 mL. The precipitation of $CaCO_3$ takes place after several minutes depending upon the solution concentration. After curing time (1 - 7 days) the solution is filtered with filter paper. The particles deposited on the filter paper and remaining in the tubes are dried to calculate the total amount of the calcite precipitated.

The precipitation ratios of various combinations of reagent and urease at various curing times are evaluated in several transparent tubes. **Fig. 1** shows a summary of the evaluated precipitation ratio. As is show in Fig 1(a), the reagent concentration of 0.5 mol/L has highest precipitation ratio. The precipitation ratio at small amount of urease (1.0 g/L) with a reagent concentration of 0.5 mol/L is more than 70%.

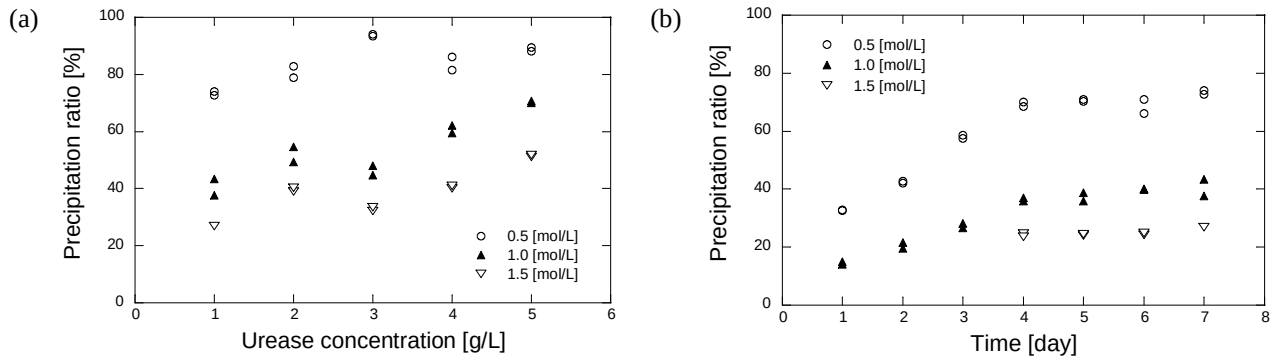

Fig. 1 (a) The evaluation of calcite precipitation at various reagent concentrations; (b) Relation between time and calcite precipitation ratio.

The precipitation ratio increases gradually and approaches the maximum 90% at the urease content of 3.0 g/L. In contrast, precipitation ratio of 1.0 mol/L and 1.5 mol/L of reagent concentration are still increasing at urease content of 5 g/L. The precipitation ratio approaches the maximum at 60% and 50% for 1.0 mol/L and 1.5 mol/L reagent concentration respectively. The small amount of urease 1.0 g/L will be applied in the next steps, because it is more economical. Fig. 1(b) represents the relation between curing time and precipitation ratio. The precipitation ratio of reagent concentration of 0.5 mol/L and 1.0 mol/L gradually increases until the curing time of 4 days, after that it is approximately constant. Meanwhile, the precipitation of the reagent concentration of 1.5 mol/L starts to appear an

fourth day and it is approximately constant until 7 days. This may be attributed to the insufficiency of the urease and hence the ureolytic activity for higher concentrations of urea.

PVC Cylinder Test

The optimal combinations of the precipitation tests are utilized in the sand specimens and the efficiency is evaluated. The PCV mid cylinders (10 cm in diameter and 20 cm in height) and the PVC small cylinder (5 cm in diameter and 10 cm in height) are used for preparing sand specimens. The prepared sand specimens are treated to evaluate the improved mechanical properties and calcite distribution. Firstly, the Keisa #6 sand is poured into the cylinders to obtain the relative density of 50% ±3. Secondly, the solutions selected are poured into the cylinders from the top. Injection number also varies from 1 – 3 times. The PVC cylinder tests take 5 days which should depend on the precipitation test results. After curing time the specimens are removed from the PVC cylinders.

The stiff samples obtained from the treated PVC cylinders are depicted in **Fig. 2**. The specimen is divided into 5 parts and distribution of calcite precipitated is evaluated by acid leaching process. In this process, the dried sand is weighed and washed with 0.1 mol/L of HCl several time until air bubbles no longer appear. The lost dry weight caused by acid leaching is evaluated and is assumed to be the weight of the calcite. The reaction taking place is showed in **Eq. (4).** The mechanical properties of the treated samples are examined by the unconfined compression test of the small cylinder. The experiment conditions for this test are listed in **Table 1**.

Fig. 2: The stiff sample obtained from PVC mid cylinder

The solution infiltrates and produces calcite precipitated that bind the soil particle and it causes changes the mechanical and hydraulic properties of the sand. **Fig. 3** represents distribution of calcite precipitated inside the sand. The distribution of calcite precipitated after one injection is almost uniform throughout the sample and the evaluated calcite content is more than 1%. As the injection number increases, the calcite precipitated increase noticeably. The calcite precipitated content is concentrated at the center, meanwhile, the amount at the top higher than at the bottom. This may be attributed to the plugging of the pores with the calcite precipitated. The plugging may restrict the flow inside the sample; therefore the precipitation take place before the solution reaches any father.

$$CaCO_3 + 2HCl \rightarrow CaCl_2 + CO_2 + H_2O \tag{4}$$

Distribution of porosity inside the sand is examined is depicted in **Fig. 4**, showing that one injection can reduce porosity uniformly. For the next injection, the decreasing of the porosity follow the tendency of calcite precipitated.

Table 1. Experimental condition of PVC cylinder test

Test Name *	Urease (g/L)	Urea – $CaCl_2$ (mol/L)	Volume of each injection (mL)	Number of injection
S-1.1	1	0.5	75	1
S-1.2	1	0.5	75	1
S-2.1	1	0.5	75	2
S-2.2	1	0.5	75	2

S-3.1	1	0.5	75	3
S-3.2	1	0.5	75	3
M-1	1	0.5	600	1
M-2	1	0.5	600	2
M-3	1	0.5	600	3

*) S : PVC small cylinder , M : PVC mid cylinder

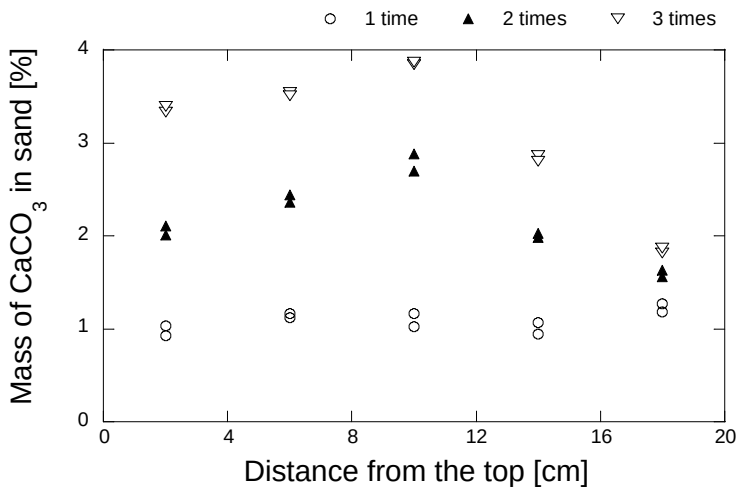

Fig. 3: Distribution of calcite precipitated inside the sand sample

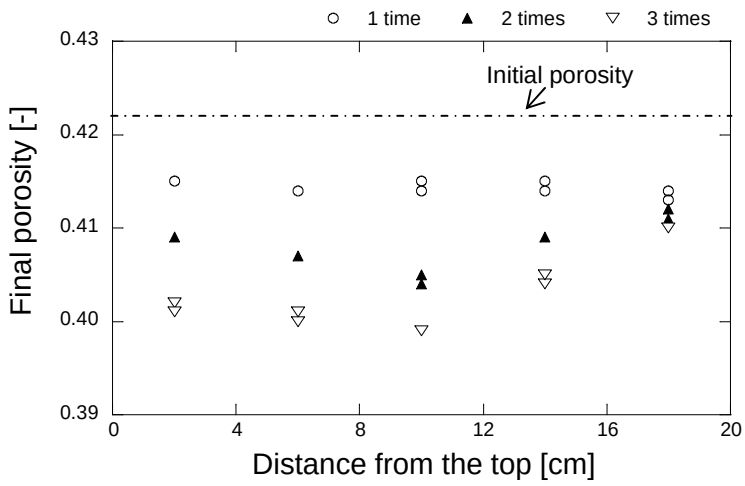

Fig. 4: Distribution of porosity in mid cylinder

A relation between the amount of calcite precipitated and UCS is depicted in **Fig. 5**. The UCS of the treated sand improves between 20–70 kPa when 1-3% of sand weight calcite precipitated takes place in the sand sample. This result is evaluated using the result shown in by (Matsuda and Shinozaki 2008), the strength more than 50 kPa is non-liquefied.

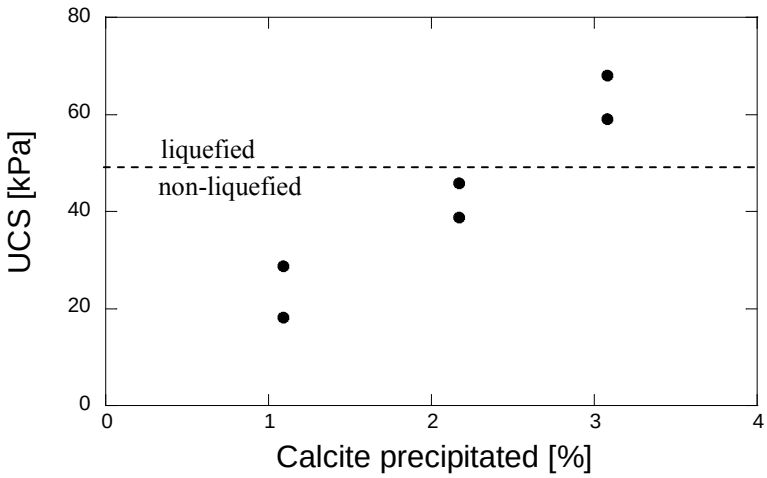

Fig. 5: Relation between calcite precipitated and UCS

Permeability Test

In this section, permeability tests are conducted to examine the effects of improvement on the permeability of the treated sand. The procedure of the sample preparation for the permeability tests is equivalent to the unconfined compression tests. It is the same in terms of the amount of Keisa sand, concentration of urease, urea and calcium chloride, the dimension of the samples and number of injection. The permeability tests are conducted by falling head method. The experiment conditions for this test are listed in **Table 2**.

Table 2. Experimental condition of permeability test

Test name	Urease (g/L)	Urea – $CaCl_2$ (mol/L)	Volume of each injection (mL)	Number of injection
P-0	-	-	-	-
P-1	1	0.5	75	1
P-2	1	0.5	75	2
P-3	1	0.5	75	3

As examine in the previous section, relation between the amount of calcite precipitated and hydraulic conductivity is depicted in **Fig.6**. The result shows that after one injection, the decrease measured in the permeability test is relatively significant, more than 50 %. This result be in accordance with permeability test by similar grouting materials (Yasuhara et al. 2012). In contrast, when the number of injection is increased, the increase of calcite precipitated does not significantly influence the change in permeability. This indicated that the small amount of calcite precipitation significantly reduce the permeability of the sand sample.

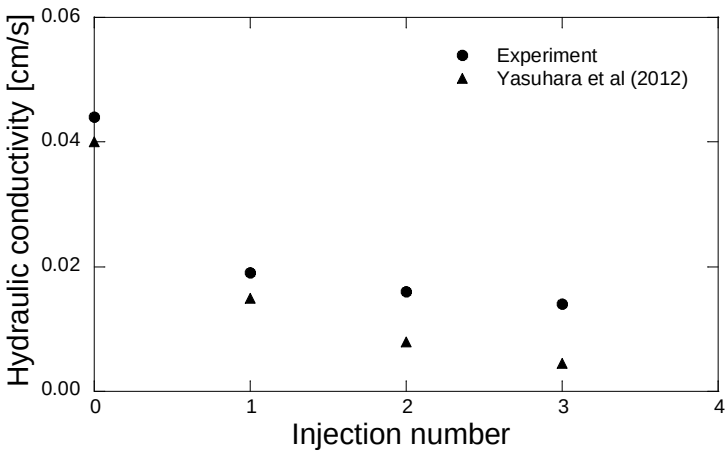

Fig. 6: Evolution of hydraulic conductivity at different injection number

CONCLUSION

The optimization of the precipitated calcite was evaluated for soil improvement technique. The precipitation ratio for various combinations of reagent solutions was evaluated. A precipitation ratio up to 70% can be obtained using a small amount of enzyme. The stiff samples obtained from the treated PVC cylinders showed the efficacy of this technique. The distribution of calcite precipitated after one injection is almost uniform throughout the sample. However, when number of injection is increased, calcite precipitated is concentrated at the center. The amount of precipitated calcite in PVC mid cylinder varied from 1% to 3%, and the corresponding reduction in porosity varied from 1 to 5%. The UCS test of the treated sand displays sufficient strength as grouting material to counteract soil liquefaction. One time injection can reduce the permeability more than 50%. However, large scale test using the same grouting condition and technique will be conducted and reported in the near future.

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EFFECTS OF DEGREE OF SATURATEION ON LIQUEFACTION RESISTANCE OF SAND IMROVED WITH ENZIMATIC CALCITE PRECIPITATION

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ABSTRACT: In recent years innovative methods of environmentally friendly and economically technique for liquefaction remediation are being developed. One of those technologies is microbial induced calcite precipitation (MICP). In MICP the microorganisms like soil bacteria are used to improve the mechanical properties of soils. The urease enzyme catalyses the hydrolysis of urea ($CO(NH_2)_2$) in a solution, and the carbonate (CO_3^{2-}) produced precipitates with the calcium ion (Ca^{2+}) as calcium carbonate crystals (calcite). The improvement in the liquefaction resistance of Toyoura sands with calcite precipitated was investigated through a series of triaxial tests in this study. Liquefaction strength curves correlating corresponding to DA=5% for treated and untreated sand were compared. The results show that the MICP can improve the liquefaction resistance of sand. The lower the degree of saturation, the higher the liquefaction resistance of treated sand in the same amount of calcite. Besides that, the greater the amount of calcite in the improved sand the stronger is the sand created.

Keywords: soil improvement, degree of saturation, calcite content, sand, triaxial test, liquefaction resistance.

INTRODUCTION

Past occurrences of earthquake were often followed by the liquefaction of soil which had caused severe damage to a number of structures. Traditionally, liquefaction mitigation measures such as various grouting methods using chemical additives (e.g. portland cement, lime, fly ash, bitumen) and mechanical methods (e.g. providing drainage ability, densification, and mixing of soils) are costly, energy consuming and composed of environmentally unfriendly materials (DeJong et al., 2010; Khodadadi et al., 2012). In recent years an innovative environmentally friendly method are being developed that utilizes precipitated calcium carbonate as a cementing material. The precipitation of calcium carbonate may be induced by the microbial metabolism (Nemati and Voordouw, 2003; Whiffin et al., 2007; DeJong et al., 2010; Harkes et al., 2010; Al Qabany and Soga, 2013).

This method, using the microbial metabolism, may be effective to strength soils, however, the evolutions in the mechanical properties induced by the microbial metabolism may not be straightforward enough to be controlled, because it may be impossible to constrain the extinction and/or the generation of living bacteria in natural environments. The research on calcium carbonate precipitation by bacteria has been mainly conducted using ureolytic bacteria. These bacteria indirectly produce precipitated calcium carbonate by a urease enzyme.

Yasuhara et al. (2012), on the other hand, employed the urease enzyme instead of using bacteria as a promoter for the hydrolysis of urea, which causes Ca^{2+} and CO_3^{2-} to precipitate as $CaCO_3$ and form into the void spaces and/or the surfaces of grains. Utilizing the enzyme itself is more straightforward than using bacteria, since the cultivation and fixation of bacteria (i.e., biological treatment) do not need to be considered.

Hayashi et al. (2010) conducted triaxial test on calcite precipitated sand to study liquefaction resistances and confirmed that this soil improving method significantly enhances not only the strength and stiffness of sand but the liquefaction resistance as well. In their tests, the specimens were prepared at saturated condition and then calcite was precipitated in the specimens. They observed sand surface with SEM and found that the precipitated calcite distributed more or less uniformly on the surface of grains and only a limited amount of calcite on the surface contributed to bond between particles. It clearly suggested that most portion of chemical regents injected into the sand were used to generate calcite on the grain surface which had nothing to do with mechanical properties in interest and only a limited amount of regents were used to bond between particles which directly contribute to improve the properties.

On the other hand, Okamura et al. (2012) has developed a technic to effectively desaturate a potentially liquefiable sand by injecting air directly into ground. They indicated that degree of saturation of sand around air injector lowered down to some 40% where sand was considered to be in the condition of the pendular saturation. In this unsaturated

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condition, pore water exists mostly at the contacts between particles. Based on these facts, one of the author, Okamura, came up with an idea that calcite precipitation at lowered degree of saturation may enhance efficiency of soil improvement. Much smaller amount of calcite precipitation may be enough to improve soil properties as compared to the previous studies. In this method, liquefiable ground may be desaturated by injecting air first and chemical regents are then inject. After allowing ample time for precipitation, air injection may be halted and degree of saturation becomes high again.

In this study, degree of saturation during calcite precipitation on the liquefaction strength of partially and fully saturated sand was investigated through a series of triaxial test.

EXPERIMENTS

Reagent (consisted of calcium chloride-C_aCl_2 and urea-$CO(NH_2)_2$) and urease enzyme were used in this experiment. The expected reactions for obtaining the precipitation of calcium carbonate-C_aCO_3 (calcite) are as follows.

$$CO(NH_2)_2 + 2H_2O \xrightarrow[\text{enzy}]{\text{urease}} 2NH_4^+ + CO_3^{2-}$$

$$C_aCl_2 \longrightarrow Ca^{2+} + 2Cl^-$$

$$Ca^{2+} + CO_3^{2-} \longrightarrow C_aCO_3 \downarrow \text{(cementation)}$$

Precipitatin test in test tube

Precipitation tests with test tube were conducted to know appropriate amount of both the urease and the regent to obtain desired calcite precipitated, as well as curing time needed to reach 100% precipitation of calcite. The combinations of the reagent concentrations and the amount of urease tested in this study are shown in Table 1.

Table 1 The Reagent Concentrations, the Amount of Urease in the Solutions, and the Expected Precipitation

Reagent Concentration (m/l)	Urease Amount (g/l)	Expected Precipitation (g/30ml solution)
0.2	2	0.6
0.3	2	0.9
0.5	3	1.5
	10	
1	3	3.0
	10	
	20	
1.5	25	4.5
	35	

Relationship between calcite precipitation ratio and curing time is plotted in Figure 1 at various content of urease enzyme (g/l) and concentration of reagent (m/l) consisted of urea and calcium chloride. For making sure the reproducibility of the materials used, two samples were prepared for each test condition. An increase in the urease amount effectively enhanced the precipitation ratio and accelerated the reactions. For example, 10 g/l urease content can only produce around 50% the precipitation ratio of 1 m/l reagent concentration along the curing time, while for 20 g/l urease level the 100% precipitation ratio had already been occurred nearby 36 hours of curing time. Contrastingly, the more the reagent concentration in the same urease level will produce the less the calcite precipitated. This is consistent with the result of the research conducted by Neupane, D, et al. (2013). The high concentration of reagent relative to urease content may restrict the performance of urease enzyme which may give a reduction in the amount of calcium carbonate precipitation. Urease amount implemented for the next experiment is based on the value resulting 100% or close to 100% precipitation of calcite with specific concentration of reagent.

Especially for urease amount of 2 g/l with reagent concentration of 0.2m/l and 0.3m/l, the observation was only conducted once when the curing time had already been three days.

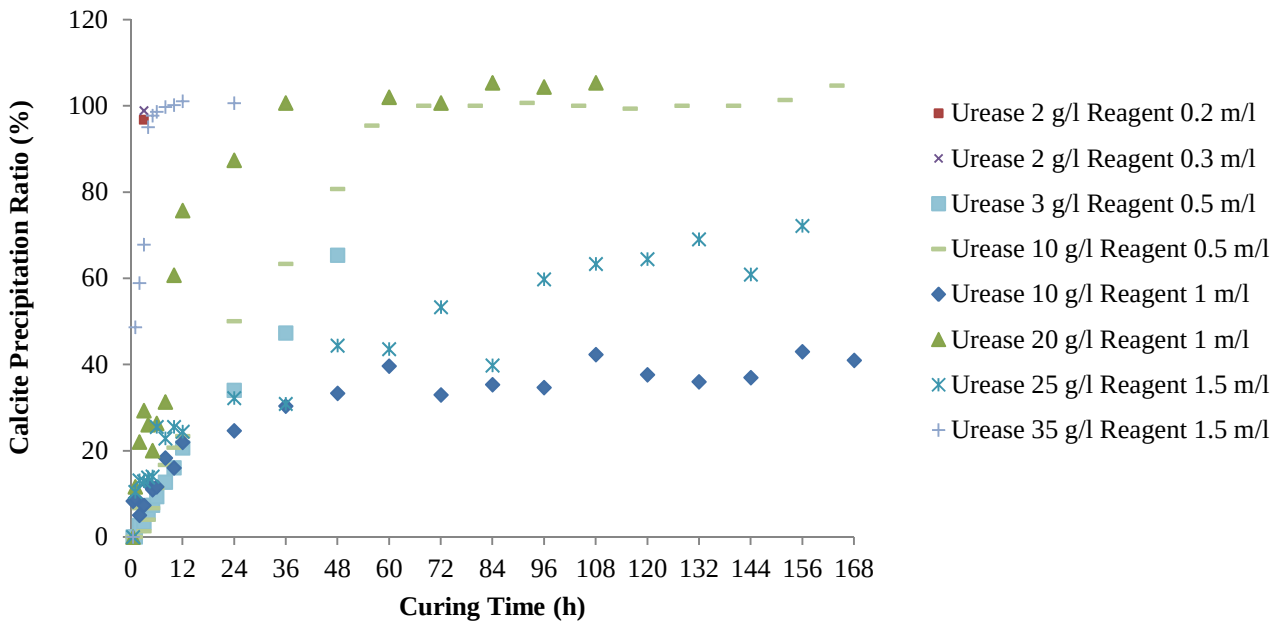


Figure 1 The Relationship Between Calcite Precipitation Ratio and Curing Time at Various Content of Urease Enzyme and Concentration of Reagent

Triaxial Test

The material used during the test performed in this experiment was Toyoura sand. It is a uniform fine sand composed mainly of quartz with the specific gravity (G_s) = 2.64, the minimum and the maximum void ratios e_{min} = O.609 and e_{max} = O.973 respectively.

Sample Preparation

All specimens were prepared using wet tamping method. Before tamping, predetermined quantity of dry Toyoura sand of around 60 gram for each layer was mixed in a transparent plastic bag with a certain amount of solution consisted of calcium chloride, urea, and urease (depended on saturation level needed). The quantities of material used are shown in Table 2. The mixed sand was then placed in a mould with internal dimension of 50 mm in diameter and 100 mm in high and tamped to the target relative density of 50% with a predetermined degree of saturation, either 40% or 70%. This procedure was repeated five time to build a 100 mm high sample in a short duration, typically 30 minutes, to minimize chemical reaction during tamping. The specimen was then subjected to a vertical pressure of 50 kPa for three days which duration is long enough to make the calcite precipitation ration of 100%.

The specimen was removed from the mold to a triaxial apparatus and was fully saturated with water. In this saturation process, more than two time the volume of water was flowed through the specimen to flush all the byproducts (ammonia) and possible chemical reagent remaining out from the specimen. Confirming that the Skempton's B-value higher than 0.95, the specimen was consolidated isotopically at an effective pressure of 50 kPa. The shear modulus at an axial strain level of 10^{-5} was measured and cyclic undrained shearing with a loading frequency of 0.1 Hz was performed.

Table 2 The Amount of Materials Used and the Expected Precipitation

Sr	Solution Concentration			Dry Toyoura Sand	Water Volume	Expected C_aCO_3 Prec.		Solution Weight (gr)		
	urea	$CaCl_2$	Urease			Weight	Percentage	Urea	$CaCl_2$	Urease
(%)	(m/l)	(m/l)	(gr/l)	(gr)	(ml)	(gr)	(%)			
40	0	0	0	300	36	0	0	0	0	0
	0.5	0.5	10	300	36	1.74	0.60	1.08	2.00	0.36
	0.25	0.25	2	300	36	0.87	0.30	0.54	1.00	0.07
70	0	0	0	300	63	0	0	0	0	0
	0.29	0.29	2	300	63	1.74	0.60	1.08	2.00	0.13

After the test, amount of calcite precipitated was measured. The specimen was oven dried and divided into three parts and weighed. Hydrochloric acid (0.1 mol/l) was used to rinse that sample. This acid leaching procedures was done three times for each sample to dissolve all the precipitated calcite followed by applying distilled water in every process

to clean both HCl and calcite that might be left in the sample. The disaggregated sand was dried again, and the loss in the dry weight was assumed to be the weight of the precipitated calcite.

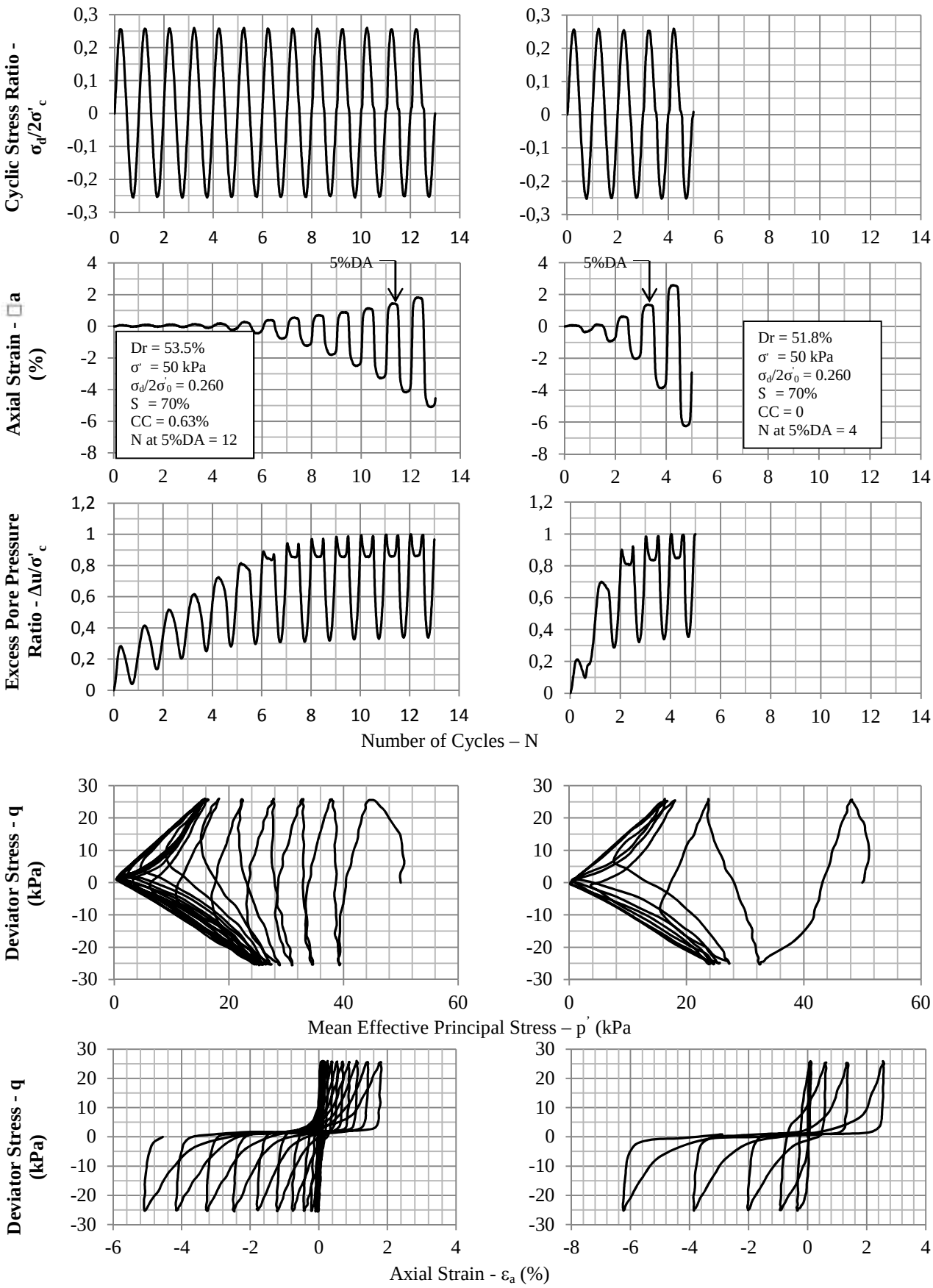
Test Conditions

The test program was designed for assessing liquefaction resistance of improved sand which is cured at different degrees of saturations (Sr) and calcite contents. The concentration of the regents was determined so that calcite precipitation ratio (CC) was either 0.3% or 0.6% at the degree of saturations as shown in Table 2. A total of 20 specimens were tested and the resultant ratio of calcite precipitation observed with the acid leaching are shown in Table 3.

It should be mentioned that the specimens were cured at different degree of saturations but fully saturated at the time of the cyclic shearing to avoid any possible effects of existence of air in the specimen. The comparisons of test results for specimens with the same CC but different Sr indicates effects of calcite precipitation characteristics, i.e. location of precipitation at sand grain surface.

Table 3 Testing Program and Testing Condition

Degree of Saturation Sr (%)	Reagent Concentration (m/l)	Calcite Content CC (%)	Relatif Density D_r (%)	CSR $\sigma_d/2\sigma'_c$	Number of Cycles (N)
40	0	0	48.1	0.260	4
			49.1	0.216	8
			52.8	0.174	16
			53.3	0.333	2
			53.1	0.151	23
	0.50	0.58	51.3	0.309	24
		0.58	52.1	0.363	11
		0.55	53.9	0.433	4
		0.61	52.9	0.211	41
		0.58	53.7	0.232	53
	0.25	0.30	52.7	0.262	4
		0.30	53.2	0.201	7
		0.20	52.0	0.146	44
70	0	0	51.8	0.260	4
			53.2	0.188	21
			54.1	0.140	26
	0.29	0.64	51.1	0.306	7
		0.63	53.5	0.260	12
		0.68	53.2	0.380	6
		0.67	53.6	0.174	145



(a) treated sand (b) untreated sand

Figure 2 Typical Records of Cyclic Triaxial Test on Treated and Untreated Sand

RESULTS OF TRIAXIAL TEST AND DISCUSSIONS

Figure 2 displays typical records of cyclic triaxial test on both improved sand with calcite content (CC) = 0.63% cured at Sr= 70% and unimproved sand. The number of cycles needed to attain a 5% double amplitude axial strain were 4 for unimproved sand and 12 for improved sand, indicating clearly the effect of calcite precipitation on the liquefaction resistance of sand. Figure 3 shows relationships between cyclic stress ratio (CSR) and the number of cycles for double amplitude axial strain DA=5%. For cases of unimproved samples (CC=0), almost the same CSR was observed irrespective of Sr at sample preparation. CSR for improved sand significantly increased with increasing CC. In order to see effects of Sr at sample preparation and curing, results of the tests on specimen with similar CC but prepared at different Sr are compared in Fig. 3(c). Although the specimens of Sr=40% had slightly lower CC than specimens of Sr=70%, the specimens at lower Sr exhibited apparently higher resistance to liquefaction. The liquefaction resistances of both the improved and unimproved specimens are shown in Fig. 4, together with that obtained by Hayashi et al. (2010) who conducted similar test on specimens with CC=1% and Sr=100% during the curing. It was found that improved liquefaction resistance of approximately 0.3 was obtained with much smaller CC by reducing Sr during the curing. This is probably due to the fact that precipitated calcite tends to concentrate to the contact points between sand grains as degree of saturation decreases. This hypothesis, however, have to be confirmed by observing grain surface. Detailed observation with SEM will be reported elsewhere in the near future.

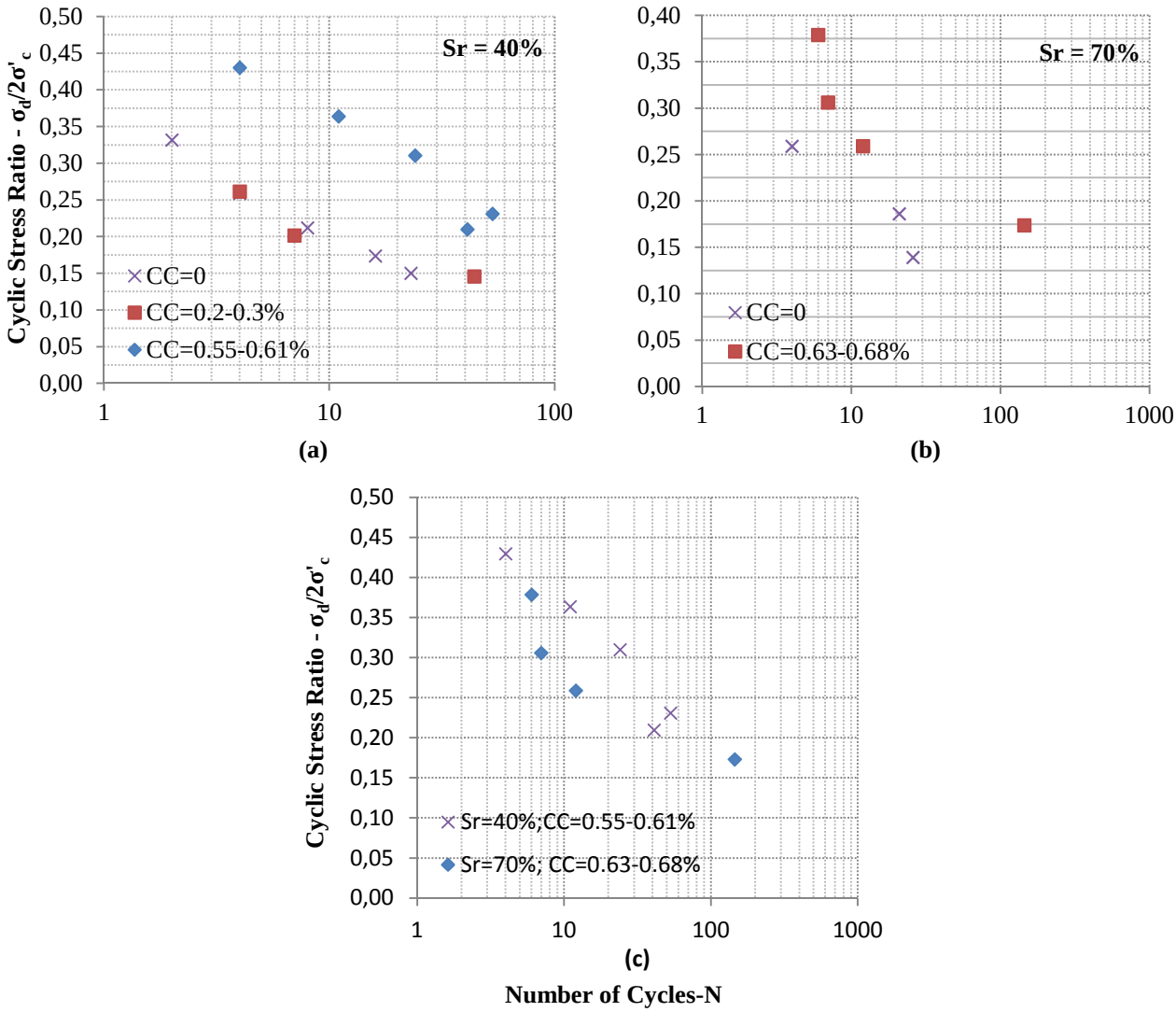


Figure 3 Liquefaction Strength Curve of Treated Sand

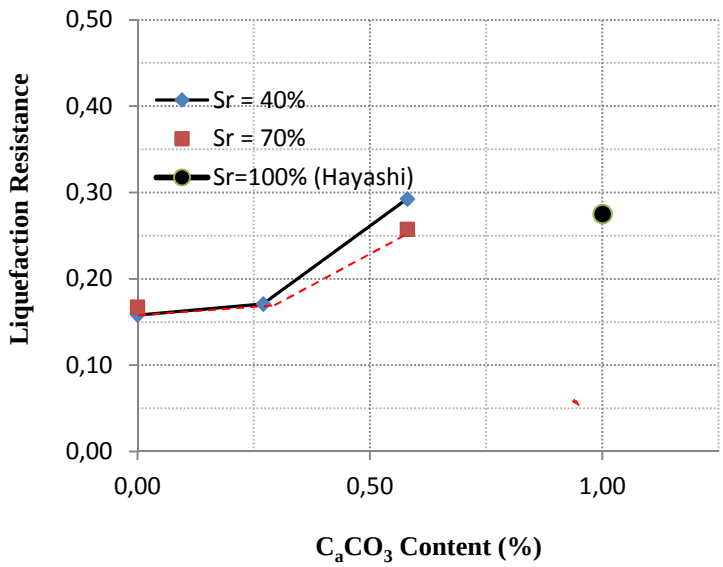


Figure 4 Alteration of Liquefaction Resistance due to the Change in Calcite Content

Shear modulus at small strain level

Shear modulus at small strain level was observed in prior to the liquefaction tests. Fig. 5 depicts shear modulus (G) for all the specimens tested in this study. Also shown in the figure is the relationship between G and shear strain of unimproved Toyoura sand at Dr = 70%. Shear modulus of sand is approximately constant in the strain level of 10^{-6} and 10^{-5}. Although the range of shear strain observed in this study was 10^{-5}, the shear modulus was significantly varied. Similar to the liquefaction resistances, the shear modulus increased with increasing CC and decreasing Sr.

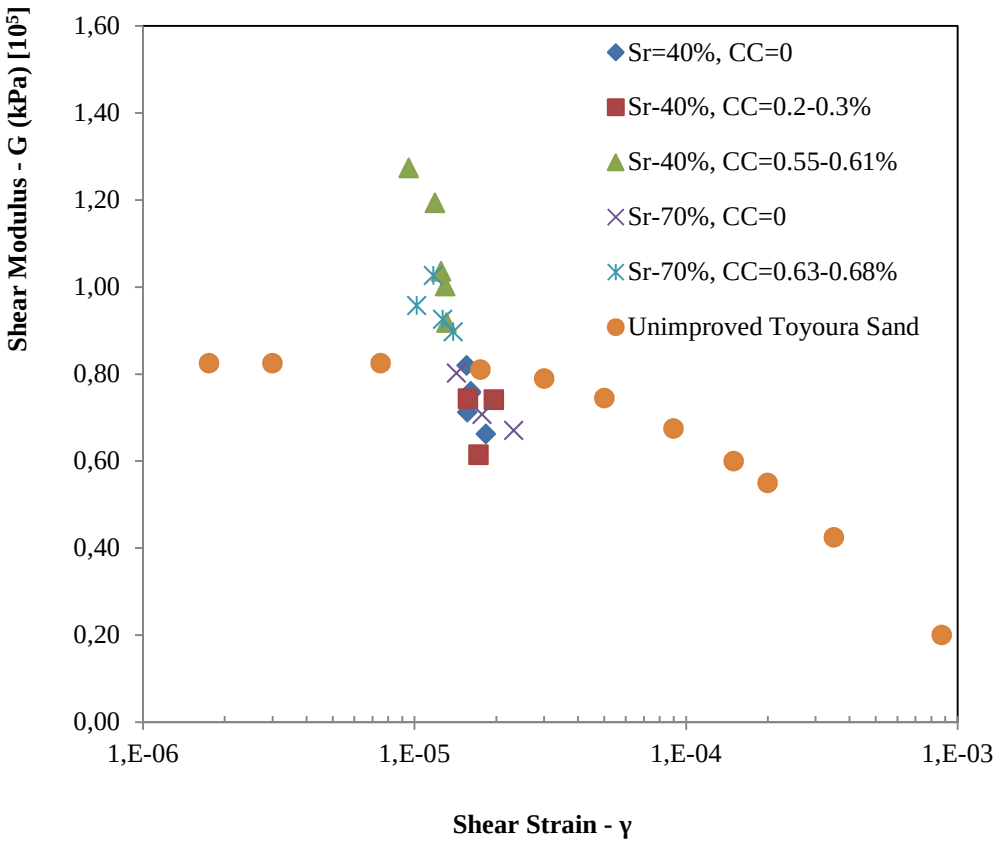


Figure 5 Shear Modulus Variations with Shear Strain

CONCLUSION

A series of undrained cyclic triaxial tests on sand improved with calcite precipitation has been conducted to study effects of calcite precipitation on dynamic properties of sand. Special attention was payed to the degree of saturation during curing the specimens. Two parameters selected as testing variables in this study are degree of saturation and calcite content. The main finding from the experiment are summarized as follows:

a. The existence of calcite content in sand sample enhance the liquefaction resistance as well as shear modulus at small strain level. This is occurred due to the restriction of buildup excess pore water pressure leading to liquefaction.
b. Cyclic stress ratio grows up due to the compound of calcite content and the decrease of saturation level in the sand sample. The bigger level of calcite and the lower the degree of saturation in the sand samples bring them stronger. This is probably due to the fact that precipitated calcite tends to concentrate to the contact points between sand grains as degree of saturation decreases. This hypothesis, however, have to be confirmed by observing grain surface. Detailed observation with SEM will be reported elsewhere in the near future.

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GIS BASED ANALYSIS FOR LANDSLIDE SUSCEPTIBILITY MAPPING USING LOGISTIC REGRESSION MODEL AT LOMPOBATTANG MOUNTAIN, INDONESIA

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ABSTRACT: Bawakaraeng and Lompobattang mountains are located at a southern part of South Sulawesi province and surrounded by the districts with high growth rates and had an important role in supporting that growth. In addition to providing a fertile land, this area is also threatened by disasters, particularly landslides. Landslide disasters occur almost every year especially during the rainy season and landslide induced flash floods or debris flow occurred in the upstream. Hence, the information of landslides susceptibility will greatly assist planners to optimize the future development planning. This study aims to predict landslide susceptibility using a statistical approach to find high accuracy. In this study 8 parameters were usedthe landslide conditioning factors namely, lithology, distance from the road, distance from the river, distance from the fault, land use, curvature, aspect and slope. The scales of data used were above 1: 50,000. The study area was chosen in a mountainous watershed where elevation is above 500 m and landslide occurred in high. All data were converted to raster forms with pixel size of 30 m. This research used ARCGIS 10.1 to process and analyzes spatial data. Besides,Microsoft Exeland SPSS were usedfor statistical data processing. This study divided landslide data as training and validation.The relative operating characteristic curve (ROC) and area under ROC curve (AUC) were usedto validate the performance of logistic regression in predicting future landslides.The best model was selected among twelve trials that were chosen from equal number of landslide and non-landslide pixels. The success rate, determined from the AUC of training data set, was found to be 0.866, which means that model has accuracy of 86.6 % accuracy to predict future landslides. The prediction rate, calculated from the AUC of the validation dataset was found0.855, which means a prediction accuracy of 85.5%. The close similarity of the success rate and prediction rate values showed how the logistic regression model is reliable in predicting future landslide with a good level of accuracy.

Keywords: *GIS, landslide susceptibility, logistic regression, ROC.*

INTRODUCTION

Background

As a developing country, Indonesia could not afford to recover from any natural disaster like landslide, flood, earthquake etc. The lack of spatial modelling in land use planning with lack of information about the disaster, puts Indonesia in a country with high vulnerable to hazards. Hence, an effort to minimize these problems is to incorporate all disaster related information in order to prepare susceptibility map. Nowadays, landslidesare serious problems particularly in hilly terrains and triggers debris flows/flash floods disaster occurs every year often during or after a heavy rainfall.

In general, GIS is focused on representation and visualization which is structured towards seeing the world as composed of geographic layers containing objects (pixels, points, lines, and polygons) along with associated attributes[Malczewski, 24].The concept of GIS is a system that can refer to a number of different technologies, processes, and methods. For that reason, GIS is widely to analyze many fields such as engineering and planning. In this decade, due to wider application of GIS concepts have been included on high school curriculum in Indonesia.In GIS technique, a model can be used to combine a set of input maps or factors using a function to produce an output map. The function can take many forms including linear regression, multiple regression, conditional analysis and discriminate analysis etc. [Fell, et al., 2008].

Many researchers were doing the studies about landslide susceptibility for good accuracy until recent days. Many methods were applied to include qualitative and quantitative modeling and analysis techniques of various types. Soetersand van Westen in (1996), classified the general techniques to analyze landslide zoning, which are heuristic, knowledge based, statistical and deterministic analyses.

The main objective of this recent study is to create a landslide susceptibility map for Lompobattang Mountain based on Geographic Information System (GIS) analysis. GIS and SPSS statistical software an equation was established using logistic regression model.Besides that, we used the ROC to examine for the best fit of the model. The landslide

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inventory data was divided intotraining and validation. Therefore,around 80% of landslide data was used as a data training to get the best model and the rest was used as validation for verificationto test the accuracy data of the model.

Study Area

Lompobattang mountain and Bawakareaeng mountain are located in South Sulawesi and surrounded by seven regions called Kabupaten, and its give a lot of beneficial for them. The mountains provide area for cultivation and farming, and some areas particularly at upstream part arecovered by forests. LompoBattang Mountain located at $119^{o}50$ – $120^{o}04$ E and $5^{o}12'$ -$5^{o}28'$ S and with altitudeis around 2876 meters above sea level with total area coverage of 351.742 km^2 (Fig.1). There are 86 small town/villages in this area and the hydrologic system have six watersheds; Jeneberang, Lantebong, Kelara,Apparang, Bijawang and Tangka.

MATERIALS AND METHOD

Materials

This study applies of logistic regression model to create a landslide susceptibility map at Lompobattangmountainin Indonesia. In susceptibility mapping, we have to assume that future landslide occurs in the same condition that caused the past landslide. Landslide data was used as a dependent variable, and eight landslide causal factors including slope, curvature, aspect, distance from fault, distance from road, distance from river, lithology and then land use were selected as independent variables for landslide susceptibility mapping.

Landslide inventory are essentially [Fell, et al., 2008], and which are important factor to predict future landslide by using probabilistic method. Landslide inventorydata were collectedfrom 2004 until 2014 google earth image interpretations of LompobattangMountain with highestaltitude range between 2833 m and 700 m above sea level. From this a total of 158 landslide were identified, which covered an area of 3.44 km^2. Most of the landslides are shallow landslides with minimum and maximum landslide area708 $m^2$and 512,765 m^2 (0.51 km^2) respectively. By using google earth historical imagery tools, we have limited the study area to an altitude 500m as no landslides data were found below this point. Before applying the landslide data in GIS analysis, we have to digitize the time series data from google earth image interpretation, step carps of noticeable landscape modification exist. Then these files are saved as GIS compatible (kml) format and the data is again subsequently changed into shapefile and then raster format.

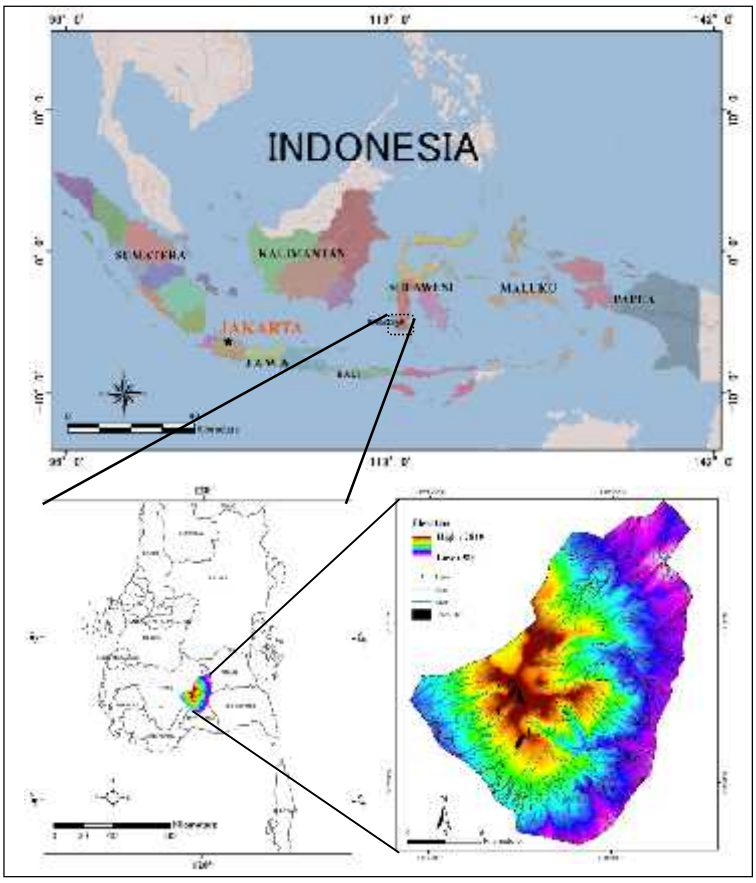

Fig. 1.Location map of study area showing elevation and landslide inventory at Lompobattang Mountain

Landslide Causal Factors

Arc GIS 10.0 was used for preparing a spatial database, particularly using the spatial analyst and 3-D analyst tools. All parameters and landslide distribution map were digitized and then processed by converting all the raster and vector maps into raster format with 30 m x 30 m pixel size. The totalnumber of cells are 390,837 pixelsand thenumber oflandslide pixels are 3827. Microsoft Excel was used to compute and manage tabular data and then analyze Frequency Ratio (Fr) values. The maps of landslide causal factors were shown in figure 2.

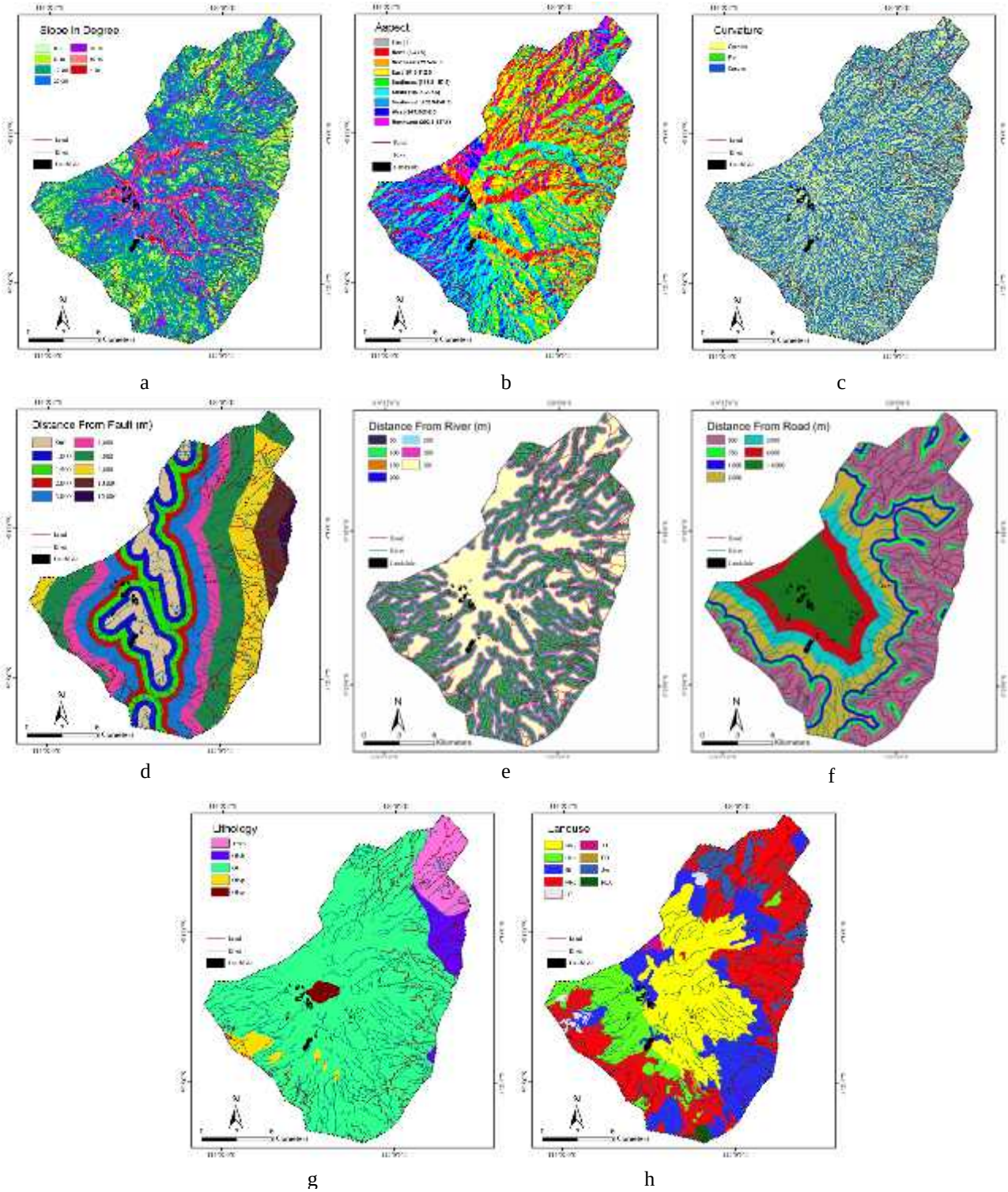

Fig. 2.Landslide causal factors (a) slope, (b) aspect, (c) curvature, (d) fault,(e) river, (f) road, (g) lithology, and (h) landuse

The geological map was used as independent variables. The geology of the area was digitized from the Geology of Puslitbang which is produced by the government board at a scale of 1:250.000. This map includes the current study area. The geology includes lithology, rock type and structure (fault or lineament).Lithology is a part of basic data or parameters for landslide map analysis. In fact, [Ermini, et al., 2005] said that lithology is a classic variable thatcontrol landslide hazard. It is related to the material strength, because they have varied composition and structure for different type of rocks [Kanungo, et al., 2006], and the resistance to driving forces depending on the power rock strength, in which the strongest rocks would be more resistance.Lineaments are the structural features, which describe the zone/plane of weakness, fractures, and faults along which landslide susceptibility is higher. It has generally been observed that the probability of landslide occurrence increases at sites close to lineaments, which not only affect the surface material structures but also make contribution to terrain permeability causing slope instability [Kanungo, et al., 2006]. For this purpose, distance from fault was used to analyze the relationship between landslide occurrences. The proximity distance from fault was identifiedby buffering from lineament or fault map.

Topography is one of classic factor to create landslide susceptibility map. The Topographic data used in the analysis include slope, aspect and curvature. These data were derived from ASTER DEM with a spatial resolution of 30m. ASTER DEM was extracted to be elevation thematic map in contour layers. Then, Triangular irregular networks (TIN) were produced in ArcGIS and convert to DEM raster with pixel size 30 m. Usingarctoolbox raster surface in ArcGIS, the slope angle, slope aspect and curvature were calculated from a DEM raster.Slope is the most substantial cause of land sliding. On a slope of uniform isotropic material, increased slope correlates with increased likelihood of failure. In this study, we used seven slope categories, 0-5°, 5-10°, 10-20°, 20-30°, 40-50°, and above 50°, which they were considered and represented in the form of slope thematic data layer. In many research for landslide susceptibility, slope angle is important factors in inducing slope instability. Aspect is defined as the direction of maximum slope of the terrain surface and has an indirect influence on slope instability [Kanungo, et al., 2006].The aspect map also plays a significant role in slope stability assessment [Chauhan, et al., 2010]. In this study, aspect is divided into nine classes for the study area, namely, flat, N, NE, E, SE, S, SW, W, and NW. To describe the variances among classes, aspect maps displayed the distribution of each direction in the topography by using different colors to each cell of the study area[Quan & Lee, 2012].Profile curvature was reclassified into three classes for landslide causal factor namely concave, flat and convex. The curvature values represent the morphology of the topography. In case of profile curvature, generally related to the puddle condition after heavy rainfall. Profile curvature slope contains more water and retains water from heavy rainfall for a longer period[Lee & Thalib, 2005].

Beside slope angle and lithology, land use (cover) is also a key factor responsible for landslide occurrences. The incidence of landslide is inversely related to the vegetation density.The landuse map was derived from Landsat 7 with 30 m x 30 m, and its wasestablished by BP DAS Jeneberang in 2013, a board for watershed issued atMinistry of Forestry in Indonesia. The landuse map is classified into several class, but especiallyfor this study area, classes such as forest (including primary and secondary), bushes, crop land (agriculture), andgrass land were identified.Drainage lines and landslide occurrence in hilly area have strong association between them due to erosional activity[Chauhan, et al., 2010] and [Kanungo, et al., 2006].The distance from river was identified by buffering and analyses of river lines that were derived from topographic map with scale 1:50.000 called Peta RupaBumi Indonesia (RBI) established by government. The class start from 0-50 m and ends withaof> 300 m. Similarly in cases of distance from river and distance from road were derived from topographic map with differentclass intervals.

Method

Several approaches to statistical modelling for the prediction of landslide susceptibility mapscan be found in different literatures. In this study, the landslide susceptibility index was created by logistic regression model. Generally in logistic regression, the spatial prediction is modeled by a dependent variable and independent variables [Shirzadi, et al., 2012] and it is useful whenthe dependent variable is binary or dichotomous[Bhandary, et al., 2013]and [Gorsevski, et al., 2006].Simple introduction of logistic regression was explainedby Chau & Chan (2005)in which logistic regression is defined as the probability of landslide occurreddivided by the probability of no landslideoccurred.The probabilities fall in the range of values from 0 to 1. Therefore, logistic regression is based on logistic function expressed as follows;

$$P = 1/(1 + e^{-z}) \quad (1)$$

Where P: is the probability of landslide occurrence that estimated values varies from 0 to 1. Variable Z is landslide causal factors and assumed as a linear combination of the causal factors $X_i(i=1,2,\ldots n)$ as

$$Z = B_0 + B_1X_1 + B_2X_2 + \cdots + B_nX_n \quad (2)$$

Where Bi are the coefficient of landslide causal factors.

Landslide causal factors as independent variables data for model contain nominal variables such as landuse type and geological type, meanwhile the use of logistic regression requiresa numeric scale. Therefore, independent variables

could be converted from nominal to numeric by using each class values from frequency ratio method. Generally, in the use of the frequency ratio, the relationship between areas where the landslide has occurred and landslide causal factors can be distinguished from the level of correlations between past landslide and landslide causal factors[Lee & Thalib, 2005]. As the result, frequency ratio values within each class described the level of distinction between them in quantitatively as shown on table 1.,frequency ratio for each class used as input for the logistic regression model and allows expressing the independent variables by the same scales [Ramani, et al., 2011].Factor classes with a frequency ratio value of > 1 will have probability of causing a landslide occurrence[Lee & Thalib, 2005]. The frequency ratio method is based on;

$$Fr = \frac{P \quad (i,j)/\sum P}{P \quad (i,j)/\sum P} \tag{3}$$

(Where, $PixelL(i,j)$ number of pixel with landslide within class i of j parameter, $Pixel(i,j)$ Number of pixel in class i of j parameter, $\sum PixL$ total pixel of j parameter, and $\sum Pix$ total pixel of the area)

Table 1.Frequency ratio value for each landslide causal factors as input logistic regression model

Factor	Number of Landslide	% Landslide (x)	Number of pixelinclass	% class (y)	Fr (x/y)
Topography					
SlopeClassinDegree					
0-5	64	2.053	27,459	7.026	0.29
5-10	136	4.363	67,750	17.335	0.25
10-20	472	15.143	137,617	35.211	0.43
20-30	609	19.538	86,336	22.090	0.88
30-40	840	26.949	45,954	11.758	2.29
40-50	734	23.548	19,573	5.008	4.70
> 50	262	8.406	6,141	1.571	5.35
CurvatureClass					
Concave	1,616	51.84	192,998	49.38	1.0499
Flat	15	0.48	5,424	1.39	0.3468
Convex	1,486	47.67	192,408	49.23	0.9684
AspectClass					
Flat (-1)	5	0.16	1,933	0.49	0.324
N (0-22.5)	159	5.10	22,786	5.83	0.875
NE(22.5-67.5)	353	11.32	53,942	13.80	0.821
E(67.5-112.5)	207	6.64	57,312	14.66	0.453
SE (112.5-157.5)	242	7.76	65,237	16.69	0.465
S (157.5-202.5)	540	17.32	56,441	14.44	1.200
SW (2025-247.5)	834	26.76	46,535	11.91	2.247
W (247.5-292.5)	372	11.93	30,493	7.80	1.530
NW (292.5-337.5)	231	7.41	35,952	9.20	0.806
N (337.5-360)	174	5.58	20,199	5.17	1.080
Geology					
LithologyClass					
Tmcv	5	0.16	28,534	7.30	0.02
Qlvb	13	0.42	17,403	4.45	0.09
Qlv	3,080	98.81	33,2091	84.97	1.16
Qlvp	-	-	8,643	2.21	-
Qlvc	19	0.61	4,159	1.06	0.57
DistanceFromfault (m)					
0-500	886	28.42	34,527	8.834	3.22

500-1000	952	30.54	35,187	9.003	3.39

Table 1. Frequency ratio value for each landslide causal factors as input logistic regression model *(continued)*

Factor	Number of Landslide	% Landslide (x)	Number of pixelinclass	% class (y)	Fr (x/y)
1000-1500	296	9.50	35,483	9.079	1.05
1500-2000	177	5.68	27,949	7.151	0.79
200-3000	411	13.19	50,855	13.012	1.01
3000-4000	263	8.44	47,642	12.190	0.69
4000-6000	63	2.02	81,203	20.777	0.10
6000-8000	69	2.21	53,027	13.568	0.16
8000-10000	-	-	21,147	5.411	-
10000-12000	-	-	3,810	0.975	-
Proximity					
Distancefrom Road (m)					
0-500	114	3.657	155,239	39.720	0.092
500-750	84	2.695	32,697	8.366	0.322
750-1000	70	2.246	24,331	6.225	0.361
1000-2000	463	14.854	63,341	16.207	0.917
2000-3000	488	15.656	36,745	9.402	1.665
3000-4000	331	10.619	31,195	7.982	1.330
> 4000	1,567	50.273	47,282	12.098	4.156
Distancefrom River (m)					
0-50	213	6.833	58,994	15.095	0.45
50-100	220	7.058	56,989	14.582	0.48
100-150	227	7.283	51,007	13.051	0.56
150-200	285	9.143	44,044	11.269	0.81
200-250	293	9.400	37,016	9.471	0.99
250-300	291	9.336	30,154	7.715	1.21
> 300	1,588	50.946	112,626	28.817	1.77
Landuse					
LanduseClass					
PrimaryDryForest	994	31.89	99,453	25.45	1.25
SecondaryDryForest	757	24.29	46,591	11.92	2.04
Bushes	977	31.34	83,215	21.29	1.47
MixDryland Agriculture	382	12.26	133,251	34.09	0.36
ForestPlant	-	-	3,962	1.01	-
OpenLand	-	-	1,601	0.41	-
GrassLand	7	0.22	1,112	0.28	0.79
PaddyField	-	-	19,533	5.00	-
Dryland Agriculture	-	-	2,112	0.54	-

Receiver operating characteristic (ROC) curves are used to plot predicted probability to comprehend issues of accuracy, criterion selection, and interpretation. For validation of results, the area under the ROC curve (AUC) was used as a measure of overall fit and comparison of modeled prediction.The success rate, determined from the AUC of training data set, and the prediction rate, calculated from the AUC of the validation dataset.

Validation

In prediction modelling, the most important and the absolutely essential component is to carry out a validation of the prediction results[Chung & Fabbri, 2003]. To do that, in this study, we divide landslide inventories into two parts i.e one for training and the other for validation. As mention in the background above, this study used 80% of landslide inventories for generate the model and the rest 20% landslide inventories for validation. To illustrate the procedure, we have chosen a small of landslide prone area.The size, area, depth of landslide and its distribution significantly varies from place to place.

RESULTS AND DISCUSSION

Table 1.showedcorrelation between landslide occurrence and each class of presence and absence landslides inventories between each class of landslide causal factors. In the case of the relationship between landslide occurrence and slope below 30° , the ratio was < 1, which indicates a very low probability of landslide occurrence. For slope above 30° , the ratio was > 1, which indicates a high probability of landslide occurrence. For curvature class, the values represent the morphology of topography. A convex indicated a positive value, a negative curvature indicates concave, and zero values indicate that of flat surface. Comparing frequency ratio values of both concave and convex, it indicates at the probability of landslide occurrence almost similar,with have a slight higher probability of landslide occurrence in the case of concave curvature. This might be due to accumulation of waterin these classes. But in the case of flat surfaces, the probability of landslide occurrence is very low. In the case of aspect class,the south, southwest and west facing slopes, frequency ratio value of >1, which indicates a high probability of landslide occurrence.

In the case of lithology classes, only Qlv has ratio of > 1, among five lithology classes, which indicates high probability of landslide occurrence.*Quarter lompobattang volcanic* (Qlv) is one of volcanic and sediment formation in South Sulawesi. In case of distances from fault, river and road, distance/proximity to such features is used to understand the level of influence on landslide occurrence. Distance from fault below 1000 m has ratio > 1, and it indicates has high probability of landslide occurrence.This shows thatas distance fromthe fault decrease, the probability of landslide occurrence increases.In case of distance from road, the frequency ratio value is higher at a distance class of > 3000 m. Similarly, for the distance from river above 300 m, the ratio value is >1. In case of distance from rivers and distance from roads, the landslide densities are higher for distance classes far away from these features showing the limited influence of these factors in causing landslides. Forests and bushes in landuse classes have a frequency ratio value of > 1, but in the case of agriculture and grass land the ratio < 1.

Table 2.Logistic regression coefficient of landslide causal factors from twelve samples with equal and unequal proportion of landslide and non-landslide points

No	Non Landslide Pixels	Variables in the Equation										
		As	Crv	Fl	Litho	Luse	Rv	Rd	Sl	Cons	Success rate	Predictive rate
1	Equal 1	0.485	1.200	0.544	1.528	0.051	0.2	0.311	0.428	-5.867	0.867	0.850
2	Equal 2	0.560	1.200	0.506	1.778	0.214	0.212	0.265	0.414	-6.300	0.864	0.851
3	**Equal 3**	**0.554**	**0.871**	**0.501**	**2.021**	**0.100**	**0.198**	**0.259**	**0.474**	**-6.152**	**0.866**	**0.855**
4	Equal 4	0.605	1.231	0.484	1.814	0.130	0.236	0.288	0.382	-6.312	0.861	0.849
5	Equal 5	0.572	0.798	0.492	1.692	0.110	0.189	0.293	0.451	-5.737	0.866	0.853
6	Equal 6	0.578	1.303	0.508	1.965	0.139	0.181	0.289	0.385	-6.506	0.864	0.848
7	Equal 7	0.573	1.008	0.518	1.922	0.111	0.174	0.305	0.447	-6.243	0.869	0.851
8	Equal 8	0.482	1.112	0.517	1.709	0.058	0.212	0.302	0.439	-5.973	0.865	0.852
9	Equal 9	0.547	1.091	0.52	1.666	0.046	0.151	0.300	0.417	-5.875	0.862	0.850
10	Equal 10	0.618	1.033	0.495	1.748	0.132	0.150	0.296	0.380	-6.012	0.858	0.848
11	50 %Non landslide pixels	0.761	0.558	0.441	2.147	0.443	0.481	0.245	0.281	-10.664	0.859	0.839
12	All Non landslide pixels	0.763	0.562	0.438	2.143	0.468	0.485	0.245	0.279	-11.389	0.858	0.839

As is aspect, Crv is curvature, Fl is fault, Litho is lithology, Luse is landuse, Rv is river, Rd is road and Sl is slope, Cons is constant

A complete set for logistic regression analysis must be containing a set of independent variables (landslide causal factors) and dependent variable (landslide inventories) which is dichotomous. Generally, the way to fixing the sample size to create an equation in logistic regression analysis [Ramani, et al., 2011]; (a) use all pixels landslide causal factors in study area (b) to reduce bias in the sampling processwe use equal number of dependent and independent variables. In

this study, thelogistic regression model is developed usingequalproportion of landslide and non-landslide pixels.The tests were carried out twelve timesto get the best equation, where ten trials were carried using equal proportions, the 11th trial from the combination of 50% of non-landslide and all landslide, where the twelve trial, was done from the combination of all landslide and non-landslide data.As shown in table 2., the best equation based on a high success rate and predicted rate, which comes from the third trial where equal proportion of landslides and non-landslide combined.. We assumed that closeness differences rate between of the success rate and prediction rate values showed how the logistic regression model has the capability to create future landslide prediction..

The constant andcoefficient of independent variableswere provided by logistic regression analysis; they were carried out using statistical software known as SPSS. The number of equal proportion of non-landslide pixels was determined by SPSS, whichare selected randomly, hence, we proposed to carry out ten times to get best result. The table showed that, among ten times experiments for half portions of equal non-landslide pixels, the success rate ranges from 0.858 to 0.869 and predicted rate ranges between 0.848 to 0.855. Then for comparisonpurpose, wefound an equation by using a half portions non landslide pixels and all non-landslide pixels that are combined with all landslide pixels.. The third rows contains successrates 0.866 and predicted rate 0.855, and used an equation to produce landslide susceptibility index shown in Fig. 3with values index ranges from 0.00458 to 0.98455. The logistic regression equations for variable Z eq. (2) for this study were obtained:

$$Z = 2.021\ (lithology) + 0.871\ (curvature) + 0.554\ (Aspect) + 0.501\ (fault) + 0.474\ (slope) + 0.259(road) + 0.198\ (river) + 0.1\ (landuse) - 0.6152$$

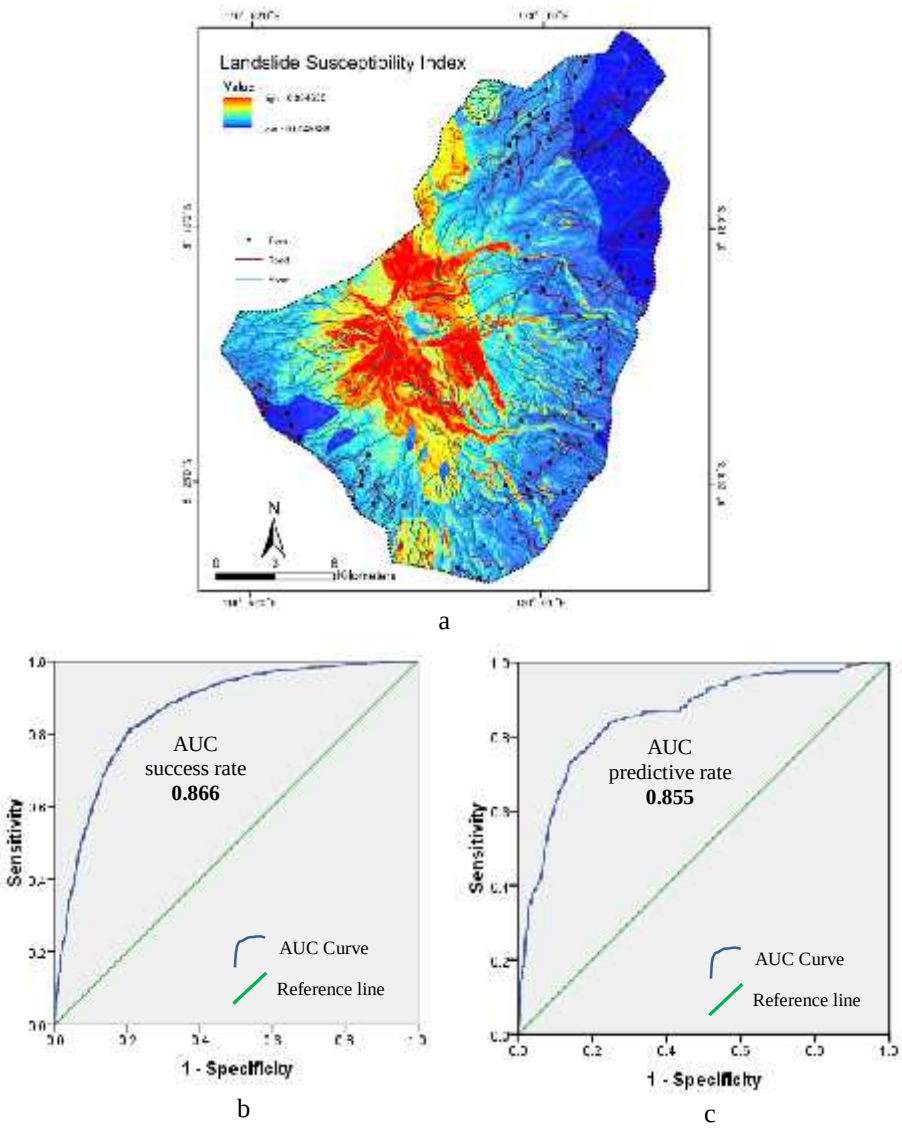

Fig. 3(a) Landslide susceptibility index map, (b) Area under the curve AUC value of the ROC curve for training data (c) Area under the curve AUC value of the ROC curve for validation data

In figure 4, the landslide susceptibility map were classified using natural breaks and grouped into five classes like very low, low, moderate, high, and very high.For verification, a susceptibility map was verified by overlaying with landslide validation data. The percentages of landslides which fall in high to very high, moderate, and very low to low comprises of 83.94%, 4.08 %, and 11.98% respectively.

Finally, landslide susceptibility map in Fig. 4 was produced from the classification of landslide susceptibility index map, and it's classified into five classes by using natural breaks classification method or Jenks optimization method. This classification method has been used widely especially for planners and itsdesigned to determine the best arrangement of values into different classes. This is because this method maximizes the variance between classes and reduces the variance within classes. The five classes include very low, low, moderate, high and very high describing the level of landslide susceptibility (proneness) in this study area.

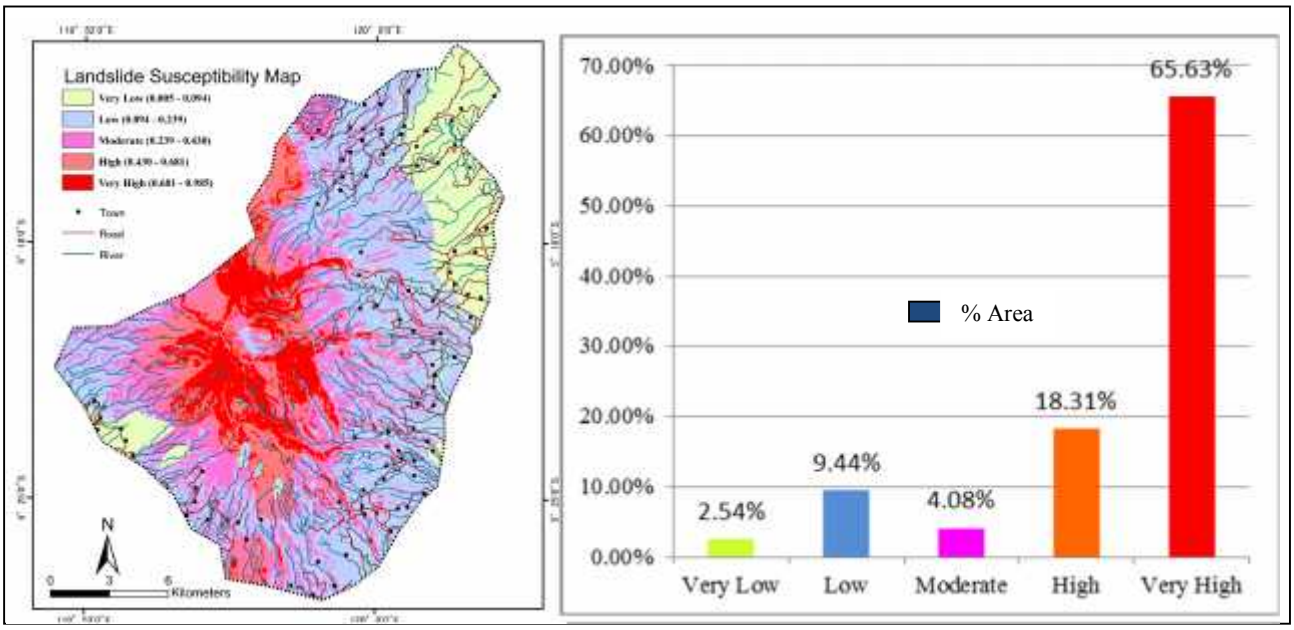

Fig 4.Landslide susceptibility map andpercentage of landslides validation data in each class

CONCLUSION

The landslide susceptibility map was produce using the statistical approach, known as logistic regression in which GIS and image interpretation at google earth for landslide inventories. The landslide susceptibility map thus produced is an effective planning tool for land managers and town planners [Ramani, et al., 2011]. Susceptibility mapping is an essential tool to delineate areas prone to landslide, and it has become an important information for decision makers and government. Validation is emphasized in this study in order to reduce any uncertainty that may appear during prediction and to increase the accuracy of the model. For this purpose, success rate and predict rate had become an important part to get the best prediction for future landslide. Besides validation, in this research, we describe the level of accuracy of prospective landslide prediction by overlying landslide susceptibility map with landslide inventories that were used for validation. The results showed that about 84% of the landslide fall in high to very high susceptibility classes, while 16% landslides fall in the very low to moderate classes as shown inFigure 4. Moreover, using statistical approach like frequency ratio method and logistic regression model, the influence of landslide causal factors on the landslide susceptibility can be known qualitatively. From the logistic regression equation, it can be concluded that the landslide causal factors that have a significant influence in causing landslides were found to be lithology, curvature, aspect, distance from fault and slope. The value ratio in frequency ratio showed the strong relationship within class of causal factors and coefficients on variable (Z) of causal factors in the logistic model showed the strong correlation between all causal factors to landslide occurrence. Nevertheless, this study also showed that predicting a future landslide by using logistic regression could be the best choice, although the result will be more accurate on a larger scale particularly at topographic map and geological map.

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THE EFFECT OF CURING PERIOD TO THE PHYSICAL AND ENGINEERING PROPERTIES OF FIBROUS PEAT SOIL STABILIZED WITH LIME CaCO3+POZOLON

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ABSTRACT: Stabilization method for Peat soil is continuously developed because it is cheaper and more environmentally friendly than other methods. Several types of admixtures have been utilized to improve the peat soil behavior; the effect of curing period to the stabilized peat behavior, however, was studied only for short period of time that was 30 days. Due to that reason, this study was conducted to determine the effect of curing period to the stabilized peat soil behavior. The admixture chosen in this laboratory-scale research was mixed of lime (C_aCO_3) and pozzolan with percentage of 10% of the initial peat unit weight. The pozzolan material selected was fly ash (FA) and rice husk ash (RHA). The effect of curing period to the physical and engineering properties of the stabilized peat soil was observed at curing periods of 10, 20, 30, 40 and 80 days. The results show that the physical and engineering properties of the stabilized peat soil increase rapidly until the curing period reaches 30 days; afterward, they tend to be constant and finally they slightly decrease. Formation of CaSiO3 gel takes place during water in the macro-pores available; afterwards, the gel and the unwrapped (by the gel) fiber getting dry and decomposition process take place. This fiber decomposition affect decrement of the physical behavior and shear strength of the stabilized peat; it also causes the increment of its compressibility. In addition, the laboratory test results show that peat stabilized with admixture-1 [70% Fly Ash (FA) + 30% Lime CaCO3] gives better improvement of peat behavior than it stabilized with admixture-2 [70% Rice Hush Ash (RHA) + 30% Lime CaCO3].

Keywords: *Curing period, fibrous peat, fly ash, lime CaCO3, rice hush ash (RHA), stabilization*

INTRODUCTION

In order to improve peat soil behavior, stabilization method is still chosen as the best method; it is due to its low cost and environmentally save (Jelisic & Lappanen, 2001; Mochtar, N.E, et.all, 2014; Yulianto, F.E.,et.all, 2014). In 2010, Harwadi, F & Mochtar, N.E also Yulianto, F.E & Mochtar, N.E, had been successfully using pozzolan (fly ash or rice hush ash) and lime CaCO3 (as a side product of PT. Petrokimia) as stabilizer material for peat soil. The research result showed that when the curing period reached 30 days, additional 10% of admixture (70% pozzolan + 30% lime CaCO3) was able to increase about 25% of its bearing capacity and to reduce its compression until 28% of its initial.

This research, however, did not show the effect of degree of decomposition to peat soil behavior. According to Huttunen and Kujala (1996), peat stabilized with any types of additives material would cause water content reduction and peat fiber decomposition. This result had been approved by Wardani, M.K and Mochtar, N.E (2012) also Yulianto, F.E., et.al (2014). In order to study the effect of water content reduction and fiber decomposition to the physical and engineering behavior of the stabilized peat, laboratory experiment was carried out by stabilizing fibrous peat with 10% admixture and cured in 80 days. The result reported in this paper will include the effect of curing period (age of stabilized peat) to the physical and engineering properties of the stabilized peat, and the effect of fiber decomposition to their properties.

PEAT SOIL STUDIED

Peat sample studied was taken from Barengbengkel, Palangkaraya, Central Kalimantan, Indonesia. Testing in the laboratory was based on the Peat Testing Manual (1992). The results are presented in Table 1. According to the Standard Classification of Peat Samples by laboratory Testing ASTM D4427-84 Reapproved 1985, peat soil studied can be classified as “peat soil (hemic) with low ash content, high acidity, and H4 degree of decomposition (Von Post)”.

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Table 1. Peat Soil Parameters

No	Peat Soil Parameters	Units	Peat Studied	Other Researchers
1	Specific Gravity (Gs)	-	1.49	1.4 - 1.7
2	Water Content (w_c)	%	630	450 - 1500
3	Unit Weight (γ_t)	gr/cm^3	0.98	0.9 - 1.25
4	Void Ratio (e)	-	10.4	6.89 - 11.09
5	Acidity (pH)	-	3.1	3-7
6	Organic Content (O_c)	%	98	62.5 - 98
7	Ash Content (A_c)	%	2.0	2 - 37.5
8	Fiber Content (Fc)	%	52.1	39.5 - 61.3
9	Internal Friction Angle (φ)	^{0}	20	30° - 50°
10	Vane shear	kPa	8.7	5 – 10

The micro structure of peat soil studied is shown in Figure 1(a) and 1(b); these pictures were from Scanning Electron Microscope / SEM. It shows that fibrous peat studied has macro-pores that are located among peat fibers and micro-pores that are located inside of the peat fibers. Due to these two types of pores, fibrous peat has unique compression behavior compared to clay compression as shown in Figure 1(c). It has 4 (four) components of compression, those are: immediate compression (s_i), primary compression (s_p), secondary compression (s_s), tertiary compression (s_t) (Yulianto, F.E and Mochtar, N.E (2010).

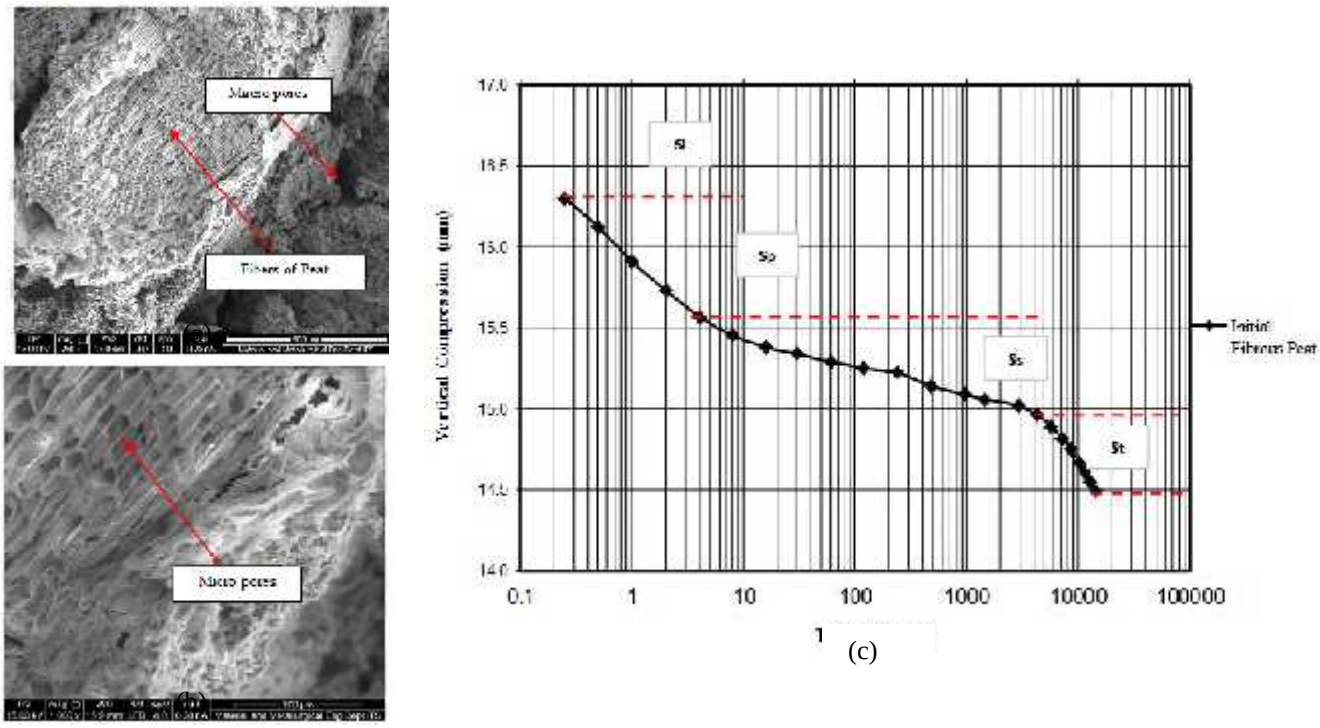

Figure 1. (a) Peat fibres and macro-pores, (b) Micro-pores inside of peat fibres
(c) Consolidation curve with one step loading ($\Delta\sigma$ = 50 kPa)
(Yulianto, F.E and Mochtar, N.E, 2015).

PREPARATION OF ADMIXTURE MATERIAL AND STABILIZED FIBROUS PEAT

There were 2 (two) types of admixture materials used in this study (adopted from study performed by Harwadi, F and Mochtar, N.E also Yulianto, F.E & Mochtar, N.E, 2010), those were:

1. Admixture-1 made of 70% Fly Ash (FA) + 30% Lime CaCO3
2. Admixture-2 made of 70% Rice Hush Ash (RHA) + 30% Lime CaCO3

The fibrous peat was stabilized with 10% admixture and placed in an impervious box (25x25x25 cm3); box-1 was for peat stabilized with admixture-1 and box-2 for peat stabilized with admixture-2. Each stabilized peat then preserved

(cured) for different periods 10, 20, 30, 40 and 80 days; it means that there were 10 boxes (5 boxes for each admixture) prepared in this study. At the end of each curing time the soil parameters, physical and engineering properties, of the stabilized peat were determined.

THE EFFECT OF CURING PERIOD TO THE PROPERTIES OF STABILIZED FIBROUS PEAT

As mentioned above that at the end of curing period, each sample was tested in order to determine its soil parameters. Physical properties of the stabilized soil determined were water content (w_c), unit weight (γ_t), and specific gravity (G_s). The engineering properties analyzed were shear strength and compression behavior.

Water content of stabilized fibrous peat at different age is shown in Figure 2. Both samples (peat stabilized with admixture-1 and admixture-2) show similar behavior; water content of stabilized peat drop significantly when the stabilization age reach 10 days. Afterwards, the water content decrement is not significant until curing period reach 80 days. It means that process to produce CaSiO3 gels take place very fast until the stabilization age reaches 10 days; it is slowing down until its age reaches 30 days and then no more water content reduction. It explains that after stabilization age reach 40 days, no more water available in the macro-pores; the only water content left is only in the micro-pores.

As shown in Figure 3, the unit weight (γ_t) of the peat stabilized with admixture-1 increases with the increase of the curing period until it reaches 30 days. The unit weight (γ_t) of peat stabilized with admixture-2, however, increases when the curing period reaches 10 days and it is constant until its age reaches 40 days. After 40 days, the unit weight of peat stabilized with both admixtures decreases and never increase again; it shows that the unit weight decrement is caused by peat decomposition because no more water available in the macro-pores. Significant increment of unit weight of peat stabilized with admixture-1 at 30 days curing period is due to fly ash has smaller granular size and has better capability in absorbing water so that CaSiO3 gel can be developed more properly by admixture-1 than by admixture-2.

The use of admixture for stabilizing peat means introducing mineral into peat soil; it causes the increment of specific gravity of stabilized peat as shown in Figure 4. It is in agreement with study performed by MacFarlane (1969) in which the specific gravity (Gs) of peat ranges from 1.4 to 1.7 and it will be equal or higher than 2.0 when mineral is introduced into peat. At 10 days of curing period, the increment of Gs value of peat stabilized with admixture-1 is

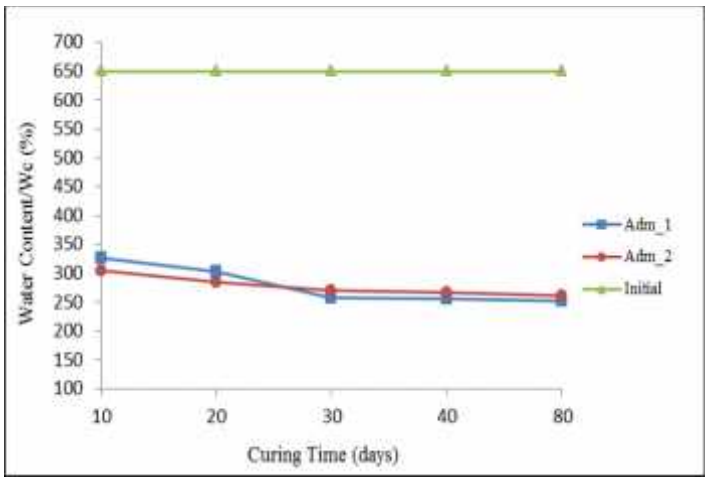

Figure 2. The effect of curing period to the water content of stabilized fibrous peat soil.

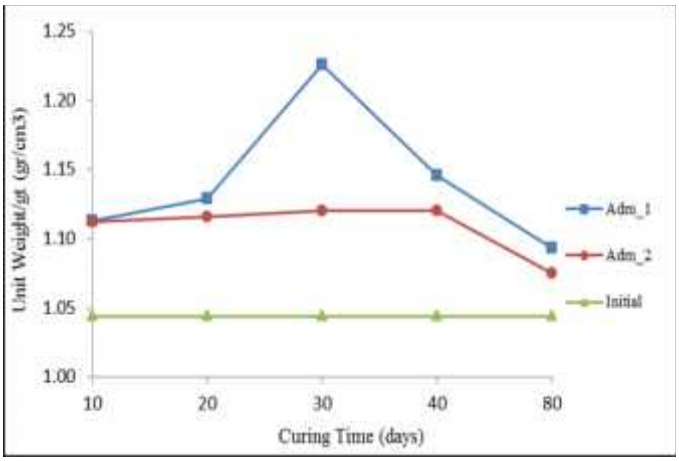

Figure 3. The unit weight of stabilized peat as a function of curing period.

higher than the one stabilized with admixture-2. It could be because admixture-1 has smaller granular size and good absorbent capacity so that the CaSiO3 gels can be developed faster than the admixture-2. It also could be because the admixture-2 is not mixed properly with the prepared peat sample so that the sample tested contains of less admixture-2; as a result, sample stabilized with admixture-2 has lower mineral content that means lower specific gravity. At 20 days curing period, the specific gravity of peat stabilized with admixture-2 increases very fast and it becomes about the same with the one stabilized with admixture-1. Until curing period reaches 80 days, the Gs value of both stabilized peat samples have similar behaviour that is Gs value slightly increases with the increase of curing period. This condition confirmed the argumentation that after the curing period reaches 30 days, there is no more water available in the macro pores so that no more CaSiO3 gel is developed. For peat stabilized with admixture-1, however, it gives slightly higher Gs than the one stabilized with admixture-2. It is also confirmed that this condition is caused by fly ash has smaller granular size and has better capability in absorbing water so that CaSiO3 gel can be developed more properly by admixture-1 than by admixture-2.

Figure 5 shows that shear strength of the stabilized peat increases significantly when the curing period is 10 days, especially for peat stabilized with admixture-1. Afterwards, the shear strength of peat stabilized with admixture-1 is not really affected by the curing period. On the other hand, shear strength of peat stabilized with admixture-2 increases significantly with the increase of curing period until it reaches 40 days. This behavior is due to the formation of CaSiO3 gel by admixture-2 takes place much slower than by admixture-1 which has smaller granular size and has better absorbent capacity. After 40 days, shear strength of peat stabilized with admixture-2 decreases with the increase of the

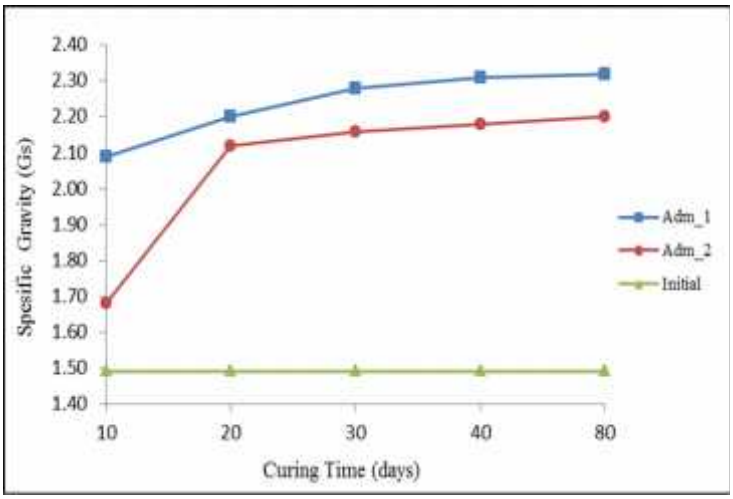

Figure 4. The increment of specific gravity, Gs, of the stabilized peat as a function of curing period.

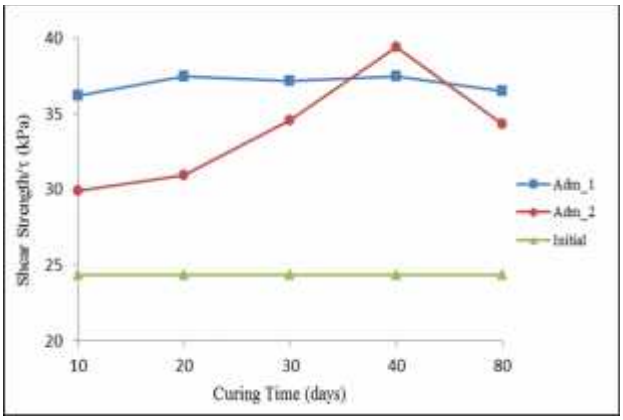

Figure 5. The effect of curing period to the strength of the stabilized peat

curing periods; this condition is not occurred in peat stabilized with admixture-1. It shows that process of fiber decomposition occurs earlier and faster in peat stabilized with admixture-2 especially for fibers which are not covered properly by the CaSiO3 gel.

Compression behavior of stabilized peat at different curing period can be seen in Figure 6. It shows that until the curing period reaches 30 days, the total compression decreases; it confirms that until 30 days of curing period the

CaSiO3 gel formation is still taking place so that the pores of stabilized peat is smaller that means its compression is also smaller. After 30 days of curing periods where no more water available in the macro pores, there is no more gel formation and the existing gel is gradually shrinking. It means that the macro pores of the stabilized peat becomes bigger and it causes the increment of peat compression. After 40 days of curing periods, the compression is still taking place due to the decomposition process of the peat fibers. Peat stabilized with admixture-2, however, shows bigger compression than the other sample; it is because formation of the CaSiO3 gel by admixture-2 is less proper so that more fibers uncovered by the gel undergone decomposition earlier.

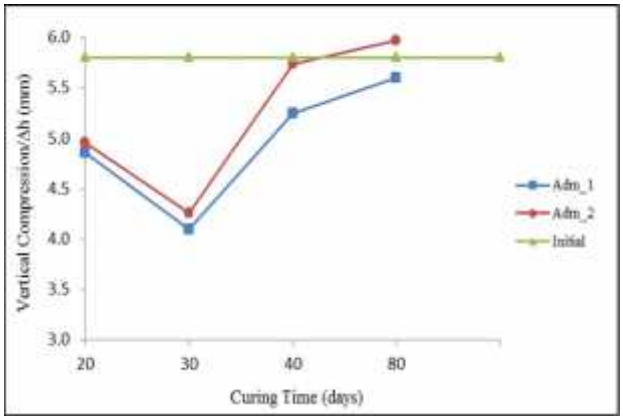

Figure 6. Compression behavior of stabilized peat at different curing periods.

CONCLUSSION

From data analyzed given above, it can be concluded as follows:

1. Physical and engineering properties of the stabilized peat increase until the curing period reaches 30 days; afterwards, they slightly decrease with the increase of curing periods.
2. Formation of CaSiO3 gel by Admixture-1 [70% Fly Ash (FA) + 30% Lime CaCO3] starts earlier and faster.
3. Formation of CaSiO3 gel takes place during water in the macro pores available; afterwards, the gel and the unwrapped (by the gel) fiber getting dry and decomposition process take place.
4. Fiber decomposition affects decrement of the physical behavior and shear strength of the stabilized peat; it also causes the increment of its compressibility.
5. Peat stabilized with admixture-1 [70% Fly Ash (FA) + 30% Lime CaCO3] gives better improvement of soil behavior than it is stabilized with admixture-2 [70% Rice Hush Ash (RHA) + 30% Lime CaCO3]

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THE PHYSICAL AND ENGINEERING PROPERTIES OF PEAT SOIL USING VARIOUS SAWDUST REINFORCEMENT

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ABSTRACT: This research is focus on the utilization of waste wood shavings (sawdust) for peat soil reinforcement with variations on the sawdust condition. This study aims to determine the change of the mixed peat soil behavior based on the waste sawdust condition. The variations of the sawdust used are original condition, burnt, and filtered. The soil used in this research is first tested for the ash content and organic content to determine that it is a peat soil. The peat soil then tested for it's physical and engineering properties in four condition, it is before mixed and after mixed with three various waste sawdust condition.The result of the four condition then compared to see the change of the behavior in it's properties. The results of the analysis obtained shows which condition of the waste sawdust used for peat soil reinforcement that gives the better result. In terms of physical properties, reinforcement with filtered sawdust gives the better result. It gives the increasing of soil density (specific gravity) to 16.284%; decrease in water content to 59.478%; increase in the volume weight of the soil to 11.101%; and a decrease in void ratio to 52.979% from the original condition. In terms of Engineering properties, the better result is using original sawdust. It gives an increase in soil shear strength to 13.761% and reduce the soil compression to 14.216% from the original peat soil condition. This result shows that the change of the physical and engineering properties is due to the shape of the sawdust that used an how it fills the peat soil pore.

Keywords: *peat, soil reinforcement, soil properties, sawdust*

BACKGROUND

Peat soil is a type of soft soil with low bearing capacity and high compressibility. Peat soil has an open texture where in addition to macro pores, peat soil texture is also dominated by micro pores inside the turf fibers. With dual pore system and variate degree of homogeneity, as well as soil bulk density approaching the unit weight of water, then the problem of compressibility can lead to a large settlement as well . Moreover, because it is a soft soil, it generally has a low bearing capacity. Study by Jelisic , Leppanen (1993) conclude that the peat soil bearing capacity is lower than the soft clay soil that could lead to collapse (bearing capacity failure) and that is a major problem for the structure to be built on it .

To fix the problems of high compressibility and low bearing capacity of peat soil, a few efforts have been done on improving the physical and technical properties of peat to increase the carrying capacity and reduce compressibility. This research is focus on the utilization of waste wood shavings (sawdust) for peat soil reinforcement with variations on the sawdust condition. In the previous research by Ma'ruf, M., A. and Ritonga, C.,S.,H. (2013) conclude that with percentage of 10% from the peat moist unit weight can increase the specific gravity to 16,284%, soil unit weight to 8,538%, and the shear strength to 13,825% from it's initiol peat condition. It is shown that waste wood shavings can be used for reinforcement of peat soil and can increase the shear strength of peat soil, although the increase is not quite significant. In terms of the waiting time after adding the sawdust to the peat, study by Ma'ruf, M., A. and Kristadi, P (2013) conclude that with 7 days waiting time showed that the consolidation settlement reduce to 14.216% from the initial peat condition.

In this study, waste wood shavings is used for reinforcement of peat soil and in the future might be able to act as an alternative for natural soil reinforcement material. The big amount of of wooden furniture industries and sawmills resulted in much more industrial wooden waste, one of which is wooden shaving waste material (sawdust). The waste is generally left strewn around the location of industrial or stacked on the side of the road. The use of sawdust is choose in hoping that the materials can fill in the pore and the sawdust is expected to reinforce peat fiber to add it's shear strength. The other reason is that sawdust wood is an organic material of natural origin so will not to damage the peat soil ecosystem. This study aims to determine the change of the mixed peat soil behavior based on the waste sawdust condition.

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RESEARCH BENEFIT

Results of this study may become one of the reference methods of peat reinforcement for field practitioners and civil construction on peat. Also to add some new reference in geotechnical science especially in fibrous peat soil reinforcement and waste utilization.

RESEARCH METHOD

There are two phases of research conducted in the laboratory scale. Phase I is determining the Physical and Engineering Properties of peat soil in it's initial condition, and Phase II is for the peat soil after adding various waste wood shavings (sawdust) reinforcement. The expected result is to obtain a waste sawdust condition which can provide maximum results.

Phase I: Defining The Peat Soil Characteristics in it's Initial Conditions

The sequence of work for Phase I as follows :

1. Taking peat samples from Sambang Lihum (Sungai Tabuk, South Kalimantan).
2. Prepare the plastic box with size 34x25x19 cm (common conmtainer box that easily found in the market) as shown in Figure 1. This box will also be function as a mixing place during the waiting time.

Figure 1. Peat Soil Sample Inside The Plastic Box

3. Ensure the soil sample is indeed a peat soil by doin ash content and organic content laboratory test as shown in Figure 2.

Figure 2. Ash Content Laboratory Test

4. Determine the physical and engineering properties of peat soil in it's disturbed conditions..

Phase II: Defining The Peat Soil Characteristics After Mixed With Various Conditions of Sawdust

The sequence of work for Phase II after finishing the Phase I is as follows :

1. Setting up the waste sawdust material in different condition. The variety of sawdust condition used in this research is powder sawdust, sawdust, and burned sawdust with the percentage of 10% of the moist unit weight of peat. Samples of waste wood shavings used come from the furniture industry and sawmill located in Banjarbaru and surrounding areas as shown in Figure 3.

Figure 3. Powder sawdut, srough sawdust, and burned sawdust

2. Adding and mixing each sawdust variation which has been prepared in step no. 1 to the peat soil inside the plastic box and wait till 7 days before testing. The mixing proces is done gently by hand to prevent the broken fiber inside the peat soil as shown in Figure 4 to Figure 6.

Figure 4. Adding and Mixing the Powder Sawdust

Figure 5. Adding and Mixing the Rough Sawdust

Figure 4. Adding and Mixing the Burned Sawdust

3. Testing each specimen which has been prepared in steps 2 after 7 days to determine the physical and engineering properties.

Figure 5. Peat Laboratory Testing

4. Conduct analysis of all the results of tests performed on initial and reinforced peat soil to determine the results of the stabilization and determine optimum stabilization material good of the three variations.

RESEARCH RESULT

Ash Content and Organic Content Check

This research was conducted as a preliminary penelitiaan which aims to determine the ash content (Ac) and organic content (Oc), Ash content test conducted as a preliminary study that aims to determine the ash content (Ac) and organic content (Oc), to know that the land included in the criteria tested peat. Based on tests conducted obtained value of Ac and Oc as shown in Table 1.

Table 1. Ash Content and Organic Content Test Result

Container No.	unit	1	2	3
1. W_{cont} + W_{soil} (oven for 24 hours)	g	32.93	36.11	44.39
2. W_{cont} + W_{soil} (burned 800^0 c)	g	31.86	34.33	42.4
3. W_{cont}	g	30.66	32.43	40.12
4. Ash Content (Ac)	%	52.86	51.63	53.39
5. Organic Content (Oc)	%	47.14	48.37	46.61

(Source: Primary Data)

From Table 1 can be seen that the soil samples is peat soil, because it's usually referred to as the peat when the organic material content (Oc) in the soil exceeds 30 % and the content of Abu (Ac) meets the requirements of the USSR (Mankinen and Gelfer 1982) as shown in table 2.

Table 2. Peat Soil Classification

system		OSRC (andrejko et al 1983)		Jarret (andrejko et al 1983)		Davis 1946	USSR Mankinen & gelfer 1982)		LGS (kearns & Davidson 1983)	ASTM D4427-92 (1997)
Ash content (%)	5	PEAT	Law ash	PEAT		PEAT	PEAT	1	PEAT (inorganic texture)	PEAT
	10		Medium ash					2		
	15							3		
	20		High ash					4		
	25									
	30	CALCARAREOUS SEDIMENT	Law ash	M U C H	PEAT Y			5	PEATY MUCH (inorganic texture)	O R G A N I C S O I L S
	35									
	40					MUCH		6		
	45									
	50								MUCH (inorganic teksture)	
	55		high ash		Clayey/ Silty/ Sandy/ Gravelly/		NON-PEAT			
	60									
	65								inorganic teksture MUCH	
	70									
	75									
	80	MINERAL SIDIMENT		ORGANIK CLAY OR SILT		MINERAL SOIL				
	85									
	90								MUCKY inorganic teksture	
	95									
	100								Inorganic teksture	

(Source: E.C. Leong & C.Y. Chin (1999))

Phase I

Peat soil samples obtained in disturbed conditions from Sambang Lihung, South Kalimantan. Tests conducted at the Soil Mechanics Laboratory, Lambung Mangkurat University, Banjarbaru to determine the engineering characteristics and the physical properties of peat soil. Tests in the laboratory for physical properties of peat include Specific Gravity (Gs) tests, water content, unit volume of soil, organic content and ash content. The results can be seen in Table 3 .

Table 3. Initial peat soil characteristics

No	Parameter		Test Result
	Phisycal Properties		
1	*Specific Grafity (Gs)*	-	1,744
2	Water Content (w)	%	553,96
3	Soil Unit Weight (ɣ)	t/m^3	1,054
4	Void Ratio (e)	-	9,821
5	Organic Content (Oc)	%	47,37
6	Ash Content (Ac)	%	52,62
	Engineering Properties		
7	Shear Strength	kg/cm^2	0,021
8	Consolidation Settlement	mm	2,04

(Source: Primary Data)

Shear strength on fibrous peat soil is largely determined by the level of fiber it contains. Peat soil shear strength value before stabilized amounted to 0,021 kg / cm^2 or 2,1 kPa. This value is obtained by calculations based on the assumption of working load 50 kPa (traffic load, pavement load and embankment work load on the ground peat) .

To find the compressibility of peat soil test used one stage consolidation (Adams , JI 1961) (Dhowian , A , W and TB Edil 1980) (Duraisamy , Y. , Huat . Single . BK , Aziz . Azlan A. , 2007) with the consolidation time of 8 days (Mochtar , NE 1985) and the result is shown in Figure 6.

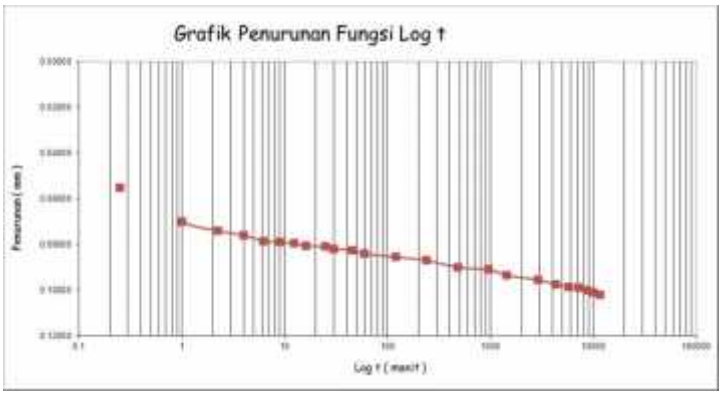

Figure 6. Initial Peat Consolidation Test Curve

Phase II

As described in the research method, peat soil samples and then reinforced by powder sawdust, sawdust, and burned sawdust to improve it's physical and engineering properties. The ammount of the admixture that is used within the percentage of 10% of the moist unit weight of peat, and then brooded for 7 days before tested. Similar to the initial peat condition in phase I, physical and engineering properties of peat soil which has been reinforced conducted in laboratory results will be compared with conditions before stabilized to see the improvement.

Water Content

From Figure 7, the tendency in water levels is decrease along with the variation of sawdust. It appears that the value of water content decreased from the initial moisture content of 553,96 % ; the rough sawdust reinforced of 231.28 % ; a burned sawdust reinforced by 236,54 %; and a powder sawdust reinforced by 224,47 %. From this description it can be concluded that within the water content, the powder sawdust reinforced is the most optimum, because the sawdust is screened in the form of a fine powder making it possible to fill the peat pores and prevent the water from filling the pores.

Figure 7. Water Content vs Variation of Sawdust

Soil Unit Weight (γt)

Soil unit weight (γt) is different due to variations of sawdust. This is due to differences in the reaction of sawdust fragments that fills the pores giving the reinforced peat a different density. Figure 8 showed different soil unit weight value from the initial condition of 1,054 g/cm^3, the rough sawdust reinforced of 1,091 g/cm^3, a burned sawdust at 1,156 g/cm^3, and a powder sawdust at 1,171 g/cm^3. The biggest unit weight provided by a mixture of powder sawdust, because there is the possibility of sawdust used is able to fill the pores in the peat soil that affect the weight of the soil volume .

Figure 8. Soil Unit Weight vs Variation of Sawdust

Specific Gravity (GS)

Determining the specific gravity in this study is done by using 3 picnometer 250 ml which calibrated in advance in order to determine the weight of the kerosene at a certain temperature, weight of the soil sample used is 50 grams. The peat soil plus kerosene is boiled, then cooled to room temperature (28 ° C) and then add the kerosene until it is full, and then the weight is measured. Based on Figure 9 obtained that the optimum value Gs occurred on powder sawdust.

Figure 8. Spesific Grafity vs Variation of Sawdust

Void Ratio (e)

Figures 9 shows the difference of void ratio according to variation of sawdust. The void ratio differs from the initial value of 9,8193 to 4.9351 using rough sawdust reinforcement; using burned sawdust to 4,793 ; and the powder sawdust to 4,617. From this description it can be concluded that in terms of void ratio, then the powder sawdust is the most optimum to use because of its size in the form of a fine powder capable of filling the pores of the peat .

Figure 9. Void Ratio vs Variation of Sawdust

Compressibility (Consolidation Test)

Consolidation test obtained total compression and peat soil Settlement. Total compression in general is different due to variations of sawdust condition, wherein the difference in the ability of sawdust fragment to withstand the vertical force is different, so the compression is variate. In this case the optimum results occur in a variation of rough sawdust

that gives smaller compression than other. This because the shape of the rough sawdust in the form of flakes and fragment, so it can reinforce the peat soil to withstand the vertical force, as shown in Figure 10 .

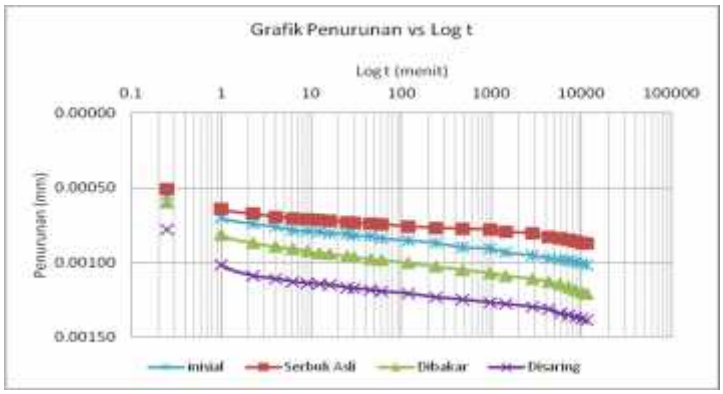

Figure 10. Consolidation Settlement vs Variation of Sawdust

Shear Strength

Peat soil shear strength parameters are associated with fibers from peat soil and unit weight parameters. This is because the peat is a frictional material /non kohesive material (Adams 1965) where shear strength depend on the bond between grains of the solid (fiber) peat. From Figure 11 shows that the value of different peat soil shear strength along with the variation of sawdust. In this case the optimum results occur in rough sawdust. This is due to the shape of the rough sawdust so it can withstand the lateral force.

Figure 10. Consolidation Settlement vs Variation of Sawdust

Data Analysis

Judging the results of research and calculation as shown in Table 4 can be seen which variation of sawdust that gives the better result for reinforcement. In terms of physical properties of soil, use of powder sawdust gives the better result. It can increase the specific gravity of 16,284%; reduction in water content (w) to 59,478%; increase soil unit weight (γ) of 11,101%; and a decrease in void ratio (e) amounted to 52,979% from the initial condition. This is because the powder sawdust is finely filtered so as to fill pores in peat soil in which we know that peat has many pores on each fiber. In terms of the engineering properties of reinforced peat using powder sawdust, shear strength increase to 13,761% and the consolidation settlement reduce to 14,216%.

In terms of engineering properties, from direct shear test obtained the value of *c* (cohesion) and φ (friction angle). In the direct shear test, the value *c* are different in every variation and the value of φ (friction angle) increases, therefore this discussion focuses on the value of *c* only. In rough sawdust reinforcement, value of *c* decreased from the initial condition due to stickiness between grains of peat soil is removed by a mixture of sawdust. In terms of burned sawdust, the value of *c* equal to the initial sample because peat is rotten fibers while sawdust is burned, so it can be said that cohesion of peat soil remain unchanged. In using powder sawdust reinforcement, *c* increased from the initial condition due to the increase of soil particle surface area.

Table 4. Physical and Engineering Properties for all Variation of Sawdust

No	Parameter		Initial Condition	Sawdust Reinforcement		
	Physical Properties	Unit		Rough	Burned	Powder
1	*Specific Grafity (Gs)*	-	1,744	1,954	1,989	2,028
2	Water Content (w)	%	553,95	231,28	236,54	224,47
3	Unit Weight (γ)	g/cm^3	1,054	1,091	1,155	1,171
4	Angka Pori(e)	-	9,819	4,935	4,793	4,617
	Engineering Properties					
5	Shear Strength	kg/cm^2	0,0218	0,0248	0,0248	0,0245
6	Consolidation Settlement	mm	2,04	1,75	2,42	2,78

SUMMARY

From laboratory testing of initial peat condition and reinforced peat soil using various condition of , the analysis of the data obtained, and the discussion of results, this research reach some conclusions as follows :

1. On the physical properties the powder sawdust gives the better result. The parameters are obtained: Specific Grafity of 2,028 from the initial condition of 1,744; water content (w) amounted to 224,47 % of the initial which amounted to 553,95 %; Unit weight (γ) of 1,171 g/cm^3 of the initial 1,054 g/cm3; and the void ratio (e) amounted to 4,617 from the initial of 9,819. This is because the powder sawdust is finely filtered so able to fill pores in peat soil.
2. In terms of engineering properties the rough sawdust gives better result. The parameters are obtained: shear strength (τ) of 0,0248 kg/cm^2 from the initial 0,0218 kg/cm^2 and the consolidation settlement of 1,75 mm from the initial 2,04 mm. This is due to the shape of the rough sawdust so it can withstand the vertical and lateral force.

ADVISE FOR FUTURE RESEARCH

Research for utilization of waste wood shavings for reinforcement of peat soil can later be continued for field -scale research to determine the real extent and depth that should be mixed with sewage. It also can be further studied for the chemical content of waste wood shavings and possible reactions that can occur with peat soil in order to look at possible uses for the chemical stabilization

ACKNOWLEDGEMENTS

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PARAMETRIC STUDY OF LATERAL DISPLACEMENT DUE TO PILE GROUP INSTALLATION

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ABSTRACT: Pile installation generates lateral ground movement and excess pore pressure surrounding it. Both are causing problem to the existing adjacent earth structures. The objective of this paper is to do sensitive analysis of lateral ground movement due to pile group installation. Pile numbers, diameter, and spacing are the parameters to be considered. Plaxis is utilized to do numerical analysis and pile installation is modeled using displacement controlled cavity expansion. The lateral displacement – pile group correlation charts are proposed based on the simulation results. The charts are useful to predict the lateral displacement on pile group design.

Keywords: *Pile Group, Lateral Ground Movement, Pile Installation, Numerical Analysis.*

INTRODUCTION

Pile installation generates lateral ground movement and excess pore water pressure surrounding it. Both are causing problem to the existing adjacent earth structures. Poulos (1994) describes the displacement of soil while installation of the following: (i) horizontal movement relatively uniform at a depth of about 0.8*L*, where *L* is the depth of penetration of the time of pile installation, (ii) The movement of the soil under the pile tip decreases rapidly according to its depth, and (iii) the maximum displacement does not depend on the depth of penetration and decreases with distance from the pile.

Prediction radial displacement of soil caused by the installation of a single pile can be approximated by cavity expansion theory given by Carter et al (1979), Vesic (1972), and also Chai et al (2005, 2007). Francescon (1983) and Pestana et al (2002) conducted experiments on soil movement due to the installation of the pile into the clay soil is saturated, the results are also consistent with the theory of cavity expansion.

Broere and Van (2006), as well as Pham (2009) has conducted a simulation jacked pile. The simulation using Plaxis 2D with axisymmetry model and the wall along the cavity which represents the pile applied displacement (prescribed displacement). The numerical results give void ratio and stress values around the pile that is acceptable when compared with the results of centrifuge tests. Stress-controlled methods cavity expansion and displacement - controlled cavity expansion into appropriate methods for the numerical simulation of displacement pile installation (Satibi et.al, 2007). For pile installation, the result will be more thorough if approached with cavity expansion method and theory of displacement (Maryono, 2014).

In the previous paper has explained the numerical simulation for analysis of lateral displacement and excess pore water pressure due to the installation of single pile. Numerical simulations using the displacement method - controlled cavity expansion results lateral displacement and excess pore water pressure when compared to the corresponding cavity expansion theory given by Carter et al (1979) and Randolph et al (1979).

Generally lateral displacement issues involving more than one pile, so it takes prediction lateral displacement due to the installation of a pile group. In this paper will be presented graphs / charts in order to facilitate the prediction lateral displacement due to the installation of pile groups.

SOIL PARAMETERS AND GEOMETRY

In this numerical analysis, pile parameter considered as linear elastic with parameters as in Table 1. As for soil behavior to be considered as an elastic-plastic model of Mohr Column using the parameters in Table 2, where the soil layer is considered as a homogeneous soil with undrained condition.

Table 1. Pile Parameter (Linier-Elastis)

Parameters	Unit	Values
Unit weight ( )	kN/m^3	24
Poisson's Ratio (v)	-	0,2

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Young's Modulus (E)	MPa	23500

Table 2. Soil Parameter (Mohr-Column)

Parameters	Unit	Values
Unit weight ( )	kN/m^3	15,66
Poisson's Ratio (ν)	-	0,35
Young's Modulus (E)	kPa	3300
Cohession (c_u)	kPa	11
Friction Angle ( )	^{0}	0,8

(References: Soil investigation reports of Project X)

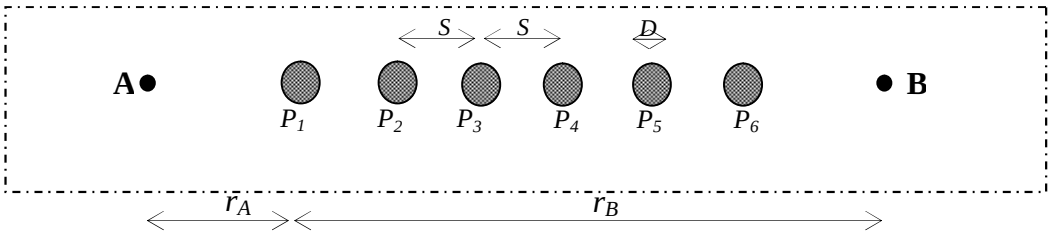


Figure 1. Plan - Pile Group Model

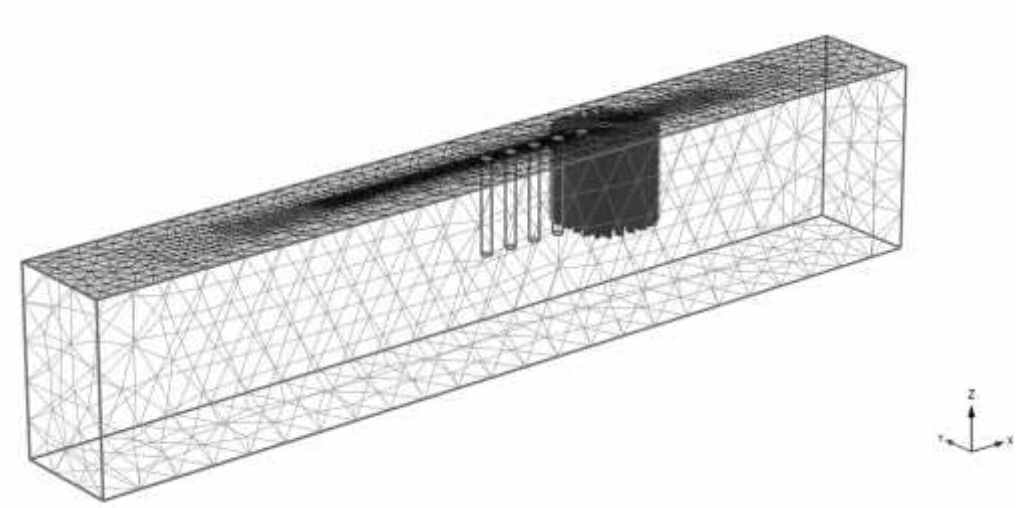


Figure 2. Geometry Model in Plaxis 3D

Pile group modeled extending in rows direction as shown in Figure 1. The diameter D of the pile is 30 cm and 50 cm of length L pile is 5 m, space of piles S taken as 2,5D and 3D, Total n piles is determined to 6 piles, point A and B are points to be reviewed when the lateral displacement simulation pile installation that begins from P1 to P6. Radial distances r_A and r_B taken from 5D to 30D. The geometry modeled on the Plaxis 3D as shown in Figure 2. Meshing elements are determined in such a way to get results when compared with the theory according to single pile given by Carter et al.

SIMULATIONS

Simulations performed using the displacement method - controlled cavity expansion. The process begins by setting the initial conditions. There after, the pile element is removed to create a cavity along the center line, then apply the priscribed displacement horizontal direction uniformly on the walls of the cavity. In addition, the vertical stress ($\sigma'_v = \gamma \cdot L$) is also applied in the bottom of the cavity, where L is the length of the pile is embedded. Stages of numerical simulation pile group for P1 to P6 are presented in Table 3, where the direction of installation begins from the point P1 to leave point A and point to point B. For Phase-1 (P1), simulation results compared with the cavity expansion theory given by Carter et al (1979), namely:

$$\square = (r^2 + r_0^2)^{0,5} - r \quad (1)$$

where $\square\square$ is lateral displacement of soil. The symbols r and r_0 denote the horizontal distance between the point reviewed and the center of pile and the outer radius of pile. The numerical simulation conducted consists of 4 types of simulations with variations in diameter D of pile and spacing of pile S as shown in Table 4.

Table 3. Numerical Simulation Stages

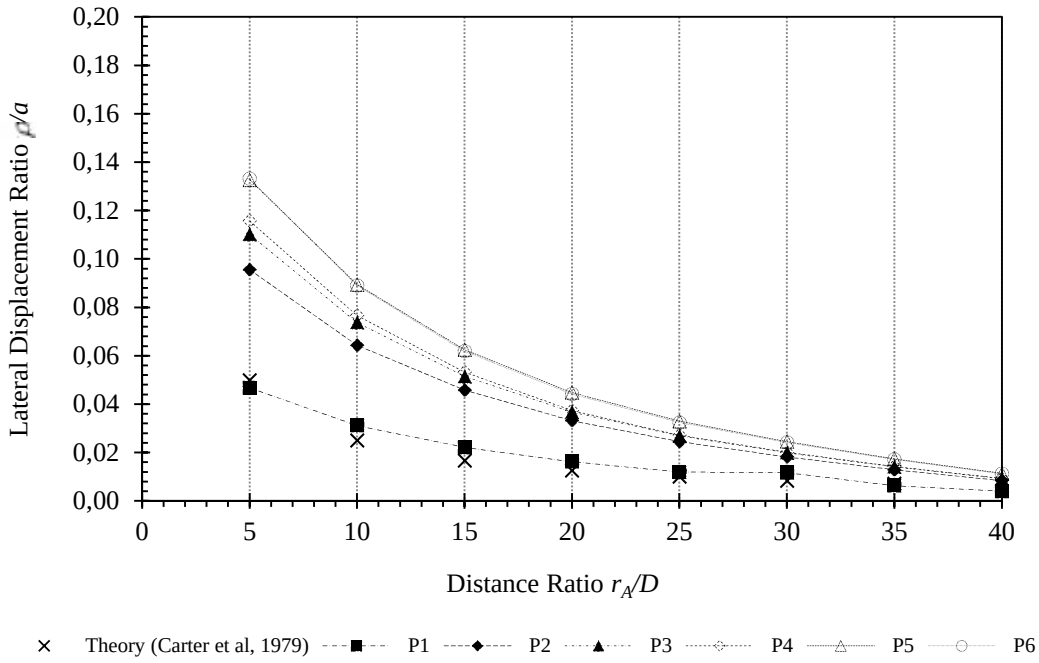
Phase	Construction Phase
Initial	Initial Stress
Phase-1 (P1)	Created cavity and prescribed displacement/stress for P1.
Phase-2 (P2)	Apply pile material and removed prescribed displacement/stress for P1. Created cavity and prescribed displacement/stress for P2.
Phase-3 (P3)	Apply pile material and removed prescribed displacement/stress for P2. Created cavity and prescribed displacement/stress for P3.
Phase- 4 (P4)	Apply pile material and removed prescribed displacement/stress for P3. Created cavity and prescribed displacement/stress for P4.
Phase-5 (P5)	Apply pile material and removed prescribed displacement/stress for P4. Created cavity and prescribed displacement/stress for P5.
Phase-6 (P6)	Apply pile material and removed prescribed displacement/stress for P5. Created cavity and prescribed displacement/stress for P6.

Table 4. Simulation Type

Type	Diameter D, cm	Space S, m
Simulation - 1	50	3 D
Simulation - 2	50	2,5 D
Simulation - 3	30	3 D
Simulation - 4	30	2,5 D

RESULT & ANALISYS

Figure 3a and 3b show the correlation between the ratio of the lateral displacement of pile radius $\square / a$ from Simulation-1, where the pile diameter D in this simulation is 50 cm and the spacing of pile S is 3D. Notation r_A is the distance between point A to pile P1, while r_B is the distance between point B to the pile P1. For Phase-1 (P1), The lateral displacement $\square\square\square$ from Simulation-1 is similar when compared with the results of calculations based on the theory of cavity expansion given by Carter et al , this is because the simulation of Phase-1 (P1) is single pile simulation.



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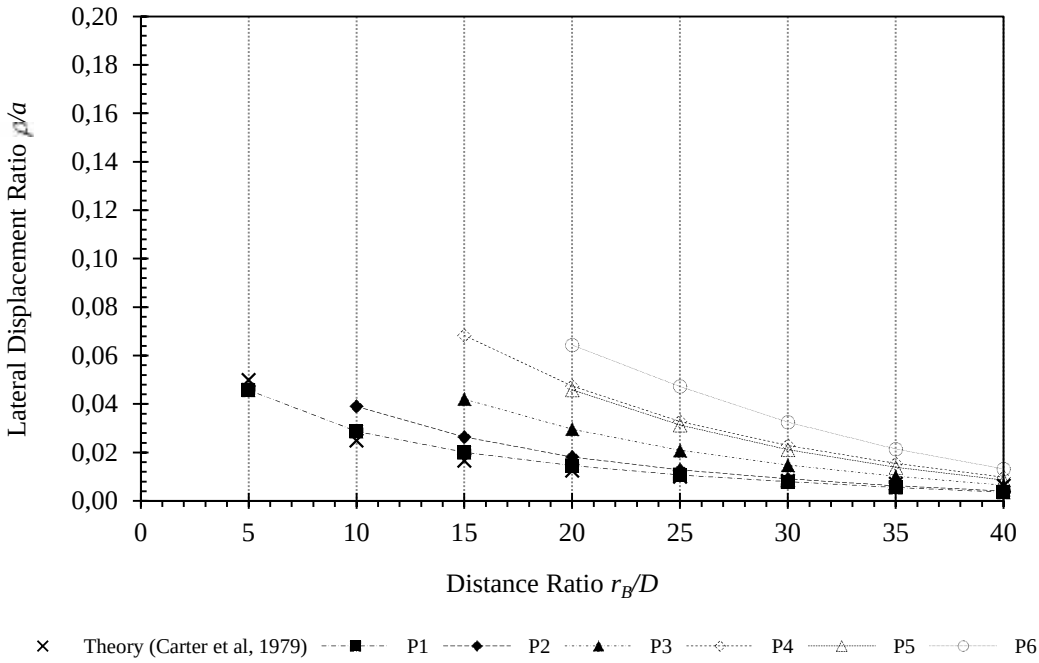
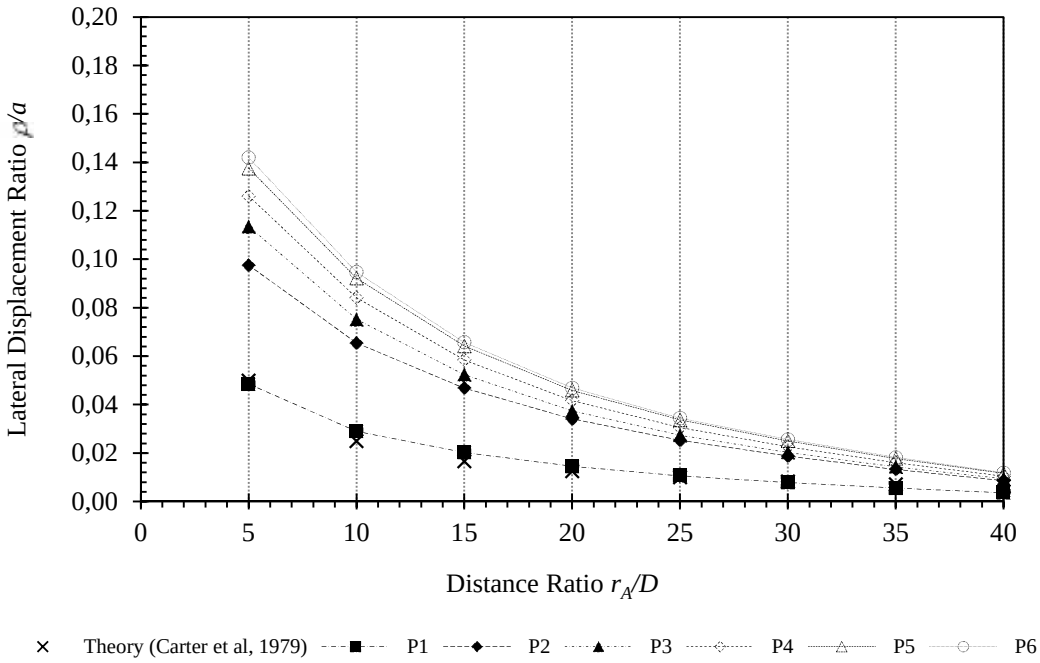
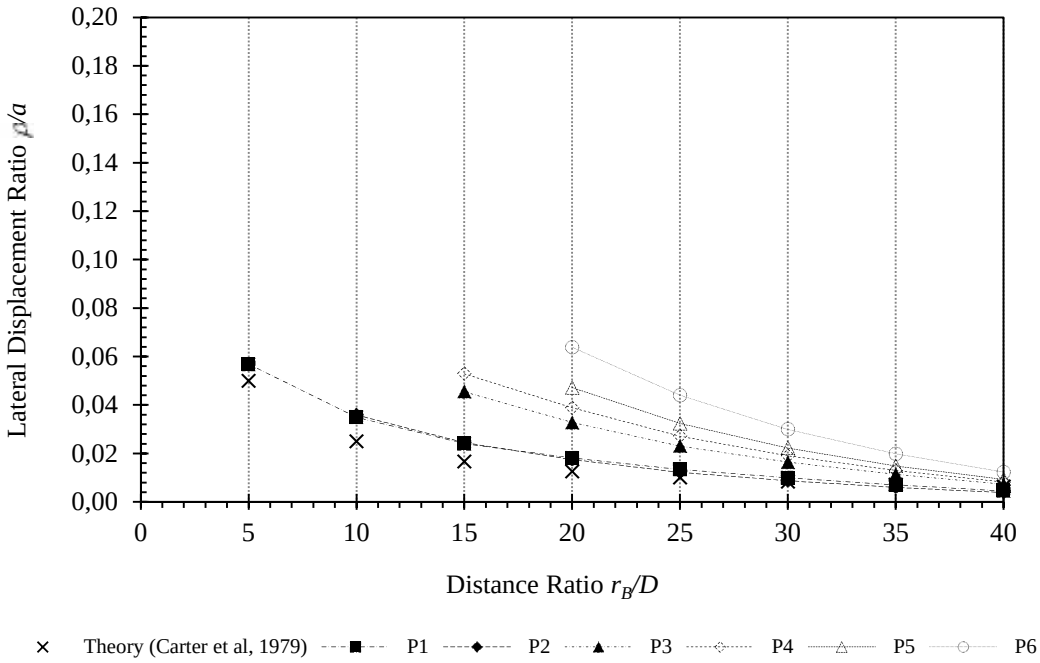


(b)

Figure 3. Lateral Displacement of Numerical Simulation Results Plaxis 3D, Diameter D = 50 cm and Space of Pile S = 3D: (a) Point A, (b) Point B.



(a)



(b)

Figure 4. Lateral Displacement of Numerical Simulation Results Plaxis 3D, Diameter D = 50 cm and Space of Pile S = 2,5D: (a) Point A, (b) Point B.

The results of Simulation-1 at point A (Figure 3a) shows the direction of pile instalation is getting away from a point (Point A). For Phase-1 to Phase 6 (Pile pile P1 to P6) showed that the displacement lateral □ is increased as the number of piles n. For Phase-6, the value of the displacement ratio □□/ a at point A with distance r_A = 5D is 13.31% and then decreased to 1.12% at a distance r_A = 40D, this show that when the distance r_A (point P1 to point A) increasingly, the lateral displacement ratio is decreases. At a distance r_A = 20D □x lateral displacement changes from Phase-5 to Phase 6 is 0.2 mm smaller than the lateral displacement changes □x from Phase-1 to Phase-2 is equal to 4.2 mm, this shows the lateral displacement changes □x tends to shrink as the numbers of piles n.

The results of Simulation-1 at point B (Figure 3b) show the direction of pile instalation that closer to a point (Point B). The results of simulation show that the direction of installation may affect the value of lateral displacement □.

When the spacing distance r_B (point P1 to point B) is increasing, then the lateral displacement ratio decreases. For Phase-6, the value of the displacement ratio □□/ a at point B with the distance r_B = 40D is 1.32%, where as for the distance r_B = 20D is 6.44%, this value is greater than the value of the displacement ratio □□/ a in A point with a distance r_A = 20D is 4.41%. However, in Phase-2, the ratio of lateral displacement □□/ a at point B looks smaller than the lateral displacement ratio □□/ a at point A at the same distance ratio.

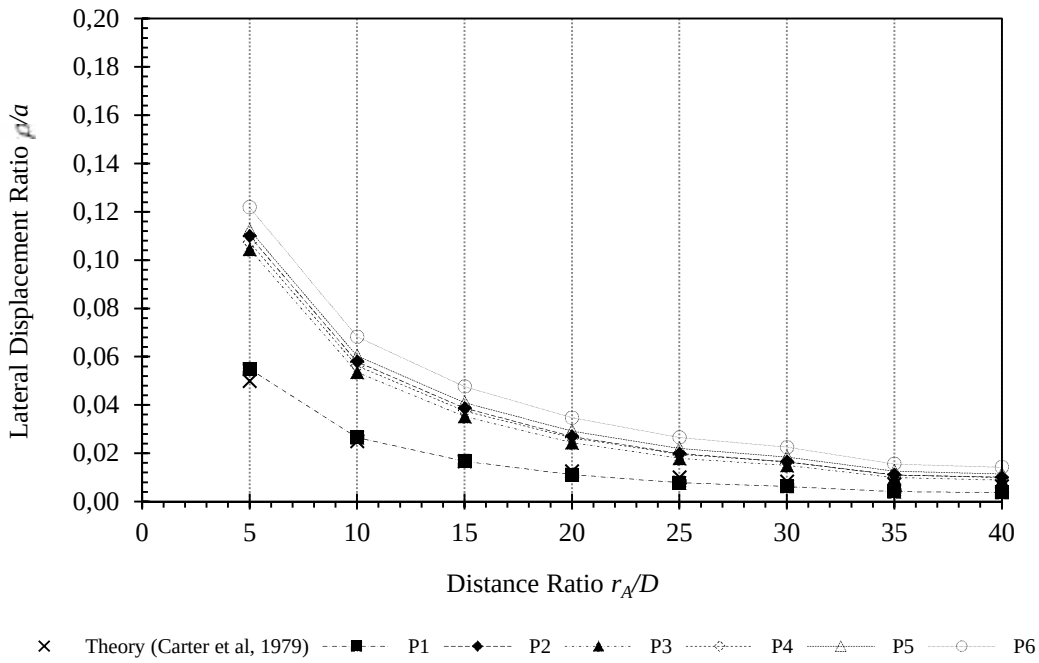
Figure 4a and 4b show the correlation between the lateral displacement ratio of radius □□/ a from simulation-2. The result of Simulation -2 at point A (Figure 4a) for Phase-1 to Phase-6 (P1 to P6) provides trends displacement ratio □□/ a is similar as the results of Simulation-1 at point A (Figure 4a), which in Phase-6 displacement ratio value □□/ a at point A with distance r_A = 5D is 9.47% and then decreased to 1.19% at a distance r_A = 40D. The result of Simulation-2 at point B (Figure 4b) also provides trends displacement ratio □□/ a is equal to the Simulation-1 at point B, which in Phase-6 displacement ratio value □□/ a at point B with the distance r_B = 40D is 1,23%, where as for the distance r_B = 20D is 6.38%, this value is greater than the value of the displacement ratio □□/ a at point A with distance r_A = 20D is 4.69%.

Figure 5a and 5b shows the correlation between the lateral displacement ratio of radius □□/ a from Simulation-3, As Simulation-1 and Simulations-2, The results of Simulation-3 for Phase-1 (Single pile) provide sufficient appropriate lateral displacement □□□current compared with the results of calculations based on the theory of cavity expansion given by Carter et al. The result of Simulation-2 in point A (Figure 5a) for Phase-1 to Phase-6 (P1 to P6) provides trends displacement ratio □□/ a is similar as the results of Simulation-1 and Simulation-2 in point A (figure 5a), where in the ratio of the value of the displacement Phase-6 □□/ a at point A with distance r_A = 5D is 7.65% and then decreased to 1.63% at a distance r_A = 40D. The result of Simulation-2 at point B (Figure 4b) also provides trends displacement ratio □□/ a similar with results of Simulation-1 and Simulation-2 at point B. For Phase-6, displacement ratio value □□/ a at point B with the distance r_B = 40D is 1.13%.

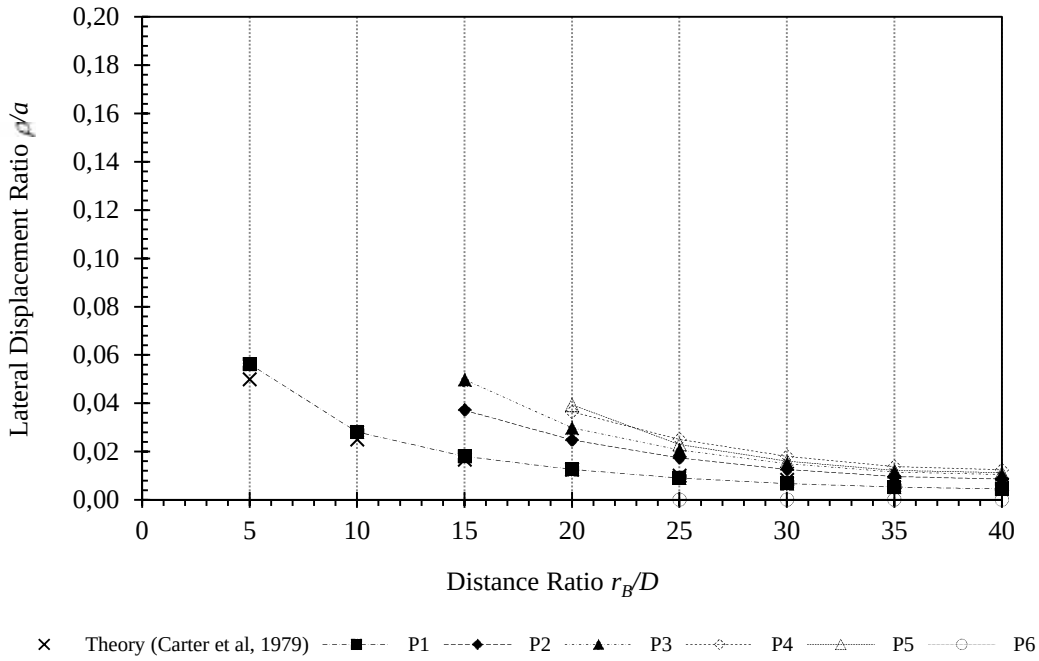
Figure 6a and 6b show the correlation between the lateral displacement ratio of radius □□/ a from simulation-4. The result of simulation-4 at point A (Figure 6a) for Phase-1 to Phase-6 (P1 to P6) provides trends displacement ratio □□/ a is similar as the results of previous simulation at point A (Figure 6a), where as in Phase-6 displacement ratio value □□/ a at point A with distance r_A = 5D is 8.41% and then decreased to 1.58% at a distance r_A = 40D. The results of Simulation -2 at point B (Figure 6b) also provides trends displacement ratio □□/ a is equal to the previous simulation at point B, which in Phase-6 displacement ratio value □□/ a at point B with the distance r_B = 40D is 0 , 98%.

Of the four aforementioned simulation can be seen that the simulation gives the similar trend that this shows that when the distance r_A (P1 to point A) and the distance r_B (P1 to point B) is increasing, then the lateral displacement ratio decreases, whereas in distance 40D, the value of lateral displacement ratio is very small, this is less than 2%.

There is little difference between the results of simulation-1 to simulation-4 for Phase-1 (single-pile) with the theory given by Carter et al is due to the determination of element mesh, where the finer the mesh arrangement of the elements then the result will be better.

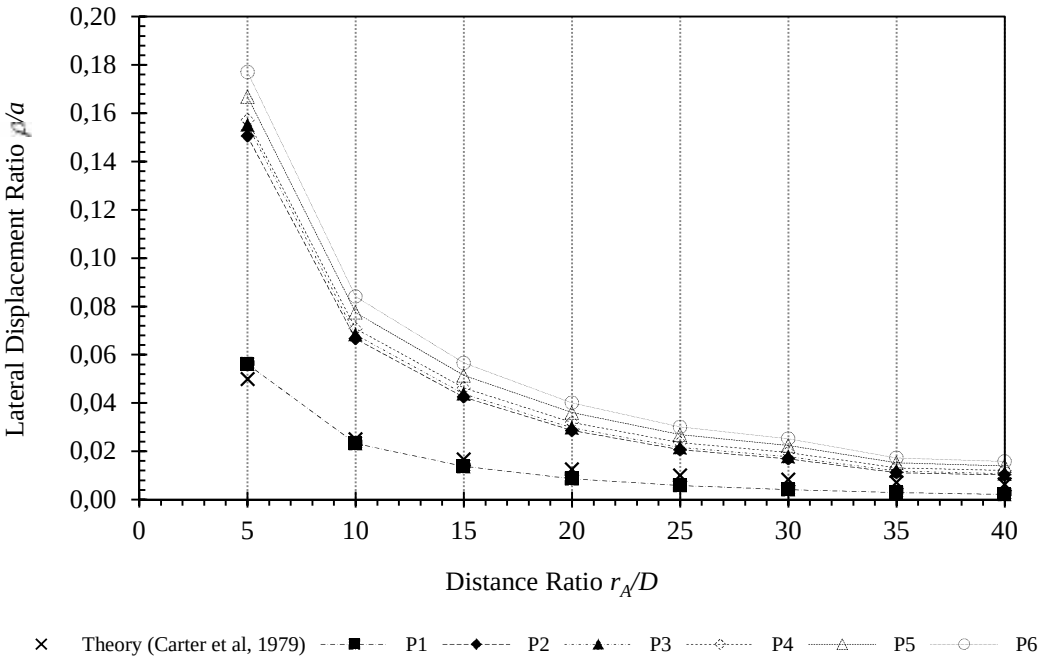


(a)

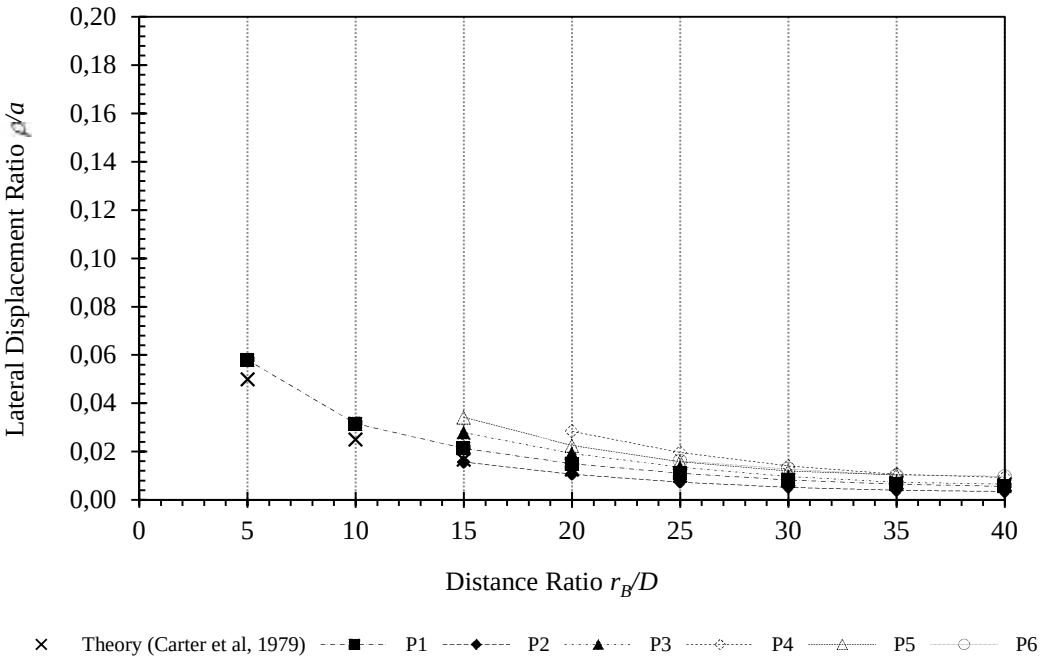


(b)

Figure 5. Lateral Displacement of Numerical Simulation Results Plaxis 3D, Diameter D = 30 cm and Space of Pile S = 3D: (a) Point A, (b) Point B



(a)



(b)

Figure 6. Lateral Displacement of Numerical Simulation Results Plaxis 3D, Diameter D = 30 cm and Space of Pile S = 2,5D: (a) Point A, (b) Point B

CONCLUSION

From the analysis and the discussion above, we can conclude several things:

- When the radial distance r farther from the pile, the value of lateral displacement has narrowed.
- The Simulation result for Phase-P1 provides sufficient lateral displacement according when compared with the results of calculations based on the theory of cavity expansion given by Carter et al
- Lateral displacement increases as the number of pile.
- The sequence of pile installation genereted affects of lateral displacement

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EXCAVATION ADJACENT TO THE EXISTING BUILDING: A STUDY CASE

M. F. Ma'ruf [1] and H. Darjanto [2]

ABSTRACT: A 17 stories building is planned to be erected at the crowded area. A 4 floors office and 2 floors house are its right and left neighbors. 2 floors surgery building of the hospital is right behind the project area. Of 3.2m depth will be excavated at the area for basement in which the ground water table is at 2m depth. The excavation could trigger instability for the surrounding buildings, especially when the dewatering process takes place. This paper intends to describe the analysis of the excavation process and the inferred ground stability. The model simulation by means of Plaxis 2D 2011 recommends the modification of construction stages drastically. Local excavation at the pile cap area is the best method to generate the smallest deformation on the nearby area.

Keywords: *excavation, ground stability, construction stage, dewatering*

INTRODUCTION

The high rise building has been planned to be built in the middle crowded building. A 4 floors office and 2 floors house are its right and left neighbors. 2 floors surgery building of the hospital is right behind the project area. Of 3.2 m depth excavation will be conducted at the area for pile caps in which 4.5 m depth has to be dug at lift area. The water table exists at 2 m depth. The excavation work is susceptible to affect the surrounding building. Careful planning of excavation should be carried out to minimize those effects.

The excavation next to existing building is one of the most problems in geotechnical practices. Research and investigation in this area had been conducted till recent time. Ramadan et al. (2013) revealed that contiguous pile succeeded to reduce lateral movement but not below the piles and the bottom of excavation. Sanjay et al. (2005) proposed grouting method combined with bore foundations to stabilize the slope of deep excavation. Meanwhile, a careful investigation was conducted by Hui (2014). He did numerical simulation on three different condition of deep excavation: slope excavation, cantilever sheet pile wall and retaining pile combined with bracing support. He recommended the retaining pile combined with bracing support among other methods. However, he revealed also that the stabilization method fully depend on the field and soil condition. Moreover, Koff (2009) did scrutiny work to access the excavation effects in the soft soil area. He proposed damage criteria to classify the building condition affected by the excavation stages. He demonstrated that the excavation induced displacement to cause the building damage depended on the stiffness of the building in axial and bending modes and the interface between soil and foundation and between foundation and building.

This paper intends to discuss about excavation case in a commercial high rise building project in Surabaya. The paper emphasizes on the excavation construction stage sequence.

PROJECT DESCRIPTION

The project was construction of 17 stories building. It had 42 columns. The number of piles to support column are vary from 3 to 12 piles. The piles had 21 m long. The piles were jacked to 24 m depth. The 3 m space above the piles to the ground surface was prepared for the pile caps and the basement. The basement was planned at 1.5 m depth supported by 50 m thick plate floor and pile caps with 1 m thickness. To stabilize the excavated soil, 6 m soldier piles was installed with 50 cm spacing. Meanwhile, the dead man piles were set 1 m from the soldier piles with 1 m spacing. The detail actual piles installed in the area can be checked in Fig. 1.

In the beginning, the contractor planned to do 3.2 m depth excavation at all the building area. However, the experience on the excavation project at the neighborhood location led the contractor to do careful examination on the excavation stage planning.

Standard Penetration Test (SPT) results shown in Fig. 2 show that the soils were soft to medium types. The upper layers to 8 m depth were loose and medium sand. About 3 m thick of very soft soil layer was found at 10 m depth. Meanwhile, the deeper soil layers were medium soil. Such soil conditions had high possibility to have lateral displacement due to excavation work.

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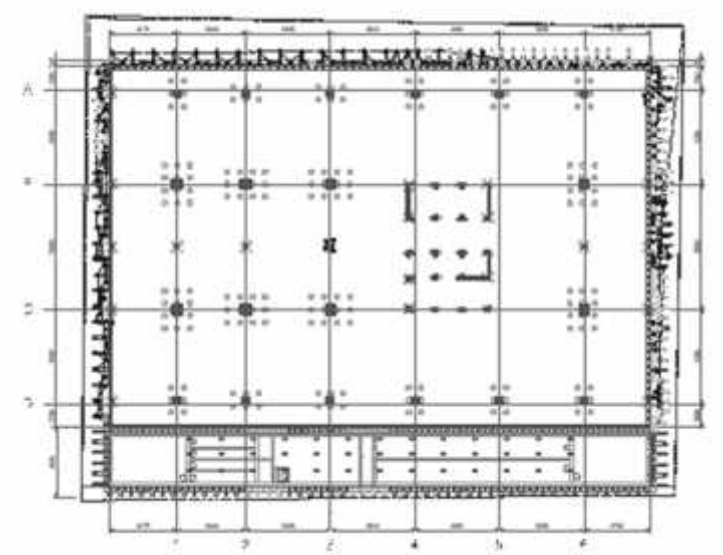

Fig. 1 The actual lay out of installed piles

Fig. 2 SPT results

SOIL PARAMETER AND GEOMETRY

3.2 m excavation of the whole area would induce very large lateral displacement that causes serious problems to the neighborhood existing buildings. The excavation stages then turned from the whole area to local excavation. At the first stage, 1 m depth excavation was conducted to the whole area. It was followed by 0.5 m excavation in the middle with average distance of 7 m from the soldier pile wall. The rest of excavation to 3.2 m depth was carried locally for each pile cap sequentially. The next pile cap excavation had been conducted after the previous pile cap casting finished. To be noticed, the sheet pile wall was installed prior the excavation process. The detail layout of the excavation is demonstrated at Fig. 3. Since the analysis was in 2D condition, 5 cross sections were chosen in which 3 of them were in vertical direction and the rest were in horizontal direction. This paper presents only a cross section of each direction (P1 and L2).

Table 1 Soil parameters.

Parameter	Fill soil	Silty clay 1	Sand	Clay 1	Clay 2	Silty clay 2
γ_{unsat} (kN/m^3)	17	16.3	18.5	15.1	16.8	17.5
γ_{sat} (kN/m^3)	18	17	19	16	18	19
E' (kN/m^2)	10E3	15E3	15E3	2500	20E3	25E3
ν	0.3	0.2	0.3	0.2	0.2	0.2
c' (kN/m^2)	5	20	7	17	47	30
φ (^{o})	30	15	33	1	9	10

The simulation of the excavation was utilized commercial finite element software named PLAXIS 2D 2011. The model geometry developed for the simulation can be seen in Fig. 4. It had 6 soil layers. Each soil layer parameters can be checked in Table 1. The model utilized in this simulation was Mohr-Coulomb. The model was chosen due to the soil data availability. Meanwhile, for dewatering and consolidation analysis, the soil permeability was chosen based on the grain size distribution classification available in PLAXIS.

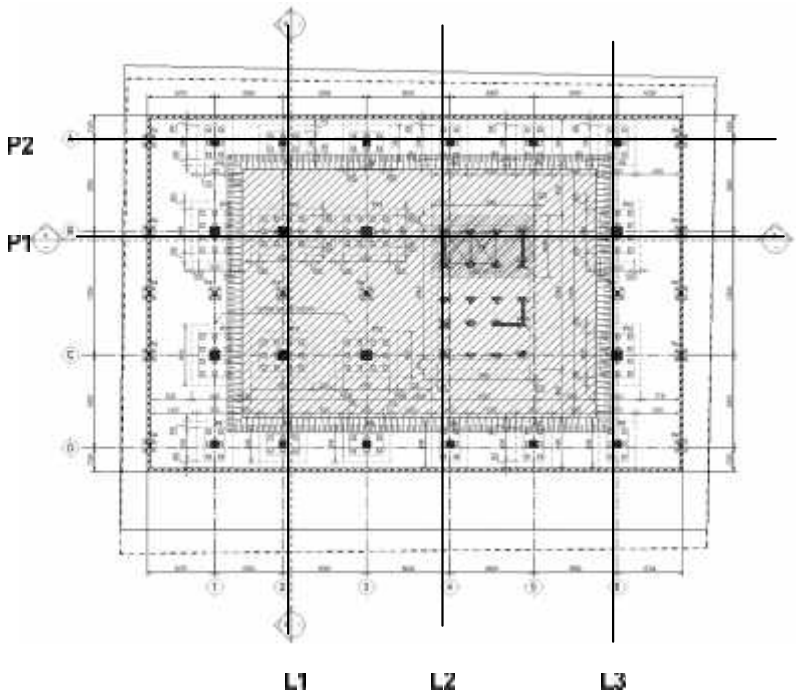

Fig. 3 Excavation layout and the cross section lines

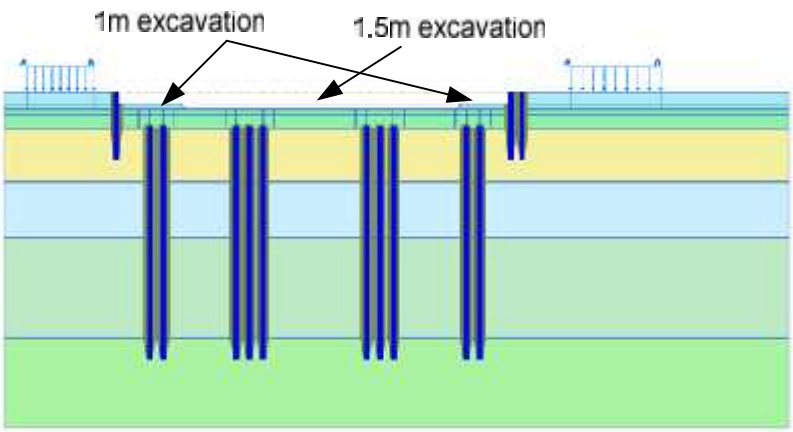

Fig. 4 Illustration of the excavation stages

SIMULATION RESULTS

Cross section P1

The cross section P1 passed 4 pile caps and the lift (Fig. 5). The lift was excavated to 4.65 m depth. Braced supports were installed at soldier piles, the edge cap and lift excavation. The critical exaction was the lift one. 2 braced supports were required to stabilize the excavation. The maximum displacement induced was 6.7 mm occurred on the top of the lift excavation (Fig. 7). The safety factor during excavation was 2.7 (Fig. 6). Interesting result is shown in Fig. 8. It shows that displacement occurred both in the excavation wall and floor. The sheet pile wall should be long enough to reduce the displacement below the bottom of the excavation. The main result associated to the building near the excavation was the deformation of the building. It was about 3.5 mm for the 3 stories building.

Fig. 5 The geometry model of cross section P1

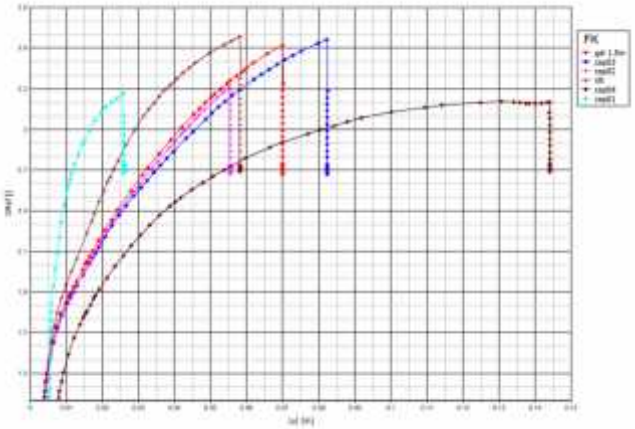

Fig. 6 Safety factor for each excavation stage

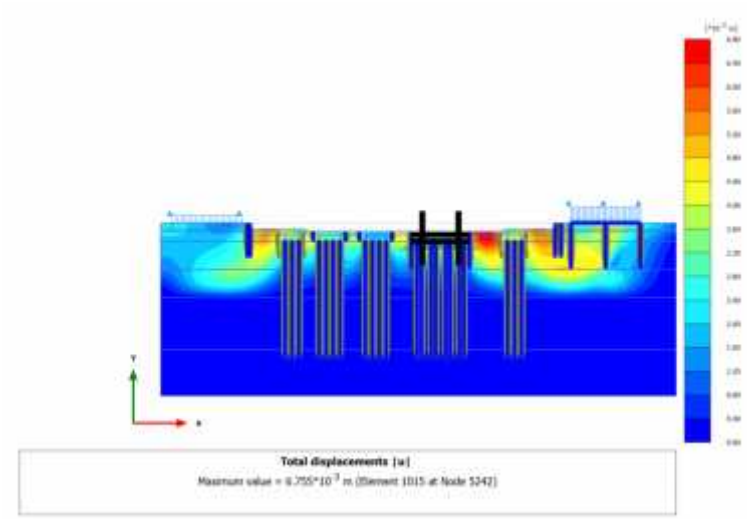

Fig. 7 Displacement distribution for lift excavation

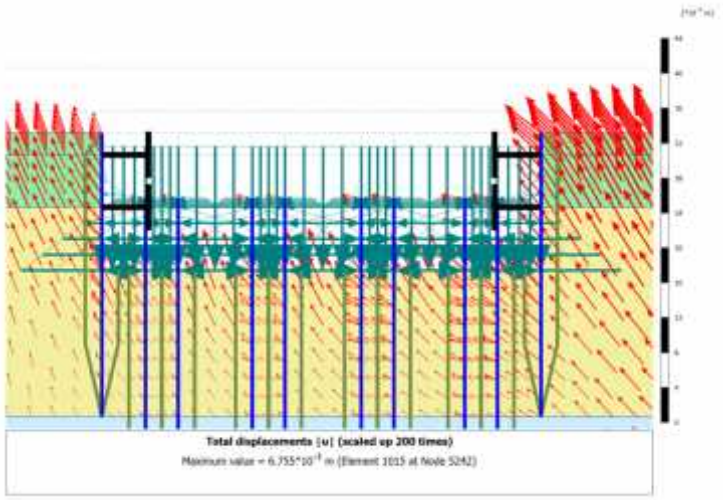

Fig. 8 displacement direction around the lift excation

Cross Section P2

Cross section P2 was important since it was a 2 stories surgery room of the hospital next to the project. Small deformation only should be tolerated during the excavation. The safety factor reached during excavation was 2.4 (Fig. 10). Meanwhile the maximum deformation occurred at the hospital was 2.4 mm (Fig. 9). The results were tolerable to keep the hospital building stable.

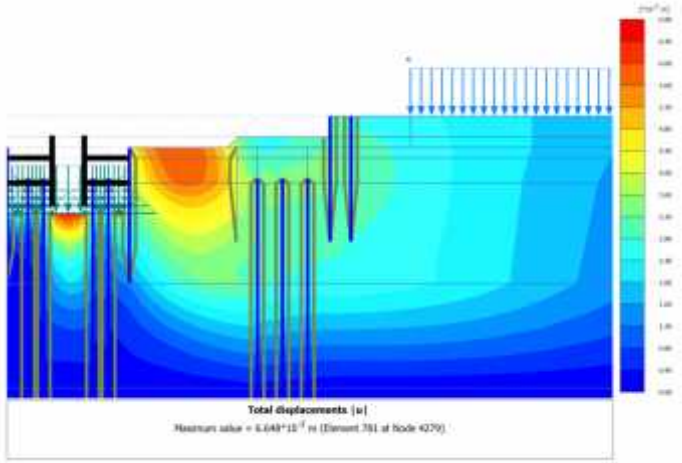

Fig. 9 Geometry model of cross section L2

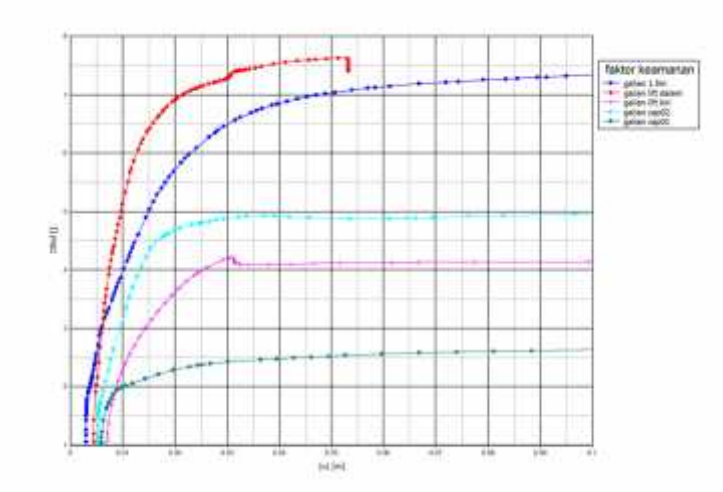

Fig. 10 Safety factors for each excavation stage of P2

CONCLUSION

An evaluation of excavation next to the existing building was conducted. The numerical simulation showed that local excavation stage yield small deformation to the neighborhood building. The deformation occurred at adjacent building was less than 5 mm.

ACKNOWLEDGEMENTS

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AN OVERVIEW OF GEOHEMICAL TRAPPING SYSTEM FOR CO_2 SEQUESTRATION STUDY IN THE ULTRAMAFIC ROCK FROM SULAWESI, INDONESIA

Adi Maulana[1]

ABSTRACT: One of the biggest environmental problems in the worlds nowadays is the anthropogenically induced climate change which has contributed intimately to the increasing of CO2 level in the atmosphere. This situation has influenced the global climate, leading to warmer temperature, increased sea melts and rise in sea level. One of the most environmentally-friendly methods to tackle the CO2 increasing is the CO2 storage in geological formation method or CO2 sequestration storage (CSS). Among CO2 storage sequestration model that have been developed, the geochemical trapping system in ultramafic rocks offer a promising result. However, despite its environmentally-friendly and economically-cost, the study on this is still lack. The ultramafic rocks in Sulawesi occur as a dismembered ophiolite body covering largely from central to the south eastern portion of the island. The composition of the rocks is ranging from peridotite to clinopyroxenite which contain Mg and Fe rich mineraland Ca- and Mg-rich minerals, respectively. The basic principal in this method is the acceleration of weathering/alteration process occurring in nature by producing reaction between CO2 with ultramafic rocks in which Mg-, Fe-, and Ca-bearing minerals are abundant. Theses minerals will act as a sink to sequester CO2 in the form of magnesium and calcium carbonates. In this study, geochemical character and approximate amount of CO2 than can be sequestered in ultramafic rocks from Sulawesi will be overviewed and estimated using standard equation. Approximately 780 mt and 34.9 mt of CO2 can be sequester in the ultramafic rocks from ESO and South Sulawesi Ultramafic rocks, respectively.

INTRODUCTION

Carbon dioxide (CO2) is the most common greenhouse gas found in the atmosphere. The emission of CO2 can be produced largely by the excessive burning of fossil fuels (coal, oil and natural gas) from power plant, industrial boilers and heaters and other kind of fuels such as wood, solid waste, ethanol, and biodiesel. This situation has influenced the global climate, leading to warmer temperature, increased sea melts and rise in sea level.There are at least three primary methods for reducing the amount of carbon dioxide in the atmosphere: employing energy efficiency and conservation practices; using carbon-free and reduced-carbon energy sources; and capturing and storing carbon to the underground or what so called carbon capture storage. Carbon capture and storage is a technology that can capture up to 90% of the CO2 emissions produced from the use of fossil fuels in electricity generation and industrial processes, preventing the CO2 from entering the atmosphere. This method is also known as carbon capture sequestration or CCS method. Among CO2 storage

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sequestration model that have been developed, the geochemical trapping system in ultramafic rocks offer a promising result (Voormeij&Simandl, 2004; Huijgen et al. 2006). Over the past several years some papers have been published which discuss the advantages of CO2 injection into ultramafic rocks formations (Hansen &Dipple, 2005; Oelkers et al., 2008). A survey of the global carbon reservoirs suggests that the most stable, long-term storage mechanism for atmospheric CO2 is the formation of carbonate mineral such as calcite, dolomite and magnesite (Oelkers et al., 2008) which has been also known as carbonation method. However, despite its environmentally-friendly and economically-cost, the study on geochemical trapping system in ultramafic rocks is still lack.

The ultramafic rocks in Sulawesi occur as a dismembered ophiolite body covering largely from central to the south eastern portion of the island (Kadarusman et al., 2000; Maulana et al., 2013). The composition of the rocks is ranging from peridotite to clinopyroxenite which contain Mg and Fe rich mineral and Ca- and Mg-rich minerals, respectively. The basic principal in this method is the acceleration of weathering/alteration process occurring in nature by producing reaction between CO2 with ultramafic rocks in which Mg-, Fe-, and Ca-bearing minerals are abundant. Theses minerals will act as a sink to sequester CO2 in the form of magnesium and calcium carbonates. In this study, geochemical character and approximate amount of CO2 than can be sequestered in ultramafic rocks from Sulawesi will be overviewed and estimated using standard equation. This result will shed the light on using geochemical trapping system for CO2 storage potential in the ultramafic rock from Sulawesi, Indonesia. As the ultramafic rocks are widely distributedin Sulawesi, a potential future utilization is highly demanded.

Carbon Dioxide Storage In Geological Formation

According to Mani et al (2008), there are three different ways for CO2 sequestration in geologic formation i.e. hydrodynamic trapping, solubility and mineral trapping.

1. Hydrodynamic trapping

It involves the storage of free CO2 as gas or supercritical CO2 in the pore spaces of sedimentary layers overlying oil and gas reservoirs, unmineable coal seams and deep saline reservoirs.

2. Solubility

   In this type of trapping, CO2 is dissolved in fluid phase comprising of aqueos brines and oils. The solution has density greater than brine in order to prevent buoyant escape of CO2 (Weir et al., 1995).

3. Mineral carbonation trapping

   It is a permanent mechanism of sequestration in which silicate minerals are converted to secondary carbonates due to reaction with CO2 (Mani et al., 2008). Detailed background of this method is given below.

Mineral CO2 sequestration: brief review

The primary requisite for using mineral carbonation method for CO2 sequestration is availability of suitable ultramafic rocks containing Mg, Ca dan Fe silicate minerals e.g olivine. Olivine is a common constituent of dunites and peridotite. These rocks are typically abundant in mantle, exposed on Earth`s surface as ophiolites. In crustal setting, they form as a part of layered complexes formed in deep-seated basaltic magma chamber as cumulates and are exposed to surface through tectonic exhumation process.

The basic principle of mineral CO2 sequestration in ultramafic rock is the acceleration of weathering/alteration processes occurring in nature, wherein CO2 reacts with Ca, Fe, or Mg containing minerals, especially silicates. The

permanent sequestration of CO2 in the form of carbonates is shown by this reaction.

$(Ca, Mg)O + CO_2 \text{-------}\rightarrow (Ca,Mg)CO_3$

$(Ca, Mg)SiO_4 + CO_2 \text{-------}\rightarrow (Ca,Mg)CO_3 + SiO_2$

Nature stores CO2 predominantly in carbonates, mainly limestone, dolomite. Litswanite (carbonated serpentinite) represents a fossil mineral carbonation system, serving as a repository of CO2 in the form of carbonates during the reaction of serpentine with CO2-rich fluids (Klump et al., 2000; Kojima et al., 1997). CO2 is made to react with mafic/ultramafic rocks which are the most common source of magnesium, iron or calcium-bearing silicate minerals present in nature (Goff and Lackner, 1998). Example of such a carbonation reaction with suitable magnesium minerals (Mani et al. 2008) are:

$Mg_3SiO_5(OH)_4 + 3CO_2 \text{-------}\rightarrow 3MgCO_3 + SiO_2 + H_2O$

SerpentineMagnesite Silica

$MgSiO_4 + 2CO_2 \text{---------}\rightarrow 2MgCO_3 + SiO_2$

(Olivine) Magnesite Silica

Potential Of Usage Of Ultramafic Rocks For Co2 For Carbon Sequestration In Sulawesi

South Sulawesi is famous for its tectonic evolution as a continent-island arc-continent collision zone involving opening of Makassar Strait and collision between the Sundaland and Australian-derived microcontinent. Ultramafic rocks in Sulawesi occur as dismembered ophiolite.In the following we briefly review the occurrence of ultramafic rocks in South Sulawesi for their possible usage in CO2 sequestration.

East Sulawesi Ophiolite

Ultramafic rocks in Sulawesi occur as dismembered ophiolite body in some areas e.g. south, southeast, east and central part (Kadarusman et al., 2004; Maulana et al., 2015). In the east, southeast and central part of Sulawesi forming some segmented ophiolite body which later known as East Sulawesi Ophiolite (Kadarusman et al., 2004). The ESO It grades upward from residual mantle peridoite, and mafic-ultramafic cumulate, through layered to isotropic gabbro, to sheeted dolerite and basaltic volcanics. The ultramafic rocks composed of peridotite which consists mainly of lherzolite and Cpx-rich harzburgite, with minor dunite, pryoxenitedanCpx-free harzburgite.

South Sulawesi Ultramafic Rocks

In South Sulawesi, the ultramafic rocks occur in two separated basement blocks, namely Bantimala and Barru Block (Maulana et al., 2015). The rocks of both blocks show strong serpentinization of original anhydrous silicates. The Bantimala ultramafic consists mainly of peridotite (harzburgite and dunite) and clinopyroxenite, with lenses of podiformchromitite. They have undergone metamorphism as shown by the occurrences of amphibolite-faciestremolitic schist. In contrast, the Barru ultramafic rocks consists of harzburgite peridotite and podiformchromitite and have been

metamorphosed to amphibolite-facies condition.

Quantification of Sulawesi ultramafic rocks for CO2 sequestration

Carbon storage capacity of various ultramafic rocks from Sulawesi will be tentatively quantified by using their MgO content (Goff and Lackner, 1998; Kohmann and Zevenhoven, 2001; Tahirkheli. et al. 2012). General information about the geology and structure of ultramafic rocks and data on their distribution, approximate thickness, chemical composition and mineralogy are used to calculate the volume of ultramafic rocks in Sulawesi. The volume thus calculated and then multiplied by the wt% of MgO to assess the quantity of ultramafic rocks needed to sequester CO_2in these area using the formula given by Zevenhoven and Kohlman (2001).

$$T = 1 \times \rho \times a \times t \times d \times (1-\varphi).$$

Where T is the amount of CO2 that can be sequestered, ρ is the wt% of MgO in ultramafic, a is the area, t is the thickness, d is the average density andφ is the average porosity of ultramafics.

East Sulawesi Ophiolite

For East Sulawesi Ophiolite, an effective sequestration of about 20% at a depth of 1 km is calculated as:

Volume of the complex = 15.000 km^2 (Kadarusman et al., 2004) x 1 km = 15.000 km^3

Effective volume of the complex for sequestration = 20% of 15.000 km^3 = 3000 km^3 = 3000 x 10^6 m^3

Average density = 3197 km/m^3

Mass of ultramafic = volume x density = 3000 x 10^6 m^3 x 3197 km/m^3 = 9591 x $10^9 m^3$.

Average % MgO in ultramafic rocks of ESO = 8.3 wt%

Total MgO in ultramafic rocks of ESO = 8.3 wt% x 9591 x 10^9 m^3 = 796 million tons (mt)

Since 1 ton of MgO can dispose of approximately 1 ton of CO2 (Zevenhoven&Kohlman, 2001), with an average porosity of 2% in ultramafic rocks, 796 mt of MgO in ESO can sequester 1 x 796 x (1-0.02) = 780.08 mt of CO” in the form of magnesium carbonate.

South Sulawesi Ultramafic Complex

For South Sulawesi Ultramafic Complex, an effective sequestration of about 20% at a depth of 1 km is calculated as:

Volume of the complex = 420 km^2 (Maulana et al., 2015) x 1 km = 420 km^3

Effective volume of the complex for sequestration = 20% of 420 km^3 = 84 km^2 = 84 x $10^6 m^3$

Average density = 3197 km/m^3

Mass of ultramafic = volume x density = 84 x 10^6 m^3 x 3197 km/m^3 = 26 x $10^9 m^3$.

Average % MgO in ultramafic rocks of ESO = 13.7 wt%

Total MgOin ultramafic rocks of ESO = 13.7 wt% x 26 x 10^9 m^3 = 35 million tons (mt)

Since 1 ton of MgO can dispose of approximately 1 ton of CO2 (Zevenhoven&Kohlman, 2001), with an average porosity of 2% in ultramafic rocks, 35mt of MgO in ESO can sequester 1 x 35 x (1-0.02) = 34.9mt of CO” in the form of magnesium carbonate.

CONCLUSION

This study outline the tentative amounts of CO2 that can be stored as mineral carbonates in ultramafic rocks from Sulawesi Island. We assumed 20% of the total ultramafic volume can be utilized as sequestration. Approximately 780 mt and 34.9 mt of CO2 can be sequester in the ultramafic rocks from ESO and South Sulawesi Ultramafic rocks, respectively.

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SOIL STABILIZATION UNDER ELECTRIC FIELD APPLICATION (A BENCH SCALE EXPERIMENTAL STUDY OF IONIC INJECTION)

R.M. Rustamaji [1] and E. Priadi [2]

ABSTRACT: This paper contains the results of laboratory experiments on a bench scale of an ionic injection system that applies under the influence of an electric field generated by an electric current of 0.01A that had passed through two electrodes that placed at both ends of the soil samples that were taken from Pontianak's soft soils. Ions such as Al^{3+}, Ca^{2+}, Mg^{2+}, CO_3^{2-} and HPO_4^{2-} were injected simultaneously into the soil through the mechanism of electromigration for 72 hours. After the injection of ionic completed, the physical and mechanical properties of the soil were measured. A sample of test no. 1 was measured shortly after the completion of the injection, while the other one, i.e. test no. 2 was measured after 72 hours from the time the completion ionic injection process. The results obtained show that, undrained shear strength of sampel from test no. 1 increased by 60% from the initial value, and similar result obtained from test no. 2 in which undrained shear strength increased up to 100%. Nevertheless, undrained shear strength was not increase uniformly along the sample. Based on the measurement, the undrained shear strength in the cathodic zone was highest increased rather than the other zones. Some other test indicators such as moisture content and ionic concentrations were also measured and indicate the changes in the value of all the soil samples. Based on the facts of the test results, it seems there had been a chemical reaction processes. It can be explained since the cross-ionic transport through the soil samples can occur by electromigration, eventually, precipitation reaction will take place as the consequence of the propagation of cation and anion in the soil samples.
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Keywords: *ionic injection, soft soils, electromigration.*

INTRODUCTION

Chemical stabilizations are usually used to improve the ground of soft to very soft soils. For soil stabilization, chemical compounds of one or more chemical species are introduced into the soil to improve its properties, both the mechanical and physical properties of soils. Chemical stabilization of clay soil is basically carried out by using mix in-place processes. In such cases, lime and cement stabilization of clay have been widely used for both surface and deep soil treatment. Sometimes, in the case of granular soil, cement stabilization called grouting is a possible technique in which cement mixtures injection can be used. However, for certain conditions, such as foundations on soft soil, this mixing technique is not appropriate. In the case of the ground water table nearing the ground surface, cement mixing is also not effective. Most soft soils mainly consist of clay and fine-grained silt. These soils are commonly indicated by their low permeability coefficient k (m/s). In such conditions, the grouting technique is not used properly since the penetration of grout materials (i.e. stabilizing agents) are difficult in fine-grained soil by hydraulic injection. To overcome the problems mentioned above, the current research regarding ionic injection shows that, chemical treatment under an applied electric field can induce the cross transport of ions throughout the soil medium. Chemical reactions (i.e. precipitations) occur in soil pores, since cations and anions react to each other due to the cross transport. The cross transport of ions due to electromigration is the main mechanism in ionic injection.

The main principles of the ionic injection are a combination of four phenomena of electrokinetics in soil consisting of electroosmosis and electromigration. Ionoc injection is performed by injecting chemical substances (i.e. stabilizing agents) into the soil which is under the influence of an electric field. The cross transport of the stabilizing agents through the soil will cause chemical reactions which will result in physico-chemical and cementation effects that will strengthen the soil.

MATERIALS AND METHOD

Experimental Setup and Materials

A series of ionic injection experiments was carried out using a bench scale apparatus especially developed for the experiment that consists of PVC tube and anolyt-catholyt compartements. Dimensions of tube were 2.5” diameter and 40 cm length. The soil specimen was placed into the tube of which at the end were attached two electrodes (i.e. anode and cathode) which were made of iron. Passive electrodes of tungsten wires were inserted into the soil along the distance in between anode and cathode. Electrical potential could be measured during treatment test through these

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passive electrodes. A constant DC electric current was generated by DC power supply for the duration of the treatment test.

Two filters of porous stone separated the soil from the anolyte and catholyte. The thickness of each filter was 10 mm with pore sizes of 200 μm. The filters functioned as separators to prevent direct contact between the electrolyte (i.e. chemical species solution or stabilizing agent) and the soil specimen. The filter prevented the soil specimen from rubbing or scouring effects at the ends while allowing the transport of the chemical species. The electrodes had direct contact with the soil specimen. A filter paper was inserted at the interface between the filter and the electrode. Each of the electrode compartments was filled with electrolyte fluid which contained stabilizing agents (i.e. chemical species or ions). The fluid levels of each of the electrode compartments were maintained at a constant level for the duration of the treatment process in such a way that the transport of several species due to hydraulic head was prevented.

A series of tests were conducted to understand the effects of Ionic Injection using multi chemical species. The four main cations used in these experiments were Al^{+++}, Ca^{++}, Mg^{++}, and K^{+} ions. The cations were dissolved in the anolyte while the main anions of CO_3^{2-} and HPO_4^{2-} were dissolved in the catholyte. Both cations and anions were introduced into the soil specimen under the influence of an electric field. The effects of treatment using Ionic Injection were analyzed by means of several different kinds of tests, measurements and observations. The undisturbed soil sample of Pontianak soft soil were tested in this experimental research work.

Procedure and Testing

To understand the physical, mechanical, chemical and mineralogical characteristics of the soil specimens, a series of tests and measurements were performed according to the ASTM standard. Undisturbed soil sample was used and it was taken directly on site using PVC tube. The electrode compartments were filled with solutions of anolyte and catholyte which were kept at a constant level during the test. Experiment tests were performed by applying a continuous DC-current for the duration of the test. The voltage and pH of the anolyte and catholyte were measured during period of 72 hours. The un-disturbed un-drained shear strength values were measured in-situ after completion of the Ionic Injection, and it was taken at the passive electrode locations to carry out the un-drained shear strength using an UCS test.

RESULT AND DISCUSSION

Properties of Untreated soil

Untreated soil properties is given in Table 1, and from the pasticity chart according to Unified Soil Classification, untreated soil is classified into MH or OH.

Table 1. Physical properties of soil

No	Parameters/Properties	
1.	Average Natural water content, w (%)	115
2.	Average Specific Gravity, G_s	2.4
3.	Atterberg Limits (average):	
	w_L (%)	83.0
	w_P (%)	35.5
	PI (%)	47.5
4.	Classification:	
	Plasticity chart	CH-OH
	AASHTO	A-2-7 & A-7-5
5.	Particle size fractions:	
	Clay particles (%)	38
	Silts (%)	57
	Sands (%)	5
6.	Activity, A (Skempton)	1.25
7.	Loss on Ignition, LoI (%)	2.64
8.	Soil pH	4.81 - 4.93
9.	Common ions (%):	
	Si^{++}	35.6
	Al^{+++}	25.4
	Ca^{++}	0.0003
	K^{+}	0.345
	Na^{+}	0.033
	Fe^{++} (total)	2.7
10.	Qu (kPa)	14.8

Electrokinetic Parameters

Ionic Injection essentially uses the principles of the electrokinetic phenomena in soil to strengthen the soil. The initial conditions and parameters of the electrokinetic are summarized in Table 2.

Table 2. Electrokinetic parameters

Sample	t (hour)	I (A)	j (A/m^2)	V (Volt)		ie (V/m)		σ (S/m)		W (kWh/m^3)
				V_0	V_t	ie_0	ie_t	σ_0	σ_t	
Test 1	72	0.01	3.15	14.0	11.7	35.00	29.25	0.090	0.108	478.45
Test 2	72	0.01	3.15	21.7	16.3	54.25	40.75	0.058	0.078	666.59

There were a changes both in electrical potential gradient and electrical conductivity. the measurement of electrical potential gradient at final test was lower than the initial condition as shown in Table 2. The change of the electrical potential gradient is related to the change of soil conductivity, and it was due to ionic strength which is a result of ionization processes. Due to electric current transmission, the electric current flow occurs primarily on ions within pore water. Thus, electrical conductivity is affected mainly by water content, porosity and tortousity (Azzam and Oey, 2001).

Electrical conductivity depends on the pH and ionic strength conditions of the soil. In such cases, the electrical conductivity of the soil is a function of ionic strength which is a consequence of electro-chemical soil processing that result in ionization. The processes of ionizations have influence on the ionic intensity in such a way that the concentration of ions is to be increased within soil-pore water. Soil pore water contains many different species such as Ca^{2+}, PO_4^{3+}, Mg^{2+}, Al^{3+}, Na^{+}, Fe^{2+}, CO_3^{2-}, etc. Therefore, a change in the electrical potential gradient is due to a change in the conductivity which is not proportional. A decreasing electrical potential gradient means an increase in electrical conductivity and vice versa (Alshawabkeh and Sheahan, 2002).

Energy Consumption

Electric power consumption over the treatment time per cubic meter soil, W (kWh/m^3) is calculated using the equations below:

$$W = \frac{U}{A \cdot L} \int_0^t I \cdot dt \tag{1}$$

Where, U = electrical potential (V); I = electric current (A); A = cross section of specimen (m^2); L = length of specimen (m); t = time or treatment period (h).

Since the specimen's length and cross section area were held constant, the consumption of electric power is a function of the electric current, electrical potential, and period of treatment time. Therefore, an increase of energy consumption is due to an increase of electric current, or electrical potential, or duration of treatment process (i.e. period of treatment), and/or a combination among them. A typical energy expenditure gradient of time is shown in Fig. 1. Energy expenditure exponentially increased over time increasing sharply in the first 24 hours. This phenomenon was exhibited at all tests. In electrokinetic, energy expenditure was controlled by the electrical conductivity of the soil.

An appropriate selection of either electrical current density or electrical gradient (i.e. electric field) is dependent on the properties of the soil and its conductivity. As conductivity of the soil (bulk conductivity) increases, the energy required for the processes increases proportionally (Mohamedelhasan and Shang, 2002). When the electric field was constantly maintained during the ECI process, the soil eventually became more conductive over time (treatment time), thus a higher current density was needed for the electro-grouting process (Alshawabkeh and Shean, 2002).

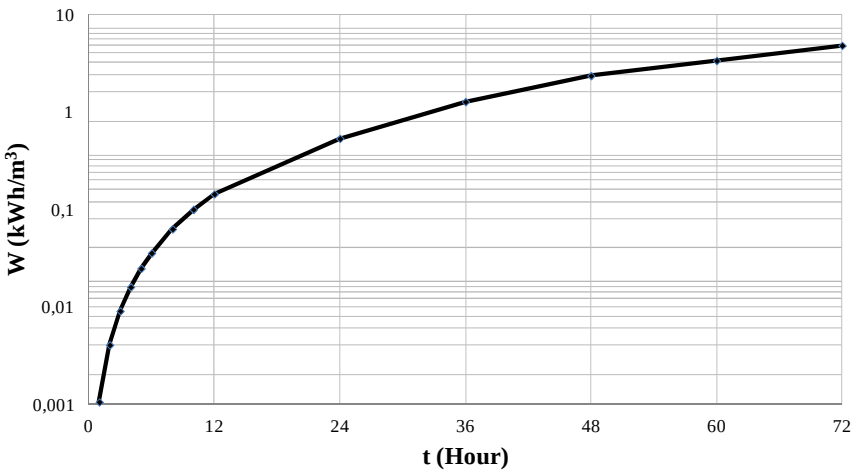

Fig. 1. Energy expenditure vs. time

Water Content of Treated Soils

Moisture content of the soil specimen was measured at the completion of test. Samples were taken directly from the treated soil specimen. Distributions of final water content across the specimen measured in these experiments are summarized in Table 3. Water content generally decreased at the anodic and cathodic zone, but it was not significant. Even though water content decreased at the anodic, but the un-drained shear strength did not significantly change. However, at the cathodic region, the un-drained shear strength measured significantly higher, and the final water content was lower than other regions. That means any chemical precipitation reactions occurred as a result of electro-chemical soil processing, since Ionic Injection treatment was controlled under a multi species cross transport without significant electroosmotic flow. Electromigration is the main mechanism that controlled the cross transport of chemical species. In this case, the species were introduced into the soil from both electrode compartments.

Table 3. Electrokinetic parameters

No.	$W_{initial}$ (%)	W_{final} (%)		
		Anodic	Centre	Cathodic
Test 1	114,18	110,66	106,22	98,88
Test 2	114,18	94,73	112,98	98,11

Undrained Shear Strength and Ion Distribution

Undrained strength across the soil specimen was non-homogenously distributed. Greater increases of shear strength were found at the cathode region than at the other regions. In ionic injection method, a formation of cementing agents primarily depends on soil pH values and the availability of free calcium ions in the system. Precipitation reactions lead to a neutral pH, influencing the formation of cementing agents (Asavadorndeja and Glawe, 2005). Therefore, the gradient of a soil specimen's pH is important due to a formation of cementing agents. The intensity of cementing agents can lead to chemical precipitation reactions that enable the strengthening of soil.

Regions of low resistance corresponded to regions of high ionic strength. A broad resistivity zone forms over the cathode half where hydroxyl ions formed in the cathode reservoir precipitate clay with dissolution products and other ions. When high resistance corresponds to precipitation zones, it also means that the high resistance corresponds to electrical conductivity. Thus electrical conductivity at precipitation zones (or at the cathodic zone) is usually lower than at other parts of the soil (or anodic zone).

Table 4. Undrained shear strength

No.	t (Hour)	I (A)	cu (kPa) initial	cu final (kPa)		
				Anodic	Centre	Cathodic
Test 1	72	0,01	14,8	18,22	22,22	29,76
Test 2	72	0,01	14,8	16,00	19,80	31,90

Table 5. pH of untreated and treated soils

No.	pH initial	pH final		
		Anodic	Center Zone	Cthodic
Test 1	7	5	6	7
Test 3	7	5	6	7

Table 6. Distribution of common ions

Common Ions	Untreated Soil (%)	Ion Concentration (%)		
		Anode	Centre	Cathode
Si^{2+}	35,6	59,9	43,5	44,6
Al^{3+}	25,4	4,93	12,9	22,5
Ca^{2+}	0,003	0,048	0,02	0,0059
K^{+}	0,345	0,151	0,153	0,18
Na^{+}	0,033	0,025	0,023	0,027
Fe^{2+}	2,7	3,75	3,87	3

Treated soils were chemically analyzed to assess the concentration and distribution of anions and cations. Samples were tested and analyzed by AAS and Gravimetry. Distribution of common ions is shown in Table 6. These ion that play an important role in the cementation reactions in the soil.

CONCLUSSION

Based on the results, a greater understanding of many of the aspects and mechanisms involved in ionic injection processing of fine-grained soil was achieved. These included transport mechanisms of ions (or species), electro-chemical processes, and the mechanics of soil.

Cementation resulted in increasing the soil strength through precipitation reactions. It was observed that by introducing one or more chemical species or ions into the soil under an electrical field, those species would dissolve within pore water and be transported across the soil so that the cations and anions chemically reacted (or precipitated) with each other thus generating cementation, and later on increases the soil strength.

Precipitation reactions in soil specimen occurred mainly at regions close to cathode. Un-drained shear strength at these regions increased more significantly than at other regions. Electrical conductivity (bulk) of the soil close to cathode was lower, while the electric field (or electrical potential gradient) became higher in this region.

Electric power consumption was a critical parameter in the process. The gradient of electric power consumption increased exponentially over time sharply increasing in the first 24 hours. The transport mechanism as well as the electro-chemical processing (e.g. electrolysis) that was induced by the electric field , was a large consumer of electric power.

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THE SUBSTITUTE EFFECT OF RESIDIUM CATALYTYC CRACKING (RCC) AND LIME ON EXPANSIVE SOIL SHEAR STRENGTH

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ABSTRACT : The expansive soil characteristic has potential of shrink – swell quite high in case of water content changing. Therefore, one way to overcome the unstable behavior of the expansive soil is to stabilize the soil. The chemsistry stabilization method are used in this study. RCC which is a waste of petroleum cracking process producted by PT. Pertamina Plaju Palembang and lime are used in this study as the admixture of the soil. The substitute effect of RCC and lime on shear strength of expansive soil are investigated using triaxial testing UU. The expansive soil samples taken from Tanjung Api-api km.27 Palembang. The stabilizing agent will be subsituted in expansive soil with each composition of RCC and lime variation were 0%, 2%, 4%, and 6% of the soil weight with a treatment period of 0 days, 7 days and 14 days. The results of the native soil testing showed that the native soil is expansive clay soil. The triaxial test of soil mixture with RCC and lime variation show the increasing of cohesion value, the value of friction angle, soil shear strength when compared with native soil and conditions without mixture. The maximum shear strength variation occurs in 4% RCC and 6% lime with 0 day treatment period, is 2.426 kg/cm^2.

INTRODUCTION

Background

In a project, the issue of soil is something that absolutely need to be considered by civil engineers. Before doing the project, the first thing to observe is about the characteristics of the native land, which characteristic that would be found at the site to support the necessity of a safe and good construction. However, not all parts of the soil have the strong bearing capacity, and one of the soils is expansive clay.

Expansive clay has the high value of swelling and shrinkage. The soil with the high value of swelling and shrinkage will be influenced by water to its physical characteristic and soil mechanical behavior (Braja M Das et al. 1995). One common way to fix this type of soil is to do the soil stabilization. The stabilization method that will be used in this research is the chemical stabilization. By using *Residium Catalytic Cracking* which is former waste of the oil refinery from PT. Pertamina at Plaju and the lime generally known as a non-brittle material, with good workability and strong bond capacity for rocks and bricks.

The uses of both stabilization agent materials are expected to fix the physical characteristic of expansive clay soil itself by doing the UU (Unconsolidated Undrained) Triaxial Test.

Problems

The problem formulation in this research is to know how big the influence of the substitute material *Residium Catalytic Cracking* and lime to the shear strength value of expansive clay soil with the UU (Unconsolidated Undrained) Triaxial Test.

Purpose

1. To know the shear strength value of expansive clay after the addition of stabilization material, *Residium Catalytic Cracking* and lime.
2. To determine the composition material of *Residium Catalytic Cracking* with 0%, 2%, 4%, 6% variations and lime with 0%, 2%, 4%, 6% variations as the stabilization materials toward the attempt to increase mechanical characteristic of expansive clay soil. Thus, the optimum measure percentage of *Residium Catalytic Cracking* and lime during the maintenance period in 0 day, 7 days, and 14 days could be obtainable

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Research Scope

This research will find the answer from the problem encountered in expansive soil stabilization. The problems are restricted at the increase of shear strength value with the addition of *Residium Catalytic Cracking* powder and lime in expansive clay soil with 0%, 2%, 4%, 6% variations for *Residium Catalytic Cracking* and with 0%, 2%, 4%, and 6% variations for the lime during the maintenance period in 0 day, 7 days, and 14 days.

LITERATURE REVIEW

Expansive Clay Soil

Expansive Soil in a simple definition is the soil with the capacity to swelling and shrinkage as the effects of water condition changes. The swelling potential of soil could be obtained based on its Plasticity Index (IP). For clay soil categorized in expansive clay soil is the soil which has the high swelling capacity and the Plasticity Index (IP) value more than 35%. The soil physical characteristics influences the swelling of expansive soil are water measurement, dry density, and properties index.

Soil Stabilization

Stabilization is an attempt to fix the characteristics of native land which is a deficient material to be the soil with better characteristics in order to meet the technical requirements used in construction design.

The fixing of native land characteristics is basically to improve the soil bearing capacity. There would be a time where the native land or the embankment soil in site could not fulfill the technical requirements as construction material, for example is the expansive soil which is easily influenced by water. Then that soil could not be used directly and need to be stabilize or fix first.

***Residium Catalytic Cracking* (RCC)**

Residium Catalytic cracking (RCC) is former waste of the oil refinery from PT. Pertamina. This material has fine grain shape with greyish white color, very light (density value 2,35 – 2,38) with silica elements and alumina and categorized as pozolan synthetic material.

Because its characteristics, many research had done about the utilization of RCC as the addition material or the substitute material in order to improve the quality of a material such as concrete, asphalt, soil and the other materials. Utilization of RCC as the stabilization material for clay soil is based on RCC's characteristics to increase the shear stress and CBR value of native land and reduce the swelling potential of expansive soil.

Lime

Lime is known as the material which has the function as the bond material in wall and pile construction. The lime characteristics are non-brittle, hardened easily and quickly, good workability and has the bond capacity for rocks and bricks (Tjokrodimulyo, 1992). The basic material of lime is the lime rocks or dolomite containing calcium carbonate element ($CaCO_3$).

Shear Strength Parameter of Soil

Shear strength is the maximum resistance of the soil in order to sustain the stress from the displacement occurred at the soil.

$$S \quad s \quad (\tau) = \frac{s \quad f}{a \quad o \quad p} \qquad (1)$$

At the rough grained soil the main influence is shear and at the fine grained soil the main influence is cohesion. Thus, in combination soil of sand and clay will occurs for the both influences. Then in soil there are three kinds of terms:

1. Friction Soil= non cohesive soil = φ soil
2. Cohesive soil = C soil
3. Cohesive – friction soil = C-φ soil

The next step is to obtain the parameter to determine the shear strength (φ a C) which could be obtain from Triaxial test.

The soil shear strength parameter needed in the bearing capacity analysis of the soil, slope stability and thrust force at the soil retaining wall. Mohr (1910) gives theory that collapse of a material could occur as the result from the combination of the critical state which is come from the normal stress and the shear stress. Furthermore the function relation between the normal stress and the shear stress on its collapse plane will be explained based on formula:

$$\tau = f(\sigma) \qquad (2)$$

With τ is the shear stress when the collapse or the failure occurs, and σ is the normal stress in that condition.

According to Coulomb (1776) the shear strength is calculated by formula:

$$\tau = c + \sigma \, tg\Phi \qquad (3)$$

Notation:

τ = shear strength of soil (kg/cm²)
c = cohesion of soil (kg/cm²)
Φ = friction angle of soil (°)
σ = normal stress at the failure plane (kg/cm²)

The shear strength of soil also stated with the formation of effective shear σ_1 and σ_3 when the failure happens. The formula of shear strength stated by:

$$\tau = \tfrac{1}{2}(\sigma 1 - \sigma 3)\sin 2\theta \quad (4)$$

$$\sigma = \tfrac{1}{2}(\sigma 1 + \sigma 3)\,\tfrac{1}{2}(\sigma 1 - \sigma 3)\cos 2\theta \quad (5)$$

$$\theta = 45° + \Phi/2 \quad (6)$$

θ = theoretic angle between the horizontal plane and the failure plane.

If the position of shear stress illustrated in *p-q* coordinate system with:

$$p = \tfrac{1}{2}(\sigma 1 + \sigma 3) \text{ and } q = \tfrac{1}{2}(\sigma 1 - \sigma 3) \quad (7)$$

The stress position could be shown by a stress point as a substitute from Mohr circle (Picture 2.1.)

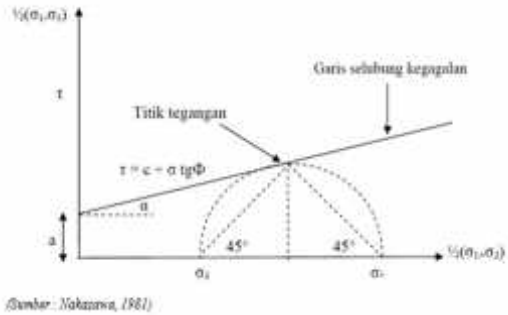

Figure 2.1. Stress condition represented $\tau = c+\sigma \text{ tg } \Phi$

In Figure 2.1. the vail line of failure is shown by the formula:

$$\tfrac{1}{2}(\sigma_1 - \sigma_3) = a' + \tfrac{1}{2}(\sigma_1 - \sigma_3)\text{ tg } \alpha' \quad (8)$$

With a' and α' are the modification parameters from the shear strength. For c' and Φ' parameters are obtain by the formula:

$$\Phi' = \arcsin(\text{tg } \alpha') \quad (9)$$

$$c' = \frac{a'}{\cos \alpha'} \quad (10)$$

Triaxial Test

These days, the triaxial shear stress is the most reliable test to determine the shear stress parameters which are *c* (cohesion) and Φ (the inside shear angle) in total stress or effective stress in order to approach the real condition at the site.

$\sigma 1-\sigma 3$ = Deviator stress ($\Delta\sigma$)
τ = Shear stress
σn = Normal stress
c = Cohesion
Φ = Inside shear angle
θ = The angle between horizontal plane and failure plane

With Mohr-Coulomb formula $\tau = c+\sigma \text{ tg } \Phi$, then cohesion (c) and inside shear angle (Φ) can be obtained with graphical way.

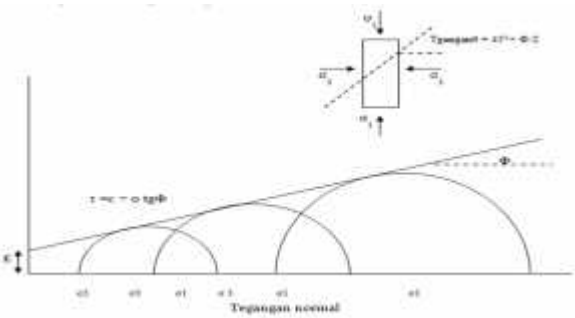

Figure 2.2. Mohr-circle (graphical way to determine c and Φ parameters.)

RESEARCH METHODOLOGY

General

This research is using the experimental method where all the research activities conducted at Soil Mechanical Laboratory, Civil Engineering Major at Sriwijaya University. The first step in this research involves the literature study, such as learn the same type of research that has been done before, theories supporting the soil stability, methods to mend the soil, examination procedure and data analysis technique.

Literature Study

Literature study or literature reviews were done by reviewing and learning books, scientific journals, past research, and the other literature study related with the subjects observed. This method was done in order to acquire the theories used as the reference or the guide in doing the experiments and discussions.

Site Works

Site works is expansive soil sampling. The sample of disturbed soil was done at Tanjung Api-Api region, South Sumatra. The site works include the expansive soil sampling and the acquirement of stabilization materials. Sample of expansive soil was done in two methods; there are the acquirement of disturbed soil sample and undisturbed soil sample.

Laboratory Experiments

After the expansive soil sampling, then the laboratory experiments performed at the Soil Mechanical Laboratory in Sriwijaya University, such as index properties test, soil compaction, and UU Triaxial test.

Soil Index Properties Test

1. Soil density is the ratio of the soil weight with the water weight to determine the density of grained soil material. This test is referring to ASTM D-854.
2. Atterberg limits. This test was conducted to know the parameter value of soil plastic limit, index plasticity, and the soil sample activities. With this test the value of index plasticity which is determined whether the observed soil is in expansive soil category or not could be obtainable. This test is using the standard of ASTM D 423-66 and ASTM D 424-74.
3. Filter analysis, to determine the division of grained size soil. This test is referring to ASTM D421 and ATM D422.

Standard Compaction Test of Soil

The purpose of this test is to determine the relation of Optimum Moisture Content (OMC = w opt.) and maximum dry density (γd max.). Result of the compaction would produce the optimum moisture content that will be used in sample making of triaxial test material. The compaction system used is the Standard Compaction Test.

Making The Material Test

In this research the material test is mixed soil made from substitute variations of *Residium Catalytic Cracking* and lime as the additive material. The maintenance period lasted for 0 day, 7 days, and 14 days.

Triaxial Test

This test includes the shear strength test for the cylinder form soil with 38,1 mm diameter and 76,2 mm height at the native soil from the site. The test performed by using conventional tools in the condition of unconsolidated soil and the pore water of unconsolidated undrained soil.

Result Analysis and Discussion

The data obtained from the result of laboratory test will be analyzed to know the influence of *Residium Catalytic Cracking* and lime towards the shear strength of expansive clay. The data analyzed such as soil properties and UU (Unconsolidated Undrained) Triaxial test of the native soil and the mixed soil.

Conclusion

After the data analysis, the discussion conducted afterward. Thus, the conclusion and suggestions could be made based on the result of data analysis and discussion.

RESULT AND DISCUSSION

Physical Characteristic of Soil

The summary of physical characteristic test and soil classification result shown in Table IV.1 below,

Table IV.1 Physical Characteristic and Soil Classification

Number	Test	Resulf of Soil Test
1	Real water measure	69,31%
2	Density (Gs)	2,64
3	Soil that past through the sieve Number 40 (<0,425 mm)	91,33%
4	Soil that past through the sieve Number 200 (<0,075 mm)	86,85%
5	Liquid Limit (LL)	67,70%
6	Plastic Limit (PL)	32,37%
7	Index Plastic (IP)	35,33%
8	Soil Type according to AASHTO	A-7-5
9	Soil Type according to USCS	Silt Clay-CH

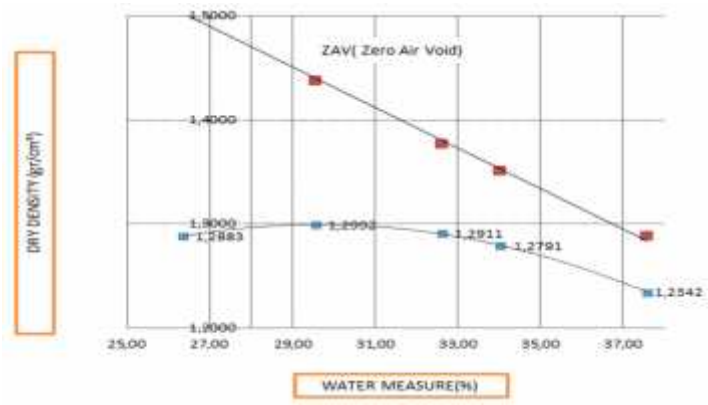

Figure IV.1. The graphic result of compaction test
(*Source: Aris, 2013*)

Standard Compaction Test of Soil

This compaction test performed in Proctor Standard Compaction's method. From the compaction test result will be obtained the maximum value of dry density (γ_k *max.*) and the value is 1.2992 kg/cm^3 and optimum moisture content with 30.8% value. The complete result of this compaction test shown in Figure IV.V below,

Triaxial Test

The purpose of Unconsolidated Undrained Triaxial test is to obtain the changes value of free variables such as the value of cohesion (C) parameter and shear angle (Φ) from the soil added with *Residium Catalytic Cracking* and lime in certain percentages.

Soil Cohesion Value (c)

The changes of cohesion value shown in Table IV.2 below,

Table IV.2 Cohesion Value (c) in mixed variation of RCC and Lime

Mixed Composition	Cohesion Value (c) (kg/cm²)		
	0 Day	7 Days	14 Days
Native Soil (Undisturbed)	0,23		
Remoulded Soil	0,38		
98% Exp. Soil + 0% RCC + 2% Lime	0,57	0.63	0,66
98% Exp. Soil + 0% RCC + 4% Lime	0,66	0,89	1,24
98% Exp. Soil + 0% RCC + 6% Lime	1,24	1,41	1,53
98% Exp. Soil +2% RCC + 0% Lime	0,67	0,78	0,89
98% Exp. Soil + 2% RCC + 2% Lime	0,44	0,56	0,62
98% Exp. Soil + 2% RCC + 4% Lime	0,63	0,67	0,9
98% Exp. Soil + 2% RCC + 6% Lime	1,14	1,39	1,74
98% Exp. Soil + 4% RCC + 0% Lime	0,65	0,76	0,89

98% Exp. Soil + 4% RCC + 2% Lime	0,63	0,75	1,06
98% Exp. Soil + 4% RCC + 4% Lime	0,96	1,14	1,32
98% Exp. Soil + 4% RCC + 6% Lime	1,56	1,77	1,98
98% Exp. Soil + 6% RCC + 0% Lime	0,64	0,68	0,95
98% Exp. Soil + 6% RCC + 2% Lime	0,72	0,79	1,24
98% Exp. Soil + 6% RCC + 4% Lime	0,89	1,23	1,28
98% Exp. Soil + 6% RCC + 6% Lime	1,33	1,41	1,59

Moreover, that changes of cohesion value in each mixed combination of RCC and lime based on the maintenance period shown in Figure IV.1 below,

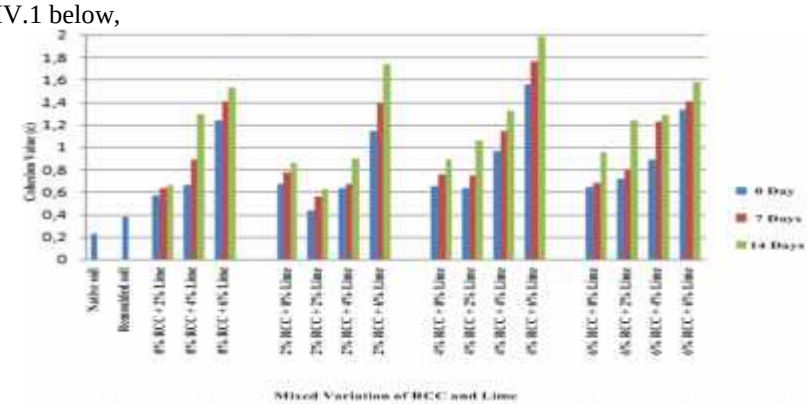

Figure IV.2. The cohesion value in mixed variation of RCC and lime
(Source: The result from the writer's research)

Shear Stress Value of Soil (Φ)

The changes of shear angle value shown in Table IV.3

Table IV.3 Inside Shear Angle Value (ϕ) in mixed combination of RCC and Lime

Mixed Composition	Shear Angle Value (ϕ) (°)		
	0 Day	7 Days	14 Days
Native Soil (Undisturbed)	1,1		
Remoulded Soil	2,98		
98% Exp. Soil + 0% RCC + 2% Lime	8,24	7,72	7,18
98% Exp. Soil + 0% RCC + 4% Lime	7,18	5,72	6,04
98% Exp. Soil + 0% RCC + 6% Lime	8,27	7,33	4,32
98% Exp. Soil +2% RCC + 0% Lime	5,98	5,05	2,74
98% Exp. Soil + 2% RCC + 2% Lime	13,7	8,81	7,11
98% Exp. Soil + 2% RCC + 4% Lime	15,67	14,31	10,8
98% Exp. Soil + 2% RCC + 6% Lime	8,61	5,71	2,24
98% Exp. Soil + 4% RCC + 0% Lime	6,69	7,35	5,33
98% Exp. Soil + 4% RCC + 2% Lime	7,95	7,57	4,247
98% Exp. Soil + 4% RCC + 4% Lime	9,08	4,51	5,07
98% Exp. Soil + 4% RCC + 6% Lime	7,85	2,17	2,31
98% Exp. Soil + 6% RCC + 0% Lime	9,99	7,72	6,89
98% Exp. Soil + 6% RCC + 2% Lime	14,52	5,85	3,11
98% Exp. Soil + 6% RCC + 4% Lime	6,93	8,32	7,09
98% Exp. Soil + 6% RCC + 6% Lime	6,52	5,74	6,77

And the changes of shear angle value in each mixed composition of *Residium Catalytic Cracking* and lime shown in Figure IV.2

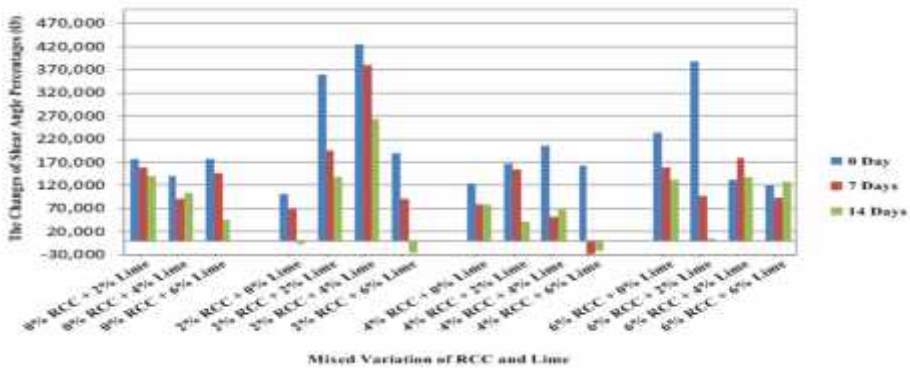

Figure IV.3 The graphical result of shear angle value
(Source: The result from the writer's research)

Soil Shear Strength Value

Soil shear strength value shown in Table IV.4 below,

Table IV.4 Shear Stress Value in mixed variation of RCC and Lime

Mixed Composition	Shear Angle Value (ϕ) (°)		
	0 Day	7 Days	14 Days
Native Soil (Undisturbed)	1,1		
Remoulded Soil	2,98		
98% Exp. Soil + 0% RCC + 2% Lime	8,24	7,72	7,18
98% Exp. Soil + 0% RCC + 4% Lime	7,18	5,72	6,04
98% Exp. Soil + 0% RCC + 6% Lime	8,27	7,33	4,32
98% Exp. Soil +2% RCC + 0% Lime	5,98	5,05	2,74
98% Exp. Soil + 2% RCC + 2% Lime	13,7	8,81	7,11
98% Exp. Soil + 2% RCC + 4% Lime	15,67	14,31	10,8
98% Exp. Soil + 2% RCC + 6% Lime	8,61	5,71	2,24
98% Exp. Soil + 4% RCC + 0% Lime	6,69	7,35	5,33
98% Exp. Soil + 4% RCC + 2% Lime	7,95	7,57	4,247
98% Exp. Soil + 4% RCC + 4% Lime	9,08	4,51	5,07
98% Exp. Soil + 4% RCC + 6% Lime	7,85	2,17	2,31
98% Exp. Soil + 6% RCC + 0% Lime	9,99	7,72	6,89
98% Exp. Soil + 6% RCC + 2% Lime	14,52	5,85	3,11
98% Exp. Soil + 6% RCC + 4% Lime	6,93	8,32	7,09
98% Exp. Soil + 6% RCC + 6% Lime	6,52	5,74	6,77

And the changes of soil shear strength in each mixed combination shown in Figure IV.3.

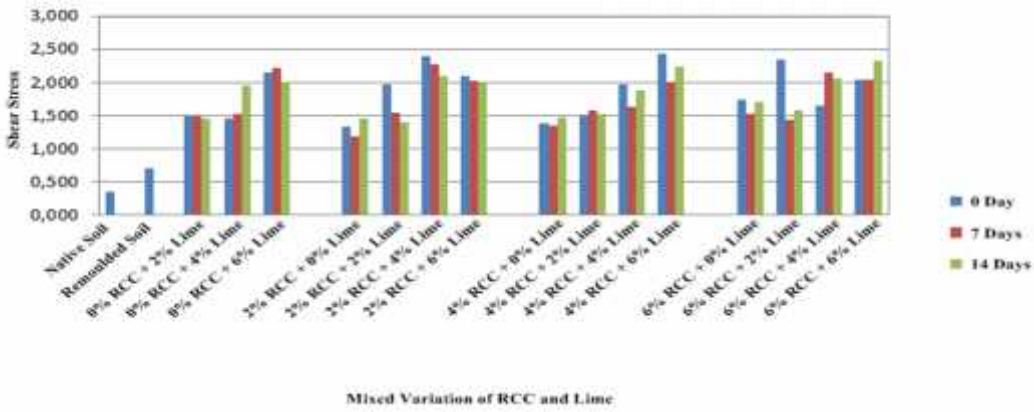

Figure IV.4 The graphical result of shear stress value
(Source: The result from the writer's research)

Discussion

From the test result in laboratory towards the index properties test of soil was shown that the soil taken from Tanjung Api-Api region, Km.27 in Palembang city, South Sumatra has the density of 2.64 kg/cm³ with Plastic Limit 32.37%, Liquid Limit 67.7%, and Index of Plasticity 35.33%. The observed soil is Silty Clay CH.

In soil physical characteristic test, the observed soil categorized in A-7-5 type, according to AASHTO classification system. From standard compaction test result, the maximum dry density value is 1.2992 kg/cm^3 with optimum moisture content of 30.8%.

From the Uncosolidated Undrained Triaxial test at laboratory, the influence of *Residium Catalytic Cracking* (RCC) and lime addition to expansive clay can be seen. The cohesion values of mixed soil seem to be increase rather than the native soil and remolded soil. In case for the cohesion value of mixed soil with lime and without the addition of RCC, is greater than the cohesion value of mixed soil with RCC and without the lime addition.

In the other hand, the lime has greater influences compared to RCC. This phenomenon happens because the lime has better capability of bond power and reduces the sensitive characteristic to water, compared to RCC. The cohesion value that was 0.23 kg/cm² for the native land and 0.38 kg/cm² for remolded soil were all increased during the maintenance period.

Cohesion value increase simultaniously with the addition of mixed composition and during maintenance period. Because by adding those material could enlarge and induces the changes of reaction among the soil particles, RCC powder, and lime and this reaction will lead into the changes of physical characteristics. From Table IV.5 the maximum cohesion value is in the variation of 4% RCC + 6% lime, with 1.98 kg/cm^2 during the 14 days of maintenance period. While in the variation of 6% RCC + 6% lime, occurred the reduction of cohesion value caused by the optimum moisture content is diminish.

For inside shear angle, the value in undisturbed soil is 1,1° and in remolded soil the value is 2,98° where these value had various enhancement in mixed combination. Nevertheless, the maintenance period that becomes longer has made the value of inside shear angle reduce. The cause of this phenomenon is sedimentation process caused by the reaction between mixed compositions with water. This sedimentation process causes the bond power to become stronger and the grain does not loose easily. These are the reasons that the bond power is increase and the grain does not loose easily. The maximum shear angle happens in variation of 2% RCC + 4% Lime during the 0 day maintenance period with 15,67°.

For shear strength value in undisturbed soil is 0,351 kg/cm² where the shear strength value in remolded soil is 0,707kg/cm². The maximum shear strength value occurred in the variation of 4% RCC + 6% lime during the 0 days of maintenance period with 2,426 kg/cm^2.

According to the observed data then can be concluded that the influence of soil treatment using the stabilization mixed material of RCC and lime could influence the shear strength, cohesion value, and inside shear angle parameters of soil. The big enhancement of cohesion percentages possibly caused by the lime and RCC characteristics that could absorb the water because the pore compaction is low and it's hollow and turns the moisture content of soil to reduce and make the soil become more compact. And the lime has the smooth grains where the elements are binding with the other particles.

That was the cause of the combination of these two materials to get the maximum percentage in each material to increase soil density and made the soil cohesion value or shear angle value increase. This thing can be seen based on the observed result with the enhancement of shear strength percentages of 243,154%.

CONCLUSION

From the recent research along with data analysis that was done before then the conclusion are:

1. The soil taken from Tanjung Api-Api KM. 27 in Palembang City, South Sumatra is categorized in expansive clay soil because the index plasticity value is 35,33%. In Unified soil classification system this soil categorized in CH group, which is the type of soil that has high plasticity.
2. From UU Triaxial test, the result changes between native soil without the compaction (undisturbed) is 0,23 kg/cm² and native soil with the compaction (remolded) is 0,38 kg/cm², for the shear angle value of undisturbed soil is 1,1° where remolded soil is 2,98°, and for shear strength value of undisturbed soil is 0,351 kg/cm² where for remolded soil is 0,707 kg/cm²
3. *Residium Catalytic Cracking* (RCC) and lime substituted to the expansive soil can make good influences such as the increase of shear strength parameters including cohesion value (c) and shear angle (ϕ) with the shear strength value itself.
4. At the combination of RCC without lime in expansive soil, the maximum cohesion value occurred at 6% percentages with 0,95 kg/cm², shear angle value in maximum occurred at 6% percentages with 9,99° , and the maximum shear strength value occurred at 6% percentages with 1,7426 kg/cm².
5. At the combination of lime without RCC in expansive soil, the maximum cohesion value occurred at 6% percentages with 1,53 kg/cm², the maximum shear angle value occurred at 6% percentages with 8,27° , and the maximum shear strength value occurred at 6% percentage with 2,218 kg/cm².

6. Cohesion value, inside shear angle, and shear strength occurred in the maximum level in the composition of 4% RCC + 6% lime where the value of each parameter is 1,98 kg/cm², 9,99°, and 2,426 kg/cm².
7. The addition of RCC and lime are effective to increase the shear strength value. This phenomenon can be seen from the increasing of shear strength parameter. Cohesion value increases with 421,05% and the increase of maximum shear angle occurred with 425,834% whereas for shear strength the increase value is 243,154%.

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THE NUMERICAL SIMULATION OF PULL-OUT TEST IN CIRCULAR AND STAR SHAPED ANCHOR PLATE TYPE

Wa Ode Sumartini[1], Ardy Arsyad [2] A. Bakri Muhiddin [3] A. Rachman Djamaluddin [4]

ABSTRACT: This study aims to compare the effectiveness of anchors of circular plate type and star-shape plate type by conducting a pull-out modeling simulation in a laboratory, and by conducting a numerical validation. The anchor modeling was made in three variations of anchor planting depth: 30 cm, 60 cm 90 cm. An anchor of circular plate type with a diameter of 100 mm and a thickness of 5 mm was used as the standard reference. Another anchor of star-shaped plate type with an area equal to the circular anchor was also used, but the diameter was changed. The analysis was conducted by using finite difference grids. A comparison of numerical geomechanics analysis in finite difference approach with those published from small scale laboratory tests show well agreement. Prediction of pull-out capacity was conducted on the two anchor types. The result showed that there was a decrease of ultimate pull-out capacity of 34.78 % in the anchor of star-shaped plate type when it was planted at 30 cm depth. It was probably due to limited amount of soil above the plate anchor. For the anchor planting at 60 and 90 cm, the pull-out capacity increased by 10.31 % and 15.07 % respectively, probably because the coverage of the star-shaped plate diameter was larger. Moreover, there was a greater amount of soil above the anchor as a result of the depth of soil. Anchor plate and star circle has the same model of failure at a depth of 30 cm, where the soil failure happened visible to the surface and anchor plate and star circle has the same model of failure at a depth of 60 cm and 90 cm, where the failure occurred in the location of the land around the plate anchor.

Keywords: *anchor, stars, circles, capacity, finite difference*

INTRODUCTION

Anchors have been developed for various purposes such as strengthening the slopes, retaining walls (plaster), the stability of the tunnel, the foundation for preventing transmission tower style pull, roll, etc. There are many types of anchors depending on the type of load and large, the type of structure and local subsoil conditions. The behavior of anchors in the field indicate that the failure mechanism and bearing capacity of the anchor can be determined by many factors. Most studies focus on massive models shaped anchor plates with various shapes (circle, square) with a variation of the dimensions, depth, and type of load.

Some studies have been conducted on anchor plates. Merifield et al. (2003) examined numerically the anchor plate circle, square, and strip solutions with the lower bound method. The development forms an anchor element with easy installation and sufficient carrying capacity. A study conducted by Djamaluddin et al. (2013) introducing anchor element type star (star anchor) embedded on cohesive compacted soil through an experimental laboratory. They investigated a pull-out capacity, failure models, and displacement due to the effect of: (1) the area of the anchor plate with a fixed diameter, (2) wide diameter anchor remains unchanged, and (3) variations in the depth of embedment. However, numerical studies for this type of anchor has not yet been done to validate the experimental model that have been carried out in the laboratory. This paper presents numerical analysis of star anchor embedded in clay soil by using finite difference.

NUMERICAL METHOD

Engineering problems in Geomechanical and Geotechnical fields are commonly treated through the infinite or semi-infinite media. The best approach to solve these problems numerically is by coupling a finite element or a finite difference with boundary element numerical methods. Coupling the bounded domain modelled by Flac3D (Itasca, 2005), a well-known program that implements an explicit finite difference method, with the boundary element method,

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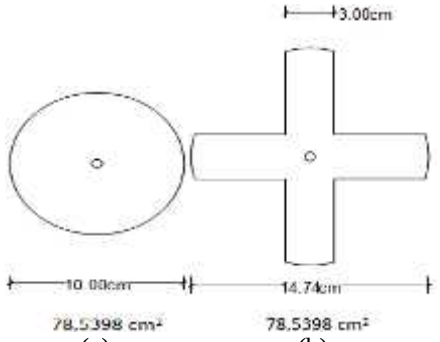
which satisfies exactly the governing Partial Differential Equations (PDE) in the surrounding infinite or semi-infinite medium, combines the capabilities and the advantages of both methods.

Anchors are typically constructed from steel or concrete and may be circular (including helical), square, or rectangular in shape. This study aimed to do the development in the form of modifications plate anchor circular becoming a plate anchor type star with 4 leaves in same area with circular anchor plate type shown in Figure 1.



(a) (b)

Figure 1. Type notation of plate anchor Circular (a) and Star (b)

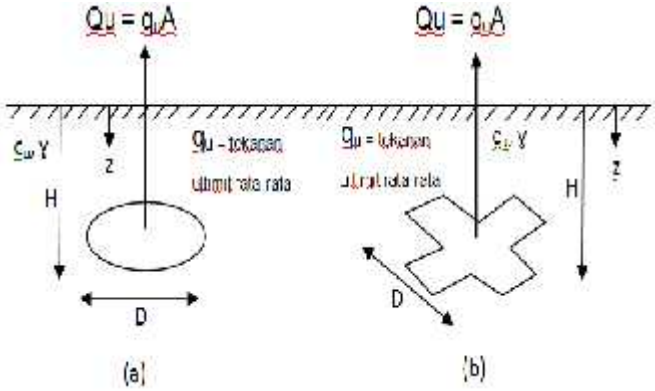


Figure 2. Problem notation of plate anchor Circular (a) and Star (b)

A general layout of the problem to be analyzed is shown in fig. 2(a) and star anchor layout of the problem to be analyzed is shown in fig. 2 (b).

Pull-out capacity of anchors in undrained clay

Ultimate pull-out capacity of anchors in clay undrained conditions proposed by Merifield et al. (2003), are usually shown as a function of undrained shear strength in the following form.

$$q_u = \frac{Q_u}{A} = c_u N_c \tag{1}$$

Where A = the anchor area, c_u = the undrained soil strength at the ground surface, and N_c = anchor break-out factor.

For convenience, Merifield et al. (2003) has been defined the anchor break-out factor for the following two cases:

1. For a homogeneous soil profile with no unit weight ($\gamma = 0$)

$$Nc = Nco, \text{ where } N_{c,0} = \frac{q_u}{c_u} \tag{2}$$

2. For a homogeneous soil profile with unit weight ($\gamma \neq 0$)

$$Nc = Nc\gamma, \text{ where } N_{c,\gamma} = N_{c,0} + \frac{\gamma H}{c_u} \tag{3}$$

Eqs. (1)- (3) reflect the complex nature of the break-factor N_c observed by Rowe and Davis (1978). The break-out factor is a function of embedment ratio (H/B) and overburden pressure, with the latter being expressed in term of dimensionless quantity $\gamma H/c_u$. this indicates that, separate from overall problem geometry, the soil property directly influence anchor behavior.

It should be noted that the break-out factor given in eqs. (1) – (3) does not continue to increase indefinitely, but reaches a limiting value which marks the transition between shallow and deep anchor behavior. The limiting value of the break-out factor is defined as Nc*.

Implicit in eqs. (3) is the assumption that the effect of soil unit weight and cohesion are independent of each other and may be superimposed. This assumption was investigated in the previous study of Merifield et al. (2005), where it was found that the principle of superposition can be successfully applied to shallow strip anchors in clay.

Anchor can be classified as shallow or deep, depending on their mode of failure. This point is illustrated in Figure 3. An anchor is classified as shallow if, at ultimate failure, the observed failure mechanism reaches the surface. In contrast,

a deep anchor is one whose failure mode is characterized by localized shear around the anchor and is not affected by the location of the soil surface.

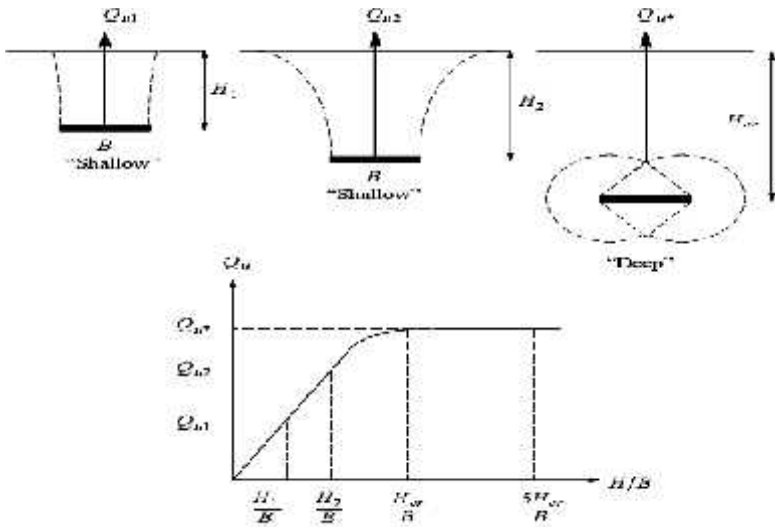
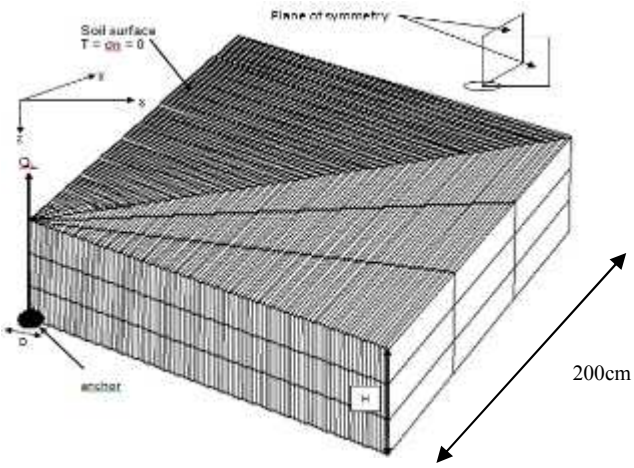


Fig. 3. Shallow and deep anchor behavior.

For a given anchor size, B, and soil properties ɣ, cu there exist a critical embedment that Hcr at which the failure mechanism no longer extend to the soil surface. This type of failure mechanism is typically observed for deep anchor, and is localized around the anchor. The significance of such localized failure mechanism is that the ultimate capacity of the anchor will have reached a maximum limiting value. This arises because the undrained shear strength is assumed to be independent of the mean normal stress.

Three dimensional modeling details

A simplified representation of the boundary grid arrangements used to be analyzed circular, and star anchors illustrated in fig. 4 and 5.



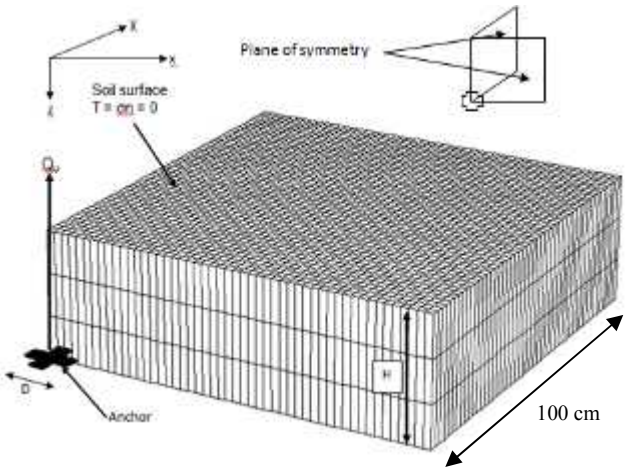


Figure 4. Typical grid used for analyzing of circular anchor

Figure 5. Typical grid used for analyzing of star anchor

The soil mass is first discretized into a number of zone where the boundaries between adjacent zone may be specified as a stress discontinuity or rigid joint. Each zone is then subdivided in three-dimensional space to form a number of radial cylinder grid for circular anchor and brick grid for star anchor within each zone. To model a smooth anchor may be modeled by insisting the shear stress is zero at all element nodes along the anchor. To allow the underside of the anchor to separate from the soil (immediate breakaway), the stress discontinuity between soil above the anchor segment is removed, and the shear stress and normal stress are force to be equal to zero. The soil properties considered for analysis are shown in Table 1.

Table 1. Soil properties

Soil properties	Unit
Dry density	1,054e3 kg/m^3
Bulk modulus	2.7942e6 Pa
Poisson'sratio	0,3
Shear modulus	1.2896e6 Pa
Cohesion	5.48e4 Pa
Friction	0° (tanah lempung)

RESULT AND DISCUSSION

RELATIONSHIP WITH THE ULTIMATE CAPACITY OF THE ANCHORS PLANTING A NUMERICAL TEST RESULTS ARE THEN COMPARED WITH THE RESULTS OF LABORATORY TESTING BY DJAMALUDDINET AL. (2013). BENCHMARKING IS THEN PLOTTED INTO A GRAPH SHOWN IN FIGURE 6.

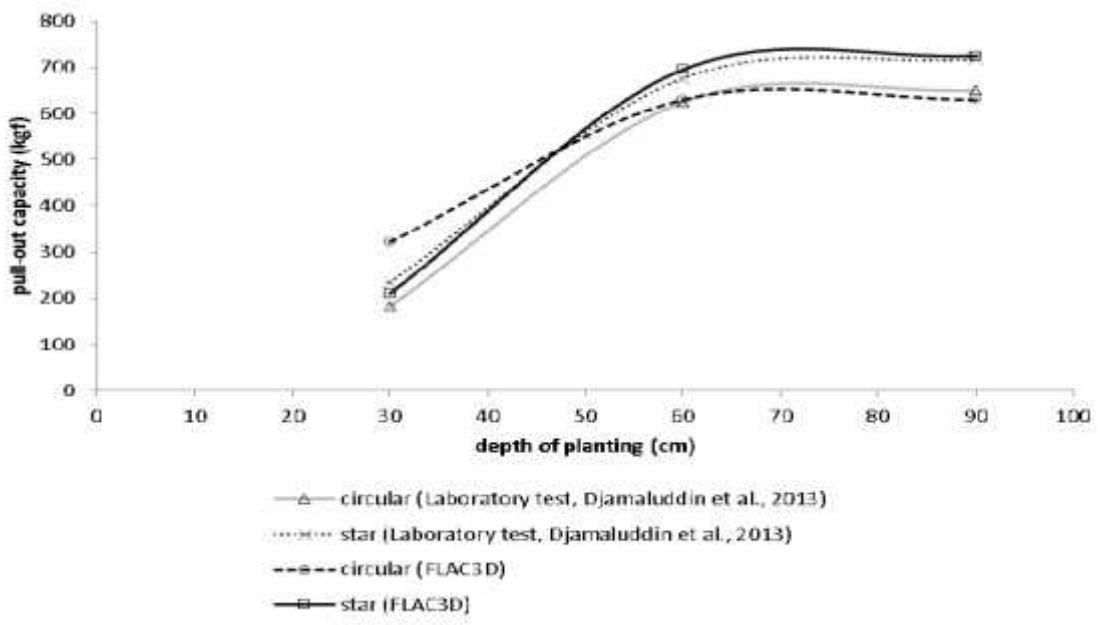


Figure 6. comparison with laboratory test results

Figure 6 shows that for circular plate type, at anchor planting 30 cm, the value of ultimate capacity on FLAC3D numerical testing has a significant difference to the testing laboratory. But for star shape type, the ultimate capacity FLAC3D numerical tests showed similarities with laboratory testing. While at anchor planting 60 cm and 90 cm for both type, the ultimate capacity FLAC3D numerical tests showed similarities with laboratory testing. Figure 6 also shows the similarity of behavior unplug capacity rise of planting 30 cm to 90 cm in planting two types of anchor plates, where the value of pull capacity increases linearly from 30 cm to 60 cm planting and from planting 60cm to 90 cm, the value of the capacity is slowly reaching to a constant value.

From the results of the operation FLAC3D program, can be obtained images contour of displacement which is represent model of soil failure as a result of land stretching caused by the movement of the anchor plate by 10 cm. failure mode of the circular anchor plate and star shaped anchor plate are Plot in any variation of anchor planting. They are shown in Figure 7, 8, and 9.

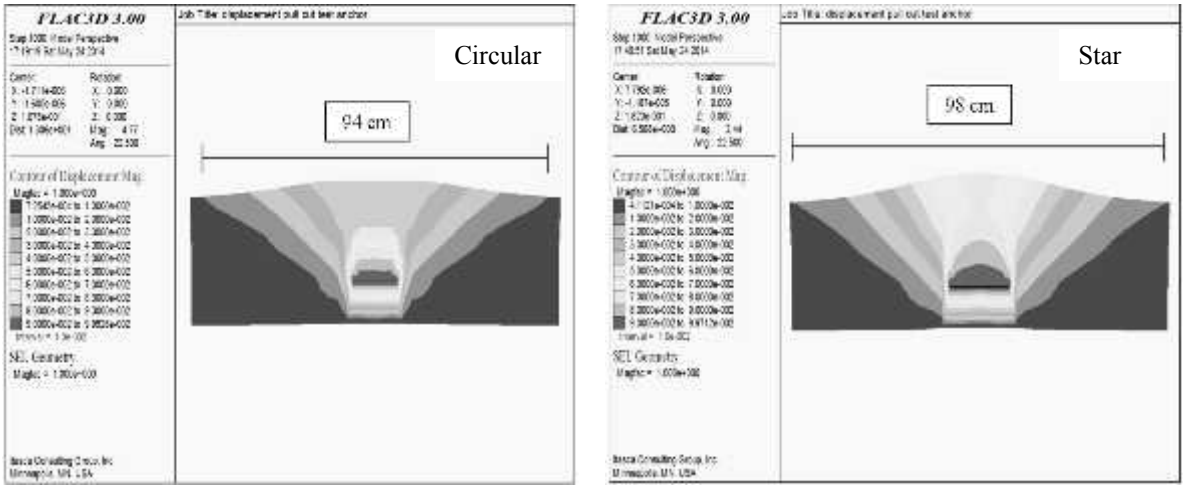


Figure 7. Failure mode at 30 cm of anchor planting

Figure 7 shows that at 30 cm of anchor planting, circular anchor and star shaped anchor has similar failure mode, where the soil failure happened looks to the surface which it is different with circular anchor and star anchor at 60 cm and 90 cm anchor planting, where the soil failure occurs localized around the plate of anchor that shows in Figure 8 and 9.

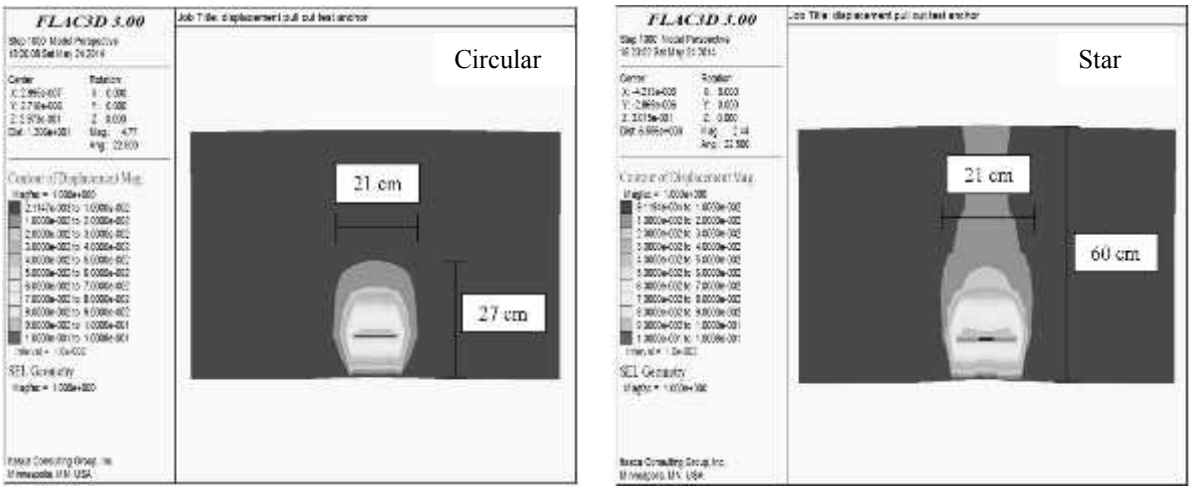


Figure 8. Failure mode at 60 cm of anchor planting

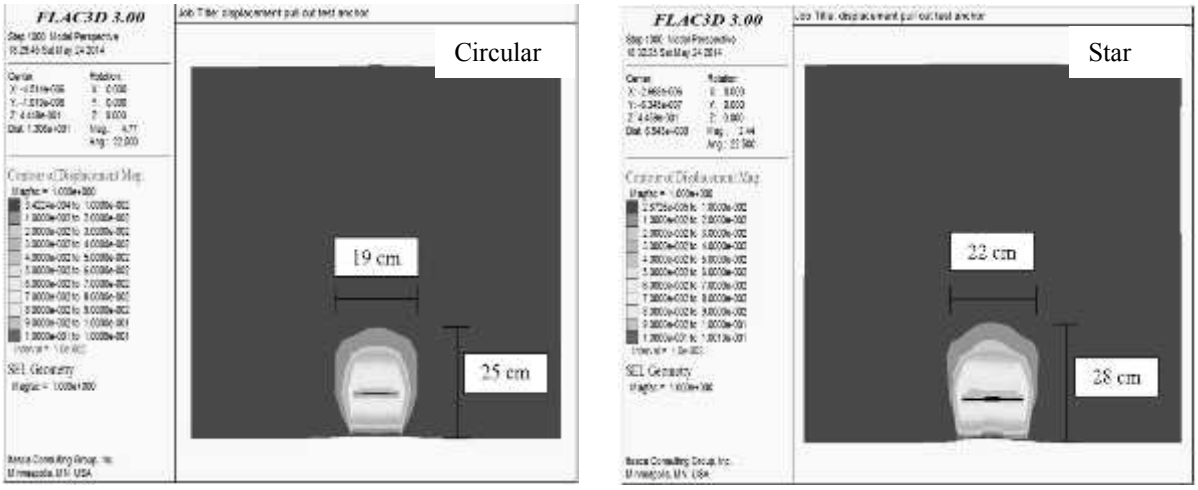


Figure 9. Failure mode at 90 cm of anchor planting

CONCLUSIONS

A comparison of numerical geomechanics analysis in finite difference approach with those published from small scale laboratory tests show well agreement. Prediction of pull-out capacity was conducted on the two anchor types. The result showed that there was a decrease of ultimate pull-out capacity of 34.78 % in the anchor of star-shaped plate type when it was planted at 30 cm depth. It was probably due to limited amount of soil above the plate anchor. For the anchor planting at 60 and 90 cm, the pull-out capacity increased by 10.31 % and 15.07 % respectively, probably because the coverage of the star-shaped plate diameter was larger. Moreover, there was a greater amount of soil above the anchor as a result of the depth of soil. Anchor plate and star circle has the same model of failure at a depth of 30 cm, where the soil failure happened visible to the surface and anchor plate and star circle has the same model of failure at a depth of 60 cm and 90 cm, where the failure occurred in the location of the land around the plate anchor.

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INVERSION GRAVITY IN MAKASSAR

Muhammad H. Syahruddin[1]

ABSTRACT: Gravity measurements in Makassar conducted in April 2012. The results of measurements of gravity in Makassar obtained average is 978,115,522.6 µGal. Highest gravity value is 978,124,333.9 µGal, while the lowest value of gravity that is 978,109,839.6 µGal. The highest value of gravity is near the mouth of the river Jeneberang and gravity of the lowest values were around Bukit Baruga Housing in Antang. Gravitational acceleration measured at the surface caused by the accumulation of body mass of the core layer of the earth to the surface layer in the earth's crust. Regional gravititasi value because of the influence mass density or density near the surface layer of the earth's gravity values corrected with latitude, altitude, topography, tidal, and Bouguer correction. The results of the gravity value correction due to the influence of the density near the earth's surface, called the complete Bouguer anomaly. Gravity anomaly inversion of Makassar used to see the dynamics of sedimentation that occurred near the surface in Makassar. Direction of sedimentation of rivers Jeneberang have a pattern such as gravity anomaly inversion results of Makassar.

Keywords: *gravity, anomalies, inversion, sedimentation*.

INTRODUCTION

Gravity measurements in Makassar were measured in April 2012. Measuring acceleration of gravity in Makassar aims to look at the direction of sedimentation. From the geological map published by BAPPEDA Makassar that sediment rocks in Makassar consists of alluvial rock units, basalt, tufa and bereksi. To understand more about the concepts deposition in geological sciences it is important to study the laws and concepts of physics which is used in geology .. The concept of uniformitarianism (James Hutton), the law of superposition (Steno), the concept of conformity and unconformity, the concept of transgression-regression, cross cutting relationship and others.

RESEARCH LOCATION

Measuring acceleration of gravity location in Makassar are the geodetic coordinates 24'4.9388"E 119 ° - 119 ° and 5 ° 6'7.5913 30'3.2231"E "S - 5 ° 11'59.7595"S. Research sites in Makassar and the surrounding areas can be seen in Figure 1. There are several points in the coordinates of which can not be measured because it is located in the area of swamps and the sea.

Fig 1. Map Of The Location Of Measurement Points Gravity Makassar

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GRAVITY DATA MAKASSAR

Gravity measurement results in Makassar were measured in April 2012 obtained an average gravitational acceleration is 978,115,522.6 μGal. The highest value of the gravitational acceleration is 978,124,333.9 μGal, while the lowest value of the gravitational acceleration is 978,109,839.6 μGal. The highest value of the gravitational acceleration to be around estuaries Jeneberang and the lowest value of the acceleration of gravity located around Bukit Baruga Housing. Gravity measurement results (g-obs) in Makassar can be seen in Figure 2.

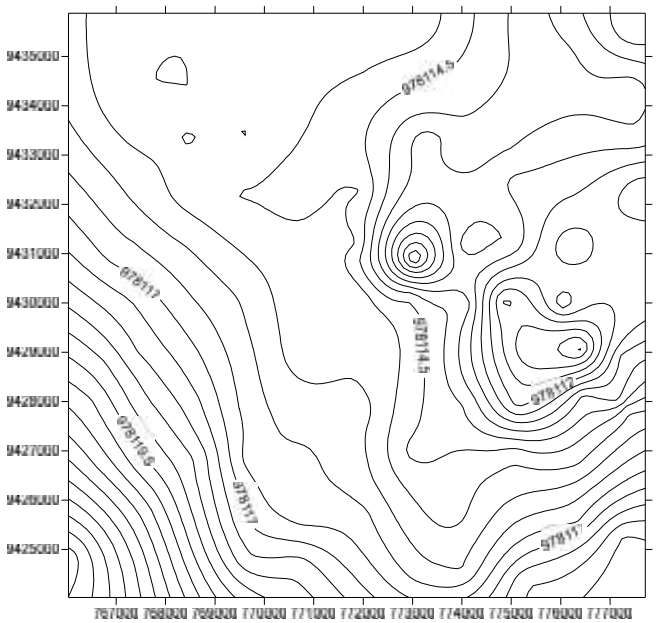

Fig. 2. Gravity Makassar (g-obs)

Complete Bouguer anomaly obtained from the measurement results of observation gravity (g-obs) which has been in correction. Correction g-obs to get CBA is drift correction, free air correction, correction topography, tide correction, terrain correction, correction of latitude, and Bouguer correction .. Complete Bouguer Anomaly (CBA) gravity Makassar city can be seen in Figure 3. Bouguer Anomalies Complete is a combination of the two anomalies that regional anomalies and anomaly residual. Is a regional gravity anomaly gravity anomaly caused by the density of the rock deeper while the residual gravity anomaly is the residual anomaly caused by the density of shallow rocks.

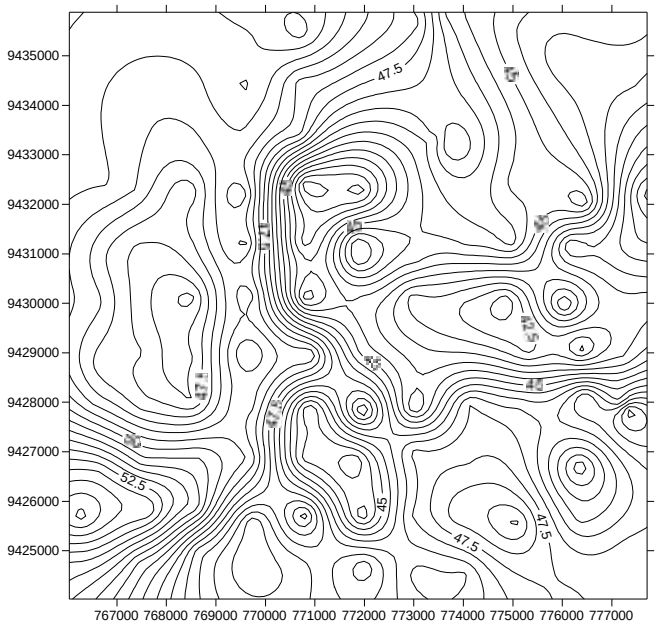

Fig. 3. Complete Bouguer anomaly (CBA) gravity Makassar

To get the regional gravity anomaly and residual gravity anomaly of CBA there should be a separation process. In general there are three kinds of anomalous gravity separation process of regional and residual gravity anomalies. These three kinds of processes are running averaging (moving average), polynomials, and inversion. Rwegional gravity separation process anomalies and residual gravity anomalies in this article using the inversion field (two-dimensional).

INVERSION GRAVITY MAKASSAR

Inversion is the processing of field data involving the completion of mathematical and statistical techniques to obtain useful information regarding the distribution of the physical properties of the subsurface. In the inversion process, we conducted an analysis of field data by conducting curve fitting between the mathematical models and field data. The purpose of the inversion process is to estimate the physical parameters of the rock previously unknown (unknown parameter). Inversion divided into certain levels ranging from the most simple as fitting a line to seismic refraction data to the complex level such as acoustic tomography and matching multidimensional resistivity curve. By using a field inversion at the complete Bouguer anomaly (CBA) can be obtained modeling the distribution of anomalous gravity values regonal. CBA inversion modeling results which is a regional gravity anomaly Makassar can be seen in Figure 4.

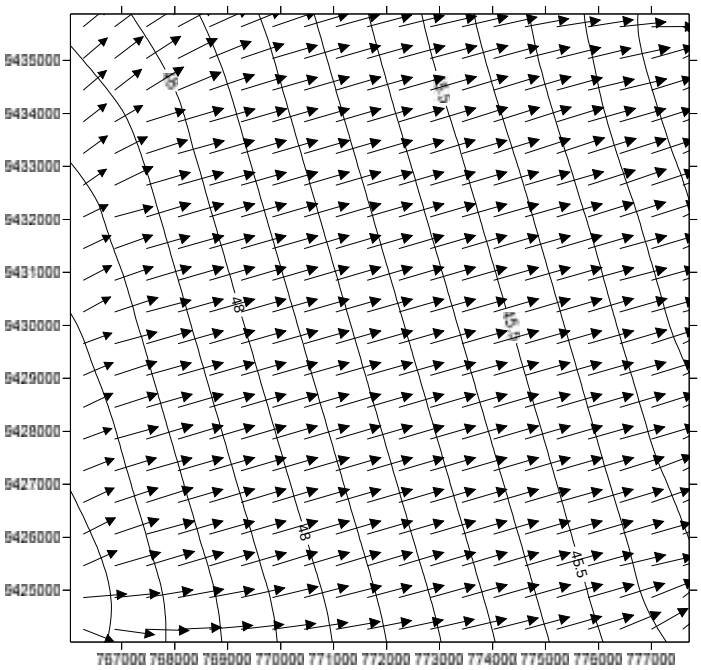

Fig. 4 Gravity anomaly Makassar

The next process is to determine the residual gravity anomaly of Makassar. How to get the residual gravity anomaly is complete Bouguer anomaly Makassar minus its regional gravity anomaly. Makassar residual gravity anomalies can be seen in Figure 5.

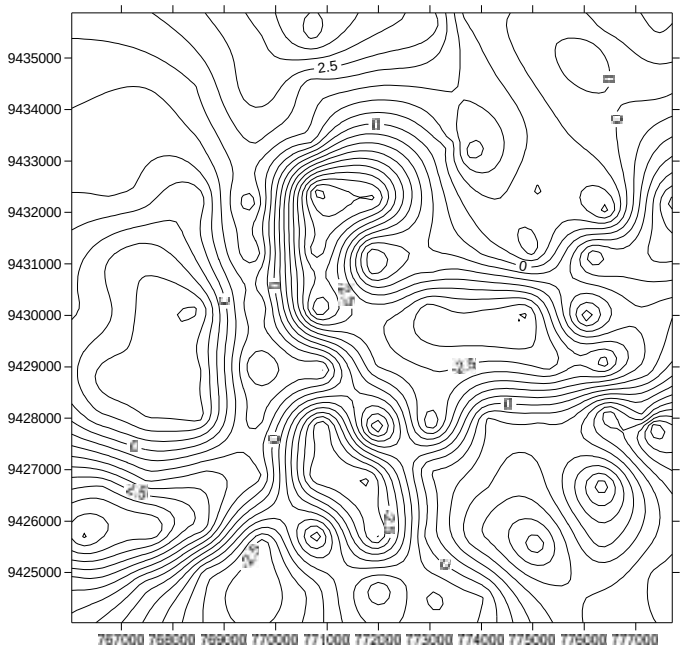

Fig. 5 Gravity anomaly residual Makassar

To get the gravity anomaly at the surface then performed a second residual gravity inversion. Surface gravity anomaly caused by layers of rock on the surface. Gravity anomaly Makassar surface rock layers can be seen in Figure 6

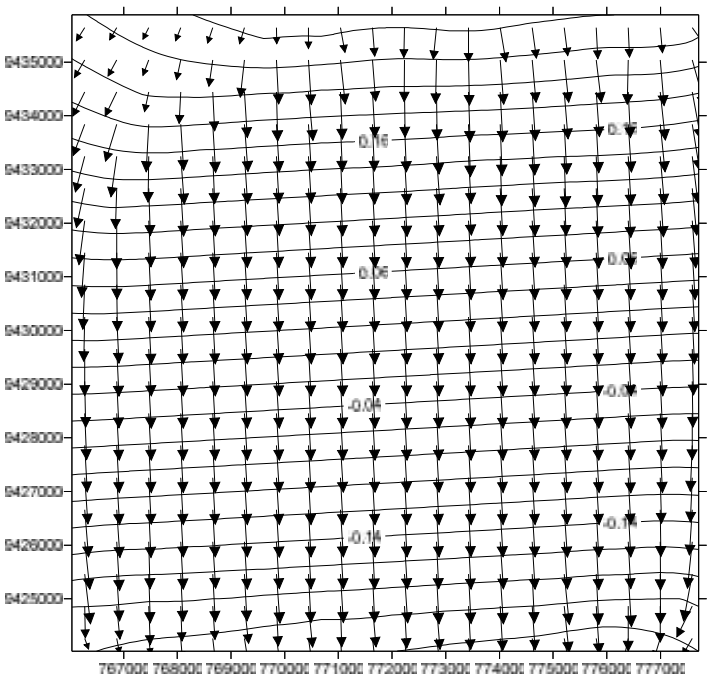

Fig. 6 Gravity Anomali Surface Makassar

Difference in residual anomalies and surface gravity anomalies as surface residual anomaly. Surface residual anomalies have the same pattern as the first inversion anomaly residual. This pattern does not change anymore when inverted higher again. Surface residual gravity anomalies can be seen in Figure 7.

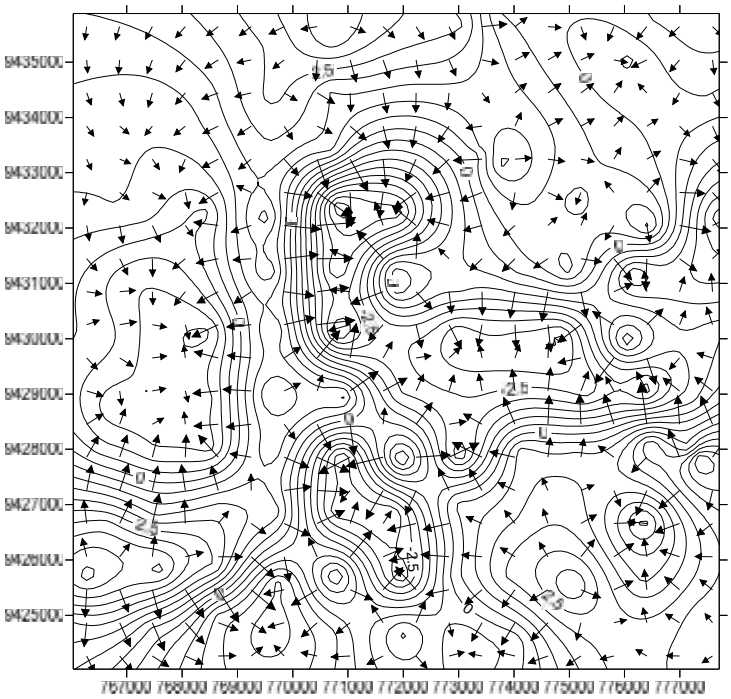

Fig. 7 Surface Gravity Residual Anomaly Makassar

DISCUSSION

Gravity inversion is done to see how the process and direction of sedimentation of makassar. Material sedimentation in makassar from the river jeneberang. At first gravity inversion process as figure 4 shows that the process and direction of sedimentation of the river jeneberang generally leads to the north are more inclined to the east. The sedimentation process is influenced by the flow of the river besides jeneberang also influenced by ocean currents such as the wind and tides. Sedimentation at the next time is the reverse process of sedimentation, the sedimentation was heading north and then turned south. This reversal sedimentation process can be seen in figure 6 the second inversion results. And towards the end of the sedimentation process is controlled by the surface morphology of sediments as shown in figure 7 .. Process and direction of this sedimentation continues periodically to form a layer of alluvial sediments in makassar and the surrounding areas as we see at the moment.

ACKNOWLEDGEMENTS

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GEOLOGY AND AVALANCHES IN THE REGENCY OF ENDE

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ABSTRACT: Avalanche is a form of natural geological, becana could threaten and harm the following human activities including destroying the work of development. Some aspects of geology can become triggers of slope movement of soil and rocks into the lower levels. It is caused due to the shape of the geometry of the slope, type of material, soil and rocks, structural discontinuity (perlapisan, cracks and fissures of rocks) as well as external factors, the weathering of surface water and groundwater. The slopes have experienced a landslide at public facilities (roads, bridges, levees and other) that are detrimental to society, rarely there are demands from party aggrieved. Ende Regency area consists of rugged rock massif which groups suffered weathering. So often experienced an avalanche of both small scale and large scale. Methods undertaken in this research include litologi, morphology and geological structure.
Results of the study revealed that the morphological region district of Ende is very rugged with kelerengan over 30° amounting to 71.3% and found the water flow of the furrows and fissures. Litologi weathering is resolved into volcanic rock, lava and tuff with stratigraphic position is special. Based on the analysis it was concluded that there were fields of gelincir between the lapukan Tuff rocks with its origin.
Therefore needs to be given a proper geological information to the appropriate mitigation efforts. So the synergy between business technocrat with decisive policies are indispensable to tackle landslide.

Keywords: geology, landslides, litologi, Ende

INTRODUCTION

A landslide is a geological phenomenon which is common and will surely happen in order to find the balance of nature. The main factors that cause the landslide is a geological factors, climate, vegetation and land use. Upon entering the rainy season in General in Indonesia has increased due to landslide hazards human events such as loss of vegetation, excavation, land use change and others.

Cursory observations in the field conditions of instability or failure of the most frequently encountered in the field has no horizontal surface (circular) as well as the affected component of gravity. When the force (load) that occurs because the component of gravity in such a large shear resistance, so the total field is exceeded, then the landslide will happen avalanches (Hardiyatmo, 2002). Events that occur at this location is in rainy season, rain triggers avalanches is rain that had a certain bulk, so that rainwater is able to seep into the soil and encourage the slope to avalanche. In general the type of rains trigger landslide in Indonesia, namely the type of heavy rain, where rainfall can reach 70 mm per hour or more than 100 mm per day. The type of heavy rain will only be effective trigger avalanches on land which slopes easily absorb water (Karnawati 1996, 1997). On the slopes of such avalanches may occur in the early months of the rainy season. Avalanche research relating to the above factors have often done, so that the whole considered carried out analyses of field experience instability or failure. The landslide occurred in the natural state is a part of the existing slopes, soil slope landslide case example in STA 22-STA 24 Tomberabu Village sub district of Ende, Maumere Ndona majors on the area thoroughly in a landslide that occurred back in the form of land is down at the time of the rainy season in January, with avalanche conditions such as in Figure 1.

Figure 1. Landslide in Sta STA 22-24
Tomberabu, Ndona, Village Kab. Ende

In addition, morphological appearance of the ground slopes in hilly area of the Regency of Ende is also categorized as a structural process of original land (endogenous) covering the area of faulting and folding. The flap of a antiklinal which has undergone erosion and changed into the antiklin Valley has a material form of sedimentary rock, the sandstone and breksi.

Structures that develop on the rocks will certainly reduce the strength (resistance) of the rocks, so some rocks become easily decayed and crumbling. The spread of these rocks almost along the road of Ende Maumere and form steep hills. Surely the State geology like this will be one of the factors causing the occurrence of mass movements. In addition the clefts have rock-intensive fissures very easily bypassed by the water, so the water deposits in the saturated soil mass movement led to.

The phenomenon of an unstable land can be classified into the collapse of a slope (slope failure) and avalanche (landslides).

THE GEOLOGICAL FACTOR AS ONE OF THE TRIGGERS OF AVALANCHES

Ende Regency is located on the island of Flores, East Nusa Tenggara province (NTT). Is about 2046,59 km^2. Ende Regency is the extreme mountainous regions beyond the mountains is as high as 1700 m and only a little flat land. Most of the region's slopes are very steep (90% more precipitous slopes region) and with the start of the rainy season many types of movement of landslides, mud and rock slides which may occur.

Geological conditions such as litologi, topographic and geologic structure of the Regency of Ende is generally considered to be very supportive of the occurrence of landslides. Regional geology show that counties Ende composed by volcanic rock with lots of fissures. This type of avalanche is considered the most dangerous area in the District of Ende is the debris flow. Such events have occurred in November 1988 and April 2003, where a very high rainfall has resulted in a large number of instances of landslides, and debris material as well as floods in the District of Ende.

In particular the flow of rombakan have been repeatedly endanger a large number of villages in the region. Aside from the immediate danger for people and houses and agricultural land that are in the path of material of this kind of avalanche, also damaged telecommunication lines, trails, including the supply of clean water. The regions suffering the most severe disasters this is a sub-district of Detusoko, Ende Ndona Timur, Nangapanda, Maurole, Wolowaru and South of Ende. Ende Regency is one of regencies in Flores Island, with the Division of land into five units of the system unit as in table 1 and Figure 2.

Table 1: Land division system Counties Ende

Forms Of Relief	The slope	Broad	
		Ha	%
The lowlands	0-3°	3000,00	1,46
The curly Plains	3-8°	3180,30	1,55
Wavy Plains	8-15°	11972,61	5,85
Hilly	15-30°	40092,89	19,59
The mountains	>30°	146413,00	71,54

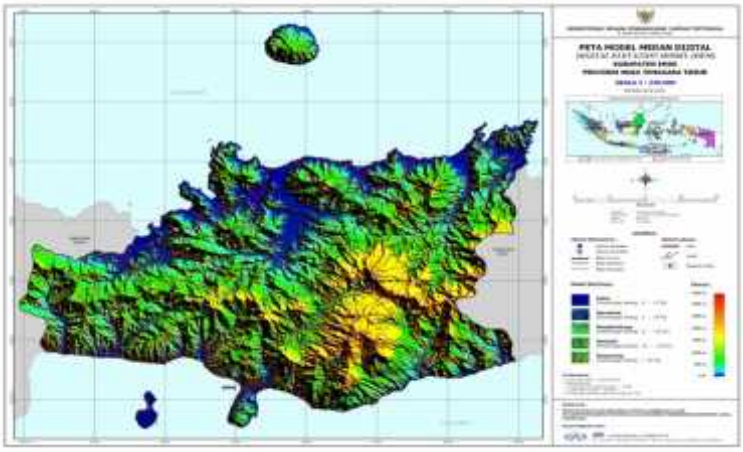

Figure 2: map of the dividing Land System Ende Regency
source: BAPPEDA Ende

Based on field observations of most avalanches usually occur in the hilly area, but the symptoms still possible landslide phenomena in land system, etc. This is caused due to other factors such as rain, weathering, and the encroachment of the sea and earthquakes that could trigger landslides. Some experts say that the success of the handling of landslide largely determined from the results of the introduction of such disasters as good and complete as determined by several factors, among others;

1. The Type and shape of landslide slope collapse
2. Relationship between the geology condition and form avalanches
3. Slide Activity are assessed from the degree of hazard and risks
4. Characteristics of the shape of the avalanche slopes can be the collapse, ruins, debris flow, rotation, which is influenced by the structure of materials, strength of materials, run off and seepage.
5. Use mathematical analysis for stability of slope

The type and shape of landslide slope collapse among others;

- Transfer of the rocks or the ground (ruins, landslide, flow)
- Forms the surface of the avalanche (straight lines, curved, irregular)
- Causes (sliding, pore water pressure, the flow of sludge)
- The number volumn of avalanches and landslides speed.

PUBLIC ATTENTION TO THE AVALANCHE OF ENDE

In General, all the mountains in the District of Ende can undergo ground motion/avalanche. Especially on the steep and slopes. Especially on steep slopes with the steepness of over 30 °, as well as the area of the bottom of the slope like this should be excluded from the activities of housing development or other means. Although there is no fixed rule, local geological conditions must always be considered.

Also in the area of the foot of the cliffs of the Valley or down the Hill will be more affected by landslide-prone area than on the top, therefore avoid to build housing in this area. □ dense forest turns out can reduce (but not stop) the risk of the occurrence of landslides. Therefore the whole village has to dwell on the danger of avalanche must protect forests and natural vegetation in the slopes above their village. When there is no longer a forest, they must be replanted.

Sometimes a slope looks cracked in a matter of days and weeks before finally experienced a landslide. Cracks the ground usually on the top of the area that would eventually become an avalanche will then override the settlement on the bottom. Therefore residents should often check state of the slopes around the rural. When they found the ground cracks like that, people should close the cracks with clay to prevent water ingress. This is important especially in the rainy season because most avalanche triggered by heavy rains.

POTENTIAL LANDSLIDE AREAS

Generally very steep mountain slopes, and at the beginning of the rainy season usually occurs some kind of ground motion/falling rocks, e.g. avalanche (rock fall), mud flow and debris flow. In addition, the island of Flores, as a part of the Banda arc of the volcano, has a number of active volcanoes and earthquakes are related, then it also should be seen as a region with a high level of seismic activity. Geologic hazards such as these became a serious threat not only to the life of local residents, also threatens the sustainability of the social and economic development in the Regency of Ende.

The landslide-prone in the Regency of Ende in terms of conditions of vulnerability of soils with high intensity has an area of 12.293 Ha area with among others; Nangapanda, Ende, Ende Tengah, Ende Utara, Ende Timur, Ndona, Ndona Timur, Detusoko, Wewaria, Maukaro, Detukeli, Wolowaru, Kotabaru, Kelimutu, Wolojita, Lio Timur, Lepembusu and Ndori.

THE RISK OF SLOPE MOVEMENT

Avalanche are the most frequent natural disaster happened in Indonesia. The term avalanche covers various types of movements on the ground, including rocks, debris flow, slump, and others. Characteristics of geology, geomorphology, geography and land use to determine the tendency of disasters that occur. The disaster was triggered by the high rainfall, earthquakes or ground movement caused by earthquakes. A landslide is a geological process that is natural, but the tendency can be increased or induced by human activities, such as land use is wrong, or deforestation. Human activities, such as construction of roads on a cliff, can also trigger landslides. However, such factors cause landslides most commonly encountered is when rainfall is high.

Geological structure also plays against the occurrence of ground motion. And of Sumaryono Triyana (2011) explained that the distribution structure in the form of very intensive and easy fault move especially if there is a trigger such as rain or vibration. land use is wrong, or deforestation. Human activities, such as construction of roads on a cliff, can also trigger landslides. However, such factors cause landslides most commonly encountered is when rainfall is high.

A disaster is something that we don't expect, therefore, an understanding of the process the following factors cause ground movement becomes very important for the Government and the community. Alternative aspects of disaster prevention (preventative), reduction (mitigation) and countermeasures (rehabilitation) need to be examined in depth.

Mitigation is any effort to minimize the consequences of a disaster at the time of disaster or disaster occurs, which in this case is performed both in the scale of local, national, or regional. Some establishments that handle this among others Geology Directorate The Directorate of volcanology, Environmental and Natural Disaster Mitigation, EIGHT, BPPT, local government level I and II, Department of mines and energy, higher education, Bakornas, Kimpraswil, and other research institutions (among others; mapping vulnerable areas of avalanche, landslide identification, mapping the way for tackling the avalanche test, environmental properties and land). However casualties due to natural disasters, landslides still occur, it indicates that the disaster mitigation should be improved implementation.

Pay attention to the explanation above sebenanya the Government and experts have done a lot of things. But a reality and a stark contrast is the landslide continues to occur on an ongoing basis. This can be summed up as that:

- Location is often the vulnerable area is landslide landslide naturally (geological)
- Landslide caused by human behavior
- Any changes the function of land in hilly area or the mountains upper slopes.

THE STRUCTURE OF THE MATERIAL SLOPE

Existing material on the slopes of the Hills District region of Ende is the result of physical and chemical weathering and originating from organic ingredients. If the results are still weathering the place originally called residual soil. Visually the residual soil material consisting of a coarse and fine-grained berbutir. Coarse grains consists of gravel and stone. Fine grain is composed of sand, silt and clay. The residual material properties reflect the properties of weathering of rocks mechanics foundations. Rotten rocks composed of mineral rock-forming stem material insoluble and is usually in the form of very fine grains (geohazard, 2007). For more detailed can be seen in table 2 below.

Table 2: Soil texture in the Regency of Ende

Soil Texture	Broad	
	Ha	%
Refined	7572,42	3,7
Medium	45618,71	22,29
Rough	116881,32	57,11
Mix	34587,54	16,9

Sumber: BAPPEDA Ende

According to the depth of the land effectively, divided into;

- 52,96% effective soil depth 0-30 cm
- 11,32% effective soil depth 30-60 cm
- 30,22% effective soil depth 60-90 cm
- 5,50% effective soil depth >90 cm

ANALYSIS OF STABILITY AND COUNTERMEASURE METHODS

How to calculate slope stability with the method of balance limit is by calculating the magnitude of the shear strength necessary to maintain stability and comparison with existing shear strength. This comparison will be obtained from the safety factor. Initially assuming that happens with a certain landslide areas of instability or failure and calculating style and moments that cause the occurrence of landslides on the field, due to the heavy ground. It's called the driving force or moment of movers. Further by calculating the force or moment of resistance to can hold the soil shear strength. With this moment together two then be acquired safety factor against sliding on areas of instability or failure in question. When the security factor obtained is 1 means that the slopes are in danger of collapse of the slope. In many ways, should be considered as well the differences between assumptions that cannot be avoided, which is used in the calculation of, including all the fault of the constants of soil mechanics with the actual price of a layer of soil.

To prevent catastrophic landslides, particularly in regions with high levels of vulnerability and, a couple of alternative mitigation measures can be undertaken, such as: a. Avoid development planning in areas of high avalanche danger, and danger areas nearby are crumbling and its surroundings. The building must be located away from steep slopes, streams and rivers, the river dried up/channel flow for a moment and basic mouth/estuary mountain stream. b. If residential developments need to be done in areas such as those above, please check in advance to the experts to evaluate the landslide hazards or designing the buildings with the right techniques to reduce landslide is likely to occur. If possible, perform a slope stability investigation for more details. c. Soon planting back ground/land damaged or open land as it lost ground to raise the possibility of avalanches and other soil mass movement. d. If possible, avoid the practice of adding a load of moisture content on slope-the slope caused by the making of a garden or irrigation.e. Consider relocation of a building or settlement in the area was in danger of a landslide, if other measures cannot be done. f. Move blocks of rocks that are blocking the flow of rivers, channels that can increase the danger of slides. If it is not possible to move, while many residents living immediately/live at the base of the flow and moved during the season of heavy rain. g. Move residents living near the slopes of the steep ravine, mount, base, to the neighbouring villages during the season with a high rainfall.

CONCLUSION

Landslide in the Regency of Ende, which often happens due to the geological aspect, among others, by the litologi, morphology and geological structure as a trigger for the stability of the slope is disturbed include;

- Geometric forms related to the morphology and topography of steep > 30°.
- The type of material the avalanche consists of volcanic rock weathering, tuff and lava with the stratigraphic position of special forming areas of avalanche. Discontinuous materials structure. factors of surface water (run off) and ground water.

By knowing some aspect is complete then it will help efforts to analyze the stability of slopes, so the suggestion of countermeasure can be done appropriately and correctly. It's time the work is carried out geotechnical professional to avoid losses and demands due to ignorance about the risks of Geotechnics.

ADVICE

To avoid the occurrence of landslides in Regency of Ende, then it is recommended;

- Perform the handling on the farm-critical land on steep slopes with the kind of litologi Tuff with perennials and deep-rooted.
- If the function changes the land held the previous geotechnical survey needs to be done.
- The need for guidance by local Government on landslide-prone areas about the threat of disaster, ordinances, anticipation and handling.

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EFFECT OVERBURDEN PRESSURE AND HOLE SIZE ON SELF HEALING CAPACITY OF GEOSYNTHETIC CLAY LINER

Kartika SARI [1] and Jinchun CHAI [2]

ABSTRACT: The Effect of overburden pressure and hole size on self-healing capacity of Geosynthetic clay liners (GCLs) has been investigated by laboratory leakage rate tests. Two types of GCL were tested, geomembrane supported GCL (GM-GCL) and geotextile encased GCL (GT-GCL). A serial test was conducted on GCLs with the hole of 5-50 mm in diameter under overburden pressure of 25 – 200 kPa. The test result indicate that For both GM-GCL and GT-GCL samples, it is suggested that a hole up to 30 mm can be self-healed. In case of overburden pressure effect, permittivity value tends to reduce as overburden pressure increase.

Keywords: Geosynthetic clay liner, self-healing capacity, leakage rate test, hole

INTRODUCTION

Geosynthetic clay liners (GCLs) are well-known as barrier system in municipal solid waste landfill application. They are widely used to protect soil and ground water from pollution originating from waste disposal landfill (Bouazza 2002, Bouazza and Bowders 2010, Touze-Foltz et al. 2008, Shackelford et al. 2010). GCLs product became famous in geoenvironmental engineering since have some advantages over conventional bottom liner and cover. GCLs are fast and easy to install, and have low hydraulic conductivity. They have capacity to self-heal any rips or holes due to swelling ability of bentonite which part of GCL product. However GCLs have some disadvantages such as easily damaged during placement, loss of bentonite during placement and lead to increase hydraulic conductivity when contacted with certain chemicals. There are possible cause defects in GCLs : (1) on-site placement and seaming, (2) handling of GCL rolls, (3) the placement of drainage gravel over the liner system, (4) traffic over the liner or the overlying protection layer, (5) placement of the waste in a landfill, (6) manufacturing defects and etc.

In 1998, Evans et al, reported that GCL damaged discovered after waste removal in Mahoning landfill. Several large tears (1 to 6 ft) and numerous small tears (less than 1 ft) in the geomembrane which is in the part of GCL and two ruptured geomembrane seams were found. Some part of bentonite was removed randomly from geomembrane. Geomembrane which is part of GCLs has also possibility to be damage in landfill especially if GCLs are installed directly under primary leachate collection system (PLCS). Nosko and Tauze –Foltz (2000) reported result from electrical damaged detection systems which installed at more than 300 sites from 16 countries. The study showed that mainly (17%) damages of geomembrane were caused by stones during installation of PLCS and the rest by heavy equipment (16%), inadequate seam (6%), workers (6%) and cuts (1%). Several studies have been conducted to investigated influence factors of self-healing mechanism on GCLs (e.g. Mazzieri and Pasqualini 2000; Babu et al. 2001; Egloffstein 2001; Takahashi et al. 1999). The tests result confirmed the presence of self-healing capacity of GCLs.

MATERIALS AND PROPERTIES

GCLS

Two types of GCL are tested, geomembrane supported geosynthetic clay liner (GM-GCL) and geotextile encased GCL (GT-GCL). The GM-GCL consists of a 4 mm-thick of granular bentonite layer glued onto a 0.5 mm thick of high density polyethyelene geomembrane (HDPE). The GT-GCL consists of granular bentonite powders encased by geotextile, i.e. a nonwoven geotextile as “cover” and a woven geotextile as “carrier”. The two layers of geotextile are connected by needle punched fibers with pitches of 3 mm x 4.5 mm and thermal treatment. The weight of the GM-GCL and the GT-GCL are about 53 and 49 N/m2 respectively. Both of the GCLs tested used Na^+ bentonite.

Liquids and interaction behavior of bentonite and liquids

Tap water, 10 g/l of NaCl, 10% of ethanol solution and 1.1 gr/l of $CaCl_2$ solutions were used as liquids in the tests. Properties of liquids and its interaction with bentonite are listed in the Table 1.

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Table 1. Properties of liquids and interact properties of the liquids and the bentonite

Type of liquid	pH	EC (µS/cm)	W_L (%)	W_P (%)	Free Swelling Index (ml/2gr)
Tap water	7.02	105	537	45.8	21.5
Ethanol	7.46	85	560	67.4	30
NaCl	7.24	17600	235	46.3	16.5
$CaCl_2$	7.60	199	165		9

Swelling pressure of bentonite were tested based on procedure of Method-C of ASTM D 4546-96 using an odometer device. Initial water content of bentonite was adjusted to about 30% - 100% and put into a mold of 60 mm in diameter and 20 mm in height. Apply a vertical pressure of 300 kPa for 2 hours to compress the sample. At the end, sample had a dry density of 760 kg/m3 – 1,020 kg/m3. Then the sample was cut into 5 mm in thickness and place again into the equipment for swelling pressure test. The tests were conducted with constant volume condition and water was supplied until swelling pressure was constant. Swelling pressure versus final water content of the samples is presented in Fig. 1. The figure showed swelling pressure reduced as water content increased.

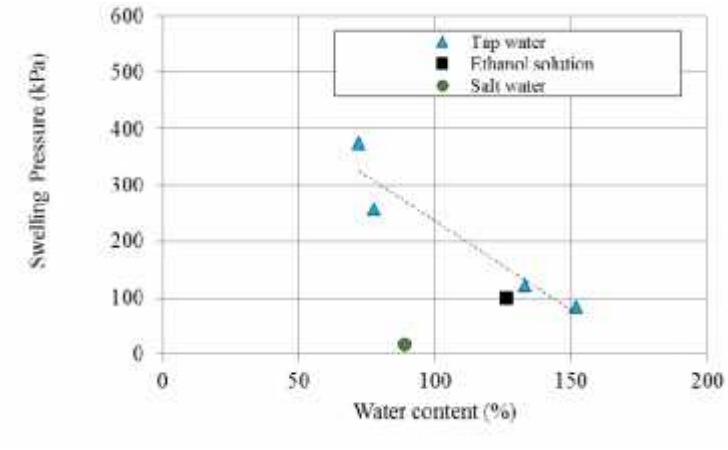


Fig. 1 Swelling Pressure of bentonite

Undrained shear strength (Su) of the bentonite

Self-healing of GCL is the expansion of bentonite into a defect hole and/or squeezing of hydrated bentonite into the hole by overburden pressure. Undrained shear strength (Su) value is one of key parameter affecting the squeezing effect. Su value were measured by laboratory vane shear equipment. The blade of the vane was 20 mm-diameter and 40 mm-height. Result of the test was shown in the Fig. 2

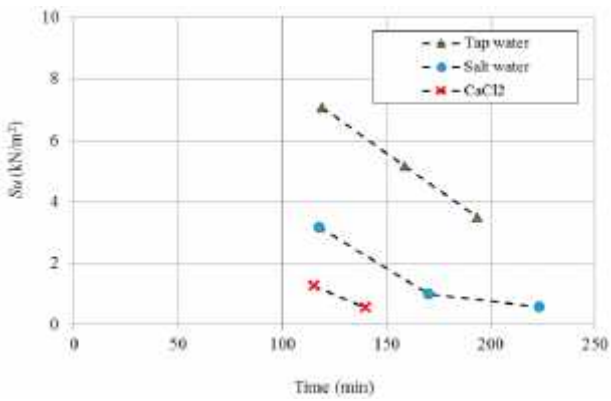


Fig. 2 Undrained shear strenght of bentonite with different liquids

LEAKAGE RATE TEST

In order to collect more data in limited time and provide a cross check on the reliability of the test result, both of constant head and falling head devices were used.

Equipment

Figure of constant head test was shown in Fig. 3. The device consist of:

(a) a transparent cylinder made of acrylic resin with an inner diameter of 150 mm (wall thickness of 5 mm) and height of about 400 mm;

(b) upper and lower pedestal made of stainless steel, and a porous stone with a diameter of about 120 mm is fixed at the top of the lower pedestal;

(c) a piston made of stainless steel, which is perforated with 3 mm diameter holes at 20 mm pitch to allow for drainage, and a ceramic porous stone with a diameter of 120 mm is inserted at the center of the bottom of the piston; and

(d) a bello–fram fixed to the top of the upper pedestal for applying overburden pressure (s).

A 4 mm diameter O ring lubricated with silicone grease sealed cylinder and piston and it fixed around the piston.

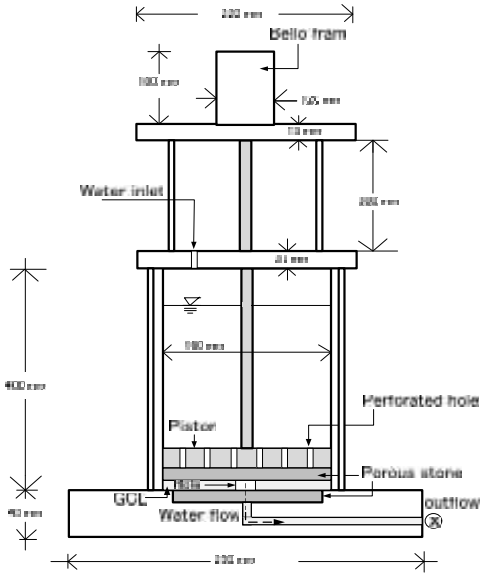


Fig. 3Schematic diagram of Constant head leakage test

Sketch of falling head test device is shown in Fig. 4. The equipment consists of upper and lower part which made of copper. The lower part consists of a container with 150 mm inner diameter. A porous stone, 50 mm in diameter is inserted at the center of the bottom of the container. A Loading plate, 150 mm diameter is in the upper part. A porous stone, 120 mm in diameter is inserted in center of upper part of it and in turn connected to a burette. Cross of sectional area of the burette is 200 mm2. The overburden pressure can be apply is up to 200 kPa for both constant head test and falling head test.

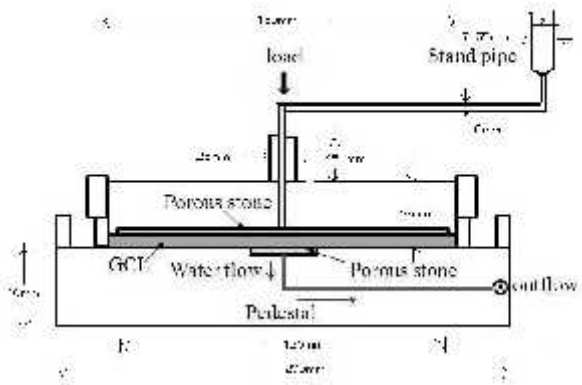


Fig. 4 Schematic diagram of Falling head leakage test

Test Procedure

(a) Preparation of GCL specimen

GCL specimen was cut in 150 mm-diameter and create a hole of 5 – 50 mm in diameter in the center by a driller and/or cutter (Fig.5). For GM-GCL, the geomembrane (GM) side is glued to the piston (constant head test) or the loading plate (falling head test) on annulus area along the outer periphery to prevent flow at GM/piston or GM/loading plate interface. Whereas for GT-GCL, the specimen is placed in the lower part of the device, and to prevent leakage through the possible gap between the periphery of the specimen and the equipment, bentonite is then carefully put around the periphery of the specimen.

(b) Constant head leakage rate test

Install the piston (with GCL specimen attached in case of GM-GCL) into the cylinder. Then apply desire pressure (0, 50, 100 and 200 kPa) and maintain for1 hour. Pour liquid with a water head of 320 mm above the GCL specimen and then the test was started. Measured the outlet flow rate periodically until stable. The fluid was added periodically to maintain a constant water head. Volume of leakage through defect of GCLs was measure within a time interval.

(c) Falling head leakage rate test

Set up the test and apply 50 kPa overburden pressure for 1hour, then apply the desire overburden pressure (25, 50, 100 and 200 kPa). Set up water head of about 1000 mm on the top of specimen through burette. Open the valve for inlet flow and start the test; and record water level in the burette periodically. The flow rate is calculated using the amount of inlet water flow and the corresponding water head. The test was continued until the calculated apparent hydraulic conductivity became stable.

(a) (b)

Fig. 5; (a) GM-GCL sample, (b) GT-GCL sample

Condition Investigated

Effect of overburden pressure and size of hole is investigated under overburden pressure of 25, 50, 100 and 200 kPa by using tap water and 1 gr/l NaCl solution as liquid. Size of hole damaged was generated to 5 mm, 25 mm, 30 mm, and 50 mm in diameter.

RESULT AND DISCUSSION

GM-GCL

The flow rates (Q) of the falling head leakage rate tests, are calculated based on Darcy law. For comparison purpose by assuming the same head difference as for the constant head test using the following equation:

$$Q = \frac{2.3 a \Delta h}{\Delta t} \log\left(\frac{h_1}{h_2}\right) \quad (1)$$

where a is the cross sectional area of the burette ($2x10^{-4}$ m^2), Δh = 0.32 m (head difference) which is the same value as used for the constant head test, Δt is duration of observation, h_1 is the water level at time (t) and h_2 is the water level at $t+\Delta t$. For the constant head leakage rate test, flow rate through defect of GCL is determined by using following equation:

$$Q = \frac{Q_T}{\Delta t} \quad (2)$$

where Q_T is the volume of leakage within a time interval of Δt. Since it is not easy to measure accurately thickness of GCLs during leakage rate test, it is considered that permittivity is preferable to assess hydraulic performance of GCLs (e.g. Gartung and Zanzinger, 1998). Term of permittivity refers to flux, that is the quantity of liquid permeates through an area under a certain hydraulic gradient in certain duration. In this study, to quantify the liquid flow through the healedor partially healed damage hole, the apparent permittivity (·) of the damaged hole is defined as;

$$\cdot = \frac{Q}{A\Delta h} \qquad (3)$$

where A is the defect area and Δh is head difference.

To quantify the self-healing capacity, a parameter of area healing ratio ($\sim_h$) is defined as:

$$\sim_h = \left(1 - \frac{A_f}{A_i}\right).100\% \qquad (4)$$

where A_f is the final unhealed area of the damaged hole which can be measured after a test, and A_i is the initial area of the hole.

Typical flow rate (Q)-time (t) curves

Typical leakage rate versus elapsed time curves are given in Figs and Figs for the falling head and Fig for constant head condition respectively. As shown in Fig. 6 for the falling head tests, flow rate (Q) reduced sharply in the first three days of the test and then became more or less stable. While for constant head test, the flow rate gradually reduced as shown in Fig. 7. The initial reduction of the flow rate of the falling head test is partially due to the use of the inlet water volume to calculate the flow rate i.e. the amount of the water absorbed by the bentonite in earlier period of hydration is include as part of the flow rate.

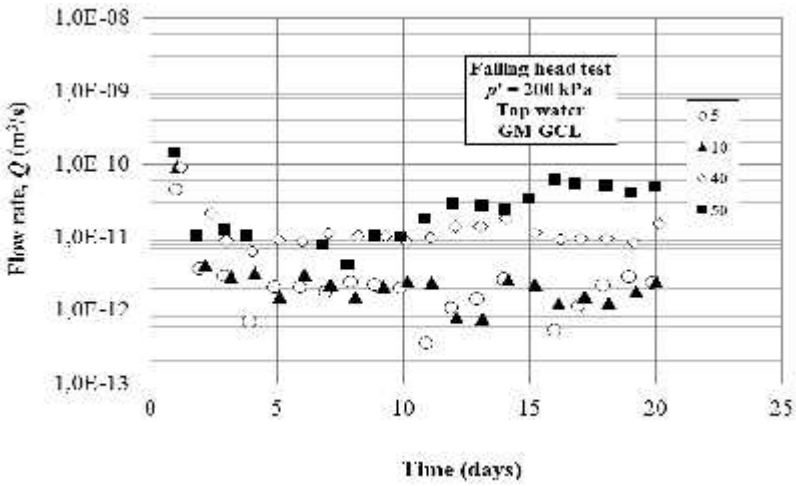


Fig. 6 Flow rate of falling head leakage rate test

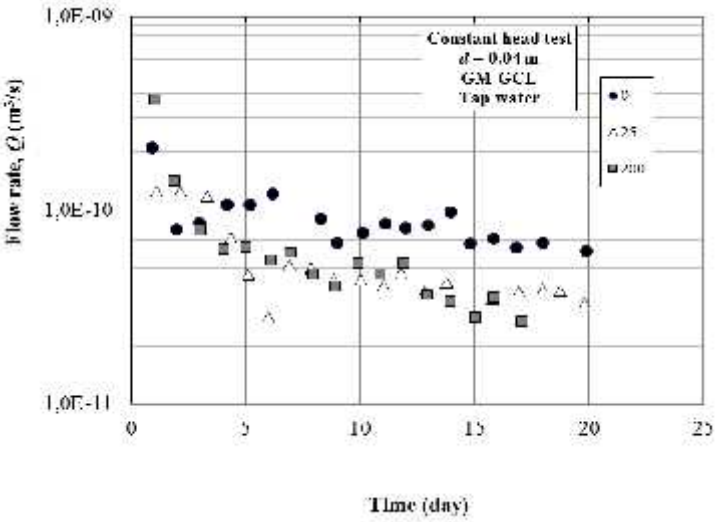


Fig. 7 Flow rate of constant head leakage rate test

Effect of the hole size

To observe effect of hole size on self-healing capacity of GCL, a serial test was conducted on GCL with the hole size (*d*) of 5-50 mm in diameter. Leakage rate test was conducted under overburden pressure of 25, 50, 100 and 200 kPa. Result of the test was shown on Fig. 8 and Fig. 9 in terms of flow rate (*Q*) and permittivity (·). Relationships between · and hole size (*d*) was depicted on Fig. 10.

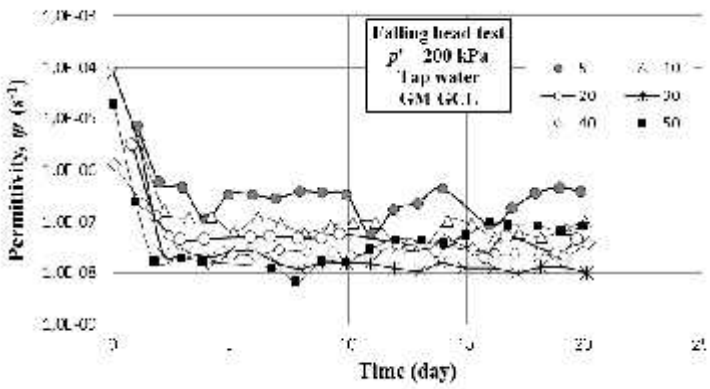


Fig. 8 Flow rate with elapsed time

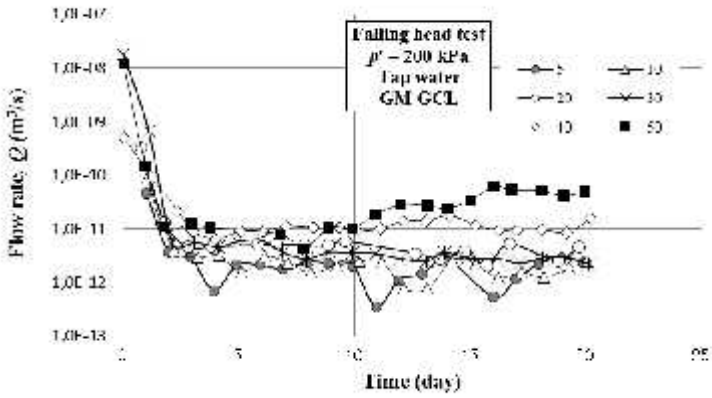


Fig. 9 Permittivity with elapsed time

From the Fig. 10, it can be seen that up to d = 20 mm, · reduced with the increase of d, for further increase of d, · reversely increased. Increasing of · with increase of d value is easy to understand because the larger d value, the higher possibility part of damage area might not be "healed" due to the expansion of the bentonite. For d less than 20 mm, · value increased with decreased of d value. When liquid enter the hole, some part of it will flow through the hole, and the other part may percolate into the surrounding bentonite or GM/bentonite interface (Chai et al 2005, Chai et al 2008). Especially after some bentonite had expanded into the hole, the lateral percolation/spreading effect will be enchanced. The percentage contribution of the lateral percolation/spreading on total flow rate is more significant for a smaller hole, and it increased apparent · value of the smaller hole.

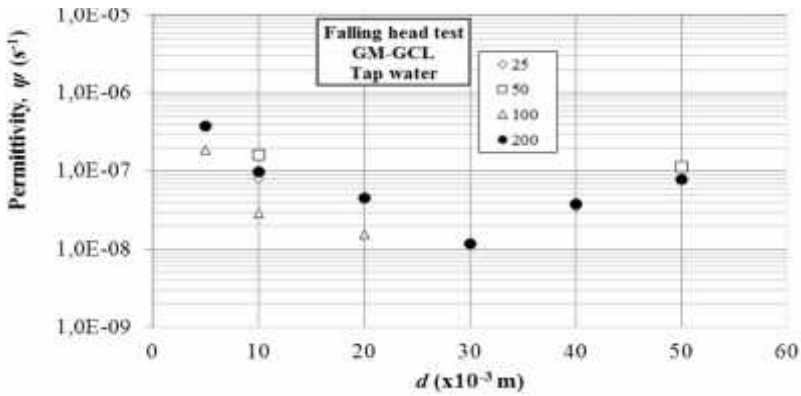


Fig. 10 Relationship between permittivity and size of hole

Effect of overburden pressure

Effect of overburden pressure (p') to self-healing capacity of GCLs was observed by conducted a serial leakage rate test under several values of p'. Comparison of permittivity (·) with elapsed time subjected by different p' is shown in Fig. 11. It shows that permittivity decreased as p' increased. There are two possible effect of p', first is squeezing effect which intends to push the hydrated bentonite into the damaged hole and increasing healing ratio, and the other is constraining effect which intend to limit the expansion of the bentonite vertically and hinder the full hydration of the bentonite. Especially when p' is larger than swelling pressure of the bentonite with a given initial density and water content. The constraining effect of p' may reduce the self-healing capacity of a GCL. Based on results in Fig. 11, it seems that the squeezing effect is more important for the condition tested. Photograph of sample after tests are shown in Fig. 13. The figures showed clearly that increasing of p' value increased the self-healing capacity (reducing of unhealed area).

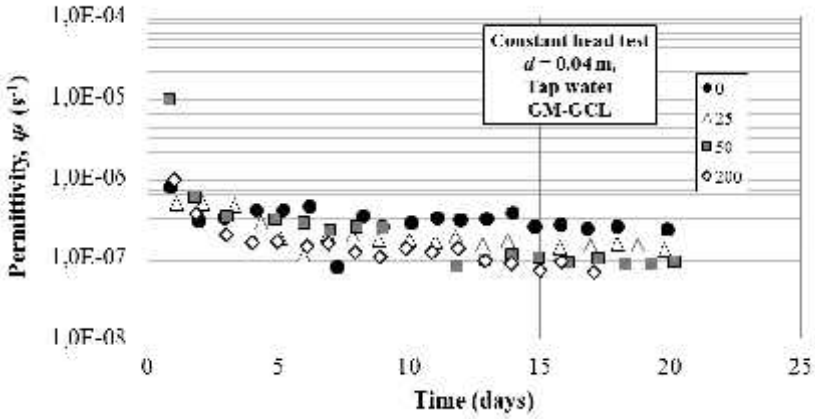


Fig. 11. Effect of overburden pressure to The Permittivity

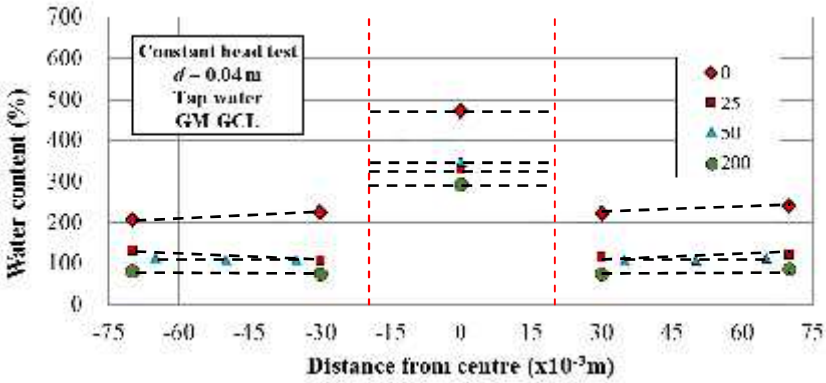


Fig. 12 Water content of bentonite after test

GT-GCL

Effect of hole-size

For the case of d = 30 mm, · value is lower than d = 40 mm and 50 mm cases. The · value of the hole of d = 30 mm case is more than 10 times of that intact GT-GCL. After the test, inspection of sample tested showed that for d = 30 mm case, the hole almost healed but for d = 40 mm and 50 mm cases, there were unhealed portion as shown in Fig.14. The figures shows that d = 50 mm cases has a larger unhealed are but the · value in Fig.15 is slightly smaller than that d = 40 mm cases, which may due to the spatial variation of GT-GCL samples tested. There were certain variations of thickness of bentonite layer in it. The apparent inconsistency for the result in Fig.15 for d = 40 mm and 50 mm cases may be due to the possible variation of the sample used.

(a) p' = 0 kPa

(b) p' = 25 kPa

(c) p' = 50 kPa

(d) p' = 200 kPa

Fig. 13 Photograph of holes after leakage rate test for d = 40 mm

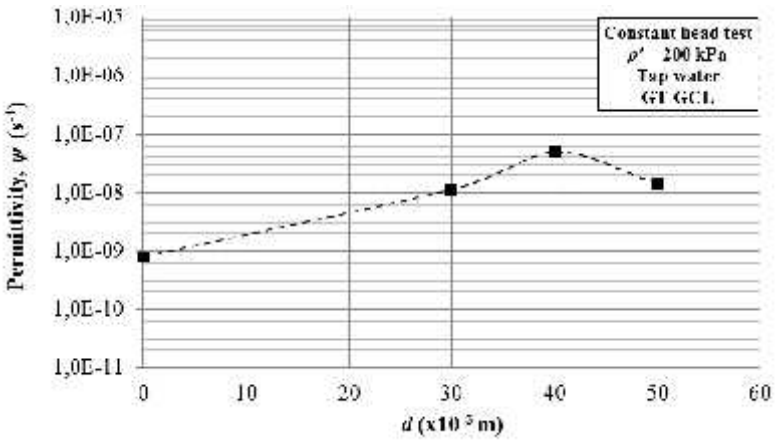
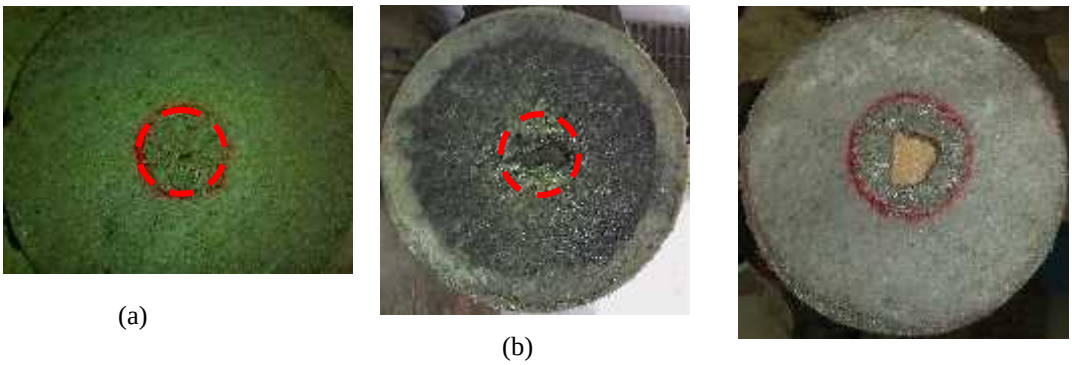


Fig. 14 versus d for GT-GCLs after constant head test



(a) (b) (c)

Fig. 15 Photo of GT-GCL samples after constant head test; (a) d = 30 mm, (b) d = 40 mm, (c) d = 50 mm

Effect of Overburden pressure

The effect of *p*' on the self-healing capacity of GT-GCL was investigated under *p*' = 0, 50, 100 and 200 kPa. Result of the tests are plotted in the Figs. 16-17. The · values tend to reduce with the increase of *p*' value . Mazzieri and Pasqualini (2000) reported that for damaged GT-GCL, its hydraulic conductivity reduced with the increase of *p*' value up to 200 kPa and remained almost constant for higher *p*' value. Although the data from this study are limited, the trend seems similar with results by Mazzieri and Pasqualini (2000).

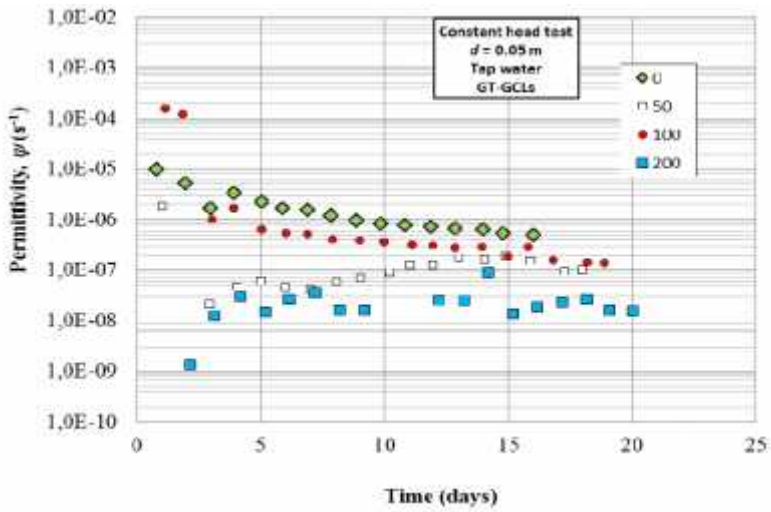
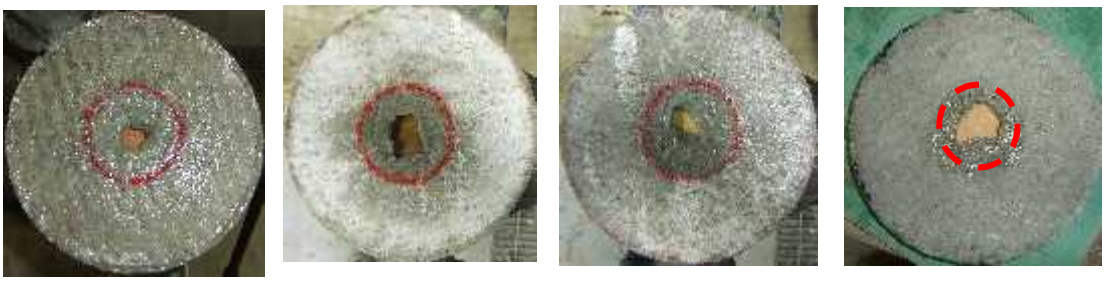


Fig. 16 Influenced of *p*' on permittivity of GT-GCLs



(a) (b) (c)

(d)

Fig. 17 Shape of the hole of GT-GCLs for d = 50 mm; (a) 0 kPa, (b) 50 kPa, (c) 100 kPa, (d) 200 kPa

CONCLUSION

From these experiments tests, the following conclusion can be drawn:

1. In the range of d = 3 -30 mm, · reduced with the increased of d value. However for d > 30 mm, · increased with the increased of d value. For d > 30 mm, possibility of part damaged area to be healed is lower, however for d = 5 – 30 mm, · reduced with the increased of d value. For d = 5 – 30 mm cases, it is considered due to the effect of lateral percolation/spreading of flow. The percentage of contribution of the lateral percolation/spreading flow on the total flow rate is more significant for a smaller hole, and it is increased apparent · value of the smaller hole. For both GM-GCL and GT-GCL samples, it is suggested that a hole up to 30 mm can be self-healed. In case liquid is a cation rich solution, a hole can be self-healed will be smaller.
2. For both GM-GCL and GT-GCL type, the · value tends to reduce with increase of p' value. Two possible effects of p' can be considered, squeezing effect which intend to push the hydrate bentonite into damage hole and increasing healing ratio. The other is constraining effect which intend to limit the expansion of the bentonite vertically and hinder the full hydration of the bentonite with a given initial density and water content. The constraining effect of p' may reduce the self-healing capacity.

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GIS GEOPROCESSING SYSTEM DEVELOPMENT FOR SEISMIC MICROZONATION ASSESSMENT: A CASE STUDY OF BENI SUEF, EGYPT

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ABSTRACT: The study of earthquakes and seismic ground motion at Beni Suef metropolitan area, Egypt is an exciting field with significant social consequences including potential catastrophic loss of life, disruption of people's lives and loss of income. This paper presents the results of a study on the spatial-temporal seismic hazard assessment in Beni Suef metropolitan area. A Geographical Information System (GIS) technology based geoprocessing system development is established to assess three-dimensional (3D) fault layers and the seismic hazard maps for specified seismic source faults. By adapting the methodology of the National Seismic Hazard Maps for Japan, GIS-based seismic hazard assessment tool is developed and applied to simulate the ground motion of the October 12, 1992 Dahshour earthquake with magnitude 5.9 and the October 11, 1999 southeast Beni Suef with magnitude: 4.9. Site amplification factor of GIS layer model has been constructed by 800-meter grid scale for part of Beni Suef metropolitan area in which digital grid data of shear velocity are available from USGS's website. Moreover, geophysical, topographical, geological data and satellite images were collected, processed, and integrated into a spatial database using GIS and image processing techniques. Because some numerical parameters of fault are not clearly recognized in the study area, sequence of assessment analysis scenario has been performed rigorously. The distribution map of peak velocity on the engineering bedrock, peak velocity on the ground surface and seismic intensity on the ground surface in the vicinity of the Beni Suef metropolitan area are simulated sequence to describe an important source parameter to affect characteristics of calculated seismic ground motions.

Keywords: *Seismic, geoprocessing, three-dimensional fault, GIS system.*

INTRODUCTION

The study area is located at the northern part of Egypt as a part of the Nile Valley (Fig. 1). The study area is mostly covered by sedimentary rocks of different formation as well characterized by flat topography. The most of populations, archaeological positions and vital structures are concentrated around the Nile valley that occupies only 1% of the total area of Egypt. Generally, Egypt buildings are not designed to resist earthquakes, therefore relatively small events can be the source of huge disasters. The earthquake of 12 October 1992 is well known example of a small but effecting earthquake in Egypt. Local site effect conditions are directly related to significant damage and loss of lives. Previous study stated that Beni Suef area has low to moderate seismicity and also may be a site of future event. So, it is importantly to study local site conditions. At the past times, the effects of local site conditions have caused disasters during several earthquakes. Some well known examples comprise the earthquake in Michoacan, Mexico in 1985 (Kobayashi et al., 1986), Loma Prieta, California, USA in 1989 (Hough et al., 1990), Kocaeli and Düzce, Turkey in 1999 (Rathje et al., 2003), Chi-Chi, Taiwan in 1999 (Rathje et al., 2005) and Wenchuan, China in 2008 (Wen et al., 2010). Several well-known examples of destructive earthquakes all over the world during the last three decades (e.g., Mexico 1985, Loma Prieta 1989, Cairo 1992, Kobe 1995, Turkey 1999, and Algeria 2003) have clearly shown that the local site conditions are an important factor in determining the seismic hazard specific to a given site. This is particularly important for the area of study where the soil conditions vary significantly from one place to another and the impedance contrast between the soft sediments of Nile alluvium and the underlying bedrock is significant.

Study of the geomorphology, geology, and tectonic setting of the study area is very important for the present work as they give an idea about the most susceptible sites for liquefaction, ground motion and amplification. The trend analysis of the surface and subsurface faults deduced from geological and geophysical data around the source area (Abou Elenean and Deif, 2003) reflects the existence of two dominant trends N60E and N45W. Field geology studies to the northwest of source area (20 km) assures the existence of a dominant normal faulting structure trending N50–65W and dipping towards NE and SW with a length ranging from 8–10 km (Hassan et al., 1978). Most of the earthquakes mainly occur in the northern part of the Egyptian territory and are related to the plate boundaries (Badawy, 1996, 1998;

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Badawy and Horvath, 1999b, c). However, the relative motion of the Sinai sub-plate with respect to the African plate (Suez rift) and to the Arabian plate (Aqaba-Dead Sea rift) represents the main source of earthquake activity in northern Egypt.

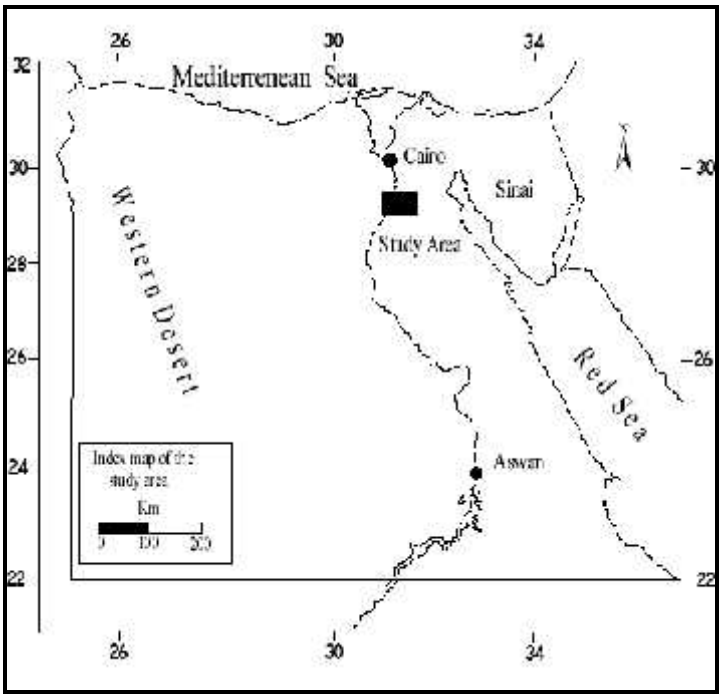


Fig.1 Location map of the study area

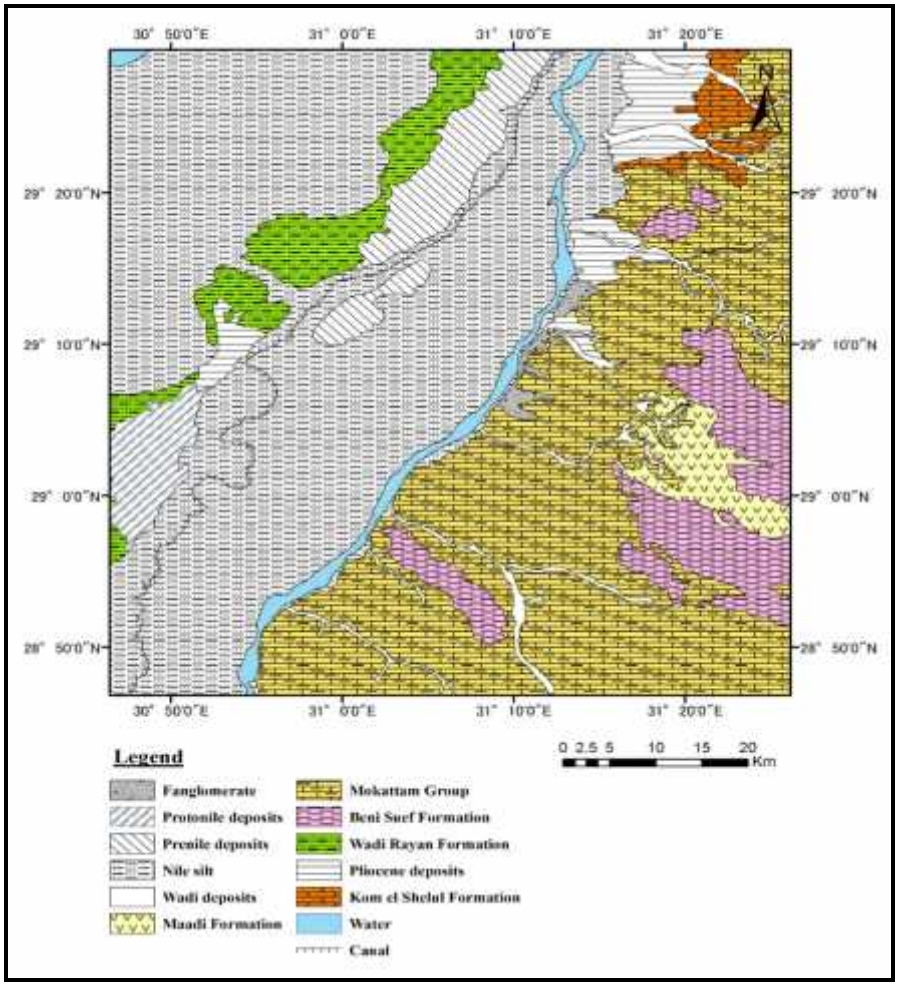


Fig. 2 Geological map of the study area (after the Egyptian General Petroleum Corporation, 1987)

GEOLOGICAL AND TECTONIC SETTING

A geologic map of the study area was simplified from Egyptian General Petroleum Corporation (1987), scale 1:500,000, (Fig. 2) in which the different rock units are represented. The area of Beni Suef is covered by sedimentary rocks of different formations belonging to the middle Eocene up to the Quaternary. The area under consideration represents a part from the Nile valley zone of Egypt. Geologically the area has been studied by a variety of authors (e.g. Zittel, 1883; Schweinfurth, 1883; Hume, 1911; Said, 1962; Said and Martin, 1964). The geology of the studied area was influenced by the history of the Nile Valley and the remnants of old settlement.

Generally, the geological formations in the studied area are composed of Mokattam Group, Wadi Rayan Formation, shallow marine limestone with Nummulities gizehensis, repeated intercalated by shale and sandy shale. Includes Midawara Sath el-Hadid and Gehannan (Ravine) Formation. Mokattam Group, Beni Suef Formation, Marine shale, marl and limestone in Beni suef area, grading northward into Giushi Formation. Mokattam Group, predominately consisting of Observatory Formation (shallow marine dense medium bedded limestone with local chert and few Nummulities); with Giuishi Formation above and Gebel Hof Formation(rich in Nummuities gizehesis) below. Toward Gebel Ataqa grading into Ramiya Suez Formations, Maadi Formation, Littoral and shallow marine shale and limestone with Carolia placunoids. Locally intercalated by shallow marine sandstone. Coeval with Qasr el-Sagha Formation. Qasr el-Sagha Formation. Littoral marine to continental clastic sequence with oyster beds, Carolia placunoides and coquina layers; intercalations of silt and claystones. In lower parts Nummulities striatus and N.fabiani abound in fossiliferous calcarenite. Includes Birket Qarun Formation (Western Desert). Umm Raqaba Formation. Yellow marine fossiliferous sandstone with shale stringers alternating with conglomerates of limestone, chert and quartz pebbles (East of Beni Suef area). Kom el-Shelul Formation. Sandstone beds and coquinal limestone, clayms, strombus (Pyramids area). Pliocene deposits, undifferentiated (North of Faiyum possibly younger). Sediments along the Nile River are classified into three types. The Proto-Nile is a Pliocene to Pleistocene Formation which consists mainly of gravel. The Pre-Nile Formation consists of intercalated beds of fluvial sand and Aeolian sand. The Neo-Nile and the Nile-silt consist of fine clastic sediments. The Proto-Nile, the Pre-Nile and Neo-Nile form terraces along the Nile River. The Nile-silt blankets the alluvial lowland along the river. Along its geological history, Egypt subjected to a number of stages since the Precambrian.

The Precambrian characterized by the development of three main fault directions, N-S, NNW-SSE and NE-SW. In the Paleozoic, a new E-W trend developed and continued to affect the Mesozoic. The Mesozoic folding structures known in the northern parts of Egypt and Sinai were also affected by a new fault trend ENE and WNW directions. The beginning of the tertiary up to the Quaternary, the E-W trend became reactivated. The Red sea became effective in the NW-SE Gulf of Suez trend and NE-SW Aqaba trend. The present day tectonic setting of Egypt should be regarded as one unit with its surroundings. And due to its location in the north east corner of the African plate, Egypt is in contact with three plate boundaries, the African-Eurasian plate margin, the Red sea plate margin and the Levant transform fault (Abou Elenean and Hussein, 2008). Based on analyzing the lineaments extracted from geological map of EGPC (1987) in the present study, it was observed that the prevailing trends are dominantly in the NE-SW and NW-SE (Fig. 3a). Also, the constructed density map of extracted lineaments (Fig. 3b) indicated that the primary maxima of lineaments are located in the northeastern region, whereas the secondary maxima are located in the southwestern part of the study area. An ETM+ VNIR color composite was overlaid on shaded relief maps constructed from the DEM data to increase the number of geological lineaments identified and clarify the topography in the study area (Fig. 4a). Rose diagram extracted from shaded relief map (Fig. 4b) reveals that the dominant trend of lineaments tend to run NW-SE and NE-SW and this is in correspondence with the previously obtained trends deduced from geological map and density map.

Historical seismicity around the study area

Spatial distribution of recent earthquake foci indicates that Egypt has suffered from both interplate and intraplate events. Most earthquake activities have been concentrated in northern Egypt along the Sinai sub-plate borders (Badawy,1996; Badawy and Horvath, 1999a,b). Away from this relatively active seismic zone, inland seismic activity (e.g. Dahshour, Aswan, Abo-Dabbab, and Abo-Simple) has also occurred. In addition to new seismically active dislocations have been discovered, for example, southeast Beni Suef and West Sohag by installation of the modern the Egyptian National Seismological Network (ENSN). The Egyptian earthquake catalog compiled by (Badawy, 2005a) lists only five moderate earthquakes at Beni Suef dislocation pre- the Egyptian National Seismological Network (ENSN) (1900–1997). However, nine small events ($3.0 \leq ML < 5.0$) and 170 micro earthquakes ($ML < 3.0$) are listed from 1997 to 2007. Generally, the southeast Beni Suef has low to moderate seismicity. The southeast Beni Suef dislocation lies approximately 45 km SE of Beni Suef. Microearthquakes are concentrated at wadi Araba depression and markedly decreased to the southwest. Another microearthquake cluster is also observed at the rampant arch between S. Galala and Ma'azah plateaus. However, the nine small-magnitude earthquakes are lie in 50 km radius circle around the October, 1999 earthquake's epicenter.

Recently, on October 11, 1999 and November 08, 2006 two felt earthquakes occurred to the southeast of Beni Suef and Cairo cities. The estimated average local magnitudes from the Egyptian National Seismic Network (ENSN) were 4.9 and 4.3 for the 1999 and 2006 events, respectively. The first event of 1999 was the largest observed earthquake to occur in this area. It was followed by eight aftershocks (ML (Local Magnitude) 1.6–3.2). Four aftershocks ($2.6 \leq ML$

≤1.8) followed the main-event with the last one occurring November 22, 2006. Indeed, it is the second felt quake from the same area since the operation of the modern digital Egyptian National Seismological Network (ENSN) in 1997. The maximum observed intensity of this event was VI on MM (Moment Magnitude) scale at the epicenter, IV at Beni Suef and III at Cairo (Abou Elenean and Deif, 2003). The shaking was strong enough to encourage some residents to leave their houses. The 2006 event was slightly felt at Cairo compared to the first one and had MM intensity around II-III.

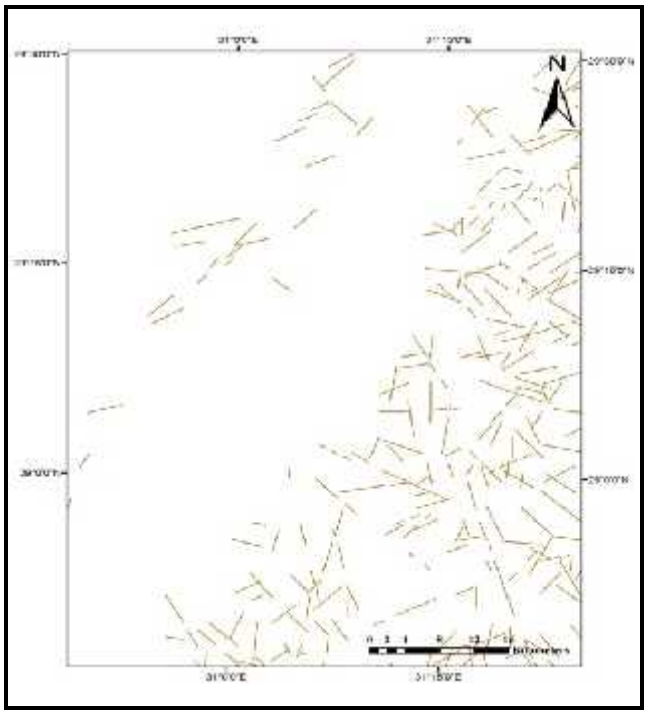


Fig. 3(a) Lineaments extracted from geological map

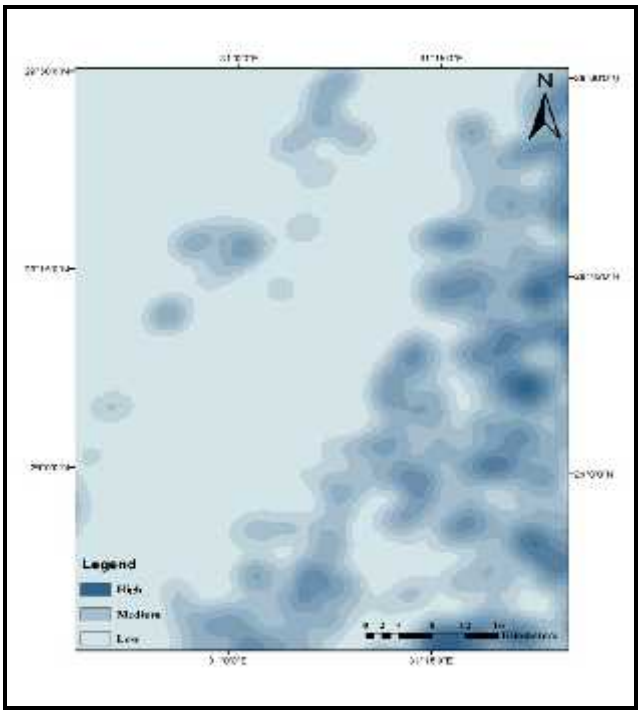
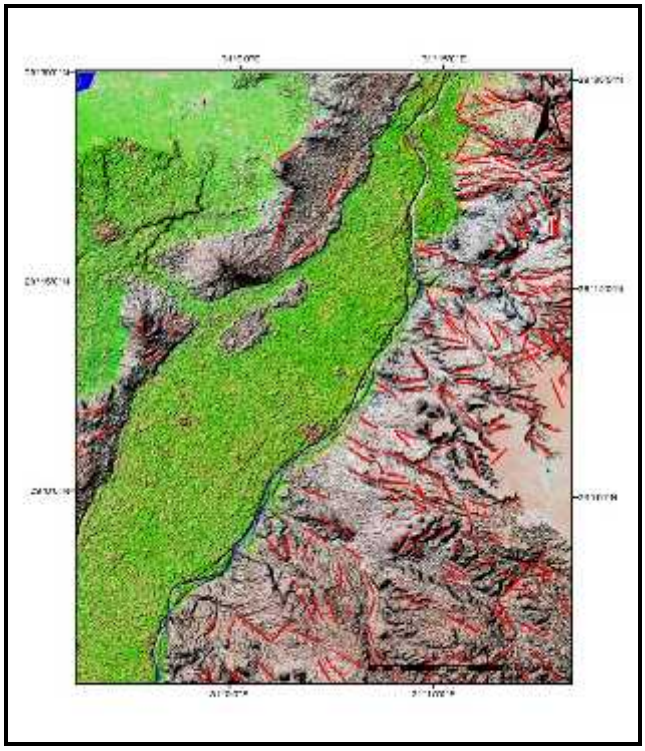
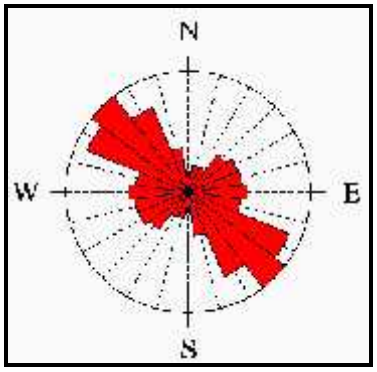


Fig. 3(b) Density map of the extracted lineaments



(a)



(b)

Fig. 4(a) Colored Shaded relief map. Solid red lines are lineaments extracted from SRTM data; 4(b) Rose diagram illustrating the lineament trends extracted from shaded relief map

GIS GEOPROCESSING SYSTEM DEVELOPMENT

Seismic hazard assessment model

The strong ground motion prediction plays an important role in prevention of earthquake disasters. Seismic hazard is a broad expression used in a general sense to refer to the potentially damaging phenomena associated with earthquakes, such as ground shaking, liquefaction, landslides, and tsunami. In the specific sense, seismic hazard is the likelihood, or probability, of experiencing a specified intensity of any damaging phenomenon at a particular site, or over a region, in some period of interest. The seismic hazard analysis refers to the estimation of some measure of the strong earthquake ground motion expected to occur at a selected site. This is necessary for the purpose of evolving earthquake resistant design of a new structure or for estimating the safety of an existing structure of importance, like dams, nuclear power plants, long-span bridges, high-rise buildings, etc. at that site.

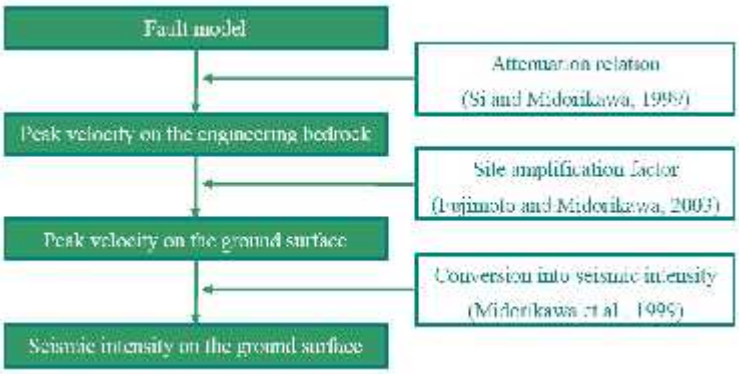


Fig. 5 Flow-chart for seismic hazard assessment

Preparation procedures of the seismic hazard assessment for specific seismic source faults are described in Fig. 5. According to flow chart, it has been described that calculate the peak velocity on the engineering bedrock by attenuation relation (Si and Midorikawa, 1999). In addition, calculate the peak velocity on the ground surface by multiplying the site amplification factor and the peak velocity on the engineering bedrock. Moreover, convert the peak velocity to the seismic intensity using the empirical formula.

Peak velocity on the engineering bedrock

Amplification of ground motion from engineering bedrock to ground surface is evaluated by a empirical relationship proposed by Midorikawa et al.(1994), described as Eqn. 1.

$$\text{Log ARV}=1.83-0.66 \log \text{AVS} \qquad (100<\text{AVS}<1500) \tag{1}$$

ARV : the factor of peak velocity amplification
AVS : the average S-wave velocity for top thirty meters in (m/s)

Peak velocity on the ground surface

The attenuation relation of peak velocity on the bedrock with S-wave velocity (600 m/s) is calculated by (Si and Midorikawa, 1999). Their equation is described in Eqn. 2.

$$\log PGV_{b600}=0.58M_m + 0.0038D + d - 1.29 - \log(X + 0.0028 \cdot 10^{0.5Mm}) - 0.002X \tag{2}$$

PGV_{b600} : peak velocity on the bedrock with average S-wave velocity (600m/s), in (cm/s)
M_m : moment magnitude X : shortest distance from fault (km)
D : hypocenter depth (km) d : fault types

By multiplying Eqn. 1 and Eqn. 2, peak velocity on the ground surface (PGV) is described, as Eqn. 3.

$$PGV = ARV \times PGV_{b600} \tag{3}$$

Seismic intensity on the ground surface

Seismic intensity is calculated using the relation between the intensity and the peak velocity (Midorikawa et al., 1999) by the following equation.

$$I = 2.68+1.72 \log PGV \qquad (4 \leq I \leq 7) \tag{4}$$

I : seismic intensity PGV : peak velocity on the ground surface (cm/s)

Shear wave velocities

The average shear wave velocity for the top 30 m (Vs30) is one of the simple predictors of earthquake ground motion amplification. One simple way of accounting for site conditions in calculating seismic hazards is to use the shear-wave velocity (Vs30). We used the average shear-wave velocity to a depth of 30 m (Vs30) to develop site categories for Beni Suef area. This parameter can further be used for modifying a calculated ground motion to account for site conditions. The shear wave velocity of shallow sediments is very important in seismic wave amplification and thus Vs30 is a well-known parameter for site classification. It has been recognized that soft and young sediments covering firm bedrock can amplify seismic waves and cause severe damage during a large earthquake. Anderson et al., (1986) noted that the strata in the top 30 m have a considerable influence on the character of the ground motions created. The Vs30-based National Earthquake Hazards Reduction Program (NEHRP) Provision classes are shown in Table 1.

Table 1 NEHRP site definition in terms of Vs30 and simple geological descriptions

NEHRP	Description	Range of Vs30 (m/s)
A	Hard rock	Vs30 >1500
B	Firm to hard rock	1500 ≥ Vs30 >760
C	Dense soil and soft rock	760 ≥ Vs30> 360
D	Stiff soil	360 ≥ Vs30 > 180
E	Soft soil	180 ≥ Vs30

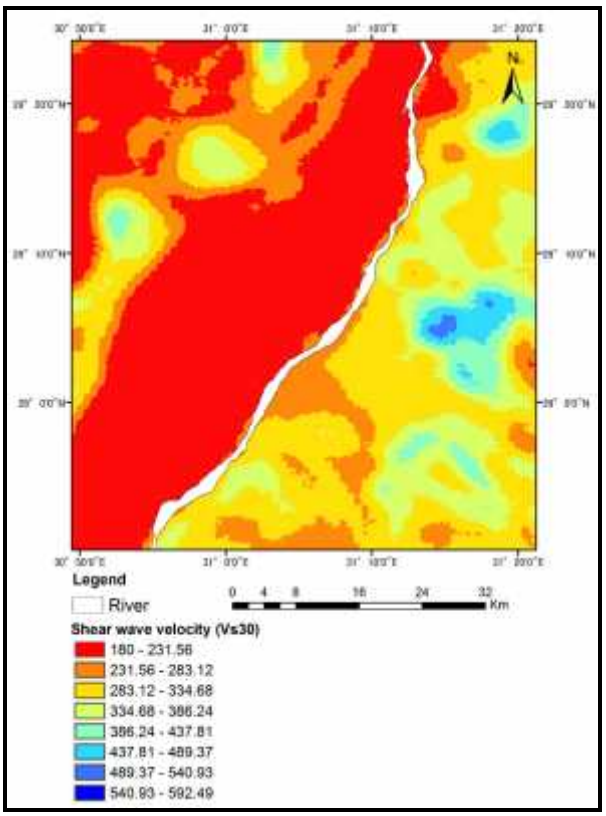


Fig. 6(a) Vs30 map for the study area from USGS

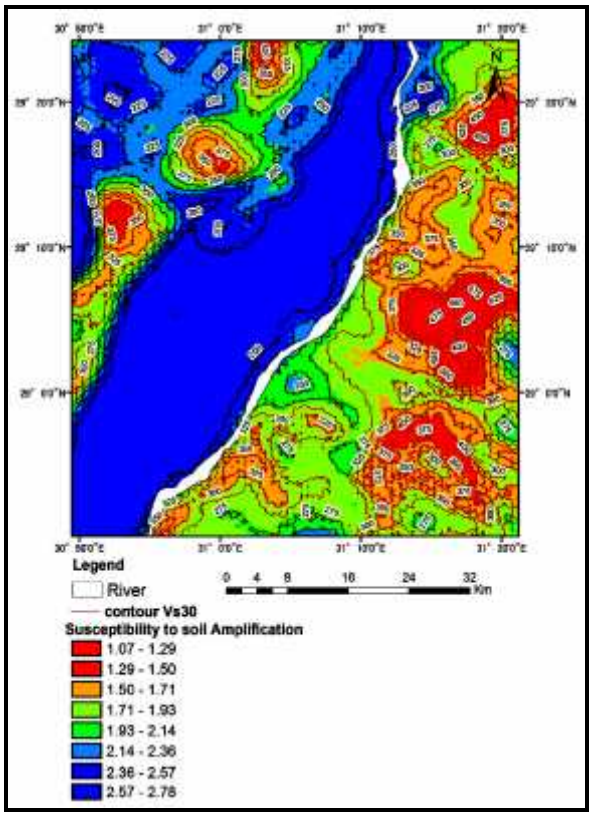


Fig. 6(b) Contour line with the soil amplification susceptibility

Wald and Allen, 2007 fully describe a methodology for deriving maps of seismic site conditions using topographic slope as a proxy. Vs30 measurements (the average shear-velocity down to 30 m) are correlated against topographic slope to develop two sets of coefficients for deriving Vs30: one for active tectonic regions that possess dynamic topographic relief, and one for stable continental regions where changes in topography are more subdued. These coefficients have been applied to continental U.S. by Wald and Allen and in other regions around the world by Allen and Wald (USGS Open File Report, 2007, in review). They also compared topographic slope-based Vs30 maps to existing site condition maps based on geology and observed Vs30 measurements, where they were available, and found favorable results. Having a first-order assessment of seismic site conditions anywhere in the world provides a valuable tool in the prediction of ground-motions following a global earthquake, the primary motivation for this research. These Vs30 maps will enable us to better quantify possible ground-shaking and rapidly deliver these predictions to emergency managers and responders. However, the Vs30 maps for the Globe will also have practical applications for numerous

related probabilistic- and scenario-based studies; hence this Web delivery service allows users to download maps and grids of seismic site conditions for specified regions. The Global Vs30 Server allows a user to select from a map or input a rectangular region of interest. It then provides (optionally) a Vs30 grid in ASCII or GMT grid format, and a JPEG Vs30 map. Earthquake dataset: compiled from the USGS Earthquake Hazard Program (http://earthquake.usgs.gov/hazards/apps/vs30/custom.php) consisting of shear wave velocity value Vs30. The Vs30 maps and grid provided herein are based a simplified approach and should not be considered definitive for any specific location or region. The maps and grids are provided for general purpose use, and are not intended to supplant existing, detailed Vs30 maps or measurements. Wald and Allen (2007) note significant limitations to this simplified approach. Users should be aware of these limitations and should exercise caution in using this approach for anything other than regional scale Vs30-based site amplification estimates. As always, site-specific Vs30 values should be used at finer scales or at particular locations. In this regard, shear-wave velocity (Vs30) (Fig. 6a) is an important input parameter in the site response analyses. Generally, seismic wave velocity depends on the nature and physical state of the materials. Fabric, aging, and cementation of the soil affect (Vs30) remarkably. Borcherdt et al. (1991) showed a correlation between amplification of ground motions and average shear-wave velocity of the upper 30 m of the earth (Vs30). Furthermore, Fujimoto and Midorikawa (2006) suggested the relation between Vs30 and amplification factor of peak ground velocity (ampv) as follows.

$$\log(\text{ampv}) = 2.367 - 0.852.\log(\text{Vs30}) \quad (5)$$

Fig. 6(b) shows qualitative estimation for the soil susceptibility to amplification. Therefore, the areas where the soils consist of soft sediments like loose sands and silty clay are more susceptible to amplification. It is obvious that lower shear wave velocities up to 400 m/s correspond quite well with the outcrop of unconsolidated, Quaternary sediments in the study area. Merging susceptibility map with the Vs-data, there is a coincidence of areas assumed to be more susceptible to soil amplification according to the overlay approach with areas of estimated lower shear wave velocities (VS<250) (Fig. 6(b)). The ground motion amplification due to local soil conditions depends on soil type, properties and thickness. (Seed and Idriss 1982), mentioned that most soils amplify peak acceleration where the firm ground acceleration is less than 0.1g, where soft to medium stiff clay and sand causing the largest amplification. (Idriss, 1990), enhanced the amplification curve for soft clay sites, based on extra analysis and data from the Mexico City and Loma Prieta actions and clarified that soft clay sites amplify peak accelerations up to 0.4g, beyond which “de-amplification” occurs as a result of the nonlinear behavior of soil under large strains.

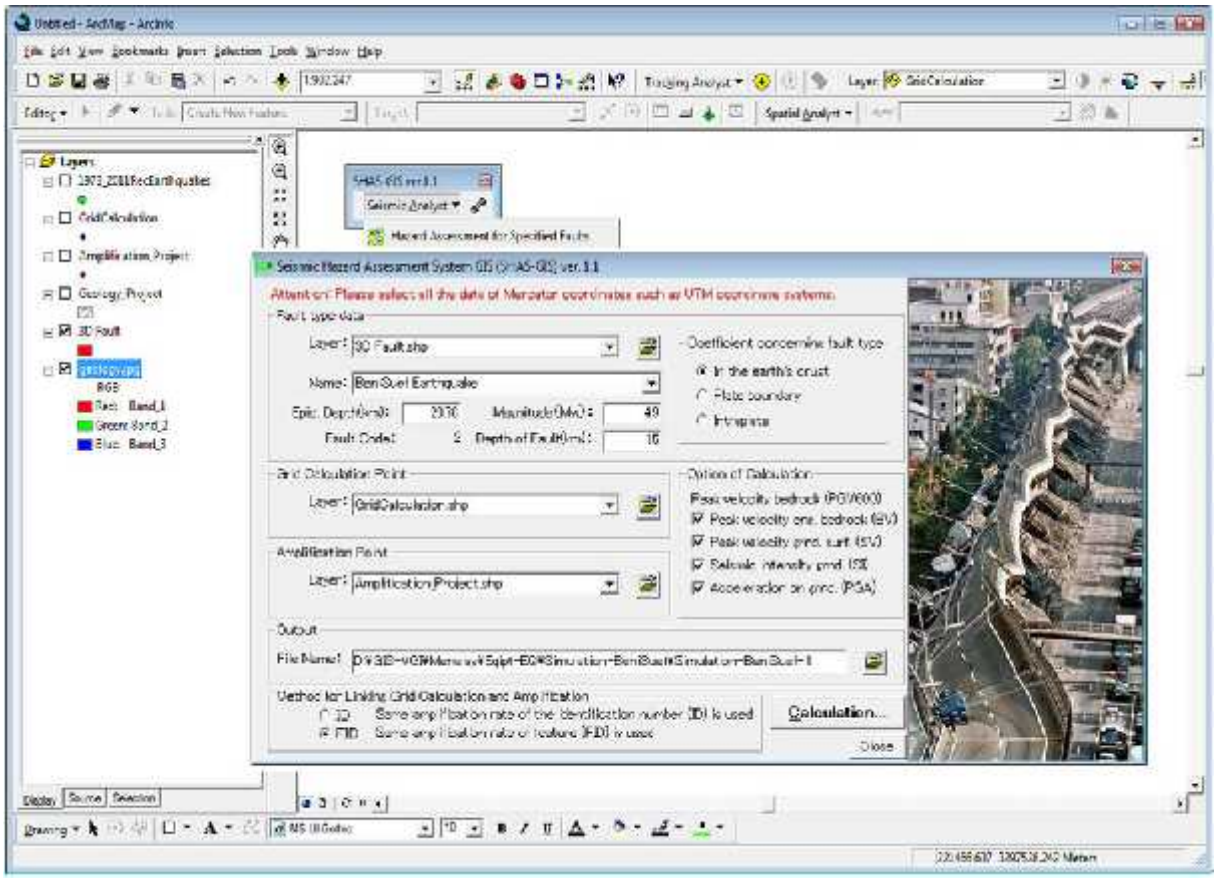


Fig. 7 Interface development of Seismic Hazard Assessment System GIS (SHAS-GIS)

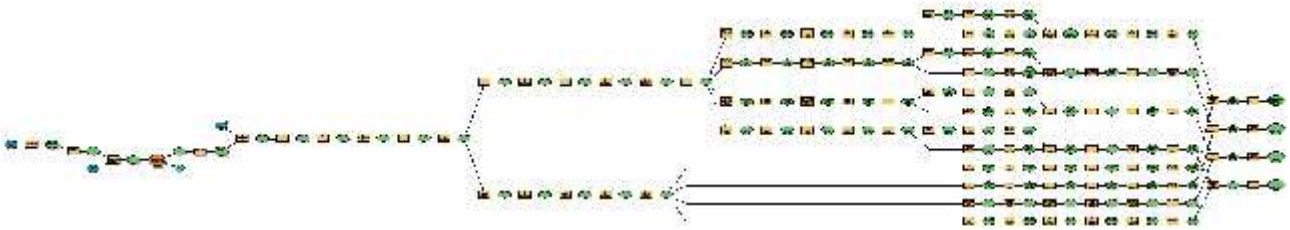


Fig. 8 Schematic flow for the development of 3D fault model by GIS geoprocessing

GIS integrated system tool

A geographic information system (GIS) is a system designed to capture, store, manipulate, analyze, manage, and present all types of spatial or geographical data. In addition to the integration of time in environmental studies, GIS is also being explored for its ability to three-dimensional (3D) model the progress of ground phenomena throughout sequences process (Djamaluddin et al., 2012).

By adapting the methodology of the National Seismic Hazard Maps for Japan, seismic hazard assessment system using GIS technology (called by SHAS-GIS) has been developed and applied to simulate the seismic hazards map of the study area (Fig. 7). In addition, the input GIS parameters for seismic hazard assessment are point grid data as an amplification factor and 3D polygon as a geometrical fault model. The 3D polygon was constructed automatically in GIS by using geoprocessing model builder (Fig. 8). Geoprocessing model builder in GIS provides functionality to make simple or complex models even more productive through the use of a number of advanced techniques. These advanced techniques improve models by adding more control to the model's logical flow, better mechanisms for managing model data, and time-saving methods for performing repetitive tasks automatically. Aim of using a geprocessing model builder is to convert fault line parameters into 3D polygon to use in the simulation model.

3D polygon fault model and grid amplification point data are used to calculate seismic hazard assessment maps. A GIS-based seismic hazard assessment tool (Fig. 7) has been applied to simulate the spatial ground motion of the October 12, 1992 Dahshour earthquake with magnitude 5.9 and the October 11, 1999 southeast Beni Suef with magnitude: 4.9 and maximum expected parameters for earthquake source with magnitude 6.5. 3D polygon was constructed in GIS by using a geometrical data from previous observation study. For the 3D fault model, with reference to the vector-based polygon, the spatial data of geometry, fault parameter, magnitude can be obtained from the 3D polygon. In this polygon fault dataset, a feature table of the 3D polygon is used to relate the depth to epicenter parameters. In the 3D polygon attribute table, ‘Code’ is the fault of 3D polygon attributes, and ‘M’ is the magnitude. ‘Dip’ is the dip inclination of the fault layer, ‘UppEdgekm’ is the distance to upper edge, and ‘Widthkm’ is the fault width. Each 3D polygon has values for fault depth vertices that give spatial geometry in x, y, and z. Therefore, a 3D polygon has spatial geometry that can be used to identify the main strike direction and dip inclination of each fault. 3D fault’s shape and attribute data in the 3D polygon GIS gives significant result for the calculation of seismic hazard in which the GIS data can be proceed automatically and faster in the GIS integrated system development. In addition, the service provided by the USGS providing Vs30 data from all over the earth’s surface based on these correlations ((Wald and Allen, 2007) USGS Seismic Hazard Program, Global Vs30 Map Server) was used to investigate amplification where this amplification values converted to grid point model.

Furthermore, the peak velocity on the engineering bedrock, the peak velocity on the ground surface and the seismic intensity on the ground surface are calculated rigorously. These maps are prepared by calculating the peak velocity using the attenuation relation with magnitude and the shortest distance to the fault. Moreover, the overview of the distribution of strength of shaking can be obtained in some areas based on the distance from the fault and the site amplification.

Study case for seismic hazard maps for specified seismic source faults

a) Case 1: Observed The October 11, 1999 Beni Suef earthquake source with magnitude 4.9, depth of epicenter 26km, fault depth to upper edge 15km, length of fault 5km, width of fault 12km and fault orientation (strike 153° and dip 72°). Results of the simulations are shown in Fig. 9(a, b, c, d).

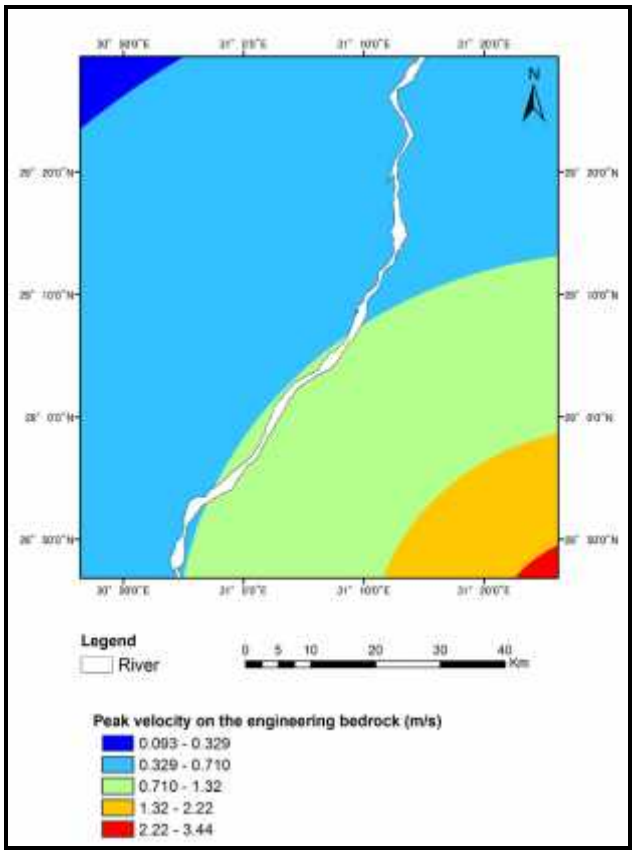


Fig.9 (a) Peak velocity on the engineering bedrock

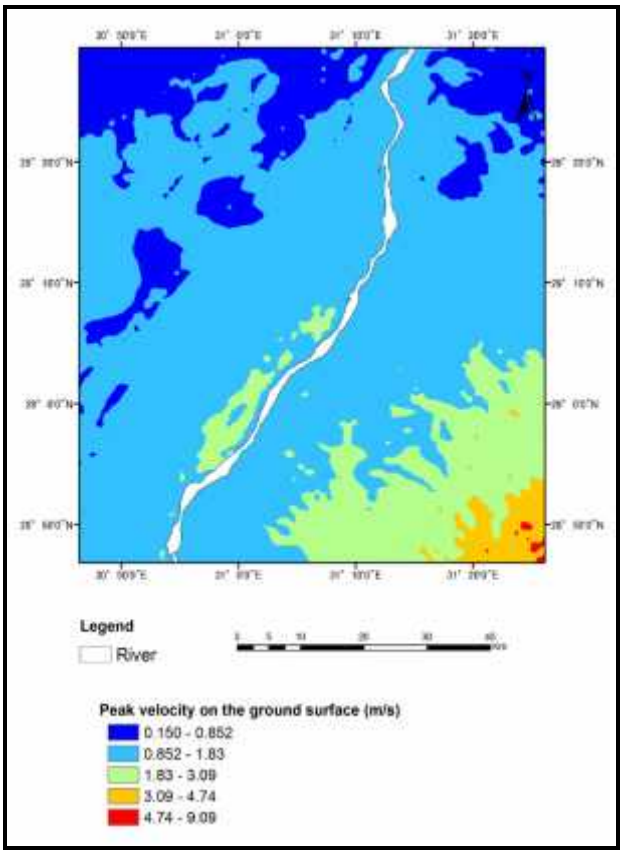


Fig.9 (b) Peak velocity on the ground surface

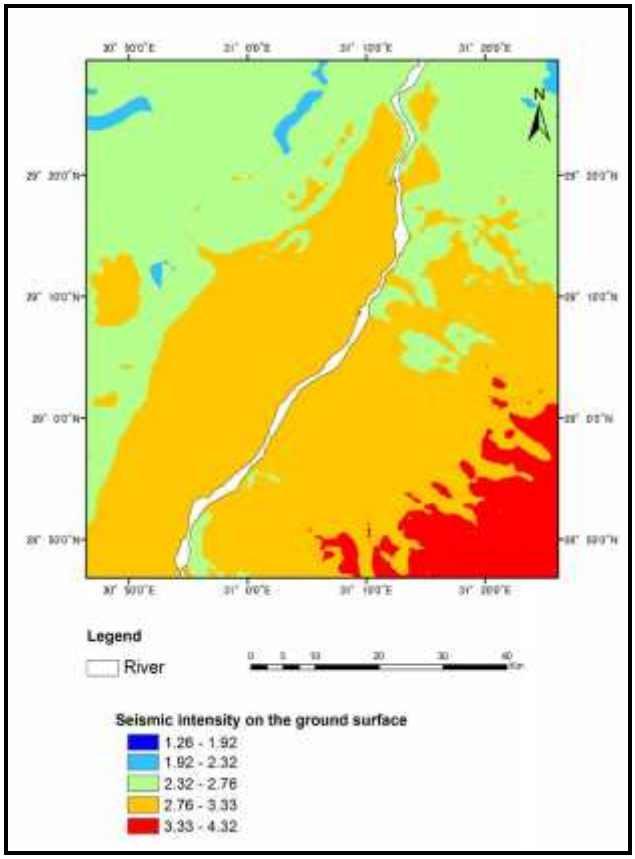


Fig.9 (c) Seismic intensity on the ground surface

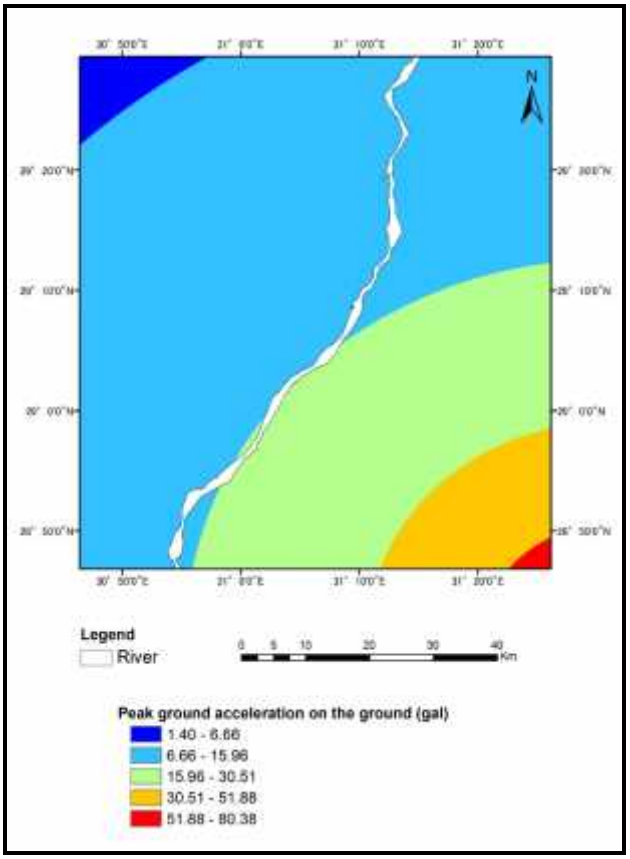


Fig.9 (d) Peak ground acceleration on the ground

b) Case 2: Observed The October 12, 1992 Dahshour earthquake source with magnitude 5.9, depth of epicenter 22km, fault depth to upper edge 12km, length of fault 12km, width of fault 10km and fault orientation (strike 270° and dip 47°). Results of the simulations are shown in Fig. 10(a, b, c, d).

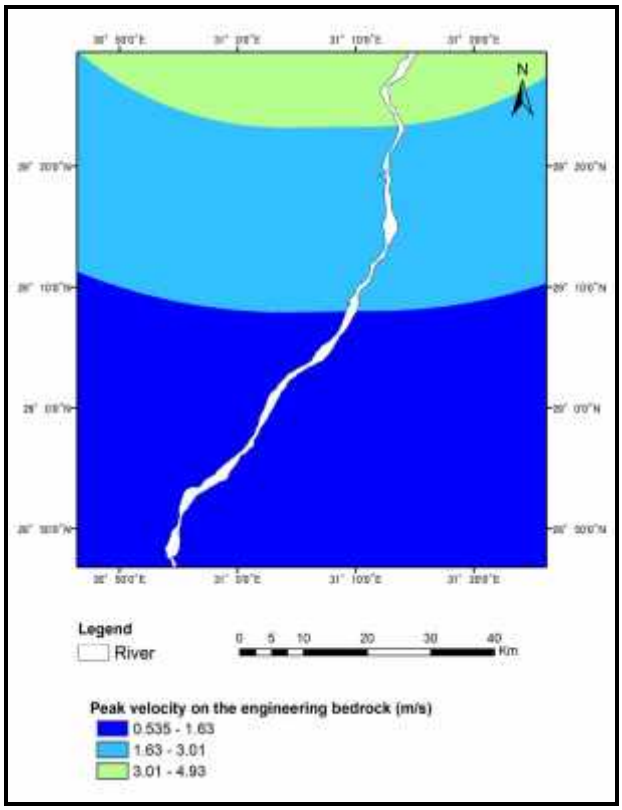


Fig.10 (a) Peak velocity on the engineering bedrock

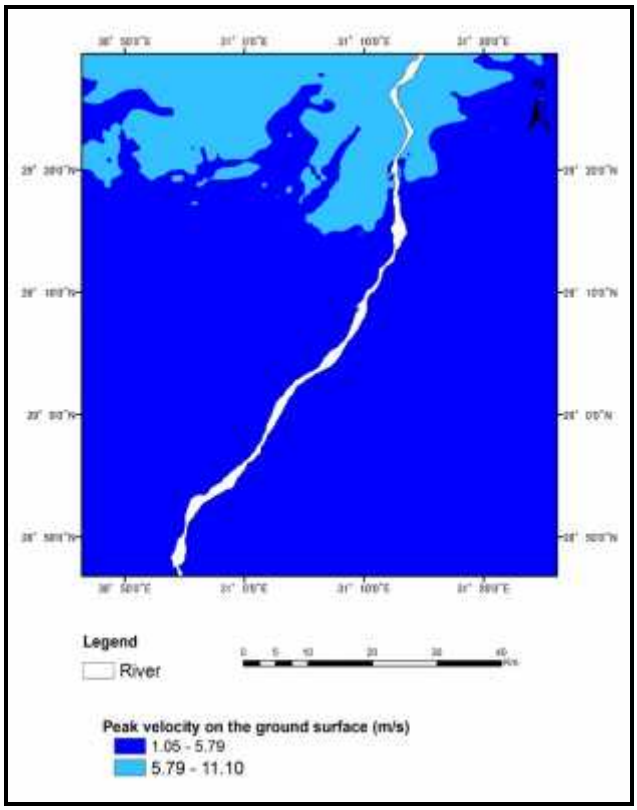


Fig.10 (b) Peak velocity on the ground surface

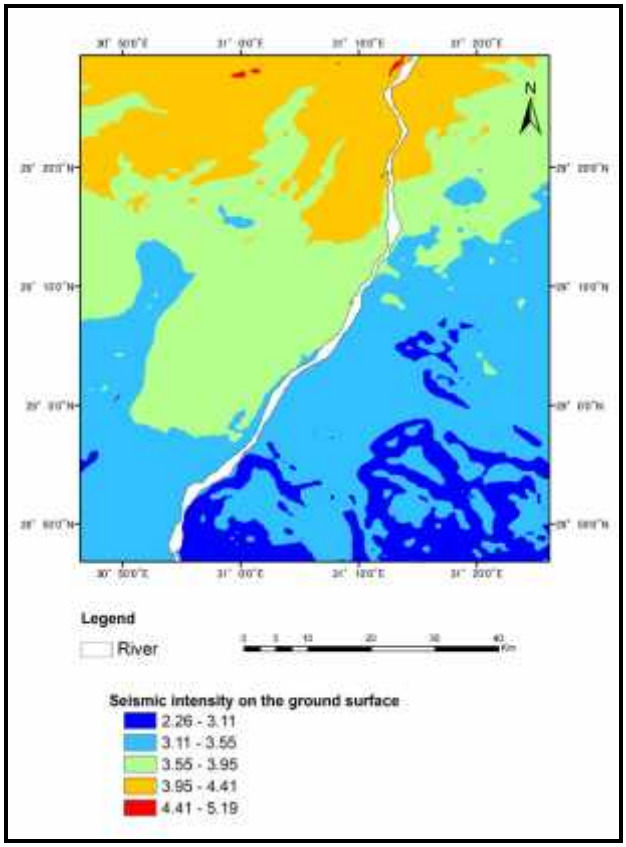


Fig.10 (c) Seismic intensity on the ground surface

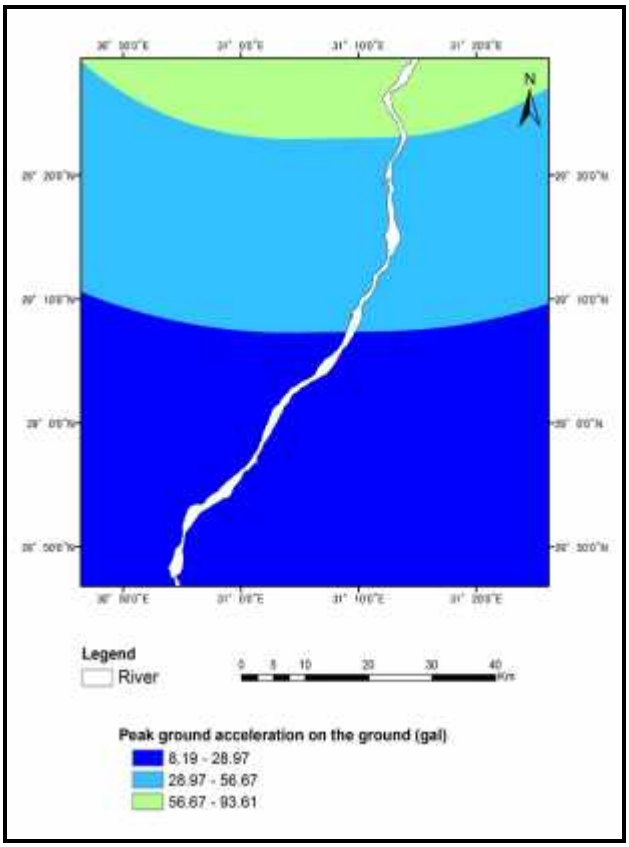


Fig.10 (d) Peak ground acceleration on the ground

c) Case 3: Parameters from maximum expected earthquake source with magnitude 6.5 in Dahshour area, depth of epicenter 22km, fault depth to upper edge 12km, length of fault 12km, width of fault 10km, and fault orientation (strike 270° and dip 47°). Results of the simulations are shown in Fig. 11(a, b, c, d).

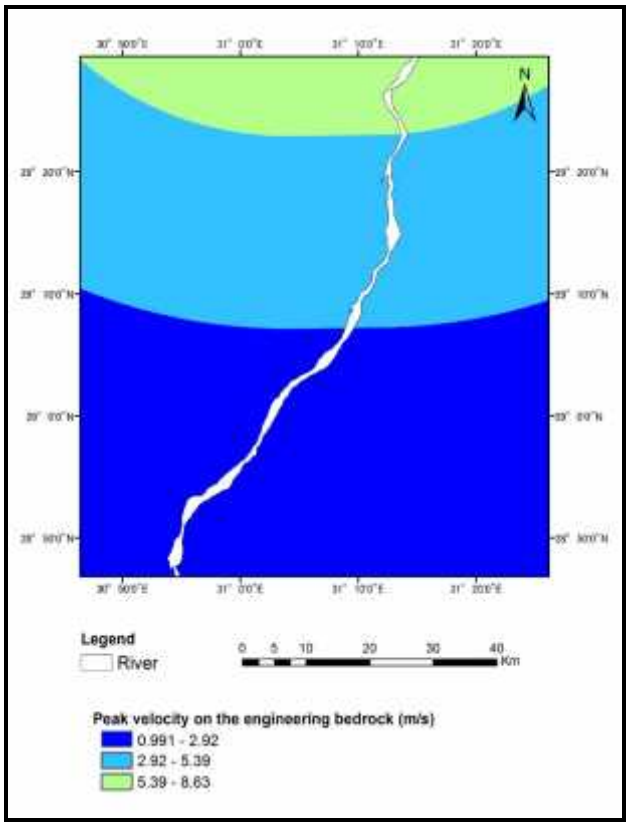


Fig.11 (a) Peak velocity on the engineering bedrock

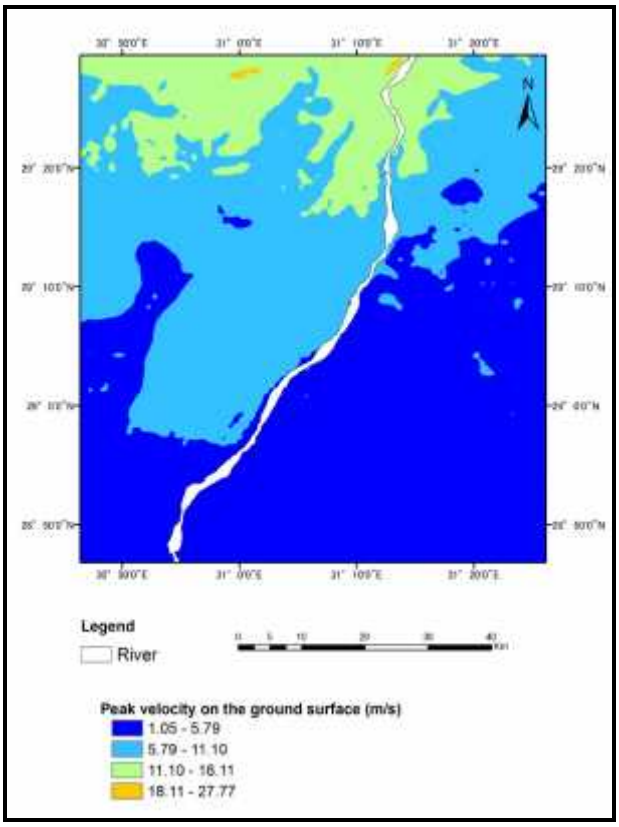


Fig.11 (b) Peak velocity on the ground surface

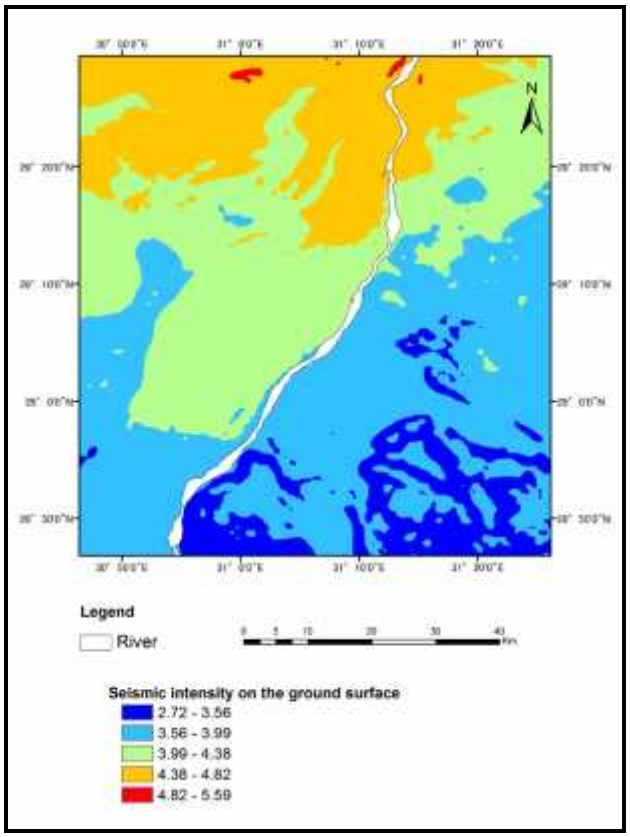


Fig.11 (c) Seismic intensity on the ground surface

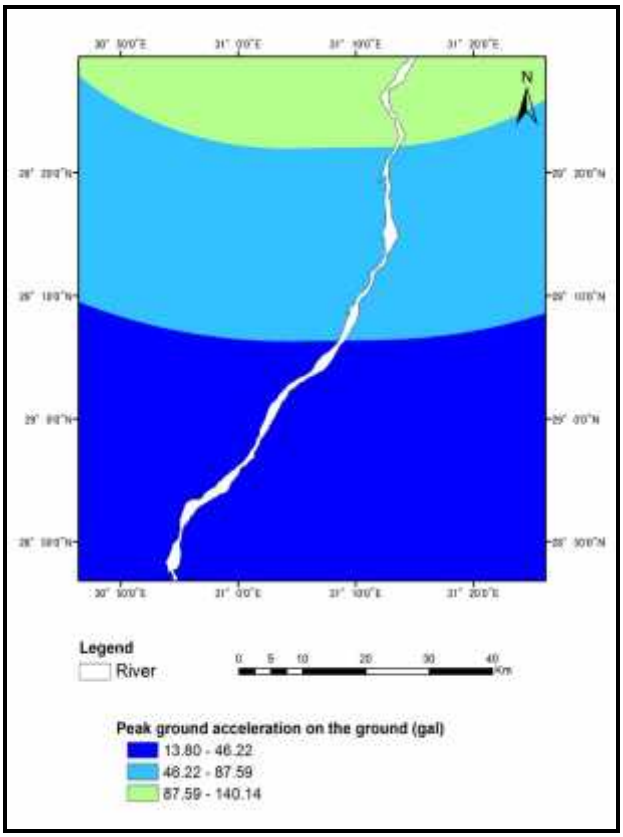


Fig.11 (d) Peak ground acceleration on the ground

It was found that the number of peak velocity on the engineering bedrock is small and gradual increase from the earthquake source because these values not depend on the amplification factor or geological units. Moreover, larger seismic ground motion is in the front area of the rupture propagation of the fault. In addition to the vicinity of the fault, there is remotely seismic ground motion because of the thick of sedimentary layers and also increased of site amplification. In the current work, the Peak ground acceleration (PGA) predicted from the October 12, 1992 Dahshour earthquake with magnitude 5.9, the October 11, 1999 southeast Beni Suef earthquake with magnitude: 4.9 and maximum expected earthquake with magnitude 6.5 are calculated by the following formula.

$$\text{LogPGA} = 0.50M_m + 0.0043D + d + 0.61 - \log(X + 0.0055 \times 10^{0.5Mm}) - 0.003X \quad (5)$$

M_m	: moment magnitude	X	: shortest distance from fault (km)
D	: hypocenter depth (km)	d	: fault type

Peak ground acceleration (PGA) is a measure of earthquake acceleration on the ground and an important input parameter for earthquake engineering, also known as the design basis earthquake ground motion. The PGA values obtained from seismic hazard assessment system GIS (SHAS-GIS) vary from 1.4 gal to 88 gal (or cm/s^2) by maximum expected earthquake. These values are rather high for the existing built environment and could cause damage and socio-economic losses for the study area. The presented results could be used as a basis for design motion specification of critical structure and for nonlinear analysis (structural, site response, landslides, and liquefaction).

CONCLUSIONS

Seismic hazard assessment maps for specified seismic source faults of Beni Suef area has been presented in this paper. A GIS geoprocessing system tool has been developed and applied to simulate the spatial ground motion of the October 12, 1992 Dahshour earthquake with magnitude 5.9 and the October 11, 1999 southeast Beni Suef with magnitude: 4.9 and maximum expected parameters for earthquake source with magnitude 6.5. The input GIS parameters for seismic hazard assessment are point grid data as an amplification factor and three-dimensional (3D) polygon as a geometrical fault model. The 3D polygon was constructed automatically in GIS geoprocessing model builder by using a geometrical lines data from previous observation study. The 3D fault's shape and attribute data in the 3D polygon, GIS gives significant result for the calculation of seismic hazard in which the GIS data can be proceed automatically and faster in the GIS integrated geoprocessing system development.

Furthermore, the peak velocity on the engineering bedrock, the peak velocity on the ground surface and the seismic intensity on the ground surface are calculated rigorously. It was found that larger seismic ground motion is in the front area of the rupture propagation of the fault. In addition to the vicinity of the fault, there is remotely seismic ground motion because of the thick of sedimentary layers and also increased of site amplification. As a result, this preliminary study of earthquake hazard assessment can be used as effective information for land-use planning in the Beni Suef Metropolitan area and strengthening of existing structures against future earthquakes.

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INFLUENCE OF SAND PERMEABILITY AT SAND COLUMN TO GROUNDWATER RECHARGE

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ABSTRACT: The need for ground water supply is increasing; however, with the excessive exploitation, the surface of the groundwater has gone down, and that has resulted in the lowering of the land subsidence, the sea water intrusion, and the decreasing groundwater quality. In order to keep the groundwater supply, various attempts have been taken, such as implementing the natural or artificial recharges. One of the artificial recharging methods which had previously been studied was the constructing a recharge reservoir on the soil with the permeability of less 10^{-5} cm^3/sec using a sand column. The purpose of research is to determine how much influence the permeability of the sand in the sand column. More specifically, the aim of the research was to determine exactly the kind of the sand column which could be used in the field, in order to reduce the groundwater problems. The measurement of the physical model test instrument was 180 cm x 115 cm x 60 cm with the parameters of the number of sand columns (4; 6; 12) pieces, the reservoir water level of (5; 7,5; 10) cm, the sand column height of (30; 32,5; 35) cm, three variations of sand permeability. The conclusion of this study namely, that the greater the density of the sand column and sand permeability, discharge into the aquifer layers greater. While the height of sand column opposite. The greater the height of sand column, the smaller discharge into the aquifer layers.

Keywords: *Sand Permeability, Groundwater, Sand Column.*

INTRODUCTION

Background

Rapid development of urban area is indicated by the existence of settlements and industries which increases water demand and disturbs the water supply and demand balance. This is worsen by the government incapability to provide clean water through Regional Drinking Water Company (PDAM). It fails to reach the overall area in supplying clean water. Thus, to obtain alternative clean water resource, people pump the groundwater by making and drilling wells. In large cities, the ground water exploitation has been intensive. Many industries and hotels have production wells up to 20 wells with more than 8.000 m^3 taking per day (Kodoatie and Sjarif, 2010), (Azis, A, 2014).

The existence of groundwater is strongly correlated to surface water. When the groundwater surface height experiences continuous decrease due to excessive exploitation, according to Tresnadi (2007), it will result in: land subsidence, sea water intrusion, degrading groundwater surface and decreasing groundwater quality.

To cope with those problems, there have been efforts of both natural and artificial recharge by developing infiltration well, bio-pores holes as well as other absorbance techniques. However the results have not been maximal. Thus, the development of recharge reservoir, which is being intensified in Indonesia, is hoped to cope with the problem. However, when recharge reservoir is to be constructed on certain area with small permeability coefficient and low absorbance power, water will be very slow to reach the aquifer layer and fail to function as a recharge reservoir. Therefore, it is required to study the use of recharge column model, within sand permeability variation, that is put in the base of recharge reservoir. This study is expected to provide solution for the problem of groundwater recharge in the above-mentioned condition.

The objective of this research is to analyze the influence of sand permeability variation of the sand column to the amount of occurring recharge.

LITERATURE STUDY

The groundwater crisis especially in large cities in Indonesia has reached danger level. Groundwater recharge almost never increases, but decreases instead, and with increasing usage on the other hand. The development of superblock and high buildings also contribute to decreasing groundwater supply. The building structure with deep basements also

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decreases the capability in absorbing rain water. Open green space and attachment area which previously function as The Office of Geology expects that Jakarta will endure groundwater crisis in 15 years ahead if no special treatments are taken. At present, groundwater uptake is quite high reaching 27 million m^3/year with groundwater absorbance of only 17 million m^3/year (Putranto, 2009).

According to Herlambang (2009), having poor quality of water surface, people of Jakarta shifts to groundwater. With uncontrolled uptakes, groundwater surface decreases from 50 m to 150 meter, causing the landscape degrades and sea water rises as tidal flood. In Yogyakarta, groundwater crisis is mostly due to many factors such as the options taken by the decision makers in the local government. According to Pramudya (2009), groundwater quality and volume degradation in Yogyakarta can be seen based on three aspects: poor layout, bad sanitation management and weak law enforcement for environment violator. There has been land changing function from wetfield into dry farm or settlements, shopping complex and other type of buildings. In Bandung, if there is no restoration of the groundwater condition, in 2013 there will be 116 % additional critical zone and 570 %, damaged zone of pressed groundwater surface which is under the upper limit of the pressed aquifer (*groundwater mining*). The estimated total area of *groundwater mining* is 244 km2 or 41 % of the total area of restricted aquifer (Hutasoit, 2009).

Groundwater

Groundwater is the most important mineral source taken from under the soil surface at water saturated zone. In Regulation No.7 year 2004 about Water Resource, it is mentioned that groundwater is water existed in the rocky layer (Sudjarwadi et, al, 2008). About 30% of daily consumption in the world is supplied by groundwater indicating its significant contribution to human lives and activities. The rest is provided by surface water from river or lake (Soedarmo and Purnomo, 2001).

Groundwater is one of the phases in hydrology cycle. It starts from an event where due to the heat of the sun, water changes its form to steam such as evaporation of river water, reservoir water or sea water and surface water, as well as evapo-transpiration, which is evaporation from the plantation surface. At certain height, the steam resulted from evaporation becomes cloud. Due to various causes, the cloud is condensed into precipitation in the form of snow, ice rain, rain or dew. Rainfall water sometimes is restrained by leaves or buildings. Some of it reaches the ground and becomes surface flow entering the catchment area and heading to the network system of river, lake or reservoir. Then it goes to coastal area and ends to the sea. Some of the flow is absorbed into the ground in the form of infiltration and percolation that becomes shallow or deep groundwater (Kodoatie and Sjarief, 2008), (Azis, A, 2014).

Groundwater flow

Groundwater requires energy to move and flow trough spaces between grains. Such energy comes from the potential energy. This is reflected from the water surface level (*piezometric*) of the related place. Groundwater flows from point with high to lower potential energy. There is no groundwater flow between points with the same potential energy. Imaginary line connecting the points with the same potential energy is called the contour line of groundwater surface or *isohypse* line. Along the contour line there is no groundwater flow because its direction is perpendicular to the contour line.

Permeability

Permeability is defined as the characteristic of pored material that enables the flow and seepage flow from the fluid, such as water or oil, to flow through the pore space connected to each other. Then, water can flow from point with high to lower energy point. For the soil, the permeability is described as the soil characteristics in flowing water through soil pore spaces (Hardiyatmo, 2010).

Soil permeability coefficient

Soil permeability coefficient is the amount of water that can be flown in each time unit through one unit of aquifer section area. The amount of water flown in the overall thickness of the aquifer is called the transmibility coefficient. The Soil permeability coefficient (k) is used to identify the amount of seepage and the problems of dams, irrigation channel, main embankment, infiltration well, etc.

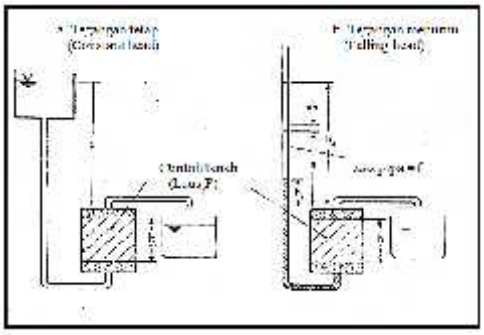

Figure 1. Fixed (a) and diminishing (b) Stress Tests (Das et.al, 1995)

Precise determination of soil permeability coefficient should be taken by laboratories method. There are two ways to determine the permeability coefficient in the laboratory: by fixed head experiment and falling head experiment. The first is used for soil with coarse grain and high permeability coefficient. The sand is set for soil with fine grain and low permeability coefficient. Both experiments are presented in Figure 1.

The Law of Darcy.

The Law of Darcy explains the ability of water to flow between pores of the soil and then give influencing characteristics. There are two main assumptions used in the Law of Darcy.

First, the fluid flow in the soil is laminar. Second, the soil is in saturated condition. The flow speed and water quantity/debit per time unit is proportional to the hydraulic gradient (Soedarmo and Purnomo, 2001), (Azis, A, 2013).

$$Q = k.i.A \quad (1)$$

$$V = \frac{q}{A} = k.i \quad (2)$$

Where : q = volume of water flow per time unit (cm3)
A = soil section area flown through by water (cm2)
k = permeability coefficient (cm/sec)
i = hydraulic gradient
v = flow speed (cm/sec)

Recharge Reservoir

The function of recharge reservoir .

One of the forms of artificial recharge is the recharge reservoir that mainly functions as water absorbance media which easily and quickly enters the aquifer layer. Such reservoir model is suitable for land with shallow and wide area (Kusnaedi, 2005), (Azis, A, 2014). According to Sudinda (2004) acting as the Team Leader of recharge reservoir development project of the Ministry of Research and Technology, the basic philosophy of the recharge reservoir development is how to minimize the surface runoff and increase the ground capacity to absorb the surface flow. The making of recharge reservoir is different from common reservoir. It is constructed with recharge base directly connected to the aquifer layer. Recharge reservoir is basically classified to *single purpose* recharge, functioning as the flood controller with a working system that increases the aquifer function optimization, such as increasing the water storage capacity in the aquifer layer (Azis, A, 2013).

The benefit of recharge reservoir includes :

1) Optimizing the aquifer function that may increase the water storage capacity in the aquifer
2) Functioning as flood controller in downstream or runoff areas.
3) Functioning as water storage in dry season.

Research by The Ministry of Research and Technology (Ristek) that has been taken since 2003 indicated that the application of recharge reservoir can cope with the annual flood and drought problems in Indonesia. The absorbance level (infiltration rate) of the recharge reservoir is quite high.

Previous researches.

Based on the above-mentioned understandings, some researchers have studied recharge reservoir as the followings:

1) The results of recharge reservoir simulation under different methods for the area of Universitas Indonesia (UI) which includes about 0.5 hectare. It shows absorbance of surface flow in infiltration rate of 1,933 m^3/day or 22.37 lt/sec. When conversed to water demand, it equals to water demand of 32,213 people of rural area or 16,106 people of urban area, which is 60-120 lt per capita and capable to supply agricultural area of 41.4 ha and potable water demand of industrial area of 30-40 hectare (Sudinda, 2004).
2) According to Djudi (2006), for Tambakboyo recharge reservoir in Regency of Sleman of Daerah Istimewa Yogyakarta, the results show that 25 m depth of drilling indicates sand of volcanic sedimentation with permeability coefficient of 1.1 x 10^{-4} cm/sec to 9 x 10^{-5} cm/sec. Therefore, the reservoir base consists of soil that is homogeny permeable and interacts to each other to aquifer that the percolation of the base soil is larger than normal. With 5.6 ha of inundation and 427,349 m^3storage volume, recharge reservoir can provide water recharge to the ground in 34.36 lt/sec averagely, indicating that it gives contribution to potable water availability for 49,479 people of Yogyakarta in. The reservoir water demand is supplied by River Tambakbayan.
3) Broto and Susanto (2008) made a model design of recharge reservoir of the City of Bogor by using the tank model application. Bogor is an area of groundwater filling. It supplies water to the urban area especially the city of Jakarta, which is susceptible to frequent flood. recharge reservoir is required as the precautions to the flood hazard. It optimizes the aquifer function by increasing the absorbance power of the aquifer in order to reduce the surface

runoff as the main cause of flood due to the river incapability to store the water. The simulation results show that water demand can then be absorbed from the recharge reservoir to the aquifer layer by 1,150 m^3/day.

4) Based on the simulation result shows that the amount of water absorbed from the recharge reservoir to the aquifer layer is 1,150 m^3/day. Based on a study by Akhmad Azis (2012) about the use of sand column for recharge reservoir for anticipating soil layer with small permeability on the reservoir base, the absorbance debit at density of 0.0157 was 62.41 cm^3/sec.

Physical model of sand column

In this study, the sand column is used as the median to absorb recharge reservoir water to the aquifer layer. The traditional method in making sand column is by drilling holes on the clay layer with small permeability and refilling it with gradated sand. Sand should be flown by water without bringing fine soil particles.

According to Liong (2000), soil with coarse grain has small surface specific (clean sand with surface specific of 10^{-4} m^2/g). Thus, the surface force can be ignored relatively to its own weight. That its own weight force is more dominant than the surface adhesive force. Thus, the composition between the particles of coarse grain soil is called single and composed as a group of randomly packed balls. Each soil and gravel particle relates to other particles or it can be said as no cohesion. Sand and gravel are non cohesive soil. Due to self weight force being more dominant than the surface force, the sand and gravel behavior are not influenced by water. Particle distribution is a relevant parameter. The influence of water is visible only for free fine sand that is water saturated and has experienced dynamic loading (Joleha, 2001), (Azis, A, 2014).

Sand columns with a Principle that water from the surface is stored in the reservoir at certain height. Then water is flown through the sand columns that sand with large permeability coefficient can accelerate and enlarge recharge as well as to become filtration that water entering the aquifer layer is in clean condition (Azis, A, 2012).

sand column density

Hypothetically, the sand column density placed in the base of the recharge reservoir has significant influence to the recharge debit. The higher the density level, the lager the recharge debit will be (Azis, A, 2012). To obtain the sand column density, the following formula is applied:

$$\xi = \frac{A\ kolom\ pasir}{A\ waduk} \tag{4}$$

$$A\ kolom\ pasir = \frac{1}{4}\pi\, d^2 . Nkp \tag{5}$$

Where : d = diameter of sand column
N_{kp} = numbers of sand column

RESEARCH METHOD

Data collecting Technique

Data collecting was carried out by the followings:

This study used a testing tub in square dimension of 180 cm x 115 cm x 60 cm. The identified soil which fulfilled the required permeability, was put into the tub. Then test was taken without using sand column. Then the sand column was installed. On the surface of the tub, it was given with input debit (Q_1) for the reservoir and (Q_2) for the energy height difference. To identify the size of the debit, time was recorded for the 5 times filling of each 1000 ml, and so was the runoff debit (Q_3) and the debit coming out of the aquifer layer (Q_4). When the soil experienced water *saturated*, observation was taken for each three variations for sand permeability, sand column density (δ) and three variations of energy height difference.

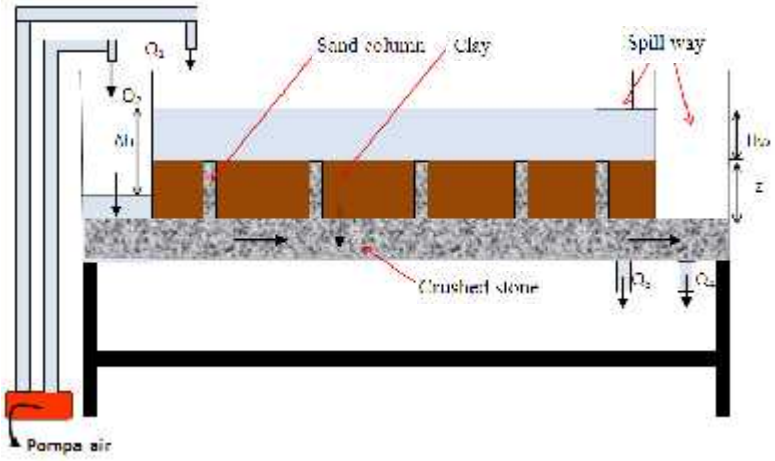

Figure 2. Model testing by using sand column

RESULTS

Characteristics of Soil and Sand Material

Soil Sample

According to USCS and ASTM systems, soil is classified as ML (inorganic silt with low plasticity, fine sand with silt or clay) and CL (inorganic clay with low plasticity, clay with silt or skinny clay), because the fine fraction (58.9%) > 50% and liquid limit (42.6%) < 50%, plasticity index (5.9%) is between required 4 < PI < 7.

Based on AASHTO and HRB system, the soil was classified as soil A4 (with silt), because fine fraction (58,9%) > 36% passed no. 200 sieve, with liquid limit (27.5%) < 40%, plasticity index (5.9%) < 10%, and group index (1.49) < 8. Soil permeability coefficient (k) = 0.000033 cm/sec that the soil type is classified into the compact silt or silt with clay.

Based on the systems of classification and the obtained permeability coefficient, in can be concluded that soil used in this research is silt soil with clay and with low plasticity.

Sand sample

Based on USCS and ASTM systems, the three types were classified as SW (good gradated sand, slight or with no fine grain) and SP (poor gradated, slight or with no fine grain), because fine fraction (2,58) < 5% and coarse fraction (97,42) > 50% passed number 4 sieve.

Under the AASHTO and HRB systems, the soil is classified in A-1-b (sand), because the fine fraction (2,58%) < 50% passed the number 200 sieve, liquid limit (4,1%) < 25%, plasticity index (0%) < 6%. permeability coefficient of sand (k) was k1= 0,201 cm/sec, k2 = 0.034 cm/sec and k3 = 0.023 cm/sec, respective. The soil type was classified as coarse sand.

Based on the four systems of classification and the obtained permeability coefficient, it can be concluded that the three sand used in this study was coarse sand.

Influencing factors to Groundwater recharge Debit

The amount of water entering the aquifer layer through the soil layer as well as the sand column in this research were influenced by sand permeability and energy height difference.

The influence of sand permeability

Based on the results of this research either with or without sand column, as well as the analysis results as seen on Table 1, Table 2 and Table 3 and Figure 3, it can be indicated that the amount of groundwater recharge debit, on the soil layer thickness or sand column height of 30 cm, 32,5 cm, 35 cm and density of (ʒ) 0 ; 0.0019 ; 0.0028 ; 0.0033 ; 0.0050 ; 0.0052 ; 0.0057 ; 0.0079 ; 0.01 and 0.0157, respectively, increased in line to the increasing sand permeability. The increasing groundwater recharge debit can be explained below :

Table 3 shows that groundwater recharge debit increased slowly from the density of 0.0019 to 0.0028 and then from 0.0019 to 0.0033 that the higher would be from 0.0019 to 0.0157 at 9 variations of energy height differences. Based on the calculation of average percentage of groundwater recharge debit, significant difference was visible between the lowest and the highest density. It was also indicated that due to the increasing density from 0.0019 to 0.0028 (47,37%) there was increasing percentage or 50.92%, and from 0.0019 to 0.0157 (726,32%) by 846.68%.

Table 1. Groundwater recharge debit at sand permeability 1

ϑ	Groundwater recharge debit , Q_a (cm^3/sec)									Average debit	% increasing debit
	Energy height difference , z (cm)										
	37.4	35	34.9	34.2	32.8	32.5	32	30.9	30.2		
0.0019	6.82	6.40	6.23	6.36	5.98	5.82	5.96	5.61	5.47	6.07	0
0.0028	10.20	9.49	9.31	9.58	8.93	8.77	8.92	8.37	8.91	9.16	50.92
0.0033	12.26	11.40	11.28	11.39	10.62	10.51	10.87	10.15	10.05	10.95	80.29
0.0050	18.37	17.03	16.77	17.26	16.09	15.84	16.12	14.99	14.80	16.36	169.48
0.0052	18.78	17.52	17.24	20.35	19.15	18.63	21.90	20.39	19.92	19.32	218.17
0.0057	20.43	18.94	18.65	19.14	17.80	17.42	17.86	16.68	16.31	18.14	198.68
0.0079	28.00	27.60	25.85	30.20	28.15	27.65	33.00	30.95	30.29	29.08	378.85
0.0100	36.48	33.83	33.31	34.25	31.92	31.44	32.16	29.91	29.52	32.54	435.81
0.0157	56.51	52.78	51.62	60.79	56.54	55.32	65.05	59.55	59.20	57.48	846.68

As seen on Table 2 below, at sand permeability 2 of 0.034 cm/sec, the condition was almost the same as sand permeability 1 of 0.201 cm/sec.

Table 2. Groundwater recharge debit at Sand permeability 2

ϑ	Groundwater recharge debit , Q_a (cm^3/sec)									Average debit	% increasing debit
	Energy height difference , z (cm)										
	37.4			34.2	32.8	32.5	32	30.9	30.2		
0.0019	3.21	3.05	2.96	3.02	2.86	2.77	2.80	2.67	2.59	2.88	0
0.0028	4.80	4.57	4.42	4.50	4.29	4.18	4.20	4.01	3.90	4.32	49.90
0.0033	5.63	5.35	5.20	5.28	5.01	4.87	4.92	4.68	4.55	5.05	75.43
0.0050	8.44	8.02	7.81	7.91	7.51	7.32	7.39	7.03	6.84	7.59	163.29
0.0052	8.89	8.45	8.22	8.34	7.91	7.69	7.78	7.40	7.17	7.98	177.09
0.0057	9.50	9.02	8.80	8.90	8.45	8.24	8.31	7.90	7.69	8.53	196.22
0.0079	13.34	12.68	12.34	12.51	11.87	11.57	11.67	11.10	10.81	11.99	316.08
0.0100	16.88	16.04	15.61	15.83	15.02	14.64	14.77	14.05	13.67	15.17	426.46
0.0157	26.67	25.32	24.68	25.02	23.74	23.14	23.34	22.21	21.61	23.97	731.97

It was indicated that groundwater recharge debit slowly increased from density 0.0019 to 0.0028 and then from 0.0019 to 0.0033 and the highest from 0.0019 to 0.0157 at 9 variation of energy height difference. Due to the increasing density from 0.0019 to 0.0028 (47.37%), the average percentage of groundwater recharge debit of 49.9% was the smallest increase. The quite large increase was obtained from density 0.0019 to 0.0157 (726,32%) by 731.97%.

For sand permeability 3 of 0.023 cm/sec as presented on Table 3, the condition was similar to sand permeability 1 of 0.201 cm/sec and permeability 2 of 0.034 cm/sec, but with smaller increasing percentage. This was because the smaller the sand permeability, it will give more influence to the increasing level. It was also visible that groundwater recharge debit slowly increased from the sand column density of 0.0019 to 0.0028 then from 0.0019 to 0.0033 that the highest was from 0.0019 to 0.0157 cm at 9 variations of energy height difference. The average percentage of the smallest groundwater recharge debit was 50.86%, due to the increasing sand column density from 0.0019 to 0.0028 (47.37%), and the largest was from 0.0019 to 0.0157 (726.32%) with increasing debit of 734.57%.

Table 3. Groundwater recharge debit at Sand permeability 3

ϑ	Groundwater recharge debit , Q_a (cm^3/sec)									Average debit	% increasing debit
	Energy height difference , z (cm)										
	37.4	35	34.9	34.2	32.8	32.5	32	30.9	30.2		
0.0019	0.87	0.82	0.80	0.81	0.77	0.75	0.76	0.72	0.70	0.78	0
0.0028	1.30	1.23	1.20	1.22	1.16	1.13	1.15	1.10	1.07	1.17	50.86
0.0033	1.54	1.46	1.42	1.44	1.37	1.33	1.35	1.29	1.25	1.38	77.86

0.0050	2.31	2.20	2.14	2.17	2.06	2.00	2.02	1.95	1.89	2.08	167.71
0.0052	2.41	2.29	2.23	2.26	2.14	2.08	2.11	2.04	1.96	2.17	178.86
0.0057	2.60	2.47	2.40	2.44	2.31	2.26	2.27	2.19	2.13	2.34	201.00
0.0079	3.61	3.43	3.34	3.39	3.21	3.13	3.16	3.01	2.93	3.25	317.29
0.0100	4.62	4.39	4.28	4.33	4.11	4.01	4.04	3.85	3.76	4.15	434.14
0.0157	7.22	6.86	6.68	6.77	6.43	6.27	6.32	6.01	5.86	6.49	734.57

The three tables above showed that the increasing sand column density was linear to the increasing recharge debit. This indicated that the sand column density is proportional to the recharge debit, in accordance to the Law of Darcy. The amount of debit is influenced by energy height difference which is a function of hydraulic gradient.

Figure 3 describes the relation between groundwater recharge debits to the sand column density at three sand permeability values. The Figure shows the increasing debit from density 0.0019 to 0.0157 at all sand permeability values. At permeability 1, groundwater recharge debit was larger than permeability 2 and 3 because sand permeability 1 was larger. At the minimum and maximum debit at sand permeability 1 were 5.47 cm^3/sec and 65.05 cm^3/sec, respectively. At sand permeability 2, minimum and maximum groundwater recharge debits were 2.59 cm^3/sec and 26.67 cm^3/sec, respectively. At permeability 3, the minimum and maximum groundwater recharge debits were 0.70 cm^3/sec and 7.22 cm^3/sec, respectively.

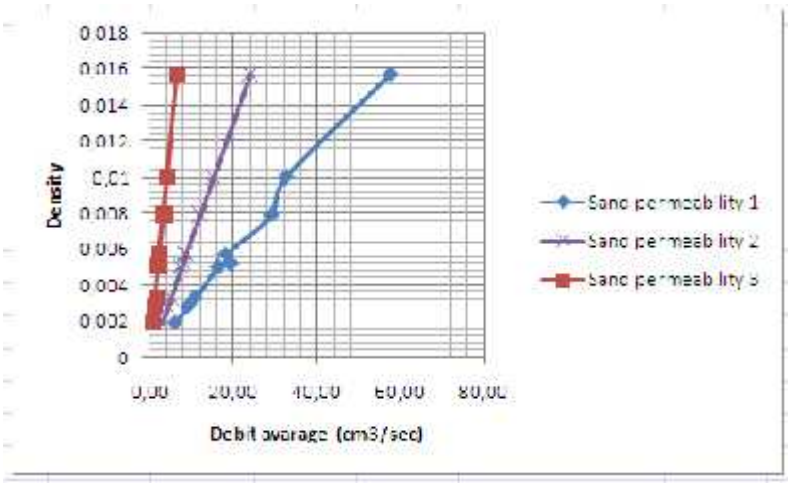

Figure 3. Average Groundwater recharge debit at 3 sand permeability values

Figure 4 describes the increasing percentage of groundwater recharge debit at three sand permeability values. It shows increasing debit from density 0.0019 to 0.0157 at all sand permeability. At permeability 1, the percentage increase of groundwater recharge debit was larger than permeability 2 and 3, and the increase of percentage at sand permeability 2 and 3 was almost the same because sand permeability 2 and 3 had insignificant difference.

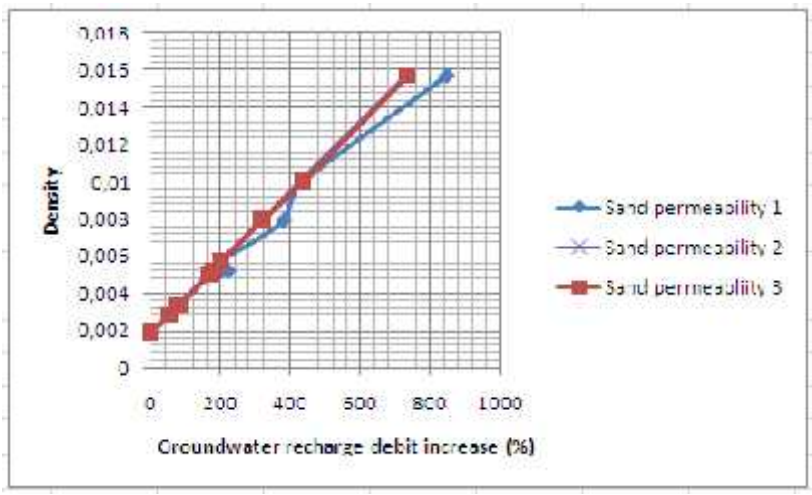

Figure 4. The percentages of groundwater recharge debit increase

CONCLUSIONS

Based on the results, it can be concluded that the larger the sand column density and permeability, the larger the debit entering the aquifer layer will be. As for the side flow debit and the sand column length, the otherwise occurs. The larger the side flow debit and the higher the sand column, the smaller the debit entering the aquifer layer will be. Based on the research results, it shows the percentage of the increasing average debit from the density of 0.0019 to 0.0157 at all sand permeability values. At permeability 1, the increasing percentage of groundwater recharge debit was larger than permeability 2 and 3, and the increasing percentage of sand permeability 2 and 3 was almost the same because sand permeability 2 and 3 had insignificant difference. The influence of energy height difference indicated increasing debit of sand column density from 0.0019 to 0.0157. At density = 0.0019, the average minimum groundwater recharge debit was 2.92 cm^3/sec and maximum was 3.63 cm^3/sec. At density = 0.0057, the average minimum groundwater recharge debit was 28.89 cm^3/sec and maximum was 30.13 cm^3/sec.

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EXPERIMENTAL AND NUMERICAL OF PULL-OUT TEST OF STAR ANCHOR PLATE EMBEDDED IN COHESIVE SOIL

A. Rachman Djamaluddin[1], Wa Ode Sumartini[2]

ABSTRACT: This study aims to compare the effectiveness of anchors of circular plate type and star-shape plate type by conducting a pull-out modeling simulation in a laboratory, and by conducting a numerical validation. The anchor modeling was made in three variations of anchor planting depth: 30 cm, 60 cm 90 cm. The analysis was conducted by using laboratory experimental and numerical analysis by finite difference . A comparison of numerical geomechanics analysis in finite difference approach with those published from small scale laboratory tests show well agreement. Prediction of pull-out capacity was conducted on star-shaped anchor types. The result showed that ultimate pull-out capacity get increase significantly from the anchor planting at 30 cm to 60. But ultimate pull-out capacity from 60 cm planting to 90 cm almost constant. Anchor of star-shaped plate type has the same model of failure both experimental and numerical analysis. at a depth of 30 cm, where the soil failure happened visible to the surface and at a depth of 60 cm and 90 cm, where the failure occurred in the location of the land around the plate anchor.

Keywords: *anchor, stars, circles, capacity, finite difference*

INTRODUCTION

Anchors have been developed for various purposes such as strengthening the slopes, retaining walls (plaster), the stability of the tunnel, the foundation for preventing transmission tower style pull, roll, etc. There are many types of anchors depending on the type of load and large, the type of structure and local subsoil conditions. The behavior of anchors in the field indicate that the failure mechanism and bearing capacity of the anchor can be determined by many factors. Most studies focus on massive models shaped anchor plates with various shapes (circle, square) with a variation of the dimensions, depth, and type of load.

Some studies have been conducted on anchor plates. Merifield et al. (2003) examined numerically the anchor plate circle, square, and strip solutions with the lower bound method. The development forms an anchor element with easy installation and sufficient carrying capacity. This study conducted introducing anchor element type star (star anchor) embedded on cohesive compacted soil through an experimental laboratory. It investigated a pull-out capacity and failure models due to the effect of variations in the depth of embedment. This paper presents experimental and numerical analysis of star anchor embedded in cohesive soil.

ANALYSIS METHOD

Anchors are typically constructed from steel or concrete and may be circular (including helical), square, or rectangular in shape. This study aimed to do the development in the form of modifications plate anchor circular becoming a plate anchor type star with 4 leaves in same area with circular anchor plate type shown in Figure 1 and general layout of the problem to be analyzed is shown in fig. 2

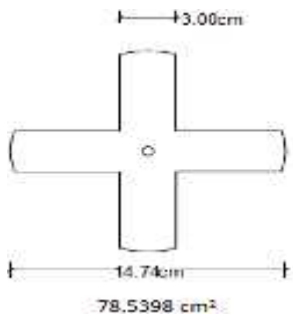

Figure 1. Type notation of star shaped plate anchor

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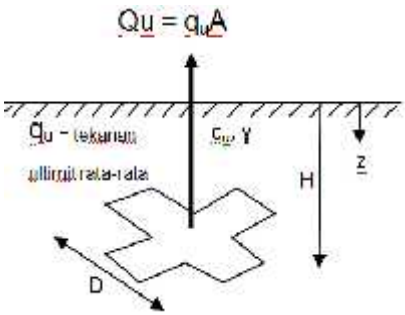


Figure 2. Problem notation of star shaped plate anchor

Pull-out capacity of anchors in undrained clay

Ultimate pull-out capacity of anchors in clay undrained conditions proposed by Merifield et al. (2003), are usually shown as a function of undrained shear strength in the following form.

$$q_u = \frac{Q_u}{A} = c_u N_c \tag{1}$$

Where A = the anchor area, c_u = the undrained soil strength at the ground surface, and N_c = anchor break-out factor. For convenience, Merifield et al. (2003) has been defined the anchor break-out factor for the following two cases:

1. For a homogeneous soil profile with no unit weight ($\gamma = 0$)

$$Nc = Nco, \text{ where } N_{co} = \frac{q_u}{c_u} \tag{2}$$

2. For a homogeneous soil profile with unit weight ($\gamma \neq 0$)

$$Nc = Nc\gamma, \text{ where } N_{c\gamma} = N_{co} + \frac{\gamma H}{c_u} \tag{3}$$

Eqs. (1)- (3) reflect the complex nature of the break-factor N_c observed by Rowe and Davis (1978). The break-out factor is a function of embedment ratio (H/B) and overburden pressure, with the latter being expressed in term of dimensionless quantity $\gamma H/c_u$. this indicates that, separate from overall problem geometry, the soil property directly influence anchor behavior.

It should be noted that the break-out factor given in eqs. (1) – (3) does not continue to increase indefinitely, but reaches a limiting value which marks the transition between shallow and deep anchor behavior. The limiting value of the break-out factor is defined as Nc*.

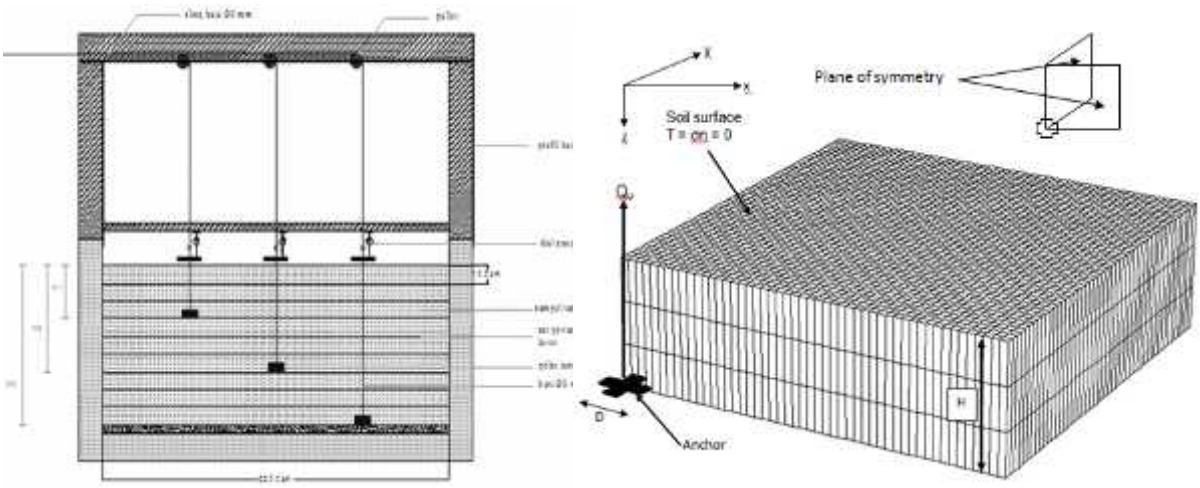
Implicit in eqs. (3) is the assumption that the effect of soil unit weight and cohesion are independent of each other and may be superimposed. This assumption was investigated in the previous study of Merifield et al. (2005), where it was found that the principle of superposition can be successfully applied to shallow strip anchors in clay.

Anchor can be classified as shallow or deep, depending on their mode of failure. This point is illustrated in Figure 3. An anchor is classified as shallow if, at ultimate failure, the observed failure mechanism reaches the surface. In contrast, a deep anchor is one whose failure mode is characterized by localized shear around the anchor and is not affected by the location of the soil surface.

For a given anchor size, B, and soil properties γ, cu there exist a critical embedment that Hcr at which the failure mechanism no longer extend to the soil surface. This type of failure mechanism is typically observed for deep anchor, and is localized around the anchor. The significance of such localized failure mechanism is that the ultimate capacity of the anchor will have reached a maximum limiting value. This arises because the undrained shear strength is assumed to be independent of the mean normal stress.

Experimental and numerical modeling details

A simplified representation of the laboratory model test and the boundary grid arrangements used to be analyzed illustrated in fig. 4.



(a) Laboratory *(b) Numerical (FLAC3D)*

Figure 4. Laboratory and numerical Set up for analyzing star anchor

Pullout capacity on star plates anchor with constant area was investigated at the anchoring depth of 30 cm, 60 cm, and 90 cm. Figure 4 shows pullout capacity with the uplift displacement at every depth. At the depth of 30 cm, the test was carry out until failure ground surface was obtained. On the other hand, at the depth of 60 and 90 cm, the test was done until the large deformation with constant load was obtained.

At the numerical analysis, the soil mass is first discretized into a number of zone where the boundaries between adjacent zone may be specified as a stress discontinuity or rigid joint. Each zone is then subdivided in three-dimensional space to form a number of radial cylinder grid for circular anchor and brick grid for star anchor within each zone. To model a smooth anchor may be modeled by insisting the shear stress is zero at all element nodes along the anchor. To allow the underside of the anchor to separate from the soil (immediate breakaway), the stress discontinuity between soil above the anchor segment is removed, and the shear stress and normal stress are force to be equal to zero. The soil properties considered for analysis are shown in Table 1.

Table 1. Soil properties

Soil properties	**Unit**
Dry density	1,054e3 kg/m^3
Bulk modulus	2.7942e6 Pa
Poisson'sratio	0,3
Shear modulus	1.2896e6 Pa
Cohesion	5.48e4 Pa
Friction	0° (clay)

RESULT AND DISCUSSION

Relationship with the ultimate capacity of the anchors planting a numerical test results compared with the results of laboratory testing. Benchmarking is then plotted into a graph shown in Figure 5.

Figure 6 shows that the ultimate capacity FLAC3D numerical tests showed similarities with laboratory testing. While at anchor planting 60 cm and 90 cm for both type, the ultimate capacity FLAC3D numerical tests showed similarities with laboratory testing. Figure 6 also shows the similarity of behavior unplug capacity rise of planting 30 cm to 90 cm in planting two types of anchor plates, where the value of pull capacity increases linearly from 30 cm to 60 cm planting and from planting 60cm to 90 cm, the value of the capacity is slowly reaching to a constant value.

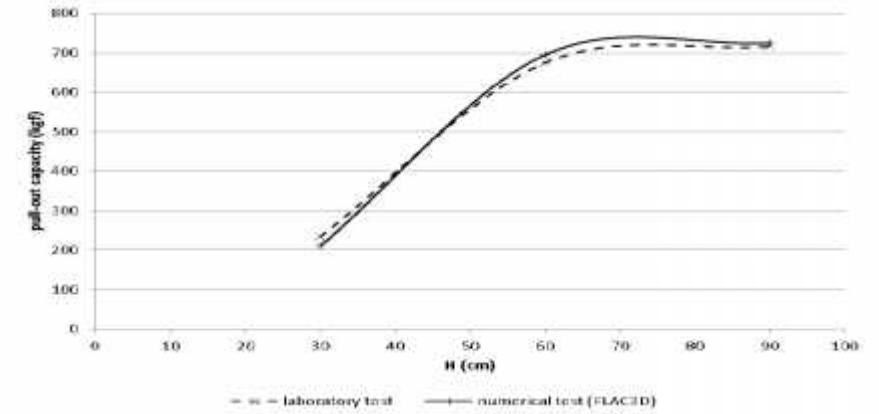


Figure 5. Comparison with laboratory test results

From the results of the operation FLAC3D program, can be obtained images contour of displacement which is represent model of soil failure as a result of land stretching caused by the movement of the anchor plate by 10 cm. it then compared with the results of laboratory testing. failure mode of star shaped anchor plate are Plot in any variation of anchor planting. They are shown in Figure 6, 7, and 8.

Figure 6 shows that at 30 cm of anchor planting, laboratory test and numerical simulation has similar failure mode, where the soil failure happened looks to the surface which it is different with circular anchor and star anchor at 60 cm and 90 cm anchor planting, where the soil failure occurs localized around the plate of anchor that shows in Figure 7 and 8.

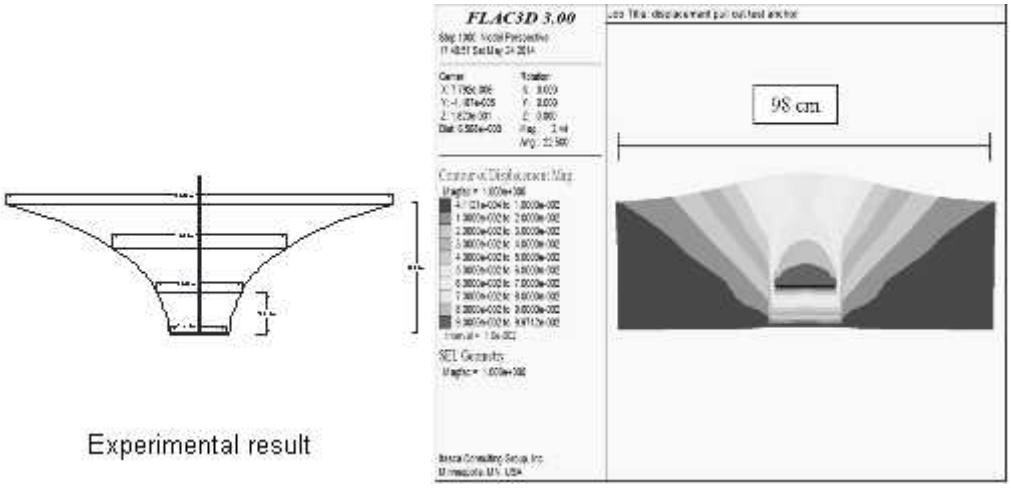


Figure 6. Failure mechanism at 30 cm of anchor planting

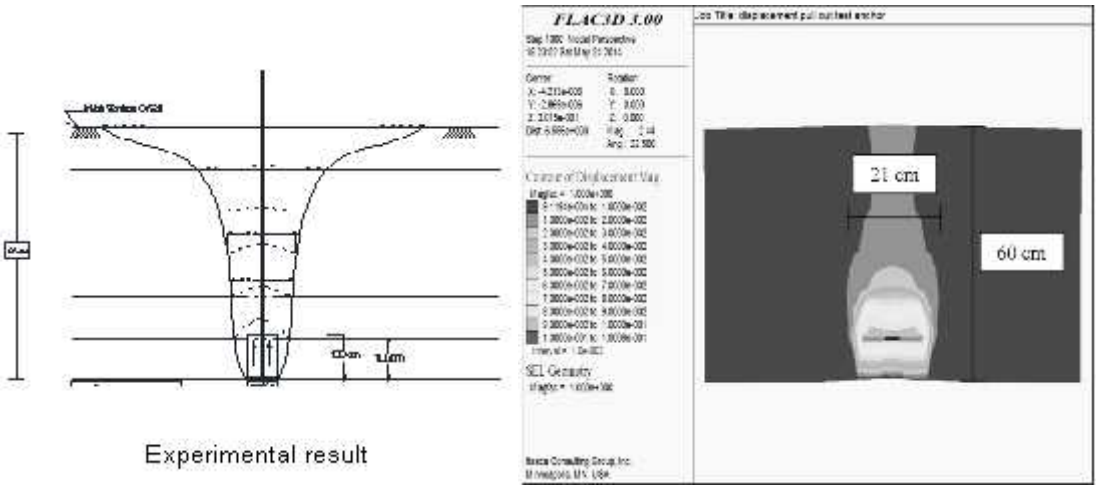


Figure 7. Failure mechanism at 60 cm of anchor planting

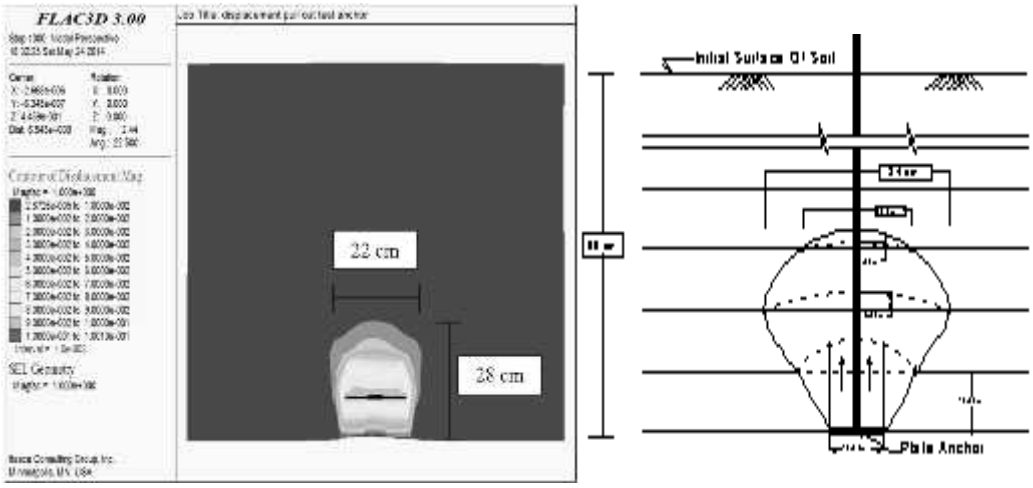


Figure 8. Failure mechanism at 90 cm of anchor planting

CONCLUSIONS

A comparison of numerical geomechanics analysis in finite difference approach with those published from small scale laboratory tests show well agreement. Prediction of pull-out capacity was conducted on star-shaped anchor types. The result showed that ultimate pull-out capacity get increase significantly from the anchor planting at 30 cm to 60. But ultimate pull-out capacity from 60 cm planting to 90 cm almost constant. Anchor of star-shaped plate type has the same model of failure both experimental and numerical analysis. At a depth of 30 cm, where the soil failure happened visible to the surface and at a depth of 60 cm and 90 cm, where the failure occurred in the location of the land around the plate anchor.

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ENERGY DISSIPATION OF MASONRY STRUCTURES UNDER CYCLIC LOADING

Milad M. Alshebani[1]

ABSTRACT: Masonry assemblages are used extensively as infill walls in reinforced concreteframes. Researchers have concluded that a proper use of masonry infills in RCframes could improve the strength and stiffness of the structure subjected toseismic excitation. Behavior of masonry walls under cyclic loading providesvital information regarding stiffness degradation, material ductility and energy dissipation characteristics. A cyclicloading can produce three distinct stress-strain curves. These curves aretermed; envelope curve, common point curve and stability point curve. Energy dissipation characteristics were examined and compared for various cyclicparameters including envelope strain and residual strain. Energy dissipationwas found to vary with the envelope and residual strains in bilinear manner. Based on experimental data, empirical relations were proposed for thevariations of the energy dissipation with the envelope strains, residualstrains and stiffness degradation. These relations indicate that the decay ofmasonry strength due to cyclic loading starts at about two-third of peak (failure) stress. This stress level is in close proximity of the peak stress ofstability point curve, a curve that has been used to identify the allowablestress level.

Keywords: *cyclic load, energy dissipation, masonry infill, stiffness, strain*

INTRODUCTION

RC frames with masonry infill walls constitute a considerable portion of existing building stocks. Researchers have indicated that interactions between masonry infills and the enclosed frame may enhance the strength and stiffness of the overall building structures, but are not capable of improving inelastic deformability (1). Others have argued that the increases in stiffness are beneficial since they lead to reduction in magnitude of deformation induced by ground motion (2,3). Cyclic lateral loads applied to RC frames strengthened with infill masonry walls sustained drift reversals with amplitudes of up to 1.5% of the height of the structure without excessive stiffness reduction (4).Cyclic loading tests on brick masonry provide vital information related to material's ductility, stiffness degradation and energy dissipation characteristics. The cumulative energy dissipation is often used as a measure of the structural seismic performance. The performance of reinforced concrete structures is widely measured by the concept of energy dissipation (5,6). Low energy dissipation characterizes the brittle behavior, while high energy dissipation indicates a ductile behavior. Energy dissipation capacity has been used to assess the ability of the structural member to withstand cyclic loading or as a damage indicator (7,8). Energy dissipation is usually expresses as a non-dimensional ratio (9) defined as the energy dissipated per cycle to the total input energy. The dissipation of energy is greatly influenced by the size and number of cracks due to cyclic loading. In brick masonry structures, it is also influenced by the cyclic load orientation, being normal to be joint or parallel to bed joint. Furthermore, the energy dissipation characteristics is greatly affected by the cyclic loading condition, being uniaxial or biaxial loading (9).

TEST PROGRAM

The test specimens were constructed from sand plast brick units each measuring 115 x 55x35 mm. a mortar mix of 1 cement:0.5 lime:4 sand by volume was used for bed joints of 5 mm thickness. The mean compressive strength of the brick was 23.4N/mm^2, and the mean cube compressive strength of the mortar was 10.2N/mm^2.

LVDTs were installed in two orthogonal directions on both faces of the panel to sensor the displacements which can be captured graphically on two X-Y plotters. Loading – unloading was controlled by Universal Testing Machine. Typical specimen is shown in Fig (1)

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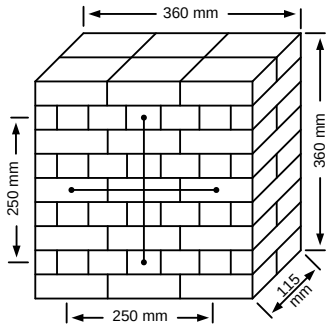

Fig. 1 The specimen and the LVDT arrangements

Three types of loading were conducted, each would produce a distinct stress-strain curve: Loading type I is a monotonic loading in which load was increased steadily to failure to obtain the envelope stress-strain curve. Loading type II is a cyclic loading in which the peak stress-strain in each cycle coincided approximately with the envelope stress-strain curve. Loading type III is a cyclic loading as in type II except the reloading and unloading was repeated several times in each cycle until the intersection of loading-unloading curves stabilizes at a lower bound point.

CYCLIC STRESS-STRAIN CURVES

The three types of cyclic loadings produced three distinct stress-strain curves. Stress and strain are expressed in a nondimensional form. The stress coordinate is normalized with respect to peak stress, and the strain coordinate is normalized with respect to the strain corresponding to peak stress for each specimen.

a- Envelope curve: Loading type I produced the monotonic stress-strain curve. The envelope stress-strain curve was obtained by superimposing the points of peak stress-strain of cyclic loading type II and loading type III on peaks of monotonic stress-strain curve. Envelope curves were produced for the two loading conditions; normal to the bed joints and parallel to the bed joints.

b- Common Point curve: Cyclic loading type II possessed a locus of intersection points called common points. These points are formed by the intersection points of reloading curve of each cycle with the unloading curve of the previous cycle. The first cycle of loading type III also produced data for common point curve.

c- Stability Point curve: This curve is the result of loading type III. If each cycle of loading-unloading is repeated several times, then intersection points of loading unloading curves keep forming in descending order. This process continuous for each cycle until the intersection point stabilizes at a lower bound. The locus of these lower bound points for all the cycles termed as the stability point stress-strain curve. The hysteresis of stress-strain loops are shown in Fig (2). Thus cyclic loading can produce three levels of stress-strain curves in which cycle peaks are bounded by the envelope curve limit. These curves can be presented mathematically in a non-dimentional coordinate system. The cyclic loading is applied uniaxially normal to bed joints and parallel to bed joints.

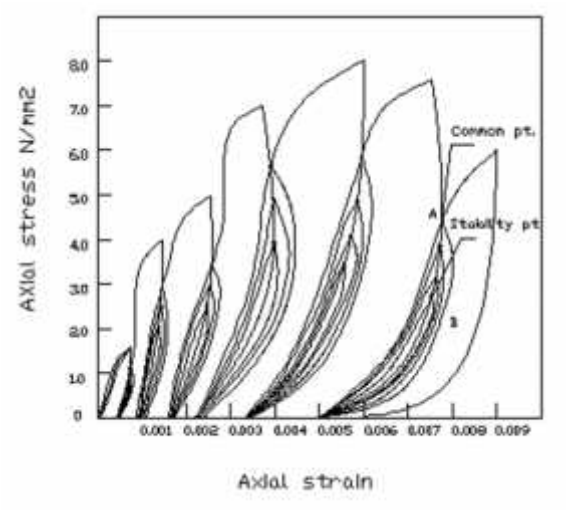

Fig. 2 Typical stress-strain hysteresis of cyclic loading

MATHEMATICAL EXPRESSIONS

Data collected from the test of the three types of loading were used to establish a mathematical model for the three stress-strain curves for the two bed joint orientations:

For envelope stress-strain curve:

$$\sigma = \varepsilon^{\beta} \exp (1 - \varepsilon/\alpha) \quad (1a)$$

$$\sigma = \varepsilon^{\beta} \exp [(1 - \varepsilon)\, \varepsilon\, (\alpha+\beta)] \quad (1b)$$

Equation (1a) is for load normal to the bed joint and equation (1b) is for load parallel to the bed joint.

For common point or stability point stress-strain curve :

$$\sigma = \varepsilon^{\beta} \exp [(1 - \varepsilon/\alpha)\, \varepsilon] \quad (2)$$

Equation (2) is valid for load normal to the bed joint and for load parallel to the bed joint.

In the above equations, the terms $\sigma$and ε are the normalized stress and normalized strain respectively.

The parameters α and β in the above equations are constants that depend on bed joint orientation under loading and on stress level. The parameter β is considered as a shape factor while parameter α accounts for the change in stress maxima. The values of the parameters α and β are listed in Table (1) along with the correlation indices i_C between the proposed curves and the test data. The mathematically expressed curves of equations (1) and (2) are shown in Figure (3)

Table 1 values for α , β, i_C

Stress-Strain Curve	Normal Bed Joint			Parallel Bed Joint		
	α	β	i_C	α	β	i_C
Envelope	1.0	1.0	0.97	0.73	0.70	0.96
Common point	0.73	0.80	0.95	0.70	0.73	0.92
Stability point	0.58	0.80	0.90	0.96	0.73	0.89

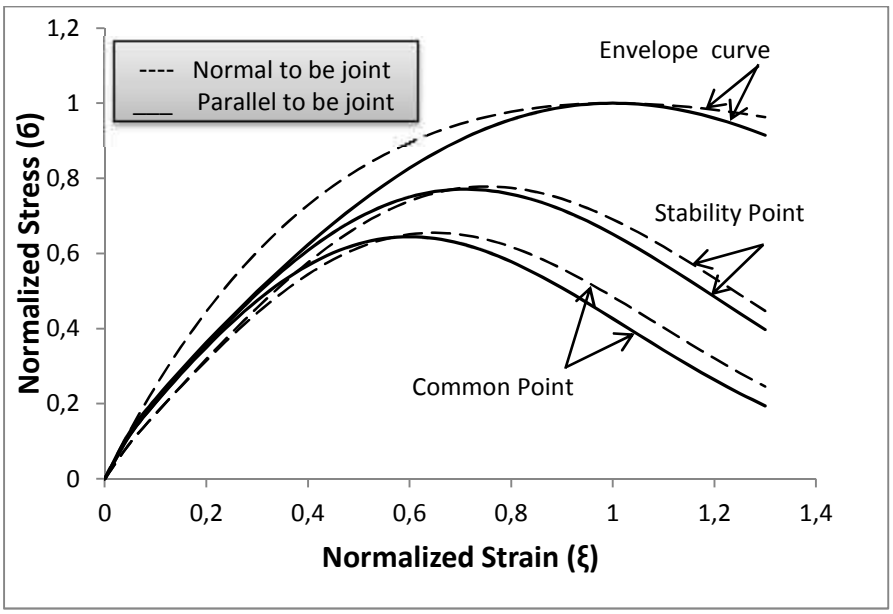

Fig. 3 Three Cyclic Curves of Brick Masonry

This paper examines the results of uniaxial compressive cyclic loading on sand plast (a form of calcium silicate) brick masonry in relation to energy dissipation characteristics.

ENERGY DISSIPATION

The cumulative energy dissipation is often used as a measure of the structural seismic performance. The energy dissipation is presented in this paper in a non-dimensional form termed the energy dissipation ratio, R_n. Thus the energy dissipation ratio is defined as the ratio of the energy dissipated (area enclosed by loading-unloading loop) per cycle to the total input energy (total strain energy) of that cycle. As shown in Fig (4), the energy dissipation ratio (R_n) can be calculated by measuring area A and area B from the curve.

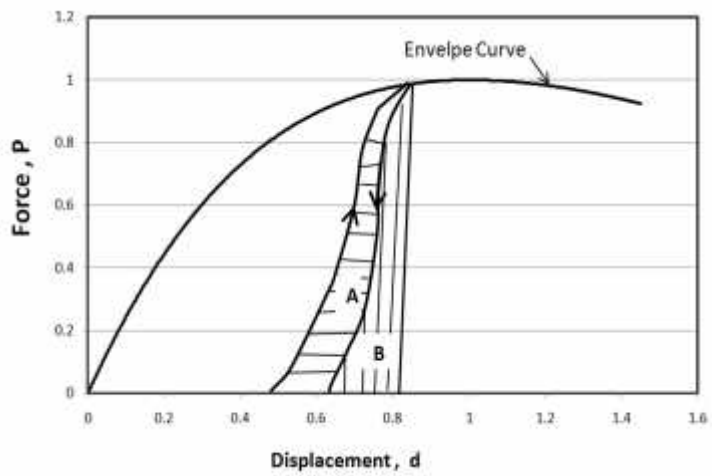

Fig. 4 Calculation of Energy Dissipation Ratio

$$R_n = B/(A+B) \quad (3)$$

Variation of energy dissipation with envelop strain

Stress-strain envelope curve is the result of superimposing the cycles' peaks on the monotonic stress-strain curve. The relation between the energy dissipation ratio, R_n and the normalized envelope strain ε_E at the peak of each cycle was found to follow the following general form:

$$R_n = \beta_1 \ln(\varepsilon_E^{\alpha_1} + 1) \quad (4)$$

Where α_1 and β_1 are constants and their values depend on the bed joint orientation to load direction. The Value of α_1 ranges from 0.37 to 0.40; whereas β_1 ranges from 0.90 to 0.93. The relation in Eq (4) showed a good fitting with test data with a correlation index equals 0.91 on the average.The variation of Eq (4) is shown in Fig (5). The relation between R_n and ε_E is characterized by initial linear portion, transit non-linear zone followed by approximately linear portion with slow rate of increase in R_n and faster rate of increase of ε_E . The extension of the initial linear relationship ends at about $\varepsilon_E = 0.2$ which in turn corresponds to normalized stress of 0.40 and 0.35 for loading normal to bed joints and parallel to bed joints respectively. These stress values are regarded as the elastic limit. The high rate of increase of energy dissipation at early stage and slower rate of increase at later stage were due to difference in the formation and size of cracks between the two stages. Micro-cracks are formed at early stage, while at later stage, the slower increase in R_n reflects the growing and widening of cracks.

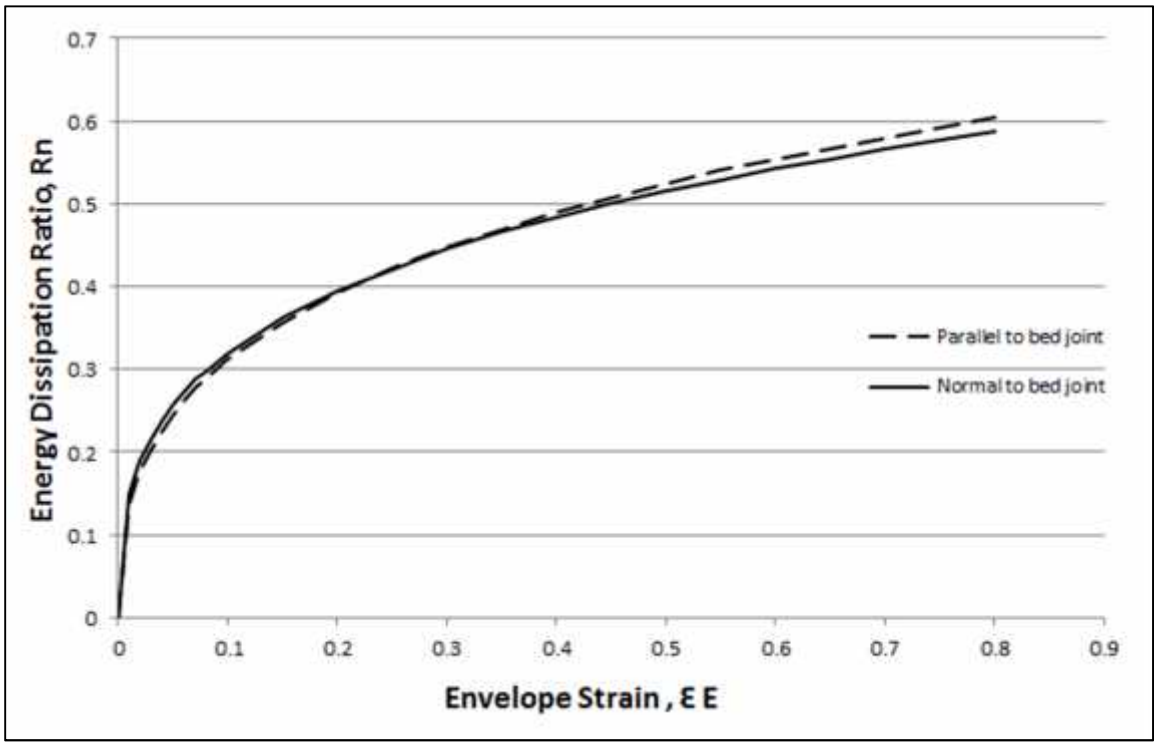

Fig. 5 Variation of Energy Dissipation with Envelope Strain

Variation of energy dissipation with stiffness degradation

The observed decay of masonry resistance to the cyclic loads can be represented by the change in stiffness at the peak of the cyclic loads. The stiffness at first crack is used as a reference point for stiffness degradation. From the stress-strain hysteresis, the stiffness associated with first crack at load level P_{cr} is termed as K_{cr}. This stiffness, K_{cr}, for each specimen is then used to normalize stiffness values, K_i, at the peak of each load cycle, P_i of that specimen. Therefore, the normalized (ratio of) stiffness degradation, termed as K_n, can be obtained at any peak load cycle by the division operation of K_i / K_{cr} . From test data, the relation between the energy dissipation ratio and the stiffness degradation ratio is found to follow the following formula:

$$K_n = \text{Exp}\,(0.25 - bR_n^3) \qquad (5)$$

The constant *b* was found to be equal to 4.0 and 3.0 for loading normal to bed joints and parallel to bed joints respectively. The correlation index between the expression and test data is 0.95 on the average. Prior to cracking, little energy is dissipated and thus the stiffness remain almost constant at low value of R_n, as shown in Fig (6). Although the relation between K_n and R_n showed similar trend for the two orthogonal joint orientation, the stiffness decays relatively more solely in the case of loading parallel to bed joint. The sharp drop in the plot of Fig (6) is due to low energy dissipation capacity of sand plast brick masonry, an indicative of brittle behavior. This type of brick masonry appears to possess less toughness than low strength clay brick masonry. Therefore, the energy input needed to approach the final collapse for sand plast brick masonry is less than that needed for low strength fired clay brick masonry.

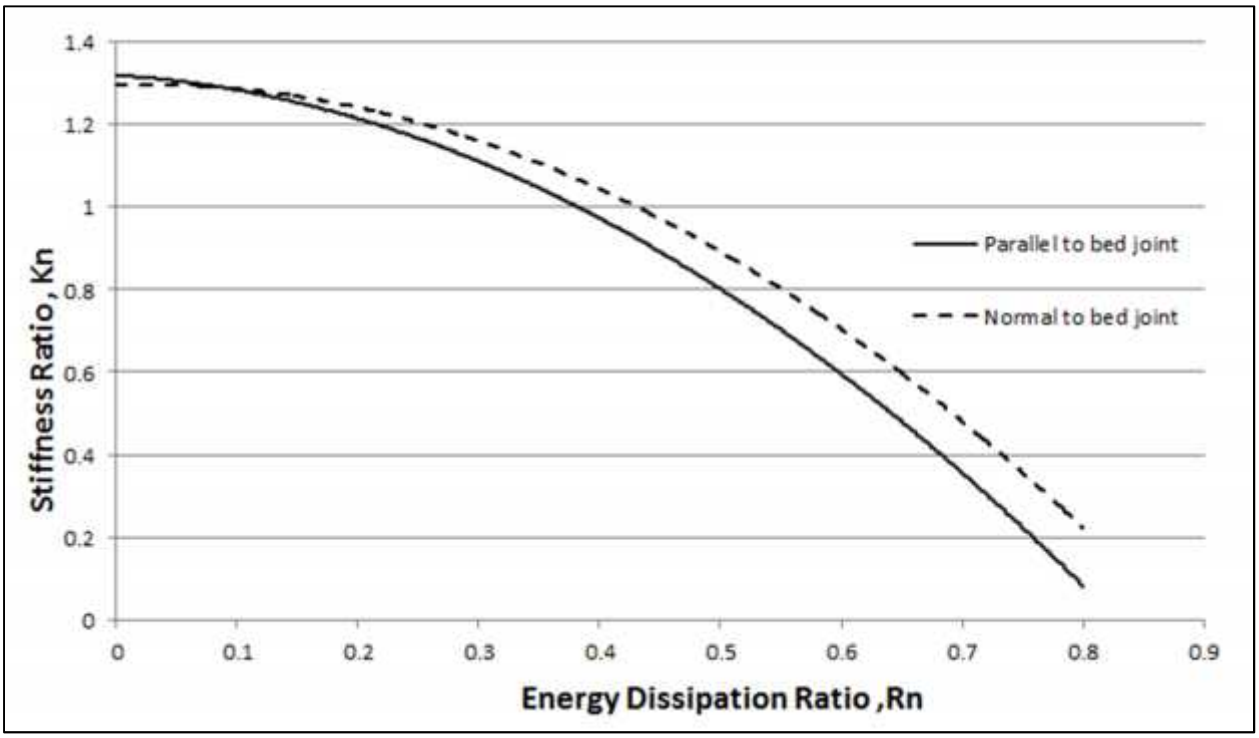

Fig. 6 Variation of Energy Dissipation with Stiffness Degradation

Variation of energy dissipation with plastic strain

The residual (plastic) strains accumulate as the number and intensity of load cycles increases. Plastic strain at the end of unloading of any cycle is defined as the axial strain minus the strain recovered when unloading is completed for that cycle. The strains are normalized with respect to strain corresponding to peak stress for each specimen and are termed ε_r. The relation between ε_r and R_n is expressed mathematically by the following formula:

$$R_n = \frac{1}{m}\varepsilon_r^n \ln(\varepsilon_r^n + c) \qquad (6)$$

Where $\ln$ is the natural logarithm and n, m and c are constants determined from test data based on the statistical analysis of the correlation index. The values of these constant are given in Table (1) along with the respective correlation index, i_c .The variation of Eq(6) is shown in Fig (7). The relation between ε_r and R_n is bilinear in nature and in general it follows three zones. Initial linear portion with faster rate of increase in R_n, followed by transit non-linear portion, followed by a relatively approximate linear portion with faster rate of increase in ε_r . The faster rate of increase in ε_r at a later stage of loading reflects the growing winding of cracks and thus faster accumulation of plastic strain. At early stage, the linearity of the relation extends up to ε_r equal to 0.14 and 0.08 for loading normal and parallel to bed joints respectively. At these values linearity starts to deviate, hence the material strength starts to decay. These values also correspond to normalized envelope stresses averaging 0.68 (Alshebani and Sinha, 2000). Such an average stressvalue is in conformity with a normalized peak stress of stability point curve and therefore, it may be considered as a damage index for the material.

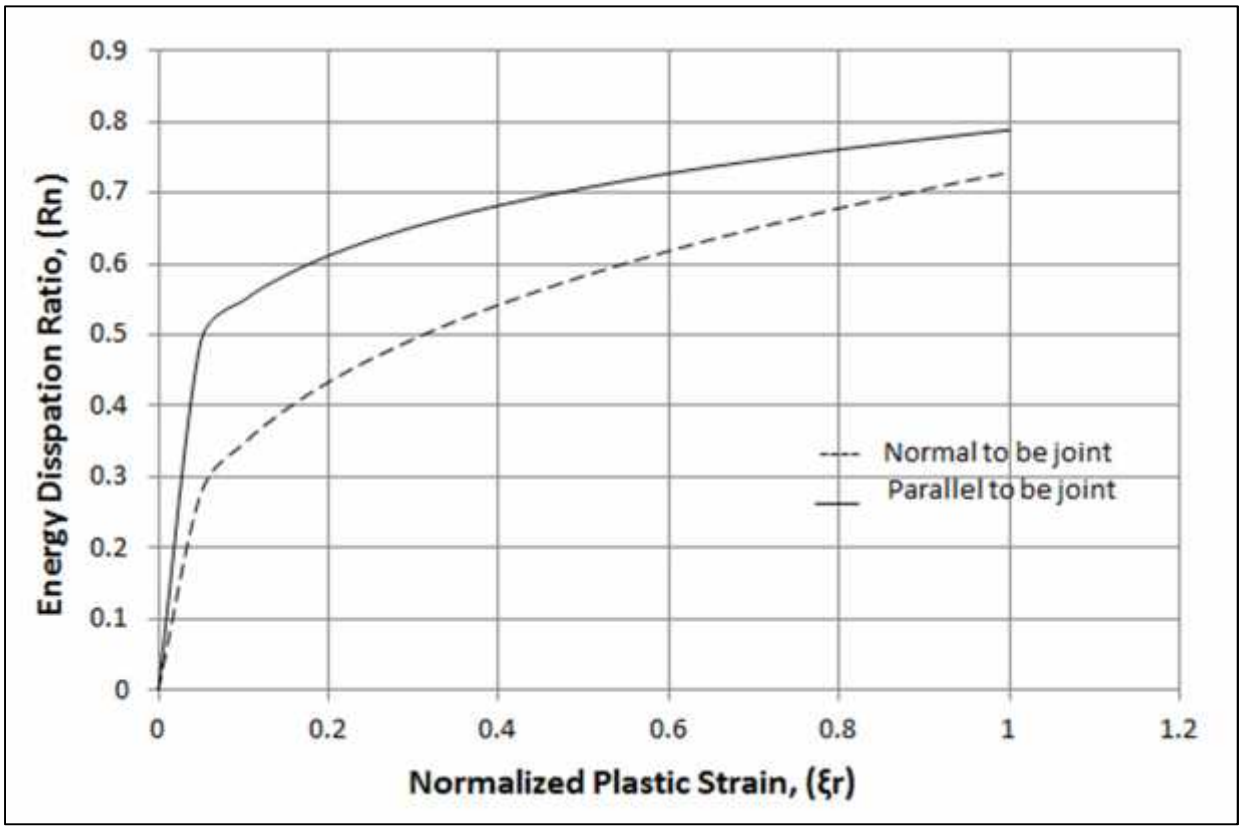

Fig. 7 Variation of energy Dissipation with Plastic Strain

CONCLUSION

1. Behavior of infill masonry walls under repeated cyclic loading provide vital information about material ductility, stiffness degradation and energy dissipation characteristics.

2. Cyclic loading can produce three distinct stress-strain curves, termed as envelope curve, common point curve and stability point curve.

3. Stability point curve can be used as a reference for allowable stress level under cyclic loading.

4. Energy dissipation of masonry walls can be expressed in a non-dimensional form as the ration of energy dissipated per cycle to total input energy of that cycle.

5. Energy dissipation is evaluated against three parameters: envelope strain, stiffness degradation and plastic strain.

6. The variations of energy dissipation against these parameters follow a bi-linear relation in general.

7. The relations between energy dissipation and these three parameters are expressed mathematically based on test data

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EFFECT OF MINERAL ADMIXTURES ON CARBONATION OF SEAWATER MIXED CONCRETE

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ABSTRACT: This study presents carbonation of seawater mixed concrete and tap water mixed concrete incorporating mineral admixtures such as Fly Ash (FA) and Ground Granulated Blast-Furnace Slag (GGBS) with W/B of 40%, 50% and 60% in tap water curing (TC), seawater curing (SC) and air curing (AC). The effect of seawater mixing, mineral admixtures, water-binder ratio, curing conditions and exposure condition on carbonation process of concrete were evaluated. Concrete prism specimens using tap water and seawater as mixing water were prepared. At 24 hours after casting, specimens were de-molded and followed by distinct curing conditions such as tap water curing, seawater curing and air curing. After curing until 28 days, specimens were exposed in air during 4 weeks before they were brought into accelerated chamber for carbonation and natural outdoor exposure. Based on experimental result, it was shown that carbonation depth of seawater-mixed concrete with and without mineral admixtures in AC was lower compared to tap water mixed concrete in both accelerated and natural exposure carbonation. Moreover, mineral admixtures increased carbonation rate of seawater mixed concrete in all curing condition. Besides, the carbonation rate of seawater-mixed concrete with and without mineral admixtures in SC was smaller than that of in TC. In addition, accelerated carbonation depth of seawater mixed or tap water mixed FA and GGBS concrete was seven times larger than natural exposure carbonation.

Keywords: *Seawater mixing, carbonation, mineral admixture, curing condition, exposure condition*

INTRODUCTION

Seawater is prohibited to use as mixing water for reinforced concrete, because it increases the risk of corrosion of steel bars in concrete. Nevertheless, in case of unavoidable circumstances, seawater may be used as mixing water, not only for plain concrete but also for reinforced concrete (JSCE 2002, BIS 2000 and Bertolini et al. 2013). Furthermore, if the utilization of seawater is permitted as a concrete material, it will be very beneficial and economical in the construction, especially in the coastal area. In addition, it can be one of the solutions to anticipate the lack of fresh water as reported by World Meteorological Organization (WMO), that more than half of the world population would not be able to gain enough drinking water by 2025 (Nishida et al. 2013).

One of major factors which cause the deterioration of structural concrete is carbonation (Chi et al. 2002). The result of carbonation may not only induce initial corrosion because surface of steel bars is not passivated anymore due to a drop in pH in concrete but also releases bound chloride into pore solution which makes the pore solution even more aggressive (Bertolini et al. 2013 and Wan et al. 2013). The carbonation rate of concrete depends mainly on the relative humidity, the CO_2 concentration, concrete composition, and environmental temperature where concrete is placed (Bertolini et al. 2013).

Discussions on effect of seawater as mixing water on carbonation of OPC concrete and Fly Ash (FA) concrete have been expressed in authors' previous study (Adiwijaya et al. 2013 and Adiwijaya[1] et al. 2014). Nevertheless, influence of mineral admixtures, curing and exposure condition are not entirely discussed. Thus, carbonation of seawater mixed or tap water mixed concrete incorporating mineral admixtures such as Fly Ash (FA) and Ground Granulated Blast-Furnace Slag (GGBS) with 40%, 50% and 60% water-binder ratio (W/B) in tap water curing, seawater curing and air curing were investigated. This study was carried out to evaluate the effect of mineral admixtures, water-binder ratio, curing conditions and exposure conditions on carbonation resistance of seawater mixed concrete.

EXPERIMENTAL PROGRAM

Materials

Three types of binder were used in this study, namely Ordinary Portland Cement (OPC), Fly Ash (FA) and Ground Granulated Blast-Furnace Slag (GGBS), specified in Japan Industrial Standard (JIS). The used two types of GGBS, namely B_{4000} and B_{6000} with different surface area were selected. Table 1 describes the physical and chemical properties

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of binders. As for aggregate of concrete, crushed stone with 20 mm in maximum size was used as coarse aggregate, density (SSD): 2.84 g/cm^3, fine modulus (FM): 6.68 and water absorption: 0.71%. The used fine aggregate was washed sea sand, the density (SSD): 2.56 g/cm^3, fine modulus (FM): 2.69 and water absorption: 1.61%. Furthermore, seawater (natural seawater) was utilized as mixing water and curing water. The chemical composition of seawater is shown in Table 2. The source of seawater was taken in seaside area in Itoshima, Fukuoka, Japan under summer season.

Table 1 Physical and chemical properties of binders

Constituent	Binder			
	OPC	FA	B_{4000}	B_{6000}
Density, g/cm^3	3.16	2.26	2.91	2.91
Fineness, cm^2/g	3330	3970	4060	5810
MgO, %	1.31	1.29	5.32	5.4
SiO_2, %	20.71	60.6	34.10	34.06
SO_3, %	2.14	0.50	-	-
LoI, %	1.97	2.4	0.33	0.36
Total Alkali, %	0.43	1.57	0.41	0.58
Chloride, %	0.02	-	0.01	0.01

Table 2 Chemical properties of seawater (natural seawater)

Specific Gravity	pH	Chemical Composition (g/l)					
		Cl	SO_4	Ca	Mg	K	Na
1.020	7.71	18.72	2.37	0.36	1.2	0.35	9.9

Concrete Mixtures

Table 3 presents concrete specimen series that were investigated in this research. Sixteen series of concrete mixtures with water-binder ratio (W/B) of 40%, 50% and 60% using tap water (T) and natural seawater (S) as mixing water were prepared. Three type curing conditions such as tap water curing (TC), seawater curing (SC) and air curing (AC) in 20°C, 60% R.H. controlled room were selected as curing conditions. The binder proportions of mixture series were OPC concrete (N), 20% FA replacement (2F), 30% FA replacement (3F), 50% $GGBS_{4000}$ replacement (B_4) and 50% $GGBS_{6000}$ replacement (B_6), respectively. Mix proportion of concrete specimens was designed according to Japan Society of Civil Engineering standard (JSCE 2002), as demonstrated in Table 4. In mix proportion table, the water content of tap water mixed and seawater mixed is slightly difference with the same water to cement ratio. This is due to difference of water density, which density of seawater is quite higher than that of tap water.

Table 3 Mixture series of concrete specimen for carbonation test

Series Mixture	W/B (%)	Mixing Water	Binder Proportion (%)				Curing Condition		
			OPC	FA	B_{4000}	B_{6000}	TC	SC	AC
T40-2F	40%	T	80	20	--	--			O
S40-2F		S	80	20	--	--	O	O	O
T40-B4		T	50	--	50	--			O
S40-B4		S	50	--	50	--		O	O
T50-N	50%	T	100	--	--	--			O
S50-N		S	100	--	--	--	O	O	O
T50-2F		T	80	20	--	--			O
S50-2F		S	80	20	--	--	O	O	O
S50-3F		S	70	30	--	--		O	O
T50-B4		T	50	--	50	--			O
S50-B4		S	50	--	50	--		O	O
S50-B6		S	50	--	--	50		O	O
T60-2F	60%	T	80	20	--	--			O
S60-2F		S	80	20	--	--	O	O	O
T60-B4		T	50	--	50	--			O
S60-B4		S	50	--	50	--	O	O	O

N: OPC; F: FA; B_4: $GGBS_{4000}$; B_6: $GGBS_{6000}$; O: Done

Table 4 Mix proportion of concrete

Series Mixture	Unit Content (kg/m³)								Slump (cm)	Air (%)	Temp. (°C)
	W	OPC	FA	GGBS	Sand	Gravel	WR*	AE*** (ltr)			
T40-2F	160	320	80	-	764	1028	1.75	1.15	5.0	3.1	23.0
S40-2F	165	330	82	-	760	1022	5.36**	0.79	12.5	5.6	24.0
T40-B4	160	200	-	200	764	1010	2.00	0.77	10.5	4.0	24.0
S40-B4	165	206	-	206	759	1004	3.71**	0.79	8.0	5.1	21.0
T50-N	160	320	-	-	805	1087	1.00	2.00	8.0	6.2	16.0
S50-N	165	330	-	-	802	1083	1.03	1.27	8.0	6.0	18.0
T50-2F	160	256	64	-	795	1074	1.00	2.00	9.5	5.8	17.0
S50-2F	165	264	66	-	791	1068	1.03	1.68	10.0	3.8	20.0
S50-3F	165	231	99	-	786	1061	1.03	1.97	8.5	3.4	18.0
T50-B4	160	160	-	160	800	1081	1.4	1.08	12.0	4.5	19.0
S50-B4	165	165	-	165	797	1076	1.24	1.11	5.0	4.0	20.0
S50-B6	165	165	-	165	797	1076	1.44	1.59	7.5	4.7	20.0
T60-2F	160	213	53	-	811	1099	0.5	1.28	7.0	3.3	23.0
S60-2F	165	220	55	-	808	1095	0.58	1.45	6.0	3.5	23.0
T60-B4	160	133	-	133	814	1077	0.83	1.41	6.5	5.0	23.0
S60-B4	165	137	-	137	811	1073	0.94	1.45	7.0	4.8	21.0

* WR: Water reducer (AE+WR) ; ** Superplasticizer ; *** 100 times dilution

Testing Methods

Concrete prism specimens of 10 x 10 x 40 cm in size were de-molded at 24 hours after casting then cured in tap water, seawater and in air. After 28 days curing, prism specimens were exposed in air within four weeks in 20°C, 60% R.H. controlled room. Afterwards, two concrete specimens were coated by epoxy resin on the two longitudinal and transversal sides, then dried for 24 hours before they were brought in the accelerated chamber for carbonation (20°C, 60% R.H. and 5% CO_2 concentrations) and natural outdoor exposure site. The accelerated carbonation of concrete was carried out in accordance with JIS A 1153:2003.

After a certain exposure period for 1, 4, 8, 13 and 26 weeks, the specimens were splitted then sprayed by a 1% solution of phenolphthalein in ethanol solution onto the concrete surface. In the carbonated region, the color of the solution was not alteration (colorless), while in the noncarbonated portion, the color solution alters into purple-red. The distance between the color change boundary and the concrete surface was measured as the carbonation depth. The accelerated carbonation of concrete was measured according to JIS A 1152:2002. Twenty readings were recorded from the two side faces of a specimen at 10 mm intervals. The average of these readings from two specimens was taken to represent the carbonation depth.

RESULTS AND DISCUSSION

Compressive Strength

Fig.1 represents compressive strength of concrete specimens of seawater mixed concrete and tap water mixed of OPC concrete, FA concrete and GGBS concrete, which was measured in according with JIS A 1108:2006. It was observed that 28-day compressive strength of seawater-mixed concrete is higher than tap water-mixed concrete in all curing conditions, in TC, SC and AC. It denotes that seawater-mixing improves compressive strength of concrete compared to tap water-mixing. This finding is probably inferred to be due to presence of chloride in seawater mixed, which accelerates the cement hydration process of concrete, thus induces a more rapid development of strength. Moreover, no significant difference on compressive strength in water curing was found for both tap water mixed and seawater mixed. It expresses that compressive strength of seawater mixing and tap water mixing is not affected by type of curing water, TC or SC.

The similar trend was also obtained on compressive strength up to 91 days that compressive strength of seawater mixing was higher than that of tap water mixing for OPC concrete and FA concrete. However, strength of seawater mixed GGBS concrete was slightly lower than tap water mixed GGBS concrete up to 91 days in water curing, as shown in Fig. 2. Also, concrete specimens in AC were increased in compressive strength up to 91 days from 28 days for both tap water mixed and seawater mixed, but strength development of concrete in AC was smaller than both TC and SC. In addition, concrete mixed with seawater in AC was significantly increased in compressive strength compared with concrete mixed with tap water. The largest compressive strength up to 91 days of seawater mixed concrete in AC was acquired for OPC concrete followed by GGBS concrete and FA concrete. It exhibits that strength gains of seawater

mixed OPC concrete in AC is more accelerated compared with FA concrete and GGBS concrete. This tendency is probably expected to be due to the chemical reaction of OPC is rapid comparing to mineral admixtures.

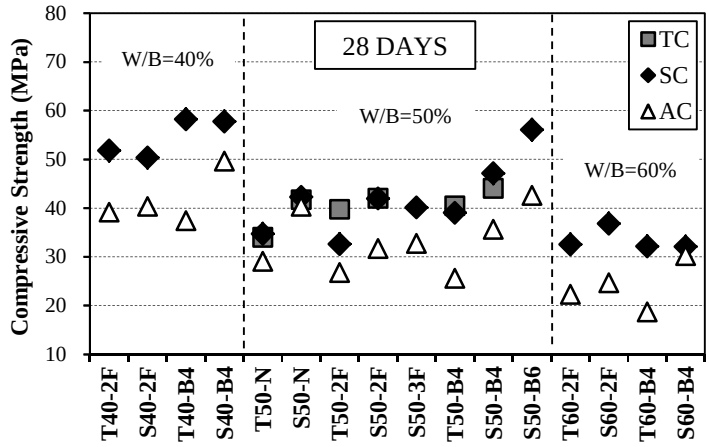


Fig. 1 Compressive strength of concrete specimens at 28 days in various curing

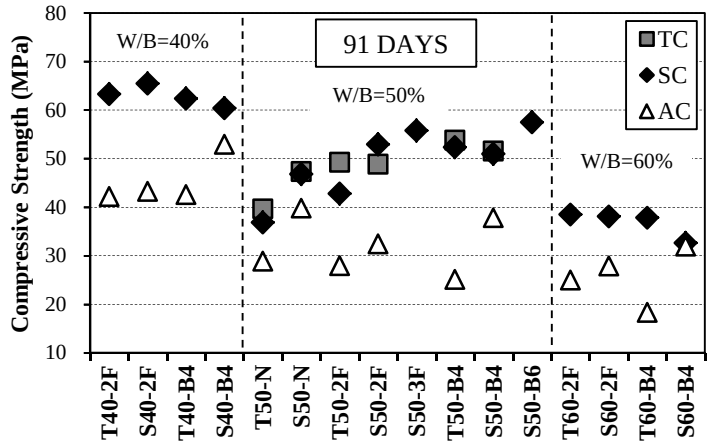
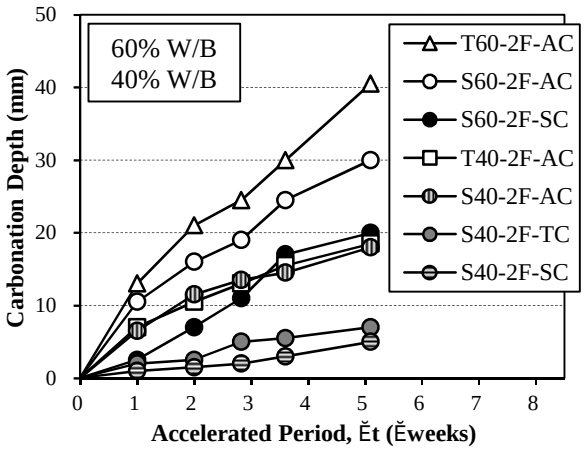


Fig. 2 Compressive strength of concrete specimens at 91 days in various curing

Carbonation Progress

The carbonation progress of tap water mixed and seawater mixed OPC concrete and FA concrete specimens in accelerated test is depicted in Fig. 3. From this result, it can be seen that carbonation depth of seawater mixed concrete was lowered compared to tap water mixed concrete in AC. It means that seawater mixing increases carbonation resistance of OPC concrete and FA concrete in AC. Similarly, seawater mixed GGBS was lower than tap water mixed GGBS concrete, as described in Fig. 4. However, progress in carbonation of GGBS concrete is higher than that of FA concrete in AC. Generally, it can be concluded that seawater mixed improves carbonation resistance of concrete with and without mineral admixtures.



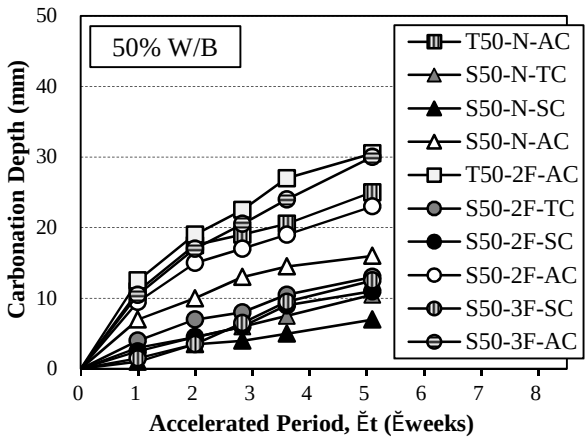


Fig. 3 Progress of carbonation of OPC and FA concrete in various curing

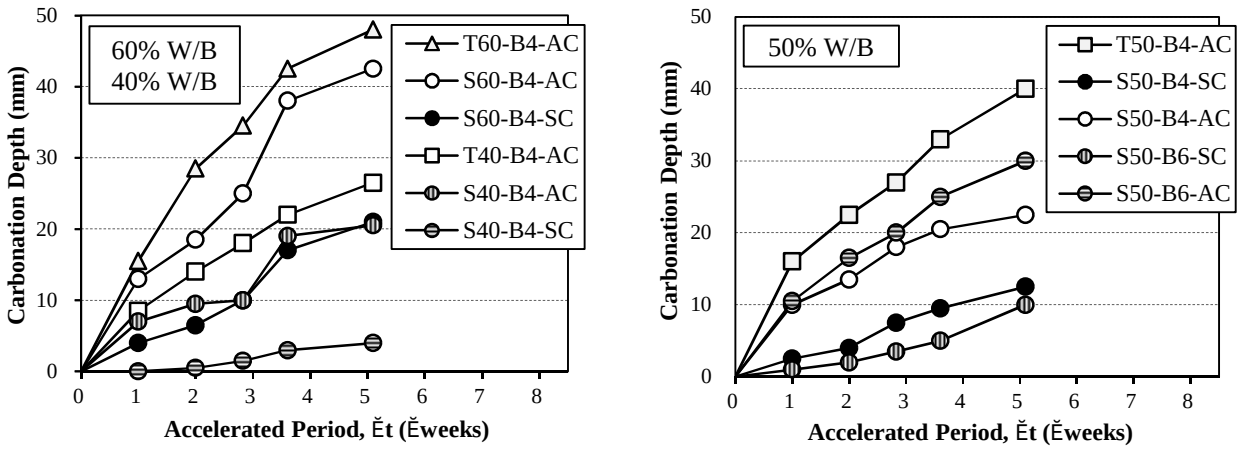


Fig. 4 Progress of carbonation of GGBS concrete in various curing

Effect of Mixing Water and Mineral Admixtures

Fig. 5 presents the carbonation ratio of seawater mixed to tap water mixed (S/T) in AC for both accelerated carbonation and natural exposure carbonation. It was clearly seen that carbonation ratio of all concrete specimens was below line of equality. It implies that seawater mixing decreases carbonation rate of concrete comparing to tap water mixing. Generally, the carbonation ratio of OPC concrete was decreased than that of FA concrete and GGBS concrete. It means that mineral admixtures increase carbonation of seawater mixed concrete, although the accelerated carbonation ratio of GGBS concrete in AC was lower than OPC concrete. In other words, the effectiveness of seawater mixing on carbonation resistance is larger for a OPC concrete than concrete incorporating mineral admixtures. This phenomenon was also revealed in compressive strength of concrete, that the strength ratio of seawater mixed to tap water mixed of GGBS concrete in AC is higher than OPC concrete, as reported by another reference (Adiwijaya et al.[2] 2014).

The influence of replacement ratio of FA on carbonation rate of seawater mixed concrete was obtained, also. The carbonation rate of seawater mixed in SC and AC increased with increasing FA replacement ratio in accelerated carbonation, as shown in Fig. 6. Nevertheless, in natural exposure carbonation, carbonation rate of 20% FA replacement and 30% FA replacement was nearly similar. In addition, the rate of carbonation of seawater-mixed $GGBS_{4000}$ (B_{4000}) was lowered compared to $GGBS_{6000}$ (B_{6000}) in air curing, AC. On the other hand, carbonation rate of B_{4000} was higher than that of B_{6000}, at the time when it was cured in seawater, as depicted Fig. 7. It suggests that the fineness impact of GGBS on carbonation rate of seawater mixing depends on curing conditions of concrete specimen.

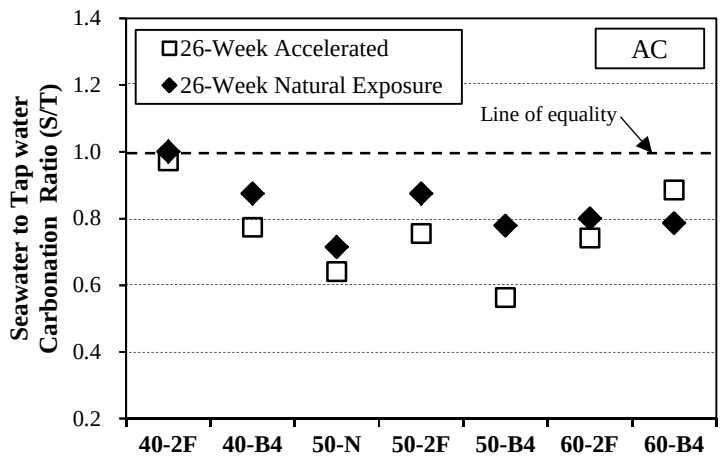


Fig. 5 Carbonation ratio of seawater-mixed concrete to tap water-mixed concrete (S/T)

Influence of Curing Conditions

Carbonation rate of seawater mixed concrete is illustrated in Fig. 8. It can be seen that the rate of carbonation of concrete specimens in air curing, AC is significantly higher than that in tap water curing, TC and seawater curing, SC. It means that carbonation rate of seawater mixed concrete with and without mineral admixtures is strongly influenced by

type of curing condition. This phenomenon can be stated logically as cement hydration in AC is not perfectly hydrated, which causes pore structures in air curing are larger than water curing, hence CO_2 gas is easily penetrate into concrete.

Furthermore, it was also found that the carbonation rate of seawater-mixed concrete in TC was higher than that in SC. It indicates that seawater curing (SC) increases the carbonation resistance of seawater mixed concrete with and without mineral admixture. It expresses that carbonation rate of seawater mixed concrete is affected by type of curing water, TC or SC. When seawater mixed concrete was cured in TC, chloride ions may be leached out, thus the efficacy of chloride ions in promoting hydration reaction of cement paste becomes lower. As result, pore structure of concrete is not effectively diminished, then carbonation rate of concrete is accelerated, accordingly.

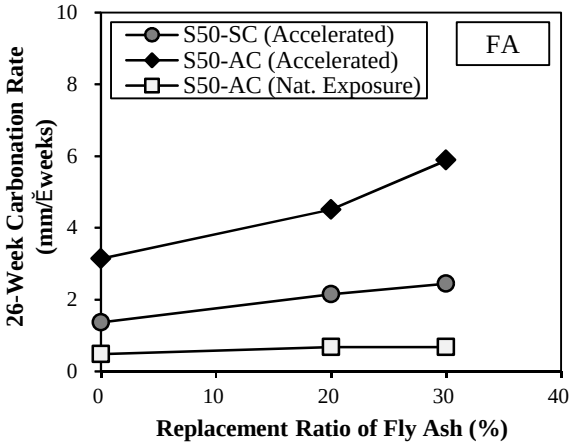


Fig. 6 Effect of FA replacement ratio on carbonation rate

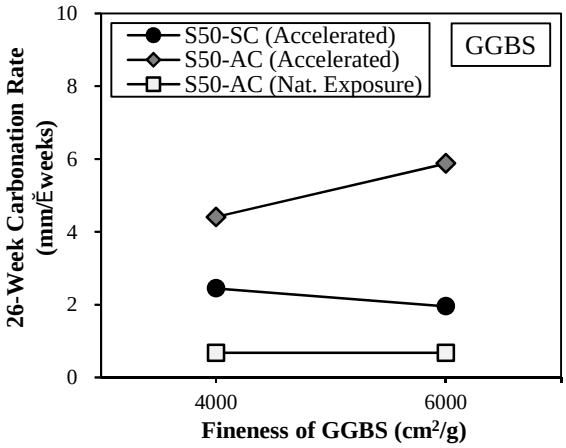


Fig. 7 Effect of GGBS fineness on carbonation rate

Effect of Water-Binder Ratio

Fig. 9 depicts carbonation rate of FA concrete and GGBS concrete for both accelerated and natural exposure carbonation. It was observed that the rate of carbonation of seawater mixed and tap water mixed was increased with the increasing water-binder ratio, W/B. From this result, it can be stated that the carbonation rate of seawater mixed FA concrete and GGBS concrete is significantly influenced by water to binder ratio. The larger water-cement ratio is, the larger carbonation rate is, which is not affected by mixing water and curing conditions. This phenomenon is also revealed in author's previous study, without FA and GGBS (Adiwijaya et al.[1] 2014). Even though carbonation rate of concrete in natural exposure carbonation is much lower than accelerated carbonation, the trend of relationship between carbonation rate and water-cement ratio are almost similar for both FA concrete and GGBS concrete.

Effect of Exposure Condition

The results of carbonation depth in natural exposure condition at 6 months, 12 months and 18 months exposure period for both tap water mixed concrete and seawater mixed concrete are shown in Fig. 10. After 4-week curing in air at 20°C, 60% R.H. controlled room, the concrete specimens were placed in unsheltered exposure site (outdoor) in Itoshisma, Fukuoka, Japan, under four seasons, from winter, spring, summer, and autumn. The average temperature surrounding condition of concrete specimens was ranging from 7°C to 30°C. It was observed that the depth of carbonation in natural exposure of seawater mixing was also lower than that of tap water mixing. The trend of natural

exposure carbonation is the same as accelerated carbonation, where carbonation depth of FA concrete is lower than that of GGBS concrete, as mentioned in earlier discussion.

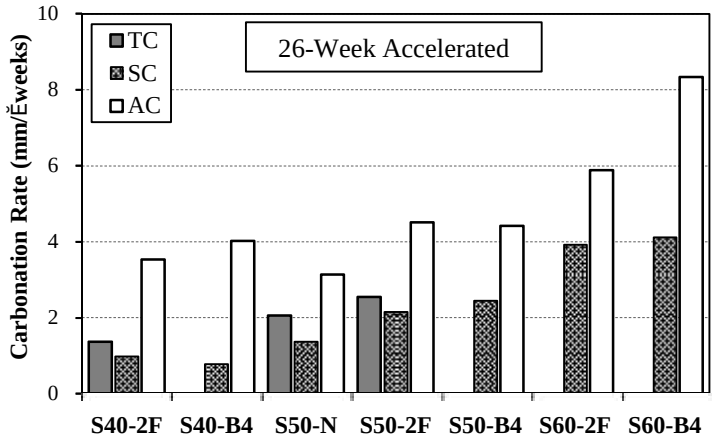


Fig. 8 Effect of curing conditions on carbonation rate of seawater mixed

In this study, the correlation between 26-week accelerated carbonation (X_{acc}) and 26-week natural exposure carbonation (X_{nat}) was also obtained, as demonstrated in Fig. 11. From this figure, it can be seen that accelerated carbonation was seven times higher than that of natural exposure carbonation in both seawater mixed and tap water mixed for FA concrete and GGBS concrete. This finding evidences that environmental condition such as humidity, carbon dioxide concentration and temperature where concrete specimen is placed plays important role in accelerating carbonation rate of tap water mixed and seawater mixed concrete incorporating mineral admixture. This phenomenon corresponds with reference, as mentioned on aforementioned introduction.

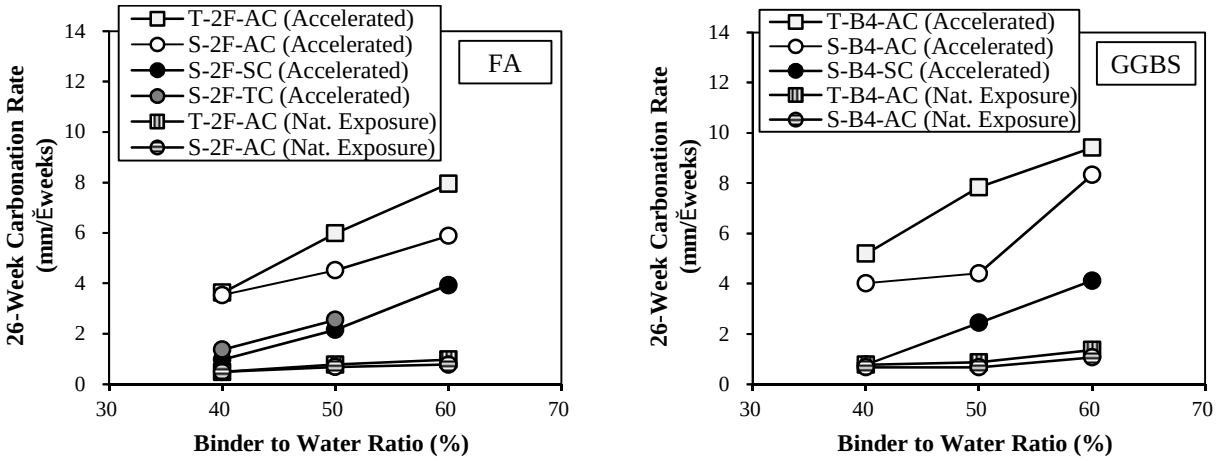


Fig. 9 Effect of water-binder ratio (W/B) on carbonation rate

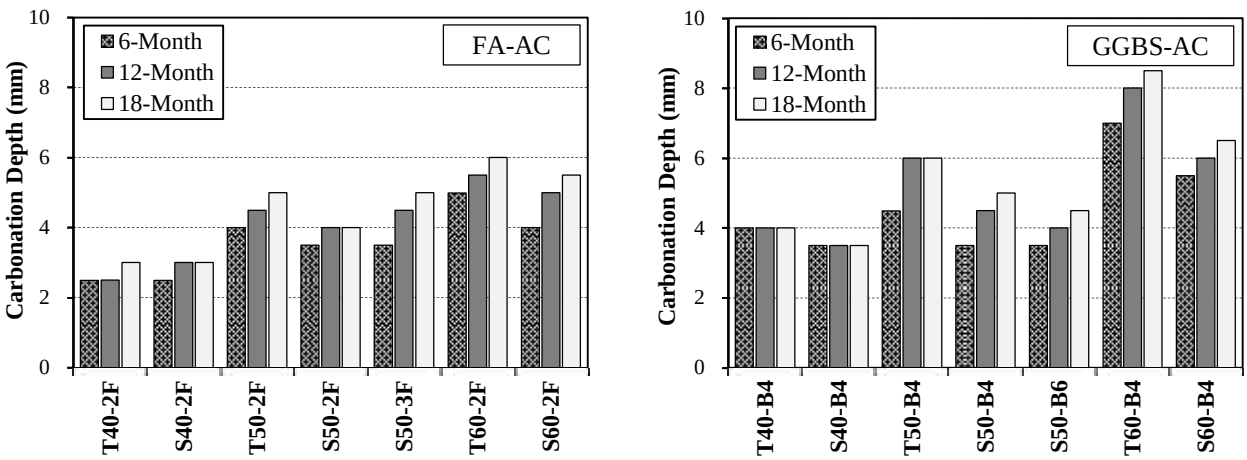


Fig. 10 Progress in natural exposure carbonation of concrete specimens

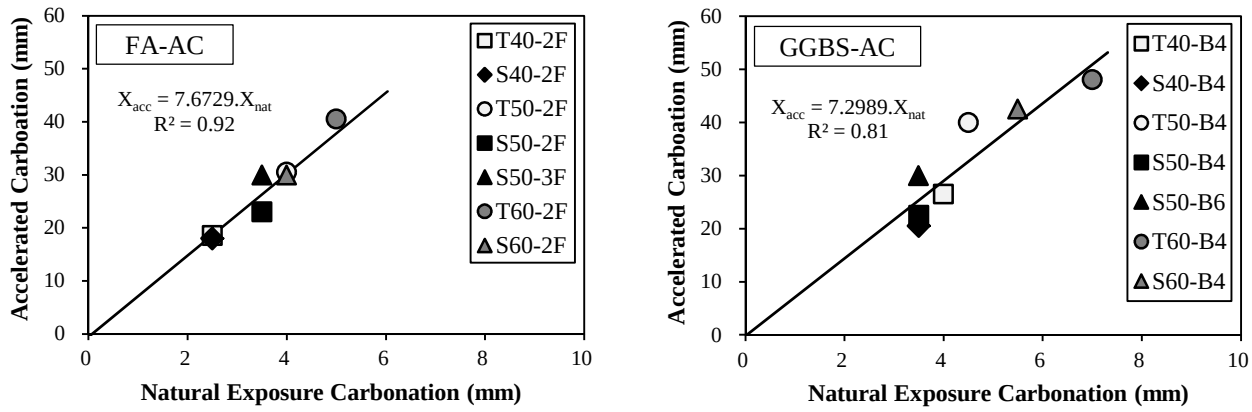
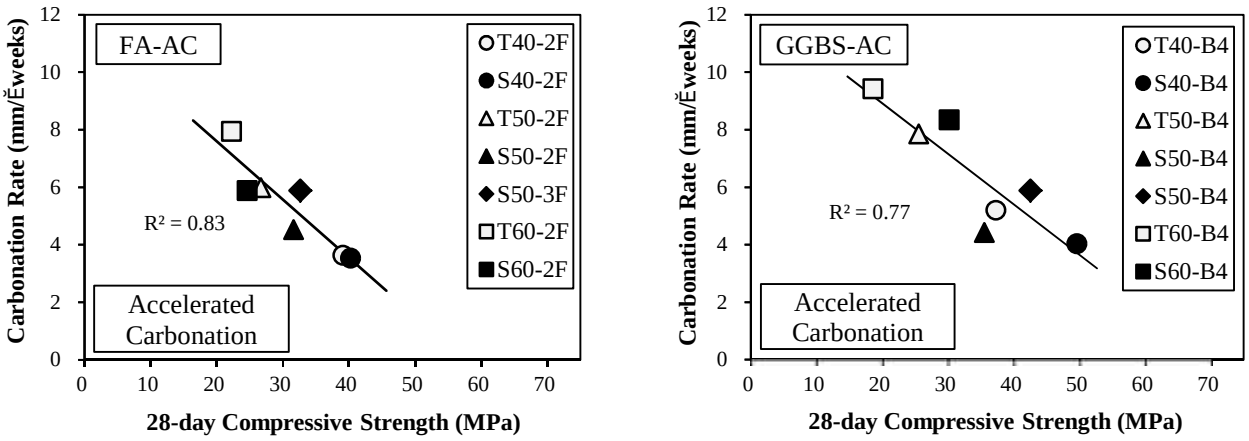


Fig. 11 Accelerated carbonation at 26 weeks (X_{acc}) vs. Natural exposure carbonation at 26 weeks (X_{nat})

Compressive Strength vs. Carbonation

Fig. 12 presents the relationship between compressive strength 28 days and 26 weeks accelerated carbonation rate of concrete specimens. From this figure, it points out that carbonation rate decreases with an increase in compressive strength. A good relationship was obtained for both FA concrete and GGBS concrete in both SC and AC. The strength-carbonation relationship is strongly achieved in FA concrete compared with GGBS concrete in accelerated carbonation, in which the correlation coefficient, R^2 for FA concrete in AC and SC are 0.83 and 0.90, respectively. While, GGBS concrete in AC and SC are respective 0.77 and 0.92 in correlation coefficient. It expresses that 28-day compressive strength can be used as indicator to control carbonation rate of seawater-mixed and tap water-mixed of FA concrete and GGBS concrete. This finding is in agreement with the findings of other researches (Atis 2003 and Sulapha et al. 2003).

Similarly, a linearly strong relationship was also acquired for FA concrete in natural exposure carbonation, as shown in Fig. 13, which was 0.94 in R^2. In general, it can be concluded that relationship between compressive strength and carbonation of seawater mixing and tap water mixing is obtained to be valid and it is linear form.



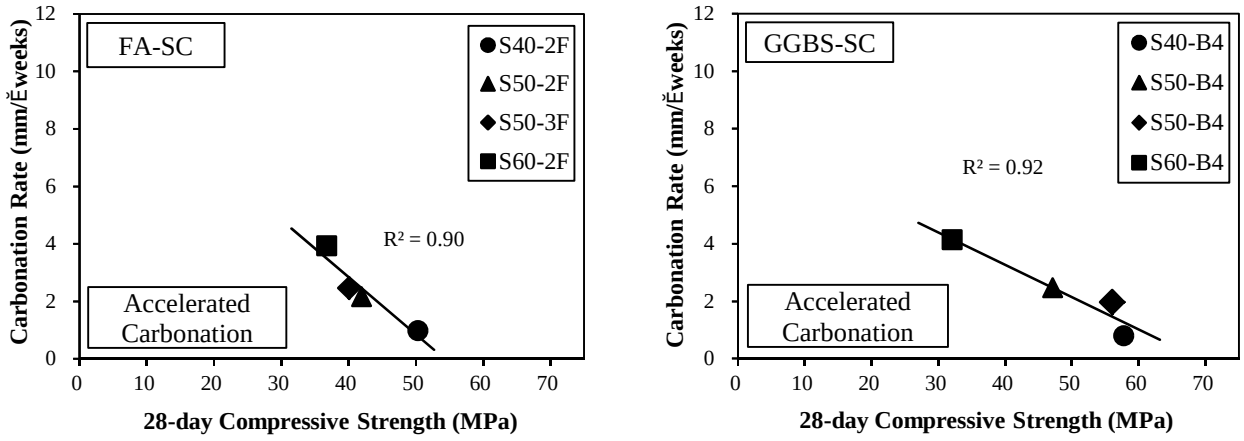


Fig. 12 Relationship between compressive strength and accelerated carbonation at 26 weeks

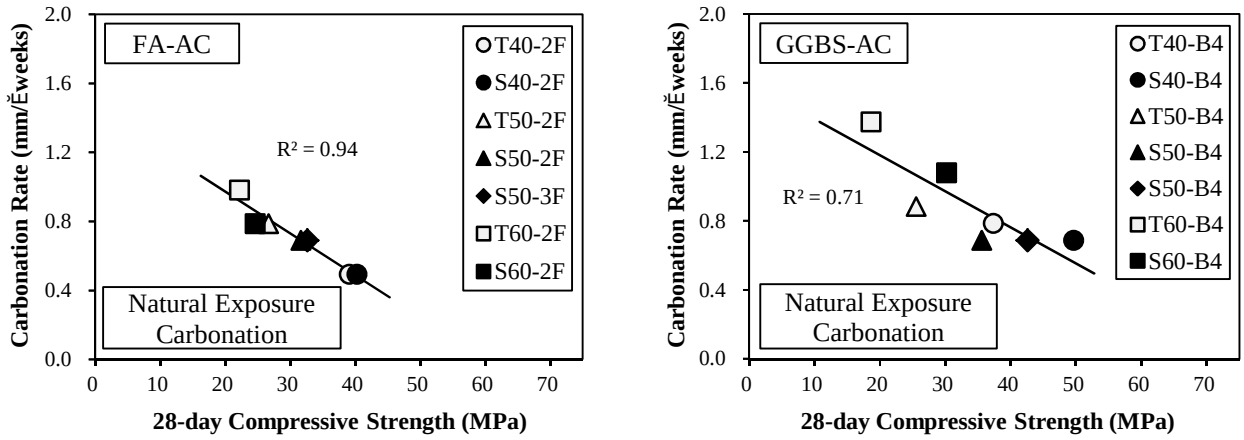


Fig. 13 Relationship between compressive strength and natural exposure carbonation at 26 weeks

CONCLUSIONS

Based on the results of this experimental study, the following conclusions can be drawn,

1. In general, seawater mixed FA and GGBS concrete is improved in compressive strength up to 91 days compared with tap water mixed in all curing condition, in TC, SC and AC. In addition, compressive strength for both FA and GGBS concrete is not influenced by type of curing water, TC or SC.
2. Mineral admixtures increase carbonation rate of seawater mixed concrete in TC, SC and AC compared with seawater mixed OPC concrete.
3. The carbonation depth of seawater mixed concrete with or without mineral admixtures in AC was decreased than that of tap water mixed concrete for both accelerated and natural exposure carbonation. However, both seawater mixing and tap water mixing were nearly similar in SC.
4. The rate of carbonation of seawater mixed concrete with and without mineral admixtures in seawater curing, SC was lower than that of in tap water curing, TC.
5. The carbonation rate of seawater mixed FA and GGBS concrete is strongly affected by water to binder ratio, which the higher of water-binder ratio is the higher carbonation rate is, in TC, SC and AC.
6. The accelerated carbonation of tap water mixing and seawater mixing FA and GGBS concrete was seven times higher than that of natural exposure carbonation.

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APPLICATION OF SACRIFICIAL ANODE TO PREVENT MACROCELL CORROSION IN REPAIRED RC MEMBER

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ABSTRACT: This paper presents results of an experimental investigation on the effectiveness of sacrificial anode to prevent macrocell corrosion of steel in repaired RC member. Experimental tests were carried out in reinforced concrete beams with sizes of 150 mm x 150 mm x 500 mm and cover thickness of 30 mm. Each specimen consisted of plain steel bar (PS), plain steel bar with sacrificial anode (PSCP) and epoxy coated steel bar (PSE). Mix proportion was divided into two categories; firstly, existing concrete (EC) with 4 kg/m^3 and 10 kg/m^3 of chloride and W/C ratio of 53.5% and 40% were placed, namely N53.5, N40 and BB40. For BB40, replacement of cement with 50% of blast furnace slag (BFS) was added. Secondly, concrete with 47% of W/C ratio without chloride was placed as repaired concrete (RPC). The potential value of steel, depolarization tests, anodic-cathodic polarization curve, polarization behavior of sacrificial anode, and visual observation were evaluated after one-year exposure. From the test results, PSCP showed good passivation film and little corrosion than PS in RPC. It means sacrificial anode is effective to delay macrocell corrosion repaired section. Also, replacement of cement by 50% BFS can raise the resistance to penetration of chloride ions into repaired concrete.

Keywords: Sacrificial anode, corrosion of steel, repaired concrete, polarization curve

INTRODUCTION

Usually, the steel embedded in concrete is protected because a high alkalinity resulting from hydration cement in concrete. However, corrosion will take place by the presence of chloride ions and carbonation. The corrosion product (iron oxides and hydroxides) can lead to concrete cracking, spalling and delamination of concrete. To prevent continuing deterioration, repair technique needs to be undertaken (Tabatabai et al. 2014). Unfortunately, many patch repairs exhibited the new corrosion damage only after a few months to a year (Qian et al. 2006).

Macrocell corrosion occurs when an electric current runs through an active and a passive area of steel. It can also occur following a patch repair of reinforced concrete member, when there is a significant difference in chloride concentration between the new patch (chloride free) and its surrounding substrate (chloride contaminated).

The corrosion rate may be very high because the passive steel surface in the patched area often shows very positive potentials the effect of which will be a very large driving force (Raupach 1996). It means the larger the difference in corrosion potential between these areas the larger the macrocell current becomes. One reason why macrocell corrosion is more severe is that negative ions like Cl^- may migrate to the anode in higher amounts than in microcell corrosion, because of the high positive potential in this area and the high current. This means that the chlorides can move to the corroded area, and the corrosion may then be accelerated. At the same time the positively charged and dissolved Fe^{2+}, migrate to the cathode zone with less volume expanding corrosion products as a consequence (Skoglund et al. 2008).

Cathodic protection (CP) has been introduced to be an effective technique to stop the corrosion of embedded steel bar in concrete. One of many different kinds of CP methods is the sacrificial point anode method (galvanic anode). Sacrificial point anode system is aimed to shift the potential of steel to the least probable range for corrosion.

Fig. 1 Sacrificial point anode

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Nowadays, sacrificial point anode is commercially available. These are typically made from zinc nucleus covered by a highly alkaline cement mortar with lithium hydroxide (LiOH) to maintain zinc corrosion activation. The type and sizes of this sacrificial point anode are variable, depending on the fabricating and distributing company. The anode used in this study is commercially available with sizes of 60 mm in diameter and 30 mm in thickness as shown in Fig. 1. This paper studies the effectiveness of commercially available sacrificial point anode to prevent macrocell corrosion of steel bars embedded in repaired concrete.

EXPERIMENTAL PROCEDURES

Preparation of Specimens

Ordinary Portland Cement (OPC), crushed river stone and sea sand were used. The physical properties of concrete materials are presented in Table 1. Concrete with water to cement ratio (W/C) of 47% without chloride was used for repaired concrete (RPC). For Existing concrete (EC), concrete with W/C of 53.5% and 40% was prepared, namely N53.5, N40 and BB40 respectively. The initial chloride content was set at 4 kg/m^3 or 10 kg/m^3. For BB40, OPC were replaced by 50% BFS. The mix proportions of concrete are described in Table 2.

Table 1 Physical properties of concrete material

Material	Description
Cement	Ordinary Portland Cement (OPC), density = 3.16 g/cm^3, SSA = 3390 cm^2/g
Mineral admixture	Blast Furnace Slag (BFS), density = 2.91 g/cm^3 SSA = 4060 cm^2/g
Sand	Sea sand (washed), density = 2.58 g/cm^3, F.M = 2.77
Gravel	Crushed stone, density = 2.85 g/cm^3, MSA = 20 mm

SSA: Specific Surface Area; F.M: Fineness Modulus; MSA: Maximum Size of Aggregates

Table 2 Mix proportions of concrete

Material	Existing concrete (EC)			Repaired concrete (RPC)
	N53.5	N40	BB40	
W/C, %	53.5	40	40	47
Water, kg/m^3	166	161	161	164
Cement (OPC), kg/m^3	310	403	201	350
BFS, kg/m^3	0	0	201	0
Sand, kg/m^3	719	680	680	844
Gravel, kg/m^3	1141	1102	1102	953
Chloride, kg/m^3	4 or 10	4 or 10	4 or 10	0
Slump, cm	8.0	6.5	9.5	7.3
Air content, %	3.8	4.0	3.5	4.0

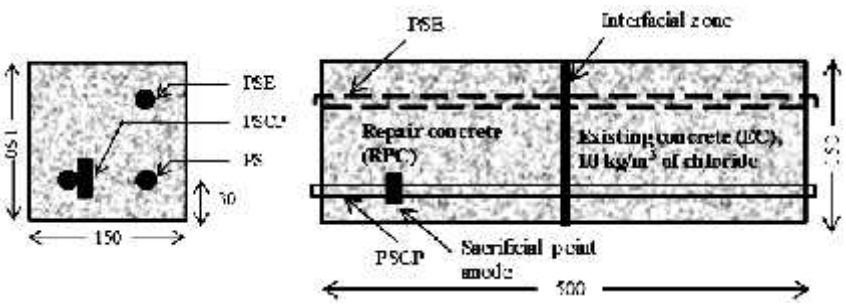


Fig. 2 Layout of the specimen

The specimen was fabricated in the form of concrete prism with dimensions of 150 mm x 150 mm x 500 mm and cover thickness of 30 mm. Each specimen consist of plain steel bar (PS), plain steel bar with the point anodes (PSCP) and epoxy coated steel bar (PSE). Reinforced concrete prisms specimen was casted in two layers. EC containing chloride of 4 kg/m^3 or 10 kg/m^3 was cast at first. Specimen was demoulded after 24 hours and cured by wet towel until 14 days. After curing for 14 days, the point anode was fixed in steel bars. Then, RPC was placed and demoulded after 24 hours and cured again by wet towel until 28 days. The layout of specimen is shown in Fig. 2. Six test prisms were

prepared as summarized in Table 3. In addition, specimens were kept in air curing (20±2°C, RH: 60%) during test periods. Exposure period was around 12 months.

Table 3 List of specimens test

No	Specimen series	W/C (%)	Chloride (kg/m^3)
1	N53.5 – 4	53.5	4
2	N53.5 – 10		10
3	N40 – 4	40	4
4	N40 – 10		10
5	BB40 – 4	40	4
6	BB40 – 10		10

Electrochemical Tests and Visual Observation

Electrochemical tests include potential tests, depolarization tests, polarization behavior, and anodic-cathodic polarization curve were carried out. Depolarization tests, polarization behavior, anodic-cathodic polarization curve were performed for the specimen with 10 kg/m^3 of chloride at the end of test periods. These tests were conducted 24 hours after removing the sacrificial point anode from steel bar. In addition, visual observation of steel bar was carried out after completely remove from the concrete specimen.

Potential value

The time-dependent change potential of PS and PSCP in repaired concrete (RPC) with 10 kg/m^3 of chloride was monitored by using rebar corrosion meter contacted to the surface of specimen. Contact method is a measuring method using double counter electrode.

On the other hand, evaluation of the potential value of each steel bar at the end of test periods was taken in the three locations namely interfacial zone, RPC and EC. The potential value of RPC and EC were taken at ±2.5 cm from the interfacial zone. Both half-cell potential of PS and the "On-potential" of PSCP were recorded by using a digital voltmeter (Yokogawa) and the silver/silver chloride reference electrode (Ag/AgCl). Potential value is converted to the copper/copper sulfate reference electrode (CSE).

The interpretation of potential readings of PS was carried out according to ASTM C876-09 (1999) as described in Table 4. Meanwhile, for PSCP it is said that the protective potential of steel bar is lower than -750 mV vs. CSE (Rincon et al. 2008).

Table 4 Corrosion probability (ASTM C876-09)

Half-cell potential (mV; CSE)	Corrosion activity
-200 < E	90% no corrosion
-350 < E < -200	Uncertainty of corrosion
E < -350	90% corrosion

Depolarization tests

The difference between 24 hours off potential and instant-off potential is defined as depolarization value. When cathodic protection is applied to steel in concrete, it is commonly considered as "effective" if the 24 hours depolarization value is more than 100 mV based on JSCE Concrete Library 107 (2011). In this study, 300 mV cathodic polarization value (on-potential – natural potential), was used to demonstrate the effectiveness of the sacrificial point anode.

Polarization behavior

Polarization behavior of the sacrificial point anode was measured after depolarization tests. Anodic polarization curve of sacrificial point anode and cathodic polarization curve of steel bar were obtained by the contact method. The contact method is a measuring method using double layer counter electrode contacted on the surface of the specimen. Potentiostat HA-151A (Hokuto Denko) and function generator HB-III (Hokuto Denko) were used to maintain potential and current.

Anodic and cathodic polarization curve

Anodic polarization curve is related to the passivity condition of steel bars. When the current density becomes larger, the grade of passivity film of steel bars becomes worse (Otsuki 1985). Cathodic polarization curve is related to the diffusion of oxygen (O_2). When the current density becomes larger, the level of O_2 diffusion becomes larger. For this, similar testing equipment in the polarization behavior test was used. The potential of the steel bar (E_{corr}) was shifted to +700 mV for anodic polarization curve and -700 mV for cathodic polarization curve from the natural potential with a scan speed of 1 mV/sec. The current at each potential sweep was recorded.

Visual observation

At the end of the tests, three specimens with 10 kg/m^3 of chloride (N53.5 – 10, N40 – 10 and BB40 – 10) were open and the steel bar was visually examined for checking the corrosion condition.

RESULTS AND DISCUSSION

Half-cell Potential of Plain Steel Bar (PS)

The time dependent change in half-cell potential of plain steel bar (PS) in repaired concrete (RPC) is presented in Fig. 3a. For N40 and BB40, it is observed that the half-cell potential of PS showed a stable value around -250 mV after 12 month. While, PS in N53.5 show the half-cell potential changed from -150 mV to -275 mV at the end test periods.

The evaluation of half-cell potential of PS after 12 months exposure is shown in Fig. 3b. In EC contained 4 kg/m^3 of chloride, the half-cell potential of PS was around -170 mV, 250 mV and -370 mV for BB40, N40 and N53.5 respectively. Comparing that with 10 kg/m^3 of chloride, the half-cell potential of PS has become more negative, around -220 mV, -420 mV and -400 mV for BB40, N40 and N53.5 respectively. This is attributed to the effect of chloride, which is accelerating corrosion of the steel bars.

From Fig. 3b, it is also observed that the half-cell potential of PS shows more negative value in repaired area for N53.5 than N40 and BB40. It is due to the potential difference between steel bar in repaired and existing concrete is higher. Thus, the larger of the macrocell current becomes and visual observation reveals that the larger macrocell corrosion existed in N53.5. In addition, the lower W/C ratio could delay the chloride ions transport to the repaired area.

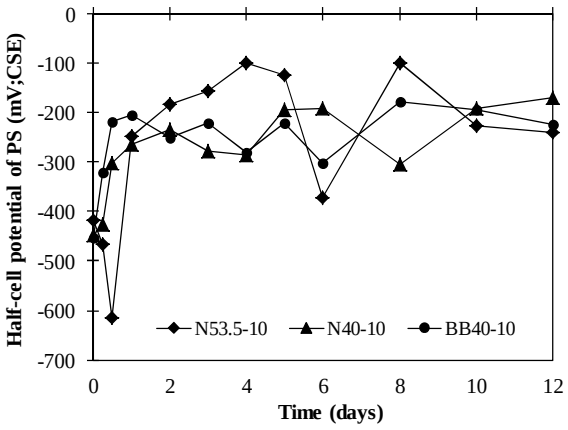


Fig. 3a Half-cell potential of PS versus time (RPC)

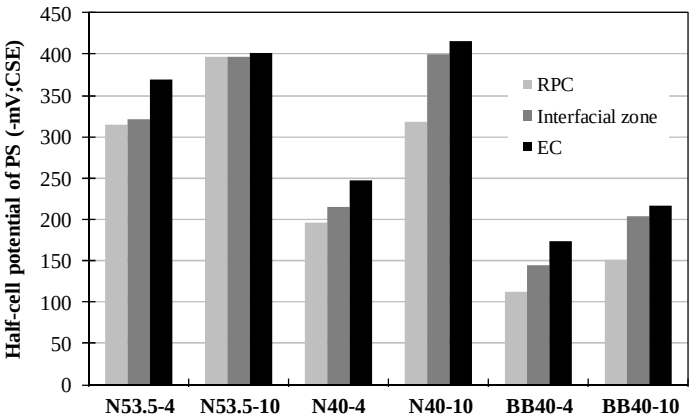


Fig. 3b Half-cell potential of PS at the end of test periods

On Potential of Plain Steel Bar with Sacrificial Anode (PSCP)

Fig. 4a shows the on-potential of steel bar with sacrificial anode (PSCP) versus time in repaired concrete (RPC). It is shown that the sacrificial anode active at an early age and reached the potential value around -1000 mV for N53.5 and BB40. While for N40, the potential value is higher than in N53.5 and BB40, which was around -800 mV. This potential difference at the early age may due to the high resistivity of concrete in N40. However, after four months the potential shifted to noble value about -250 mV for all mix concrete. Even though the potential value is shifted to noble value, the potential of steel bar in the stable condition during exposure periods. It indicates that the steel bar is protected.

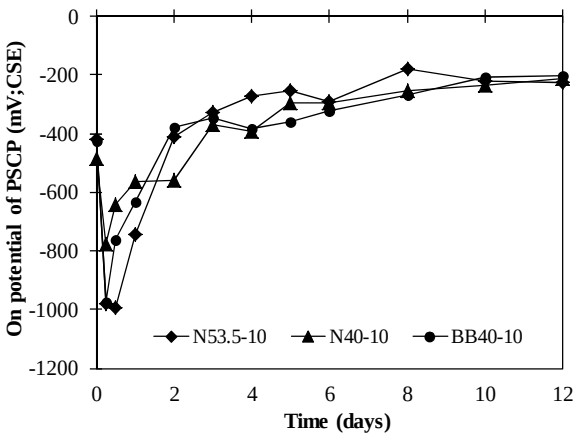


Fig. 4a On-potential of PSCP versus time (RPC)

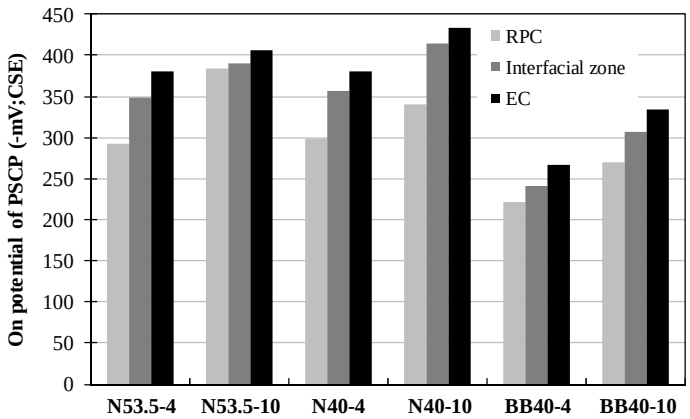


Fig. 4b On-potential of PSCP at the end of test periods

Fig. 4b represents the evaluation of the on potential of plain steel bar with sacrificial anode (PSCP) at the end of test periods. It is observed that the on potential of steel bar varied between -200 to – 450 mV. This value is higher than the potential protective of cathodic protection, around -750 mV vs. CSE. Also, BB40 showed the higher on-potential value compare to N53.5 and N40. On reason might be due to the high resistivity of concrete, which decrease the current output of sacrificial anode to the steel bar.

Although the protective potential of steel bar higher than the protective level requires, visual observation of PSCP show no corrosion appearance in repaired area (RPC). It means that the sacrificial anode can delay the macrocell corrosion in repaired section. From this test results, it can be said that the protection level criteria of this sacrificial anode depends on the environmental condition of concrete. In the present study, all specimens were exposed in the air curing.

Half-cell Potential of Epoxy Coated Steel Bar (PSE)

Fig. 5 shows the half-cell potential of epoxy coated steel bar (PSE) at the end of test periods. It is observed that the half-cell potential of PSE more positive than in PS. The value of 4 kg/m^3 of chloride show the potential value around -100 to -300 mV. While concrete with 10 kg/m^3 of chloride shows the potential value between -150 to 330 mV. This indicates that the PSE provided excellent corrosion protection than PS.

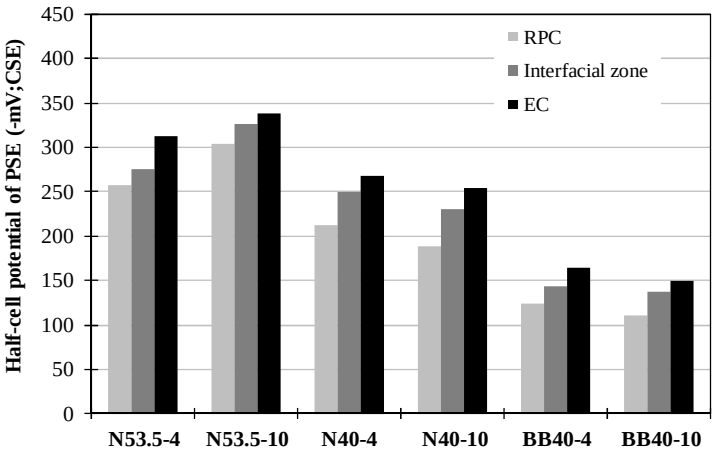


Fig. 5 The half-cell potential of PSE

Depolarization Tests

At the end of test periods, three specimens with 10 kg/m^3 of chloride (N53.5-10, N40-10 and BB40-10) were open and the sacrificial anode removed from the steel bars. Fig. 6 shows the specimen condition after removing the sacrificial anode from the steel bar and measurement of the rest potential. Table 5 shows the depolarization value of steel bar taken at RPC area after 24 hours remove from the sacrificial anode. It is observed that the depolarization value PSCP less than 300 mV in this condition. It is due to the low humidity condition, which decrease the protective potential of the sacrificial anode. However, the rest potential (after 24 hours off potential) of PSCP is around -250 mV as shown in Fig. 7, which indicates that 90% no corrosion probability per ASTM C876-95 (1999). In addition, it was also observed that the rest potential of sacrificial anode is about -800 mV vs. CSE.

Fig. 6 Specimen condition after removing the anode

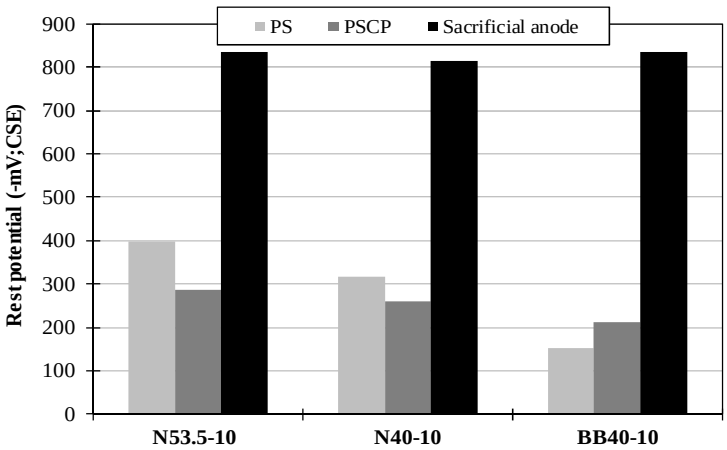


Fig. 7 The rest potential of steel bar and sacrificial anode

Table 5 Depolarization value and the 24h off potential of steel bar

Specimen	On-potential (mV vs. CSE)	24h off potential (mV vs. CSE)	Depolarization value (mV vs. CSE)
N53.5 – 10	-384	-287	97
N40 – 10	-341	-261	80
BB40 – 10	-270	-211	29

Polarization Behavior of Sacrificial Anode

Fig. 8 shows the polarization behavior of sacrificial anode after 12 months exposure. The current densities of the sacrificial anode were around 0.25 μA/cm^2, 0.5 μA/cm^2 and 0.7 μA/cm^2 for N53.5-10, BB40-10 and N40-10 respectively. As reported by previous studies (Bertolini et al. 1993 and Pedeferri 1996), the current density of 0.2 μA/cm^2 is sufficient for cathodic prevention before corrosion initiation. From the test results, the current densities of all sacrificial anode are still over the range of cathodic prevention of steel bar in concrete.

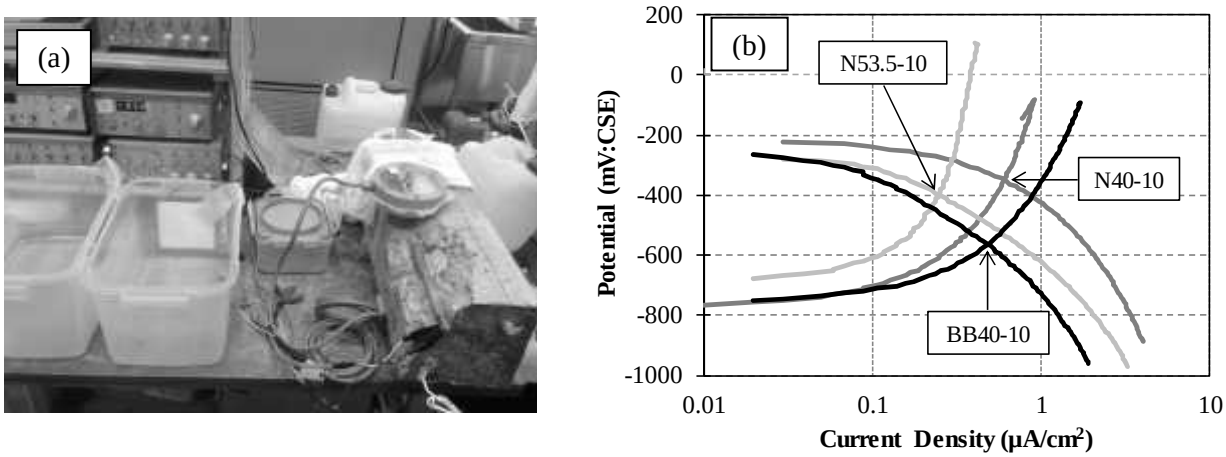


Fig. 8 Polarization behavior of sacrificial anode: (a) Measurement by contact method; (b) Anodic polarization behavior of the sacrificial anode and cathodic polarization curve of steel bar

Anodic-Cathodic Polarization Curve

The anodic-cathodic polarization curve of steel bar is shown in Fig. 9. Except N40, it is observed that no significant different in anodic-cathodic polarization curve between PSCP and PS for all mix concrete. It means that the sacrificial anode does not influence on the anodic-cathodic polarization curve of steel bar exposed in the air curing.

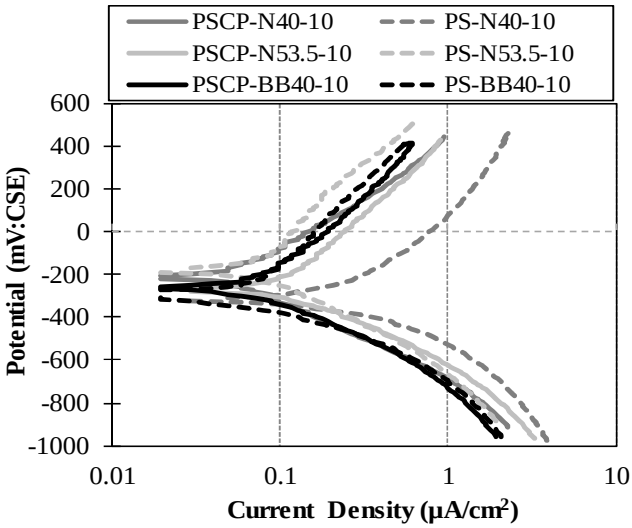
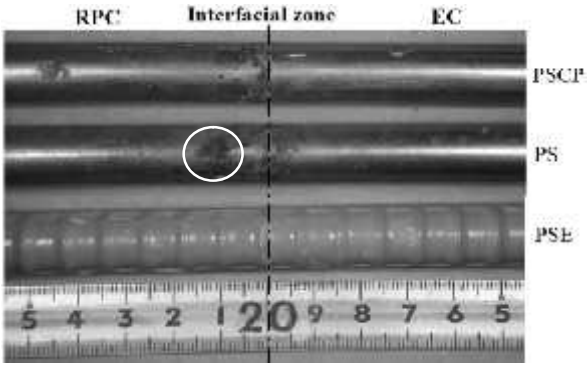


Fig. 9 Anodic-cathodic polarization curve of steel bars

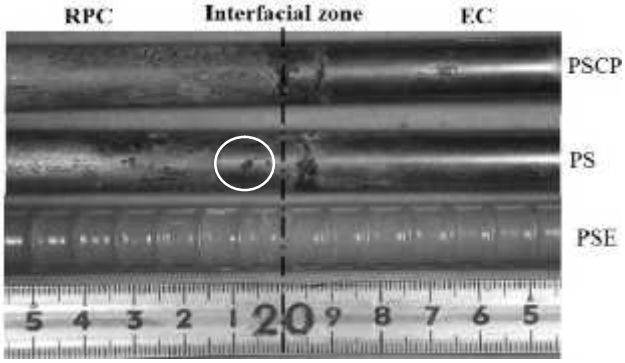
Visual Observation

Fig. 10 shows the results of visual observation of steel bars in concrete with 10 kg/m^3 of chloride. It was observed that PS for N53.5 exhibited larger corrosion area than N40 and BB40 in repaired area. It seems that the lower W/C ratio

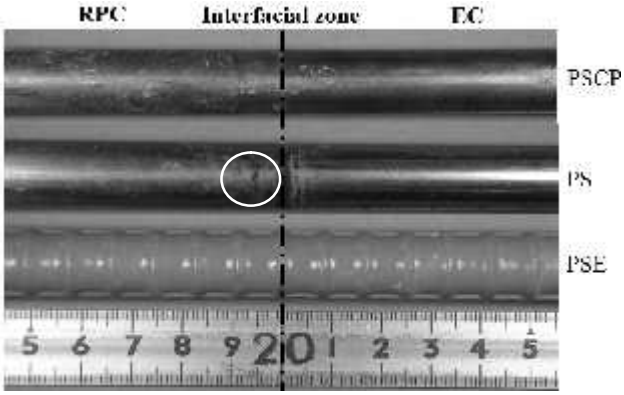
gave influence in preventing macrocell corrosion in RPC. Also, replacement of cement by 50% BFS showed a good condition. On the other hand, it is clearly seen that PSCP showed no corrosion appearance in RPC in all mixed concrete. The results indicate that the sacrificial anode can delay anodic reaction in RPC even if the protective potential higher than -850 mV vs. CSE during exposure. In addition, it was also confirmed that PSE show a good resistance against corrosion.



(a) N53.5 – 10



(b) N40 – 10



(c) BB40 – 10

Fig. 10 Visual observation of steel bars

CONCLUDING REMARKS

The use of fresh repair material in chloride contaminated concrete may accelerate new corrosion in repaired area due to the different potential between repaired concrete (passive condition) and existing concrete (active condition). Chloride ions level in existing concrete (EC) significantly influence on the corrosion of steel bar at repaired area.

The application of sacrificial anode in repaired area was effective in preventing macrocell corrosion and no corrosion was shown on the steel surface and this was confirmed by visual observation of the steel bar after removing from the specimen. Also, the lower W/C ratio and partial replacement of cement by 50% BFS in existing concrete (EC) could significantly prevent macrocell corrosion of steel bar in RPC and raise the resistance to penetration of chloride ions into repair concrete.

ACKNOWLEDGEMENTS

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EXPERIMENTAL STUDY OF ENGINEERED CEMENTITIOUS COMPOSITE MATERIAL FOR STRUCTURAL APPLICATION

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ABSTRACT: Engineered cementitious composite (ECC) is commonly used in the research field to obtain the ductile behavior of concrete. ECC itself addresses a unique type of high performance fiber-reinforced cementitious composites with high tensile strain of more than 2% compared to 0.01% of conventional concrete. On the other hand, extensive inelastic deformation in ECC is achieved via multiple micro-cracks with width of less than 80 μm. This study purposes the use of local material i.e. silica sand and lumajang sand for ECC specimen in order to obtain the high tensile strain capacity. Polyvinyl alcohol fibers (PVA) were employed in this study because of the characteristic as reinforcing material for cementitious composites. The experimental investigation was carried out in compression test and direct tensile test of ECC (M45) due to the effect of different type of fine aggregate. The results showed that both ECC specimens exhibited the large of strain hardening capacity through the multiple micro-cracking due to the use of PVA fiber. Furthermore, through this test, it can be confirmed that ECC with high tensile strain capacity and micro-cracks behavior has much feasibility to upgrade the structural performance.

Keywords: *Engineered cementitious composite (ECC), polyvinyl alcohol fibers (PVA), silica sand, lumajang sand, strain hardening capacity, multiple micro-cracking.*

INTRODUCTION

In the last several decades, concrete with high compressive strength have been commonly used for potential civil engineering application. However, as the material goes up to the higher strength, the behavior of the material becomes more brittle. This certainly can cause the lack of ductility of structure during the seismic response due to its brittleness. Furthermore, the potential danger can worsen the behavior of structure because of the significant fracture failure in the concrete (Li, 1998). As the matter of fact, it is strongly needed to develop the high performance elements for seismic structural applications which is based on a new materials technology.

Engineered Cementitious Composites (ECC) addresses a special type of high performance fiber-reinforced cementitious composites with high tensile ductility (Sahmaran & Li, 2008). Fibers have been used to improve the toughness of quasi-brittle cement based material (Roth, 2008). As the ability to produce high tensile ductility, ECC is originally designed strain-hardening behavior using micromechanical concept, thus the excessive tensile strain capacity can be achieved more than 2% through multiple cracking (Lee et.al. 2010).

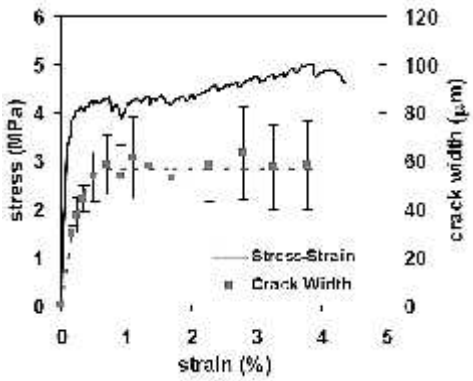

Fig. 1 Typical tensile stress-strain response and crack width of ECC-M45 (Yang et.al 2008)

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The typical stress-strain response from direct tensile test by applying 2% volume of PVA fiber can be seen in Fig. 1. Strain hardening of ECC after first cracking is several hundred times greater than conventional concrete (Yang et.al. 2008). Also, the tight crack widths occurs in ECC, less than 80 μm (Sahmaran & Li, 2008). Compared to the Fiber Reinforced Concrete (FRP) which gives tension-softening after the first cracking, ECC performs the raising stress accompanied by increasing strain (Li, 1998). When the strain capacity is exceeded, the material continues to the tension softening behavior throughout failure. In addition to its pseudo-strain hardening branch, ECC also has high fracture toughness, isotropic properties (Li 1994) and remains ductile even when subjected to high shear stress (Li et al. 1994).

Based on the unique performance, ECC is suitable to be applied for critical members of structure in seismic applications. The use of ECC in seismic structures is expected to meet damage tolerant and high dissipation energy (Fukuyama et.al. 2000). Despite of the potency to prohibit brittle manner and shear failure, bond splitting failure can be prevented by using ECC material.

Objective of this study is to investigate the behavior of ECC under direct tensile tests by using local materials. Compressive strength of ECC material were presented in this paper in order to validate the proposed mixture proportion for concrete grade M45. Polyvinyl alcohol fibers (PVA) were employed in this study as the reinforcing material for cementitious composites. As the results, this paper can give the strain-hardening behavior of ECC when subjected to uniaxial tensile load. The design basis of ECC material which supports the development of ECC are also briefly presented.

DESIGN BASIS OF ENGINEERED CEMENTITIOUS COMPOSITE

To ensure the behavior of strain hardening and multiple cracks occur in ECC material with the tight crack width, the design basis of ECC was made and characterized by Marshall and Cox (1988). A foremost requirement is to permit steady-state cracks criterion takes place under tension in a form of flat cracks. An understanding of ±J¢ relationship is critically required in order to satisfy the criterion (Sobolev et al. 2013). ±J¢ response is controlled by tailoring of the fiber, matrix, and interface between both of fiber and matrix.

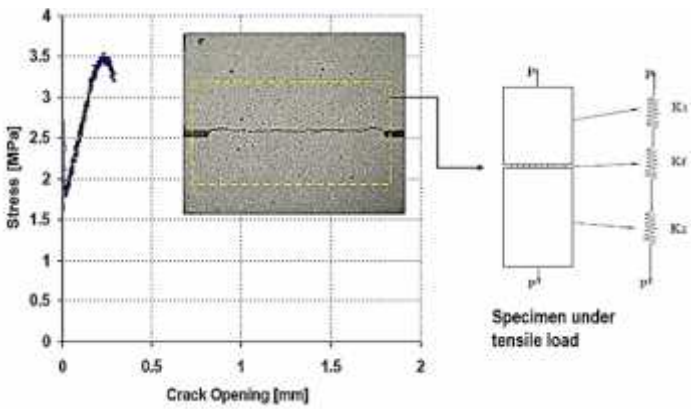

Fig. 2 σ-δ curve of a single crack (Fischer et.al 2004)

Fig. 2 illustrates the experimental test to obtain the ±J¢ response by using ECC material. ±J¢ curve can be used to detect the range of fiber bridging stress and crack opening responses. As reported by Fischer and Stang (2004), the parameters that govern the behavior of the composite material are the first cracking strength $\pm_{fc}$, the peak of bridging stress $\pm_0$, the crack opening $¢_0$ at peak of bridging stress, and fiber bridging stiffness K_f.

By forming steady-state "flat cracks" which preserve a constant crack without rupturing under tension, loading can initiate another micro crack from another damage site and subsequent flat cracks propagation (Wang & Li 2007). As aforementioned, the behavior of steady-state cracks can deliver the multiple cracks.

Marshall and Cox (1988) analyzed the condition for steady-state "flat cracks" using J-integral method as can be seen in Eq.(1).

$$J_b^{'} = \pm_0 ¢_0 - \int_0^{¢_0} \pm(¢)\, d¢ \geq J_{tip} \approx \frac{K_m^2}{E_m} \tag{1}$$

where J_b' is the complimentary energy of the bridging fibers and must be equal or greater than the fracture energy of the mortar matrix J_{tip}. On the other hand, $\pm_0$ and $¢_0$ are the maximum crack bridging stress and crack opening, respectively. K_m is the fracture toughness of the mortar matrix, and E_m is the elasticity modulus of the mortar matrix.

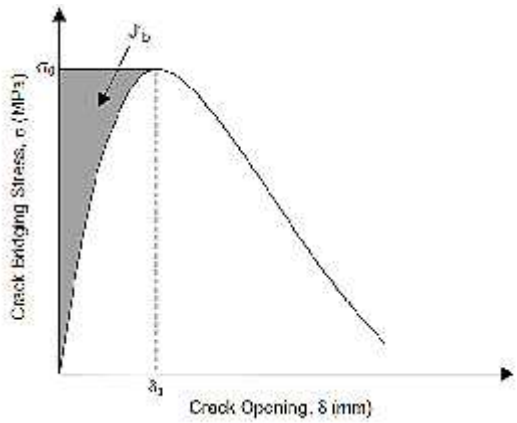

Fig. 3 Complimentary energy due to crack bridging stress versus crack opening (Sahmaran et.al 2008)

The formation of multiple steady-state cracks and strain hardening branch can be attained after performing an ECC mixture which adequately proportion. However, in order to maintain the cracks width below 80 μm, tailoring the crack bridging stress and crack opening should be taken into account as expressed in Eq. (1).

Before achieving the maximum crack opening that corresponding to the maximum crack bridging stress, it allows ECC to perform the multiple cracks to the limit of $¢_0$. If the crack width exceeds $¢_0$, the crack bridging stress would begin to fall and the formation of multiple cracking would stop. Therefore, by preserving the crack opening of ECC below 80 μm, the ECC material can exhibit multiple cracking with the tight crack width.

MATERIALS

Based on the several previous studies, ECC using local material compounds have been successfully applied in various countries such as Europe (Mechtcherine and Schulze 2006), South Africa (Boshoff and Zijl 2007) and Japan (Kanda et. al. 2006). Therefore, it is possible to carry the experimental investigation in order to obtain the tensile performance using local materials in Indonesia.

The standard mixture design of current ECC materials consists of cement, fly ash, silica sand, water, chemical admixture, and randomly deployment of PVA fibers. These materials components are typically similar to the fiber reinforced composite with cement based. The ordinary portland cement (C), class-F fly ash (FA) with lime content 15.24%, water, a high range water reducing admixture (HRWR), and PVA fibers were employed in this study. Furthermore, the use of two different sand (S) types were also carried out using silica sand from Bangka Belitung and lumajang sand that obtained from Lumajang, East Java. According to the typical standard design of ECC, the use of sand must meet an average and maximum grain size of 100 μm and 200 μm, respectively.

The PVA fibers employed in this study are manufactured by Nycon Corporation, USA. The type of PVA fibers is RECS15 and deliberately chose because it can be used in all types of concrete. As presented in Table 1, the mechanical properties of PVA fibers are purposely made with a high tensile strength and elastic modulus matching those needed for proper strain-hardening behavior.

Table 1 Mechanical and geometrical properties of PVA fiber

Diameter	Length	Young's modulus	Tensile strength	Specific gravity
38 *micron*	8.0 *mm*	40 *GPa*	1600 *MPa*	1.30

EXPERIMENTAL PROCEDURES

Mixture Proportion

Two appropriate cement based ECC mix proportion denoted as ECC-S and ECC-L employing silica sand and lumajang sand, respectively and were designed through experiments using the principles of micromechanics for strain hardening behavior. The use of lumajang sand was based on the previous research regarding the possibility to produce non-standard ECC that is expected to meet the strain hardening performance under tension.

ECC mixture proportion for concrete grade M45 was used in this study where details of mixture are shown in Table 2. Water (W) to cementitious material ratio of 0.26 was prepared in a standard mortar matrix for ECC-M45. In addition, fly ash-cement ratio (FA/C) and sand-cement ratio (S/C) of 1.2 and 0.8 by mass, respectively. On the other hand, the percentage of PVA fibers is 2% by volume of specimen.

Table 2. Standard of mixture proportion for ECC

	ECC (M45)
FA/C	1.2
W/CM[a]	0.26
S/C	0.8
Cement (C), kg/m^3	570
Fly Ash (FA), kg/m^3	684
Sand (S), kg/m^3	455
Water (W), kg/m^3	331
Fiber (PVA), kg/m^3	26
HRWR, kg/m^3	4.9

[a] *CM: Cementitious Materials (cement + fly ash)*

Casting, Curing and Testing of Specimens

The fresh ECC matrices were cast into standard ∅50 × 100-mm cylinder specimens for compression test. After placing the fresh ECC matrices into the moulds, compactions were done using elastomer table. Compactions were conducted in order to release any entrained air voids contained in the concrete. If the ECC matrices were not compacted in the proper manner, the strength of concrete under compression cannot be attained.

The compressive strength of ECC mortar was computed as an average value of three cylinder specimens. The compression tests were performed at 7 and 28 days under moist curing. After 24 hours of casting, all the specimens were demoulded and moisture cured in plastic bag with a controlled temperature of 25 °C.

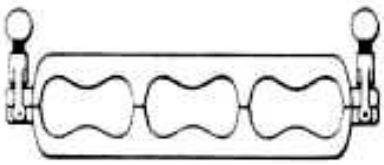

Fig. 4 Dog-bone mould for tensile test (CRD-C 260-1)

Direct tensile tests were also conducted to evaluate the behavior of ECC mixture under tension. Cross section 25 × 25-mm dog-bone specimens (Fig. 4) according to CRD-C 260-1 were prepared. For each mix, the specimens were cured similar to the cylinder specimens. The specimens were tested in uniaxial tension using MTS machine with 5 kN capacity under displacement control at a rate of 0.01 mm/s. A setup of uniaxial tension test is shown in Fig. 5.

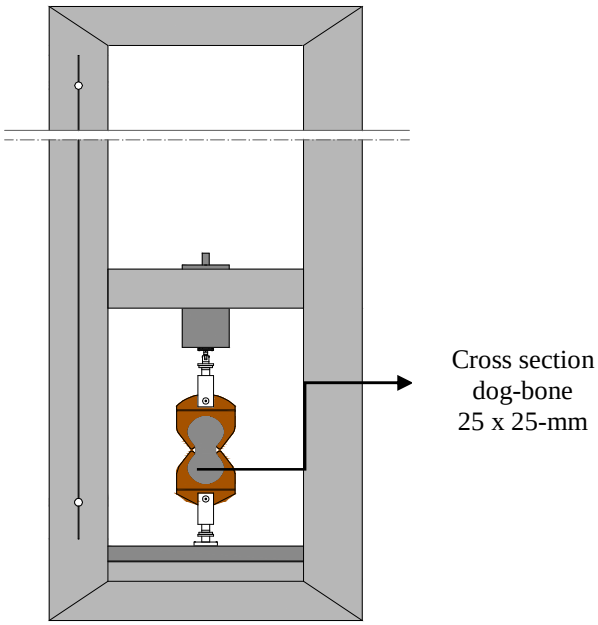

Fig. 5 Setup of uniaxial tension test

RESULTS AND DISCUSSION

Density and Compressive Strength

The average density at 7 and 28 days obtained from the specimens is presented in Table 3. It can be said the density of ECC-S is slightly lower than ECC-L. However, the composite densities of both ECC are relatively lower compared with conventional concrete. This may be caused to the lack of coarse aggregate.

Table 3. Density and compressive strength of each mix

Day	Mix ID	Density (kg/m^3)	Compressive Strength (MPa)
7	ECC-S	2062.65	34.09
	ECC-L	2088.11	34.77
28	ECC-S	2088.11	48.70
	ECC-L	2139.04	49.67

The average compressive strength of cylinder specimens of ECC-S and ECC-L are also listed in Table 3. The results showed that ECC-L gave the better compressive strength compared to ECC-S. This could be mainly due to the gradation characteristic contained in lumajang sand. However, the compression strength between two different types of sand did not give significant value.

Uniaxial Tensile Performance

Fig. 6 shows the tensile stress-strain behavior of dog-bone specimen of ECC-S under moist curing at different ages. It can be said that ECC-S exhibited the strain hardening of more than 3.0%. The result also showed that strain capacity measured after 28 days is slightly lower than 7-day strain capacity (Fig. 6 and Table 4). However, the observed more than 3% strain capacity remains acceptable for an ECC. In addition, the significant improvement of tensile strength value of ECC-S from 7-day to 28-day are also shown in Table 4.

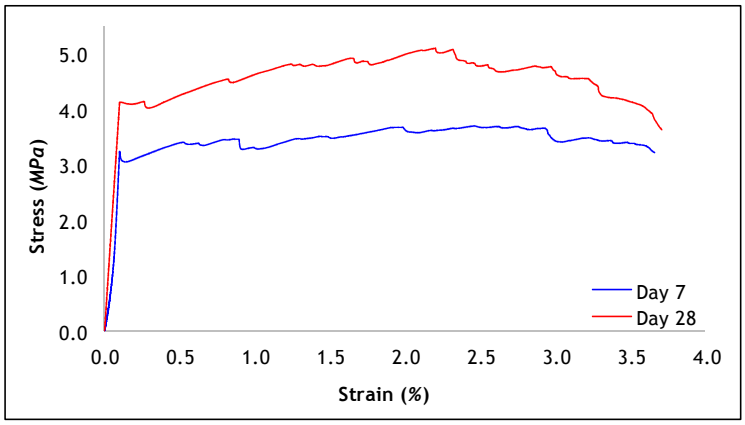

Fig. 6 Tensile behavior of ECC-S cured in the air after 7 and 28 days moist curing

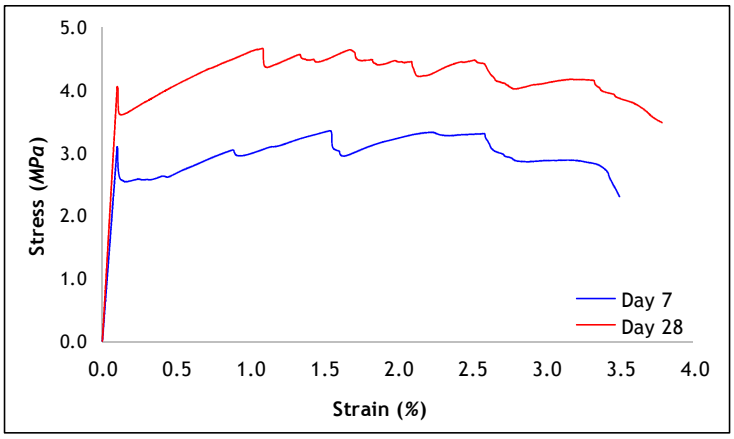

Fig. 7 Tensile behavior of ECC-L cured in the air after 7 and 28 days moist curing

The strain hardening behavior of lumajang sand is also proven as can be seen in Fig. 7. ECC-L dog-bone specimen indicated the strain-hardening capacity where it performed the raising stress accompanied by increasing strain. As the matter of fact, the non-standard mixture proportion of ECC remains acceptable for an ECC due to its strain hardening behavior. Table 4 shows the precise value related to the stress-strain response of ECC-L.

Table 4 Tensile properties of ECC (M45) at different ages

Specimen Type	Tensile Strain (*%*)	Tensile Strength (*MPa*)
ECC-S after 7 days moist curing	3.56	3.71
ECC-S after 28 days moist curing	3.44	5.11
ECC-L after 7 days moist curing	3.34	3.35
ECC-L after 28 days moist curing	3.31	4.67

Despite of the strain hardening capacity of both dog-bone specimens, the tensile strength capacity measured from ECC-L is a bit lower than ECC-S (Table 4 and Fig. 8). The pseudo-strain hardening performance on ECC-S also remains stable compared to ECC-L. Besides, ECC-S has lower strength degradation after the yield point than ECC-L. One of the underlying reasons of the difference in tensile strain capacity of the ECC-S compared to ECC-L lies in their different characteristic of fine aggregate. As known that silica sand have been used for standard of mixture proportion on ECC composites, thus the micromechanics performance can be achieved by using its material. On the contrary, lumajang sand is the new typical of aggregate that is proposed to be used as the replacement of silica sand. Therefore, using the non-standard of mixture proportion can cause different behavior.

In addition, the considerably different tensile strain capacity of the ECC-S and ECC-L composites is also attributed to their different energy performance (J_b'/J_{tip}). The higher ultimate tensile strength and strain capacity of ECC-S indicates its higher complimentary energy (J_b') compared to that of ECC-L. Thus, it can be said that the higher complimentary energy, the higher stress-strain capacity can be achieved.

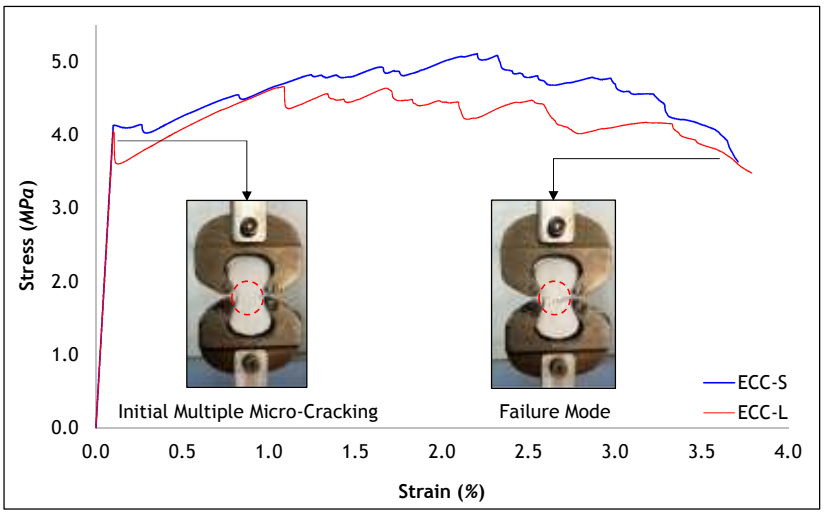

Fig. 8 The comparison of direct tensile performance between ECC-S and ECC-L

Fig. 8 also presents the typical crack of ECC dog-bone specimen under direct tensile test. The initial multiple micro-cracking on specimen occurred after the first yielding point. It can be seen that tight crack width was exhibited under tensile load. Furthermore, unlike the conventional concrete that shows brittle manner where the specimen splits into two parts, in this figure, the ECC material remained intact at strain more than 3% due to high properties of PVA fibers. Thus, it can be said that the performance of ECC material is highly better than conventional concrete.

CONCLUSION

Based on the experimental results reported in this paper, the following conclusion are drawn:

1. The compression strength at 28-day of ECC-S and ECC-L meet the standard of compression strength of mixture proportion for ECC (M45).
2. The density for an ECC is slightly lower than conventional concrete due to the lack of coarse aggregate
3. The tensile strength capacity of both ECC-S and ECC-L can achieve 10% of its compressive strength.

4. The tensile strain of ECC-S and ECC-L exhibits the strain hardening behavior up to 3.56% where it remains acceptable for an ECC. Thus, it is acceptable to be used in the structural application.
5. The multiple micro-cracking occurs on the ECC dog-bone specimens under direct tensile test.
6. From the compression test and direct tensile test, it can be confirmed that ECC-S has lower compressive strength than ECC-L. However, in term of direct tensile performance, ECC-S has higher tension strength than ECC-L.

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FLEXURAL STRENGTHENING EFFECT OF CFRP STRAND SHEET TO RC BEAMS WITH VARIOUS TYPES OF ADHESIVE

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ABSTRACT: CFRP strand sheet is one of FRP material made from carbon. The CFRP strand sheets can be easily attached on the surface of concrete members with putty like adhesive and no impregnation needed. Thus, the advanced strand sheet method can assure better construction quality and cut down the application time and costs.
This paper will discuss about strengthening effect of CFRP strand sheet to RC beams. To obtain sufficient results, three types of adhesive, namely; epoxy, MMA, PCM, and JCM are used in this study. There are two models of test in this study. They are bonding test, this test have been done according to the JSCE – E543 – 2007 (testing method for bonding properties of continuous fiber sheet to concrete), and beam test. Simple applications of CFRP strand sheet in Japan will be shown at the end of this paper.

Keywords: *CFRP strand sheet, bonding test, RC beams, Epoxy, MMA, PCM, JCM*

INTRODUCTION

Many innovations have been carried out for strengthening reinforced concrete structures. Strengthening is required for structural elements in which its strength have been declining due to, such as, age, environmental influences, poor design, lack of maintenance, change of function and damage caused by events such as earthquakes and others.

FRP (fiber reinforced polymer) is proposed as a solution for solving this problem, despite having a fairly expensive price, FRP has many advantages such as corrosion resistance, high tensile strength, durability, good fatigue resistance, lighter specific gravity, easy and fast in application, as well as adjustable processing method of concrete. FRP is a composite material that consists of high strength material fiber embedded in a polymeric resin. The type of fiber that is often used in the fabrication of FRP is carbon, then called carbon fiber reinforced polymer (CFRP), aramid (aramid fiber reinforced polymer, ARFP) and glass (glass fiber reinforced polymer, GFRP). Fig. 1 shows a comparison between CFRP, AFRP, GFRP, and reinforcing steel in the stress-strain relationship diagram. It can be seen that CFRP is 7 to 8 times stronger than reinforcing steel.

CFRP strand sheet, shown in Photo. 1, is one of CFRP interface bonding system that has been recently developed. CFRP strand sheet has a unidirectional reinforcement and is suitable for structural beam. Compared with other strengthening materials, CFRP strand sheet has several advantages, such as, thinness, adjustable processing method of concrete, easy to cut, can be used without impregnating adhesive onto RC beams and less possibility of air bubble occurrence in interface between CFRP strand sheet and RC beams.

This paper discusses about strengthening effect of CFRP strand sheet and concrete. Prior to discussing about strengthening effect would also talk about bond behavior. To investigate the bond behavior, a doubled faced shear type bond test was conducted. The bonding test was done by adopting the (JSCE-E543 –2007), about test method for bonding properties of continuous fiber sheet to concrete. This study used four types of adhesive namely epoxy, MMA (methyl methacrylate), PCM (polymer cement mortar), and JCM (jet concrete mortar).

MATERIAL PROPERTIES

CFRP Strand Sheet and Adhesives

The properties of CFRP strand sheet and adhesives were obtained from the manufacturer. High tension type of CFRP strand sheet was used in this study. For more detail, its mechanical properties can be viewed in Table 1. Table 2 shows the mechanical properties of adhesive. Epoxy is commonly used as standard adhesive in the use of FRP. MMA is a quick-hardening adhesive can also be used in low temperature conditions. MMA can be used after two hours of installed. PCM, consists of polymer-emulsion, cement and sand, is used for wet surfaces. JCM is inorganic material which consists of ultra-rapid hardening cement and sand. In application, JCM is used together with epoxy adhesive.

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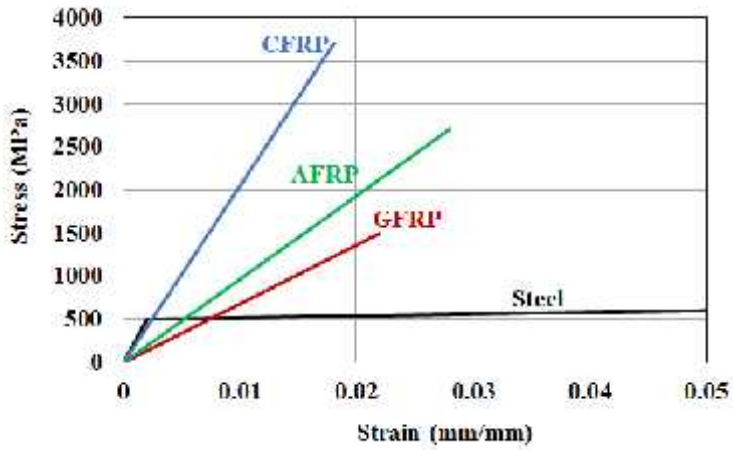

Fig. 1 Stress-strain relationship of reinforcing materials

Photo. 1 CFRP strand sheet

Table 1 Mechanical properties of CFRP strand sheet

Type	High-tension type
Tensile Strength (MPa)	3,400
Tensile Modulus (MPa)	245,000
Design thickness (mm)	0.333
Unit Weight (g/m^2)	600

Table 2 Mechanical properties of adhesive (MPa)

Type	Epoxy	MMA	PCM	JCM
Compressive Strength	78.3	79.0	11.3	55.3
Compressive Modulus	3,970	2,500	4,800	26,200
Tensile Strength	35.8	43.0	2.4	5.3
Flexural Strength	58.8	71.0	6.5	9.5
Lap-share Strength	25.8	22.0	-	

The use of adhesive in specimen

Fig. 2(a), Fig. 2(b), and Fig. 2(c) are the cross-section of CFRP strand sheet reinforcing element using Epoxy resin as a bonding agent with 1 ~ 3 layers. The process of implementation was begun after Epoxy resin prepared coated on the concrete, then followed by attaching process of CFRP strand sheet. It was placed while pressed in order that adhesive was evenly distributed. After it was done, the next layer was coated in the same way.

Fig. 2(a), Fig. 2(b), and Fig. 2(c) illustrate in brief strengthening types that were used MMA as an adhesive for 1 to 3 layers. Stages of implementation are as follows; prepared the primer material, then given evenly on the CFRP strand sheet and the surface of the concrete beam. Next, sprinkled MMA material on the concrete surface. Then the CFRP strand sheets were attached to the concrete surface while pressed in order to be spread evenly, after it was done, did overcoat for the next layer. In this study, the use of Epoxy and MMA adhesive was about 2.5kg/m^2.

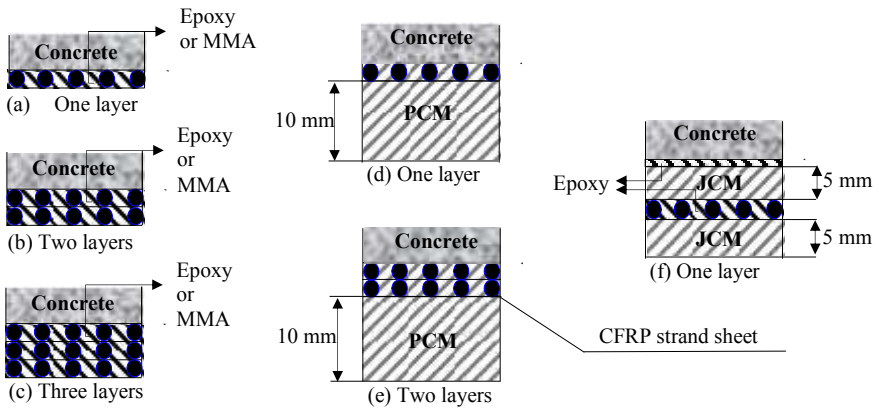

Fig. 2 Cross-section of CFRP strand sheet reinforcing element

The use of PCM as an adhesive in retrofitting methods can be seen in the cross-section at Fig. 2(d) and Fig. 2(e). Stages of implementation areas follows; after prepared PCM mixed with the water, spread thinly on the surface of concrete beams. Subsequently, CFRP strand sheet was attached with on the whole surface and was pressed such that the filling could be evenly distributed. After that, followed by the next layer. Finally, CFRP strand sheet was covered with the PCM until designed thickness. In this research, PCM, for surface protection, had a thickness of 10mm from CFRP strand sheet layer.

Fig. 2(f) presents cross section an application of JCM adhesive for one layer. In this case, firstly, epoxy adhesive was applied to the surface of specimen about 1.4kg/m^2, followed by 5mm in thickness of JCM. Next, before CFRP strand sheet was installed, epoxy adhesive once again is given at the surface of JCM about 3.0kg/m^2. The final step is surface protection with JCM about 5mm in thickness. All the procedures should be completed before all the materials has hardened.

Concrete and Steel Reinforcement

The quality of concrete was used in both bonding test and beam test can be seen in Table 3. Table 4 presents mechanical properties of rebar in beam test. Deformed bars of 19mm in diameter were used as tension reinforcement, 10mm in diameter were used as compression reinforcement, and stirrups as shear reinforcement have a diameter of 13mm.

Table 3 Mechanical properties of concrete (MPa)

Compressive Strength	Modulus of Elasticity
37.1	26,900

Table 4 Mechanical properties of rebar (MPa)

Material Properties	Rebar		
	Dia.10 mm	Dia.13 mm	Dia.19 mm
Modulus of elasticity		200,000	
Tensile strength	548	551	559
Yield strength	376	395	407

BONDING TEST

Test Set Up

The specimens of this research are shown in Fig. 3 and Photo. 2. They were made three for each of sample. The sample are one, two, three layers for Epoxy and MMA adhesive. For PCM adhesive, the sample are one and two layers and the one layer sample is for JCM adhesive. The bond test specimen was a double-face type consisting of a concrete prism (100mm x 100mm x 620mm) with two CFRP strand sheets with 50mm width which were bonded at two opposite sides of the specimen. One side of the specimen was given a confinement sheet. There were two notches at the center of the prism which made at the time of casting, the two steel bars also had no connection. This means that the two prisms were connected only with the CFRP strand sheet and debonding failure occurred only on the opposite side of confined area, where strain gauges were set. The strain gauge had a distance of 30mm, the bond length total of CFRP strand sheet was 280mm. To avoid stress concentration, at each end of the specimen had two or three layers, the upper layer was made shorter than the previous layer by 25mm (Fig. 2 Detail J). Monotonic static loading test method was carried out in

this investigation. A universal testing machine with a capacity of 1000kN was used, as shown in Photo. 3. The increase rate of the load was approximately 5kN per minute and the strain gauge was recorded at 1kN of load increment.

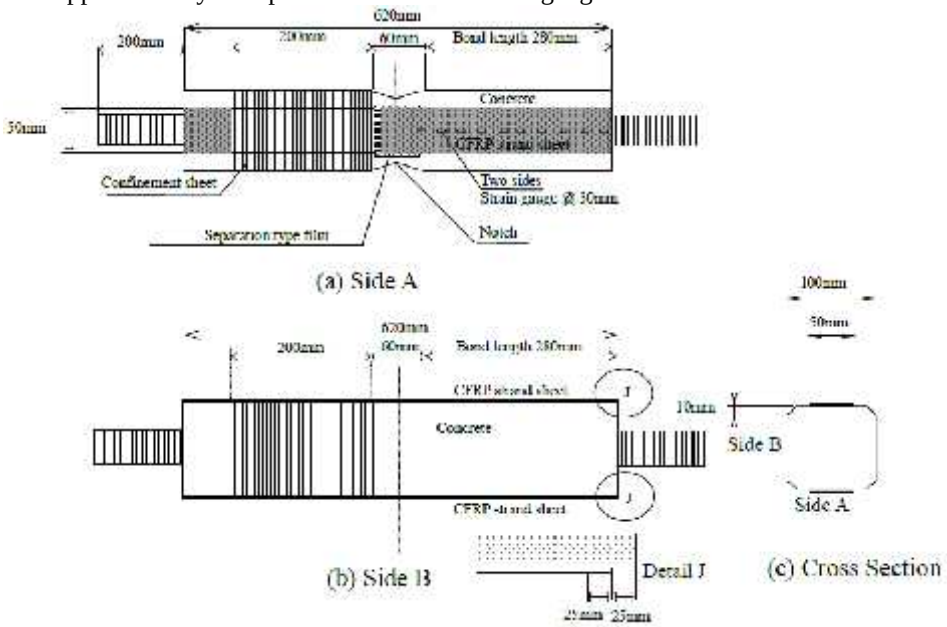

Fig. 3 Geometry for bonding test specimen

Photo. 2 Specimen for bonding test (epoxy 1 layers)

Photo. 3 Testing of specimen

Test Results and Discussion

Maximum load

Table 5 presents the results of maximum load. For all types of adhesives shows that the more layer increase the maximum load. When comparing to a single layer of the specimen, it became clear that the largest maximum load is on the adhesive type JCM followed epoxy, MMA, and PCM, respectively. The maximum load of JCM is greater up to 40%, compared with other types of adhesive on the one layer of CFRP strand sheet. For two layers and three layers specimen, epoxy has larger value compared to the MMM and PCM with larger values up to 20%.

Bond strength and interfacial fracture energy

Table 5 also capture the bond strength and the interfacial fracture energy results. The interfacial fracture energy, G_f, is area enclosed the interfacial bond stress (^{2}) and slip (s) curve relationship that is a significant parameter for the bonding properties. In the analysis, $^2{}_u$ and G_f were calculated by equation 1) and 2), based on JSCE – E543 – 2007.

$$^2{}_u = \frac{P_{max}}{2bl} \tag{1}$$

$$G_f = \frac{P_{max}^{\ 2}}{8nb^2E_f t} \tag{2}$$

Where,

G_f = interfacial fracture energy (N/mm)

$^2{}_u$ = bond strength (MPa)

P_{max} = maximum load (N)

b = average width of CFRP strand sheet (mm)

E_f = tensile modulus of CFRP strand sheet (MPa)

t = thickness of CFRP strand sheet (mm)

l = length of bond (mm)

n = number of layer of CFRP strand sheet

Table 5 shows that the bond strength of JCM is the greatest than the other in the one layer specimen. For one layer, the bond strength of JCM are 14%, 34% and 40% higher than those of Epoxy, MMA, and PCM, respectively. For two layers, the Epoxy results are 22% and 14% greater than those of MMA and PCM, respectively, and it has 20% higher than that of MMA for three layers. In general, it can be seen that adding the number of layers will increase the of bond strength.

For results of interfacial fracture energy G_f can be seen that the value of G_f is related to number of the layer, increasing the number of layers the results of G_f will be decrease. On single layer specimen, JCM has the greatest value for the interfacial fracture energy, followed by Epoxy, MMA and PCM, respectively.

Table 5 Results of bond test

Type of Adhesive	Numb. of layers	No. of spec.	Max. Load, P_{max} (kN)		Bond Strength, τ_u (MPa)		Interfacial Fracture Energy, G_f (N/mm)	
			Result	Ave	Result	Ave	Result	Ave
Epoxy	1	1EB1	43.3	42.1	1.55	1.51	1.15	1.09
		1EB2	41.9		1.50		1.08	
		1EB3	41.2		1.47		1.04	
	2	2EB1	51.2	51.7	1.83	1.84	0.80	0.82
		2EB2	46.6		1.66		0.67	
		2EB3	57.2		2.04		1.00	
	3	3EB1	56.4	56.7	2.01	2.02	0.65	0.66
		3EB2	54.7		1.95		0.61	
		3EB3	59.0		2.11		0.71	
MMA	1	1MB1	40.6	36.0	1.45	1.29	1.01	0.80
		1MB2	32.2		1.15		0.64	
		1MB3	35.2		1.26		0.76	
	2	2MB1	42.2	42.3	1.51	1.51	0.55	0.55
		2MB2	41.4		1.48		0.53	
		2MB3	43.3		1.55		0.57	
	3	3MB1	43.7	47.1	1.56	1.68	0.39	0.46
		3MB2	50.7		1.81		0.53	
		3MB3	47.0		1.68		0.45	
PCM	1	1PB1	35.8	34.5	1.28	1.23	0.79	0.73
		1PB2	32.6		1.17		0.65	
		1PB3	35.0		1.25		0.75	
	2	2PB1	44.1	45.1	1.58	1.61	0.60	0.63
		2PB2	42.3		1.51		0.55	
		2PB3	49.0		1.75		0.74	
JCM	1	1JB1	54.6	48.21	1.95	1.72	1.83	1.44
		1JB2	43.5		1.55		1.16	
		1JB3	46.6		1.66		1.33	

Photo 4 Typical failure model of bonding test (epoxy 2 layers)

Failure mode detail

JSCE –E543 –2007 confirms that there are two categories for test specimen failure; (a) the interfacial failure, which is a debonding failure between surface CFRP strand sheet and concrete, and (b) base material failure, which is the CFRP strand sheet reaches its ultimate strain.

Typical failure of bonding test is shown in Photo 4. It can be seen that the typical failures of the specimen were the interfacial failure which occurred only one side of the prism. It was caused by eccentricity of the load in the loading machine. Besides that, these facts were related to the adhesive at interface between CFRP strand sheet and concrete. In this case, the bonding strength did not depend on the concrete strength or the CFRP strand sheet strength.

BEAM TEST

Test Set Up

Seven RC beams comprising one non retrofitted beam (specimen N) as control beam and six retrofitted beams with one layer and two layers of CFRP strand sheet for each of adhesive have been tested in this study. Detail of the experimental program can be seen in Fig.4. Detail A in Fig.4 explains the position of the CFRP strand sheet for the two-layer specimen. To avoid stress concentration at the end of beam, the CFRP strand sheet was shortened by 25mm for the next layer.

All the beams are four-point loading beams that have a span of 2.2m with two small rollers as supports that have a span of 1.6m. The load was applied by a hydraulic jack and measured by a load cell. Deflection control was used in all the tests and deflection measurements were taken at the mid span of the beam. The strain gauges were located 100mm on each CFRP strand sheet, four on the steel reinforcement (two for tension rebar and two for compression rebar), and one on the top of the concrete in the mid span.

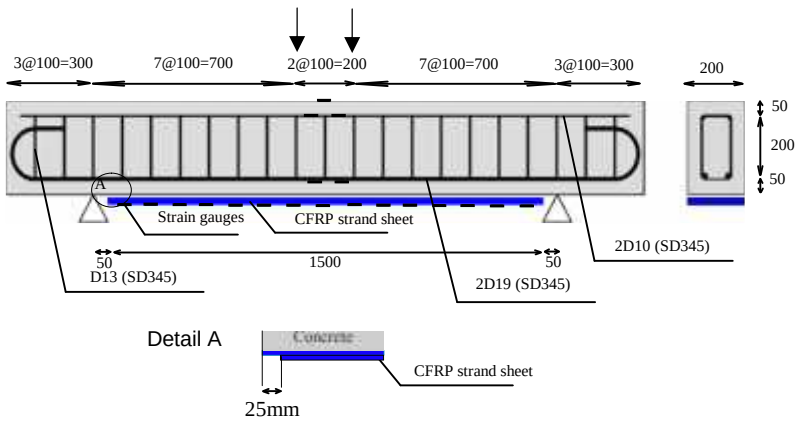

Fig.4 RC Beams specimen detail

Test Results and Discussion

Experiment load result

The experimental results for each loading stage are given in Table 6. The results show an increase for almost all stages of loading. When in the crack state, there is an increase in almost the crack load. For epoxy adhesive, an increase in crack load is 38% and 27% for specimen Epoxy1 and Epoxy2, respectively, compare with control beam (specimen N). For MMA adhesive, the results are 26% and 39% for specimen MMA1 and MMA2, respectively, higher than specimen N. For JCM1, there is an increase of 25% in crack load. Meanwhile, for specimen PCM1, there is almost same result. Moreover, for specimen PCM2, a decrease in capacity of load crack compared with specimen N occur. It is possible that initial crack has happened prior to loading on the specimen PCM2.

From Table 6 can be seen that all strengthened beams with CFRP strand sheet have an increase in capacity of yield load compared to the normal beam at the yield loading stage. These show an increase of 46% and 80% in the yield load capacity for specimen Epoxy1 and Epoxy2, respectively, over the specimen N. For MMA adhesive, there is an increase of 56% and 65% than control beam. An increase of 31% and 46% occur in the specimen PCM1 and PCM2, respectively compared with specimen N. The same tendency occurs in JCM specimen, an increase of 25%. It means that the CFRP strand sheet and the adhesives could delay the reinforcing steel to experience yield.

From Table 6 can be seen the results of maximum load for each beam. An increase in capacity is observed 46% and 80% of normal beam for Epoxy1 and Epoxy2, respectively. An increase in maximum load for specimen MMA1 and MMA2 can be observed in Table 6. The values are 55% and 65% higher than specimen N. For PCM adhesive, there is an increase of 35% and 22% for PCM1 and PCM2, respectively. An increase of 59% from normal beam also occurs in JCM1 specimen.

Table 6 Experiment load results

Spec.	Layers Numb.	P_{Crack} (kN)	P_{Yield} (kN)	$P_{Ultimite}$ (kN)
N	-	39.8	131	165
Epoxy1	1	50.4	191	278
Epoxy2	2	55.0	236	295
MMA1	1	50.1	205	255
MMA2	2	55.3	216	272
PCM1	1	40.1	171	223
PCM2	2	35.1	191	201
JCM1	1	49.9	185	262

It could be noted that although there is an increase in maximum load but the use of one layer of CFRP strand sheet is more effective than two layers for PCM. This is different from assumption that increasing the number of layers will increase the capacity of beam. This is because in the two-layer specimen (PCM2 specimen) the interface layer between CFRP strand sheets is not too effective to resist shear stress and so reduces the capability of existing composite mechanisms.

Load and displacement

Fig. 5 shows the relationship between total load and mid-span deflection of the specimen. In general, it may be noted that after the maximum load is achieved, load will be closer to the yield load on the specimen N. It means that the specimens show ductile behavior up to failure, so it may be able to absorb the energy well and can avoid the collapse that occurs suddenly.

Load and strain of CFRP strand sheet

Load and strain of CFRP strand sheet is shown in Fig. 6. It can be seen that the first crack occurred in the surface of beam at about 40–55kN. An exception in specimen PCM2 is also confirmed in that the initial crack happened in specimen PCM2. The maximum ultimate strain that can be reached is around 7000μ for Epoxy1 and JCM1, next ultimate strain for MMA1 is around 6000μ. It can be seen that CFRP strand sheet's ultimate strain (about 13500 μ) can never be reached. So, although epoxy, MMA, PCM and JCM, as a bonding agent, can improve the capacity of the RC beam, it is not able to maximize the strength of the CFRP strand sheet.

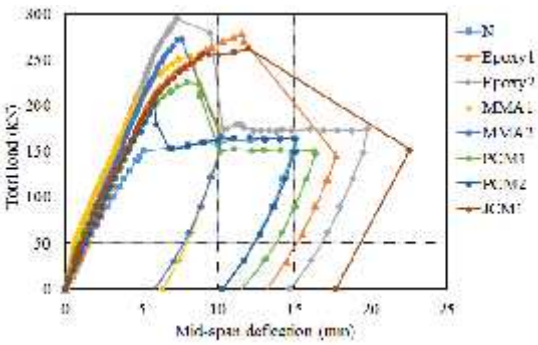

Fig. 5 Total load versus mid-span deflection

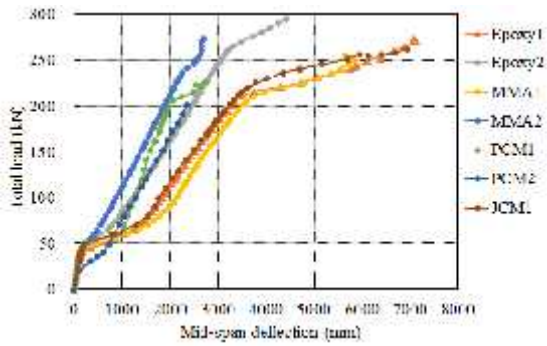

Fig. 6 Total load vs strain of CFRP strand sheet

EXAMPLE OF APPLICATION IN JAPAN

Rapid Construction of Bridge Deck

The work that require quick time in construction without a long treatment time has applied by using CFRP strand sheet in the bridge retrofitting. By using this retrofitting method was possible to be able to use the adhesive before JCM becomes hard. It was obviously making the time of construction become short. After installation, it was possible to carry out deck waterproofing and pavement restoration in about three hours of curing time. Since the surface finishing was JCM, it was no matter for adhesion of waterproofing or asphalt pavement. In these circumstances, it is possible to approximately 20m^2 construction per one day. Photo 5 show the actual construction of bridge in Okayama Prefecture.

(1) Floor Cleaning	(2) Epoxy adhesive (1.4kg/m^2)	(3) JCM coating (5mm)
(4) Epoxy adhesive (3.0kg/m^2)	(5) Installation of CFRP Strand sheet	(6) JCM for surface protection (5mm)

CONCLUSION

This paper provided results from the CFRP strand sheet and concrete strengthening effect. The following conclusions can be drawn up;

1. From the results of bonding test, it can be seen that Epoxy and JCM have better performance, for the maximum load (P_{max}), fracture energy (G_f) and bond strength (τ_u), among the types of adhesive which are used in this experiment.
2. Typical failure of bonding test was the interfacial failure occurred on only one side of the prism. It can happen due to eccentricity of load in the loading machine. The specimens were supposed to be under pure tensile force, but they were not possible to avoid the moment which is caused by eccentricity of load in loading machine.
3. Strengthening with CFRP strand sheet and PCM can improve the capacity of RC beam.

4. Although epoxy, MMA, PCM, and JCM as a bonding agent, can improve the capacity of the RC beam, it is not able to maximize the strength of the CFRP strand sheet.

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THE EXPERIMENTAL TEST OF CORROSION INDUCED CRACKING IN REINFORCED CONCRETE USING ACCELERATED METHOD

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ABSTRACT: This study presents an accelerated corrosion test for reinforced concrete specimens to investigate the cracking process induced by reinforcement corrosion. The reinforcement corrosion is major durability problem in the reinforced concrete structure. The growth of corrosion product can exceed two to six times the volume of the original steel and it generate expansive pressure to the concrete. As the result, it can cause cracking and damage to the structure. In this study, the concrete specimen was immersed in a 5% NaCl solution. A constant electric current of approximately 190 μA/cm2 was applied to accelerate the corrosion process. Based on the experimental test it was obtained that the cracking properties of concrete were influenced by exposure time and volume of the corrosion product. This paper also discussed about non-uniform distribution of the corrosion product which can cause non uniform distribution of cracking in the concrete.

Keywords: *Non-Uniform Corrosion, Reinforced Concrete, Accelerated Corrosion Test, Cracking.*

INTRODUCTION

Reinforcement corrosion is a classical durability problem in reinforced concrete structure. This problem can affect the service life of the structures (Muthulingam & Rao, 2015; Shi, Xie, Fortune, & Gong, 2012). Corrosion can initiate by several mechanism such as carbonation, chloride ingress, freezing thawing etc. Based on the previous research it was known that chloride ingress is one of the major problems which can initiate the corrosion process.

Corrosion initiation process indicate by depassivation of passive layer of the steel reinforcement (Cao, Cheung, & Chan, 2013a; Zhao, Xu, & Jin, 2013). This process involves chloride diffusion through concrete pores. Once the chloride content reached a certain threshold value, it can accelerate the corrosion process (Angst & Vennesland, 2009). As corrosion develops, volume of the corrosion product increase and cause internal expansive pressure to the concrete (Zhao, Hu, Yu, & Jin, 2011; Zhao, Yu, Wu, & Jin, 2012; Zhao, Karimi, et al., 2011). This expansive pressure will lead to cracking and progressive deterioration to the concrete.

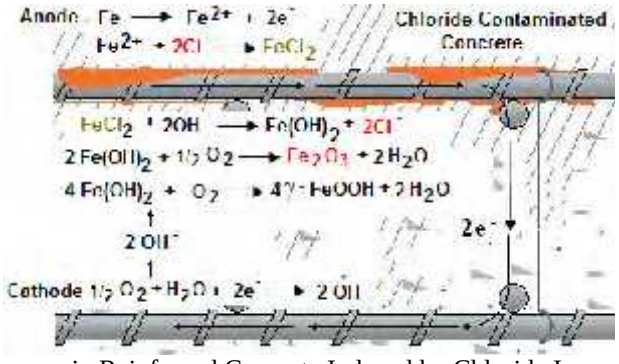

Fig 1. Corrosion Process in Reinforced Concrete Induced by Chloride Ingress (Shi et al., 2012)

Field studies regarding crack to the concrete suggest that cracking and spalling due to corrosion are most concern problem of the construction owner. This problems can lead to structural distress and damage overtime if it is not handled properly (Vu, Stewart, & Mullard, 2006).

The purpose of this study is to investigate the cracking process of the reinforcement due to reinforcement corrosion. The study conducted by experimental test. Conventional corrosion testing often takes long periods of time and costly (Williams, 1996). Therefore, this study used accelerated corrosion with galvanostatic method to simulate the corrosion

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induced cracking process. Analysis of cracks which occur in reinforced concrete due to corrosion still use the assumption that the corrosion products formed uniformly along the perimeter of the steel bar (Cao, Cheung, & Chan, 2013b; Li, Lawanwisut, Zheng, & Kijawatworawet, 2005; Liu, 1996; Zhao, Yu, Hu, & Jin, 2012; Zhao, Yu, & Jin, 2011; Zhao, Yu, Wu, et al., 2012). By using these assumptions, the corrosion products can be assumed to be a thick wall which formed around the perimeter of reinforcement (Li et al., 2005) and generate uniform cracking pattern on the expansive concrete. However, in subsequent studies it found that the development of corrosion products (rust) is formed unevenly along the perimeter of reinforcement (Cao & Cheung, 2014; Muthulingam & Rao, 2014; Zhao, Hu, et al., 2011; Zhao, Karimi, et al., 2011). Based on this problem, this paper also discussed about non-uniform distribution of the corrosion product which can cause non-uniform distribution of cracking in the concrete.

STEEL BAR CORROSION IN REINFOCED CONCRETE STRUCTURE

Corrosion is an electrochemical process that occurs due to the interaction between the metal, water and oxygen. Reinforced concrete in aggressive environments, such as marine environments is very susceptible to corrosion due to sea water. Sea water actually containing aggressive ions which can accelerate the corrosion of reinforced concrete structures. In reinforced concrete, corrosion is major cause of durability problem which can lead to overall damage to the structures (Chen & Mahadevan, 2008; Zhao, Hu, et al., 2011; Zhao, Yu, Hu, et al., 2012).

Corrosion process begin with deppasivation process. Depassivation actually is a damage process of passive layer of reinforcement due to infiltration of aggressive agent such as chloride. Damage of the passive layer cause steel bar react with the water and oxygen which cause the corrosion. Development of corrosion cause worsening the bonding between concrete and steel reinforcement. As the result, the ability of the reinforcement to provide tensile strength and ductility to the concrete decreased (Fang, Lundgren, Chen, & Zhu, 2004).

Furthermore, corrosion process will continue if the concentrations of O_2 and H_2O are sufficient in the concrete. This condition cause increase of the corrosion product or known as rust. Development of rust will generate internal expansive pressure which can initiate crack at the concrete surface (Cao & Cheung, 2014; Fang et al., 2004; Guzmán, Gálvez, & Sancho, 2011; Li et al., 2005; Liu & Weyers, 1998).

EPERIMENTAL SET UP

All specimens were 150 x150-mm reinforced concrete prism with 150 mm thickness. The sample contain 19 mm single deform reinforcing bar which placed at center of the cross sectional area. Steel reinforcement used in this experimental has 50-mm exposed length to the concrete matrix as shown in figure 2. The concrete cover used in this study is 40 mm.

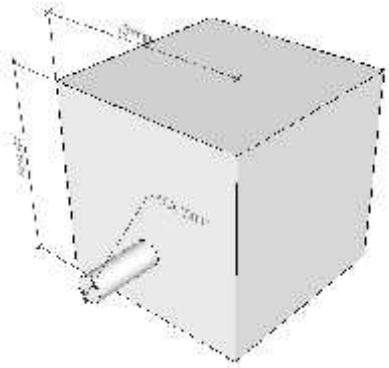

Fig. 2. Concrete Speciment for Corrossion Test

Six sets of reinforced concrete speciments were used in this study. The conrete mix used water per cement ration (w/c) 0.5. Cement used for all speciment are portland pozzoland cement. This test used local coarse and fine aggregate with maximum size for coarse aggregate is 20 mm.

The Accelerated Corrosion Test

To generate cracks due to corrosion the accelerated corrosion test used approximately 200 $\mu A/cm^2$ of electric current. The concrete specimen placed in the 5% NaCl solution. The accelerated corrosion equipment shown in Fig 3. The accelerated corrosion process was achieved by applying a constant electrical current to the bars by a power supply via a current regulator. The current regulator kept the current constant over time. The steel bar acts as an anode, the stainless steel plate submerged in the NaCl solution acts as the cathode, and the pore fluid in the concrete is the electrolyte.

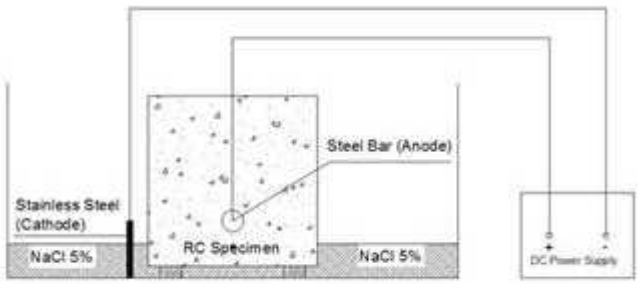

Fig. 3. Accelerated Corrosion Test Set Up

RESULT AND DISCUSSION

An accelerated corrosion test was performed to investigate the corrosion induced cracking process in reinforced concrete structures. The first corrosion bleeding and crack initiation at concrete surface was observed using daily visual observation. The process of corrosion bleeding investigated from discoloration of the solution. Furthermore, the cracking initiation and propagation was observed by a magnifying loop. At the end of the experimental test, the concrete specimen were sliced into 4 identical pieces to observe the corrosion layer and cracking pattern at the inner part of the concrete specimen.

Rust Distribution

To observe the rust distribution of the sample, the sample was cut into four same pieces. The rust of the sample observe using digital measurement to get accurate result. The non-uniform thickness of the rust layer around the circumference of the steel bars is described in terms of a polar coordinate. Gaussian method purposed by Zhao (Zhao, Hu, et al., 2011) was used to analyze the non-uniform distribution of the sample. Fig 5. Below show the analysis result at day 6 and 28 of the rust layer based on Gaussian method purposed by Zhao.

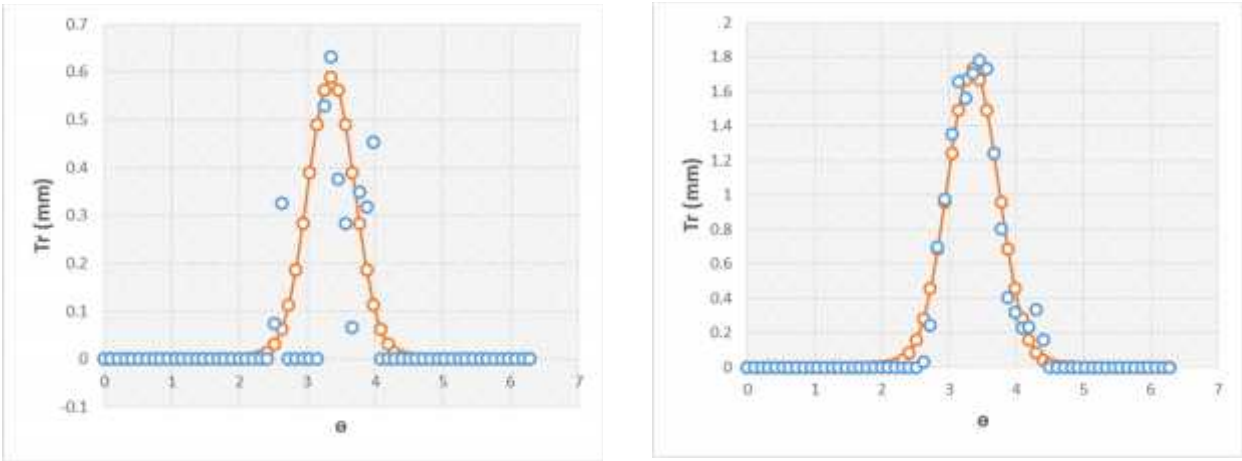

Fig. 5. Non Uniform Distrinution of Rust Layer

Corrosion and Crack Initiation

It was observed that first corrosion initiation occurred less than one day. After the initiation process, the corrosion starts to develop and generate cracks. The first visible hairline cracks can only be seen through magnifying loop with approximately 0.07 mm width. This crack appears at the bottom part of the reinforcement which face the concrete cover.

After crack initiation, cracks propagate with their length and width until reach the concrete cover. Beside cracks at cross sectional direction, cracks due to corrosion also propagate at the longitudinal direction. Crack propagation at cross sectional direction as expected has slower speed compared with longitudinal side. For example at the end of experimental test the cracks crack width at longitudinal direction reach 3.5 mm and at the cross sectional direction the crack width only reach approximately 1.5 mm. Fig. 6 below shows the crack at cross sectional and longitudinal direction. Cracks length and width increase as increase of exposure time. After crack initiation crack excessively increase the length and width. By the time the crack propagation starts to propagate slower and at the end concrete start to spalling.

(a) (b)

Fig 6. Cracking pattern at cross section direction (a) and longitudinal direction (b).

Types of the Rust

At the end of test, concrete was remove and rust was collected to study its type and characteristic. This test performed using X-Ray Diffraction (XRD) analysis. The test result showed that the type of rust is Iron Oxide Hydroxide (FeOOH). Table below show the pattern list of XRD test.

Table 1. XRD test Result of The Rust

Visible	Ref. Code	Score	Compound Name	Displacement [°2Th.]	Scale Factor	Chemical Formula
*	01-073-2326	25	Iron Oxide Hydroxide	-0.008	0.597	Fe O O H

Chloride Profile Analysis

Chloride profile analysis performed using argentometric titration. The concentration of the chlorides at the given depth are determined through titration with Silver Nitrate ($AgNO_3$). The chloride concentration per concrete mass is increased as increased of the exposure time. Fig.7 below shows the chloride profile of the sample.

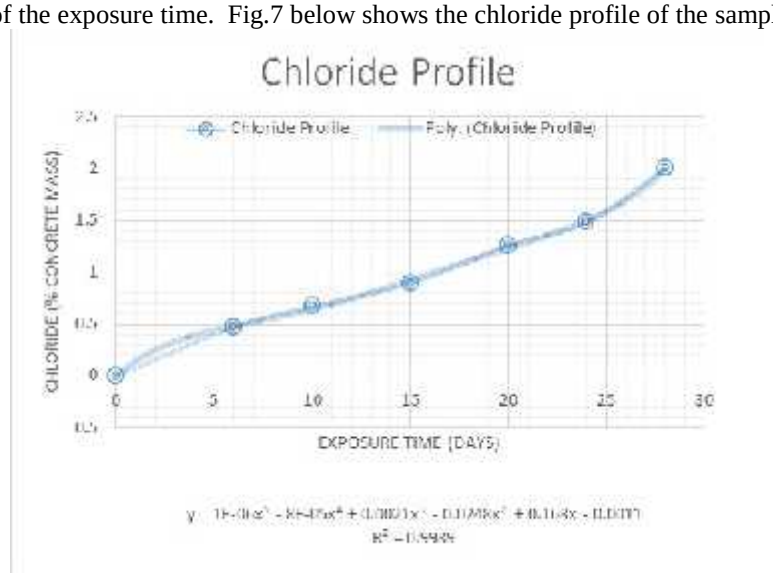

Fig. 7. Chloride Profile of the Concrete Sample

CONCLUSION

An experimental test was conducted to investigate the cracking process of reinforced concrete under chloride environment. In a short period of time the damage process of concrete due to corrosion was obtained. The experimental result showed that corrosion product thickness form non-uniformly around the circumference of the steel bar. This condition cause the non-uniform distribution of cracks pattern at the concrete surface. It also found that the chloride content, crack width and length increase as increase of the exposure time.

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FINITE ELEMENT MODELING OF TWO WAY HALF SLAB PRECAST CONCRETE SUBJECTED TO MONOTONIC LOADING

Djoko Irawan [1], Priyo Suprobo [2] and Data Iranata [3]

ABSTRACT: Half slab precast concrete has been commonly use in the bridge construction and designed as one way slab systems. Due to the rapid construction method, most of the building designers and constructors have changed the monolith cast in situ slab system into the half slab precast system. However, for two way slab systems these changes are lead to the severe concrete cracking as well as section strength reduction. Hence, the main problem that to be solved in this study is the changes system from the monolith cast in situ into the half slab precast for two way slab with large dimension. Commonly, the slab is divided into several precast parts but still maintain to have two way slab behavior. Thus, it is necessary to develop the precast connection systems that have adequate capacity to resist the acting moment for the two way slab systems. This paper presents finite element modeling and simulation of two way half slab precast concrete systems that subjected to the monotonic loading. The connection between precast concrete slabs using wet joint system with adequate reinforcement bar is proposed. To verify the capacity of the connections as well as the half slab precast concrete systems, the model is compared with the monolith cast in situ slab systems. Comparison results show good agreement between half slab precast and monolith cast in situ slab systems. Though, the half slab precast system still has some discontinuities between the precast slabs components, the additional reinforcement bar ratio in the wet joint system still can resist the acting forces that occur in the corresponding direction. Therefore, based on the modeling and simulation results, the proposed two way half slab precast concrete systems can be applied to the real structural systems.

Keywords: *half slab precast concrete, one way slab, two way slab, precast concrete connection*

INTRODUCTION

Half Slab Precast concrete slab structure is made in the form of a combination of a system of precast and cast in situ. Most of the thickness of the plate at the bottom made of precast and will also function as formwork for the concrete to be cast monolithically there on. So that it becomes an integral plate structure with a total thickness as desired. The use of half precast slab for the floor lately is being rapidly adopted, because the process of slab work faster and easier because it does not require much formwork or scaffolding during construction. Precast component can be made either at the project site or in the concrete plant.

Therefore, in designing the precast floor slabs need to pay attention to the ability of conveyance and lifting equipment is used, it makes a half slab of precast be the right option than full depth precast slab. The use of slab with precast slab system half still found obstacles, such as occur cracks on the precast components plate, as shown in Figure 1. It is alleged in the half slab of precast system for two-way plate (Two Way Slab) where precast plate will behave plate in one direction (One Way Slab), while the component plates are casted on it (overtopping) still behave as Two Way Slab.

Fig.1. Concrete cracking on the precast components plate

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PROBLEMS

The problems that need to be answered in this study is, if the system changes cast in situ (monolith) on the plate into a system of two-way half slab of precast conducted on a large plate panel, so that the precast components had to be split into several parts, but should still be able to plate behaves as a two-way, it is necessary to create a system that can guarantee the connection between the tensile strength of the precast component parts so that the lower layer plate panel as a whole, both part of the bottom layer and the top layer can behave as a two-way plate without adding beam - joist.

SCOOP OF PROBLEMS

This study is limited to the observation of the behavior of the model measuring 220 cm x 220 cm x 20 cm supported on four sides, especially the behavior of the voltage and the deflection caused by monotonic loading. The test specimen is divided into two different thickness type, which is 12 cm thickness of the bottom of the precast components, while 8 cm top is cast in situ component. Precast components lower part is divided into three sections, where the third part in the implementation given a special connection system which is the core of this study, so that the third part of the precast slab can be strung into a single unit that can withstand the torque in both directions. The study was carried out with the help of software analytical Finite Element Method.

RESEARCH METHOD

The methodology used in this study was to study the behavior of the half slab of precast slab system compared to the monolithic system. Before performing the analysis, it is necessary to be designed model proposed model connection by referring to previous research by Irawan et al (2012) which have been made to the model half precast slab with precast components on the bottom plate is equipped with connection systems and tested simple pedestal on two sides as well as studying its behavior. The design of the proposed model of connection between precast components on the half slab of precast then modeled with the help of software finite element method. Analyses were also conducted to the plate with the same dimensions, but made in the monolith, where both models supported on four sides, so it behaves as a two-way slab and plate will be used as a comparator behavior. After analysis to model monolith and precast slab half, then do comparisons and ultimately evaluated for use as the manufacture of conclusion. In summary, the methodology of this study can be seen in the following flow diagram as shown in Figure 2.

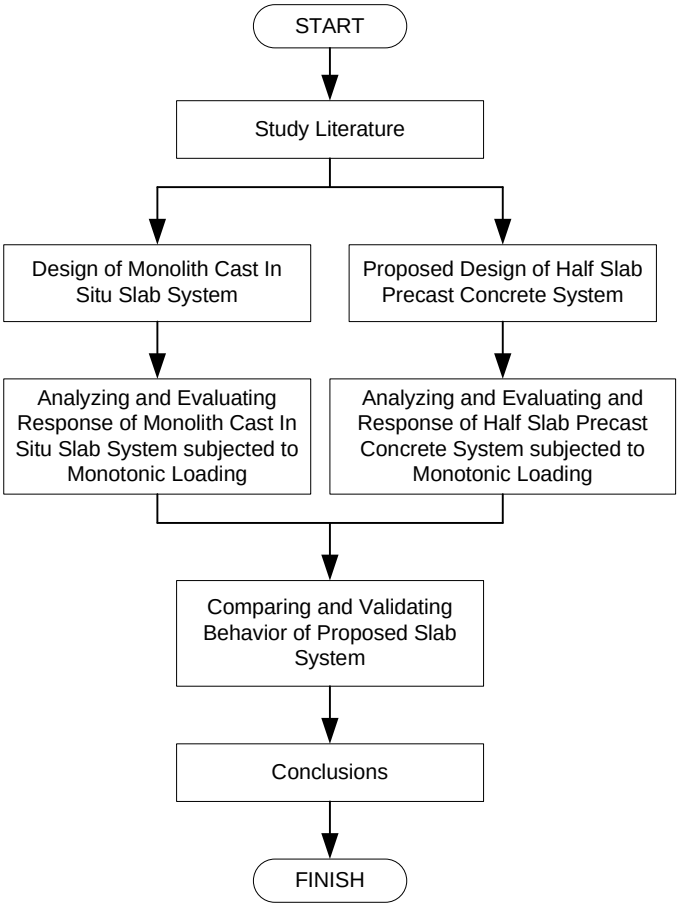

Fig.2. Research method

SPECIMEN MODEL AND MATERIAL PARAMETER

The specimen model used as the object of study is as shown in Figure 3 to Figure 7. Furthermore, material specifications are present as follows:

a. Monolith concrete is made with fc '= 23 MPa and fc '= 31 MPa
b. Precast concrete is made with fc '= 31 Mpa.
c. Reinforcing bar is used 10 mm in diameter with fy = 240 MPa.

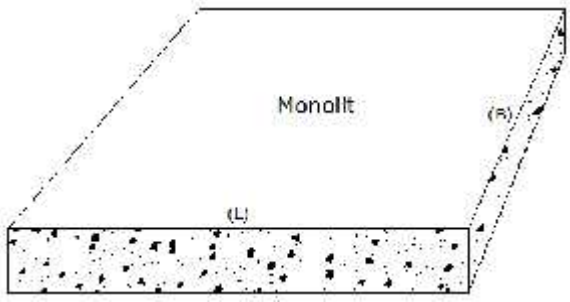

Fig.3. Monolith slab system

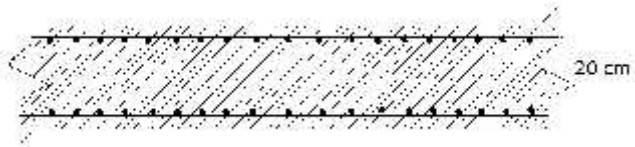

Fig.4. Section and rebar of monolith slab system

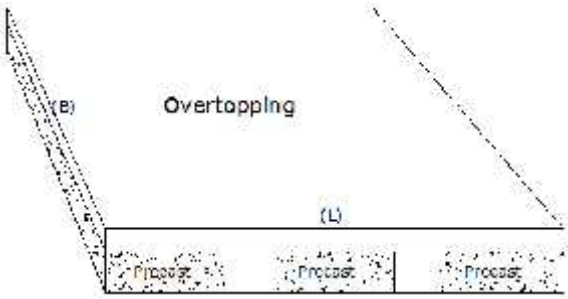

Fig.5. Half-slab precast system

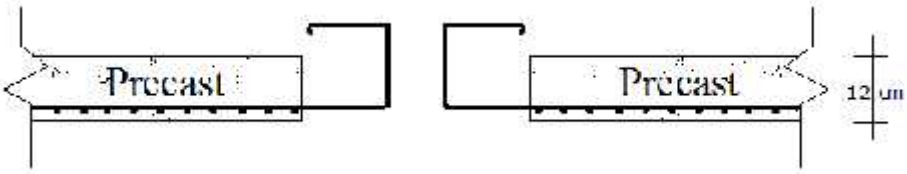

Fig.6. Section and rebar of precast slab system

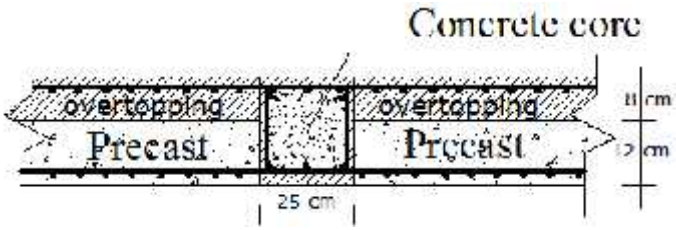

Fig.7. Section and rebar of half-slab precast system

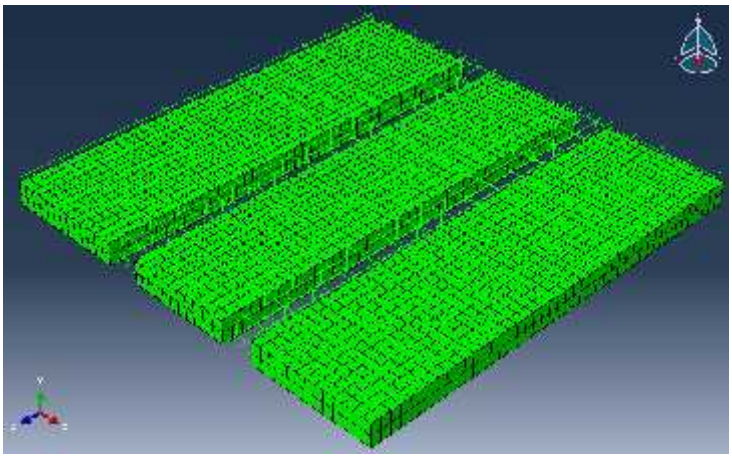

Fig.8. Precast slab system FE model

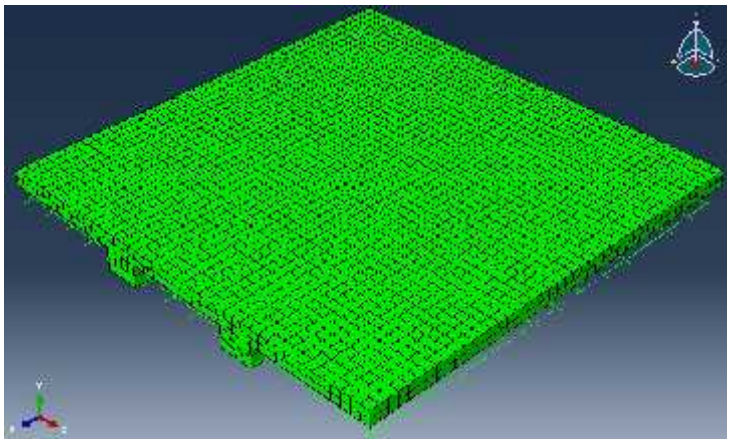

Fig.9. Overtopping slab system FE model

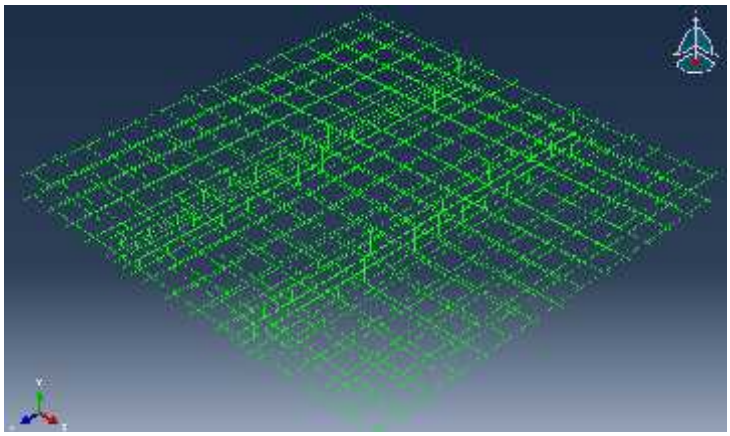

Fig.10. Reinforcing bar FE model

MODELING AND ANALYSIS

Modeling and analysis is conducted with the software of Finite Element Method. The monotonic loading position is applied in the middle of the span with an area of 200 mm x 200 mm and is done gradually as 6 step, namely: 1 MPa, 4 MPa, 7 MPa, 9 MPa, 11 MPa and 13 MPa. Supported restraint is applied on four sides form the foundation of joint support, which is roller supported on two sides and hinge supported on the other two sides. Modeling illustration using finite element software is presents in Figure 8 and Figure 10.

Table 1. Plasticity Parameter

Dilation Angle	Eccentricity	Fb0/fc0	K	Viscosity Parameter
36	0.1	1.16	0.6667	0

Table 2. Concrete compressive strength parameter of the FE model

Model	**Slab System**	**Concrete Compressive Strength**	
		Precast	**Overtopping**
		MPa	**MPa**
A	Half-slab	23	31
B	Half-slab	31	39
C	Monolith	23	
D	Monolith	31	

Analysis was performed in inelastic condition, hence the analysis of Concrete Damage Plasticity incorporating elements taken from the reference approach by FHWA (2006) to the concrete conditions of the press and the formula approach by Frank and Collins (1986). Parameters taken concrete plasticity of reference developed by Kmiecik and Kaminski (2011), as presented in Table 1. Additionally, the concrete compressive strength parameter of the FE model is shown in Table 2.

The illustration of deformed shape of half-slab precast system is presented in Figure 11. Moreover, the calculation of stress and strain relations of concrete in compression and tension condition can be seen in Figure 12 and Figure 13, respectively. Comparison results of both FE model are showed as shown in Figure 14 to Figure 18.

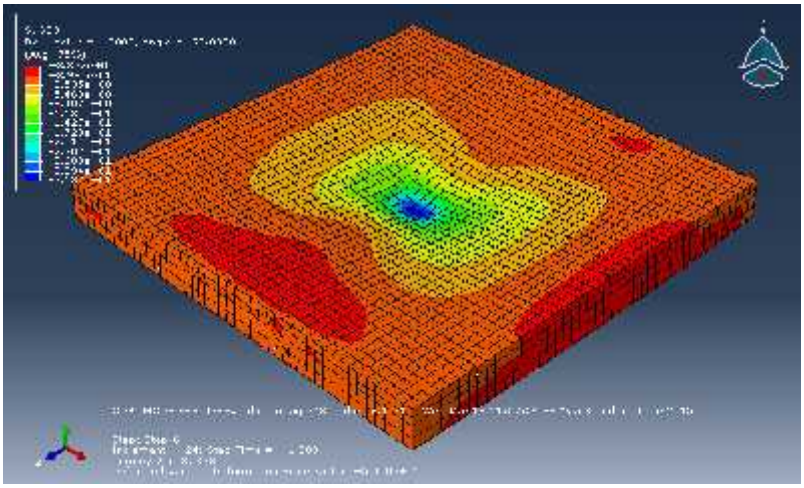

Fig.11. Deformed shape of half-slab precast system

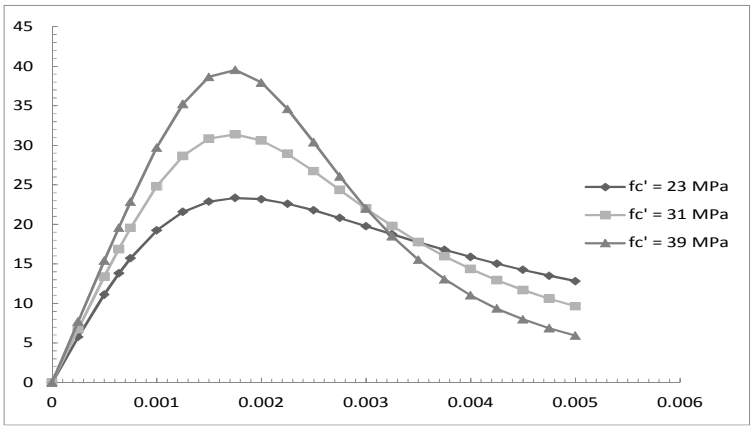

Fig.12. Stress and strain curve of concrete in compression condition

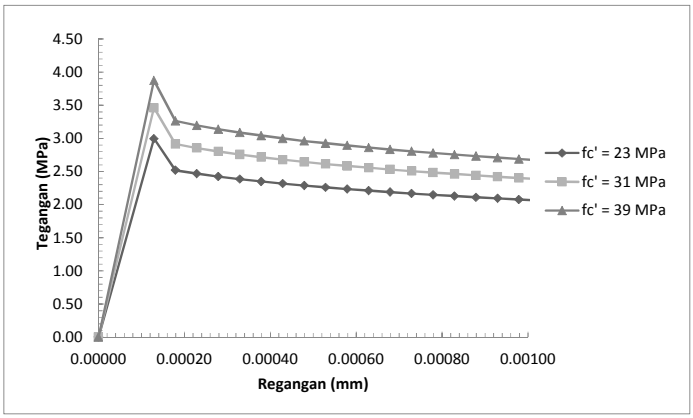

Fig.13. Stress and strain curve of concrete in tension condition

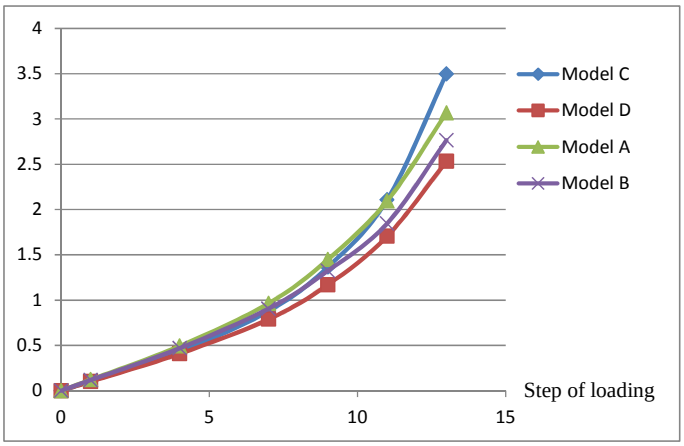

Fig.14. Comparison results of vertical displacement

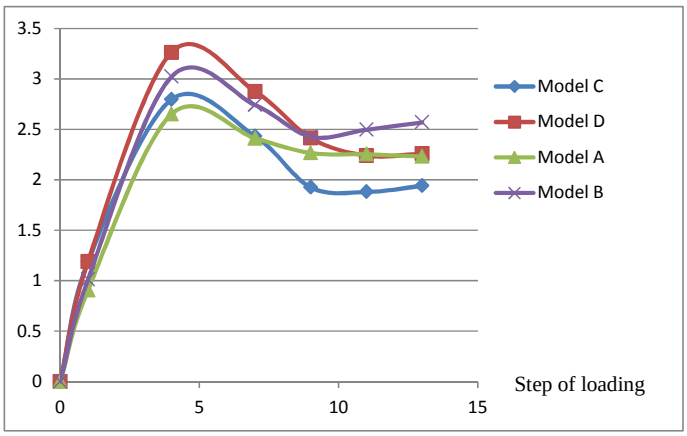

Fig.15. Comparison results of concrete tension stress S11

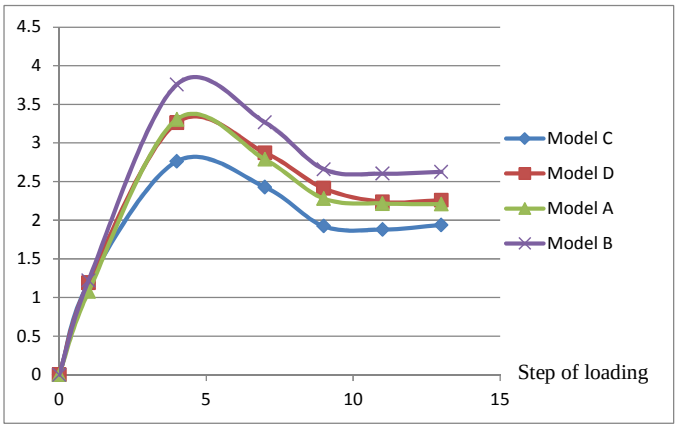

Fig.16. Comparison results of concrete tension stress S22

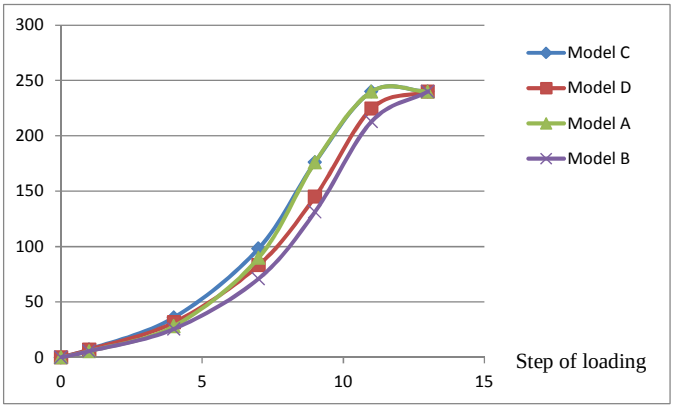

Fig.17. Comparison results of rebar stress in X direction

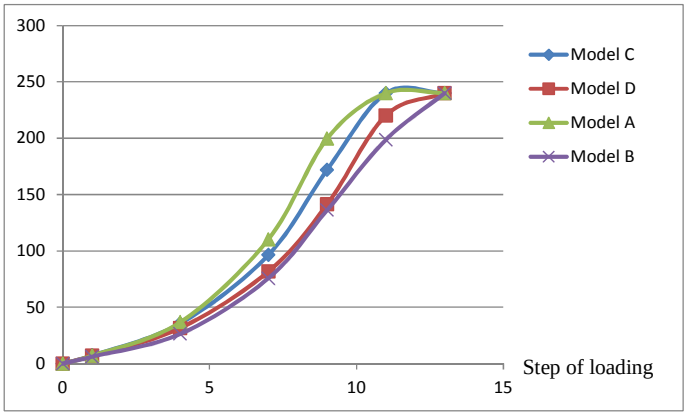

Fig.18. Comparison results of rebar stress in Y direction

RESULTS AND DISCUSSION

From the analysis evaluated the behavior of four different kind of model studies are present as follows:

Deflection

Deflection comparison as shown in Figure 14, shows that Model A has a greater deflection about 20% compared with the Model D, but the has same approach with the Model C. Moreover, the deflection of Model B has similarity with Model A and also slightly larger than the Model D.

Concrete tension stress

Tension concrete stress occurs at middle span of the slab system. As shown in Figure 15 shows that the maximum tensile stress concrete X axis direction (S11) to model a similar approach with the Model C, with a difference of 5% lower. However, when compared with the Model D, the Model A has a lower value of 20%. While Model B has a maximum stress difference of approximately equal or 7% lower than in Model D. Regarding the tension stress at the Z-axis direction (S33) as shown in Figure 16, the maximum stress on the concrete tensile Model A has a value approximately equal to the Model C, although the quality of concrete overtopping just has a quality fc '= 23MPa, while Model B has a higher maximum stress 13% compared with the Model D. This is because the joints between precast in Model A and Model B has more reinforcing bar the Z direction compared to the Model C or D.

Reinforcing Steel Bar Stress

Reinforcing steel bar Stress is also carried out at the bottom of the tensile steel reinforcement. As seen from Figure 17 and Figure 18, indicated that the maximum tensile reinforcement bottom of the Model A and Model B has reached the yielding point (fy = 240 MPa) at loading 11 MPa (Step 5). Furthermore, for the Model C and D, the maximum tensile reinforcement steel bar yielding point reached at 13 Mpa of loading (Step 6).

CONCLUSION

From the explanation of the results and discussion, it can be concluded some of the following points:

a. Differences in behavior that occurs in a system where the half slab of precast components precast plate behaves as a plate in one direction (One Way Slab), while the components overtopping plate behaves as a two-way (Two Way Slab), can be resolved with a joint system proposed in this study to make precast slab half as a single entity as a plate that has the same behavior that is as a two-way plate (Two Way Slab) either on or overtopping and precast components, yet still there was a slight difference in stress and deflection compared to a monolithic system, although not significant. For that further research is needed to validating with laboratory testing.
b. When the construction of a concrete slab on the ground made the change from a monolithic system into a system of precast slab half, hence the need for concrete action to improve the quality of the precast components, while the quality of overtopping can be equal to the quality of the proposed design, and it is also necessary to add reinforcement connection system between components precast as proposed role in this study, so that the behavior of precast slab half as a single entity can behave as a two-way slab system.

SUGGESTION

This study is only done with the supporting of Finite Element Method based software. So as to ensure the correctness of this study, then as it goes, need to be validated by conducting an experiment in the laboratory with full scale condition.

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UTILIZATION OF COPPER SLAG AS A CEMENTITIOUS MATERIAL IN REACTIVE POWDER CONCRETE

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ABSTRACT: This research studies the use of copper slag from a plant in Belgium as a cementitious material in reactive powder concrete (RPC). The quickly cooled granulated copper slag (QCS) was ground intensively using a planetary ball mill. A low water-to-binder ratio of 0.18 was chosen for the RPC in this study. Various concrete and cement paste samples were produced with increasing copper slag contents from 0 to 20 wt% in steps of 5 wt%. Particle size distribution (PSD) and specific surface area (SSA) of the copper slag were assessed using laser diffraction and the Blaine permeability test, respectively. The results obtained, showed that the strength of RPC with different copper slag proportions was similar to or better than the control mixture at 90 days. The presence of copper slag in the paste tends to decrease the total heat production of the paste. The pozzolanic reactivity of QCS determined by the Chapelle test was found to be low.

Keywords : *Copper slag, Reactive Powder Concrete, Compressive Strength, Hydration, Pozzolanic Reactivity*

CONTEXT

In recent decades, concrete has grown to be the most popular material for infrastructure projects due to its relative low cost in production and treatment. However, Portland cement as ingredient of concrete requires a huge amount of energy from fossil fuels and non-renewable resources and for clinker burning and grinding. Moreover, the toxic gases from a cement kiln also contribute to emissions of air pollutants and pose a hazard for human health. The alternative way, to protect the environment and saving energy in the near future, is to utilize recycled waste material within the cement and concrete industry.

The use of industrial by-products as cement replacing material is very popular. Typical by-products that are already widely used are e.g. fly-ash and blast-furnace slag. Copper slag is one of the by-product materials from the copper smelting. In Belgium, about three million tons of slag are produced by the copper industry every year (Flanders Cleantech Association, 2012). In Europe, approximately 40 million tonnes of slag were generated by the European metal industry according to Euroslag data in 2004 (Böhmer et al. 2008). Since this material needs a large area for landfilling, of which the availability is insufficient, and to avoid problems related to the leaching of heavy metal and other harmful elements, it would be interesting to upgrade these 'waste' products in high-value applications. Moreover, the amount of natural resources is declining due to a large consumption in the cement and concrete production. A possible breakthrough can thus be sought in exploiting by-products within cement and concrete production.

Reactive powder concrete (RPC) is classified as an ultra-high performance concrete (UHPC) (Zenati et al. 2009) which has a compressive strength of 200 MPa when applying heat curing and up to 800 MPa when using steel aggregate (Richard 1995). In the RPC compositions, active powders dominate as the main constituents to obtain a relatively dense and homogenous microstructure (Yu et al. 2014), (Zang et al. 2008), (Yasici et al. 2013).

Looking into literature, many researchers have investigated the use of copper slag as cementitious material in high performance concrete. Hydrated lime can be used as activator in high performance concrete since it accelerates the pozzolanic reaction. The highest compressive strength is achieved for 15% copper slag and 1.5% lime at 90 days of curing, which is twice as high compared to the strength at 28 days (Mobasher et al. 1996) and (Tixier et al. 1997). The effect of copper slag as cement replacement in concrete and cement by-pass dust as activator is limited to the compressive strength (Al-Jabri et al. 2002). The positive effect of copper slag on the strength of concrete and mortar is caused by an increasing amount of calcium silicate hydrates formed by the pozzolanic reaction of copper slag and calcium hydroxide from cement during the hydration process (Moura et al. 2007). Opposite results were obtained by De Schepper et al. (2014) when using the copper slag as cement replacement up to 60 wt% in steps of 10 wt%. They found that the effect of copper slag on the hydration process was minor. Copper slag was used in ultra-high performance

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concrete as fine aggregate by Ambily et al. (2014). Their conclusion is that the copper slag has a promising future in the production of UHPFRC.

This paper presents the study of the effect of copper slag as cementitious material in reactive powder concrete. This study is a continuation of previous studies which focus on ultra high performance mortar (Edwin et al. 2015). The author used Glenium ACE 30 as superplasticizer in the current study since Glenium 51 previously delayed the setting time of the mortar.

MATERIALS AND METHODS

Materials

The secondary slag used in this research was quickly cooled granulated copper slag (QCS) from a Belgian recycling plant. This slag was produced by using waste materials such as copper wires, dust from tin production, old television and radiators from cars as raw materials to generate copper, lead, tin and nickel.

Besides copper slag, an undensified silica fume (type 940U, Elkem) was used as SCM. As cement, a CEM I 52.5 N-HS/NA (low C_3A) was used throughout all experiments. An overview of the chemical composition of the binders is given in Table 1.

Table 1. Chemical composition of the applied binders reactive fine aggregate determined by XRF analysis [wt%]

Material	QCS	Cement	Silica fume	Flour (M400)
CaO	7.1	63.7	0.6	0.02
SiO_2	25.9	20.9	94.2	99.5
Al_2O_3	5.9	3.6	1.0	0.2
Fe_2O_3	45.5	5.2	0.5	0.03
MgO	0.8	0.8	0.7	-
Na_2O	0.8	0.2	1.0	-
K_2O	0.2	0.6	1.1	0.05
SO_3	0.4	3.0	0.3	-

Table 2 depicts the RPC compositions used in this research. A very low water-to-binder ratio (w/b. 0.185) was chosen in order to produce ultra high performance concrete (UHPC). To obtain the desired workability, a polycarboxylate ether (Glenium ACE 30 con 35% S) was used. The concrete was made with copper slag contents varying between 0 and 20 wt% in steps of 5 wt%. For all concretes, a flour (type M400, Sibelco) with a d_{50} of 12 µm was used.

Table 2. Composition of the RPC mixtures with copper slag

Material		Reference	5% CS	10% CS	15% CS	20% CS
CEM I 52.5 N HS/NA	(g)	960	912	864	816	768
Copper slag	(g)	0	48	96	144	192
Silica fume (940 U)	(g)	301	301	301	301	301
Sand M31	(g)	1320	1320	1320	1320	1320
Flour M400	(g)	240	240	240	240	240
Glenium ACE	(g)	37	37	37	37	37
Water Total	(g)	233	233	233	233	233
Water Glenium	(g)	24.	24	24	24	24
Water Compensated	(g)	209	209	209	209	209

Methods

Grinding process

Before using the copper slag as cementitious material, QCS was intensively ground using a planetary ball mill. The copper slag was ground at 390 rpm for 6 times 5 minutes. To avoid agglomeration of the slag upon grinding, a superplasticizer (Sika Viscocrete-3095x; 0.122 wt%) was added

The particle size distribution of the copper slag obtained by laser diffraction is given in Figure 1. To disperse the materials, isopropanol was used since it does not react with copper slag. To avoid agglomeration, the copper slag was put in a sonication bath (5 min) before the measurement. An overview of the parameters used to determine the PSD of the SCMs by laser diffraction can be seen in table 3.

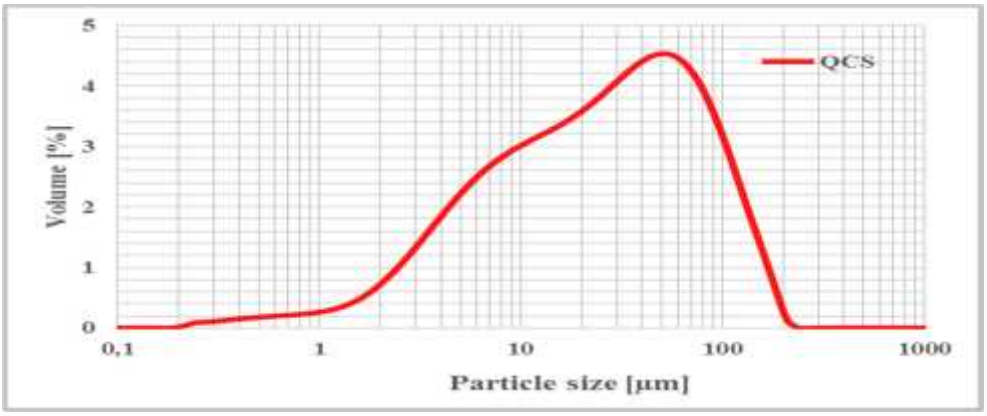

Figure 1 : Particle size distribution of copper slag.

In addition to the PSD by laser diffraction, the fineness of the binders was evaluated by their specific surface area (SSA) using the Blaine air permeability test according to EN 196-6:2010. To start, the pycnometer method was used to measure the density of all binders. Both the density and SSA of the binders are presented in Table 4.

Table 3. Overview of the parameters applied to determine the PSD of the different SCMs by laser diffraction

Optical parameters	Copper slag	Silica fume
Refractive index (RI)	1.731	1.53
Absorption coefficient	0.055	0.001
Obscuration [%]	10 - 20	5 - 10
Stirrer rate [rpm]	1700	1500
Dispersant RI	1.390	1.390
Sonication times [minutes]	5	15

Table 4. Density and SSA of the different binders

Materials	QCS	CEM I	SF
Density (g/cm^3)	3.761	3.152	2.017
SSA (cm^2/g) :			
Blaine permeability	2533	4955	-
Laser diffraction	2610	-	56200

Mixing Procedure

A mixer with two speeds (140 and 285 rpm) was used. First dry powder materials were inserted and mixing was started at low speed (140 rpm) for about two minutes. Afterwards, water and superplasticizer were dosed and mixed for about 1.5 minutes at 140 rpm. The highest speed (285 rpm) was then started for about 6 minutes in order to obtain a homogeneous mixture. Finally, normal speed was applied for about 2 minutes to obtain an optimal workability.

Compressive strength testing

After curing in a room at relative humidity of 95 ± 5%, the specimens were tested to evaluate the compressive strength according to NBN EN 196-1 (2005) at the age of 7, 28, 56, and 90 days.

Isothermal Calorimetry

Isothermal calorimetry was carried out at 20°C on cement pastes that were mixed manually using a small container. After 4 minutes of dry mixing, the water and Glenium ACE 30 were added and mixing continued for another 2 minutes. Afterwards, around 14 g of paste was injected into an ampoule using a modified syringe. The water-to-binder ratio of the cement pastes was 0.185. Similar to the RPC, the cement pastes were made with an increasing copper slag content from 0 wt% to 20 wt% in steps of 5 wt%. More details about the paste composition are given in Table 5.

Table 5. Paste composition.

Material		Reference	5% CS	10% CS	15% CS	20% CS
CEM I 52.5 N HS/NA	(g)	25.00	23.75	22.50	21.25	20.00
Copper slag	(g)	0.00	1.25	2.50	3.75	5.00
Silica fume (940U)	(g)	7.84	7.84	7.84	7.84	7.84
Glenium ACE 35% solid	(g)	0.97	0.97	0.97	0.97	0.97
Water Total	(g)	6.08	6.08	6.08	6.08	6.08
Water Glenium	(g)	0.63	0.63	0.63	0.63	0.63

Water Compensated	(g)	5.45	5.45	5.45	5.45	5.45

Chapelle test.

To evaluate the pozzolanic activity of copper slag, the Chapelle test was performed according to NF P 18-513. In this test, 2 g of pozzolan and 1 g of CaO were inserted into an Erlenmeyer. In order to obtain decarbonized water, 250 ml of distilled water was boiled in a heater to remove the CO_2. The agitation system was started using a magnetic stirrer to create a circular motion in order to obtain a homogeneous material after mixing. During agitation, the material was boiled at constant temperature (90 $^{\circ}$C) during 16 hours to facilitate the reaction of the solid particles and the liquid. The test reflected positive if there is no evaporation throughout the heating process. A smooth condenser column with circulating water was installed to create a cooling system to prevent evaporation. A reference mixture was prepared using the same method by mixing 1 g of CaO and 250 ml of distilled and decarbonized water. After cooling at room temperature, the solution was mixed with 60 g of sucrose and shaken for about 15 min using a magnetic stirrer in order to extract non reacted calcium hydroxide. After filtering, the solution was titrated using 0.1M HCl with phenolphthalein as indicator instead of methyl orange. This test is considered positive if CaO is consumed by the pozzolan.

RESULTS AND DISCUSSIONS

Figure 2 describes the effect of copper slag replacement on the compressive strength of RPC at 7, 28, 56 and 90 days. In general, the strength of RPC at 7 days decreases with rising copper slag substitution in the concrete mixture. The strength increase for RPC with CS between 7 and 28 days was similar as for the reference mortar. However, for longer curing periods the strengths of mixtures with 10% CS and 20% CS were higher than those of the reference mixtures. Although the strength development of the 5% CS mixture was delayed at early ages, it was higher than that of the reference specimens at 90 days. The highest compressive strength (165 MPa) was achieved for 10% copper slag at 90 days of curing, which increased about 7% compared to the control specimens. The lowest RPC strength at 90 days (150 MPa) was obtained for 15% copper slag replacement. Nevertheless, the strength of this mix remained stable between 28 and 90 days as well as for the control mixture as seen in figure 2. Generally it can be stated that the use of the finely ground copper slag for up to 20% cement replacement will have no or a positive effect on the strength of RPC.

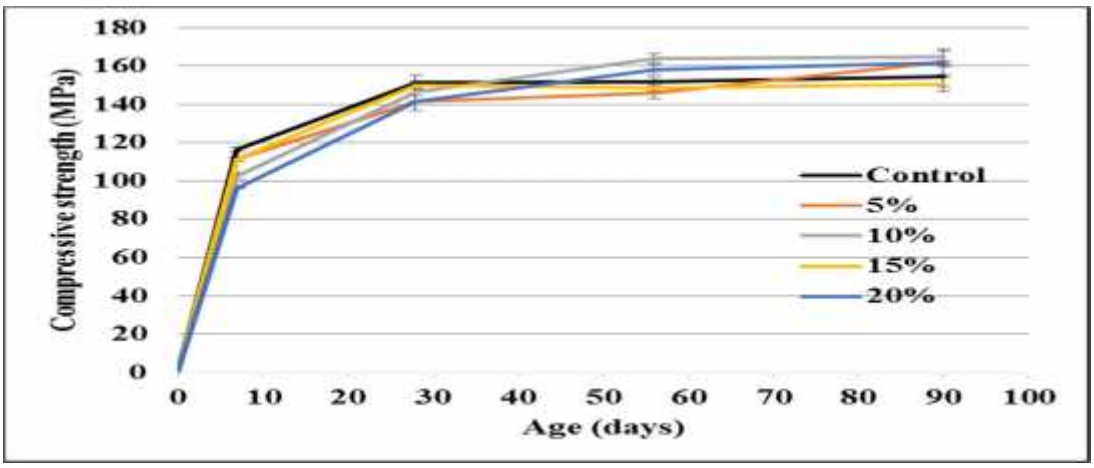

Figure 2 : RPC compressive strength results (error bars represent standard errors)

Figure 3 shows the process of the heat production in function of time for all mixes. The maximum value of the heat production rate during the second peak decreases with increasing copper slag replacement, and comes later in time. Furthermore, the maximum heat production rates for pastes containing copper slag are lower compared to the reference. This phenomenon is probably caused by heavy metals present in the copper slag [Zain et al. 2004]. This can be due to the dilution of the clinker content in the paste and to the limited pozzolanic activity of the copper slag during the first weeks of hydration. Also heavy metal compounds such as Zn, Pb, and Cu may play a role in the delay of the setting time and hydration process of the cement paste containing copper slag. In comparison with earlier work of the authors (Edwin et al. 2015), the superplasticizer is no longer responsible for a retardation of the hydration process of the cement (Glenium ACE 30, a new generation of polycarboxylic ether polymer superplaticizer, was used in this research, instead of Glenium 51 in Edwin et al. 2015).

The cumulative heat production is shown in Figure 4. It is seen that the highest total heat production is achieved for the reference. In addition, in this study, an increase of the copper slag amount did not further promote the increase of total heat production of the paste. This is in line with the discussion presented above.

In this study, the concentration of CaO consumed by 2 g of copper slag was lower than for the reference. These test results indicate the non-reactivity of the copper slag with CaO to produce calcium silicate hydrates, giving an explanation for the obtained lower hydration heats in figure 3 and 4. The slightly higher strength results obtained are probably related to an improved microstructure. In subsequent research it will be studied whether the reactivity can be improved by obtaining higher fineness.

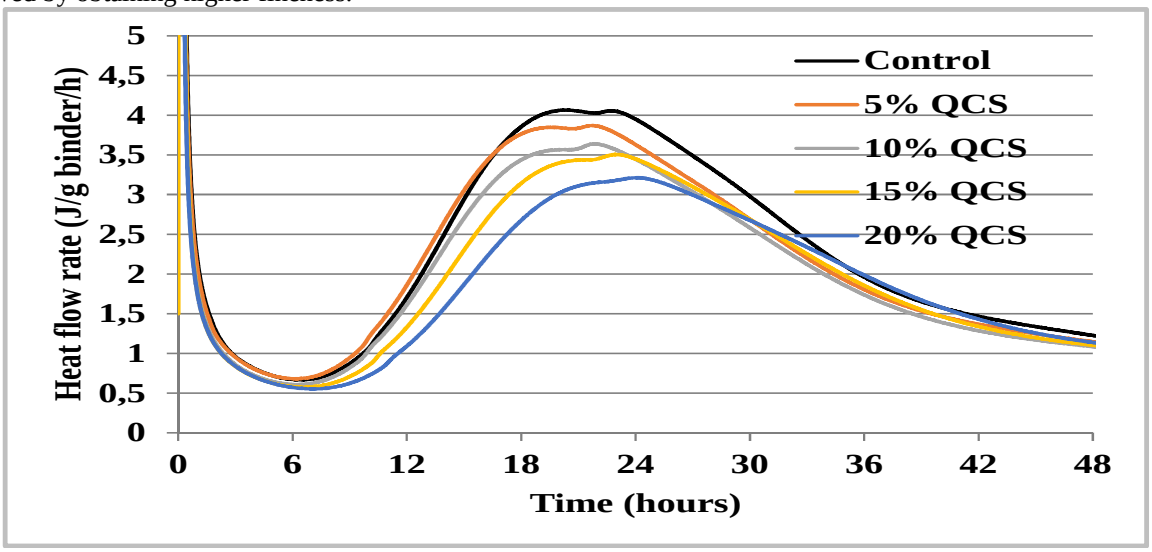

Figure 3. Influence of copper slag addition on the binder hydration

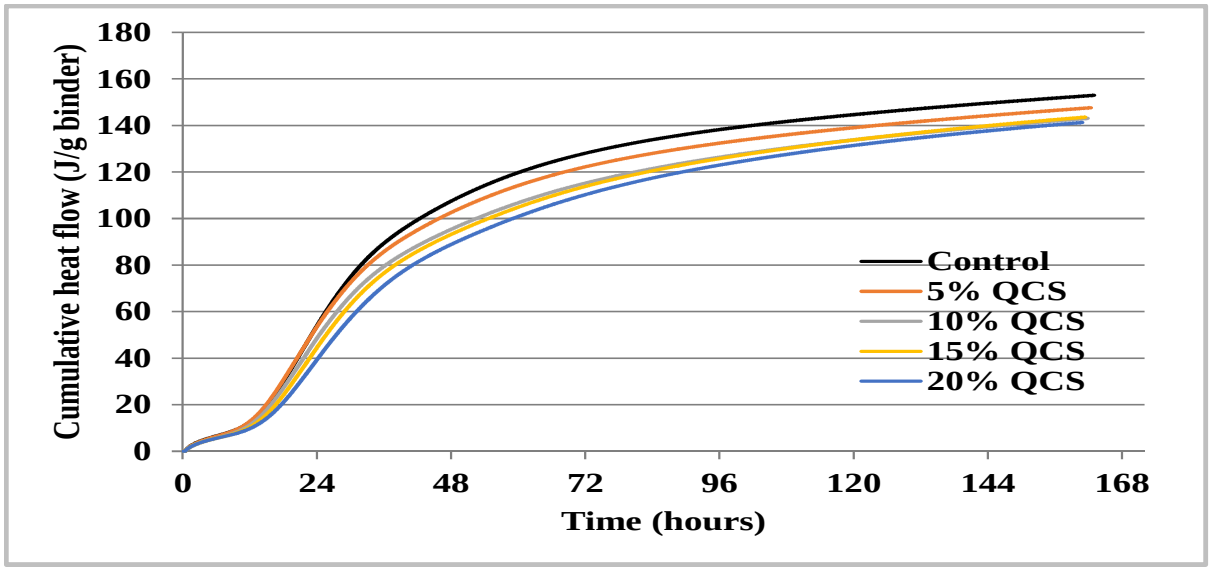

Figure 4: Total heat production for pastes with copper slag

CONCLUSIONS

Within this paper the utilization of quickly cooled copper slag (QCS) as cementitious material in Reactive Powder Concrete (RPC) was investigated. The following conclusions can be made:

1. The use of copper slag as cement replacement up to 20% has no adverse effects on the compressive strength. For some replacement levels minimal strength increases at later ages were observed. The presence of copper slag in the cement paste decreases both the maximum of the heat production rate and the total heat production after 7 days.
2. The copper slag used in this research is not reactive with CaO (Chapelle test) and is thus not considered to be pozzolanic. This explains the effects of copper slag on the heat production of the cement paste. The minimal effect on the compressive strength is probably related to an improved microstructure in the presence of copper slag.

ACKNOWLEDGEMENTS

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DAMAGE INVESTIGATION AND SEISMIC RETROFITTING OF WEST SUMATRA GOVERNOR'S OFFICE BUILDING AFTER 2009 SUMATERA EARTHQUAKE

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ABSTRACT: West Sumatra Governor's office building is one of the buildings that damaged by September 30, 2009 Sumatra Earthquake. The earthquakes was 7.6 Richter Scale centered in the ocean around 100 miles from Padang city. Damage of the building structure was identified after demolition of all the partitions and column cover panels on September 2014. In order to know the cause of the building damaged, an investigation has been done. This paper discusses the result of a building assessment to find the causes of the damage and the recommendation method for retrofitting the building structures. Based on the results of field investigation, structural damages occurred on almost all columns in first floor due to lack of shear reinforcement. A numerical simulation was carried out using structural analysis software (ETABS 9.7.4) to evaluate the capability of building structures to resist loads, especially earthquake load based on new Indonesian Earthquake Standard Code (SNI 03-1726-2012). The result of simulation indicates that the damaged of the columns was shear failure due to the big distance of shear reinforcement. Retrofitting of the structure was proposed using two methods: concrete jacketing and installed steel bracing. Results of structural analysis for retrofitted building using combine jacketing of all columns in first floor and installed steel bracing in the corners of the building until third floor shows that the building have good performance to resist the earthquake load based on SNI 03-1726-2012.

Keywords: *earthquake, damage, retrofitting, jacketing, steel bracing*

INTRODUCTION

Indonesia was in the area of four tectonic plates main namely eurasian plates, indo-australia, pacific and the philippines so that known causing much of earthquake due to movement on the plates. On 30 september 2009, an earthquakes 7.6 SR has hit West Sumatra Provinces (BMKG, 2009).This earthquake centered on coordinates 0°50'24" south latitude and 99°39'0" east longitude. Padang as the capital of the province with a distance of 100 km from earthquake center couldn't escape severe damage especially on multistoried building (Fauzan et.al., 2010). With the depth of the quake 70 km, this was recorded as hardest earthquakes hit padang since standing this city (Ismail et.al.2011).

One building that did not escape the damage is the West Sumatra Governor's Office building located in Padang. This building was built in 1961 with 10 years construction period. The major structural component of the building is reinforced concrete. Conditions Governor of West Sumatra Governor's office buildings that suffered structural damage after the earthquake that occurred in 2009, especially the first floor will be identified when the repairs/renovations will be started. This happens because of finishing the building, especially the finishing column using wood panel materials that are not monolithic columns so that cracks occur in the column is not visible from the outside. The cracks visually looks after the demolition of the wooden panel column cover. With the crack in this column is feared the building is not safe so that the necessary repairs and retrofitting of structures. 1st floor of the building structure damage before and after all demolished column cover panel can be seen in Figure 1.

(a) (b)

Figure 1. Existing Condition of Structure ,(a) after earthquake, (b) after demolished column cover

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Based on the above it is necessary to do an analysis of the existing condition of the building. The analysis carried out on the damage and repair methods to be implemented to restore the building to be worthy of reuse.

EVALUATION OF EXISTING BUILDING STRUCTURE

Before analyzing the existing condition of the building structure is necessary to collect data and information on the structure of the building. Preliminary data obtained are building repair planning documents Governor's Office of West Sumatra in 2012 from the Department Prasjaltarkim West Sumatra. To match that data with the actual conditions, the field investigation must be done. Data needed the dimensions of the building, the size of the cross-section beams and columns, concrete quality and reinforcing details attached. Looked outside the office building of West Sumatra governor can be seen in Figure 2.

Figure 2. Governor's Office of West Sumatra

Quality Concrete and Reinforcement Details

To get the existing concrete quality testing by performing test hammer. Hammer test results showed that the quality of the existing concrete is K-225. Meanwhile, the amount of reinforcement is obtained by performing a visual data collection (on the broken column) and ferroscan tool. Quality reinforcement used for the analysis is reinforced U-24 (fy = 240 MPa). Quality reinforcement is based on planning data obtained from the Department Prasjaltarkim West Sumatra.

Damage Columns and Beams

Column damage occurs in the middle column (column 30/80) as shown in Figure 3. Visually visible cracks on the slanted column, especially the central part of the column so that the column reinforcement is clearly visible. In some middle column even concrete cover can be detached and the distance and position of reinforcement (Figure 3a). Damage to the column caused by shear failure which is characterized by sloping cracks in the middle of the column (Figure 3b).

(a) (b)

Figure 3. Damaged Column

Damage in this column, is caused by the flexural reinforcement mounted one direction and shear reinforcement columns fitted with a distance too far (± 30 cm). This condition is suspected as the cause of damage / failure due to lack of shear reinforcement column. Location columns 1st floor damage can be seen in Figure 4.

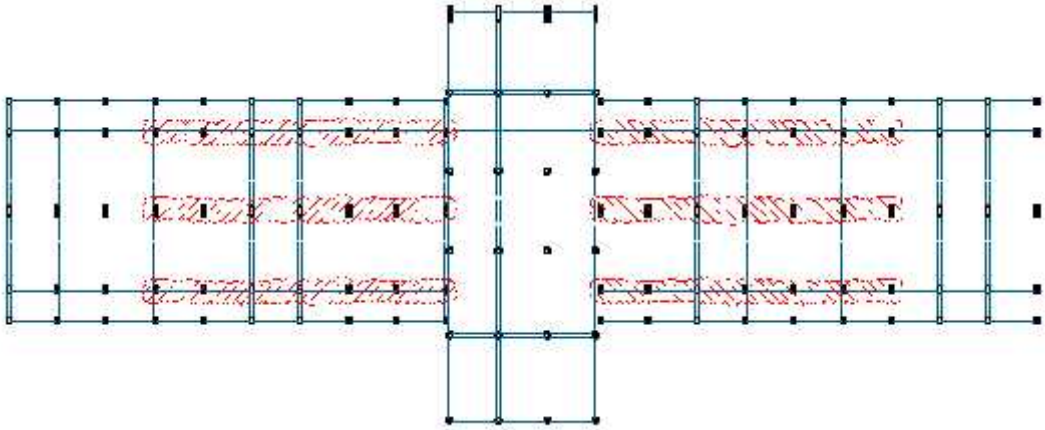


Figure 4. Location of damaged columns

In addition to damage to the columns, there is also damage to the beam discharge of concrete covers the lower part of the 2nd floor beams at multiple locations. This beam damage including minor damage. Damage to the beams can be seen in Figure 5.

Figure 5. Damaged of beam on second floor

Structural Analysis of Existing Condition

Based on the data that has been described previously as well as by function and location of buildings, structures of West Sumatra Governor's Office building modeled and analyzed with ETABS 9.7.1 (Computers and Structures Inc., 2005). Analysis of seismic loads using dynamic analysis (spectral response) to the city of Padang earthquake by Earthquake Spectra SNI 03-1726-2012 (State Agency for Standardization, 2012) with the help of the online application program "Design spectra Indonesia" issued by Puskim PU in 2011, where to get earthquake response spectrum based global coordinates of the location of the building. After analysis of the structure, the cross-section capacity analysis based on the force in that case.

Based on the analysis capacity of the column cross-section through the interaction diagram PM, the entire column considered able to withstand axial loads that occur but most fields are not able to withstand shear forces that work for shear capacity of the column is not sufficient. Reinforcement diameter of 8 mm were installed at a distance of 30 cm is not sufficient capacity is required. As for the beams, reinforcement installed currently have sufficient capacity to carry the load acting on the structure.

The maximum displacement of West Sumatra governor's office building can be seen in the image below. It can be seen that the dicplacement that occurs is large enough so that there was great potential pounding of the building.

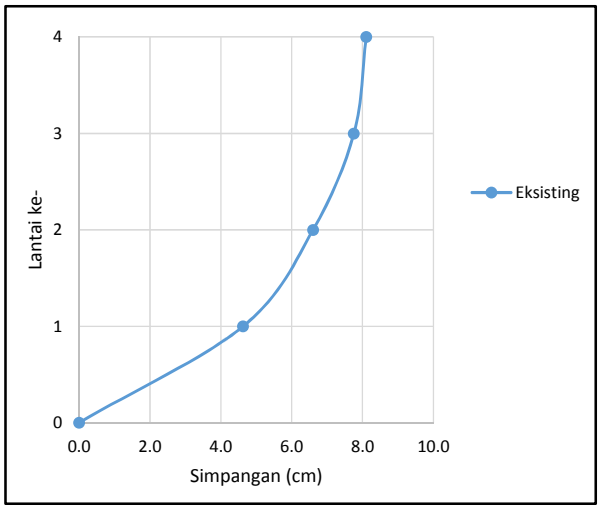


Figure 6. Maximum Displacement of Left/Right Building (x-dir)

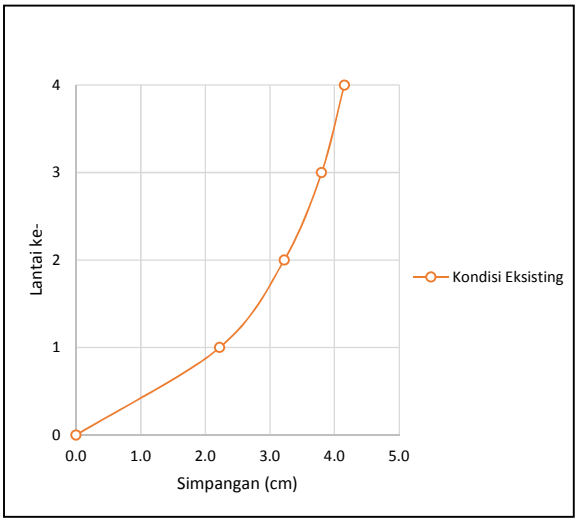


Figure 7. Maximum Displacement of Center Building (x-dir)

The Fundamental Structure Period (T)

The fundamental structure period (T) shall not exceed the upper limit of the coefficient for the period is calculated (Cu) and a period of fundamental approach (Ta). Fundamental period in seconds is specified in the equation listed in SNI 1726-2012 7.8.2.1 article as follows:

$$Ta = C_t h_n^{\ x}$$

h_n is the height above ground structure up to the highest level structure and coefficients of C_t and x based on SNI 1726-2012. Based on the requirements of the natural structure of the vibrating time Ta = Cu .0,0466 . $17{,}5^{0{,}9}$
The value of time limit structure of the natural vibration of Ta = 0,857 dt. From the analysis with software obtained the value of period structure of the existing condition of the left/right building 1.371/dt and 0,779 dt for the center building. It is concluded from the data structure of the existing building left/right is too flexible and does not meet the requirements of the applicable maximum period.

From the results of the evaluation of the performance and the strength of the structure against earthquake regulations in effect that SNI 03-1726-2012, building structures that are not obtained as able to withstand loads that work especially the lateral load (load of the earthquake), so that the structure retrofitting of the Office of the Governor of West Sumatra should be done.

RETROFITTING METHODS OF BUILDING STRUCTURE

In retrofitting there are two kinds of methods that can be done, local retrofit and global retrofit (Central Public Works Department and the Indian Building Congress, 2007). Local retrofit is a method of repair and strengthening of structures on certain structural elements (columns, beams, sloop), one of which is concrete jacketing. Global Retrofit is

a method of strengthening and stiffener structure by adding a retaining elements lateral loads on structures such as shear wall (shear wall) and steel bracing. In this paper retrofitting method used is the method of concrete jacketing and installation of steel bracing. Based on the results of research conducted by Fauzan et al (2011), an increase in the capacity of the round column-jacketing on the building of the Grand Mosque Andalas is significant both flexural and shear capacity of the column. For seismic retrofitting of buildings as a whole have been mounting steel bracing.

Jacketing Method

The method can be performed for retrofitting Office of the Governor of West Sumatra is concrete jacketing. Concrete jacketing is done by enlarging the cross-sectional dimensions and increase the amount of reinforcement columns. In this study, column retrofitting is done by installing a wiremesh type of M5-150 with quality fymin = 400 MPa. and enlarge the column dimensions of 30x80cm to 40x90cm. Wiremesh use have been easier in the process and does not take long when assembling reinforcement.

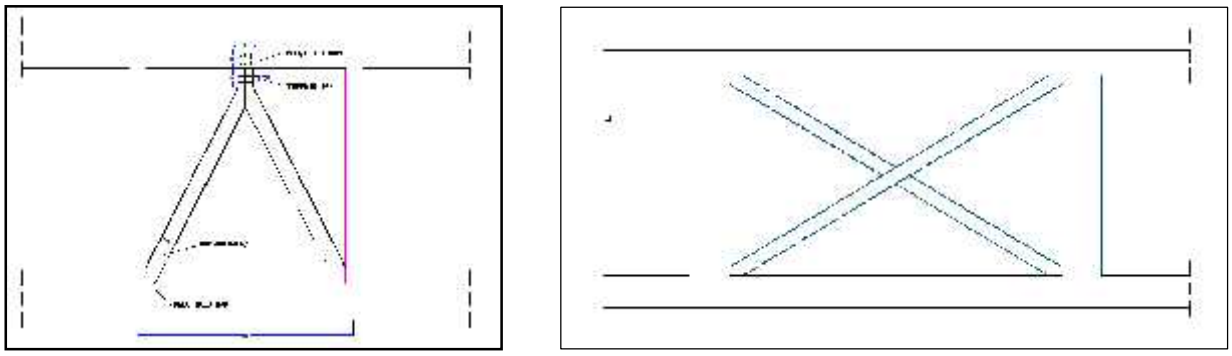
The calculations show that shear capacity of the column cross-section can be improved by installing wiremesh on column. Wiremesh planned as shear reinforcement columns receiving shear load (lateral). In addition, the increase in the capacity of the column is also given by the dimensions of the column that is larger than the existing condition. Increased column shear capacity 1st floor after in-jacketing reached 60% compared to the existing condition. In Table 1 are presented first floor column capacity ratio after jacketing compared to the existing condition.

Table 1. Capacity Comparison of Jacketing

Tipe	Kapasitas Kolom	Kondisi Eksisting	Jacketing dengan Wiremesh	Rasio Peningkatan
K5	Momen Rencana (kNm)	550.653	630.825	14.56%
	Aksial Tekan Rencana (kN)	2974.556	3999.053	34.44%
	Kuat Geser Rencana (kN)	165.620	248.668	50.14%
K4	Momen Rencana (kNm)	346.372	370.479	6.96%
	Aksial Tekan Rencana (kN)	2312.355	3122.107	35.02%
	Kuat Geser Rencana (kN)	121.454	190.158	56.57%
K3	Momen Rencana (kNm)	276.055	300.163	8.73%
	Aksial Tekan Rencana (kN)	2069.429	2798.206	35.22%
	Kuat Geser Rencana (kN)	99.372	160.903	61.92%

Installation of Steel Bracing

Installation of steel bracing on reinforced concrete portal is one method that can be applied to the building. This method is one option for implementation methods that are not too complicated. In this study determined the location of the placement of steel bracing itself by analyzing the structure with the placement of a wide range of locations. Models of steel bracing used can be seen in the following figure.



(a) **(b)**

Figure 8. *Steel Bracing* Type V-Inverted (a) V-Inverted ; (b) Type-X

The structure of the building consists of three buildings separated by two dilatation. In 1726-2012 SNI explained that the building is structurally separated must have sufficient distance to avoid damaging collisions. Separation of the structure must be able to accommodate a maximum displacement with equations that have been defined in the regulations. From the analysis of the structure of the existing condition of the building was found that the maximum displacement is potentially causing the collision, the installation of steel bracing is done on all three buildings. Steel bracing location can be seen in Figure 9.

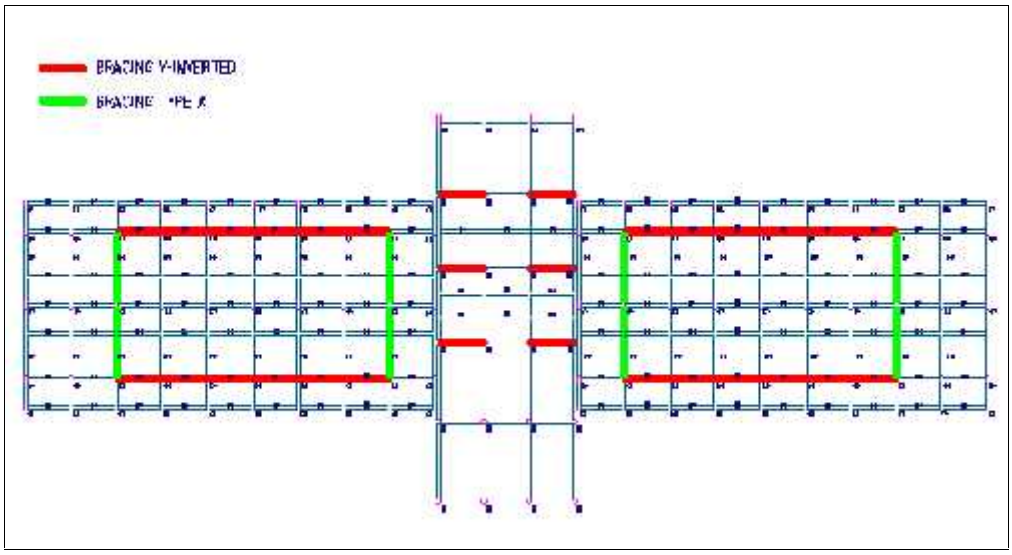


Figure 9. Location of steel bracing

From the analysis of the structure obtained the maximum displacement of the two buildings is smaller than the existing condition (Figure 10 and Figure 11). Installation of steel bracing expected to minimize pounding between buildings.

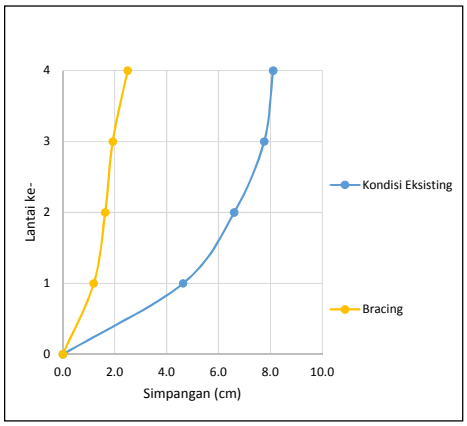


Figure 10. Maximum displacement left/right building (x-dir)

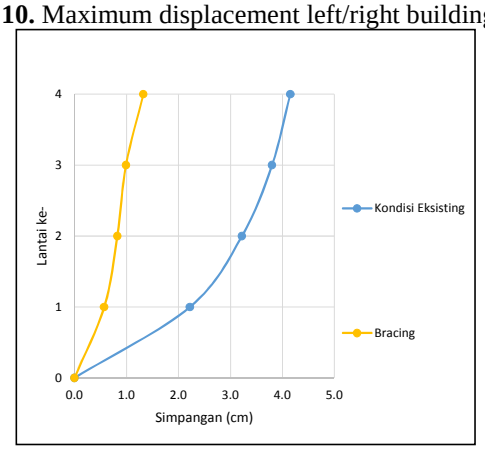


Figure 11. Maximum displacement center building (x-dir)

Fundamental period of the structure

Fundamental period of the structure (T) must not exceed the upper limit results for the coefficients of the period are counted (Cu) and approach the fundamental period (Ta). Natural vibrating structure is Ta = 0.857 sec. From the analysis of the value obtained left/right building 0,467 dt and 0.496 dt to the center building. From this data deduced the structure of the building after the installation of steel bracing has met the requirements of the applicable maximum period.

Retrofitting Implementation method

The first step that needs to be done to the West Sumatra Governor's office building are doing the restoration actions on the elements of structure. Restoration needs to be done on the part of the element structure gets damaged (cracked). Restoration actions that can be implemented (Boen, 2009):

1. Cracked small on the beams and columns injected with cement or epoxy materials. Small cracks are cracks that have wide gaps less than 0.6 cm.
2. The elements of the structure that had large cracks should be dismantled and replaced with a new concrete.

After the act of restoration was done the next step is retrofitting action (strengthening). By installing a wiremesh jacketing method is more practical because it does not need to dismantle the entire column of concrete. Concrete column shelled approximately approx 0.5 cm. And then mounted around the column surface wiremesh and casted again with a thickness of 5 cm so that the dimensions of the cross-section of a column is larger than the original size. Form fitting wiremesh can be seen in Figure 12.

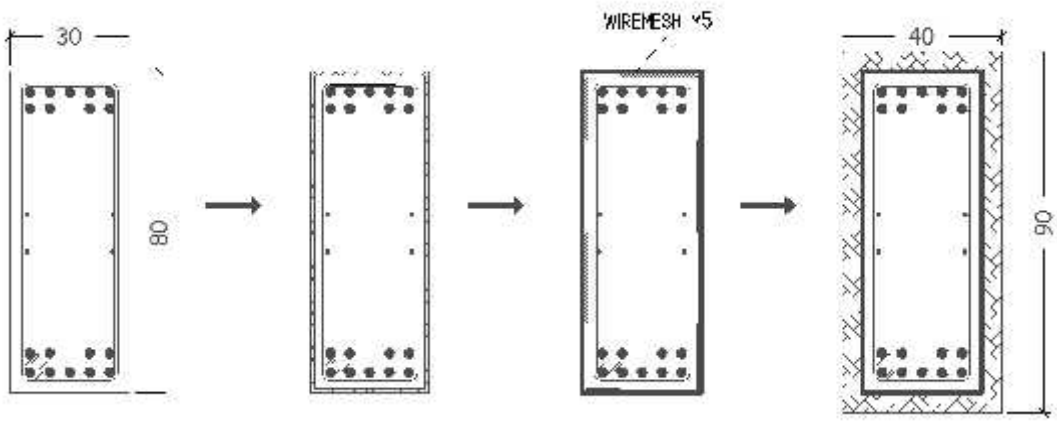


Figure 12. Jacketing methods of column

Conditions of implementation method of the jacketing was held on West Sumatra Governor's office building can be seen in Figure 13.

Figure 13. Process of wiremesh installation

CONCLUTION

Based on the investigation and analysis of the condition of structure of the West Sumatra Governor's office building can be summed up as follows:

1. The main damage which occurred in the structure of the Office of the Governor of West Sumatra which is damage to the concrete columns of the 1st floor, caused by shear failure. Reinforcement of shear mounted on a column do not meet the needs and requirements that apply.
2. Recommended method for retrofitting West Sumatra Governor's office building that is a combination of jacketing method and installation of steel bracing.

ACKNOWLEDGEMENTS

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EXPERIMENTAL STUDY ON SCANNING ELECTRON MICROSCOPE OF ASPHALT CONCRETE BINDER COURSE (AC BC) MIXTURE USING BUTON GRANULAR ASPHALT (BGA)

Abdul Gaus [1], Wihardi Tjaronge [2], Nur Ali [3] and Rudy Djamaluddin [4]

ABSTRACT: Indonesia has a natural asphalt known as buton. Levels of bitumen in asbuton varies from 10 -40%, even at several locations found with 90% bitumen content that can be found in the area Kabungka and Lawele. Asbuton have a big enough deposit of around 600 million tons, equivalent to the deposit asbuton estimated 24 million asphalt oil. This study was conducted to determine the chemical composition of the constituent BGA with SEM, EDX, the effect of the use of BGA in the asphalt mixture to strain. EDX results showed constituent of BGA BGA consisting of carbon (C), oxygen (O), calcium (Ca), silicon (Si), sulfur (S), fluorine (F) and aluminum (Al). The content of 43.1% carbon is the most dominant element in point BGA then oxygen at 38.3% and 1.5% sulfur. Asphalt mixture using 5% BGA stress shows an increase of 25% compared with using BGA, it gained the maximum stress 3.54 MPa.

Keywords: *Buton Granular Asphalt, SEM, EDX, Asphalt Concrete.*

INTRODUCTION

The current asphalt Indonesia still imports petroleum asphalt to meet domestic needs. Required to use local materials to reduce imports of petroleum asphalt 60/70 to meet the needs of rehabilitation and new road construction. Indonesia has a local asphalt that has not been utilized to the fullest. It found on the island of Buton in Southeast Sulawesi.

Asbuton estimated to have deposits of about 677 million tonnes of mineral contents of about 20% to 80% (furgon 2006 and hermadi 2008). asbuton use in hot mix asphalt can affect the properties of the asphalt mixture as a binder properties and characteristics marshall (hermadi 2008). The current asphalt Indonesia still imports petroleum asphalt to meet domestic needs. Required to use local materials to reduce imports of petroleum asphalt 60/70 to meet the needs of rehabilitation and new road construction. Indonesia has a local asphalt that has not been utilized to the fullest. It found on the island of Buton in Southeast Sulawesi (gaus, 2014).

Investigate the stress-strain relationship for asphalt concrete in compressive to described the limit of elasticity (Starodubsky 1994)

MATERIALS AND METHODS

This paper using the asphalt mixture is mixed with natural aggregate stone crusher, asphalt binder used oil pen. 60/70, and used asbuton that has mechanically processed with a crusher is BGA (ministry of public works).

Buton Granular Asphalt

This paper used a type of grains asbuton processed is BGA. BGA has a relatively uniform grain size with a maximum grain size of 1.18 mm. Table 1 shows the properties of the BGA.

Table 1. Some Properties of BGA

Parameter	Value
Bitumen content of BGA (%)	23.00
Asphalt mineral levels (%)	77.00
Water content (%)	1.70
Penetration of bitumen (dmm)	16
Melting point of bitumen (^oC)	86
Flash point before extract (^oC)	168

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Figure1. Buton Granular asphalt (BGA)

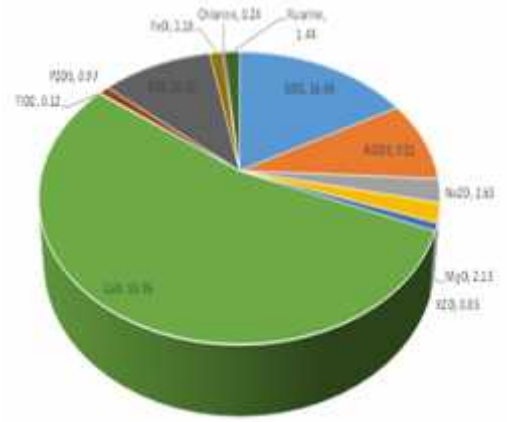

Figure 2. SEM RDX buton granular asphalt

Petroleum Asphalt

Asphalt concrete mixture in Indonesia generally use petroleum bitumen 60/70. Standard properties of the asphalt is Showed in Table 2.

Table 2. Some Properties Of Petroleum Asphalt

Parameter	Value
Penetration (25°C)	65
Softening Point (^{o}C)	52
Flashpoint (^{o}C)	310
Ductility (25°C)	110
Specific gravity	1.01
Weight (with TFOT)	0.2
Penetration after TFOT	54
Ductility after TFOT	25

Table 3 shows the results electron dispersi x-ray of petroleum asphalt**.**

Table 3. Chemical Compounds Petroleum Asphalt

Elements	**Percentage**
Carbon	93.50
Oxygen	4.80
Sulfur	1.70

Carbon is the main constituent elements of the petroleum asphalt with a large 94.4% of the total weight of the asphalt

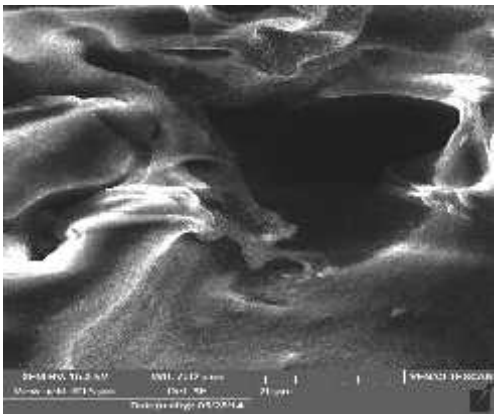

Figure 3. SEM petroleum asphalt

Aggregates

Crushed stone and river sand were used as coarse aggregate and fine aggregate, respectively. These aggregates were from Jeneberang river, Gowa Indonesia. The physical properties of coarse aggregate and fine aggregate are showed in Table 3 and Table 4.

Table 4. Some properties of course agregat

Parameter	Value
Coarse Aggregate	65
Abrasion (%))	22.64
Affinity to Asphalt (%)	96
Particle flat and oval	8.21
Material through sieve No. 200	0
Absorption (%)	2.37
Bulk Density	2.55
SSD Density	2.61
Apparent Densyty	2.72

Table 4. Some properties ff fine agregat

Parameter	Value
Fine aggregate	
Sand equivalent value	83.4
Material through sieve No. 200	7
Angularity	90
Absorption (%)	2.29
Bulk Density	2.55
SSD Density	2.61
Apparent Density	2.71

Asphalt Mixtures

This paper studied AC BC mixture without BGA, AC BC mixture with 5% of BGA, and AC BC mixture with 8% of BGA. All mixtures used a aggregate combination as shown in Figure 2. The aggregate combination met the requirement of mixture AC BC with BGA.

Asphalt bitumen and aggregates of all AC BC mixtures were mixed at temperature of 150°C, and compacted into the

cylindrical mould with capacity of 1,200 gram and diameter of 101.6 mm, respectively. The specimens were compacted with 75 blows each face.

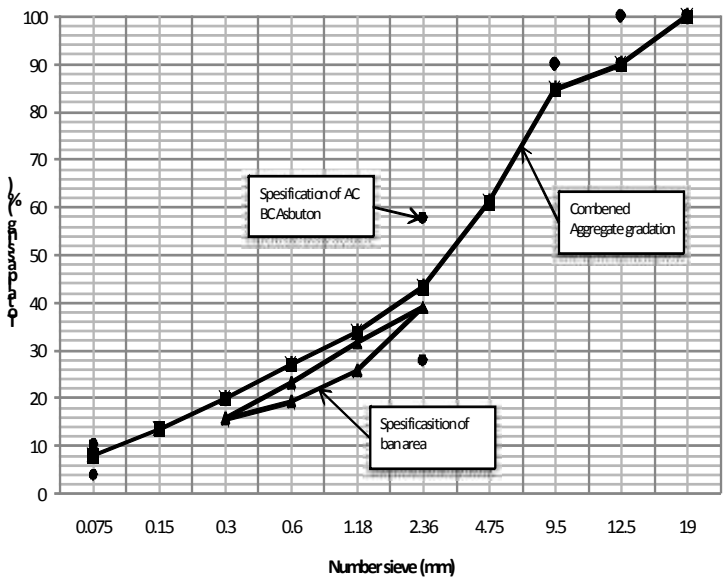

Figure 4. Combined aggregate gradation

Scanning Electron Microscope

SEM imaging was performed in the photo 2d and 3d to see the microstructure as well as the content of chemical elements and chemical compounds in petroleum asphalt and BGA.

Unconfined Compressive Strength Test

This method is used to get the value of the stress, the deformation horizontal and deformation vertical to the sample experienced a failure. Samples were tested by using a UTM appliance, the reading of the data using LVDT deformation that is read by the data logger.

Figure 5. Set-up UCS test

Samples were made in cylindrical shape with a diameter of 101.6 mm and height 100 mm. Used a sample of 3 pieces of samples for each composition of AC BC mixture.

RESULTS AND DISCUSSION

Scanning Electron Micscroscope on AC-BC Mixture

Figure-6. Shows image 2D from Scanning Electron Micscroscope (SEM) on buton granular asphalt (BGA).

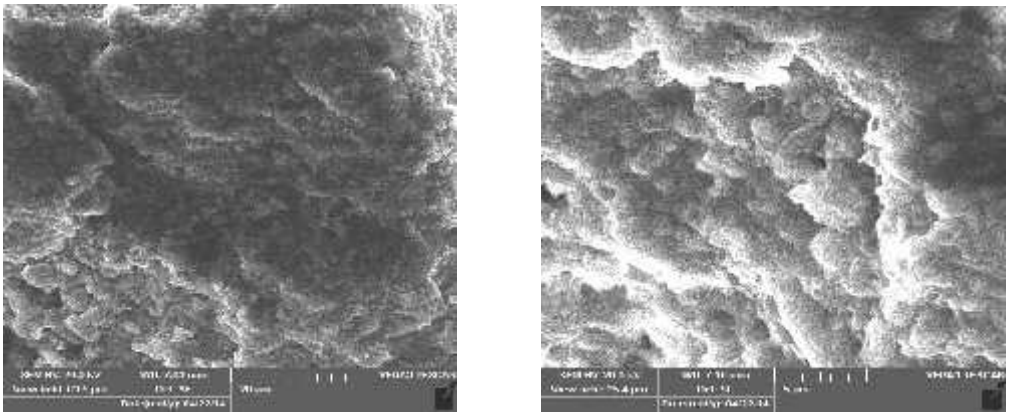

Figure 7. SEM buton granular asphalt (BGA)

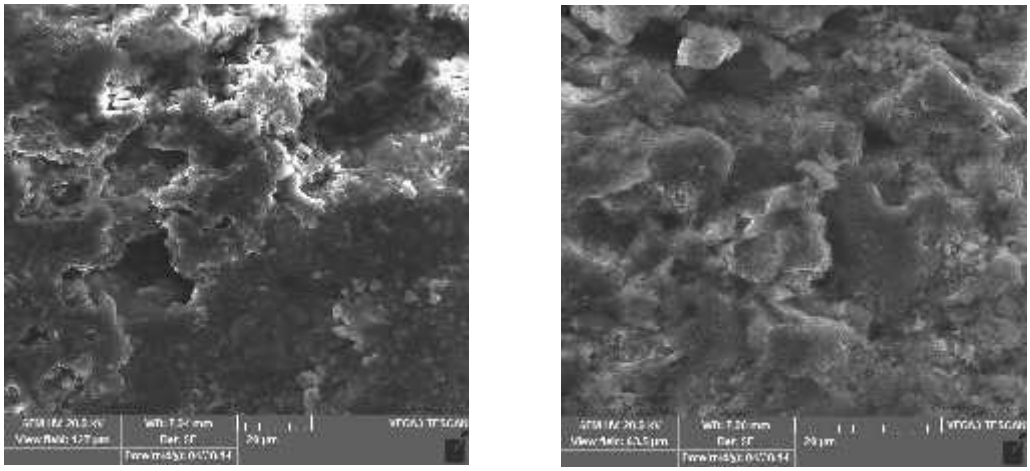

Figure 8. SEM Asphalt concrete mixture with 5% BGA

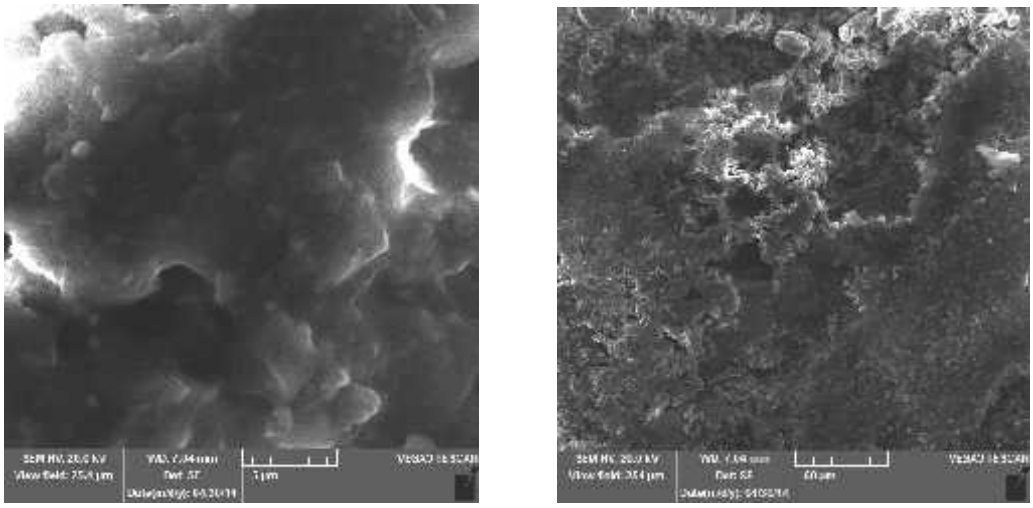

Figure 9. SEM AC BC mixture with 5% BGA

Figure 1 and Figure 2 shows SEM images of 2D. SEM tests showed that the chemical compounds contained in the mixture of asphalt concrete with BGA is SiO2, Al2O3, Na2O, SO3, MgO, respectively 0.29%, 0.57%, 0.72%, 95.36% and 1.06%.

Compressive Strength

In compressive strength testing was concluded as shown in figure 10.

Figure 10. Crack pattern on the sample

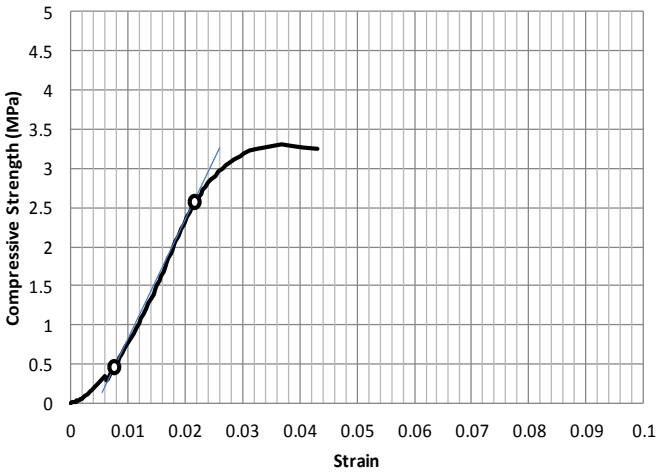

Figure 11. Stress and strain asphalt concrete bearing coarse mixture without BGA

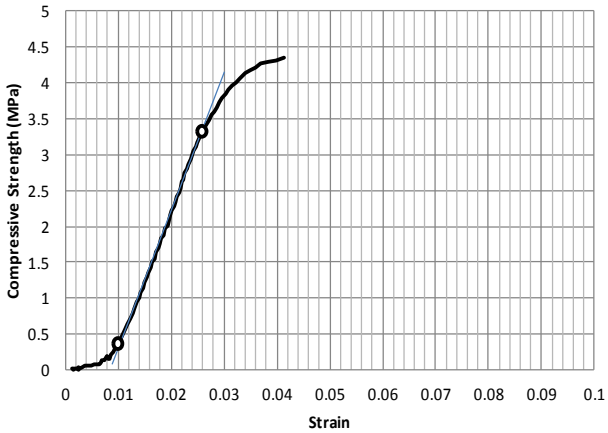

Figure 12**.** Stress and strain asphalt concrete bearing coarse mixture with 5% BGA

Curves in Figure 10, Figure 11 and Figure 12 can be divided into 3 parts, namely the non-linear curve first, the second and the third linear form of non-linear curve.

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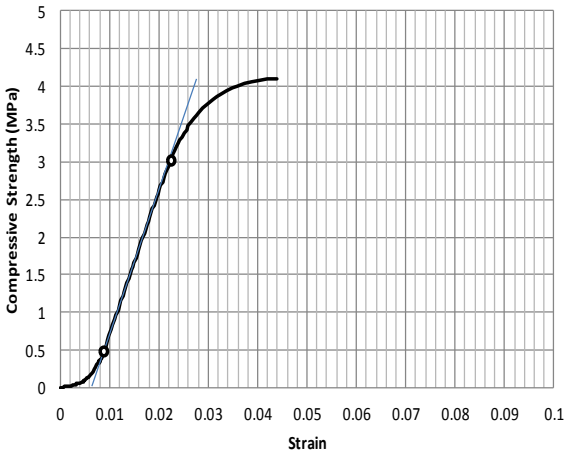

Figure 12. Stress and strain asphalt concrete bearing coarse mixture with 8% BGA

Figure 10 shows that the strain of 0.008 by 0.022 mm/mm to linear-shaped curve with a maximum stress of 0.5 MPa. Figure 11 shows that to linier curve. Figure 11 shows the relationship with the strain on the compressive strength to AC BC mixture with 5% BGA. Linear curve given the strain of 0.01 to 0.026 with a stress higher than the asphalt mixture without BGA. Figure 12 shows the compressive strength-strain relationship in the asphalt mixture AC BC with 8% BGA. Strain curve is linear in the 0.01 to 0.24 is almost the same as the asphalt mixture AC BC with 5% BGA.

CONCLUSIONS

1. Chemical compounds buton granular asphalt (BGA) consists of SiO2, Al2O3, MgO, K2O, CaO, TiO2, P2O5, SO3, FeO, Chlorine and Flurine. Total content of compounds CaO, SiO2 and FeO very dominant about 80% of the total weight.
2. The content of chemical compounds in asphalt concrete mixtures with 5% BGA is SiO2, Al2O3, Na2O, SO3, MgO. SO3 is a compound with a weight of more than 95% in the asphalt mixture.
3. Asphalt concrete bearing coarse with BGA can bear a greater burden than the asphalt mixture without BGA indicated by stress values greater.

ACKNOWLEDGEMENTS

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EFFECT OF SEA WATER TO THE FLEXURAL BEHAVIOUR OF RC BEAM STRENGTHENED GFRP-S

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ABSTRACT: Structures built in aggressive environments such as in the sea/marine environment need to be carefully designed, due to possibility of chloride ion penetration into the concrete. One way to reduce the strength degradation in such environment is to use FRP, which is attached to the surface of reinforced concrete using epoxy. A key issue in the design of FRP strengthening RC structures using externally bonded FRP composites is the bong behavior between FRP and concrete. A series of specimen of reinforced concrete beams with dimension 150×200×3300 mm were casted. Beams were immersed in seawater for 6 months (BF_6) and 12 months (BF_{12}). Three specimens were prepared control beam without immersion to the sea (BF_0). The result indicated that the flexural capacity of BF_6 and BF_{12} compared to BF_0 decreased for 3.78% and 4.16% respectively. The decreasing was caused by the weakening of the bonding capacity GFRP due to the influence of sea water immersion. Based on the experimental result, on model to estimate the effect after sea water immersion to flexural capacity of reinforced concrete beams strengthening using GFRP-S was proposed $MFt = MF0.e^{-0.0036t}$

Keywords: *GFRP, Seawater, Flexural Capacity*.

INTRODUCTION

Recently present the construction of the concrete structures around the beach line or even under water is increasing such as buildings, bridges, highway road, etc. Concrete structures that are not protected or close to the sea may be affected by corrosion, than if maintenance or preventive repairs is not done on the structure, it may cause the collapse . Fiber Reinforced Plastics (FRP) has been accepted as an alternative material for the conventional steel reinforcement.

Common FRP types are aramyd fiber reinforced plastic (AFRP), glass fiber reinforced plastics (GFRP), carbon fiber reinforced plastic (CFRP), respectively. FRP has been applied to many purposes for civil engineering structures not only for new structures but also for strengthening of the deteriorated structures. There has been an important increase in the use of FRP as strengthening structures with externally bonded, because of their inherent advantages in terms of light weight, high specific strength and stiffness ratios and their non corrosive properties. FRP has been developed in the various forms, such as grid, rod, sheet and plate. Glass fiber sheet as showed in Fig. 1 is most commonly used due to its relatively lower cost compared to the other FRP materials.

Fig. 1 Glass fiber sheet

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Studies using retrofitting of beams have been conducted by several researchers. Banthia (2009) reported that using GFRP composite materials in the area interested in the beams and plates. The increase of the moment capacity. Z. G. Guo *et al*., (2005) reported that using FRP composites were successfully used for strengthening of existing reinforced concrete structures because of their superior properties. Alam F (2010) conducted research using GFRP as reinforcement flexural in reinforced concrete beams, The result indicated is an increasing in load up to 75.13 %. However further study needed to clarify the behavior of beams with GFRP sheet reinforcement influenced by the seawater immersion.

SPECIMEN AND TEST SETUP

Specimen

The test program consisted of seven beams. The test span of all beams were 3300 mm. The cross section was 150 x 200 mm. All beams were reinforced with 2D14 mm steel bars in tension side (bottom) and 2φ10 mm in compression side (top). All beams were provided with φ10 mm steel stirrups @ 200 mm center to center. All beams were tested simply supported and were subjected to two points loads symmetrically placed at equal distance (150 mm) from the centerline of the beam. Further details of the test set-up and cross-section of the beams are shown in Figs. 2.

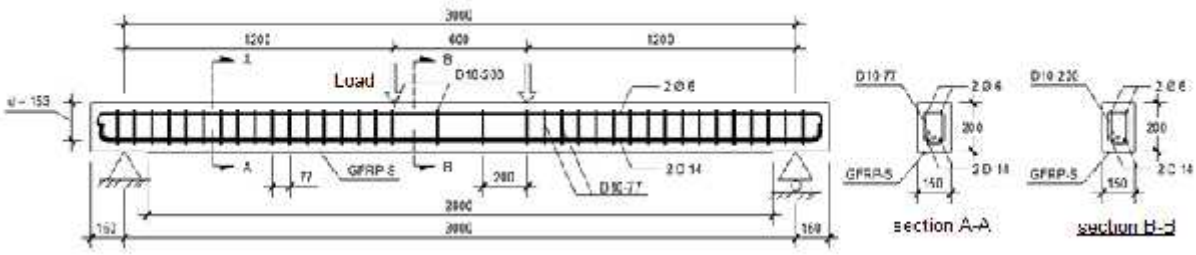

Fig. 2 Detail of specimen

The GFRP sheets were bonded to the tension face of the specimens after 28 days of casting. Before applying the epoxy, the concrete surface was roughened and cleaned to insure a good bond between the epoxy and the concrete surface. The epoxy was hand-mixed and hand-applied at an approximate thickness of about 1 mm. All specimens were immersed in a tank containing seawater for 6 months (BF_6) and 12 months (BF_{12}). Three specimens were prepared control beam without immersion to sea water (BF_0). Figure 3 shows a tank containing seawater the immersion of the specimens. Table 1 shows the material properties of the manufacturer data GFRP sheet, and Table 2 shows the manufacturer data of epoxy resin, respectively.

Table 1 Material Properties of GFRP-S

. Items	Properties
Tensile strength (MPa)	22.20
Modulus young (GPa)	22.14
Laminate thickness (mm)	3.3

Table 2 Material Properties Epoxy Resin

. Items	Properties
Tensile strength (MPa)	72.40
Modulus young (GPa)	3.18
Bending strength (MPa)	2.12

Figure 3. Tank containing

Test Setup

The beam specimens were tested under two point load in loading frame capacity of 1000 kN, as shown figure 4. Each specimens was instrumented by strain gauges on concrete at extreme compression, on steel bar and on the GFRP sheet. The deflection and loading were measured using LVDT and a load cell. The load was applied gradually with constant 0.05 mm per second up to failure. All instrumentation was connected a computer based data logger for data recording.

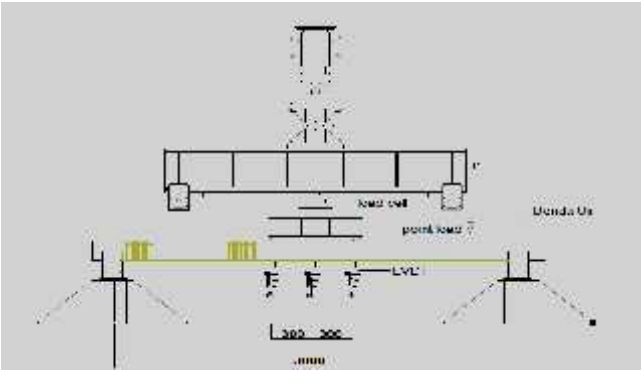

Figure 4. Set up beam specimen

RESULT AND DISCUSSION

Flexural Capacity

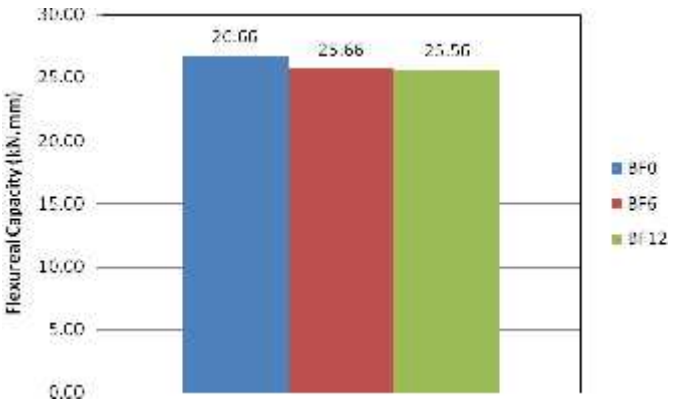

Figure 5. Flexure capacity of the specimens beam BF_0, BF_6 and BF_{12}

Fig. 5 shows the flexural capacity of the beam specimen BF_6, and BF_{12},. It can be observed that the beam specimens after seawater immersion flexure capacity compared to BF_0, decreased for 3.78% and 4.16% respectively.

Load Deflection Relationships

Fig. 6 shows a relationship between the applied load and the deflection at span center for the loading step up to the concrete crushing. At initial stage of loading, the beams were un-cracked beam. The concrete resisted both compression and tension force. When the applied load reached to the rupture strength of the concrete, the concrete started to crack at approximately 8 kN of applied load. This caused a decreasing of beam flexural stiffness. Once the tension zone of concrete cracked, its tensile force resistance becomes negligible. The tensile force due to external load was primarily carried by GFRP sheet reinforcement. Interaction between GFRP reinforcement with the concrete is only due to interfacial bonding. Further loading caused the cracks propagated toward to the top of beams. This caused a crushing of concrete. On the specimen BF_0, the concrete crushed when the applied load achieved to 40.59 kN. On the specimen type BF_6, concrete crushing occurred at approximately 40.05 kN of applied load, while concrete crushing of specimen type BF_{12} occurred when the applied load achieved to approximately 40.00 kN, respectively. Faster crack propagation on the specimen was caused by the lower elastic modulus of the GFRP composite and the lower shear modulus of interfacial epoxy resin. On the flexural beams, the rupture bonding stress of the GFRP may be influenced also the flexural cracking. The failure load for beams BF_6 and BF_{12} were 41.42 kN and 41.26 kN with 3.90% and 4.28% decrease from BF_0, respectively

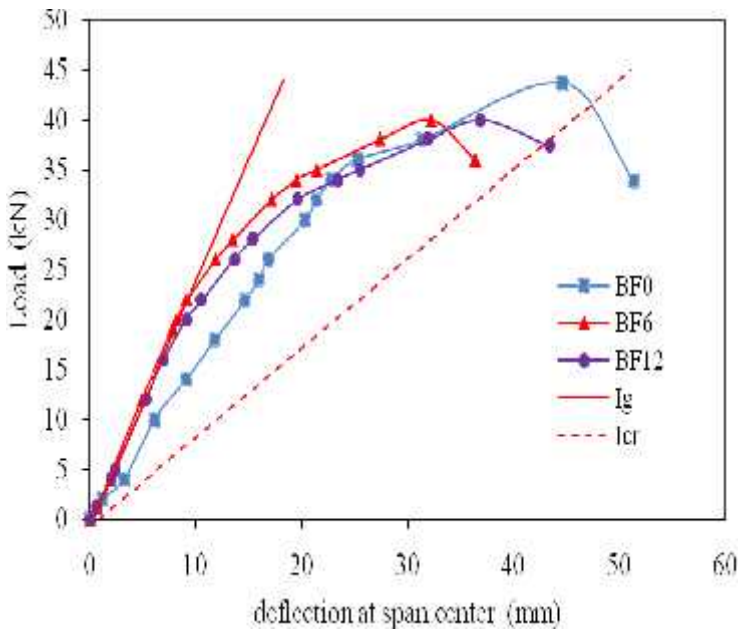

Figure. 6. Load-deflection relationship up to maximum load

Failure Mode

Based on the results of flexural was testing of specimen beams as shown in Fig. 7 pattern of cracks occurred at the 1/3 of the span, so it can be said to be cracked due to flexural moment. The cracks propagated upward to the compression region as the results of the deformation of both tensile reinforcement and concrete. On the strengthened pre-cracked specimens, the loading brought forth some new cracks as well as continuing the pre cracks. Once the cracks occurred, the cracks continued to propagate upward to the compression region as well as appearance of new cracks. This caused the bond stress increased on the other part. Finally, suddenly debonding occurred, as the results, the beam capacity decreased.

Figure 7. Failure model of Specimens

Effect Immersion Sea water to Flexural Capacity

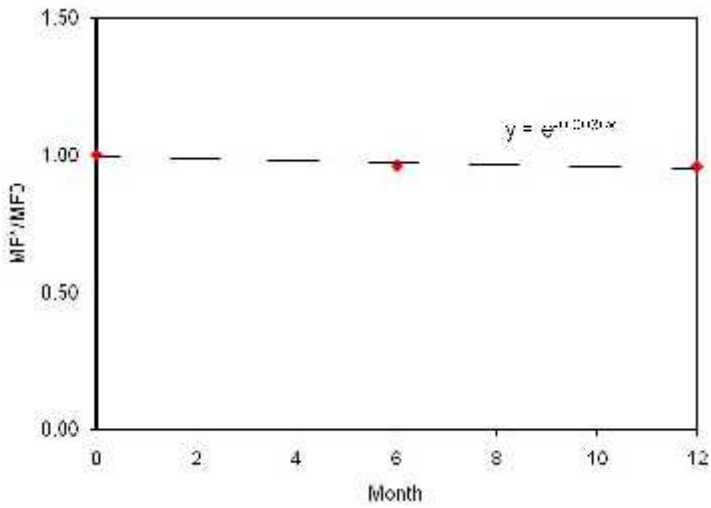

Figure 8. Effect immersion of seawater than Flexural capacity ratio

Fig. 8 show that after 6 months and 12 month reduces the flexural capacity ratio 3,90%. And 4.28%, from BF_0, respectively. This indicates that the reduction in beam flexural capacity after sweater immersion. Based on the Fig. 8, on model to estimate the effect after sea water immersion to flexural capacity of reinforced concrete beams strengthening using GFRP-S was proposed $MFt = MF0.e^{-0.0036t}$

CONCLUSIONS

Based on the experimental study, the following may be concluded:

1. After seawater immersion The decrease flexure capacity of the beam specimen BF_6, and BF_{12} 3.78% and 4.16% respectively, compared to BF_0,
2. Failure is a general loss of bond between GFRP and concrete beam which called debonding failure.
3. The model to estimate the effect after sea water immersion to flexural capacity of reinforced concrete beams strengthening using GFRP-S was proposed $MFt = MF0.e^{-0.0036t}$

ACKNOWLEDGEMENTS

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FLEXURAL CAPACITY OF REINFORCED CONCRETE BEAMS STRENGTHENED USING GFRP SHEET AFTER SEA WATER IMMERSION DUE TO FATIGUE LOADS

Arbain Tata [1], Herman Parung [2], Wihardi Tjaronge [3] and Rudy Djamaluddin [4]

ABSTRACT: Girder of a bridge is experiencing a fatigue loading due to vehicles therefore the study on fatigue durability of a girder is important. This paper presents the result of an experimental study of the flexural behavior of beam strengthened using glass fiber reinforced polymer (GFRP-S). The Experiment was carried out to determine the effect of fatigue loads to the flexural capacity of reinforced concrete beams. Specimen was 3300 mm long with 150 x 200 mm rectangular cross section. Two specimen where tested under monotonic loading of BNK and BFK (maximum load control) and six specimens were tested in fatigue load. Specimen of BN, BF0, BF1, BF3, BF6 and BF12 was loads under sinusoidal under fatigue load with range of 4 kN to 24 kN. BF1, BF3, BF6 and BF12 respectively immersions under sea water to zero until six month. He result indicated that the beams failed after nine hundred thousand of cycles. GFRP-Sheet reinforcement showed an increase in the capacity of the test beam, the failure mechanism, fatigue of the steel reinforcement, remains the same in both strengthened and no strengthened beams. Based on the experimental result on model to estimate the effect of the sea water to flexural capacity of reinforced concrete beams strengthening using GFRP-S was proposed $MFf_t = MFf_0.e^{-0.0061t}$

Keywords: *fatigue loads, flexural capacity, GFRP-S, reinforced concrete beams*.

INTRODUCTION

Fiber reinforced polymer (FRP) became important in improving structure strength. Carbon fiber reinforced polymers (CFRP) that used for bridge reinforcement able to increase durability and capacity (Amr and Sami 1999), (Elkenel and John 2009). These things cannot be avoided especially for developing countries. Followed by the technology improvement in this mature construction material, the used of Fiber Reinforced Plastic on repairing and strengthening building structure were well used in variety of functions, including repairing and enhancing the capacity service moment on a bridge structure. FRP material is a type of material with major advantages such as high tensile strength, does not rust, and light in term of weight, so relatively it does not increase the weight of its own structure (Richard et all 1999).

(a) Glass fiber sheet (b) Application Glass fiber sheet

Fig. 1 Material and application GFRP-S

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Fig. 1a showed the glass fiber sheet that been used, FRP application in structural repair does not require a big energy like in conventional material, so it will be environmental friendly. There have been many studies that have been done, implemented and applied widely for improvement or enhancement on bridge moment service function (Sobhy et all 2002). Fig. 1b showed the reinforcement of bridge girder by using FRP material with carbon fiber as the base material. Several studies have proven the reinforcement with FRP material attachment. However, the capacity of beam was tested with FRP reinforcement material by attach on the pull side conducted by using monotonic static load. In fact, the bridge structure included bridge girder, principally bear the repeated load that resulted by the passing vehicle. In several studies showed that repeated load in a structure can cause a collapse or failure even the level of load far below the ultimate flexural capacity (Vladimir et all 1999), because of that, the durability of reinforced concrete beam by FRP reinforcement material due to fatigue load need to be applied (Papakonstantinou et all 2001) . The guideline procedure in designing the beams with FRP reinforcement that have been used in this study refers to ACI standard which is "ACI 440.2R-08 Guide for the Design and Construction of Externally Bonded FRP system for Strengthening Concrete structure. *Ms* from Eq. (1) and (2) equal to the moment due to all sustained loads plus the maximum moment induced in a fatigue loading Tension condition on concrete that experienced bending as shown below Fig. 2 and Fig. 3 **(**ACI Committee 2008).

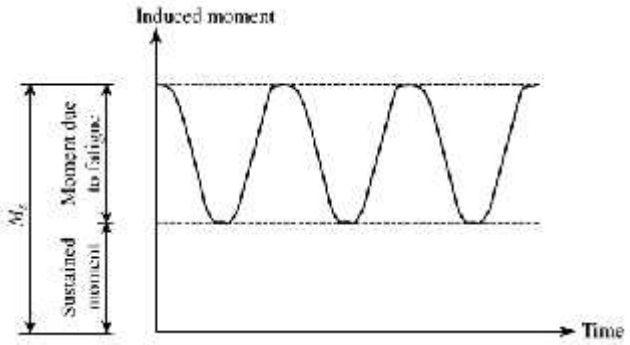
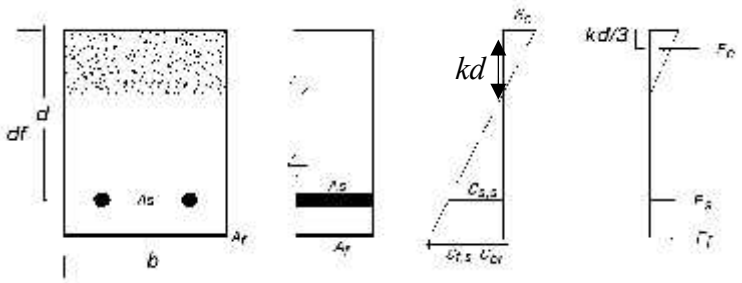


Fig.2 Illustration of the level of applied moment to be used to check the stress limits in the FRP reinforcement. (ACI Committee. 2008)



(a) Reinforced concrete section (b) Strain distribution (c) Stress distribution

Fig.3. Internal strain and stress distribution for a rectangular section under flexural beams. (ACI Committee. 2008)

$$f_{s,s} = \frac{\left[M_s + \varepsilon_{b1} A_f E_f \left(d_f - \frac{kd}{3}\right)\right](d-kd)E_s}{A_s E_s \left(d - \frac{kd}{3}\right)(d-kd) + A_f E_f \left(d_f - \frac{kd}{3}\right)(d_f - kd)} \quad (1)$$

$$f_{f,s} = f_{s,s} \frac{E_f}{E_s} \frac{(d-kd)E_s}{\left(d - \frac{kd}{3}\right)} - cE_f \quad (2)$$

$f_{s,s}$ = Stress level in non prestressed steel reinforcement at service loads, (MPa)

$f_{f,s}$ = Stress level in FRP caused by a moment within elastic range of member, (MPa)

M_s = Service moment at section (N.mm)

E_f = Tensile modulus of elasticity of GFRP-S (MPa)

E_s = Modulus of elasticity of steel (MPa)

SPECIMENS AND TEST SETUP

Specimens

The concrete that used is normal concrete with compressive strength plan of 25 MPA. Compressive strength and flexural strength were checked through concrete cylinder and concrete test. Casting was conducted according to casting standard, and mold was opened after 3 days and furthermore the treatment of concrete was performed with wrapped by wet sacks for 28 days. Specimen of reinforce concrete beam planned in accordance with the principle that set out in the planning standard. Fig. 4 showed the detail of beam specimen that prepared for testing. Concrete beam cross section dimension is 100 mm x 120 mm with a length of 3300 mm. Beam was planned with 2D13 concrete as the tensile rebar. To prevent sliding failure, 6 mm diameter of rebar was used as stirrups. This study used FRP material with material based on fiber glass sheet shaped which called Glass Fiber Reinforced Plastics-Sheet. The unification of glass fiber material and epoxy resin would create a reinforcing GFRP-S material on the beam. Technical parameter of GFRP-S showed on table 1 and parameter of properties epoxy resin showed on table 2. Application of glass fiber sheet as the beam reinforcement was conducted after concrete beam is 28 days-old.

Application process for beam reinforcement showed on Figure 4a with process in accordance to standard procedures. First of all, the surface of beam on pull side cleaned by polished method. Then, glass fiber sheet cut according to the size that would be used and covered with epoxy resin material. After that, the glass fiber sheet would be paste on the concrete surface after the concrete also covered with epoxy material. After the sheet material has been positioned properly, next the sheet material sealed and smeared again with epoxy by using brush roller, so the entire surface of sheet became saturated with epoxy material. Then, the specimen allowed to quiescent in order that epoxy resin hardens and fused with glass fiber material to form GFRP-S which attached at pull side (bottom side) of reinforce concrete beams.

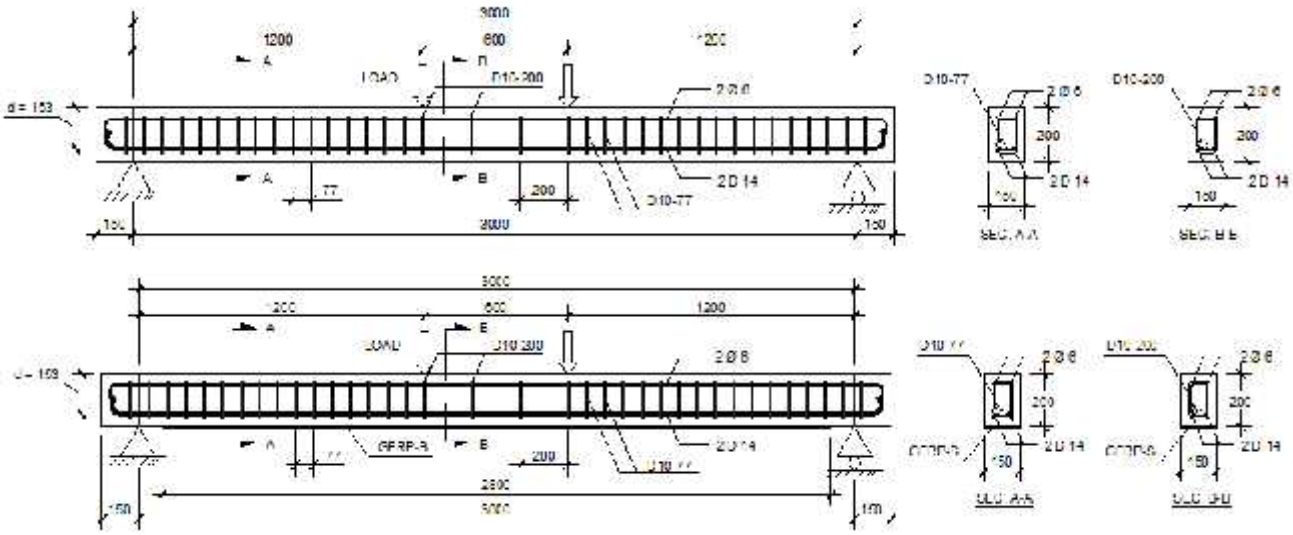


Fig. 4 (a) Beam normal (BN) ; (b) Beam of GFRP-S (BF)

Tabel 1. Properties GFRP-S type SHE-51A

Items	Properties
Modulus young (GPa)	3,24
Tensile strength (GPa)	72,40
Strain Maximum (%)	4,5
Density (gr/cm^3)	2,55
Thickness fiber (mm)	0.36

Tabel 2. Properties epoxy resin

Items	Properties
Tensile strength (MPa)	72,40
Modulus young (GPa)	72,40
Bonding strength (MPa)	2,12

(a) Installation GFRP-S (b) Photograph test beams

Fig. 5 Installation and photograph test beams due to fatigue loads

Test Setup

Loading fatigue test was conducted by using 100 tons fatigue capacity equipment test. Fig. 5b, before the tested strain gauges affixed on the concrete surface and on the GFRP-S surface to measure the strain that occurs on the specimen during the loading process. Specimen placed on two simple pedestals which behaves joint rollers placement system. Loads were given to two points with distance of 50 cm loading point at the middle of beam span. The load that given was measured by using Load-Cell, while deflection measured by using LVDT that placed on the middle point of beam span. All measurement instruments were connected to tabulation data system that connects to a computer in order to monitor all parameters measured during the loading process. Beams planned for fatigue loaded with minimum load 4kN (around 5% from ultimate flexural capacity) and maximum load 24kN (around 45% from ultimate compressive concrete strain). The minimum and maximum load patterns were given repeatedly with frequency 1.5 Hz continuously until the occurrence of failure on specimen. Data reading conducted after the beam experienced repeated load with period of 10 cycles, 100 cycles, 10000 cycles, and multiples of 50000 cycles until the beams failure happen.

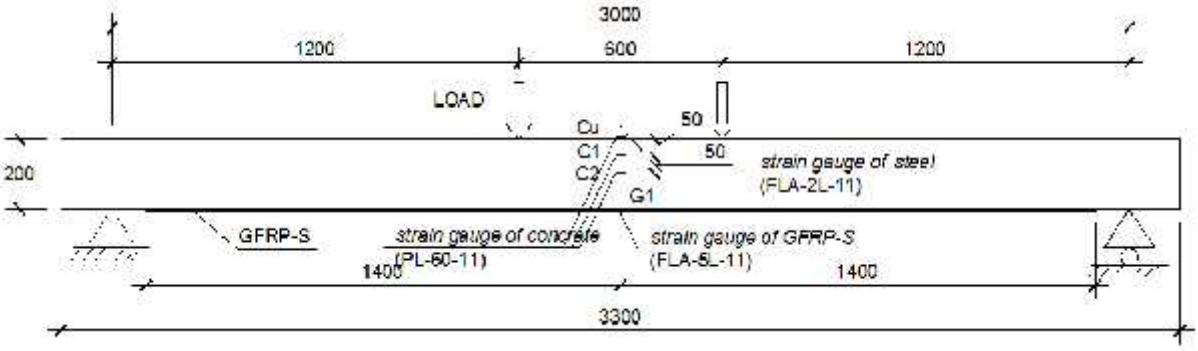


Fig. 6 Position record data.

Further fatigue test carried out on the concrete beam specimen that been soaked during age of 1, 3, 6 and 12 months. During that fatigue, deflection measurement also conducted by installation of LVDT on the bottom side of beam and the examination of pattern cracks that occur conducted by using phi gauge. The imposition performed on the beam with two load points at BN and BF, using fatigue load. Fatigue load was able to cause fatigue and failure, with constant frequency of 1.5 Hz sine load models. In sinus loading Fig. 7, fatigue load applied with load of 1 cycle, 10 cycles, 100 cycles, 1000 cycles, 10.000ycles, then multiples of 50.000 cycles until wrecked or until 1.000.000 cycles. Load was given with minimum and maximum load for each beam without reinforcement (BN) with beam which strengthened by GFRP-S (BF) 4 kN - 19 kN and 4 kN - 24 kN. Minimum load obtained was from estimation of dead load on the concrete beams. Maximum load 19 kN or 24kN was obtained from estimation of 45% f’c control beam from each reinforcement GFRP-S and without reinforcement as shown in Fig 6.

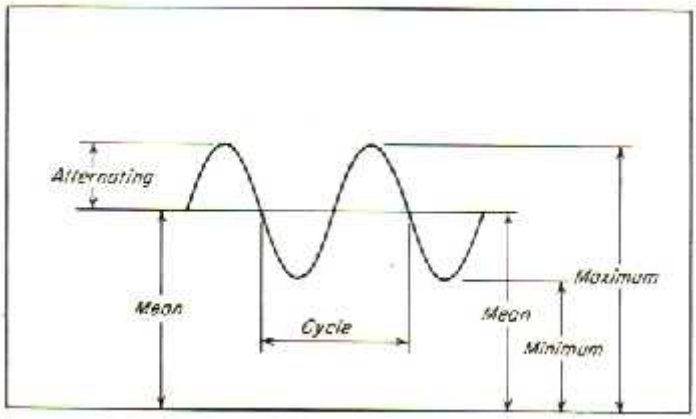


Fig. 7 Model sin wave on fatigue loads

Data reading in data logger was taken every increment of 1 kN load under normal conditions while the observation of beam test continuously observed visually, especially to the development of cracks that occur because of the burden that increased and also to collapse behaviors that occur.

RESULTS AND DISCUSSION

The influence of fatigue load on the beam deflection

Normal beam (BN) wrecked at 635.100 cycles. At load levels that still relatively small (P=4 kN), the deflection change that occurs still relatively small after fatigue load. But, at the higher load level the deflection change quite significant. At 11.5 kN, the deflection was quiet big. At maximum load (19) kN the effect of fatigue load increased 11.03% after loaded with fatigue up to 635.100 cycles.

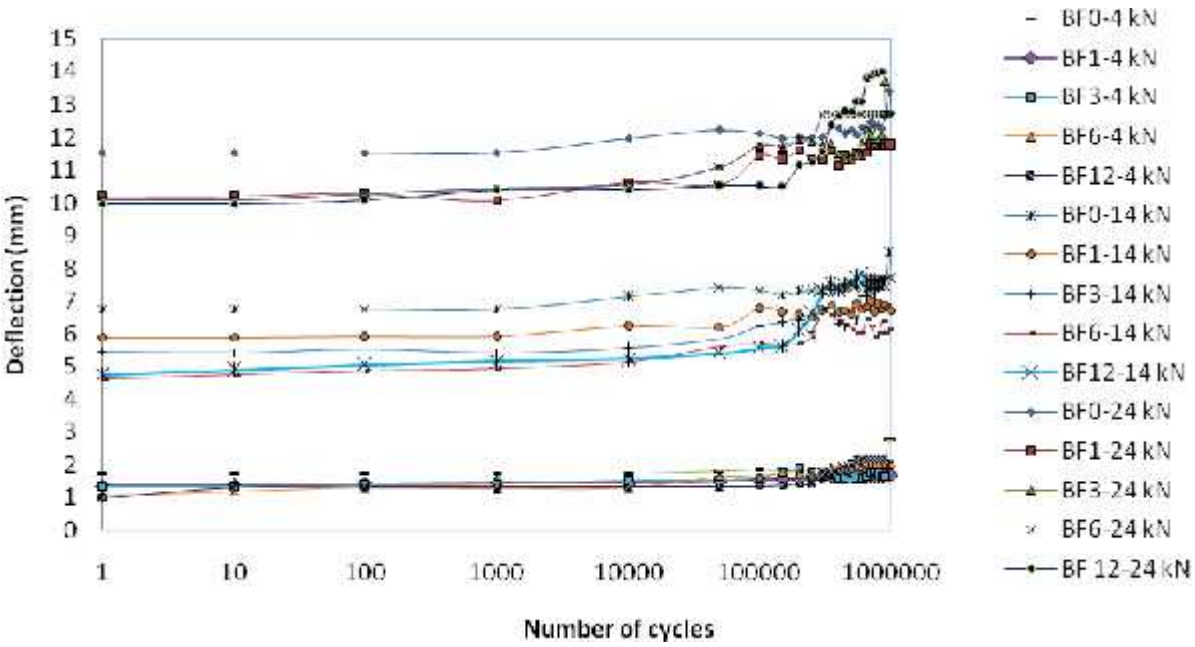


Fig. 8 Deflection versus number of cycles beams BF0, BF1, BF3, BF6 and BF12

What happened in normal beam also happened as well in reinforce concrete beam that strengthened by GFRP-S. Fig. 8 showed the deflection on normal beam for various loading level after through fatigue load cycle until 1.000.000 cycles. On the beam that strengthened by GFRP-S averagely were destroyed at 1.000.000 cycles. On the load level that relatively still small (P=4 kN) the deflection that occurs seems not affected by fatigue load. On the higher loading level it seems that the effect of repeated load has begun to appear. At 14 kN load, there is an increase in deflection after beam through 1.000.000 fatigue load cycles. At 24 kN load, the effect of fatigue load becoming more apparent where the deflection experienced an increase after loaded with fatigue load 1.000.000 cycles. The effect of fatigue load after

1.000.000 cycles for various loading level which compared with the age of immersion tend to decrease. This can be indicated as a result of weakened in the GFRP-S adhesiveness against concrete at the pull side. Generally the weakened structure also caused by fatigue load which caused by the emergence of micro cracks due to repeated load.

The influence of fatigue load against concrete strain

On Fig 9 showed the strain that happened at the side of compressive concrete (fiber outer compressive) on beam that strengthened by GFRP-S for various loading level after through fatigue load cycle until 1.000.000 fatigue loads. At the load level that relatively still small (P=4 kN) the strain that occurs relatively still small.

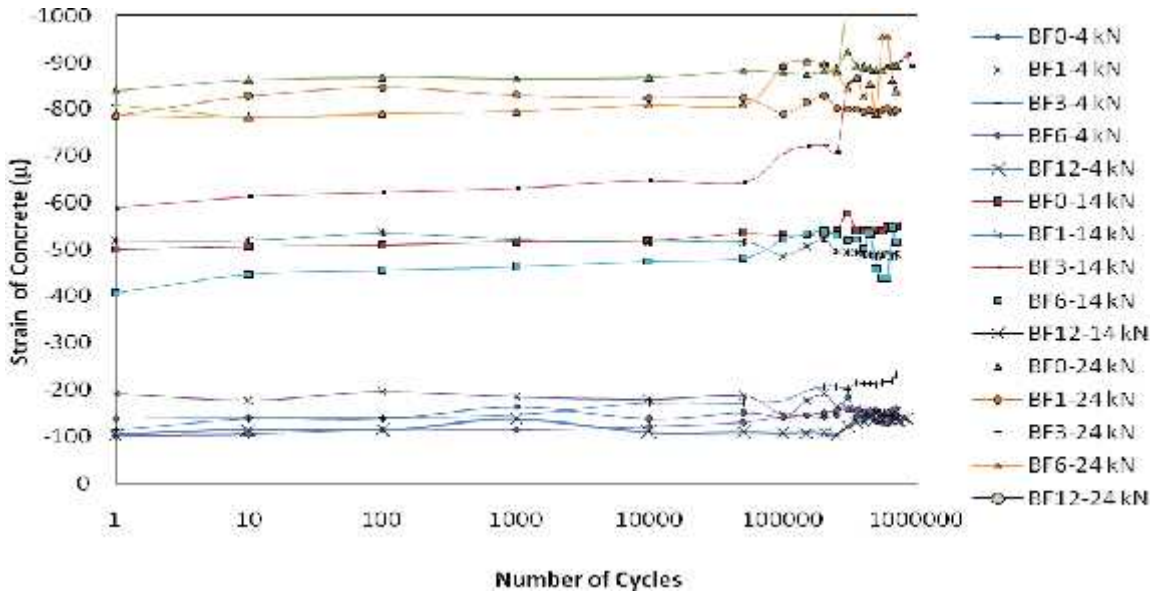


Fig. 9 Strain of concrete versus number of cycles beams BF0, BF1, BF3, BF6 and BF12

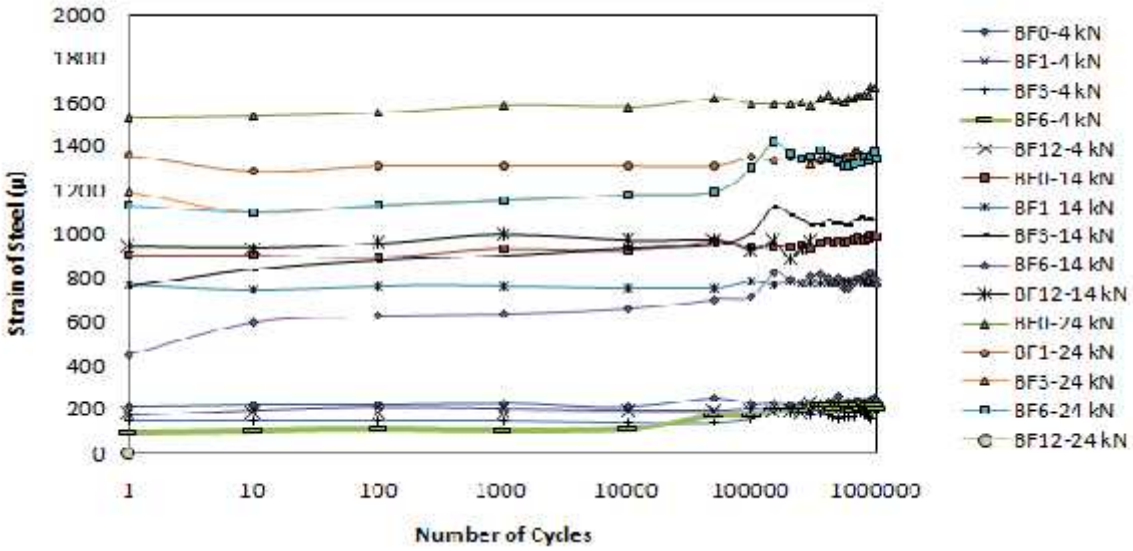


Fig. 10 Strain of steel versus number of cycles beams BF0, BF1, BF3, BF6 and BF12

But, on the higher load level it seems that the effect of repeated load has begun to appear. At 14 kN load, there is an increase in deflection after beam through 1.000.000 fatigue loads. At 24 kN load, the effect of fatigue load becoming more apparent where the deflection experienced an increase after loaded with fatigue load 1.000.000 cycles. At the beam with the GFRP-S reinforcement for various level of duration in immersion after through fatigue load until 1 million cycles repeated load, each load level showed the increase in concrete strain and steel strain Fig.10. These things indicated a decline in the flexural capacity moment due to weakened of GFRP-S adhesiveness against concrete skin due to sea water immersion.

The influence of fatigue load against GFRP-S strain

The influence of fatigue load against GFRP-S strain showed in Fig. 11 the result showed on the load level that relatively still small (P=4kN) strain that occurs still do not have any change after fatigue load. But on the higher load level it seems the effect of repeated load has begun to appear. At 14 kN load, it seem there is an increase in strain after beam experienced fatigue load 1.000.000 cycles. At 24 kN the effect of fatigue load tend to be more appear where the strain of GFRP-S experienced a decrease after given fatigue load 1.000.000 cycles. These things were the composite action from concrete, steel and GFRP-S. It appears that the weaker the GFRP-S action means that there is a weakening on adhesiveness of GFRP-S against concrete skin.

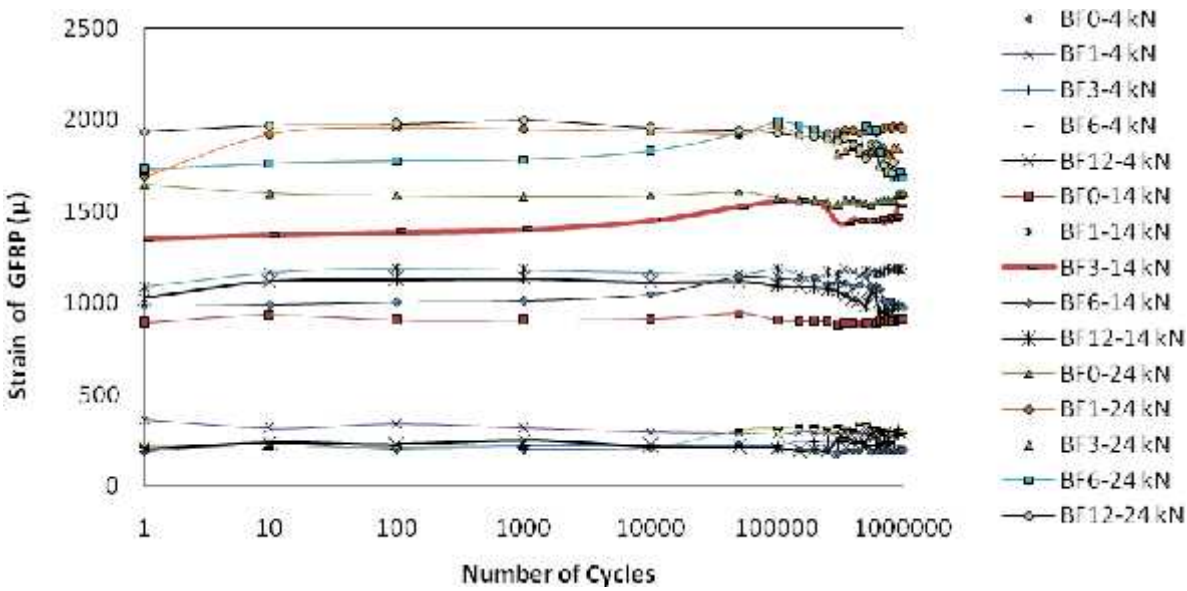


Fig. 11 Strain of GFRP-S versus number of cycles beams BF0, BF1, BF3, BF6 and BF12

Service moment capacity on beam

Based on fatigue test on beam, so the maximum load for GFRP-S beam was 24 kN at 500.000 cycles can be determined how big the service moment like showed on Table 3. The service moment capacity reduction size can be approximated by the equation $MFs_t = MF_0 e^{-0.00681t}$, as showed in fig. 12.

Table 3. Service moment beams of BF0, BF1, BF3, BF6 and BF12

Bulan	ε_c	ε_f	ε_s	M_s
	mm	mm	mm	kN.mm
0	8.92E-04	1.558E-03	1.621E-03	1,893.3
1	7.98E-04	1.958E-03	1.355E-03	1,725.3
3	1.062E-03	1.804E-03	1.650E-03	2,285.8
6	8.04E-04	1.714E-03	1.327E-03	1,723.9
12	8.83E-04	1.790E-03	1.1659E-03	1,774.4

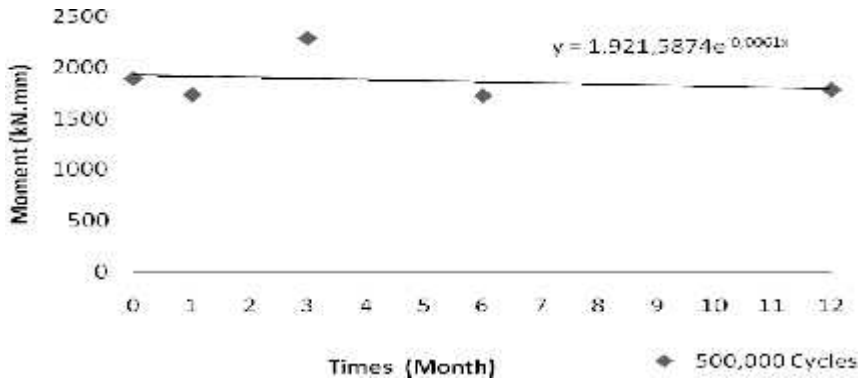


Fig. 12 Graph of relation between service moment relationship with time (month)

CONCLUSION

From this study and discussion can be conclude as follows:

1. There was a tendency of deflection increment and deflection decrement after immersed on sea water and fatigue load on the beam that immersed within one month (BF1), immersed 3 months (BF3) immersed each one year (BF12) each beam that not immersed (BF0). These things indicated by a weakened capacity GFRP-S adhesiveness capacity that affected by sea water immersion and fatigue load.
2. To calculate moment capacity value from reinforce concrete beam that strengthened by GFRP-S due to sea water immersion and fatigue load in the immersion pool can be approximated with equation model of $MFs_t = MF_0 e^{-0.0061t}$ where M_{st} is service moment capacity at t year, MF_0 service moment at the beginning of immersion and t is the time of sea water immersion expressed in year.

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PARAMETER ANALYSIS OF NON-LINIER BEHAVIOR UNCONFINED CONCRETE BY TRIAXIAL LOADING USED SOFTWARE BASED FINITE ELEMENT ANALYSIS

Data Iranata[1], Alfin SeptyaNugroho[2]

ABSTRACT :Finite element method is the world techniques method it used to study the behavior of an element, either in elastic or plastic conditions. FEM is used to get the behavior of the element in the actual condition of an element is assumed as an equation or matrix component. Along with the development of technology in the field of Civil Engineering users have been facilitated with FEM-based aid programs one of which is Abaqus. Abaqus is one of which programs that can be used for modeling the behavior of test specimen, both the behavior of linier and non-linier. The modeling by ABAQUS aims to obtain the behavior and parameters to be converge, modeling was performed using plastic analysis with CDP method (Concrete Damage Plasticity) and DPA (Drucker-Prager Analysis). Simulations are made based on previous experimental studies, in which cube dimensions on 100 mm x100 mm x100 mm using isotropic material data. Then it will be given as a boundary condition form of cube elements, so the compatibility has consist of the elements that will be subjected by triaxial loading of pressure, biaxial loading, and uniaxial loading. Triaxial loading applied tensile stress and compression stress in the form accordance with the stress ratio of previous studies. The results of modeling with simulation will be compared with the results of experimental research laboratories. The output of the ABAQUS will be stress-strain, and shear of the main axes x, y, and z. From the results ofthis study foundthat the methods usedin DPA most effective when use line arequations(exponent order1), the plan chart parameter sp, q, and t toget the value of a =0.394, b=1, pt=3.7, dilatation angel=28°, friction angel= 53°. The results obtained from theunique behaviorby using multiple loading triaxial stress ratio,which is yield due to the tensile load but failure caused by the compressive load as it did inthe experiment DPA in the ratio 0:25-1 -1. Which the comparisons results of the two experiments ABAQUS modeling experiments resulting curveis relatively coincide with the results of experiments in which the accuracy of modeling with experimental results of the CPD is equal to16.10%, while the accuracy of modeling with experimental results of the DPA is equal to7.52%. These condition results also can be seen that there is convergence of iterations in the DPA or CDP presented through charts.

Keywords : *Finite Element Methode, Abaqus, Parameter, Concrete Damage Plasticity, Drucker-Prager Analysis*.

INTRODUCTION

Concrete is the main ingredient in the construction of this century, as well as being a very useful material for various types of techniques in building structures. Concrete has the advantages of various aspects of economy, efficiency, strength, and stiffness make this material become very attractive for application in long-span structures or other building (Kwak and Filippou, 1990). Concrete is generally applied to all buildings either general or specific structures, such as dams, bridges, nuclear buildings, storage buildings radioactive substances, the president's house, and others (Shiming and Yupu, 2012). Practice baton as special building materials used high quality concrete as a substitute or alternative to the traditional normal concrete. High quality concrete is generally formed to have a high strength, high fluidity, as well as the addition of admixture can actually regulate the durability of concrete. Density micro structur high quality concrete also affect the strength of the concrete and reduce the permeability of concrete, the concrete so the ability to accept bending (deformation) is very weak, or in other words, the concrete becomes brittle (Zhen-jun and Yu-pu, 2009). According to (Kwak and Filippou, 1990) in use, the concrete must meet the following conditions:

1 Structure that uses concrete must be strong and secure. In the sense thatits application is very fundamental and must satisfy the principle of equilibrium analysis and mechanical components of the material, so that in case kruntuhan due to load more inevitable on set of an accident.
2 The structure must meet therigor and have no damaged. The need to control deflection occurs when the load service ability and minimize the occurrence of cracks in concrete.

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3 The structure should be more economical. The material used should be tested and the concrete planned as efficiently as possible so that it can be seen differences in the unit price of the use of concrete with steel.

Application of concrete in building structures generally form plate elements, beams, columns, and foundations. Plates are part of the structure for distributing the dead load and live on the beams, but there is also used as a wall plate. Plates are unique elements and can be used in other ways. Beams are elements that are designed to receive the load so that the resulting bending and shear stress σ1 and σ2, so that the expenses incurred can be reviewed biaxial load (Zhang, 2002). Column is the most important in the design of the structure so that the SNI 2012 set of "strong coloumn weak beam" in which the design is based by making the structure more robust columns of structural beams, so if there is a load that exceeds the design load, the building does not immediately collapse but is expected to still can survive and give the time for people who are inside to save themselves. The column is a part that receives the force comes from three directions, in other words, the column is expected to disperse the triaxial stress. Poer and is part of the same column in which the possibility of these elements must be able to receive a load from several directions multiaxial. Some research have been done to investigate the characteristics of the concrete strength against from biaxial and triaxial load. The research states that stress and strain that occurs can be affect the strength of concrete against from multiaxial compressive load. (Shiming and Yupu, 2012) mentions that the strain and stress of biaxial load more dangerous when compared with uniaxial load. So when taken hypothesis of the statement, the triaxial load more dangerous than the uniaxial or biaxial load. So in this opportunity will be made of concrete engineered to model the triaxial load.

Along with the development of technology in various fields, in the field of computer technology and engineering (simulation) also continues to evolve and become more advanced. Computer technology in manipulating modeling has been widely used in the fields of engineering example of civil engineering, machinery, microelectronics, outside angakasa, flights, bioengineering. Simulation or modeling approach is useful for calculating many cases the strength of materials, thermal analysis, structural strength even fluid mechanics (Sylvya A, et al 2013). An understanding of the behavior of an element of structure in general obtained from experimental tests in the laboratory. This experimental test is very important to get results concerning the structure of the corresponding response based on real circumstances. However, these experimental results have constraints in terms of time and costs due to produce accurate experimental experimental data is needed very much so require many specimens (Setiawan, 2008).A success of each approach to the analysis of simulation depends on the balance between accuracy and practices, with obstacles in the form of modeling parameters required, and the lack of information from previous research into its own difficulties in combining the data with predicted data yet into modeling. For modeling or simulation approach followed by numbers that will be required calibration input into modeling which is based on minimum numbers that may be used as a parameter of a material (Vassiles and Andreas, 2007). In doing approach or numerical simulations carried out based there are various methods that can be used as finite diference method, finite volume, boundary element and finite element method. Among these methods, finite element method be one that is accepted and widely use in a variety of engineering applications, including the analysis of elastic structures (Endah, et al 2013).

Currently, allfaced withthe ease ofoftechnologywithFEM-based software. Programsor softwarethat resolves theissuestructurewithinputfrom simple tocomplex. But convenience should be balanced with the basic principle sin the analysis of the structure. Despite the ease of analyzing by using FEM-based program, but the application is required to enter data soul in material engineering and property in order to obtain maximum results. A user or user FEM-based programs need tounderst and the usefulness of the program and how too perate it. The out put or result of the program being used will be the behavior of the structure or structural elements, either isotropic or orthotropic. These behaviors requires a special strategies either in a line are lastic and non-linear matrix (Katili, 2008). Considering and reviewing the above background, then it will be analyzed by using the auxiliary program based software FEM (ABAQUS). Toget the parameter sand the behavior of concrete with triaxial load, which will be compared to the results of research in modeling software with existing experimental modeling. These programs were chosen because it has its own advantages compared with similar programs in which not only can analyzethe structure of the frame but also the material elements oft he structure.In addition the program can enter some data the material properties of concrete with a fairly complete. The analysis will be carried out will refer to the CDP method (Concrete Damage Plastisity) and DPA(Drucker-Prager Analysis) It is expected that the result s will be close to the experimental results of the.

MATERIAL AND MODELING PROCEDURES

This research will be focused on the parameters used to define non-linear behavior of concrete. This study aimed to compare the stress the laboratory experimental results with modeling using Abaqus program. That requires data from experimental material for dikomparasikan and input into the program. Input is needed in the form of parameters which will bediscussed in the next chapter. While the results of behavior that will be reviewed include the stress that occurs on the surface of the concrete in plastic state. Modeling on the program will be adapted to the experimental conditions at

the time. At this modeling will be used the same withe xperimental dimension is, the cube-shaped test specimens with dimensions 100mm x 100mm x 100mm. Boundary condition which will be usedis a roll, hinged joint, fixed on three sides corresponding x, y, and z. As for the loading is done differs from experimental, using load displacement (displacement) which will value the load will continue to rise until it reaches aplastic condition and failure is in dicated by the stress strain curve on the test object. loading pattern will be adapted to the ratio of the stress in **Table1**, to value ratio with a negative sign is assumed as the pressure (opposite the main axis) while the positive sign is assumed as tension (the direction of the major axis). Modeling results form of non-linear behavior of the concrete stress to be compared with experimental studies (Songand Zhen, 2009).As for the details of this methodology will be discussed in the next chapter.

AnalysisProgram Based of FiniteElement (Abaqus)

Analysisis performed using the modeling/ simulation are made equal with the implementation of the experimental test by (Zhenand Song,2009). In the simulations used finite element programs (Abaqus). Simulation is used form of three-dimensional modeling, using the same dimensions according to research eksperimental Zhen Song (2009), and configured according to the reality at the time of the study.Results from these simulations will be controlled form of an analysis of elements in conditions of plastic and elastic. Simulation will generate stress and strain in the concrete simulation, which will be compared with research (Zhenand Song, 2009) form often sile and compressive concrete analysis using a comparison stress(stress ratio).

Material Data

The data used in the modeling or simulation is based on previous research which done by Zhen-junand Song, 2009. The type of concrete used is High Strength Concrete with the water-cement ratio(0.31), while the poisson for steel taken as 0.17 based on SNI 03-1729-2002 Steel Structures Planning Procedures for Building and for concrete ranged between 0.15 and 0.20 (Wahyudi & A.Rahim, 1997, p. 28). In reaserch of Vassiland Andrewin 2005, resulting in that thevalue of Ec is used fo rconditions of HP Cor high quality concreteis to use **Equation 1**.Where f'co is not confined concrete compressive strength and elastic modulus Ecis not confined concrete. While there search (VassilK.Papanikolaouet al, 2005), resulted in that the value of Ecis used for conditions of HP Cor high quality concrete is the formulation

$$E = 0.9x22000x\left(\frac{f'c}{1}\right)^{0.3} \quad (1)$$

Compressive Strength (f'c)	: 60,16 MPa
Tension Strength (fcr)	: 5.08 MPa
Temperatur Ruangan (T)	: 20°C
Poisson Ration (υ)	: 0,17
Modulus Elastis (E)	:0,9.22000.(60,16/10)^0,3= 33920,132 MPa
Density	: $\frac{2 \quad .7 \ x9.8}{1 \ 5}$=24,1 x 10^{-6}ton/mm^3

Table 1. The stress ratio on the experimental result

DimensiSimulasi	Stress Ratio			Stress Value (Mpa)		
mm	σ_1	σ_2	σ_3	σ_1	σ_2	σ_3
	0.00	0.00	-1	0.00	0.00	-60.16
	1	0.00	0.00	5.08	0.00	0.00
	0.05	0.00	-1	2.20	0.00	-43.92
	0.10	0.00	-1	3.23	0.00	-32.26
100x100x100	0.25	0.00	-1	4.28	0.00	-17.11
	0.50	0.00	-1	4.73	0.00	-9.45
	0.75	0.00	-1	4.86	0.00	-6.48
	0.05	-1	-1	2.23	-44.64	-44.64
	0.10	-1	-1	3.02	-30.18	-30.18

0.25	-1	-1	3.76	-15.05	-15.05
0.35	-1	-1	4.11	-11.73	-11.73
0.50	-1	-1	4.19	-8.37	-8.37
0.75	-1	-1	4.22	-5.63	-5.63
1	-1	-1	4.23	-4.23	-4.23

Source :(Zhen-judan Song yu-pu 2009)

Boundary Condition

In this step it will do is determine boundary condition on this modeling. In the modeling using the software will be matched with experimental modeling, that is assumed to be a roller in areas or parts that become boundary condition.

Loading

The loads used in the simulation are the same load used on the experimental modeling that is the burden of pressure on the three sides of the elements reviewed (x, y, and z). Pressure given form of non-linear multi level to reach failure condition. Simulations are expected to come from the state elastic, plastic, and reaching failure.

Table1 shows the ratio of loading of each side of the element, for stress is zero=0 then it is not done loading, where as when the value 1 or number then there is the axial force on the that direction, while for positive and negative values is the effect on (tension=positive and press=negative). And will be an additional press modeling for concrete conditions of the three main axes (-1, -1, -1) which will be the behavior of the test object.

TES RESULTS AND DISCUSSION

One form of stress-strain curve up to the limit before the stress reaches the maximum value presented by Hognestad which uses parabolic equation degree of two. From the graph, fc'is the maximum stress reached by the concrete (in this study using the value fc'). The value off'' can be differs from value fc' due to differences in the size and shape of the concrete being tested press(Park & Paulay, 1974, pp.12-14).

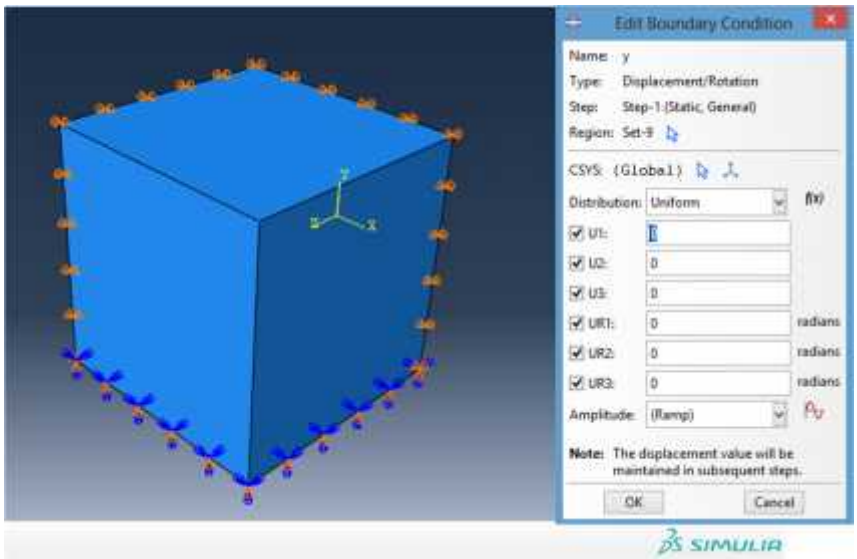

Fig. 1 Boundary Condition on Abaqus Simulation (left for BC x and z axis, right for BC y axis)

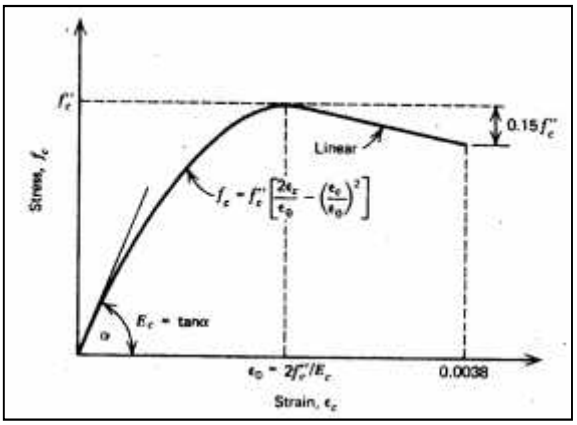

Fig. 2 Stress-Strain Curvefor compression concrete (Park &Paulay, 1974)

The **Fig.2** can be write as the **Eq.2** and **Eq.3**:

$$0 \le \varepsilon_c \le \varepsilon_0,\ f_c = f_c'\left[2\frac{\varepsilon_c}{\varepsilon_0} - \left(\frac{\varepsilon_c}{\varepsilon_0}\right)^2\right] \tag{2}$$

$$0 \le \varepsilon_c \le 0{,}0038,\ f_c = f_c'\left[1 - \frac{0{,}15}{0{,}003 - \varepsilon_c}(\varepsilon_c - \varepsilon_0)\right] \tag{3}$$

Where :

fc' = Maximum concrete stress have been reached

ε_c = Concrete strain in a point that we define

ε_0 = Concrete strain on the maksimum stress in which

Be define as : $\varepsilon_0 = 2\frac{f_c'}{E_c}$

E_c = Concrete Elastic Modulus

To get the **Fig.3** we can take **Eq. 4** and **Eq. 5**

$$\sigma_1 = E_c\,\varepsilon_1 \qquad \varepsilon_1 \le \varepsilon_{cr} \tag{4}$$

$$\sigma_1 = f_c\left(\frac{\varepsilon_c}{\varepsilon_1}\right)^{0.4} \qquad \varepsilon_1 > \varepsilon_{cr} \tag{5}$$

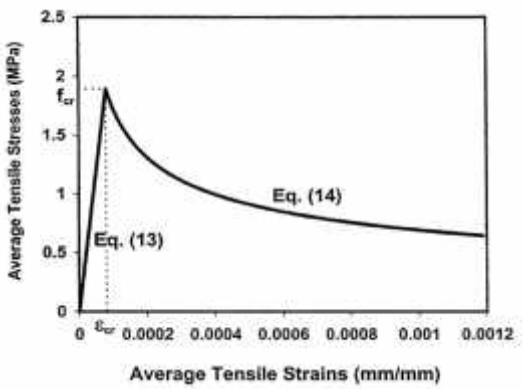

Fig. 3 Stress and strain graph for konstitutif tension modeling
(Taijundan Thomas, 2001)

Table 2. Default parameter for CDP model

Parameter Name	Value
Dlatation Angel	29
Eccentricity	0.1
fbo/fco	1.16
K	0.778
Viscosity Parameter	0

Concrete Damage Plasticity

The units used in Abaqus program is dependent on the use of the initial modeling unit, there aren't unit which can be used in Abaqus we must take MPa for constants unit.**Table 2** used for input parameter on which modeling in triaxial, biaxial, or uniaxial loading.Parameters used in **Table 2** to put the data into the modeling material using analysis of Concrete Damage Plasticity. This can be seen in **Figure 4**, in addition to the loaded behavior compression and tension of concrete at the CDP material selection. Parameter from Table 3 will be produce modeling as showin the **Figure 4** produce modeling as in the picture, which shows a contour stress on the specimen by giving pressure.

Statican alysisis used Genaral with default parameters, by adding the number of increments to 100 and change the minimum time in crement become 1E-200 in order to be closer to the analysis desired. In the triaxial loading conditions (-1, 1, -1) factorin the failure of the concrete specimen material is in tension that occurs in the field load test object. So that either the stress from the direction of x, y, and z equal value either compression or tension. **Figure 4** show the crack location that shows the pattern of fractures or cracks that occur in the test object.

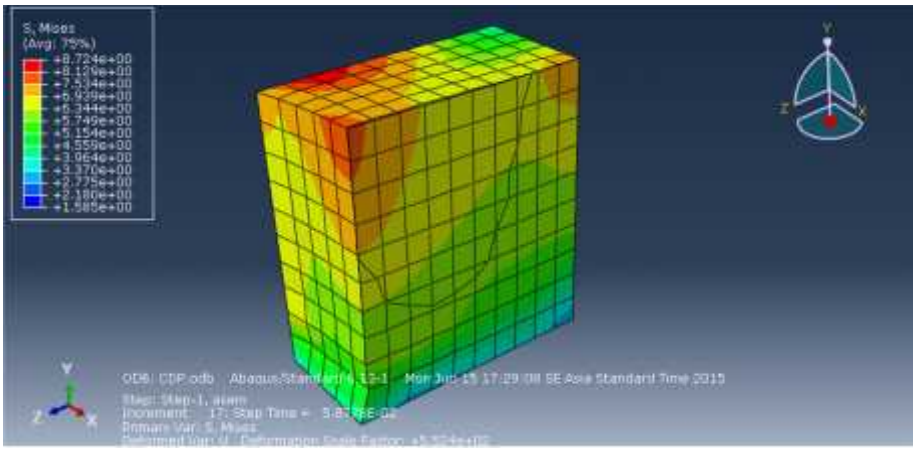

Fig. 4Von Misses ratio 0.75 -1 -1 CDP triaxial

Drucker-Prager Analysis

Ptis the value derived from the following formulation, ptis a factor hydrostatic triaxial tests. **Eq 6** When hardening is defined as the in uniaxial compression yield stress **Eq 7** when hardening is defined as uniaxial tension yield stress

$$p_c = a\sigma_c^b - \frac{\sigma_c}{3} \quad (6)$$

$$p_t = a\sigma_t^b - \frac{\sigma_t}{3} \quad (7)$$

$$s \qquad r \qquad \dot{\eta} = \frac{p}{q} \quad (8)$$

Toget the value of the initial tension is used the average valueof the **Eq(8)**, the value is taken based on **Table4** where the comment"OK" on the two sides of both positive and negative with the comments definition value must not be $0 < pt < \sigma_t$ *(Analysis User's Manual Vol III, 2010)*

$$F = a^{\ b} - p - pt \tag{9}$$

Which **Table. 3** we can make the chart depend on **it**, for define the plan p-q chart wecan use Equation 9 for the exponent suboption on druckerprager in Abaqus with **Eq. 9** we get **Fig. 8**.

$$\tan\beta = \frac{6\sin\ \Psi}{3-\sin\ \Psi} \tag{10}$$

Source: (ABAQUS, Inc., 2003.)

$$\varepsilon^{\ p} = \frac{0.1f^{\ 0.3}}{1} - \frac{f}{E} \tag{11}$$

Source: (Papanikolaoudan Andreas, 2005)

Table 3. Calculation for parameter friction angel and dilatation angel approach the parameter used

Ψ Dilatation Angel	β Friction Angel
26	51.37973684
27	52.25142736
28	53.07247156
29	53.84596761
30	54.57481153

Figure 5 shows the behavior of concrete against multi axial load, where the plan will be formed tangential angle sin p-q plan that called frictionangel. This research used frictionange l± 53,2^{O} for modeling the cubes. For more details can beseen in **Table4**, where frictionange l have relevance with dilatation angel, it is shown in **Eq10**.

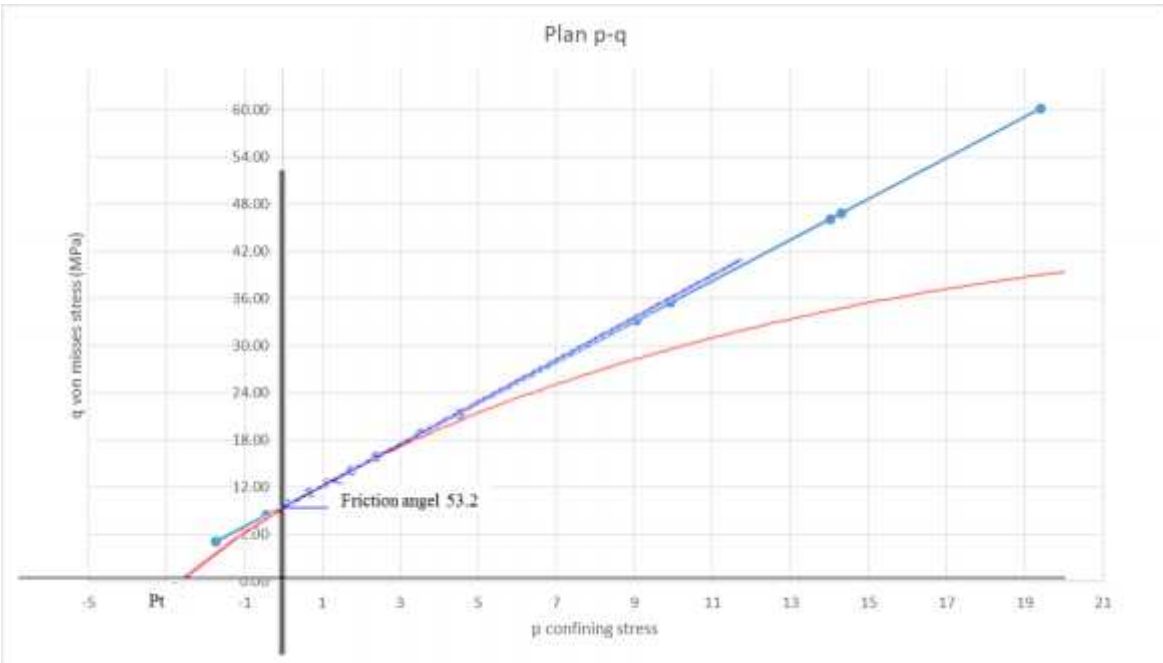

Fig 5. Curve from plan p-q for hardening definition tension and compression

Table 5 can show the possible parameter for multiaxial loading, while to determine the strain rate of load used can use the **Eq.11**, it is on show on the **Table 4**.By using some of the comparisons from previous modeling by researchers generated several possible parameters that can be used to model the multi axial test specimen, as shown in**Table 5.**

Table 4. Parameter used for Drucker Prager Analysis

Drucker Prager Triaxial Data			**Strain Rate**	
t	**q**	**p**	**η = p/q**	**Eq Plastic Strain Rate**
95.34073	60.16	20.01304	0.00938071	0
8.050713	5.08	-1.68848	0.05217021	0.00011431
71.41093	46.12	14.48128	0.05860317	0.0001244
53.86735	35.49	10.29306	0.09021019	0.00016501
31.06752	21.39	4.73766	0.12377433	0.00019822
19.81768	14.18	1.89692	0.15104545	0.00022072
15.6166	11.34	0.77796	0.18782775	0.00024684
74.27892	46.87	14.77678	0.21148948	0.00026175
52.6149	33.2	9.3908	0.27285542	0.00029544
29.80983	18.81	3.72114	0.28002705	0.000299
25.10301	15.84	2.55096	0.30399133	0.00031045
19.90491	12.56	1.25864	0.3052716	0.00031104
15.61014	9.85	0.1909	0.32266356	0.00031891
13.40729	8.46	-0.35676	0.34237795	0.00032746
Drucker Prager Eksponensial				
a	b	pt	Ψ	Use Triaxial Data And Strain Rate
0.394	1	3.69	28	
0.006052	2	1.84	28	
Drucker Prager Linier				
β	K	Ψ	Use Hardening parameter for compression strength	
69	0.778	29		

Table 5. Possible parameter for multiaxial loading

Parameter	**friction Angel**	**Dilatation Angel**	**Flow Stress Ratio**	**Material Constanta**		
	o (derajat)	**o (derajat)**	**K**	**a**	**b**	**Pt**
Van Gool, 2007	41.2184	0	0.778	0.0155	-	-
JiaFeidan Yu Fei	36.5838	-	-	-	-	-
NPL Manual	-	-	-	0.0394	2	6.956311
Conclution	**53o-69o**	**28o-29o**	**0.7780**	**0.3940**	**1.0000**	**3.6900**

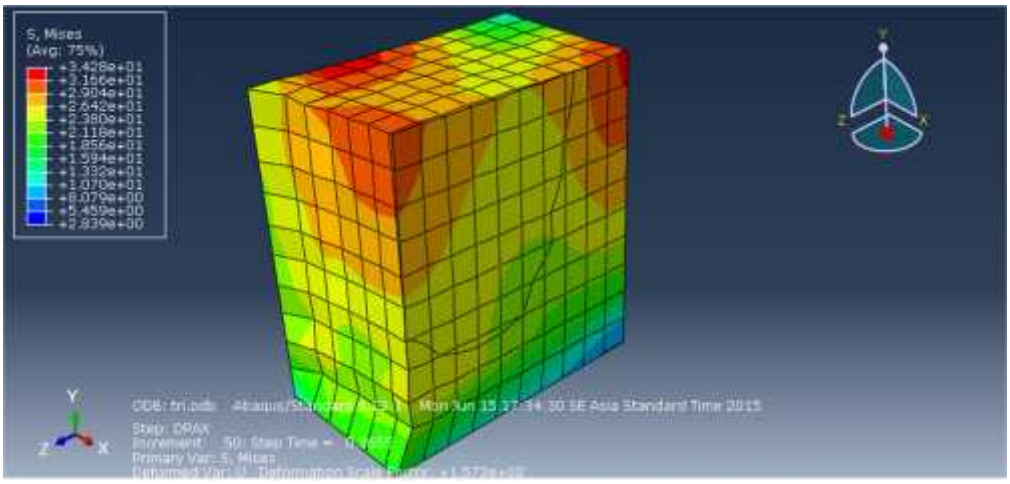

Fig. 6 Von Misses ratio 0.1 -1 -1 DPA triaxial

As for multi axial stress can be seenin **Fig** 6, where the yield stress has been achieved in some areas of the test specimen and has resulted in the test object in to collapse or in the sense that could no longer deform. For the result we can see the **Table. 6**, there are deviation from the von misses experimental just than vons misses from Concrete Damage or Drucker Prager.

Table 6. Percentagecalculation resultscomparisonusingCDPparametersorDPA

Stress Ratio			Stress Value (Mpa)			CDP (Mpa)			DPA (Mpa)			Percentage Deviation CDP	Percentage Deviation DPA
σ_1	σ_2	σ_3	σ_1	σ_2	σ_3	σ_1	σ_2	σ_3	σ_1	σ_2	σ_3		
0.00	0.00	-1	0.00	0.00	-60.16	0.000	0.000	60.160	0.000	0.000	60.980	0.00%	1.36%
1	0.00	0.00	5.08	0.00	0.00	5.080	0.000	0.000	4.869	0.000	0.000	0.00%	4.15%
0.05	0.00	-1	2.20	0.00	-43.92	2.715	0.000	-30.780	2.348	0.000	-42.270	28.49%	3.48%
0.10	0.00	-1	3.23	0.00	-32.26	2.840	0.000	-22.110	3.240	0.000	-33.060	30.40%	2.36%
0.25	0.00	-1	4.28	0.00	-17.11	3.230	0.000	-12.910	4.568	0.000	-19.000	24.54%	10.43%
0.50	0.00	-1	4.73	0.00	-9.45	3.680	0.000	-8.326	5.214	0.000	-10.840	14.80%	13.43%
0.75	0.00	-1	4.86	0.00	-6.48	3.867	0.000	-6.310	5.377	0.000	-7.320	9.70%	12.02%
0.05	-1	-1	2.23	-44.64	-44.64	3.916	-22.760	-22.760	4.320	-46.490	-46.490	43.09%	8.41%
0.10	-1	-1	3.02	-30.18	-30.18	3.711	-18.510	-18.510	4.670	-33.460	-33.460	33.07%	14.85%
0.25	-1	-1	3.76	-15.05	-15.05	3.957	-11.920	-11.920	4.410	-16.730	-16.730	15.59%	12.39%
0.35	-1	-1	4.11	-11.73	-11.73	3.800	-9.800	-9.800	4.095	-12.112	-12.112	14.14%	2.32%
0.50	-1	-1	4.19	-8.37	-8.37	3.811	-7.718	-7.718	4.428	-8.780	-8.780	8.21%	5.16%
0.75	-1	-1	4.22	-5.63	-5.63	3.860	-5.816	-5.816	4.505	-4.730	-4.730	1.77%	6.24%
1	-1	-1	4.23	-4.23	-4.23	3.900	-4.700	-4.700	4.560	-4.639	-4.639	1.65%	8.73%
Summary			N	N	N	N	N	N	N	N	N	16.10%	7.52%

CONCLUSION

From the analysis above it can be concluded several things including :

1. Compression Stress affects the criteria of failure in the concrete uniaxial press, while for the concrete analysis of biaxial and triaxial criteria of failure is strongly influenced from the stress tension on the concrete unconfiment. Boundary condition will affect the behavior of the test object to be modeled, especially inconditions of triaxial and biaxial.
2. Drucker Prager parameters used shows the behavior at a rate that is not too far from the experimental results, where we can see for the parameter using the DPA always be from thea constant percentage so well used in modeling triaxial or biaxial. While concrete damage plastisit is howed good initial behavior under conditions of stress ratio of 0.75-1 -1while when the ratio is getting smaller between the tension to the compression so approach from modeling the percentage of the experimental results also decreased.

3. Analysis with parameter of Concrete damage plasticity has advantage and deficiency, for stress from Abaqus would appoach the eksperimental for the stress ratio bigger than 0.1 while if the stress ratio smaller than it, the stress would keep away from the experimental data.
4. Drucker Prager Analysis have the advantage too, there are have the constant deviation stress on modeling between experimental, although the ratio tension and compression either the axis are small or big.

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KERANG DARAH (ANADARA GRANOSA) CLAMS AS FINE AGGREGATE IN CONCRETE MIX

Wiku A. Krasna[1], Pebri Putra Hidayat[2]

ABSTRACT: Concrete is composed of three main element of cement, aggregates (sand and gravel), and water. A large number of the use of concrete in the construction world resulted in an increased need for concrete materials, thus triggering the large scale sand mining as one of the concrete-forming material, this led to a decrease in the amount of natural resources. therefore an alternative to the sand needs to be developed as a substitute for sand in concrete mixtures. This experiment is substitute fine aggregate used in the concrete mix is kerang darah clam. it used as a substitute for reducing the use of sand. Kerang darah (Anadara granosa) can be found on almost every coast in Indonesia, the local communities around the coast consume shellfish meat and discard the clam shells. So that the clam shells become unused waste. This experiment is using five type of concrete with the use of clam shells for comparison, first type is B0 with 0% of clam shells as control sample, and BK (BK1, BK2, BK3, BK4) with a substitute for Sand with the following percentage, 10%, 25%, 35%, and 50% of the volume of fine aggregate. The sample use standard cylinder with measurements 30 cm in high and 15 cm in diameter. This experiment tested the compressive strength of concrete with compressive machine based on SNI standards. From the results of the experiments can be concluded that the use of kerang darah clam can be used as a substitute or a deduction of sand in the concrete. It is seen from the difference in the compressive strength is not much different between control sample (B0) and combination (BK), with the difference between B1(21,13 MPa) and BK1 (22,08 MPa) is 4,46%, with BK2 (23,59 MPa) or 11,61%, with BK3 (24,16) or 14,29%. and with BK4 (22,08) or 4,64%.

Keywords: *fine aggregate, concrete, Kerang Darah (Anadara Granosa)*

INTRODUCTION

Background

The number of concrete results in an increase the use of material needs concrete , could trigger sand mining as one ingredient in the form of concrete that means a drop in the number of natural resources available for the purpose of concreting.

Answer the needs, that should be thought an alternative aggregate substitute in a concrete material to be able to reduce the use of the aggregate .In this research one kind of substitute on a mixture of concrete, that is material used is a "kerang darah" clams as substitute to reduce the use of the fine aggregate (sand).

"Kerang darah" (Anadara granosa) is a type of clams fish that is commonly eaten by the citizens of East Asia and Southeast Asia , not maximum utilization of waste clams rationale for the use of blood into the clams as a substitute for sand in concrete mix. This study aims to determine the effect of the use of clams blood as a sand substitute material with compressive strength comparison of normal concrete with compressive strength of concrete using clams as a substitute for reducing the use of sand. The results of this research, is expected to contribute to the development of science and technology about concrete material, producing concrete with the power of a good, environmentally friendly, by addressing the problems of waste disposal clams

LITERATURE

"Kerang darah" clams have thick and chalky clams that can be used as coarse aggregate in the concrete mix. Shells are the name of a bunch of mollusk of the family cardiidae on who was one of fish products that has been long cultivated as one effort by-product coastal communities. Simple blood clam cultivation techniques in doing, it does not require large capital and can be harvested after the age of 6-7 months. Yields per hectare per year can reach 200-300 tons of clams intact or about 60-100 tones of scallop meat. (Porsepwandi, 1998 in Marito Shinta Siregar, 2009).

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"Kerang darah" clams containing lime, silicate and alumina. The chemical composition contained in clams as follows:

Table 1 Components contained in the clams of blood

Component	Scallop clams (wt% concentration)
CaO	67.072
SiO 2	8.252
Fe 2 O 3	0.402
MgO	22.652
AL 2 O 3	1.622

(Siti mariyam, 2006 in sinta Marito sirigar, 2009)

Mix Design

Mix Design is a process that involves two interrelated phases, namely:

1. Election of the materials suitable for the manufacture of concrete mixtures such as, cement, fine aggregate, coarse aggregate and others.
2. The determination of the relative amounts of the ingredients to produce concrete mix well.

In planning the concrete in this study used a method DOE (Department of Environment) in accordance with SNI 2847: 2013 (BSN, 2013). Article 5.6.3 SNI 2847: 2013, which set the treated specimen standard is as follows:

1) Test specimens for strength test shall be taken in accordance with ASTM C172.
2) Cylinders for the strength test should be printed and treated in accordance with SNI standards 03-4810-1998 and tested in accordance with SNI 03-1974-1990. Cylinders should be size 100 mm in diameter, 200 mm height or 150 mm diameter, 300 mm in height.
3) A power level of individual concrete quality should be considered eligible if two of the following are met:
   a) Each arithmetic average value of all three consecutive strength test (see section 5.6.2.4 frequency of testing, SNI 2847: 2013) has a value equal to or greater than.
   b) There is no test of strength (see section 5.6.2.4 frequency of testing, SNI 2847: 2013) under the more than 3.5 MPa if it is 35 MPa or less, or by more than 0.10 if more than 35 MPa.

Influence of shells of oysters as Material Added Fine Aggregate Concrete Compressive Strength Against Using FAS 0.4; 0.5; and 0.6

In the study oyster shells are used as a substitute in fine aggregate, this was due to oyster clams powder contains chemical compounds that are pozzolan which contained calcium (CaO), alumina and silica compounds thus suitable for use as an added ingredient manufacture of concrete. This study aims to determine how the effect of oyster shells as a partial replacement for fine aggregate for concrete compressive strength characteristics of the test object sat only 28 days. In this study the variation FAS used is 0.4; 0.5; and 0.6 percentage oyster clams with 30% of the weight of fine aggregate. The test results normal concrete (without substitution oyster shells) with a variation of the results obtained FAS is to FAS 0.4 = 151.13 kg/cm^2; FAS 0.5 = 123.69 kg/cm^2; and to FAS 0.6 = 102.03 kg/cm^2. Concrete with 30% substitution of oyster shells are FAS 0.4 = 174.482 kg/cm^2; FAS 0.5 = 170.914 kg/cm^2; and FAS 0.6 = 169.272 kg / cm^2 (Deviana, 2014). Based on the results obtained compressive strength characteristics that oyster shells can be used as a partial replacement for fine aggregate compressive strength of the resulting able to develop a higher strength than normal concrete.

RESEARCH METHOD

Time and Place

Implementation of this study began in November 2014 and ended in March 2015. While the place of execution of this research is in the Structures and Materials Laboratory, Faculty of Engineering, Lambung Mangkurat University, Banjarbaru, South Kalimantan.

Tools and materials

The equipment used in this study include: Formwork concrete cylinder diameter of 150 mm and a height of 300 mm, Scales with accuracy of 0.01 grams, a medium-sized machine concrete mixer along with stirrer, A set of tool inspection slump, machine vibration (vibrator machine), soaking tub, and a compression testing machine.

The materials used in this study are:

Coarse aggregate : Crushed Stone (Martadah)
Fine Aggregate : Sand River (Mataraman)
Cement : Portland Cement Type I

Substitutes : The clams clams (Kotabaru)

Test objects

Concrete mix design are made based on inspection results data quality materials with concrete plans f c = 22 MPa. Test object used is concrete compressive strength test specimens in the form of standard concrete test samples; concrete cylinder diameter 15 cm, height 30 cm, a total of 30 samples of concrete, each type of sample of 3 samples for 5 variations percentage "kerang darah" clams with the weight of fine aggregate in the mix. Sample A: Clams 0%; Sample B: Clams 10%; Sample C: Clams 25%; Sample D: Clams 35%; Sample E: Clams 50%. Concrete mixtures with variations in composition will be tested at 28 days

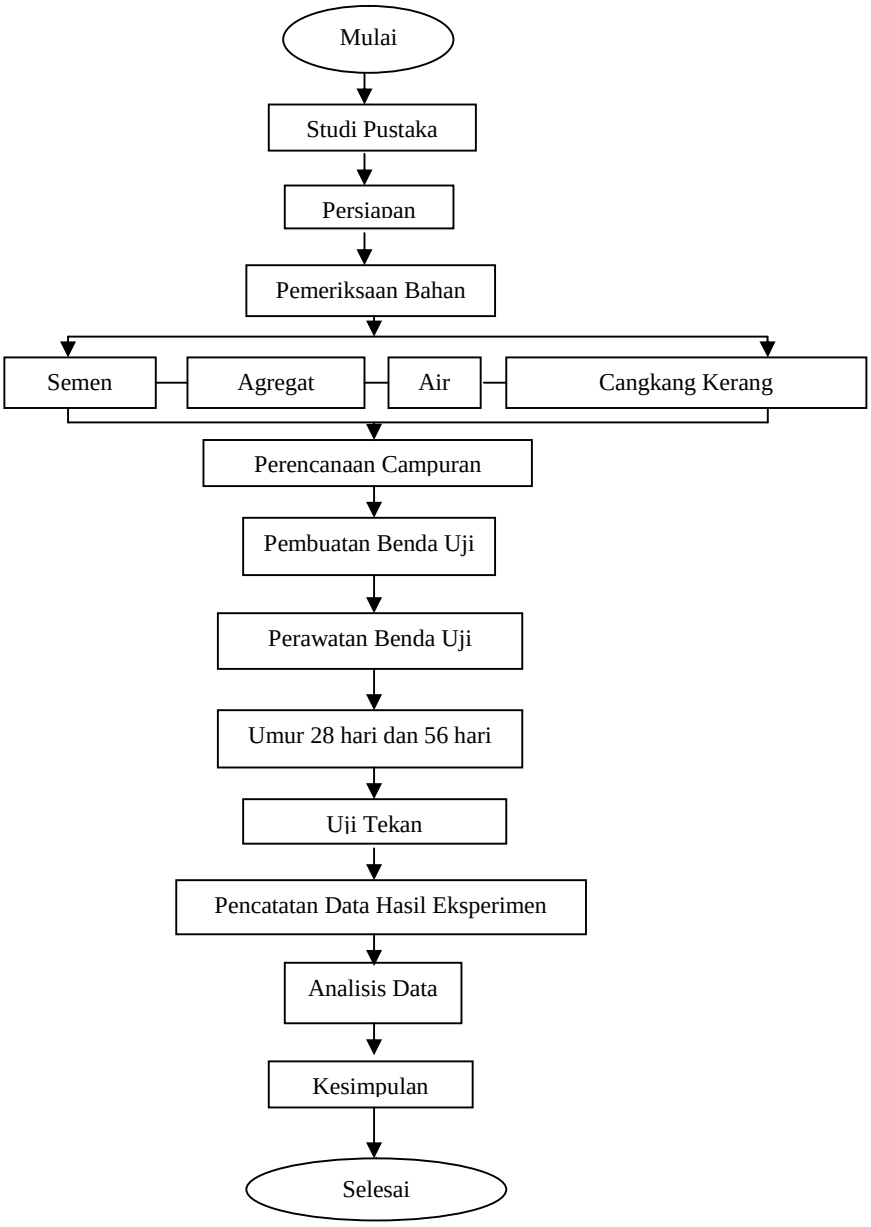

Figure 1. Flowchart of Research Methods

RESULTS AND DISCUSSION

Examination Material

The cement used in this study is Portland cement type I. Results of examination of the cement used can be seen in Table 2 as follows:

Table 2. Semen Examination Results

No.	Type Examination	Value Inspection
1	Density	3.56 g / cm 2
2	Consistency Normal	27%
3	Weight Volume Loose condition Solidification conditions Conditions wobble	 1.03 g / cm 3 1.16 g / cm 3 1.17 g / cm 3
4	Binding Time Early End	 82.5 minutes 135 minutes

Fine aggregate used comes from Matraman. Results of the tests can be seen in Table 3 as follows:

Table 3. Fine Aggregate Examination Results

No.	Type Examination	Value Inspection
1.	Water Content	5.32%
2.	Clay Levels	1.33%
3.	Organic levels	Color no. 2
4.	Sieve Analysis	Zone III
5.	Weight Volume Remove Conditions Wobble conditions Compaction condition	 1.27 g / cm 3 1.40 g / cm 3 1.44 g / cm 3
6.	Density Apparent specific gravity Bulk specific gravity on basic cleaning Bulk specific gravity on basic SSD Water absorption percentage	 2,65 2.52 2,57 1.87%

Coarse aggregate used in the form of crushed stone that comes from Martadah. Results of the tests can be seen in Table 4 as follows:

Table 4. Coarse Aggregate Examination Results

No.	Type Examination	Value Inspection
1.	Water Content	2.38%
2.	Clay Levels	0.5%
3.	Abrasion	33%
4.	Sieve analysis (maximum grain size)	20 mm
5.	Weight Volume Remove Conditions Wobble conditions Compaction condition	 1.30 g / cm 3 1.33 g / cm 3 1.40 g / cm 3
6.	Density Apparent specific gravity Bulk specific gravity on basic cleaning Bulk specific gravity on basic SSD Water absorption percentage	 2,63 2,49 2,54 2,17%

Clams are used as fine aggregates derived from Kotabaru. Results of the tests can be seen in Table 5 as follows:

Table 5. Examination Results clams of "kerang darah" (fine aggregate)

No.	Type Examination	Value Inspection
1.	Water Content	1.94%
2.	Organic levels	Color no. 1
3.	Sieve Analysis	Zone I
4.	Weight Volume Remove Conditions Wobble conditions Compaction condition	 1.39 g / cm 3 1.48 g / cm 3 1.52 g / cm 3
5.	Density Apparent specific gravity Bulk specific gravity on basic cleaning Bulk specific gravity on basic SSD Water absorption percentage	 2.97 2.75 2,82 2,67%

The Test Results Workability

The results of the test and the weight of the concrete slump can be seen in Table 6 as follows:

Table 6. Results of Testing Slump Concrete and Weight Content Fresh Concrete Testing

Variations Mixed	Decrease (cm)	Weight Content Fresh Concrete (gr / cm^3)
Mix I (0%)	6	2,17
Mix II (10%)	4	2,10
Mix III (25%)	4	2,17
Mix IV (35%)	3.2	1.98
Mix V (50%)	4.5	2,14

From the results in Table 6 above, it can be seen that the variation affects the clams mixture slump test results that vary between 3.2 to 6.0 cm. This is because the rate of water absorption in the clams higher than the sand used is the absorption of water in the sand as much as 1.87% and 2.67% so that the clams of fresh concrete mixture experiencing water shortages.

The Results of The Testing Levels of Salt pH And Sulfate

This examination is intended to determine levels of salt, pH, and sulfuric acid clams which is highly corrosive to steel and often lead to rust on steel reinforcement. The test results can be seen in Table 7 as follows:

Table 7. Results of Assays salt, pH, and Sulfate

No.	Sample Code	Parameter		
		pH	The salt concentration (%)	Sulfate (ppm)
1	K1	6,99	0.88	106.364
2	K2	5.12	0.92	113.112
3	K3	8.98	0.90	379.394
4	K4	7.51	0.91	334.759
Method		Electrode	Titration	SNI th. 2009

From the test results four samples, all indicate that the sample is alkaline clams. While the levels of salt are contained does not exceed 1% and the sulfate content of small means relatively mild.

Concrete Compressive Strength Testing Results with variation Mixed Clams

Modification casted concrete test specimen with an average compressive strength of 22 MPa planned. Results of the test can be seen in Table 8 - Table 12 as follows:

Table 8. Concrete Compressive Strength Testing Results with Mixed 0%

No.	Sample Code	Weight (Kg)	Maximum load (Kg)	Testing Compressive Strength (MPa)	Average Compressive Strength (MPa)	Age (day)
1.	A-1	12	37000	20.95	21.14	28
2.	A-2	12, 2	35000	19.82		
3.	A-3	12, 1	40 000	22.65		

Table 9. Concrete Compressive Strength Testing Results with Mixed 10%

No.	Sample Code	Weight (Kg)	Maximum load (Kg)	Testing Compressive Strength (MPa)	Average Compressive Strength (MPa)	Age (day)
1.	B-1	12, 0	39 000	22.08	22.08	28
2.	B-2	12, 2	35 000	19.82		
3.	B-3	12, 0	43 000	24.35		

Table 10. Concrete Compressive Strength Testing Results with Mixed 25%

No.	Sample Code	Weight (Kg)	Maximum load (Kg)	Testing Compressive Strength (MPa)	Average Compressive Strength (MPa)	Age (day)
1.	C-1	1 2.2	430 00	24.35	23.59	28
2.	C-2	1 2.3	42 000	23.78		
3.	C-3	1 2.2	400 00	22.65		

Table 11. Concrete Compressive Strength Testing Results with Mixed 35%

No.	Sample Code	Weight (Kg)	Maximum load (Kg)	Testing Compressive Strength (MPa)	Average Compressive Strength (MPa)	Age (day)
1.	D-1	1 2.5	420 00	2 3.78	2 4.16	28
2.	D-2	1 2.4	42 000	2 3.78		
3.	D-3	1 2.2	44 000	24.91		

Table 12. Concrete Compressive Strength Testing Results with Mixed 50%

No.	Sample Code	Weight (Kg)	Maximum load (Kg)	Testing Compressive Strength (MPa)	Average Compressive Strength (MPa)	Age (day)
1.	E-1	12, 3	39 000	22.08	22.08	28
2.	E-2	12, 1	39 000	22.08		
3.	E-3	12, 2	390 00	22.08		

The results of the above table can be seen in Figure 2. as follows:

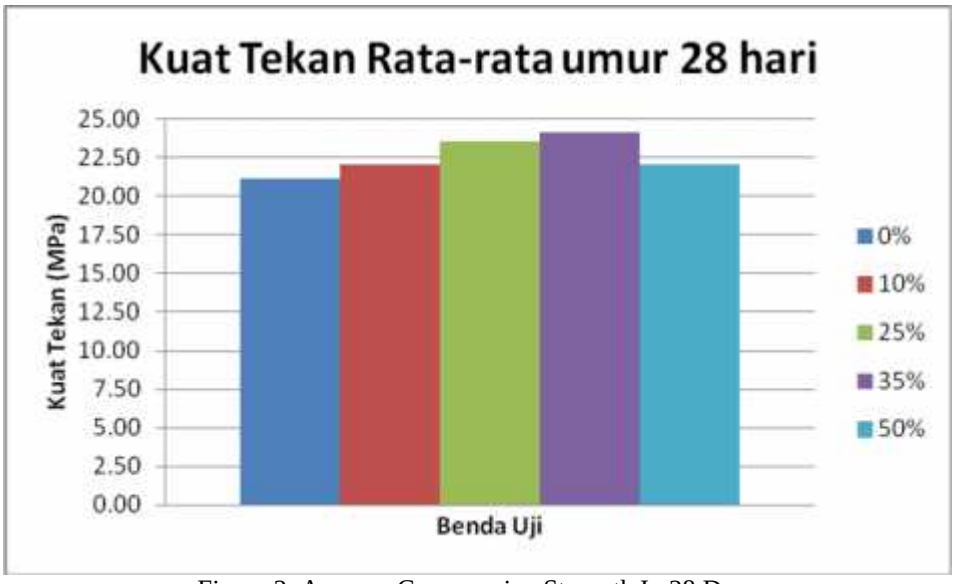

Figure 2. Average Compressive Strength In 28 Days

Based Figure 2 images can be seen that the highest compressive strength at 28 days is concrete with a mixture of 35% amounting to 24.16 MPa.

Comparison of the results with a Mixed Concrete Concrete Concrete Compressive Strength Against Normal Age 28 Days

From the results of experiments that have been done obtained a percentage of the compressive strength of concrete age of 28 days, the calculation is done in the following way:

From the above data it can be concluded that the concrete mix mussel clams has increased an average compressive strength of the concrete of the average compressive strength of normal concrete in a mixture of 10%, 25%, 35%, and 50% is 4.45%; 11.61%; 14.29%; and 4.46%.

CONCLUSIONS AND RECOMMENDATIONS

Conclusion

From the results of research on the use of blood clam clams as fine aggregate in the concrete mix can be concluded as follows:

1. "Kerang darah" clams can be used as a substitute for fine aggregate
2. From the test results that have been made known comparison of the average compressive strength of the concrete mix to 28 days increased from an average compressive strength of normal concrete (A = 0%), namely sample B (10%) as much as 4.45%; sample C (15%) as much as 11.61%; sample D (35%) 14.29%; and sample E (50%) as much as 4.46%.
3. From the test results that have been made known to the use of clams in concrete mix slump affecting the value resulting from the high water absorption by "Kerang Darah" clams that is 2.67% compared to 1.87% as much sand and crushed stone as much as 2.17%

Implication

From the results of research on the use of clams of compressive strength of concrete can provide the following benefits:

1. Provide another option in terms of waste handling clams.
2. Contribute to the development of science and technology of concrete admixture are clams as fine aggregate

Suggestion

As for suggestions that can be made for subsequent research are as follows:

1. Further research on this can be done by clicking the crushed oyster clams to resemble a finer grain of sand in general.

2. For further utilization of clams can also use cement type II, which is the cement that is specific to the area salinity / sulfate medium.

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THE EFFECT OF REED AS FILLER UPON BRICK PERFORMANCE OF BUILDING MATERIAL

Kurniati Ornam[1], Masykur Kimsan[2]

ABSTRACT:This paper examines performance of reed as filler for the modified bricks. As shown on previous research, modified hollow bricks with sawdust as filler meet the standard of ASTM C216-15, ASTM C652-14, ASTM C67-14, ASTM C62-10, and SNI 15-2094 for construction material. In accordance with sawdust properties as insulator, reed (*imperata cylindrica*)is expected to share precise result as sawdust does upon brick's production. Brick's production follows conventional process in home industry, with 1:8 of water-clay ratio. Nine compositions of reed mixed with clay i.e. 9% (type A), 13% (type B), 17% (type C), 20% (type D), 23% (type E), 26% (type F), 29% (type G), 31% (type H) and 33% (type I), will be evaluated on its dimension, weight, and compressive strength (NI-10 standard). Moreover, in terms of economic aspects and sustainability, the speed of burning process and the number of firewood being utilized were scrutinized. The result shows that all types meet ASTM C216-15, ASTM C652-14, ASTM C67-14, ASTM C62-10, and SNI 15-2094requirements which their compressive strength surpass 10 MPa (level 1 on ASTM C216-15, ASTM C652-14, ASTM C67-14, ASTM C62-10, and SNI 15-2094standard) and 0,02% reduction of weigth following burning process. Compared to conventional brick, there is approximately 50% time reduction for production of brick with reed as filler. Hence, 50% firewood diminution ensues, which is of paramount importance for environment.

Keywords: *Brick, reed, dimension, compressive strength, firewood*

INTRODUCTION

In 2014, based on statistical yearbook of Indonesia,there were46,77 million Indonesianwho were still homeless or 22% of the total population of Indonesia, where Southeast Sulawesi contributed as many as 4,23 million people or 1,60%.Moreover, per capita income of Indonesian society is considered low, hence to meet the need of housing is limited as well, especially for farmers.Therefore, engineering design and material construction technology had been developed to be cheaper and more sustainable e.g. hollow brick design with sawdust as filler (Ornamet al., 2010 &2015).

Bricks are most preferred building materials due to its cheap and reasonable price, good thermal insulator properties, and that they are exposed and can be produced locally through home industry. Nevertheless, bricks production in home industryhad encountered classic situation where the level of productivity is akin to the previous period and tends to decrease as a consequence of modest technique of production process. In addition, environmental impacts e.g flood and erosion might ensue resulted from the excavation of the soil and logging.

Hollow brick design reducesapproximately 25% of soil (ornam et al. 2010). Heat transfer performs acceptably in this model where it spreads more quickly and evenly in brick structures than on conventional model and hence minimizing the possibility of crack following unloading phase. On the other hand, the use of sawdust as inner burning,accelerates burning time twice and it also can reduce the use of firewood for about 50% (ornam et al, 2015).

On those basis, as an organic material that has most similar characteristics to sawdust,reed (*Imperata Clindrica*) is then developed to beengineered materials ofas a mixture for bricks. Reed is categorized as wild plant and agricultural nuisance plant thatgrows easily and rapidly in various places, especially in a fertile soil that might reach 1,0 to 2,0 meters of height. Some agricultural experts estimate the total area of reed in Indonesia might reach 200.000 million hectares with the growth rateat16.000 hectares that regularly continues over time (Isnadi et al. 2011). Meanwhile in Papua, reed is utilized as roof material of the building by some native people whose lives are still austere.Additionally, in Southeast Sulawesi, the use of reed is limited as fodder and the rest is burned because it disturbsmain agricultral plants. Conversely, higroscopicproperties of reedwhen it experiences heat tends to ameliorate the burning process. Several other studies in the development of bricks have been conducted by mixing natural materials i.e. fiber or mixed waste material including corn and coconut (Christiawan and Darmanto 2004), lime sand (Sudarsana 2011), and ash of

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bagasse (Disurya et al. 2002). Therefore, as similar organic materials, reed might be used as a mixture of brick and through this study, it is aimed to scrutinize the effect of reed on the bricks as a building material as compared with standard bricks (conventional) from the economical aspect and sustainability.

PRODUCTION AND STANDARD

Brick is building material that is originally created from clay and additional mineral that can be shaped in certain size. There are 3 (three) methods in production of bricks (Civil Engineering Materials 2001), i.e.

a. Stiff-mud process, with 12 - 15 % water content
b. Soft-mud process, with 20 - 30 % water content
c. Dry-press process, with 7 - 10% water content (low plasticity).

Here are some steps of bricks production (Brick Industry Association 2001):

a. Mining and storage of raw materials
b. Size reduction and screening
c. Forming and Molding
d. Coating and drying
e. Burning and cooling
f. Storage and shipping

Brick, which is well known as *batu merah* for Indonesian people, is commonly produced by small scale home industry, using these following ingredients (Heinz 1999):

a. Six part of total weigh of clay, which consists of 50% - 70% silica;
b. Two part of total weigh of husk, which is used to cover the base of duplicator,in order the duplicated brickis not attachedwith the ground (base);
c. One part of total weigh of animal's faces, such as cow's; horse's; pig's and other herbivore's, which is functioned to moderate the clay and is functioned as agent to speed up the brick burning;
d. Four part of total weigh of water, which is used for moderating and soaking the dough. The mixed clay; husk;and animal faces is soaked into the water.
e. Sand or red cement, if necessary.

The raw material of brick is clays, which is mixed and processed with other ingredients. In conventional home industry of bricks, that process is described as follow (Heinz 1999):

a. Clay, animal faces and water are mixed and stirred until smooth. Gravel or other material is pulled out, because it can decrease the quality of brick.
b. This dough is soaked for a day (24 hours).
c. After 24 hours, the dough is ready for duplication. First, the surface of ground, where the duplication done is covered by husk, in order the dough is not attached to the ground as the base. The duplicator is made of wood or steel consisted of two frame, outside and inside. The outside frame is bigger ± 1 mm of its every side, in order the duplicated brick is easy to be let out. Commonly, duplication is done in dry season under the sun shine.
d. After that, the wet bricks are kept for a while to let it dry. The dry brick is arranged to upper side 10-15 bricks, and is repeated in rows. They are kept 2-7 days; in order wet bricks can completely dry well.
e. The dry bricks are burned using wood as the fuel, by arranging it triangularly on the trivet. In particular part of dry brick arrangement is let to be empty. The outside of this arrangement is covered with clay to keep the high temperature, which is up to 800 °C, is still inside the arrangement. By which, the brick is expected to be burned completely well. This burning is repeated 4-5 days.
f. The produced bricks are kept at work station to be distributed.

Conversely, properties of brick with treatment is called mechanical properties.Compressive strength is one of them as stated on ASTM C67-14, 2014 where the minimum requirement is 10,40 Mpa.

As comparison, there are some standards on SNI 15-2094-2000 that bricks production should meet i.e.

1) Texture and shape

   Square, rough surface, red color, having loud voice when it is tapped, no crack, and unbreakable. Dimension requirement is as shown on table 1.

2) Compressive Strength

   Brick is classified into 6 (six) classes based on its compressive strength i.e. class 25, 50, 150, 200 and 250. This class represents its compressive strength of 30 bricks that have been tested as shown on table 2.

3) Red brick does not consist of soluble salt hence its crystallization covers more than 50% of bricks surface and might reduce its preservation.

4) Water absorption is not more than 22%. Based on water absorption, brick is classified into 5 (five) groups as shown on table 3.

Table 1Bricks dimension

Thickness (mm)	Width (mm)	Length (mm)
115 ± 5	175 ± 7	300 ± 12
	115 ± 5	240 ± 10
71 ± 5	115 ± 5	300 ± 12
	105 ± 4	240 ± 10
52 ± 3	115 ± 5	240 ± 10
	105 ± 4	200 ± 10

Source: SNI 15-2094-2000

Table 2Standard of compressive strength test

Class	Minimum average compressive strength of 30 bricks		Allowable coefficient of variations
	Kg/cm^2	N/mm^2	
50	50	5	22
100	100	10	22
150	150	15	15
200	200	20	15
250	250	25	15

Source: SNI 15-2094-2000

Table 3Water absorption of bricks

Class	Maximum water absorption (%)
50	22
100	20
150	20
200	20
250	20

Source: SNI 15-2094-2000

METHODOLOGY

This research was performed in two approaches i.e. laboratory testing and production process in the home industry ward.

a. Laboratory Testing

Two laboratorieswere facilitated to perform this research i.e. Soil Mechanics Laboratory and Construction MaterialLaboratory of Faculty of Engineering, Halu Oleo University. Laboratory test included testing on soil and reed i.e.sieve analysis, atterberg limit, water absorption and compressive strength of modified brick. Measured production of modified brick was also carried out from mixing, molding, drying and cooling, burning and storage.

Fig. 1 Testing and production of conventional and modified brick's in laboratory

b. Production in the home industry ward

Production of bricks with reed as filler was conducted on home industry scalein the Said Jaya ward located in Punggolaka-Puuwatu, Kendari, Southeast Sulawesi. The production process includedexcavation, mixing, molding, drying, burning and unloading/storage.

Fig. 2Production of conventional and modified brick's in home industry

Prior to laboratory works, fresh/green reed was first obtainedwhere its height reached ± 1 meter and then dried in the open space to gain direct sunlight until reed color changed to amber. After that, the reedwas then severed to the smallest size (± 2 mm), followed by arranging the composition of the water –clay ratio that meets the ASTM standard C216-15 (2015) and SNI 15-2094 (2000) i.e. 8: 1.Furthermore, thecomposition of the soil and reed was also determined where the average weight of soil was 2 kg while the reed varied in 9 (nine) composition i.e. 2: 0,2 (9%); 2: 0,3 (13%); 3: 0,4 (17%); 2: 0,5 (20%); 2: 0,6 (23%); 2: 0,7 (26%); 2: 0,8 (29%); 2: 0,9 (31%); and 2: 1 (33%). Weight measurement of soil and reed was performed by digital measuring scales. In addition, soil testing was conducted in laboratory to determine soil classification in accordance with ASTM C216-15 (2015) and SNI 15-2094 (2000).

Next, production process was performed in the ward. Clay was mixed with reed and water in accordance with the predetermined composition, where30 pieces of brick for each composition were produced. Each composition was separately and evenly stirred by trampling it until the mixturewas not adhere and might be molded. Bricks were molded by rectangular wood with 21,5 cm length, 11 cm width and 5 cm thickness. Once bricks were completely molded, they were arranged for drying purpose in the outer side of the ward to obtain direct sunlight for ± 2 weeks. The dried bricks were then arranged in a rectangular furnace by using firewood as fuel. Furnace is in the form of a pile of bricks on the ground that its depth reaches 50-70 cm from ground level, where the brickwork was made such that there was a hole to insert the firewood, and the burning process took 2 days. Conventional bricks were also produced with similar size and method to be compared with modified bricks in the analysis.Following the burning process, bricks were unloaded and prepared for examination in laboratory that coversdensity, color, dimension, textures and shapes, compressive strength,

water absorption, and salt content. After examination and laboratory tests, analysis was performedto examine the effectof reed on bricks mixture compared to conventional brick as building material.

As a comparison to the productionin labor ward, production process was also carried out in the laboratory begun with the mixing and molding modified brick. Fresh green reed was cut and put into an oven at temperature of 100°C for 24 hours until the color changed to amber. Once dry, reedwas then evenly blendedto formreed powder.Subsequently, conventional and modified bricks were molded with similar size i.e. 21,5x11x5 cm^{2}where there were two pieces of bricks for each composition. In addition, to obtain the corresponding compressive strength that meets standard dimensions of compressive strength machines, bricks with dimension of 14 x 8 cm^{2}were also produced to be compared with the other dimension both from labor ward and laboratory. Next, bricks were dried by putting into oven at temperature of 150 ^{o}C. After that, the brickswere burned on a stove with average fire temperature of 350 ^{o}C (measured by fire thermometer). To achieve anentirely heat transfer, brickswere wrapped in aluminum foil and baked for 5-6 hours. Then, the bricks were tested and examined to obtain its physical and mechanical properties as bricks from labor ward.

RESULT AND DISCUSSION

As shown on table 4 and 5, there were reduction in dimension and volume in comparison of modified bricks to conventional bricks.Dimensions of conventional bricks after burning were reduced with 20,8 cm length, 10,6 cm width, 4,9 cm thickness and 1.8 kg weight. On the other hand, dimensions of modified bricks were reduced ranging at16,2 - 19,3 cm length; 9 – 9,7 cm width; 4,1 –4,5 cm thickness; and 1,2 –1,4 kg weight. This was generated by the influence of mixture of reed, where the reed was also burned as inner burning that their dimensions and volume decreased. The lighter brick will provide the better quality of the bricks, as italso consequentlyreduces the distribution or delivery cost.

Table 4 Brick's dimension

Reed content	After drying				After burning			
	Length (cm)	Width (cm)	Thickness (cm)	Weight (Kg)	Length (cm)	Width (cm)	Thickness (cm)	Weight (Kg)
Conventional brick	20,8	10,8	5	2	20,8	10,6	4,9	1,8
9% (Type A)	19,2	10,2	5	1,6	16,2	9,4	4,5	1,4
13% (Type B)	19,7	10,1	4,8	1,6	19	9,4	4,5	1,2
17% (Type C)	19,2	9,6	4,6	1,55	18,7	9,3	4,3	1,3
20% (Type D)	19,5	9,7	4,6	1,4	19,3	9,4	4,4	1,3
23% (Type E)	19	9,5	4,6	1,4	18,8	9,3	4,1	1,3
26% (Type F)	19	10	4,6	1,5	18,9	9,2	4,3	1,2
29% (Type G)	19,2	10	4,7	1,4	19	9,7	4,4	1,3
31% (Type H)	19,4	9,7	4,3	1,4	19	9	4,5	1,2
33% (Type I)	18,7	9,5	4,6	1,5	18,8	9,7	4,3	1,3

Table 5 Shrinkage modified brick with filler reed

Reed Content	Shrinkage		
	Length	Width	Thickness
Conventional brick	3%	4%	2%
9%	13%	12%	14%
13%	12%	15%	10%
17%	13%	14%	16%
20%	10%	15%	12%
23%	13%	15%	18%
26%	12%	16%	14%
29%	12%	12%	12%
31%	12%	18%	10%
33%	12%	16%	10%

Table6The time comparison between conventional brick and modified brick

Reed content	Burning time (day/hours)	Percentage of Firewood reduction (%)
Conventional Brick	5/20	100
9% (Type A)	3/21	63,83
13% (Type B)	3/18	63,83
17% (Type C)	3/12	59,57
20% (Type D)	3/9	57,45
23% (Type E)	3/5	54,61
26% (Type F)	2/22	49,65
29% (Type G)	2/17	46,10
31% (Type H)	1/23	33,33
33% (Type I)	1/16	28,37

Table 7Final brick production comparison between conventional brick and modified brick

Reed Content	Number burned brick	Number final brick
Conventional Brick	500	252
9% (Type A)	500	384
13% (Type B)	500	397
17% (Type C)	500	414
20% (Type D)	500	432
23% (Type E)	500	452
26% (Type F)	500	461
29% (Type G)	500	478
31% (Type H)	500	482
33% (Type I)	500	498

It can be seen from table 6 and 7 that burning time reduction between conventional bricks and modified bricks. On the conventional bricks, burning time of 500 pieces reached up to 141 hours with damage to the brick after unloading reaches 50%. Beside, on modified bricks, it only took40 to 93 hours for burning time, percentage reduction of firewood usage reached 28,37 to 63,83% and the percentage of damage after unloading variedfrom 0 to 23%. Moreover, higher reed composition reduced more burning time, firewood and damage. This might be as a resultof inner burning and/or hygroscopic properties of reed that will facilitate heat transfer to the whole parts of bricks, either single brick or the whole arrangement of bricks.

Table 8 Water absorption and salt

Reed content	Weight in water (gr/dm^2/minute)	Saturated weight (gr/dm^2/minute)	Dry weight (Kg)	Salt content
Conventional brick	796,6	1707,4	1494,3	< 50%
9% (Type A)	720,8	1549,8	1304,1	< 50%
13% (Type B)	751,2	1611,9	1353,4	< 50%
17% (Type C)	769,7	1605,9	1375,7	< 50%
20% (Type D)	717,9	1521,7	1291,2	< 50%
23% (Type E)	753,2	1618,9	1356,2	< 50%
26% (Type F)	786,6	1672,5	1415,3	< 50%
29% (Type G)	693,0	1496,5	1259,2	< 50%
31% (Type H)	731,8	1544,8	1304,7	< 50%
33% (Type I)	739,0	1551,5	1323,2	< 50%

Evaluation of water absorption and salt content is presented on table 8 both for conventional bricks and modified bricks. In terms of water absorption, conventional bricks did not meet the requirement that exceeded 22%. However, their salt content met the requirement as specified in ASTM C216-15, ASTM C652- 14, ASTM C67-14, C62-10 ASTM, and SNI 15-2094. Furthermore, at the immersion, it wascaptured that the white crystal did not cover most of the surface of the bricks. On the other hand, both water absorption and salt content, modified bricks met all the requrements andnot more than 50% of white crystal covered the bricks surface. Lesser percentage of crystalization will increase the durability of the brick itself. Bricks immersion over 24 hours with cold water (cold water absorption) was intended to look at the maximum capability of brick to absorb cold water (at room temperature) to saturation. Absorption value of

bricks indicate less dense composition of bricks so that water can fill cavities in them. itaffects on the strength or durability of bricks on bad weather.

Table 9 Brick's density of brick modified

Reed Content	Brick Density (gr/cm^3)
Conventional brick	2,37
9% (Type A)	1,68
13% (Type B)	1,60
17% (Type C)	1,73
20% (Type D)	1,79
23% (Type E)	1,84
26% (Type F)	1,71
29% (Type G)	1,71
31% (Type H)	1,64
33% (Type I)	1,65

In terms of bricks density, conventional bricks exceeds the requirement i.e. 1,60 – 2,00 gr/cm^3 (table 9). On the contrary, modified brick density is between 1,60 and 1,84 gr/cm^3. Density of bricks is influenced by the composition of the raw materials, mixing process either manual or using blender, and the duration of drying and burning process.

However, as can be seen on table 10, conventional brick reaches 8,21 Mpa for compressive strength, and classified as third grade based onASTM C216-15, ASTM C652-14, ASTM C67-14, ASTM C62-10, and SNI 15-2094. On the other hand, in average, modifid brick shows better performance than conventional brick. All types are included in first grade, with maximum compressive strength on type A (9%) for 15 Mpa. Brick's compressive strength is influenced by the density and color of the bricks. Should the density is larger, then compressive strength will increase as well.

Table 10Compressive Strength of ModifiedBrick

Reed content	Compressive Strength (N/mm^2)
Conventional brick	8,21
9% (Type A)	15
13% (Type B)	12,4
17% (Type C)	9
20% (Type D)	12,25
23% (Type E)	12,7
26% (Type F)	10,86
29% (Type G)	14,5
31% (Type H)	10,4
33% (Type I)	11,4

CONCLUSION

Test results indicate that the effect of reed in a mixture of brick with certain composition show better quality compared with conventional brick as a building material based on ASTM C216-15, ASTM C652-14, ASTM C67-14 , C62-10 ASTM, and SNI 15-2094Increase of brick's density at the beginning will not share similar distribution after burning due to insulator's properties of reed, modified brick filled improves the quality and quantity of produced bricks.

From the economic and sustainability aspect, it can be concluded thatmodified bricks will contribute in time reduction in burning process and in utilizing the amount of firewood up to 50% so that flood or erosion can be minimized. Modified brick with reed filler can reduce the labor cost and increase the number of mobilized brick. Hence, it will reduce the total price of bricks

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STUDY ON RESIDUAL STRENGTH AND CARBONATION OF CONCRETE IN CENTRAL MARKET SHOP AFTER FIRE

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ABSTRACT: Temperature increase significantly during the fire caused a fundamental change of the properties of concrete structures. Based on this, the objective of this study are: (i) to identify damage of concrete beams, columns and slabs; (ii) to determine the level of carbonation; and (iii) to evaluate the residual mechanical properties of concrete after fire. The study was conducted in the field and in the laboratory. Study in the field include the Schmidt Hammer Test, Core Drill Test, and carbonation, while laboratory studies was compressive strength test. The results showed that damage to the beams, columns and slabs are undistributed, after fire the carbonation level by 1/3 of the slab thickness or ± 30 mm which was neutralized from the bottom of slab, and the residual strength of concrete after fire as follows: (a) the temperature that occurs at the time of fire was estimated at 900 ° C to 1000 ° C for about 6 hours; (b) the compressive strength (f'c) from Schmidt Hammer Test in the range of 10 MPa - 32.044 MPa, while from Core Drill Test in the range 6,41 MPa - 16.57 MPa.

Keywords: *Residual strength, concrete, after fire.*

INTRODUCTION

Building fire is one of the urban problems until today. With the development and progress of the construction, which grew rapidly, increasing the risk of fire. Fire could be fatal and results in a great loss to both of materials and immaterials.

The roles of the expert in dealing with the structure of the building after fire are: (a) estimating highest temperature experienced by the structural elements in the event of fire, (b) estimating residual strength of the structure of the building after fire, and (c) proposing reinforcement technique elements structure in such a way as necessary so that the building can function as before the fire.

On May 7, 2014 Central Market Shop on fire. Based on *Kompas.com*, the fire occurred at 20.40 PM until the early morning on the next day, so the fire can be estimated occur for 6 hours. Central Market Shop, which is located at Jl. H. Oemar Said Tjokroaminoto and Jl. KH. Ramli, is a 4 stores whole market building with reinforced concrete construction design.

High temperatures during the fire has great affected on the structural elements of the building shop. Temperature increased significantly during the fire caused a fundamental change of the properties of concrete structures. So that the construction of the building structure is not capable to support the load and led to collapse in some part of Central Market Shop.

Loss of strength occurs due to a gradual change in the chemical composition of the cement paste. In order to protecting the steel reinforcement from the high temperature of fire, the concrete cover need to be designed with sufficient thickness.

In addressing the problem scientifically and accurately, using a variety of assessment methods, both destructive and non-destructivetest were conducted to determine the residual strength of concrete in Central Market Shop after fire.

The objectives of this study are:

1. To identify damage of beams, columns, and slabs in Central Market Shop after fire
2. To determine the level of carbonation slab in Central Market Shop
3. To evaluate the residual strength of Central Market Shop after fire

LITERATURE REVIEW

Concrete is a mixture consisting of sand, gravel, crushed stone or other aggregates are mixed together with a paste made of cement and water to form a rock-like mass. Sometimes, one or more additives are added to produce concrete with specific characteristics, such as ease of processing (workability), durability, and hardening time.

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Like substances similar to other rock, concrete has a high compressive strength and tensile strength are very low. Reinforced concrete is a combination of concrete and steel reinforcement which serves to provide the tensile strength whose is not owned by concrete.

Concrete consists of aggregates, cement and water are mixed together in a plastic state and is easy to work with. Because of these properties causes the concrete is easy to be formed in accordance with the wishes of the user. Shortly after mixing, the mortar chemical reactions are generally hydration and produce a hardening and increase the strength.

Concrete is a building material that has resistance to fire relatively better than other material such as steel, especially wood. This is caused by the concrete are materials with low thermal conductivity, thus preventing the propagation of heat to the inside of the concrete structure. Therefore, concrete covers are usually designed with sufficient thickness that are intended to protect the reinforcement of the high temperatures outside in case of fire, because as we know that reinforcing steel will decrease the strength / stress fairly drastic melting at high temperatures.

Concrete structures affected by fire, the strength of concrete will be affected by changes in temperature, level and duration of heating. Concern on whether the burned concrete strength is still able to withstand the various loads including axial force is, bending and shear forces which can be seen in the stress-strain relationships that occur within the concrete.

As the axial load is increased gradually, the length of the gauge length measured at each increment load and this continued until damage (fracture) on the specimen. By knowing the initial cross-sectional area of the specimen, then the normal stress, which is expressed by fc, can be obtained for any axial load value by using the relationship where P states in the axial load (Newton) and A represents the initial cross-sectional area (mm^2).

CONCRETE COMPRESSIVE STRENGTH

The compressive strength characteristics calculated f’ck = f’cm - 1.64 with a significance level of 5%. As for the other factors that can affect the quality of concrete the strength as proposed by Mulyono (2006), namely: (1) the proportion of constituent materials, (2) a method of mixing, (3) curing, and (4) the circumstances at the time of casting. Regulation of Reinforced Concrete 1971 in Indonesia NI - 2 (1971) are described the class and quality of concrete as presented in Table 1.

Based on SNI 03-1974-1990, the compressive strength of concrete can be calculated by the formula:

$$fc' = \frac{P}{A} \quad (1)$$

where:

f'c = concrete compressive strength (N / mm²)

P = maximum load (N)

A = cross-sectional area (mm²)

In calculating the compressive strength must also take into account the ratio of the height and diameter of the test specimen. In addition, if it was found any reinforcement in the test specimen should be a correction.

Value after correction of compressive strength of concrete core testing are as follows:

$$f'cc = C_0 \times C_1 \times C_2 \times f'c \quad (2)$$

where:

C_0 = modification factor for directional drilling

C_1 = modification factor for ratio of the height and diameter

C_2 = modification factor of the reinforcement

f'cc = compressive strength of concrete core which is corrected in MPa

f'c = compressive strength before corrected in MPa

EFFECT OF FIRE ON CONCRETE

Increasing temperatures will cause the carbonation process is the formation of calcium carbonate (CaCO3) that has off-white color to change the color of the concrete surface becomes brighter (whitish pink). Furthermore, at temperatures above 600°C or 700°C other elements hydration results changed the composition so that the strength of concrete loss at all.

To determine whether the concrete surface is contaminated or not, it can be tested by using a solution of pH indicator by using 1% Phenolphtalene solution. If the concrete is contaminated then the reaction pH indicator solution is colorless (transparent) and if the concrete is still in good condition (still alkaline with pH = 13.5), the reaction of the pH indicator solution is pink. (Triwulan, 2006).

As for the effect of fire on the mechanical properties of concrete, namely:
1. Changes in the color of the concrete
2. Spalling and crazing on concrete
3. Cracks on concrete
4. Effect of fire on the compressive strength

TYPES AND CLASSIFICATION OF CONCRETE DAMAGE AFTERFIRE

From observations made on various cases of damage to buildings after fire, can be grouped into:

1. *Slight damage*. Damage in the form of flaking on the outside plaster concrete and the surface color changes to black due to smoke that may be accompanied by cracks in plaster.
2. *Moderate damage*. This damage is the appearance of light cracks (a depth less than 1 mm) on the outside of the concrete in the form of stripes are narrow and not too long with the spread pattern.
3. *Severe damage*. Cracks that occur deeper own size and width, occur singly or in groups. If it occurs in the beam, sometimes accompanied by a deflection which can be seen by vision.
4. *Total damage*. Damage that occurs is such that concrete broken / chipped so that reinforcing steel can be seen, or even broken off and crushed concrete core

RESEARCH METHODOLOGY

Research Framework

Research framework to be conducted, it can be seen as described in Figure 1.

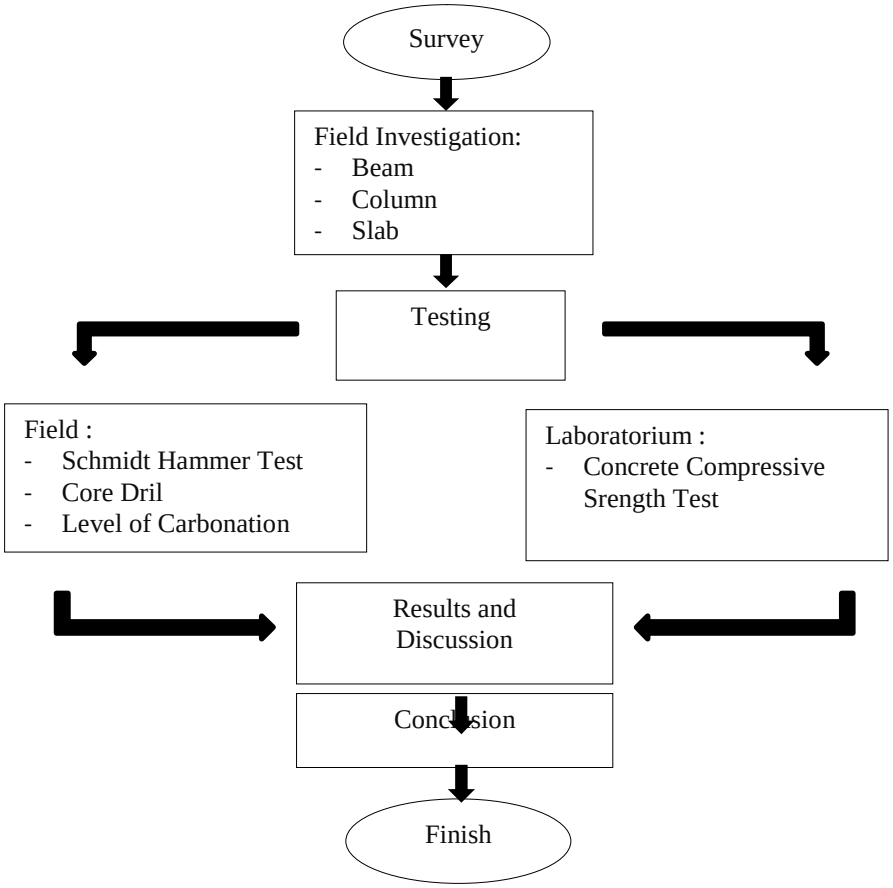

Fig. 1 Research Framework

Time and Location Research

This research was conducted at the Central Market Shop in Laiya St. and Tjokroaminoto St., Makassar, which consists of Block 1, Block 2, Block 4 and Block 5 particularly on the first floor of Central Market Shop and the Structures and Materials Laboratory of the Department of Civil, Engineering Faculty, Hasanuddin University. The research conducted for about two months

Procedure Research

a) Field Investigation

Field investigations in the form of visual inspection which is the beginning of a whole series of investigations carried out in the field. The aims are to estimate and to classify the type and level of damage to the three conditions, namely good, moderate and severe based on the visual condition.

b) Test of Carbonation

The level of carbonation in concrete can give an indication of how deep the concrete carbonation process has undergone after fire.

c) Schmidt Hammer Test

Schmidt hammer test conducted in each column at the field, as shown in Figure 3. This test reference based on SNI 03-4430-1997, Method of Testing Structural Elements Concrete Compressive Strength by using hammer Type N / NR

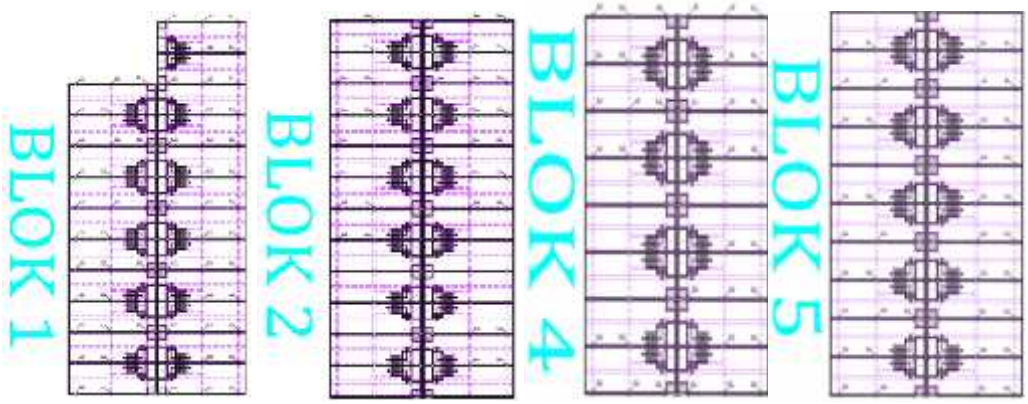

Fig. 2 Location of Schmidt Hammer Test

d) Concrete Compressive Strength Test (Core Drill Test)

In general, the results of semi-destructive tests is to determine the strength of the concrete in the field is still feasible or not. To perform the test of the strength of concrete is required two stages, namely the first stage is a sampling method refers to the SNI 03-2492-2002, and the second stage is a method of testing the strength of concrete which refers to the SNI 03-1974-1990.

RESULTS AND DISCUSSION

Analysis of Fire Temperature

Visual conditions on structural beams, columns, and slabs after fire showed physical and color changing. The physical changes in the structure such as changes in shape (deformation) in beams and slabs, flaking on the surface of the concrete (spalling), until the main reinforcement in the column can be seen due to structural failure. This is cause by significant changes in temperature at the time of the fire. It can be viewed in the following image:

Fig. 3 Collapse Condition of Structure

Fig. 4 Spalling Condition After fire

From the above pictures can be seen changes in the structure of columns, beams, and slabs at Central Market Shop after fire. Colors changing, deformation, the occurrence of spalling and cracking, and the appearance of reinforcement to the surface due to the high temperatures that occur.

With the change of color on the surface of the structure, it can be seen a change in the color of the concrete surface from gray to white wrinkles (buff) with an estimated minimum temperature <300°C until it reaches a maximum of 1000°C.

Besides analyzing the structure after fire condition visually, the highest temperature that occurs at the time of the fire can also be taken into account by the visual observation of the objects that are flammable or melt due to the heat contained in the fire location.

By viewing and analyzing materials flammable such as plastic, glass and copper were found in the field, it can be seen that occurs based on the estimated temperature melting point of each of the flammable objects.

Table 1. Evaluation of Materials on Fire

No.	Materials	Yield Point (°C)
1.	Glass	± 793
2.	Copper	± 660,4
3.	Plastic	± 112

Residual Compressive Strength of Concrete

The compressive strength of concrete residue obtained from Schmidt Hammer test only shows the residual strength on the concrete surface, which can be affected by surface flatness, moisture concrete, characteristic and type of coarse aggregate, the level of carbonization and the age of the concrete. The results from Schmidt Hammer evaluation as follows:

Table 2 Schmidt Hammer Evaluation

No.	Location	f'c max (kg/cm^2)	f'c min (kg/cm^2)
1.	Block 1	320,44	8,29
2.	Block 2	96,87	0,72
3.	Block 4	199,78	8,26
4.	Block 5	159,88	0,68

Besides Schmidt hammer test, residual compressive strength of concrete can also be known by core drill testing. From concrete compressive strength test on core drill samples so can be obtained the data of residual compressive strength after correction (F'cc) caused by drilling vertically and the reinforcement steel on core drill samples at the time of testing. From the results of the core drill tests show that there has been a degradation of concrete strength after fire and fire temperature is undistributed, causing variation on residual strength of concrete after fire.

Table 3 The Results of Concrete Compressive Strength Test (Coredrill Test)

Sample	Diameter (mm)	Length (mm)	Max. Load (kN)	f'c (N/mm2)	f'cc (N/mm2)
Sample 1	108	130	83,352	9,10	13,73
Sample 2	108	135	58,704	6,41	9,65
Sample 3	108	117	117	12,06	15,09
Sample 4	108	120	83.38	9,22	12,23
Sample 5	108	130	86,48	9,44	12,83
Sample 6	108	140	79,4	8,67	11,39
Sample 7	108	108	136,6	14,92	22,91
Sample 8	108	134	72,1	7,87	10,70
Sample 9	108	115	151,7	16,57	23,08
Sample 10	108	133	133	12,47	21,24

Test of Carbonation

Carbonation testing performed on concrete slab. By spraying a solution of Phenolpthelene against the concrete, it can be seen a large part of the concrete structure undergoing a process of carbonation due to fire. Carbonation process can be determined by the change in color.

From the carbonation test results in Table 4 showed that less than 1/3 of the slab thickness does not change color. No discoloration due to spraying a solution of Phenolpthelene it can be concluded that the third slab thickness or ± 30 mm was contaminated from the base slab.

The bottom of a concrete slab easier to have carbonation process than others due to a plate of work on the top floor, so it prevents carbonation process on the top plate, than at the bottom of the concrete slab that is carbonized after fire because it is easily exposed on the outside.

Table 4 Carbonation on Slab

Sample	Picture	Level of Carbonation (cm)	
		(Top)	(Bottom)
Sample 1		• 2,3 • 2,0 • 2,1 • 2,36 • 2,15	• 3,75 • 3,9 • 3,36 • 4,1 • 2,05

CONCLUSIONS

Based on data obtained from the test results of concrete residual strength after fire, it could be drawn as follow:

1. From some testing can be identified that the damage to the structural elements of the Central Market Shop after fire is undistributed. This is because the uniformity fire temperature occurred at the time of fire.
2. From carbonation test results that level of carbonation on slabs of Central Market Shop after fire is 1/3 part of the slab thickness or ± 30 mm was contaminated from the base slab. This is because the top floor there is a plate of work that prevents carbonation process on the top slab. So it's easier for carbonation occured at the bottom of the concrete slab because it is easily exposed from the outside
3. From some testing can be obtained residual mechanical strength concrete of Central Market Shop as follows:
   - The physical changes in the concrete structure is analyzed through visual observation, then the temperature that occurs at the time the fire was estimated at 900 ° C to 1000 ° C, which lasted for ± 6 hours. In addition, the presence of flammable materials such as glass on each floor (± 793 ° C), copper (± 660 ° C), and plastics (± 112 ° C), it can be estimated temperatures that occur by looking at its melting point. With the temperature difference occurs, it can be concluded that the temperature variation on fire.
   - The value of concrete compressive strength (f'c) column obtained by Schmidt Hammer test ie 32.044 to 9.687 MPa. While the compressive strength of concrete (f'c) on the plate is best obtained by Core Drill test namely from 16.57 to 6.41 MPa. Compressive strength uniformity is due to the effect of uneven temperature during the fire causing degradation of different concrete compressive strength.

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ANALYSIS OF OFFSHORE PLATFORMS LIFTING WITH FIXED PILE STRUCTURE TYPE (FIXED PLATFORM) BASED ON ASD89

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ABSTRACT : Platform construction GTS (*Gathering Testing Satellite*) is offshore platform construction with fix pile structure type/*fixed platform* functioning to support the mining of petroleum process exploitation. After construction fabrication process platform was moved to barges, then shipped to the installation site. Moving process is generally done by pull or push based on construction design determined when planning. But at the time of lifting equipment/cranes available in the work area then the moving process can be done by lifting so that moving activity can be implemented more quickly and the risk of work during moving process can be minimized. This analysis investigates moving process of GTS platform in a different way that is generally done to GTS platform types by lifting using problem formulation : is construction reinforcement required so the construction can be moved by lifting with analyzing and checking structure working stress that occurs due to construction moving process by lifting AISC code standard. The analysis was done by collecting project data in the form of construction type and working location, technical data, material specifications, structure details and construction model also loading applied at construction. All data will be processed and analyzed using SAP2000 structural analysis program that has been validated and done verification manually in order to obtain stress strain to determine which structural parts that require reinforcement. The analysis result showed that existing condition cannot be moved by lifting because the produced stress ratio is above maximum allowable value that is 0.950 (*AISC-ASD89*) . *Overstress* occurs on the member 295 and 324 with stress ratio value 0.977 and 0.955 so that it is required structural reinforcement. Box plate aplication at both members so that it produces stress ratio values 0.78 at the member 295 and stress ratio of 0.77 at the member 324. These results indicate that the construction have qualified structural reinforcement for being moved by lifting.

Keywords : *Lifting methode , Analisys,offshore Platform*

Background

Offshore construction is a construction that was built in the water location which serves to support the process of mining exploration or exploitation of petroleum and natural gas located shallow ocean waters and deep sea, in practice the offshore construction can function as exploration platforms, production platforms, distribution platforms and rigs accommodation.

The process of building the offshore construction work is different from the construction of buildings that were made on land. If the building construction process carried out in the same place while the offshore construction is done in different places at the installation location.

With consideration of economic issues, building site offshore cultivated as close as possible to the installation location. But there is also the location of manufacture and installation locations are island or even different countries. Such as the West Seno TLP platforms, primarily structures made in companies Hyundai Heavy Industries, South Korea, while the location of the installation is in the Makassar Strait, Indonesia.

The transfer process by pulling

Platform is transferred from land to the ship. This process is done slowly to avoid uncontrolled movements. This process involves many workers, tools and equipment as shown in Figure 2.1.

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Figure 1. Pull Transfer

Figure 2. Lifting Platform

Source: www.heavyliftspecialist.com/projects/

The transfer process by lifting

The process of removal by this method can be done with lifting equipment/crane which has a large lift capacity.

Formulation of the Problem

Once completed through the fabrication process, the construction of the GTS platform will be moved onto a barge in a generic way to do that is by pulling or pushing.

At the time of tool Lift / Crane available, construction removal process can be done by lifting way so that the process of moving is carried out with a relatively faster. In this study the problems that occur when the removal process is carried out by means of liftingare:

a. If the construction is strong enough to be moved by lifting ?
b. Is it required additional structures or structure reinforcement in certain parts so that the construction can be moved by lifting ?

Research Purposes

The intent and purpose of this paper is as follows:

a. To analyze the strength of the structure if the transfer process is done by lift.
b. To calculate the necessary reinforcement structure so that construction can be removed by lifting.

Benefits of Research

The benefits of this research are:

a. Reference for implementationof the transfer of offshore construction by lifting way for similar work.
b. Knowing construction part which requires structure reinforcement so that the construction can be moved by lifting.

Scope of Problem

a. Writing limited to a static analysis of offshore structures GTS types (Gathering Testing Satellite),
b. Calculated Analysis is stress analysis that occurs at construction structures during lifting process based on the standard American Institute of Steel Construction Allowable Stress Design 9th Edition (AISC-ASD89).
c. Research limited on the analysis of moving process of GTS platform with four lifting points as is done in the lifting process GTS platform in general.
d. Type of steel strap (sling) used were not analyzed. Assumed capacity sling used has met the required standardsstrength.
e. Stress analysis of the structure does not include stability ship analysis and lift/crane capacity.

RESEARCH METHODOLOGY

Approach and Types of Research

GTS (*Testing Gathering Satellite*) platform construction which is the main building of some oil wells were built around it and operates in the concession area owned Total Indonesie. The studymade the author is an analysis of the removal / lifting platforn by using software SAP2000 with data taken from the field.

Data Source

In the process, the author takes the field data on Total Contract Tunu project area in preparation classified into two types of data, namely primary data and secondary data.

Primary Data
From the field data obtained as follows:
Project General Data
Project Name: Tunu Development Project
Location: Muara Mahakam
Building Name: Testing Gathering Satellite (GTS)
Building function: Regulatory Pumping and distribution of oil wells to the processing center (Central Processing Unit)
Building Structure: Steel Construction
Steel Quality : Material Grade S355
Construction Description
Construction is a type of GTS (Gathering Testing Satellite) to be established at sea with water depth of 3 meters. The building is a type of production building which has function to collect oil from small wells built around it. GTS building just made one level consisting of a main structure as follows:
- 2 main beam of 1.45 meters high and 15 meters long.
- 3 main beam size of 1.45 meters and a length of 17 meters.
- Small Beams W21x62, W12x65, W12x30 and W8x15.

Building storey has outside size of 17 meters x 20.4 meters at a +8.45 meters height and accommodate appliances / equipment, electrical devices, instruments, piping, safety equipment / safety and Pipe rack. The building structure is supported by 4 pieces piles measuring 36 "which are piled to a depth of -40 meters below the ground surface with dimensions of 17 meters x 15 meters.
GTS construction parts
Construction platforms consists of the main parts as follows:
- The main deck, *GTS platform*main construction consisting of a laydown area, shelters are located at an elevation of + 8.450m. The main deck is also connected with the construction of the access road to wellheads, ESDV platform and access to the boat landing.
- Pipe Rack
  Pipe Rack is construction part ofpiping system that sustains the GTS platform. Pipe rack is located above the main deck and at an elevation of 9.700m - 14.900m.
- Open Close Drain Platform
  This section is located on the bottom side of the main deckhas function to locate and access the equipment Open & Close Drain and located at an elevation +6520 m.

Secondary Data
References relating to the subject matter in the form of planning design, supporting literature books, charts and tables.
Structure Geometry Model GTS
GTS Platform sized 17mx20.4m made at + 8,45m height above sea level. Construction is supported by 4 feet structure using pipe with 36-inch diameterthat will be sitting on a pile.
In the SAP2000 program construction models made at an elevation:
- Platform Open Close Drain + 5.970m, 6.520m +,
- Main Deck + 8.450m,
- Pipe Rack is built at an elevation of + 9.700m, + 11.650m, + 12.530m, and+ 14.900m.

Platform Construction
GTS platform construction consists of structure arrangement built at an altitude of 8.450m above sea level as modeled.
Construction Modelof Drain Open-Close Access Platform
Open Close Drain construction part is on the side of the platform with 18,4mx3,6m size. Drain Open-Close Access Platform construction is installed depends on the primary structure with structure frame arrangement.
Model Construction Pipe Rack
Pipe rack construction measuring 12m x 6,3 meters attached on the deck. This construction is modeled.

Loading
In the lifting process, the calculated load isself weight and equipment weightin empty conditions according to construction condition as shown in Appendix E page 167.
Structure Individual weight / Self weight (DL)
Structure self weight calculation is calculated automatically by the programbased on the material weight for profile that is used:

No	Code	Description	Weight Total
1	1100	Structure self weight	126.698

Equipment Load / Equipment (LL)

Equipment weight is calculated in empty condition classified into several types.

Construction Center Calculation

Calculation of construction weight and location of construction centeris calculated using moment static theory with SAP2000 program with pedestal flops modelsat joint 62 as shown Figure 3.

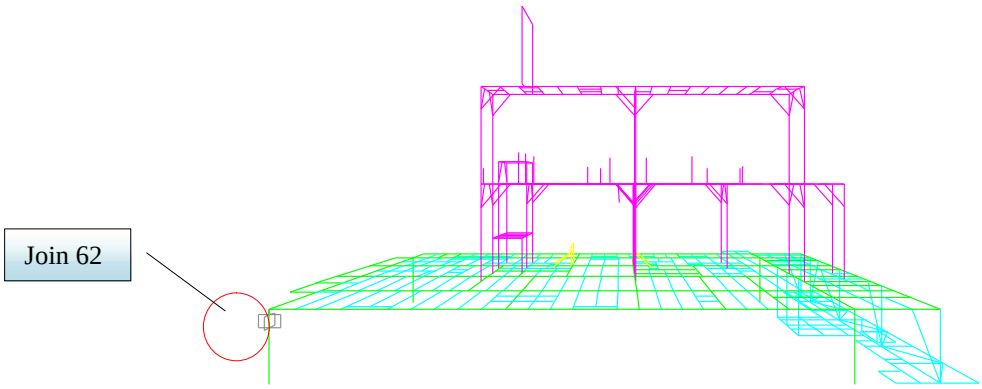


Figure 3. Moment Static Model

Construction center location is x and y coordinates as shown in Figure 4 is calculated from the value of combination moment division with the greatest value in the direction of x and y and support reactions generated by program calculation.

TABLE: Base Reactions

OutputCase Text	CaseType Text	GlobalFZ Kgf	GlobalMX Kgf-cm	GlobalMY Kgf-cm	GlobalMZ Kgf-cm
COMB4000	Combination	210909	175303964	-211935963	-498608

Coordinate x = GlobalMx/Global Fz = 1.753E+08 / 210909 = 831cm
Coordinate y = GlobalMy /Global Fz = 211935963 / 210909 = 1005cm
Coordinate z = GlobalMz /Global Fz = 498608 / 210909 = 2.6 cm
Z coordinate location is determined at 2150 cm coordinate with sling condition doest not intersect with construction as shown in Figure 4.

Lift Equipment Components

From the results it is obtained lifting equipment components as follows:

- Sling: Using four sling with diameter 13/8 "capacity of 83.5 tons with respectively length of 18.45m (sl.1), 16.25m (sl.2), 16:44 (sl.3) and 18.62 (sl.4).
- Shackle: Using four shackle sizes 21/2 "95 ton capacity.

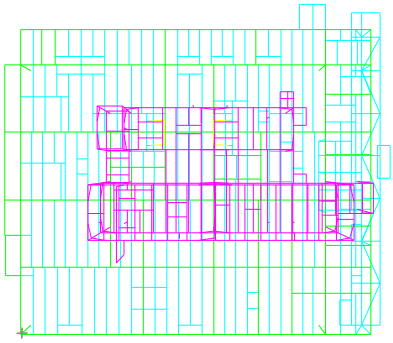
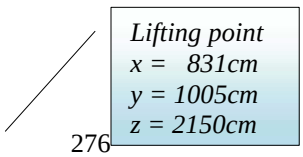


Figure 4. Construction Center Coordinate



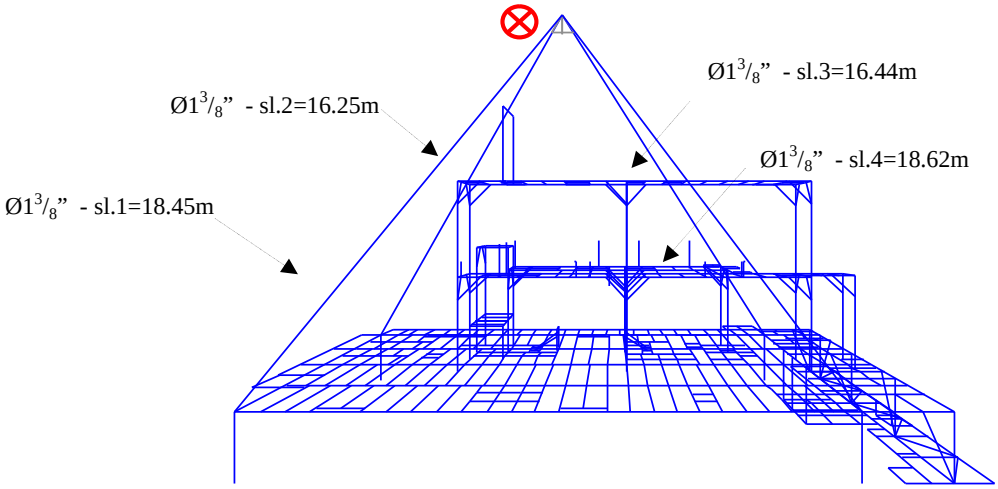


Figure 5. Lift point location

Structure Analysis

Load Resume

- Load case

  The structure was analyzed by grouping base load.

Load combination

The amount of force that caused by base load combination as a whole as shown in the table base reaction (page 169).

Table: Base Reactions, Part 1 of 3

OutputCase	CaseType	Global FX Kgf	Global FY Kgf	GlobalFZ Kgf	GlobalMX Kgf-cm	GlobalMY Kgf-cm	GlobalMZ Kgf-cm	Global X cm
COMB3000	Combination	1.67	17.75	126967.76	105455447.6	-127666457	-4460.31	0.000
COMB4000	Combination	69.27	-513.83	212247.48	176284568.7	-213071257	-505700.93	0.000
COMB5000	Combination	1.67	17.75	126967.76	105455447.6	-127666457	-4460.31	0.000
COMB6000	Combination	69.27	-513.83	212247.48	176284568.7	-213071257	-505700.93	0.000

Table: Base Reactions, Part 2 of 3

OutputCase	GlobalY cm	Global Z cm	XCentroidFX cm	YCentroidFX cm	ZCentroidFX cm	XCentroidFY cm	YCentroidFY cm	ZCentroidFY cm
COMB3000	0.000	0.000	-4561.101	5524.501	-11797346.0	269.525	1262.184	-576372.342
COMB4000	0.000	0.000	-392847.675	384130.301	-283011863	7556.963	30832.091	-13832666.2
COMB5000	0.000	0.000	-4561.101	5524.501	-11797346.0	269.525	1262.184	-576372.342
COMB6000	0.000	0.000	-392847.675	384130.301	-283011863	7556.963	30832.091	-13832666.2

Table: Base Reactions, Part 3 of 3

OutputCase	XCentroid FZ cm	YCentroid FZ cm	ZCentroidFZ cm
COMB3000	850.016	749.986	2358.453
COMB4000	21250.405	18749.643	58961.320
COMB5000	850.016	749.986	2358.453
COMB6000	21250.405	18749.643	58961.320

Table: Steel Design 1 - Summary Data - AISC-ASD89

Frame	DesignSect	DesignType	Status	Ratio	RatioType
295	W21x62	Beam	Overstress	0.977417	PMM
324	W21x62	Beam	Overstress	0.954880	PMM

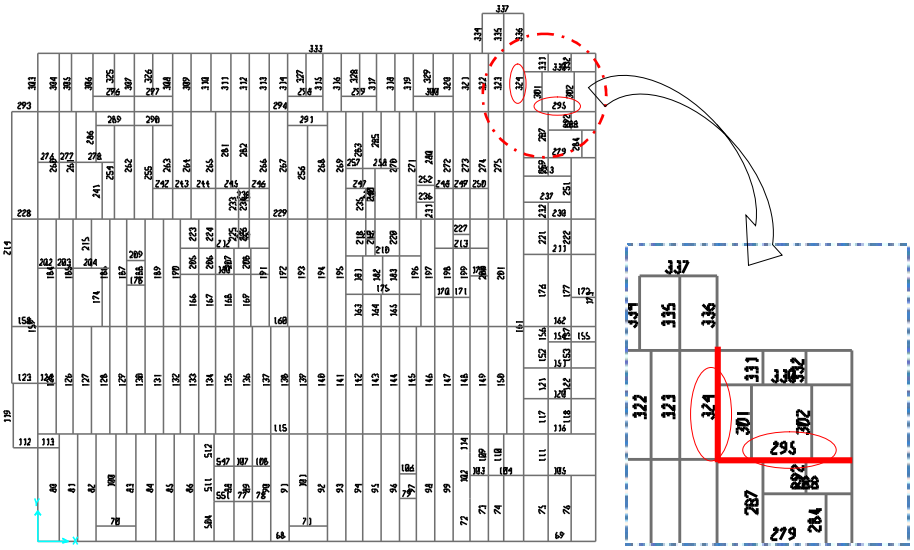


Figure 5. Beam Location 295 and 324

Structural modification

Because there are two beams that overstress it is necessary to modify the structure so that the construction can be moved by lifting. Modifications can be done in two ways: Replacing the profile size and adding structure reinforcementon weak beam as described below.

Replacing the profile beams 295 and 324

Profile replacement process is done by replacing the overstressed beam with larger size profile that is to replace beam 295 and 324 into size WPG 750 with consideration voltage ratio value that is produced does not exceed the maximum voltage structures (beam 69) with value of the voltage ratio 0.82.

This method is suitable for new construction so that the replacement of the profile can be determined before construction begins. Analysis result shows profil profile replacement produced voltage ratio valueless than the maximum ratio of 0.95.

Table: Steel Design 1 - Summary Data - AISC-ASD89

Frame	DesignSect	DesignType	Status	Ratio	RatioType
295	WPG 750	Beam	No Messages	**0.710240**	PMM
324	WPG 750	Beam	No Messages	**0.672812**	PMM

Profil cross-section 295 and 324 reinforcement
In addition to replacing profile sizemodifications can be done by increasing the strength of the structure in the required part by adding plat material to beam 184 and 324 on the left and right sides of the beam (box plate).
This selection can be performed on existing constructions in a condition where it has been determined that the platform should be moved to a different location by lifting.
To make reinforcement on the beam 295 and 324 (reinforcement plate) is done by adding plat material on both sides of the stem plate (box) as shown picture 6.

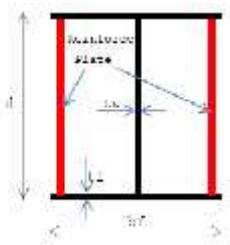


Figure 6. Addition of plat at beam profile

Calculation result as follows:
- Beam Reinforcement295 (page 239)
- Profile: W21 x 62
- Reinforce Thick plate: 1,2 cm (both sides)
- Unity Check: 0.78

- Unity Check (UC)

$$UC = \frac{Fb\text{-}y}{Fb\text{-}all} + \frac{Fb\text{-}z}{Fb\text{-}all}$$

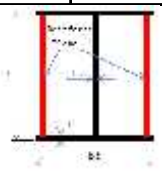
$$= \frac{1.18}{18.15} + \frac{12.97}{18.15}$$

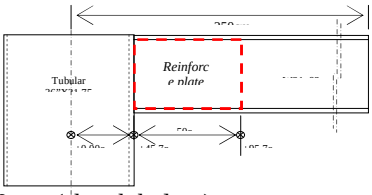
$$= 0.78 \text{ ---- } < 1.00 \text{ ------OK !!}$$

- *Unity Check* Las : 0.48

Tabel tegangan

Member	Tegangan (kN)	1.5*Fx/A (Mpa)	UC	Kondisi	Kesimpulan
295	300401	53.07	0.48	Tarik	Ok





- The length of reinforcement plate (reinforcement plate): 50 cm (sketch below):
- Shear stress:

$$fv = \frac{Vmj}{A} = \frac{132.4}{228.5} = 0.5792$$

$$Fv = 11.000$$

$$UC = \frac{fv}{Fv} = \frac{0.579}{11.000} = 0.05 \quad \text{---- } < 1.00 \text{ ------OK !!}$$

Sliding stress test results showed beam does not require stiffener plate (page 210).
• Beam 324 (page 241)
- Profile: W21 x 62

- Reinforce Thick plate: 1,2 cm (both sides)
- Unity Check: 0.77

- Unity Check (UC)

$$UC = \frac{Fb\text{-}y}{Fb\text{-}all} + \frac{Fb\text{-}z}{Fb\text{-}all}$$

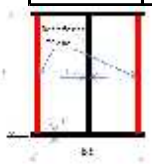
$$= \frac{1.56}{18.15} + \frac{12.45}{18.15}$$

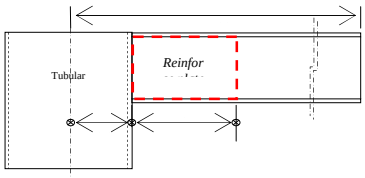
$$= 0.77 \quad \text{---- < 1.00 ------OK !!}$$

- *Unity Check* Las : 0.62

Tabel tegangan

Member	Tegangan (kN)	1.5*Fx/A (Mpa)	UC	Kondisi	Kesimpulan
324	388041	68.56	0.62	Tarik	Ok





- The length of reinforcement plate (reinforcement plate): 50 cm (sketch below)
- Shear stress :

$$fv = \frac{Vmj}{A} = \frac{119.337}{116} = 1.025393$$

$$Fv = 11.000$$

$$UC = \frac{fv}{Fv} = \frac{1.025}{11.000} = 0.09 \quad \text{---- < 1.00 ------OK !!}$$

Shear Stress test results showed beam does not require stiffener plate.

Conclusion

Based on the analysis, it can be taken conclusion as follows:

1. The GTS existing structure can not be removed by lifting without structure modification or reinforcement because the value of stress ratiooccurring on the beam 295 of 0977> 0.95 and the beam 324 of 0955> 0.95 (page 185).
2. The construction can be transferred by lifting with modification or reinforcement of the beams 295 and 324 to increase the capacity of structure strength, the modification can be done in two ways:
   a. Replacement beams profiles 295 and 324.
   If the GTS construction is new construction that is being planned, the replacement can be done on stem profiles 295 and 324 of the profile W21x62 to be WPG profile 750 with the results of the stress ratio at the beam 295 by 0.71 <0.95 and the beam 324 by 0.67 <0.95. (Pages 228 and 231).
   b. Beam 295 and 324 profile reinforcement
   beam 295 and 324 structures reinforcement is done if GST construction is existing building as follows:
   - To make structure reinforcement at the beam 295 by adding thick plates of 12 mmon both sides of the profile (box plate) of 500 mm length so that the stress ratio value can be lowered from 0.977 into 0.780 as described in Appendix J.
   - To make structure reinforcement at the beam 324 by adding thick plates of 12 mm on both sides of the profile (box plate) of 500 mm length so that the stress ratio value can be lowered from 0.955 into 0.77 as described in Appendix A (pages 239 and 241).

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BEHAVIOR OF THE PRECAST AND MONOLITH CONCRETE ON BEAM-COLUMN JOINTS UNDER CYCLIC LOADING

Mardewi Jamal [1],Herman Parung [2] and M. Wihardi Tjaronge [3] and Victor Sampebulu [4]

ABSTRACT: This research is an experimental study to investigate the behavior of the structure Beam Column Joint (BCJ) precast regarding ductility and strength , and then formulate a relationship ductility and strength. The test specimen consists of 4 models of beam-column joints , 3 of which are precast beam-column joints and 1 beam column joint monolith (BCJ - 1). The area is on a beam connection with a distance of 2 x hb (high beam) from the face of the column . Precast construction joints are distinguished in the area , with 3 different models , namely a straight line (BCJ - 2) , a jagged line (BCJ - 3) and the oblique connection (BCJ - 4) . The loading is cyclic lateral load representing earthquake load , testing the method with the type of gradual displacement control. Experimental test with the "full scale test" , the cross-sectional dimensions of the beam 200 mm x 250 mm and column 300 mm x 300 mm. From the analysis shows a hysteresis loop characteristic of the load-displacement relationship at the third joint approach precast shows a hysteresis loop model thin. The third connection of precast, models oblique connection (SBK-4) showed better behaviour of all specimens. The formulation produced in this study are a function of the parameters of the test, the data precast concrete is represented by SBK-4, so that the function of the relative strengths can be written as ε (μo) precast = ε (μo) x 1.073 monolith.

Keywords*: precast concrete, beam-column joint, cyclic loading and ductility*.

INTRODUCTION

In the construction of buildings, the panel meeting (continued) beams and columns are part-prone on earthquake resistant structure because of the specific nature of transmitting energy. When the structure was hit by an earthquake, there will be a very large shear forces at the panel meeting of the beam-column joints, especially when the onset of plastic beams on the face of the column. These shear forces can lead to the collapse of the core of the joint panel either because it exceeds the shear capacity or because of the collapse of the bond of reinforcement or as a result of both. Beam column connections are a vital component in a building structure. This part is very important because it has a major role in the transfer of force in an element to another in a structure. Sometimes the effect of lateral forces such as earthquakes, beam-column connections have experienced and horizontal shear force greater than the beam elements and the adjacent column.

Based on the author examines the connection beam column, by making it into a precast construction. Therefore precast construction has several advantages compared with conventional construction, and its use is increasing in recent years. So the object of this research is devoted to the connection beam precast columns, which were given cyclic loading. Strength and behavior will be compared with the conventional structure.

The research objective is assessing the influence of the model on the strength precast connection and Formulate a function of the parameters of the test on the connection of precast beam column to beam connection monolithic columns.

Results from this study are expected to an overview of anticipated maximum load and displacement that occurs in connection beam column in case of cyclic load so as not to exceed the required limits, add references to the behavior of precast concrete structures in receiving cyclic loading and by knowing the behavior of precast structures, especially in connection beam column other researchers can develop an interest in discussing it..

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BASIS THEORY

Precast Concrete

Precast concrete is concrete structure construction technology with the constituent components are printed in advance at a special place), sometimes these components are prepared and put together in advance, and then installed on site, so precast system will be different from the monolith construction mainly on the planning aspects of dependent or determined by the implementation of the manufacturing method, unification and installation, as well as technical also determined by the behavior of precast system in terms of how the connection between components join.

Some of the principles that are believed to benefit more from precast concrete technology, among others, related to the time, cost, quality, predictability, reliability, productivity, health, safety, environment, coordination, innovation, reusability, and can be relocated.

The principle of connection planning in precast elements can be classified in two categories, namely:

1. Strong Connection, when joints between precast elements still behave elastic during a strong earthquake, the connection system and proved to be theoretically and experimentally have the strength and hardness of the minimum equal to that of the structure of the connection equivalent concrete monolith.
2. Connection Ductile, when the connection inelastic deformation may occur, the system connection must be proven theoretically and experimental meet reliability requirements and stiffness of earthquake resistant structures.

Ductility

Ductility are used in the planning consists of a cross-μφ ductility and ductility μδ, namely:

$$\mu_\varphi = \frac{\varphi_u}{\varphi_y}, \qquad \mu_\delta = \frac{\delta_u}{\delta_y} \tag{1}$$

Where φu and φy is curvature boundaries and melting of the foregoing, while δu and δy is the deflection limits and the melting of a structure.

Plastic deflection capacity according to the European Convention for Constructional steelwork (ECCS) which divides the two forms ductility partial and full ductility.

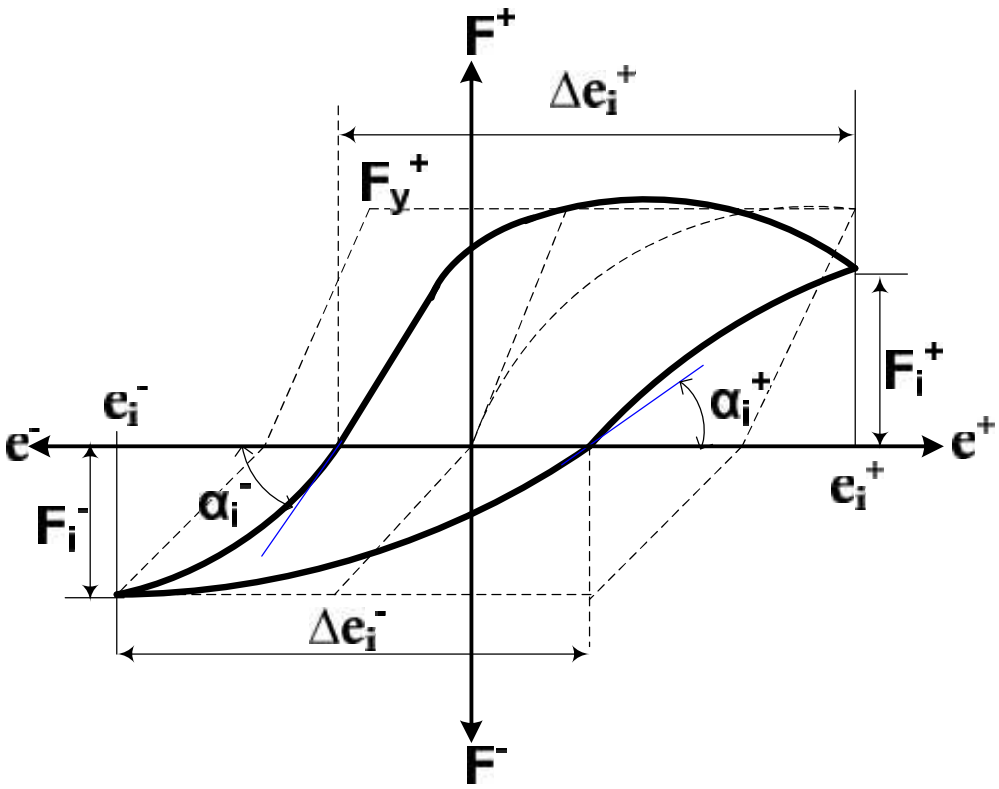


Fig 1. Load and deflection on the hysteresis curve

Partial and full ductility to positive and negative moments regions according to ECCS, namely:

$$\mu_{oi}^{+} = \frac{\mu_{e}^{+}}{\mu_{y}^{+}} \quad (2)$$

$$\mu_{oi}^{-} = \frac{\mu_{e}^{-}}{\mu_{y}^{-}} \quad (3)$$

$$\mu_{i}^{+} = \frac{\Delta\mu_{e}^{+}}{\mu_{y}^{+}} \quad (4)$$

$$\mu_{i}^{-} = \frac{\Delta\mu_{e}^{-}}{\mu_{y}^{-}} \quad (5)$$

Where μoi + and μoi- is partial ductility, μe + and μe- deflection limits, μi + and μi- is daktitilias partial, Δμe + and Δμe- is the total deflection limit, μy + and μy- is melting deflection.

Strength degradation

Beam strength is the ability of a beam in shouldering the burden. Strength degradation is presented in the form of the power ratio, ie the ratio between the maximum load (Pmaks.) With a load at the beginning of melting (Py).

$$\varepsilon^{+} = \frac{P_{maks}^{+}}{P_{y}^{+}} \quad (6)$$

$$\varepsilon^{-} = \frac{P_{maks}^{-}}{P_{y}^{-}} \quad (7)$$

Where ε + and ε- each is the ratio of the power of the positive and negative regions. Specific parameters ε + (μoi +), is the minimum value of ε + is evaluated from three cycles of each group. And ε- (μoi-), is the minimum value of ε- evaluated from three cycles of each group. Ability beams will carry the load maximum voltage that occurs when the voltage reaches its melting and will decline after experiencing sectional cracks in certain areas which would reduce the amount of cross-sectional moment of inertia.

RESEARCH METHODS

Research Sites

Research conducted in the laboratory Structure and Earthquake Engineering Department of Civil Engineering Faculty of Hasanuddin University.

Study Design

1. Type of Research
   In addition to literature review, this research will be experimental test is the implementation of the actual conditions in the field. Experimental studies will be conducted in accordance with the following stages of this research.
2. Stages Research
   a. Preparation Phase
   This phase includes a literature review on the basic theory and previous studies relating to, preparation of tools and materials, including mixtures mix design calculations for the planned concrete quality.
   b. Design Samples
   Beam and column sizes to be tested shown in the following table:

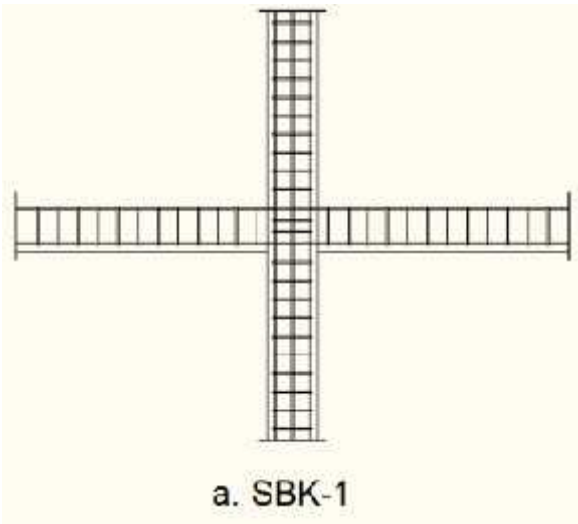
Table 1. Dimensions Beams and Columns

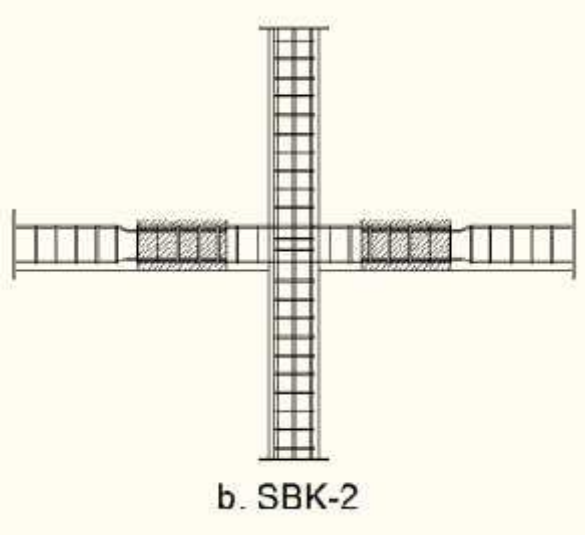
Description	Beam	Column
Width (B)	200 mm	300 mm
Height (H)	300 mm	300 mm
Concrete cover (d)	40 mm	40 mm
(f'c)	25 MPa	25 MPa
The length (L)	1500 mm	1180 mm

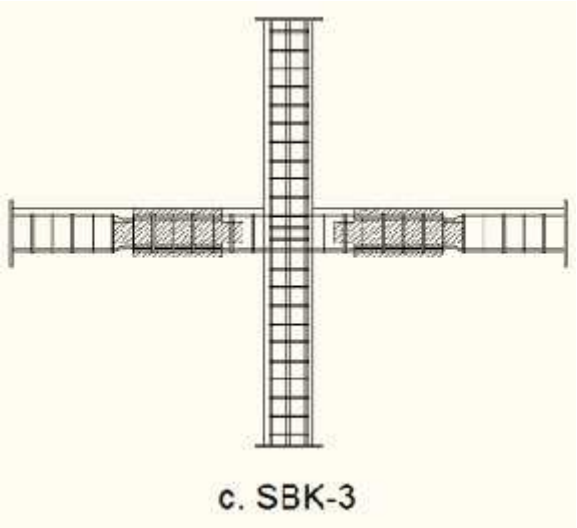
Details of the connection beam column models (SBK) can be seen in the following table:
Table 2. Specimen Number of Beam-Column Joints

No.	Shape	Code Testing	Number
1.	Connection Beam Column monolith	SBK-1	1
2.	SBK Precast straight connection	SBK-2	1
3.	SBK Precast gear connection	SBK-3	1
4.	SBK Precast oblique connection	SBK-4	1

The fourth model of beam-column connections can be seen in Figure 2.







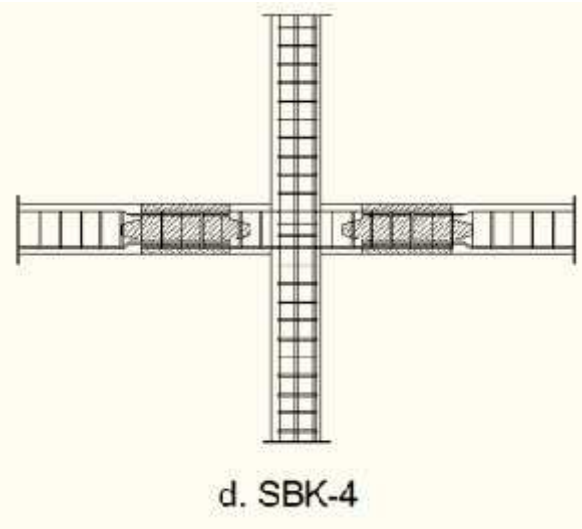


Fig 2. Model Connection Beam Column

Testing phase

Testing consists of :

1. Physical Testing of concrete material, in the form of testing compressive strength testing with the test object cylinder, split tensile strength testing and testing of elastic modulus and Poisson figures.

2. Physical Test steel material, this test includes testing the tensile strength of the reinforcing steel that will be used as plain reinforcement. This test was conducted to determine the yield stress of reinforcing steel.

3. Physical tests column beam connection, this test includes testing for precast structures and conventional full scale. Tests carried out after the 28-day-old concrete.

4. Testing Methods
   Tests conducted by the method of displacement control, with the gradual type of loading starting from the smallest displacement gradually up to the largest displacement can be achieved.

5. Model testing is as follows:

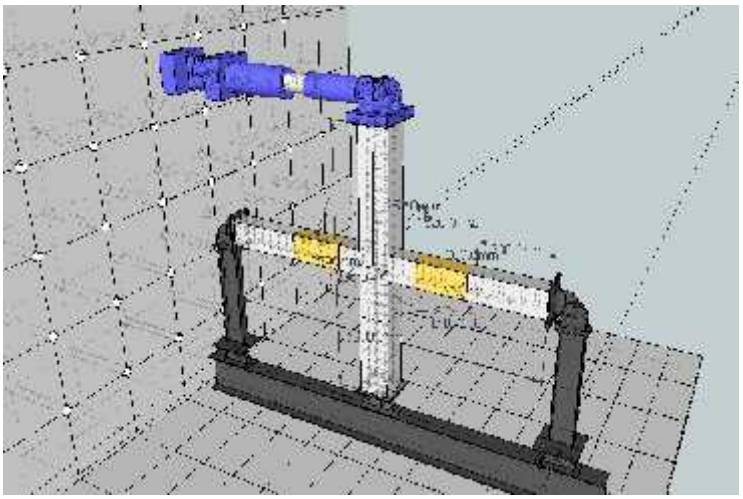
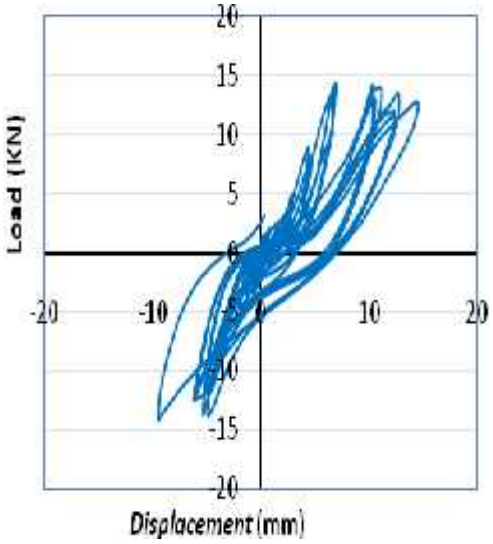


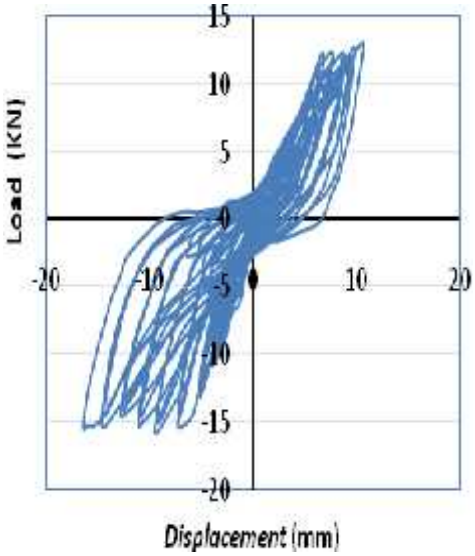
Fig 3. Test set-up

RESULTS AND DISCUSSION

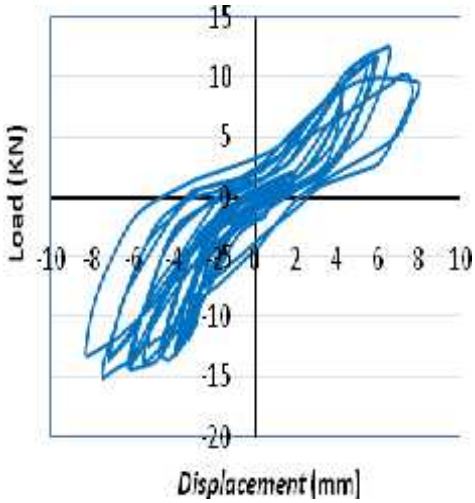
Based on test results are reviewed on a beam, obtained graph the relationship between load and displacement for each specimen as in Figure 4.


Load (KN)
Displacement (mm)
20
15
10
5
0
-5
-10
-15
-20
-20
-10
10
20

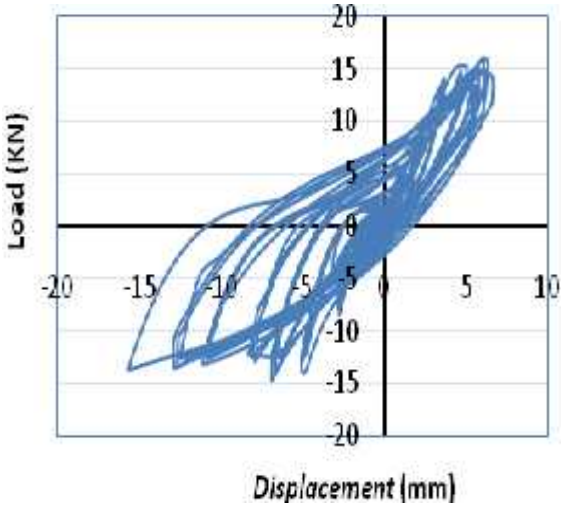
SBK-1


Load (KN)
Displacement (mm)
15
10
5
0
-5
-10
-15
-20
-20
-10
10
20

SBK-2



SBK-3



SBK-4

Fig 4. The load-displacement hysteresis curve on the beam

Based on the maximum load figure 4 largest known specimen occurred at SBK-4 which amounted to 16.139 KN.

After reviewing the behaviour of each specimen to the various parameters of ductility, strength, and patterns of breakdown, then do a comparison between the respective models Columns Precast Beams Connection, and then compared with Beam Column Connection monolith. Comparison of values of the parameters of the fourth test specimen can be seen in Table 3.

Table 3. Parameter comparison specimen

Speciment	Parameter							
	1			2			3	4
	Max Load (KN)			Max Displacement (cm)			Partial	Resistance
	(+)	(-)	Average	(+)	(-)	Average	Ductility	Ratio
SBK-1	14,280	14,200	14,240	12,340	6,015	9,178	2,928	1,515
SBK-2	13,015	15,665	14,340	9,265	14,015	**11,640**	2,189	1,602
SBK-3	12,590	15,115	13,853	6,400	5,230	5,815	1,180	1,103
SBK-4	16,319	14,644	**15,482**	5,605	12,525	9,065	**4,225**	**1,803**

Based on the parameters in Table 3 it can be seen that the test object SBK-4 has a maximum load average higher than other specimens. Likewise with the ductility and strength ratio is also higher than other precast test specimen and specimen monolith.

One of the factors that cause SBK-4 is better than the other specimens because the connections are angled so that when the grouting done all sides in the area filled with good connection and the possibility of a cavity in the corner of the very small, unlike the case with the SBK-3 have a connection to straight serrations can occur if the cavity in jagged angles less than perfect workmanship. While at SBK-2, which is a straight line, in the case of workability better than 3rd, and SBK SBK-4 but not the grouting part that goes into normal concrete area that does not cause the maximum force transfer.

According to the European Convention for Constructional steelwork (ECCS), there are several parameters to interpret (interpret) the results of cyclic testing. Within these parameters, the partial ductility is taken as a variable. The results are expressed in several functions, namely the continuous function is defined based on the number of limited value than the value of the full ductility and strength ratio. One function of these parameters is a function of relative strength (resistance Relative function): ε (μo). Where ε is the ratio of the strength and ductility μo is partial. Based on the analysis mentioned SBK-4 is the best precast connection, then further analysis to obtain a function of the parameters mentioned above using the data SBK-4, which is then referred to as $SBK_{precast}$. And beam-column connections monolith called $SBK_{monolith}$.

Partial ductility and strength ratio can be seen in Table 4.

Table 4. Values of partial ductility and strength ratio

cycle	SBKmonolith		SBKprecast	
	μo	ε	μo	ε
5	1.00	1.00	1.00	1.00
6	1.45	1.21	1.73	1.48
7	1.80	1.38	2.29	1.56
8	2.22	1.29	2.64	1.59
9	2.92	1.51	3.24	1.62
10			3.51	1.67
11			4.22	1.80

Based on Table 4 can be obtained which is a function of the relative strength of a polynomial function of order two. Graph function relative strength of each test specimen can be seen in Figure 6.

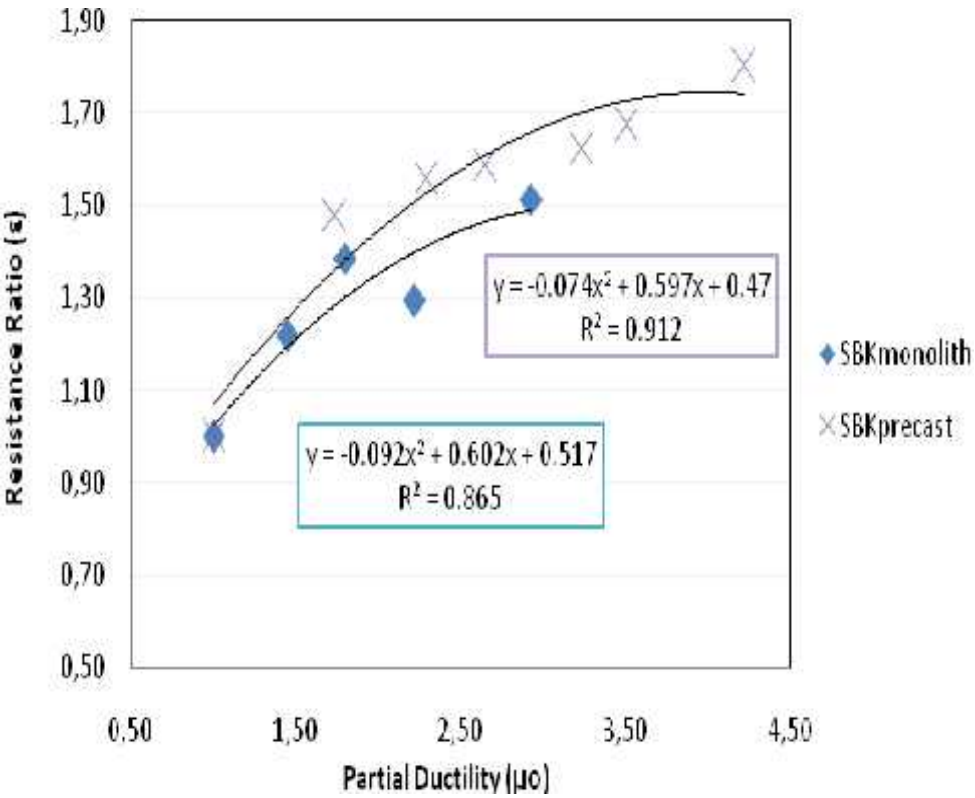


Figure 5. Curve function relative strength

Based on the function obtained in Figure 5, it can be written functions relative strength for each specimen by changing the symbol x as μo and y as ε, so we get the function ε (μo) for specimen as follows

- Specimens SBK $_{monolith}$:

  $\varepsilon = -0.092\ \mu_o^{\ 2} + 0.602\ \mu_o + 0.517, \qquad 1 \le \mu_o \le 2.928$ (8)

- Specimens SBK $_{precast}$:

  $\varepsilon = = -0.074\ \mu_o^{\ 2} + 0.597\ \mu_o + 0.547, \qquad 1 \le \mu_o \le 4.225$ (9)

Comparison of the function of the relative strength of the connection monolith precast connection can then be formulated as follows:

$\varepsilon\ (\mu_o)_{precast} = \varepsilon\ (\mu_o)_{monolith} \times 1.073$ (10)

where :

$\varepsilon\ (\mu_o)_{precast}$ = Relative resistance function precast connection

$\varepsilon\ (\mu_o)_{monolith}$ = Relative resistance function monolith connection

CONCLUSIONS

The hysteresis loops characteristic of precast connection shows curves similar to monolithic connection is increasing with the addition of load and displacement. Of the three precast connections, models oblique connection (SBK-4) showed better behavior of all specimens, because it has the ductility and strength higher than the other specimens. The formulation produced in this study is a function of the parameters of the test. Formulation is made based on the best connection, namely SBK-4. Function of the relative strength formulated with $\varepsilon\ (\mu o)_{precast} = \varepsilon\ (\mu o)_{monolith} \times 1.073$.

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PERFORMANCE ANALYSIS OF SHEAR WALL RC BUILDING DUE TO SEISMIC LOAD UPON BAR BUCKLING LIMIT STATES

M. Kimsan [1], L. Rahman[1] and M. R. Alhas [1]

ABSTRACT: This paper examines how deformation limit states on longitudinal bar buckling affects structural performance of shear wall reinforced concrete (RC) building as a result of seismic loading. Various preceding experiments and numerical simulation signify that bar buckling may occur following reversal from tensile strain during cyclic loading. In addition, recent finding with regard to design equations on deformation limit states for reinforcing bar buckling has provided benefit in evaluating structural performance by cogitating bar buckling effect. Nonlinear time history analysis is performed on 5, 10 and 20 stories of shear wall RC building by utilizing nine ground motions to capture existence of buckling on longitudinal bar. For that purpose, displacement in each step in each ground motion is prudently scrutinized, and thenceforth evaluated to deformation limit states curve. Furthermore, as preliminary study for future research, higher mode effect is also assessed. Constraint of two key parameters i.e. the ratio of transverse steel spacing to the longitudinal bar diameter and the ratio of the hoop diameter to the longitudinal bar diameter is provided to result proposed design to achieve performance demand of shear wall RC building.

Keywords: *Reinforced Concrete, Reinforcing Bar Buckling, Displacement, Ground Motion, Performance Demand*

INTRODUCTION

Structural design based on structural performance (direct displacement based design) has been developed in capturing more details of behaviour of structural elements due to strong earthquake. This conceivably eventuate from the need of further risk reduction which pertains to financial loss. Bar buckling effect is by way of illustration where any particular behaviour, especially nonlinear behaviour that might be cause of structural failure, necessitates to be evaluated.

At some periods, in terms of earthquake loading, buckling of reinforcement has not been gained serious attention from both researchers and engineers. However, Feng et.al (2014) demonstrated that bar buckling in column might occur due to cyclic-seismic loading following tensile strain. Cyclic movement results in tension and compression on longitudinal reinforcement and hence, this reversible movement might generate buckling.

Buckling of reinforcement obviously reduces strength capacity of structure, and consequently yield additional deformation. That the structure continues to receive axial load will exacerbate structural damage, particularly if the structure experiences further ground motion acceleration folowing previous earthquake loading. Therefore, reinforcement buckling design should take into account to be considered as the initial collapse of the structure.

Since structural column retains to be particularly prone to reinforcement buckling, shear wall building system will also be preserved to it. More energy dissipation of earthquake force in shear wall and column will engender buckling. In addition, distinct ground motion might yield different response of structure due to nonlinearity.

METHODOLOGY

Evaluation of reinforcement buckling in structure was conducted by modelling 4 (four) symmetrical shear wall building system with variance in number of storeys i.e 5, 10, 15 and 20. Standard of structural design procedure is in accordance with ASCE 7-10 and ACI 318-11. Non linear time history anaysis was performed with nine ground motions i.e. *El centro, Kobe, Cape Mondecino, Santa Felicia Dam, Livermore, Tar Tarzana, Aceh, Selat Sunda*, and *Padang*.

Column section works on this study were by constructing deformation limit state curve of elements based on equations proposed by Feng et.al (2014) which shows tensile and compressive strain relationship.

$$\varepsilon_T = \frac{-1\left(\varepsilon_C - \frac{0.0}{3\sqrt{\frac{s}{d_b}-1}}\right)}{\left(\frac{d_b}{h}-1\right)^2} \tag{1}$$

$$\varepsilon_T \geq -1.7\frac{s}{d_b}\sqrt{\frac{d_h}{d_b}}\varepsilon_L + 0.045\sqrt{\frac{s}{d_b}} \tag{2}$$

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$$\varepsilon_t \leq 0.09, \quad \text{if} \quad \frac{s}{d_b} \leq 3 \tag{3a}$$

$$\varepsilon_t \leq 0.06, \quad \text{if} \quad \frac{s}{d_b} \geq 4 \tag{3b}$$

$$\varepsilon_t \leq 0.09 - 0.03\left(\frac{s}{d_b} - 3\right) \quad \text{if} \quad 3 \leq \frac{s}{d_b} \leq 4 \tag{3c}$$

Where:
ε_t : tensile strain
ε_c : compressive strain
s : spacing (mm)
d_b : diameter of bar longitudinal (mm)
d_h : diameter of hoops

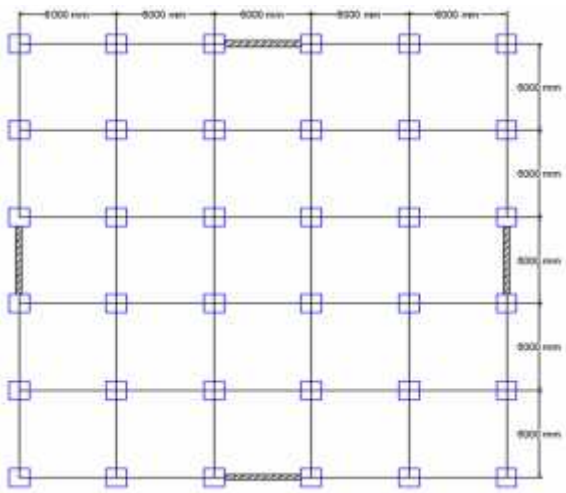

Fig. 1 Plan View of Structure

Each displacement was captured and then being superimposed on limit state curve. Moreover, minimum requirements for displacement regarding transversal reinforcement spacing to prevent buckling were also generated and might be utilized for design purpose of symmetrical shear wall building system.

STRAIN LIMIT CURVE FOR BUCKLING REINFORCEMENT

The boundary for reinforcement to be buckled or not is depended on the value of tensile and compressive strain on structural elements. Due to slight difference of transversal reinforcement spacing, The deformation limit state of all types of storeys is as shown on figure 2

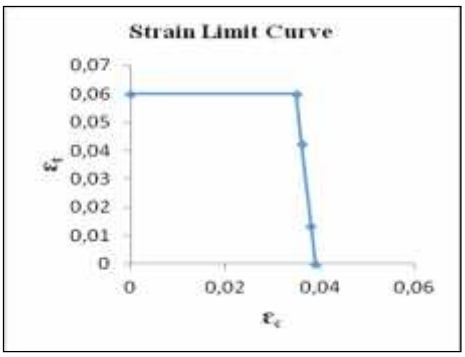

Fig. 2 Strain Limit State of Buckling of Reinforcement

Based on tensile strain and compressive strain on the structure, displacement limit of the structure is then determined subjected to bending displacement either in compression or tension condition by using capacity design equations of Direct displacement based design methods (Priestley et al. 2007) as shown on figure 3.

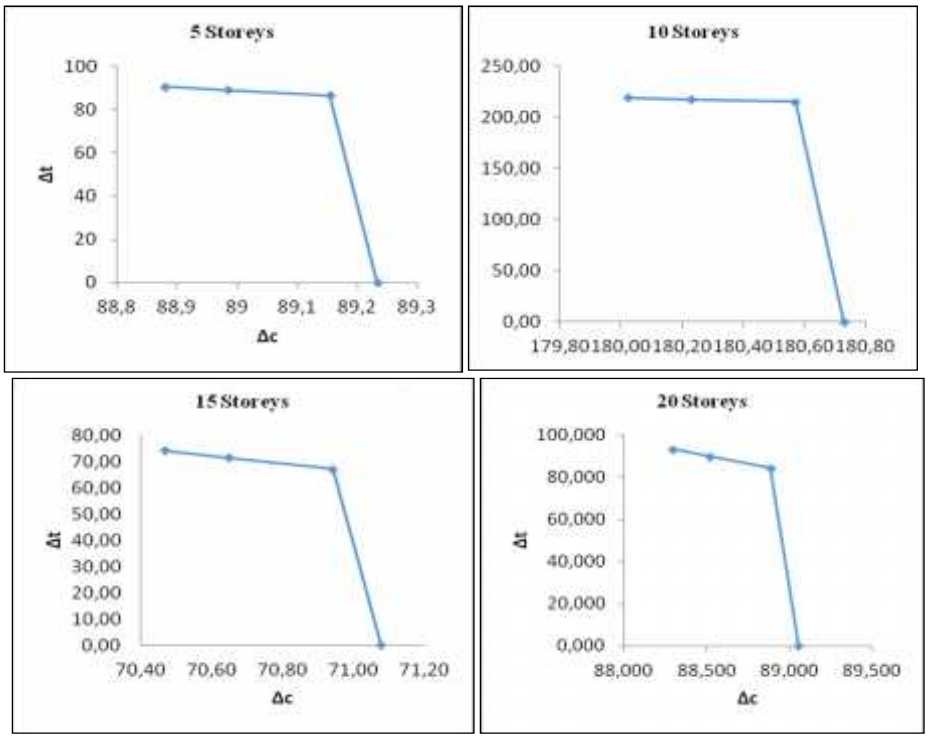

Fig. 3 Deformation Limit State of Buckling of Reinforcement

DISPLACEMENT ANALYSIS ON BUCKLING LIMIT STATE

As shown on figure 4, roof displacement following nonlinear time history analysis of each storey is presented, superimposed by deformation limit state curve. And pushover analyisis is also performed for comparison purpose.

As can be seen on figure 3, buckling does not occur on 5 storeys and 10 storeys building, regardless the type of ground motions and pushover analysis. On the other hand, buckling occurs on 15 and 20 storeys for *Kobe*, *Livermore* and *Padang* ground motions.

VARIATION ON LONGITUDINAL REINFORCEMENT

For further analysis, proper reinforcement dimension will possibly prevent elements from buckling. For each number of storey, specific minimum and maximum space of reinforcement will yield different deformation limit states as shown on table 1-4. It can be analyzed that transversal reinforcement spacing determines the capacity of element on buckling. Should the space be more adjacent to each other, the displacement limit state will be greater. Conversely, if the space is looser, then the capacity will decrease and buckling will be more likely to occur.

In addition, diameter of longitudinal reinforcement also shares similar effect on buckling possibility. The bigger dimension will absolutely provide smaller risk for buckling to happen during earthquake. Careful consideration need to be carried out in terms of optimizing the diameter of longitudinal reinforcement and space of transversal reinforcement due to buckling of reinforcement. Selection of bars dimension will affect the number of bars required. Larger bar diameter indicates larger space of longitudinal reinforcements and smaller number of bars. On the contrary, smaller dimension of bars will increase the number of longitudinal reinforcement and decrease their space. Optimum dimension might avoid buckling behavior of longitudinal reinforcement.

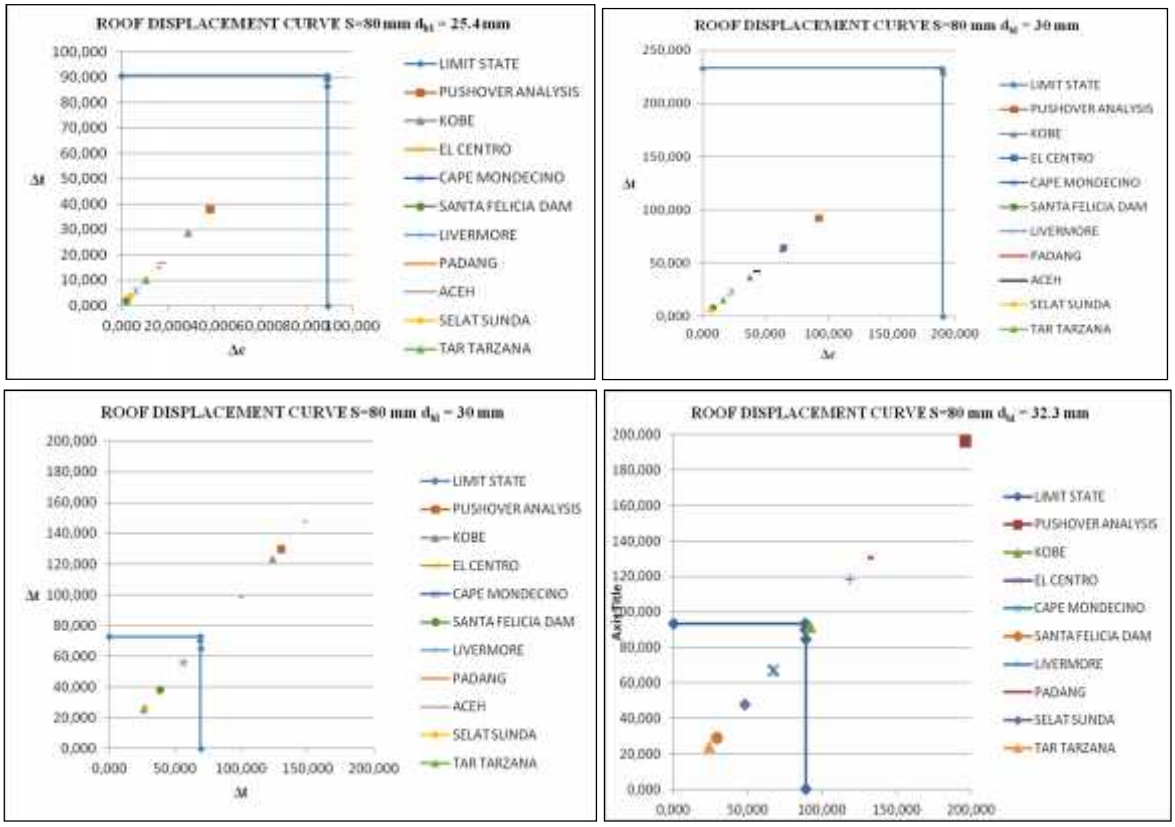

Fig. 4 Actual Roof Displacements on Buckling Limit State

Table 1 *Roof Displacement Limit Curve* for 5 storeys

Transversal reinforcement spacing (S) = 80 mm		Transversal reinforcement spacing (S) = 52.17 mm	
d_{bl} = 25.4 mm		d_{bl} = 15.9 mm	
Δ_c	Δ_t	Δ_c	Δ_t
mm	mm	mm	mm
88.879	90.709	105.463	100.688
88.984	89.143	105.573	99.052
89.154	86.586	105.751	96.381
89.233	0.000	105.833	0.000

Table 2 *Roof Displacement Limit Curve* of 10 Storeys

Transversal reinforcement spacing (S) = 80 mm		Transversal reinforcement spacing (S) = 52.96 mm	
d_{bl} = 25.4 mm		d_{bl} = 32.3 mm	
Δ_c	Δ_t	Δ_c	Δ_t
mm	mm	mm	Mm
180.022	219.300	181.962	219.300
100.231	217.714	182.171	217.714
180.572	215.126	182.512	215.126
180.730	0.000	182.670	0.000

Table 3 *Roof Displacement Limit Curve* Of 15 Storeys

Transversal reinforcement spacing (S) = 80 mm		Transversal reinforcement spacing (S) = 54.32 mm	
d_{bl} = 30 mm		d_{bl} = 30 mm	
Δ_c	Δ_t	Δ_c	Δ_t
mm	mm	mm	Mm
70.467	74.300	90.875	93.448
70.648	71.590	91.087	90.281
70.943	67.164	91.431	85.409
71.080	0.000	91.591	0.000

Table 4 *Roof Displacement Limit Curve* of 20 Storeys

Transverse reinforcement spacing (S) = 80 mm		Transverse reinforcement spacing (S) = 55.12 mm	
d_{bl} = 32.3 mm		d_{bl} = 32.3 mm	
Δ_c	Δ_t	Δ_c	Δ_t
mm	mm	mm	mm
88.294	93.272	90.274	93.272
88.512	89.907	90.498	89.907
88.884	84.412	90.865	84.412
89.054	0.000	91.035	0.000

CONCLUSIONS

Subsequent to previous analysis, some conclusion might be pointed out i.e.:

1) For structure in which first mode is dominant (5 and 10 storeys) and experiencing nine ground motions, longitudinal bar buckling might not occur. Therefore, design procedure as implied in code might be applied.
2) For higher mode analysis of structure where first mode is no longer dominant, buckling might occur on particular ground motions. Nonlinear time history analysis should be conducted to capture bar buckling.
3) Pushover analysis is not appropriate method for determining internal forces due to earthquake, especially for higher mode effect of structure and capture buckling behaviour. It is excessively conservative for design purpose.
4) Most advantageous approach for preventing buckling is by selecting precise combination of longitudinal bar diameter and its spacing. Larger or smaller diameter does not necessarily mean that it will be apposite to design.

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THE EFFECT OF FIRE DURATION ON MECHANICAL PROPERTIES OF CONCRETE

I.A. Ahmad[1]

ABSTRACT: Concrete structures that have burned can be used again along the constituent material still has enough strength to sustain the external load. An effort to assess the residual strength is a complicated matter and is determined by various parameters. The purpose of this study is to determine the reduction on mechanical properties of concrete due to fire at various durations. Cube concrete specimens that were 15 cm x 15 cm x 15 cm prepared for compression tests. Meanwhile prism specimens that were 15 cm x 15 cm x 60 cm prepared for flexural tests. Specimens were made with the composition according to the mix design for compressive strength of 20 MPa, 30 MPa, and 35 MPa. At the age of concrete at 90 days, the specimen burned with varying heating times ((3 hours, 6 hours and 9 hours) and temperatures (400°C and 600°C). Based on the research results, it was concluded that due to heating, the higher the quality of the concrete, the smaller the reduction in compressive strength and flexural strength were occurred.

Keywords: *Fire, duration, compression, flexural, concrete.*

INTRODUCTION

Fire in Indonesia is still the most common disaster. Based on data from the Fire Department of Makassar, at least 183 cases of fires have occurred during the period of 10 months in 2014. It also said that starting in July 2014, the number of fires increased by 20%. The Fire Service and Disaster Management of Jakarta recorded the fires during January to April 2014 as many as 280 cases. Fire disaster management generally been slow due to various factors. One of them can be caused by the absence of guidance in determining quickly and accurately about the level of damage caused to a building that has caught fire.

Challenge faced by the construction experts is how to assess the residual strength of the building after the fire. To answer this challenge has been scientifically developed various methods of assessment, both non-destructive and destructive assessments. This method is also known as Forensic Engineering, aims to accurately estimate the high temperatures experienced to the building. This may help people in dealing this fire problem.

The strength estimation method consists of various activities in the field and in the laboratory. The data obtained will be used to reanalyze the residual strength of the building. Results of this analysis will give a conclusion on whether the building still be used, must be repaired or even not be reused.

Many studies have been carried out to investigate the effect of the elevated temperature on different concrete specimens, for example studying the effects of concrete compressive strength and cover thickness on the structural behavior of reinforced concrete (RC) beams under fire (Choi and Shin 2011), the multiaxial mechanical properties of High Performance Concrete structure enduring high temperatures, and provides the experimental and theory foundations (testing data and correlated formula) for fire-resistant structural design, and for structural safety assessment and maintenance after fire (Zhen-jun and Yu-pu 2010). Murtiadi (2013) reviewed about The Indonesian Standard for concrete building subjected to fire. Development of fire engineering method is required from the study of whole concrete structures rather than individual member behavior. Fares et al. (2010) concerned to study the behaviour of Self Consolidating Concrete subjected to high temperature. The physical and chemical properties and the microstructure after exposure to 150, 300, 450 and 600 °C were studied.

Additionally, spalling, thermal, and hydrous behavior of ordinary and high-strength concrete subjected to elevated temperature were studied by Kanema et al. (2011). Specimens were subjected to various heating-cooling cycles. The effect of the thermocouple in the concrete, the temperature difference (i.e., the thermal gradient), the heating rate, the specimens size, the weight loss, and the concrete density were studied.

When concrete is exposed to temperature changes, its durability is reduced because of the decomposition of cement metrics generation of cracks within its structure as its component materials undergo different volumetric changes. Coarse aggregates play an important role in such behavior of concrete. Yoon et al. (2015) evaluated the influence of coarse aggregates on the fire resistance performance of a concrete structure by conducting a fire experiment under loading on two types of concrete, one with a granite-based coarse aggregate (NWC: normal weight concrete) and the other consisting of a clay-ash lightweight aggregate (LWC: lightweight concrete). LWC displayed a higher residual compressive strength than NWC under thermal load condition. NWC suffered from a large number of cracks at its interior at high temperatures, while the interior of the LWC demonstrated fewer cracks because of the voids in its

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interior to the mitigation of thermal expansion stress. Abdulkareem et al. (2015) studied about the effect of lightweight aggregate size and grading on the residual strength of lightweight geopolymer concrete exposed to elevated temperature. The results of this study suggested that the lightweight aggregate size and grading are totally applicable to high temperature environment application when the high fire-resistance is desired.

According to the statistics from China, 80% and 95% of the building fires on the ground are extinguished within 1 h and 2 h, respectively, i.e., the total duration of most fires is no more than 2 h. However, some serious fires occurring in underground spaces (e.g., in a building, tunnel, or cave) are sustained more than 10 h, or even several days (see Section 14.2.1), usually because of difficulties in extinguishing the fire (Guo and Shi 2011). Therefore, the object of this study is duration of fire in a building. The purpose of this study was to determine the effect of duration of fire against the residual strength of a building. If the overview is already known, it is expected to provide a solution to the experts for analyzing the strength of building after fire.

METHODOLOGY

Materials

In this study, natural siliceous river sand (fineness modulus of 2.6 and specific gravity of 2.42) is used as fine aggregate, crushed granite (maximum nominal size 20 mm and specific gravity of 2.3) as coarse aggregate. The Portland composite cement (PCC) was used as the binder material in this study, to product 20MPa (A), 30 MPa (B) and 35 MPa (C) of strength concrete.

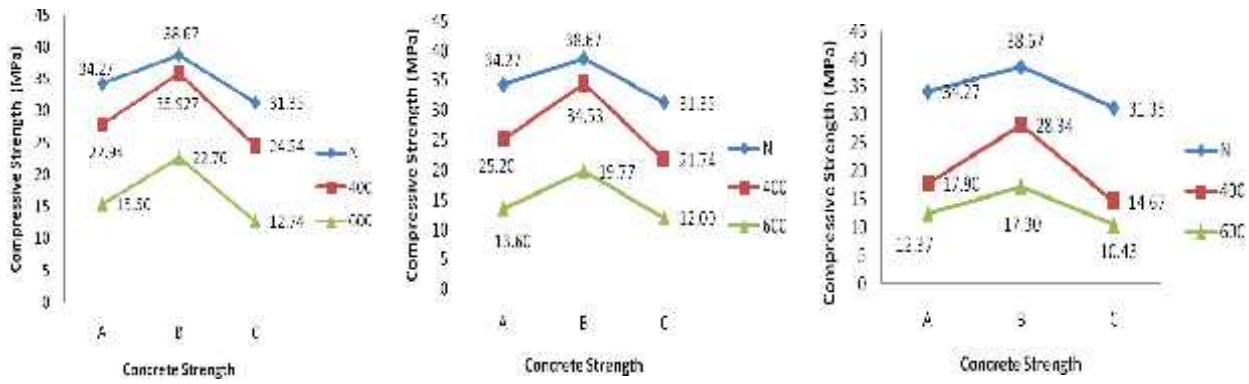
Methods

Cube and prism concrete specimens that were 150 mm x 150 mm x 150 mm and 150 mm x 150 mm x 600 mm respectively, were prepared for tests. Both specimens were divided into three groups. First group which was not burned was called control specimen. Second and third group were exposed to fire in oven respectively at temperature of 400°C and 600°C. The second and third groups in this study were designed to have different fire duration of three hours, 6 hours, and 9 hours.

Test carried out on concrete specimens after 28 days of curing. The first test was conducted to cube specimens to find compressive strength. The second test was conducted to prism specimens to find flexural strength. Both tests were performed at the age of 90 days on specimens.

RESULTS AND DISCUSSIONS

Compressive Strength



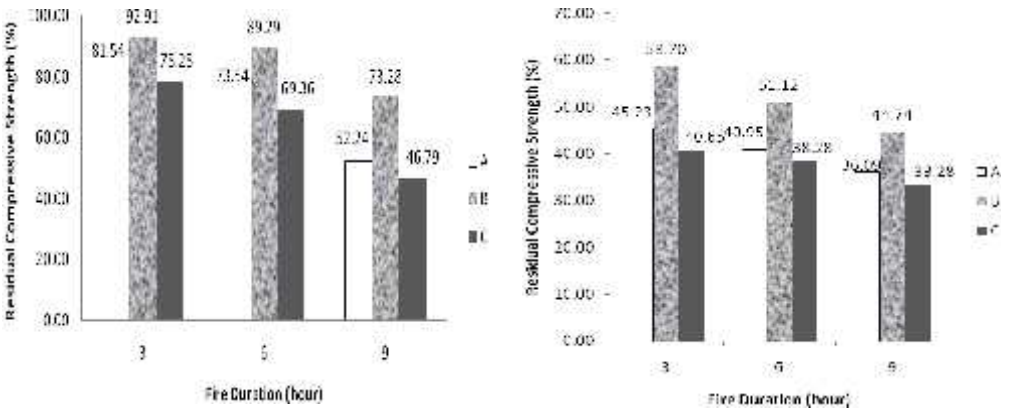
(a) after 3 hours heating (b) after 6 hours heating (c) after 9 hours heating

Fig. 1 Compressive Strength

The effect of temperature on compressive strength for three concrete strengths (A, B, and C) and variations in fire duration can be seen in Fig. 1(a), Fig. 1(b), and Fig. 1(c). The three graphs show that the reduction in compressive strength for A and C have the same tendency, but for B is not the same.

Specimen A and C, which were heated for 3 hours and 6 hours, had almost the same reduction in compressive strength which occurs between 400°C and 600°C. While for the specimen B, the magnitude of the decline is much different than the other two specimens. Decrease of compressive strength at temperatures 400°C was less than that at temperatures 600°C. While on fire duration of 9 hours, the tendency reversed, for specimen B, decrease of the

compressive strength at temperatures between 400°C and 600°C were almost the same. While the reduction of compressive strength on specimen A and C, at temperatures between 400°C and 600°C was different.



(a) exposed to temperature 400°C (b) exposed to temperature 600°C

Fig. 2 Residual of Compressive Strength

Fig. 2(a) shows the residual compressive strength of concrete for each quality with 3 variations due to heating at a temperature of 400°C. The heating in the oven for 3 hours caused residual compressive strength of specimen A, specimen B and specimen C, respectively for 81.54%, 92.91%, and 78.28%. After heating for 6 hours, residual compressive strength of specimen A, specimen B and specimen C, respectively for 73.54%, 89.29%, and 69.36%. Heating for 9 hours caused even greater reduction in compressive strength so that the residual of the specimen A, specimen B and specimen C, respectively for 52.24%, 73.28%, and 46.79%.

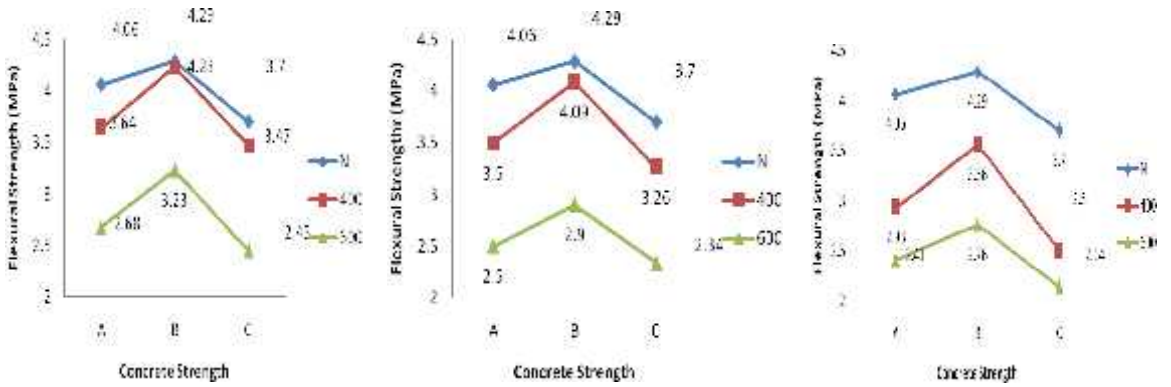
Residual compressive strength of concrete for each quality with 3 variations due to heating at a temperature of 600°C is presented in Fig. 2(b). Residual compressive strength of concrete was very low which under 60%. The heating in the oven for 3 hours caused residual compressive strength of specimen A, specimen B and specimen C, respectively for 45.23%, 58.70%, and 40.65%. After heating for 6 hours, residual compressive strength of specimen A, specimen B and specimen C decreased to 40.95%, 51.12%, and 38.28%. Heating for 9 hours, caused an even greater reduction in compressive strength so their residual of specimen A, specimen B and specimen C, respectively were 36.09%, 44.74%, and 33.28%.

Furthermore, it is concluded that for the specimen A and specimen C, heating at a temperature of 400°C to 6 hours, the residual compressive strength was still worth (above 80%), so the building does not need to be dismantled, but can be renovated or strengthened. But at a temperature of 600°C, the residual compressive strength was no longer feasible, so that the building cannot be used. For specimen B, heating at a temperature of 400°C to 9 hours, the residual compressive strength was still feasible, so that the building can be renovated or strengthened. But at a temperature of 600°C, the residual compressive strength was no longer feasible as in specimen A and specimen C, so the building cannot be used.

Flexural Strength

The effect of high temperature on flexural strength for three concrete strengths (A, B, and C) and variations in fire duration can be seen in Fig. 3(a), Fig. 3(b), and Fig. 3(c). The tendency of decrease in flexural strength is almost similar to the compressive strength.

Specimen A and C, which were heated for 3 hours and 6 hours, had almost the same reduction in flexural strength which occurs between 400°C and 600°C. While for the specimen B, the magnitude of the decline is much different than the other two specimens. Decrease of flexural strength at temperatures 400°C was less than that at temperatures 600°C. While on fire duration of 9 hours, the tendency reversed, for specimen B, decrease of the flexural strength at temperatures between 400°C and 600°C were almost the same. While the reduction of flexural strength on specimen A and C, at temperatures between 400°C and 600°C was different.



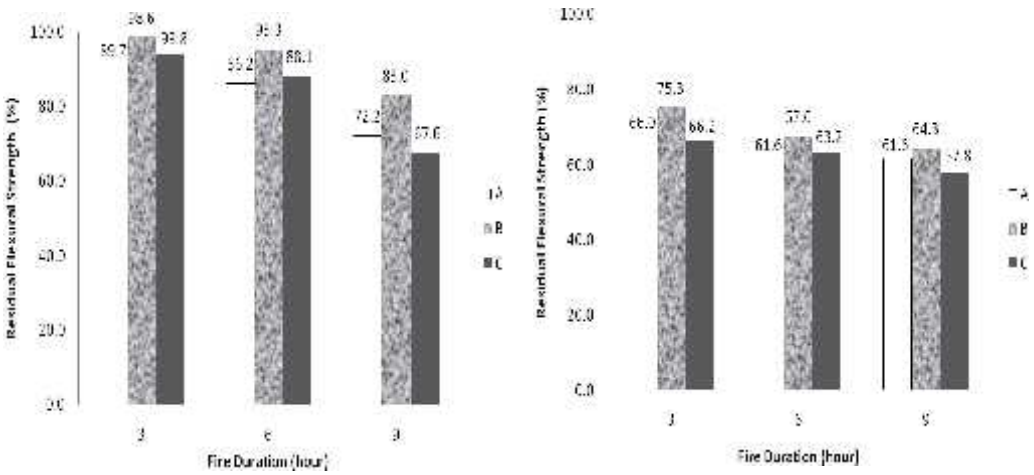
(a) after 3 hours heating (b) after 6 hours heating (c) after 9 hours heating

Fig. 3 Flexural Strength

The relationships between fire duration and residual of flexural strength exposed to temperature of 400°C and 600°C are shown in Fig. 4(a) and Fig. 4(b), respectively. As shown in Fig. 4(a), the heating in the oven for 3 hours caused residual flexural strength of specimen A, specimen B and specimen C, respectively for 89,7%, 98,6%, and 93,8%. After heating for 6 hours, the residual flexural strength of specimen A, specimen B and specimen C, respectively for 86,2%, 95,3%, and 88,1%. Heating for 9 hours caused even greater reduction in flexural strength so that the residual of the specimen A, specimen B and specimen C, respectively for 72,2%, 83%, and 67,6%.

The residual of flexural strength concrete for each quality with 3 variations due to heating at a temperature of 600°C is presented in Fig. 4(b). The heating in the oven for 3 hours caused the residual of compressive strength of specimen A, specimen B and specimen C, respectively for 66,0%, 75,3%, and 66,2%. After heating for 6 hours, residual compressive strength of specimen A, specimen B and specimen C decreased to 61,6%, 67,6%, and 63,2%. Heating for 9 hours, caused an even greater reduction in compressive strength so their residual of specimen A, specimen B and specimen C, respectively were 61,6%, 64,3%, and 57,8%.

It is concluded that for the specimen A and specimen C, heating at a temperature of 400°C to 6 hours, the residual of flexural strength was still worth (above 80%), so the building does not need to be dismantled, but can be renovated or strengthened. But at a temperature of 600°C, the residual of flexural strength was no longer feasible, so that the building cannot be used. For specimen B, heating at a temperature of 400°C to 9 hours, the residual of flexural strength was still feasible, so that the building can be renovated or strengthened. But at a temperature of 600°C, the residual of flexural strength was no longer feasible as of specimen A and specimen C, so the building cannot be used.



(a) exposed to temperature 400°C (b) exposed to temperature 600°C

Fig. 4 Residual of Flexural Strength

CONCLUSION

Based on this study it appears that the longer of the fire, the smaller the residual strength of concrete (compressive strength and flexural strength). But if viewed from the quality of concrete, the decline in the strength of 30 MPa is smaller than that of 25 MPa and 35 Mpa. Samples of 30 MPa still persist with residual of strength above 80% although they had burned at 400°C for 6 hours.

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TESTING MULTI LAYER ON THE ROAD PAVEMENT

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ABSTRACT: multi- layer has been widely used broad spectrum of various discipline is science , specifically on the pavement is used as a medium that provides theoretical information about the response of pavement layers . Multi layer as a concept used as a planning approach adds a thick layer of pavement as the main principle of the mechanistic method , and become part of the artificial neural network method , also used as part of a multi-layer on the analysis of pavement komputansi system with specific loading models . In this paper a multi layer as a form of testing on the road pavement layers that focus on literature review ,. A literature review using the internalization approach - adduct .

Keywords: *Multi layer internalization approach adduct approach*

INTRODUCTION

Indonesia, like an emerald in the equator, the country is endowed natural resources preconditions such as aggregate, cement and bitumen which should be fully utilized as one solution to the premature destruction of the pavement (under design).

The phenomenon of early damage (under design) in flexible pavement construction since last two decades proves the shortcomings of conventional design and durability of the asphalt mix design results are vulnerable to structural damage, such as plastic deformation and material fatigue crack (Quintus et al, 1991, Hunter, 1994) in Wahyudi Mudji 2011.

Historically Suaryana N (2012) wrote that the effort to eliminate premature damage in Indonesia was started in the 70s, where the year started to use materials Asphalt Concrete (AC), but the use of air conditioning with a minimum thickness of pavement damage has not been established in the form of cracks -retak. Furthermore, in the mid-80s, introduced Hot Roller Sheet (HRS) which has a high flexibility, a thicker asphalt membrane. Crack problems disappear, but new problems arise with the occurrence of permanent deformation, especially on roads with heavy traffic. In the 1990s introduced the use of Stone Matrix Asphalt / Split Mastic Asphalt (SMA), but is considered less successful because of the difficulty of mixing the cellulose fibers evenly in the mixture Split Mastic Asphalt.

2000s occurred against the development of Stone Matrix Asphalt Stone Matrix Asphalt become Asbuton (SMAB), the results showed that the addition asbuton drainage can reduce the occurrence of bitumen so that it can replace the function of the cellulose fibers as stabilizers, also adding asbuton mixture increases the resistance to permanent deformation. In the era of the 2000s in addition to the development of Stone Matrix Asphalt by Pusjatan also conducted full-scale testing of porous asphalt on a road segment Jatibarang - palimanan (sta 30 600 - sta 31 050) towards Jakarta and Cirebon in the slow lane and fast lane, the northern coast of Java West, with a 5 cm thick layer of dense (Iriansyah 2010).

The description above is a partial effort in reducing early damage pavement, on the other hand there are events in the form of (traffic congestion, vehicle stop when the red lights, vehicles parked on top of the pavement, stopping the vehicle when taking a passenger, the vehicle stopped at the door toll, the vehicle stopped at the entrance of the airport exit, the vehicle stopped on the road that bertebingkan ravine (subgrade to the width of pavement), which gives the effect of its own (monotonic static) to the road pavement structure where the worst condition if at the time the incident took place there vehicles charged more.

The above conditions as well as a variety of related information into a source of inspiration importance of a form of testing that can reveal a phenomenon seen in general as well as flexible solutions. In this paper in question is a form of testing Testing Multi layer.

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METHOD

As a literature review, the method used to adjust any literature review methods commonly used in general, the method used is the internalization approach - an addition as part of the epistemology of science (U. Kh Maman, 2009. According to Big Indonesian Dictionary (2005: 439) internalization namely the appreciation of a doctrine, doctrine, or the value of that is the confidence and awareness of the truth or value manifested in attitudes and behavior. Meanwhile, according to Prof. Mulyasa (2012: 167) in Umar page 1 internalization is an attempt to live and deepen the value, so that embedded within every human being.

Another understanding of internalization by a radical fundamental approach popularized by Ismail Yusanto is to do the work of putting Islamic values into materials that comply with Islamic law and the adduct is adding some new submateri into existing material. From the definition of internalization as well as the addition of the above, the authors conclude internalization approach - an addition that efforts to create a multi-layer into a shape of its own testing with less add some significant things into multi-layer theory.

RESULTS AND DISCUSSION

Khairi Fadhlan and Zulkarnain A. Muis. in planning evaluation thick flexible pavement construction methods clan Pd T-01-2002-B by using the program kenpave, page 2. writes that a multi-layer or elastic theory is the main principle of the mechanistic method of calculation. This concept assumes that each of pavement has properties such as homogeneous, isotropic and linear elastic which means it will return to its original shape when the load is moved. In modeling pavement road with this elastic layer model input data required to determine the stress and strain on the pavement structure and response to load. The parameters used were:

Parameters of each layer. Parameters layer is meant, among others, are as follows:

a. Modulus of elasticity
b. Poisson ratio
c. The thickness of each layer
d. Load conditions

In this method the work load of the vehicle above the pavement structure is considered as a static load evenly, then there will be stress and strain on the structure. Voltage, strain, and deflection is the response of pavement material which experienced loading. Calculation of the response voltage, strain, and deflection at each layer for a pavement is extremely complex requiring extremely high accuracy that will be tricky done manually for the necessary calculations using computer assistance. The use of computer tools should also be the time of data collection.

According to Lu Sun, et al (2011, page 2) layered elastic media described as the picture. 1 and used as the basis of a physical model of the pavement system.

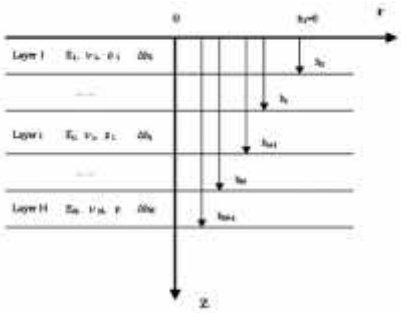

Figure 1. Medium Multi layer

Should be understood that the multi-layer theory strung from a variety of theories, among others:

1. Theory Boussinesq

   The assumption: pavement and subgrade is homogeneous and isotropic linear elastic modulus of elasticity (E), and Poisson's ratio (μ) as a result of the static concentrated load on the surface of the pavement (Tu, W., 2007).

   A series created by Boussinesq equation to determine stresses, strain and deformation. The illustrations can be seen in Figure 2.

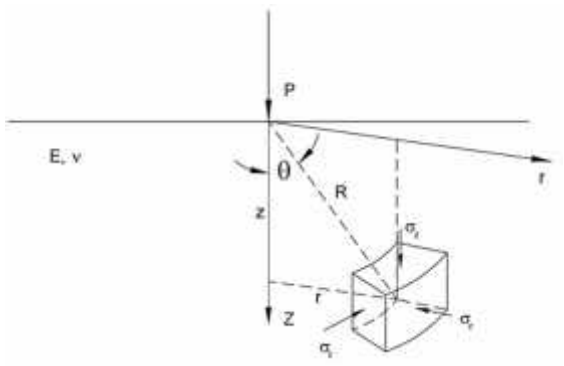

Figure 2. The axis coordinate system for Boussinesq equation (Tu, W, 2007)

Several Boussinesq equation shown in the equation below, where:

$$R = \sqrt{z^2+r^2} \quad (1)$$

Normal stress:

$$\sigma z = \frac{3P}{2\pi R^2}\cos^3\theta \quad (2)$$

$$\sigma r = \frac{P}{2\pi R^2}\left[3\cos\theta\sin^2\theta - \frac{1-2\mu}{1+\cos\theta}\right] \quad (3)$$

$$\sigma t = \frac{(1-2\mu)P}{2\pi R^2}\left[-\cos\theta + \frac{1}{1+\cos\theta}\right] \quad (4)$$

Shear stress

$$\tau rz = \frac{3P}{2\pi R^2}\cos^2\theta\sin\theta \quad (5)$$

$$\tau rt = 0 \quad (6)$$

$$\tau tz = 0 \quad (7)$$

Normal strain

$$\varepsilon z = \frac{(1+\mu)P}{2\pi R^2 E}\left[3\cos^3\theta - 2\mu\cos\theta\right] \quad (8)$$

$$\varepsilon r = \frac{(1+\mu)P}{2\pi R^2 E}\left[-3\cos^3\theta + (3-2\mu)\cos\theta - \frac{1-2\mu}{1+\cos\theta}\right] \quad (9)$$

$$\varepsilon t = \frac{(1+\mu)P}{2\pi R^2 E}\left[-\cos\theta + \frac{1-2\mu}{1+\cos\theta}\right] \quad (10)$$

Displacements

$$dz = \frac{(1+\mu)P}{2\pi RE}\left[2(1-\mu) + \cos^2\theta\right] \quad (11)$$

$$dr = \frac{(1+\mu)P}{2\pi RE}\left[\cos\theta\sin\theta - \frac{(1-2\mu)\sin\theta}{1+\cos\theta}\right] \quad (12)$$

$$dt = 0 \quad (13)$$

in the above equation, the elastic modulus does not have any influence on the stress and strain of normal and shear stresses.
The next Boussinesq equations developed by researchers with the load evenly.

2. Burmister Theory (1945) / theory of two layers

The assumption: pavement consists of two layers:
- Pavement
- Subgrade
- The top layer and bottom layer is homogeneous, elastic and isotropic
- Great rigidity against the rigidity layer on the bottom layer, the ratio give a decisive influence

Formula:

$$D = 1{,}5\left(\frac{p.a.F2}{E2}\right) \tag{14}$$

with:

D = deflection
p = load evenly split (Pa)
a = radius of the circle (cm)
F2 = coefficient of deflection, depending E1 / E2
E2 = modulus of elasticity of the soil (Pa)

3. Theory of Westergaard (1926)

- The theoretical analysis ttg slab laid on the ground that burdened wheel vehicles
- Stress occurs analyzed

The assumption:
- slab is homogeneous, isotropic, and elastic
- subgrade elastic (the vertical direction)
- due to load, subgrade reaction vertical direction and magnitude
- to proportioned slab elastic deflection load evenly split,
- with the contact area of a circle and an ellipse

Westergaard made form analytical solutions approach for stress and deflection on rigid pavement with illustrations (figure 3) and the following formula.

Figure 3. Three loading conditions Westergaard theory

corner loading

$$\sigma c = \frac{3P}{h^2}\left[1 - \frac{a\sqrt{2}}{\ell}\right]^{0.6} \tag{14}$$

$$\Delta c = \frac{P}{k\ell^2}\left[1.1 - 0.88\left(\frac{a\sqrt{2}}{\ell}\right)\right] \tag{15}$$

interior loading

$$\sigma i = \frac{3(1+\mu)P}{2\pi h^2}\left(ln\ \frac{\ell}{b} + 0.6159\right) \tag{16}$$

Where:

$b = a$ jika, a> 1.724h (17)

$b = \sqrt{1.6a^2 + h^2} - 0.675h$

if, a< 1.724h (18)

$$\Delta i = \frac{P}{8k\ell^2}\left\{1 + \frac{1}{2\pi}\left[ln\left(\frac{a}{2\ell}\right) - 0.673\right]\left(\frac{a}{\ell}\right)^2\right\} \tag{19}$$

edge loading

$$\sigma e = \frac{3(1+\mu)P}{\pi(3+\mu)h^2}\left[ln\left(\frac{Eh^3}{1\ \ k\ ^4}\right) + 1.84 - \frac{4\mu}{3} + \frac{1-\mu}{2} + \frac{1.1\ \ (1+2\mu)a}{\ell}\right] \tag{20}$$

$$\Delta e = \frac{\sqrt{2+1.2\mu P}}{\sqrt{Eh^3k}}\left[1 - \frac{(0.7\ \ +0.4\mu)a}{\ell}\right] \tag{21}$$

The other side is required for the addition of the results of an empirical approach as a helper to define theoretically cause and effect relationship right from the phenomenon. Or in other words to establish the context of the multi-layer at the level of correlation of input-process - output and results of the management context .. The empirical method as a design approach that is based on the results of the experiment or experience, in which observations are used to establish the correlation between the input and the results of process. Khairi Fadhlan and Zulkarnain A. Muis. in planning evaluation thick flexible pavement construction methods clan Pd T-01-2002-B by using kenpave program, it 3. Jot that empirical method was developed based on the experience of existing research. By him the necessary steps observation that definitely requires major equipment and support as well as software to facilitate the gathering of data on the observation proficiency level. The equipment according to the following picture:

Figure 4. The multi-layer test tool

It inspires a multi-layer used as a form of its own testing to obtain preliminary information on the early symptoms of damage to road pavement including previous research as follows:

Table 1. Relevant Research

Researchers	Title	Objects	Test Results
Hary Christady Hardiyatmo	Naileb Slab for Strengthening Concrete Plates on Rigid Pavement	concrete plate 50x50 cm and 70x70 cm, 10 cm thick with and without poles there are placed on the ground .	The average deformation on specimen without poles with dimensions of 50x50 cm is 3:39 mm while the dimensions are 70x70 mm 2,54
Luca G. Sorelli, dkk	*Steel Fiber Concrete Slabs on Ground: A Structural Matter*	A Matter Structural concrete plate 300x300 cm with 15 cm thick is placed on the placement is made of steel as many as 64 pieces	the average deformation on specimen without steel fiber is 2 mm

CONCLUSION

1. Multi layer worth increased to testing his theory because the device is clear.
2. Multi layer worthy of testing because it meets the context of input-process-input and assumptions.
3. Approach - an addition strengthens the argument that the multi-layer deserves to be considered a separate test able to provide early information in order to reduce the premature deterioration of road pavement

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ATTRIBUTES AFFECTING THE BUS NETWORK SYSTEM OF DHAKA CITY AND THEIR DEFICIENCIES.

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ABSTRACT: Transportation system of Dhaka city is under huge challenge of managing growing number of private cars and non-motorized vehicles. Traffic congestion is an everyday scenario of this city. Buses are the only available organized public transport system in Dhaka. This research assesses the quality of service provided by the bus network system and its deficiencies. Under the study, demand-supply conditions and performance of the public buses had been identified by processing field survey based on 22 user attributes data according to a suitable mathematical procedure. For this purpose, 15 different bus routes were chosen and a total of 500 passengers of buses were interviewed with a predetermined structured questionnaire to know their experience, level of satisfaction and opinion about the existing service as well as their expectations. This study generates a ranking of the attributes that influence bus transport system. It was observed that the existing bus service quality is very poor. There is no specified time schedule for the buses operating; hence passengers waiting time at station is longer. Except a few bus of seating service, all the buses carry extra passengers than the seat capacity and always remain over-crowded. Thus buses are often not accessible for disabled or elderly persons as well as for women.

Keywords: *Traffic congestion, bus network system, deficiencies, , demand-supply conditions, over-crowded.*

INTRODUCTION

Dhaka, the capital of Bangladesh is one of the least motorized mega city in the world but most densely populated city with a current population of almost 16 million at annual growth rate of 8% per annum (STP,2004). Anyone living in this city will certainly speak of overcrowded city streets. The roads of the city are clogged with vehicles all day long making travel unbearable. It has become a dream for city dwellers to travel on traffic jam free road. Unfortunately, Dhaka's struggles will prevail as it is growing at the second fastest rate of the twenty most populated metropolises in the world(World Urbanization Prospects:, 2006).The impact of such rapid growth has major consequences on the ability of the transport sector to provide mobility for all people, as they seek to take advantage of employment, education, health and social opportunities. As Dhaka is also the economic, cultural and administrative hub of Bangladesh, huge amount of people travel daily by pedal rickshaws, auto-rickshaws, tempos, taxis, private cars, and a wide array of buses. Among these travel modes, buses are the only available conveyance medium for dedicated mass transport system in Dhaka city. The bus system mostly financed by private owners. These buses typically ply overcrowded with very low frequency, very poor maintenance and service quality but the extent to which they do so, differs a lot by the type of bus service.

BACKGROUND

Dhaka is a city with a transportation system predominantly road based having absence of any mass transit system like metro rail or bus rapid transit (BRT). As a third world country, the majority of trips in Dhaka are served on public transport and non-motorized transport modes (NMT) or Para-transits because a significant numbers of people are poor who cannot afford personal vehicle. As the fare of NMT (such as rickshaws) or other Para-transits are more expensive than the bus fares (Rahman, M. S. U., 2009); most of the people are heavily dependent on public transport for their travel (Hossain, M, 2006) .Among households, 7% either have or have access to a car, 4% owns motorcycle, 3% cycle rickshaw, 5% bicycle, and 2% auto-rickshaw. Auto-rickshaw and cycle rickshaw are for commercial uses. The results revealed that 84% of household do not have any sort of transport vehicles and depend on public transport and only 16% have some sort of vehicles. This explains the importance of and dependency on the public transport system (STP,2004). Although only mechanized transport buses run the highest passenger-km per day(Rahman, M.S.U and Nahrin, K., 2012) , according to 2005 Strategic Transportation Plan (STP) it's only 44%, which is very low. However, the demand of public transport system is increasing with the increase of passengers for the last 20 years(Karim, M. M., and Mannan, M. S.,2008).

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As Dhaka is a city with poor economy and low private car ownership, it needs a very sustainable and economy friendly bus transport system. Dedicated bus transport system for mass transit should be the backbone of transportation system in this city. However, at present, passenger transport scenario of Dhaka shows that the existing bus transport facilities is not sufficient to keep pace the growing demand. In addition, this service is inefficient, unproductive and unsafe. Passengers are frequently facing long waiting time, delay on plying, over-crowding, lack of comfort, long walking distance between origin and nearest bus stops as well as between bus stops and destination. Again, a well-planned city should have 25% of total area for its road network whereas in Dhaka; only 9% of total area is occupied by road. This paper is an attempt to explore existing service quality and deficiencies of the bus network system of Dhaka city.

USE OF BUS AS PUBLIC TRANSPORT

All shapes and sizes of vehicles are part of the traffic. The multiplicity of modes of transport, private cars, motor cycles, buses, taxis, cycle rickshaws, rickshaw vans, trucks, pushcarts and recently revived animal driven transports offers a rich range of choices to travelers and transporters of goods. But lack of relevant measures and firm decisions to cope with the complexity of traffic has created chaotic conditions. Different types of modes using the same road space characterize traffic environment. Current very diverse traffic mix is increasing the traffic delays constantly. Figure-1 is an illustration of the complex traffic composition in Dhaka.

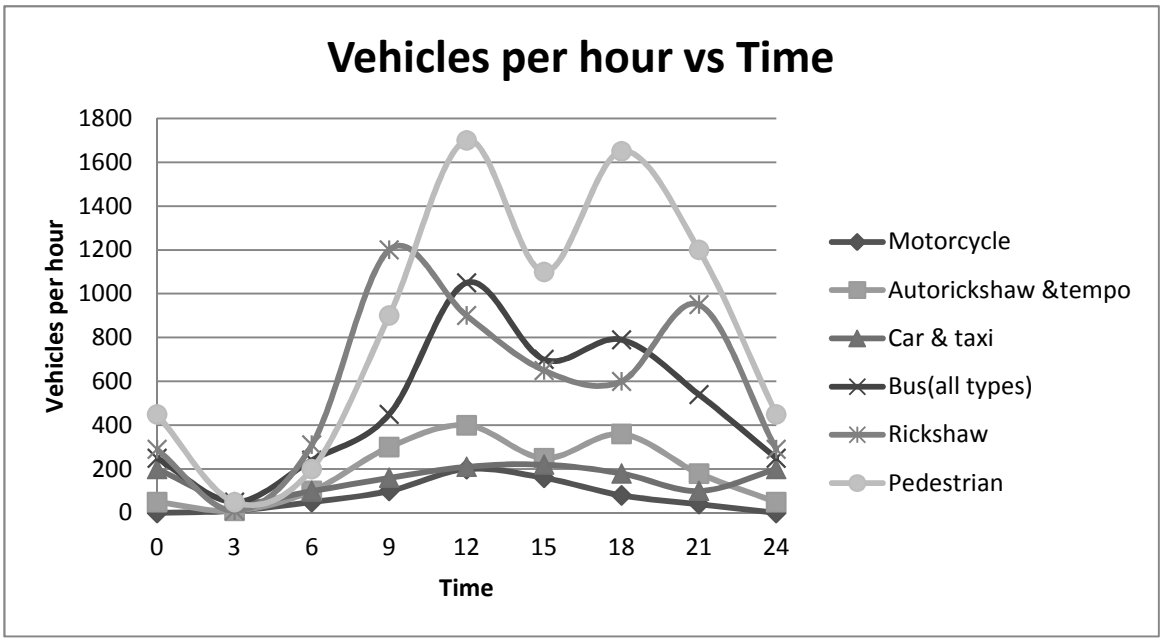

Fig 1: Illustration of traffic composition in Dhaka (Source: *STP, 2004*)

An attitudinal questionnaire survey was made by STP in 2004 on public transport. In reply to a question on 'reasons for using bus as usual travel mode' respondents said they preferred bus journey because of various reasons. Almost 73% said it is cheaper, 69% said they don't have any other alternatives, 31% said it is more or less reliable, 21% said it save time, 26% said it is convenient and comfortable too and 22% said it is comparatively safer. Only 10% said that they take bus because they cannot use their car to get to their work place due to the problem of parking. Table-1 shows parametric comparison of different transport modes of Dhaka. The table shows comparing to all other transport modes, buses serve the best with low cost, greater passenger carriage covering maximum trips.

Table-1 Parametric comparison of different transport modes in Dhaka.

Mode types	Passenger boarding at peak hour	Load factor	One way Avg. trip length (km)	No. of trips/ day/ vehicle	Passenger Km/ Day/ vehicle	Journey speed (km/hr)	Fare demand (Tk/km)	No. of conductor or helpers
Rickshaw	2	1	4.8	35	252	9.3	2.7	Nil

Baby taxi	3	1	8	22	396	10.8	5.4	Nil
Mishuk	3	1	6	22	297	15.2	5.1	Nil
Auto tempo	12	1.2	6	18	927	21.9	0.47	1
Double Decker bus	102	1.4	14.5	12	8874	15.5	0.25	2 to 3
Single Decker bus	77	1.5	14.5	12	6700	15.5	0.25	2 to 3
Private bus	77	1.5	14.5	12	6700	15.5	0.25	2 to 3
Mini bus	42	1.2	16	14	4704	18.3	0.37	2

PREVAILING BUS SERVICES AND BUS ROUTES IN DHAKA

In Dhaka, according to services, buses are classified into two types: (1) Counter bus service; and (2) Local bus service. Counter buses have specified stoppages for loading and unloading passengers. Passengers have to purchase ticket before boarding on counter buses. Few of the counter buses are air-conditioned. Whereas, local buses have no defined stoppages. They stop anywhere on the road for loading and unloading passengers. Passengers pay to conductor after boarding on the bus. Both counter and local buses remain over-crowded due to gap between demand and supply. However, there are a few seating service buses operating in certain limited routes which allow boarding passengers only if there is an empty seat available for the person (e.g. Bikalpa having start and end termini at Motijheel and Mirpur 12, respectively as in Table 3).

Again according to size, Dhaka buses can be grouped in to 3 types: (1) Large bus; (2) Mini bus; and (3) Double decker. Large buses are those with at least 32 seats. However, more generally buses with length more than 10m are considered as large bus. The most significant recent change in the bus fleet composition is the increases in the number of large buses. This trend began with Sino Dipon in early 2003. They are now operating on four different routes with 105 buses. Minibuses have capacity of 15 to 30 seats excluding driver's seat and normally are around 8m long. There are around 9311 numbers of registered large buses and 8459 numbers of registered mini buses in Dhaka city(http://www.brta.gov.bd/) .

STP conducted a survey on average trip length, travel time, speed and carrying capacity of existing mass transit system and got the following scenario as shown in Table-2.

There are 39 different routes of bus service in Dhaka city approved by Dhaka Metropolitan.

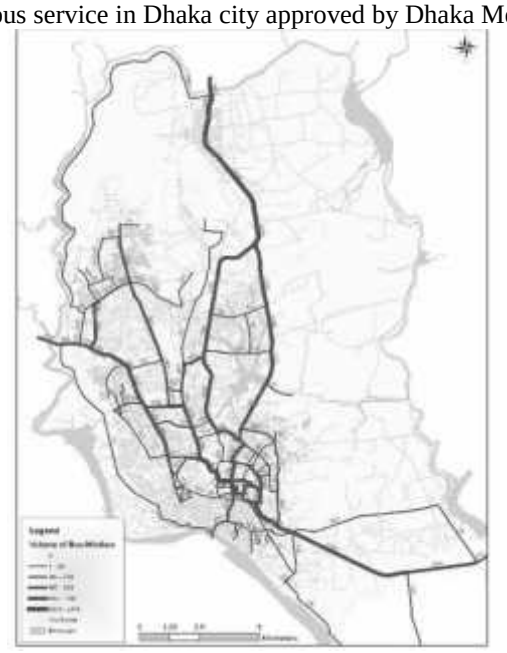

Fig 2: Prevailing bus routes and number of buses operating on those routes.

Regional Transport Committee (DMRTC) in April, 2009. However, almost all of these routes have variations in routes for operation. No scientific methods or planning process are applied for identifying such bus routes for operating and bus stoppage (Rahman, M. S. U. ,2011). Fig 2 demonstrates prevailing bus routes and number of buses operating at those routes.

Table-2 Average trip length, travel time, speed and carrying capacity of existing mass transit system in Dhaka City.

Mode	Average Route length (km)	Average Journey time (min)	Average travel speed (km/hr)	Average running speed (km/hr)	Average boarding passenger	Pass-load at max. point	Average stops
Mini Bus	20.12	65.00	17.14	23.19	79.27	39.1	16.56
Large Bus	14.7	71.09	13.7	17.8	89.6	44.9	12.3
Double Decker	17.81	56.83	17.22	23.45	100.76	83.71	13.67
School/ Staff bus	13.5	37.25	20.9	23.0	44.4	43.4	7.8

METHODOLOGY

Dhaka inter-city bus transport companies have a wide variety of buses. The reactionary behavior of bus staffs (bus driver, bus conductor, bus helper and counter staffs) created a difficult situation to explore the condition of service provided by bus transport system of this city. Although maximum bus staffs were friendly and helpful to share their experience and answer the question asked for survey, a few were afraid of sharing as their buses do not follow the rules imposed by Bangladesh Road Transport Authority (BRTA). The structure of the study was aimed at certain variations in bus types and examined the way they performed, their physical condition, average journey speed and passengers comfort level while travelling. The ultimate goal was to explore the level of service provided by bus transport system of Dhaka city by collecting individual experience at certain stops. It was decided that a variety of buses to be captured so that the results represent the bus network system. Counter buses have wider variety of door arrangements. Bangladesh road transport corporation (BRTC) operates some double decker and articulated buses. Local buses have less variation in size and all of them fall in to mini bus category.

The data was collected by discussion with the bus employees (i.e. bus conductors, bus drivers, employees at counter). Data given by bus staffs were cross-checked by riding on that particular company bus three times at different periods. To know the riders opinion on riding bus, a total of 500 persons were interviewed with selected structured questions. The survey form was ten questions long and was conducted through face-to-face interviews. Few questions (e.g. your comfort level, your movement inside the bus) were phrased for satisfactory, moderate and un-satisfactory responses and other questions (e.g. are you a regular passenger on this route) were phrased for yes or no responses. The survey was written in English and translated into Bengali before delivered to subjects. Interview places were selected considering two facts: (1) Starting or ending point of bus route; and (2) Places where counter bus passengers and local bus passengers assemble and remain for a sufficient period to board or alight. By this, counter bus passengers and local bus passengers were easily separated. The interview was conducted during the month of October to December 2013 at sunny days.

The data of speed was collected by riding on that particular bus and by using GPS device (route length and travel time). After taking three readings on the same bus company on same route, an average travel time was obtained. Average journey speed data was obtained by dividing the total route length by average travel time. During weekdays, data was collected at peak hour, from 8.00 am to 10.00 am in the morning and 4.30 pm to 7.00 pm in the evening. And during holidays data was collected from 3.30 pm to 7.00 pm in the evening.

All buses would be ridden only within the city limit. For this, routes were chosen from all parts of the city, and when possible, a route which started and ended within city limits. For this study, the study riding limits were set at Uttara in the northeast, Jatrabari in the southeast, and Mirpur 12 in the northwest of Dhaka. To collect data during journey each route was observed from origin to destination but data which valid within city limits were taken. To assess the condition inside the overcrowded buses, the surveyors ride on the bus and collected necessary data. The number of people standing before the first row of seats and immediately in front of the middle or back doors was recorded as a measure of a bus's crowding levels. The number of people who remained at the bus stop who could not board, the number of passengers with some part of their body outside the doorframe and passengers riding on the roof of bus were also recorded.

It was quite impossible to cover the whole city or all the bus routes and operators of the city due to time and resource constraints. Hence, only the seven major bus routes: (1) Motijheel to Mirpur; (2) Motijheel to Gulshan; (3) Motijheel to Mohammadpur; (4) Azimpur to Kuril (Bishwa road); (5) Azimpur to Mirpur; (6) Azimpur to Uttara; and

(7) Jatrabari to Mirpurhad were chosen for this study. While selecting the bus routes, it was considered that they cover the whole city (i.e. the central area and the periphery, the planned and unplanned area, and the higher-income and lower-income residential areas). Ultimately, 90 ticket bus trips and 45 local bus trips were collected.Table-3 shows the collected survey data.

Table-3 Survey on selected bus routes.

Bus routes	Bus stops (*Places where passengers were interviewed*)	Selected bus services	Average journey speed during working days (km/hr)	Average journey speed (km/hr)	Type of Service
Motijheel to Mirpur	Mrpur12, Mirpur1, Farmgate, Gulistan, Motijheel	Bikalpa Auto Service (M.B.)	11.48	28.7	Sitting
		Bahan paribahan (L.B.)	8.32	15.33	Local
		Shakalpa paribahan (L.B.)	8.92	14.49	Counter
Motijheel to Gulshan	Motijheel, Malibag, Doinik Banglar Moor, Gulshan 1	Bus no 6 (L.B.)	6.52	10.56	Local
Motijheel to Mohammadpur	Motijheel, Pressclub, City college	Moitri (L.B.)	7.23	13.17	Counter
		Rajacity (L.B)	8.05	14.23	Counter
		Dipon Transport(L.B)	6.87	14.98	Counter
Azimpur to Kuril (Bishwa road)	Azimpur, Kalabagan, Bashundhara city, Nabisco	Winner (L.B)	7.67	18.18	Counter plus boarding
Azimpur to Mirpur.	Azimpur, Kalabagan, Mirpur12,Mirpur1.	Safety bus service(M.B)	7.5	19.54	Boarding
		Bihanga (M.B)	7.45	16.93	Boarding
Azimpur to Uttara	Azimpur, Kalabagan, Farmgate, Mohakhali.	Bus no. 27(M.B)	6.56	14.87	Local
		Duldul(L.B)	8.37	16.29	Counter
Jatrabari to Mirpur	Jatrabari, Gulisthan, Farmgate, Mirpur 10, Mirpur 12	Bus No.14	7.27	17.28	Counter

(Note: L.B. = Large Bus and M.B. = Mini Bus.)

RESULTS

Overcrowded buses are the main reason of long waiting time at bus stops. This is due to low frequency of bus headway and high demand. Waiting time at a bus stoppage was reported less than 5 minutes by 22% of passengers, waiting time greater than 5 minutes but less than 10 minutes was reported by 31% of passengers. And 34% reported of waiting between 10 to 20 minutes .However, 13% reported that they frequently have to wait more than half hour to board in a bus. Waiting time only for local buses was reported more than 20 minutes by 72% of passenger. This long waiting time is also due to overcrowded buses causing no room for new boarding. It is common sight in Dhaka that a few passengers is unable to board in the bus as there is no room and they have to wait for the next one. Figure-3 shows the queue of the passengers waiting for bus and Table-4 shows the number of passengers at bus stops waiting per hour along with the prevailing demand and supply of buses at that particular stop.

Fig 3: Passengers waiting for bus.

Table-4: Demand and Supply in Different Stops.

Stops	Total number of awaiting passenger per hour	Total demand of bus per hour	Current supply of bus per hour
Mirpur 12	52	0.35	0.35
Mirpur 10	78	0.65	0.33
Mirpur 1	105	0.88	0.38
Kalabagan	48	0.3	0.35
Azimpur	201	1.68	1.1
Gulistan	426	3.84	3.05
Jatrabari	480	4	3.7
Motijheel	224	2.25	2.25
Press club	55	0.32	0.68
Nabisco	35	0.23	0.36
Khamar Bari	386	3.56	3.18

Besides waiting at the bus stops, this survey shows that about 83% riders are not satisfied with the delay time incurred at each and every stoppage. And the rest are moderate. Whereas, no one is satisfied with the delay time of the local bus only. A local bus will commonly remain at a stop for several minutes until the conductor feels the bus is sufficiently loaded, which typically means every possible space is filled. Some local buses will leave a stop when a second bus of the same route arrives, but schedules are roughly kept so that the first bus will already be sufficiently crowded by the time of the second bus's arrival. In case of counter buses, counter staff makes the bus wait to sell some more tickets. Passengers on-board often shout at bus staffs to stop this delay. This often creates chaotic situation. Even, they often board and off-load passengers at unspecified stops. These unexpected halt cause unnecessary delay for on-board passengers. Besides, higher alighting and boarding time might be another cause of delay. Passengers reported that average delay time at each station for counter buses ranges from 2 minutes to 5 minutes while for local buses it varies from 3 minutes to sometimes more than 8 minutes. This delay time affects average journey speed.

Difference in average journey speed is showed on Fig 4. For weekdays this difference is less significant because of traffic jam, all buses are bound to move at same speed. But during holidays, this difference is significant. Delay time while travelling is the main reason of this difference. Compared to local buses, counter buses have fewer stops and less delay at each stop.

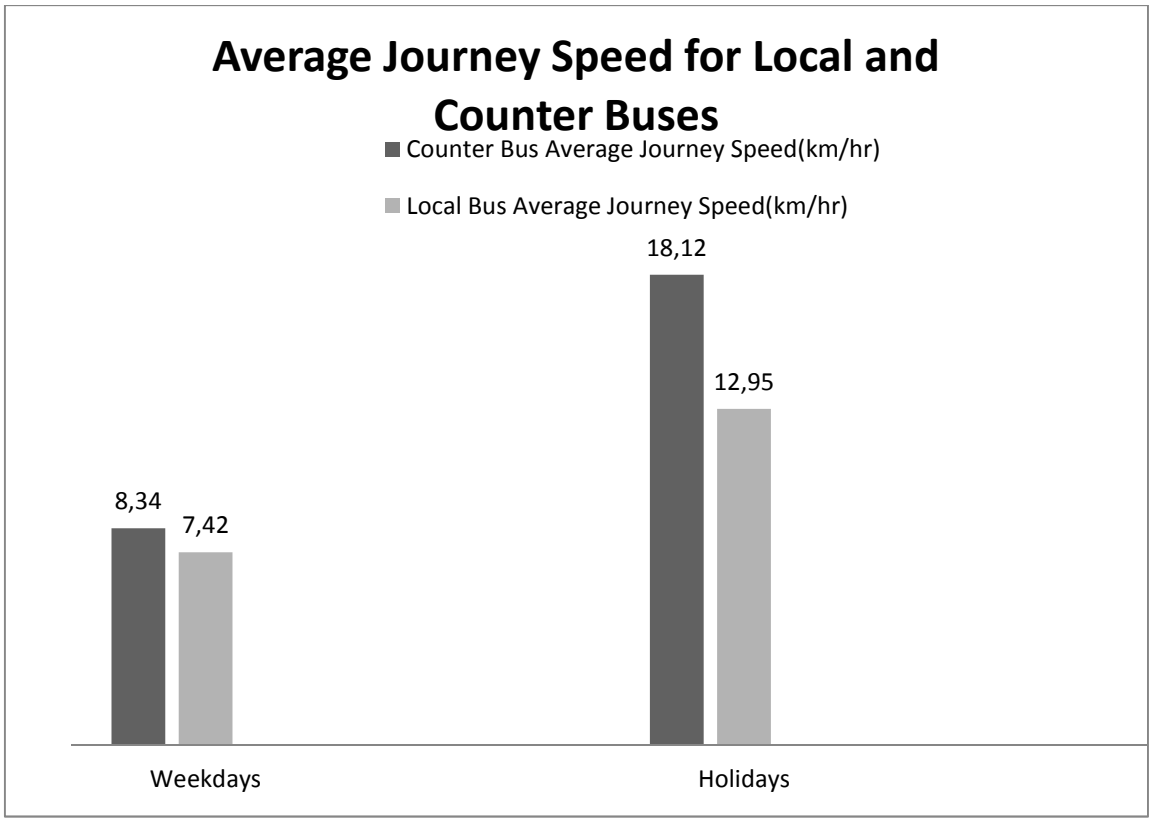

Fig 4: Average Journey Speed for Local and Counter Buses

Considering the bus frequency, only 10% are satisfied where 55% and 35% are respectively moderate and un-satisfactory. However, considering only local buses, un-satisfactory passengers rises up to 70%. Often due to low frequency and high demand, it becomes impossible for older people or women or children to get room inside the bus. Consequently, these groups of vulnerable people do not have access to bus service during rush hours. Most buses especially local buses carry passenger exceeding their capacity.

Only few buses carry passenger as much as their seat capacity ('no standing passengers') e.g Bikalpa Auto Service (Motijheel to Mirpur12). Due to high demand, passenger boarding in the bus risking their lives (see, Figure 3). Due to overcrowded situation, comfortable movement inside the bus is restricted often impossible. Survey shows that more than 73% of bus passengers do not feel comfortable during travel. This discomfort increases at office starting and ending hours. This situation is worse in local buses where conductor moves inside the bus to collect fare of travel often make the condition of passengers standing miserable. Also, he shouts for passengers to compress more so that he can board in more passengers beyond the bus capacity. Figure-5 shows overcrowded bus during the typical peak hours.

Fig 5: Overcrowded buses of Dhaka.

For female passengers the travelling environment of buses especially local buses is very much unsatisfactory. Not all local buses have special female seats. Compared to local buses, counter buses are much better, have 6 seats in front

especially for female passengers. But this number is very few compared to the female passengers travel. This survey found that there is no especial care for pregnant female passengers. So riding buses in Dhaka city for pregnant woman is a matter of great risk. Also sexual harassment to women in crowded buses is a common problem in Dhaka. Many women reported that they experienced physical harassment while boarding or alighting bus by bus staffs, even by male passengers. Figure-6 shows the satisfaction level of riders riding on buses.

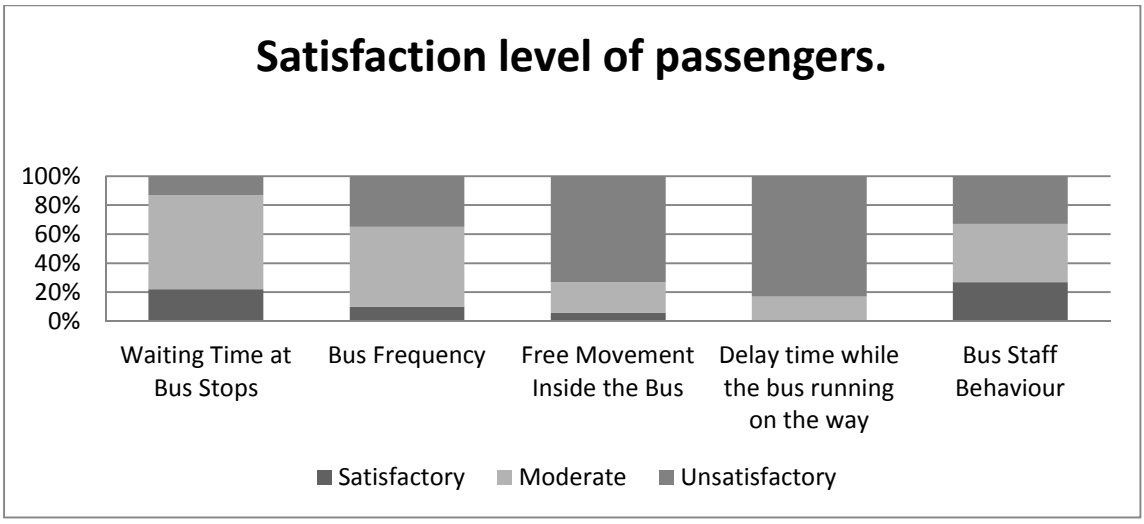

Fig 6: Satisfaction level of passengers boarding on buses.

The physical condition of the buses plying in Dhaka city is not satisfactory. This is because the majority of bus fleet is very old and the maintenance is almost absent or very poor. Despite the poor bus condition riders not feeling it or mentioning about it because their main concern might be getting a seat or room inside the bus irrespective to its overall condition. For instance, one of the passengers told 'interior of the counter buses are moderate, however, the local bus is very poor'. Not only interior but also bus engine condition is very poor. They reported that for a large bus with more than 50 seats have almost one fifth of seats are out of order. However, if only local buses are considered, almost all the passengers reported uncomfortable seats: (1) having holes in the seats; (2) back side of the seats broken; (3) no foam in the seats; and (4) bad odor. also the size of the seat is unsatisfactory Due to poor maintenance of the bus fleets, often these become out of order on the way and causing problems for its passengers. About 40% of the respondents reported that they had experience of unexpected breakdown of the bus while travelling, mostly due to fitness problem of bus or staff related problem. Researcher had such experience of sudden breakdown in Dhaka for two times during the survey in 2013. On that occurrence, the passengers had to wait until another bus, which was passing the spot, to pick up them or some passengers either walked to their destination or took alternative modes. This causes serious harassment for passengers by compelling them to find new bus or to find the nearest bus stop to board.

The survey shows that about 47% respondents reported that their closest bus stop is within 0.5 km while 36% mentioned within 0.5 km to 1 km and more than 1km was reported by 17%. So, it is clear that position of bus stops is satisfactory. Even bus stops within half km cause reduction in bus frequency. Although there are sufficient bus stops, the condition of bus stops are not good. Very few bus stops offer protection from the sun, rain, dust, and other elements that have significant implications for health or safety. Passengers shade is almost absent in Dhaka city. There are a few, however, occupied with vendors or shops and no use for the bus passengers. Counter bus stops have big umbrellas for their counter staffs but does not provide any shade to passengers from sun and rain. There is no seating facility for passengers, often they sit on sidewalks blocking pedestrian movement or stand in a queue while waiting for buses causing temporary bottleneck by blocking a lane of road. Also there is very few designated place for bus stoppage in road side of Dhaka city. Most of the bus stop in road side haphazardly with competition attitude and alighting and boarding passenger dangerously. This makes always crowed on road side and influence pedestrian to move on road as well as decreases the effective width of the carriage way.

Buses, which are main mass transit system in Dhaka, are operated by government authority and private financing entities. The BRTC, which operates under immunity from regulation by licensing authorities, owns a total of 306 buses operating on 15 routes in Dhaka *(1).* BRTC does not operate the buses directly, rather sub-contracts bus operations to private operators at different routes. The main problem of privately operated bus services is that buses are owned by a large number of operators. There are many operators who have only 2 to 3 buses. Drivers and crew in this category either own the vehicle individually or rent the bus on a daily or monthly basis. They then operate the vehicle at their own revenue risk, requiring enough passengers per day to repay the bus rental fee, cover fuel and basic maintenance costs, and make a profit. So, to keep their business upright they forget about giving minimum level of service to riders. Here in Table-5 the ranking of the attributes taken during survey is shown. From the below table it is justified that passengers give higher importance to bus punctuality and reliability even more than bus comfort.

Table-5: Attributes ranking according to public opinion survey.

Attributes	RANK
Access at Home	18
Access atDestination	19
Per day Journey	21
Frequency of bus service	2
Average travel time (weekdays)	7
Average travel time (weekends)	22
Fare Paying System	5
Daily expenditure on bus fare	20
Punctuality and Reliability	1
Seat availability	4
Comfort Level of Seats	6
Ease of Exit and Entry	16
Ventilation system	8
Security inside bus	11
Women harassment inside bus	12
Service quality	10
Behavior of bus staffs	15
Structural condition of the bus	9
Cleanliness	17
Noise Level (Bus Engine)	13
Experience of bus travel	3
Route Information	14

Dhaka is perhaps the only city of its size without a well-organized, properly scheduled bus system. Dhaka Metropolitan Regional Transport Committee (DMRTC) gives route permits without using statistics, without judging travel demand. There are some routes in the city where more than 5 counter bus companies along with more than three local operators exist. Whereas, there are some routes with only one counter bus operator. So it is obvious that due to too many bus operators and lack of statistical data, no route is operated on the basis of travel demand and supply. In Dhaka, there is no coordinated bus network information center. It is not easy to travel to desired route for unfamiliar riders. Especially for local bus service as there is no counter; passengers have to rely on fellow passengers of same route to collect information. For a vast city like Dhaka with its huge population where bus is the only mean of public transport this is certainly unacceptable.

DISCUSSION

Bus is the only mode of public transport system, the choice for the majority and the only means of mobility that can be afforded by the urban middle and lower economy people. Public transport system by definition is a system which provides scheduled service carrying mass people on a fixed route. However, public transport system of Dhaka city is not maintaining any time schedule. Due to heavy congestion on road and absence of any time schedule, waiting time for

the buses is unpredictable for riders. Due to low frequency and supply-demand deficiency the buses remain always overcrowded making the journey experience a miserable one for passengers. Due to the extreme levels of congestion in the streets of the city, buses cannot keep consistent schedules. This means that passengers will prefer to board a crowded bus, even at the risk of their personal safety, rather than wait an unknown length of time for the next bus to start (Katz, Donald and Rahman, Md. Mizanur,2010). However, there are some sitting services but their numbers and routes of operation are very limited. The average journey speed is about 7.5 to 8 km/hr which is few kilometers greater than normal walking speed of a matured human being. Also, there is no organized authority to operate day to day operation of bus system.

The only organized bus transport authority BRTC started with the motto of "*Service is our motto: Comfort is our commitment*", it cannot fulfill any of their objectives(BRTC. 2011). With few exceptions, the bus owners or operators, including government owned BRTC buses in Dhaka City do not pay adequate attention to passengers' comfort(Andaleeb, Syed Saad, Haq, Mahmudul, and Ahmed, Rubina I, 2007). Not only operational deficiencies but also Dhaka bus system lacks of quality bus service. Facilities inside the bus, such as fans and lights are frequently out of order; many of these do not have lights and fans at all. Other requirements, such as airflow and ventilation, lighting during night must be monitored to ensure a desirable environment for riding. Basic passenger's requirements, like comfortable seats and windows for airflow also do not meet up the standard (Rahman, M.S.U and Nahrin, K, 2012). The results derived from the interview of bus passengers suggest that if comfort level could be improved it could also increase significantly the passengers' satisfaction, leading perhaps to greater proclivity to use public buses (Rahman, M. S. U. , 2011)

To reduce demand-supply deficiency authority prefer increasing the number of buses without any sort of research. These unplanned decisions increase congestion thus reduce the flow of buses more. These add up to the waiting time. In Dhaka, the number of bus stops is satisfactory but boarding-alighting system is unacceptable. Most of buses have only one door even if they have two doors, they use two doors both for alighting and boarding. Also the behavior of bus staffs is an important issue for service level. Often they halt the bus in the center of road for boarding or alighting passengers, causing temporary bottleneck and safety hazard for passengers.

There is no guarantee that anyone can reach to their destination within certain time because of the ever increasing traffic jam, congestion and shortage of public transport service. Public bus is the most affordable mode of transport in developing countries like Bangladesh. According to the study, performance of bus service will not be developed only by increasing bus number. Service regularity, on-time performance and service quality are also mandatory factors to be considered. Traffic congestion and excessive demand are behind all the problems. Reducing the traffic congestion is not in the hands of the respective bus authority but maintenance of the service quality of the bus is still manageable. This study gives a picture about the service condition of the bus that can help the bus authority to take remedial measures to upgrade the present situation of the bus. From the research, it is very clear that, the demand-supply deficiency is prevailed, but not so acute. But still the people are fully deprived from a quality bus service. The results of the research point out that the excessive travel time, waiting time and dreadful services in terms of comfort, regularity and on-time performance mainly hinder the prospect of the public bus transportation in Dhaka. Hence, transport facilities of the city should be provided keeping in mind the population growth, economic development, future travel demand and attributes ranking provided by the public of the city.

Dhaka is currently trying out new bus transportation options to help maximize the use of the road. BRTC has recently added air-conditioned buses to its fleet at different routes. Articulated buses have recently been purchased to be used by private companies, meaning one bus can carry higher loads if current crowding conditions remain the same. Also plans to institute a digitalized ticket system on the busy Uttara to Azimpur route. All existing companies on that route will run under one name and ticket system instead of competing with each other. Dhaka is also planning for elevated expressway, BRT and Metro Rail inside the city. These new operational implementations should be studied to see their effect on Bus network system of Dhaka.

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A GEOGRAPHICAL INFORMATION SYSTEM BASED APPLICATION FOR REGIONAL PLANNING DEVELOPMENT

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ABSTRACT: In various natural, social and cultural environments that are conducted in those environments, diverse matters related to the land and the lives of the people involve much information about position and time, and the state when, where and how, which are treated as geospatial information. Thus, because we are surrounded by a wealth of geospatial information, we must have an accurate understanding of this information and conduct analysis to reveal complex phenomena in order to resolve a wide range of problems from the very local level to regional issues. Therefore, in order to study the regional planning development, data-sharing and utilization of high level of geospatial information in the region is important to be considered. Moreover, developing and updating the geospatial information, designing the system that enabling greater efficiency and higher quality of services in region, and analyzing the regional planning spatially should be established. In this paper, a new methodology has been designed for regional planning development study. Firstly, a framework of a GIS based system which makes full use of the capabilities of a GIS technology as core system for the data-sharing and utilization of geospatial information in the region has been developed. Second, various data compilation in particular datasets from local government have been performed to provide the fundamental geospatial information. Third, multiple geospatial information such as natural hazard maps and photos from field surveys, topology maps, infrastructures facility maps, etc. which related to the regional planning development study have been integrated in order to obtain integrative spatial analysis and more intelligent use of location to the regional planning study. Finally, geospatial information services (geo-services) have been considered in this study to provide faster and more intelligent ways for developing various maps which is valuable to the regional planning development study.

Keywords: *GIS, data-sharing, regional planning, WebGIS, geo-services.*

INTRODUCTION

During this time, social conditions that involve geospatial information have been changing dramatically. Information and communication technology has undergone rapid development, and new and diverse services have been appearing (Goodchild, 2011). Highly functional personal technologies for individuals such as smart-phones have seen explosive growth as well, and people's needs are becoming even more diversified. In addition, Japan is facing many social issues, such as an environmental problems, energy problems, globalization of the economy, maintenance of public infrastructure, and preserving the safety and security of communities. However, there are expectations that the development of geospatial information and the use of Geographic Information System (GIS) and information communication technology (ICT) can be used as tools for solving these issues (Battrick, 2006). Furthermore, demands are being made for geospatial information to play an even greater role for regional planning development in rebuilding and restoration efforts in the wake of the 2011 Great East Japan Earthquake and subsequent disaster preparedness. In order to respond to regional planning issues, and to promote even greater use of geospatial information in society, a new basic plan for the advancement of utilizing geospatial information has been formulated to continue to address these issues (Shibasaki, 2008).

The development and utilization of geospatial information by public organizations of the national and local governments are enabling greater efficiency and higher quality of services, and cooperation between residents and local governments through GIS can be used for community development. Most of the information handled by the national and local governments is closely related to land use and community. Therefore, by developing and sharing this information as easy-to-use geospatial information, it is possible to get an accurate understanding of conditions and make administrative decisions and responses based on regional planning analyses, as well as promote greater efficiency and quality of administrative work. Moreover, in individual administrative fields such as regional planning, the combination with information from various other fields can help to implement more precise measures and policies (Alshuwaikhat

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and Aina, 2006). In addition, “visualizing” information about the community, such as having local governments provide and distribute their geospatial information in a format that can be utilized with GIS, collecting opinions and data on the state of activities of residents and others, etc., as geospatial information, can help local governments and residents work together to solve regional planning problems, promote alliances of various groups, and provide greater vitality to local community. In this way, the development of geospatial information and the utilization of GIS can be expected to make a wide array of contributions, from providing support for land management and daily living, to expanding regional planning development and helping local governments become more efficient and provide a higher level of services.

The local governments are responsible for formulating and implementing policies for the advancement of utilizing geospatial information for regional planning in accordance with the circumstances of their respective regions based on the basic principles (MLIT, 2010). However, local government holds a large amount of geospatial data which is often not shared effectively across departments and sometimes not released at all. There is also a lack of knowledge as to what data are available where and how one can access them. Implementation of better use and data-sharing and utilization of geospatial data and services in local government is not yet performed for regional planning. Therefore, in order to promote the development of regional planning by utilization of geospatial information in the region, a GIS based system for enabling the comprehensive analyzing and utilization of geospatial data is indispensable.

GEOGRAPHIC INFORMATION SYSTEM (GIS)

Role of geospatial technology

The world as a whole is becoming highly geospatially-enabled and there is the need to study trends and patterns and also communicate these phenomena with the evolving spatial technologies such that end users would benefit from this advancement in technological development. Since climatic changes and human activities could amount to the degradation of our natural environment, there is the need for us to acquaint ourselves more and embrace the vast emerging geospatial technologies in solving a number of environmental challenges to humanity today (Goodchild, 2008). Geospatial technology serves as a good decision support system in environmental issues and spatial planning. It has been proved as an appropriate tool for managing decisions relating to environmental concerns and challenges (Neuvel et al., 2010). The connection between geospatial technology and information communication technology (ICT) runs deep. Geomatics fields, especially GIS, have used information and communication technologies such as database management, data-sharing, networking, computer graphics and visualization. GIS is one of the most evolving aspects of geospatial technology. It evolved from desktop application in the 1980s into enterprise GIS in the 1990s and into distributed GIS. Even the technology of distributed GIS is evolving. It has changed from mobile GIS to web GIS and it is currently developing into cloud GIS. These emerging sources of geospatial information have become valuable to different societal and governmental applications such as geospatial intelligence, disaster management, regional planning development and real time data collection and tracking and property and services search.

GIS cloud and Geoportal

There has been a huge upswing in interest in geospatial data, and the cloud computing infrastructure to support the visualization and analysis has been developed (Chaowei and Goodchild, 2011). The rise of cloud computing just might be the most important thing happening in information technology today. While not everything will move into the cloud, it's fair to say that nearly every organization will use this new approach in some way. Given this reality, cloud computing has the potential to change many aspects of our world. For users of GIS technology, the cloud opens a number of new possibilities. Like the applications that run in your data center, cloud applications have users. The big difference is that cloud applications run in a shared data center that’s accessed via the Internet. The cloud platforms can run applications, store data, and provide other useful services. And while one important role of cloud platforms is to support cloud applications, cloud platforms themselves are of interest to developers rather than end users.

GIS software technology developed by ESRI (Environmental System Research Institute) uses the cloud today in several different ways. ArcGIS Server is a platform for delivering GIS services to software on other systems. These capabilities are exposed and in other ways, and they can be consumed by clients written using various technologies, including JavaScript, Adobe Flex, and Microsoft Silverlight. Geographic information providers, including government agencies and commercial sources, use geoportals to publish descriptions (geospatial metadata) of their geographic information. Geographic information consumers, professional, use geoportals to search and access the information they need. Thus geoportals serve an increasingly important role in the sharing of geographic information and can avoid duplicated efforts, inconsistencies, delays, confusion and wasted resources.

RESEARCH DESIGN

Promotion of data-sharing and utilization of geospatial information in local government

To promote the use of geospatial information for local government, it is necessary to create an environment where the information that is developed by various departments or sections at the local government can be easily searched, acquired and utilized. For this purpose, an entirely research framework for data-sharing and utilization of the geospatial

information has been established (Fig. 1). The required environments for such geospatial information will be collected, managed and utilized by users that should be developed and improved. A GIS based system which will facilitate the procedures for secondary use of geospatial information should also be created.

Enhancing the efficiency and quality of administration for regional planning development

Local governments are enhancing the efficiency and quality of their operations by utilizing geospatial information with GIS (Fig. 2). Local governments have already been using geospatial information to create various types of maps. However, by integrating and sharing basic map data, they can significantly reduce their mapping costs. Specifically, if administrative staff, sitting in front of their PCs, can compile and view administrative information from other sections, they will be able to respond more rapidly to residents' inquiries. If they can collect information by address and merge it with corresponding map information to gain a more accurate understanding of the situation, they will be able to provide better services, even when they go to respond to problems on-site. In this way, a GIS based system will be fully utilized to create WebGIS applications for sharing administrative information as geo-services thus they will be able to reduce the time they spend collecting and referencing documents.

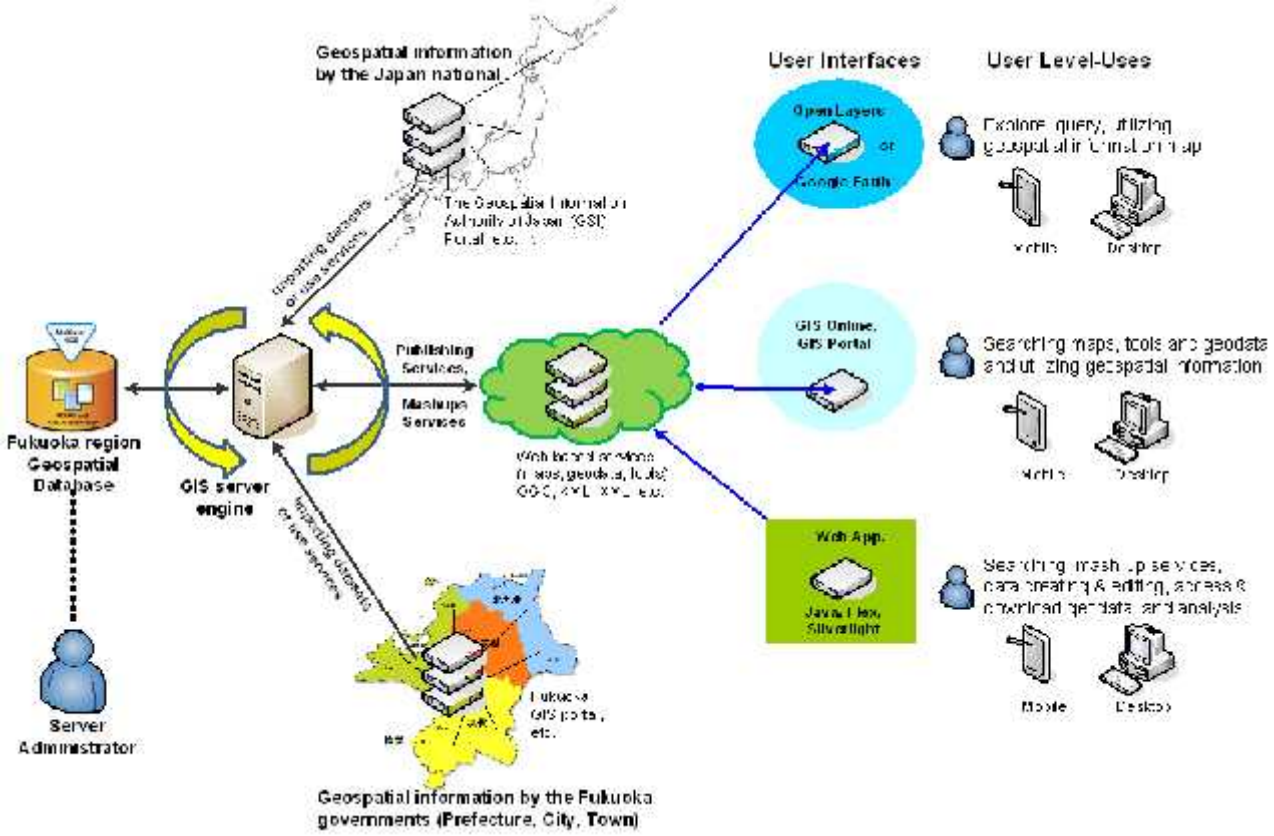

Fig. 1 Framework for development and utilization of geospatial information in the region (Indrayani et al., 2013)

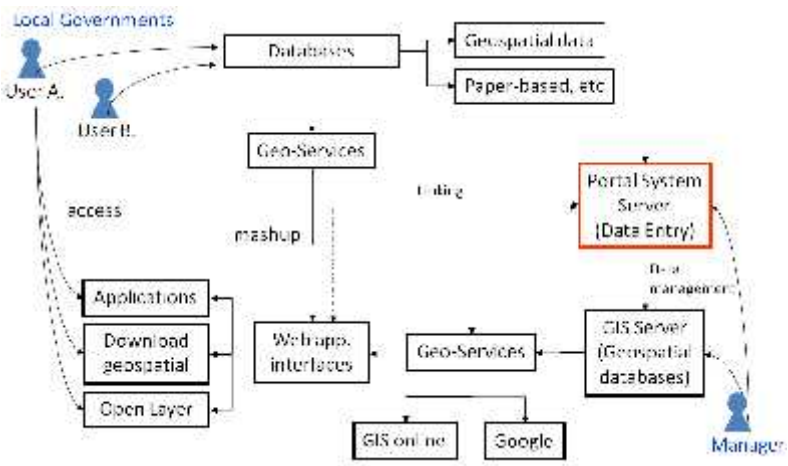

Fig. 2 Concept model of a GIS based system for collecting data-sharing data, and utilizing the geospatial information

Fig. 3 An example of the interface of the developed portal system using a NetCommons technology

RESEARCH METHOD

Given the above section of entire research design, a detail research will be performed to create and design an interface system which makes full use of the capabilities of a GIS, as follow.

Developing a GIS based system

A research is performed to establish a GIS based system interfaces for data-sharing and utilization of geospatial information in particular for local government. In this system, various types of geospatial datasets provided by local government and accumulated through the designed portal site which built by the GIS related technology (Fig. 3). A GIS web portal is most often one specially-designed Web page at a website which brings information together from diverse sources in a uniform way. NetCommons technology will be used that is an information-sharing support system that provides a user-friendly environment for constructing information portals, hosted databases, document management, data entry system, and more. Another component in the designing mechanism system is a GIS geodatabase server, the central data storage and management framework for the system platform. Storing geospatial data in the geodatabase enables to define advanced geospatial relational models and support many multiuser workflows. Geodatabases manage a complete information model of geospatial data in heterogeneous and scalable storage environments. These include DBMSs, such as Oracle, and Microsoft SQL Server. Furthermore, WebGIS is becoming an essential component of many GIS application solutions. GIS server lets share and publish geospatial data resources on the web as geo-services. To work with these geo-services, web client applications using the web mapping for Silverlight have been developed. Silverlight enable to easily configure and deploy a custom web mapping applications to users who are not GIS profession (such as local government's staffs).

Collecting, updating and managing the geospatial databases for designed region

In this study, various data collection from many sources has been performed to provide the fundamental geospatial information in the region. In particular, multi scale digital elevation data have been downloaded to provide high value of geospatial data. Each geospatial theme has been managed as an information set independent of other themes. Because the various independent themes are spatially referenced, they overlay one another and have been combined in a common map display. Therefore, to implement the data-sharing and utilization of geospatial information in the region, geospatial related datasets from the local government, etc. have been collected as a study case. Moreover, local governments develop and store various types of data for various administrative purposes, including a lot of geospatial

information datasets that is useful for many departments or for society in general. However, there is still insufficient progress being made in providing such geospatial information. It is thus necessary to digitize, analyze, and convert their datasets into geospatial information. To implement the data-sharing and utilization of geospatial information in region, geospatial related datasets from the local government, etc. have been collected, as a case study. Registered users (organization and department) can upload their datasets through portal site by selecting the data license categories (Open Data (OD), Closed Data for Organization (CDO) and Closed Data for Department (CDS)).

Data-sharing and utilization of the geospatial information services for regional planning study

Data-sharing and utilization of geospatial information services (geo-services) have been implemented in an organization such as local government, allowing different departments or sections in an organization to collaborate in the development and utilization of geospatial information. For example, a city's planning department (Section A), engineering and land information department (Section B) may each have their own geospatial information that supports each other's planning. The data-sharing and utilization of geospatial information allows each department to use its own geospatial information supporting other department-specific geospatial information and at the same time geospatial information services onto the network for users in other departments (Fig. 4). Geospatial information may be opened beyond the organization. Moreover, by combining the applications and geospatial information services, such as publishing a hazard information map, infrastructure management information map, etc., the organization can obtain the advantage on the utilizing the information effectively to enhance the efficiently of the organization for regional planning. To study further, the compilation of geospatial datasets is very significant to conduct the development of regional planning study. In currently research, the datasets from designed region have been collected to develop geodatabases and provide geo-services. Various datasets are provided from department's level and organization's level which covered several sub-study themes of related regional planning (such as disaster prevention and safety, environmental aspects, and city development and infrastructure) which converted into geodatabase scheme (Fig. 5).

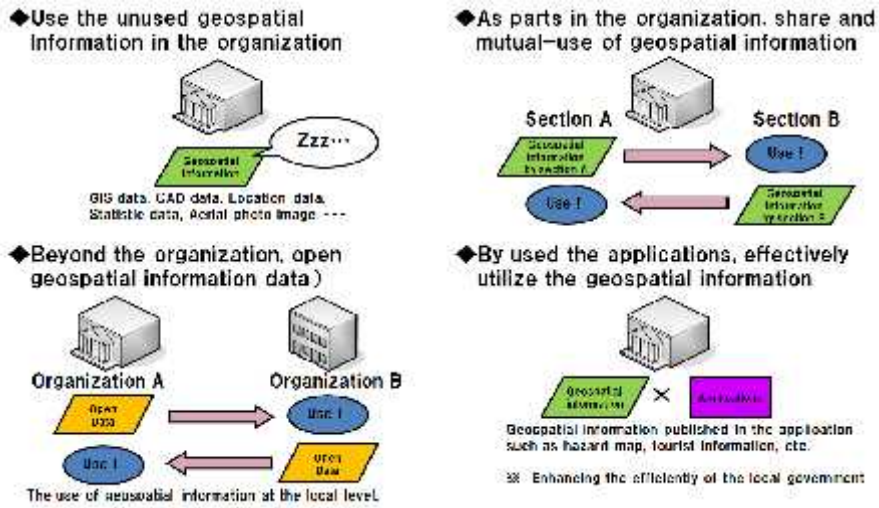

Fig. 4 Concept of the utilization of geospatial information in departments, sections, and organizations

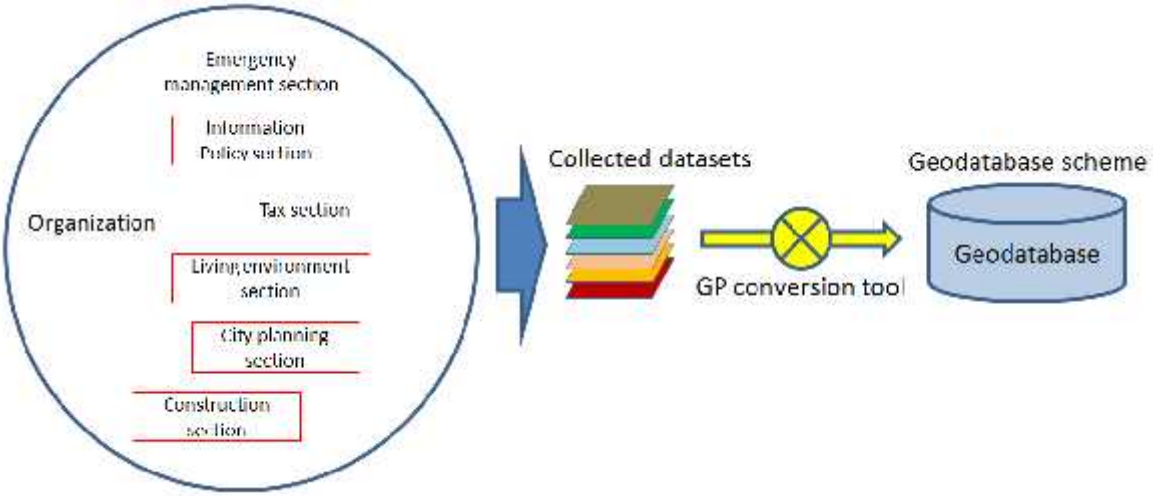

Fig. 5 Abstracting the migration datasets into the geodatabases by geoprocessing (GP) conversion tool

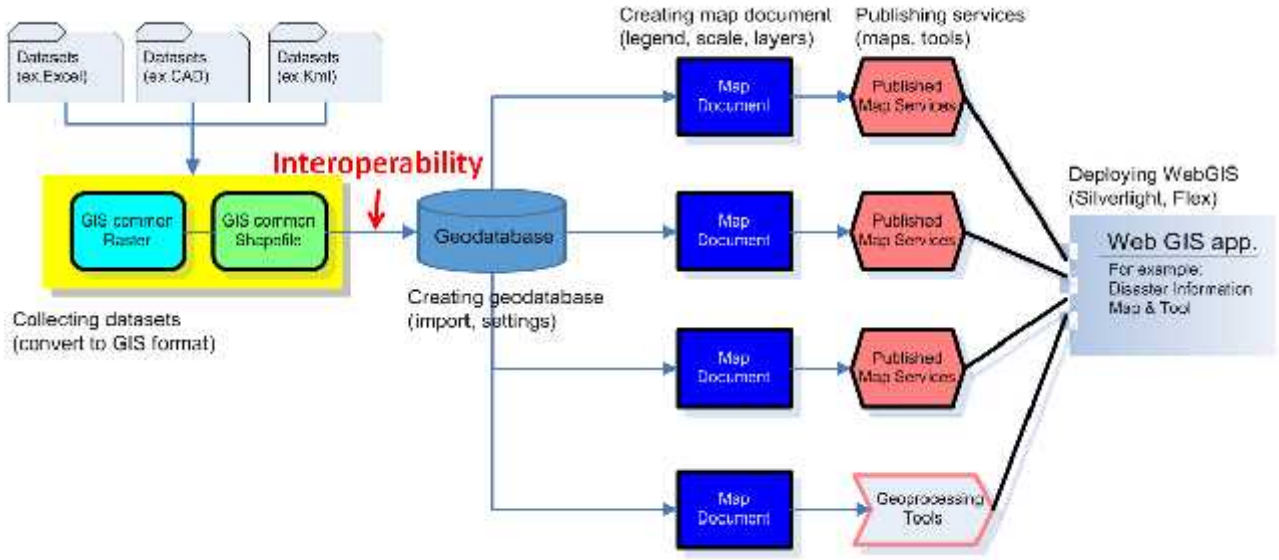

Fig. 6 Schematic flow of the registered datasets to the geo-services on the WebGIS application

However, the collected datasets from local government will mainly not be used for general public purpose, but the geospatial information related datasets will be used primarily in the local government such as organization's users and department/section's users. Datasets created by local government is not always a GIS common format, but frequently datasets are created in the Jpeg, Excel, CAD, pdf, and so on. All the datasets provided by local government are stored into geodatabase scheme in order to provide geospatial information as geo-services. Registered datasets provided by local government can be linked as geospatial information (open data or closed data) through WebGIS applications. Prior to publish the geospatial information by WebGIS application, the registered dataset are converted and stored into GIS geodatabase as common format. Therefore, various map documents as geo-services can be created from GIS datasets in the geodatabase. WebGIS applications have been customized following required theme of applications such as disaster prevention and safety, database editing and updating which related to regional planning development (Fig. 6).

Application model of disaster prevention and safety related to regional planning study

Fig 7 shows the illustration of the WebGIS application model that contained the several geo-services such as hazard related maps, field photo surveys, slope detail information as the pdf attached file, and aerial photographs to give the geospatial information related landslide prone areas.

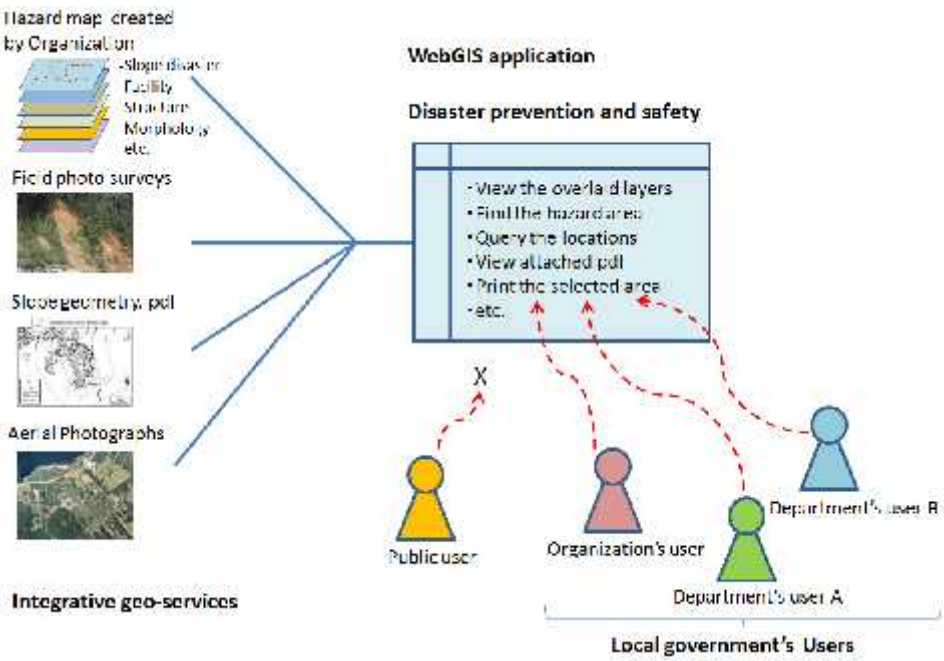

Fig. 7 Illustration of a model for disaster prevention and safety

Fig. 8(a) Interface shows the geospatial information related to the attribute tables and photos

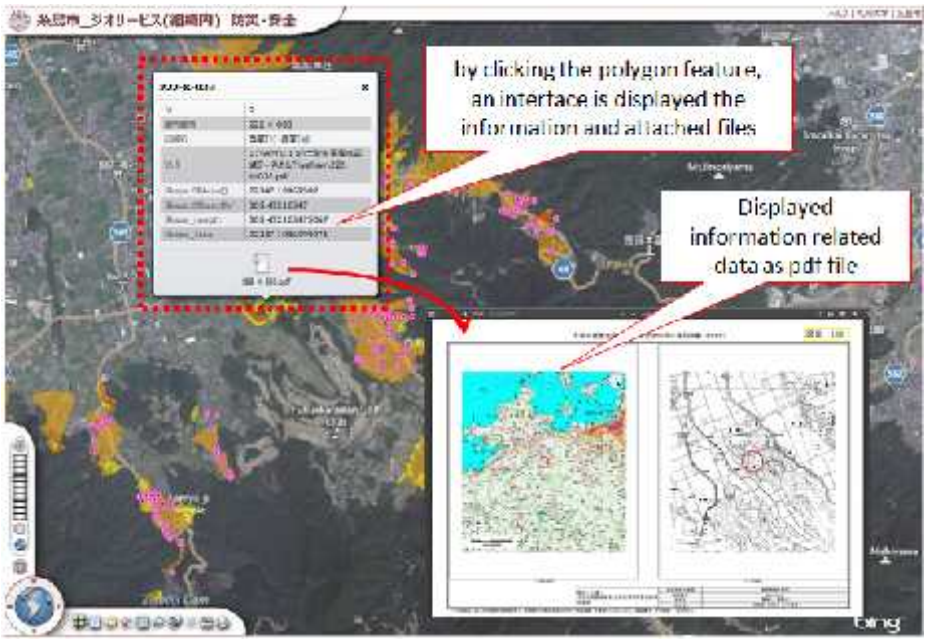

Fig. 8(b) Interface shows the geospatial information related to the additional scanned paper maps

In this case, geospatial information combined by several datasets such as base maps, photos from field surveys, slope hazards detail map papers, etc. which displayed by the designed WebGIS application to give integrative information to the disaster prevention and safety on the regional planning development study. As shown in Fig. 8 (a) to (b), the designed interfaces of WebGIS application as a model for regional planning maps related to the disaster prevention and safety information. Local government can access the geospatial information and explore the data related regional planning information such as hazard area and slope detail maps, photos related to the observed point at field, etc. Moreover, attribute data of geospatial information can be displayed as an excel table in this application.

Application model for the creating and updating the geospatial data related to the regional planning study

Fig 9 shows the illustration of the application model for creating and updating the geospatial data through WebGIS application and mobile. Editing and updating the geospatial data can be done by multiple users simultaneously. Using base map geospatial information such as updated aerial photographs as a reference map, member users can update their geospatial data related to the regional planning study by editing, updating and creating features to obtain new digital data. Furthermore, the geospatial related files such as image, pdf, word, etc. can be attached into the updated information data. Fig. 10(a) depicts interfaces of WebGIS application to create feature geodatabases (point, polyline,

and polygon) and update the attribute table and attach the files. Furthermore, Fig. 10(b) shows interfaces of mobile to create, update, and upload the geospatial data related to regional planning development study.

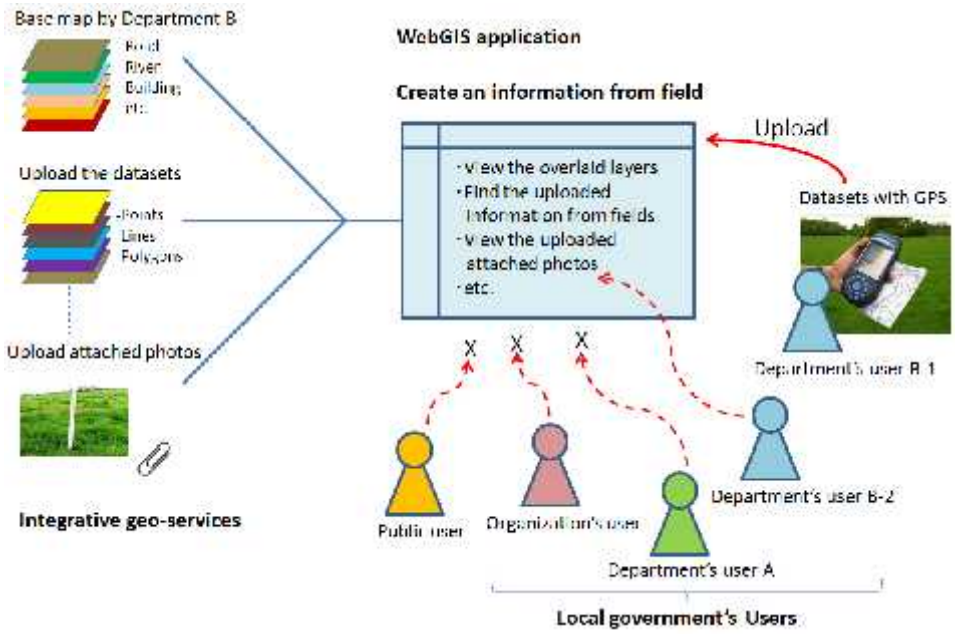

Fig. 9 Illustration of a model for creating and updating the geospatial information through the WebGIS application

Fig. 10(a) Interface model shows the created polygon feature, attribute information, and attached files

Fig. 10(b) Interface model of mobile application (smartphone) shows the process to access the maps through internet and to upload features (geometry and attribute) to the designated features of geodatabase

EFFECTS OF DATA-SHARING AND UTILIZATION OF GEOSPATIAL INFORMATION TO THE REGIONAL PLANNING DEVELOPMENT

The value to the policy decisions

In view of the fact that it is possible to perform superimpose of various geospatial information services (geo-services) and to conduct spatial analysis by the developed WebGIS application. Therefore, the identification of the problems in the region are easily carried out by clarification of the issues by means of the visualization of overlaid distribution information (spatial data, statistical data, etc.), thus effective policy decisions and regional planning can be obtained. By utilizing the designed system in local government, the geospatial information can be collected from various departments in the organizations, and integrative problems across departments can be assessed effectively.

The value to the quality administrative services

The combination of geospatial information with new technologies of developed WebGIS application and mobile devices such as smartphones that are quickly coming into widespread use can help to increase the quality for example, providing easy access to a high level of useful geo-services and information. Furthermore, using the GIS function tools of the designed system (portal and WebGIS application), government staffs can quickly search the information rather than search information of digital papers, manually. Therefore, benefits can be obtained such as cost saving, faster and more intelligent delivery of services to the regional planning information.

The value to the new public benefits

Easy public access to the most current and relevant geospatial data provides a number of other societal benefits as well. One such benefit is economic, because reference to the information contained in geospatial data greatly facilitates economic improvements. For example geospatial data enables online WebGIS mapping services such as geo-services. Greater support of self service through use of WebGIS application will reduce time spent by the general public finding information in the region.

CONCLUSIONS

In this paper, a GIS based system and geo-services have been designed to facilitate the users to access and manage their data in the region. Furthermore, WebGIS applications have been established to provide an easy-to-use application for local government and general public to utilize various fields of the geospatial information to study regional planning development, effectively. Various types of geospatial datasets from local government have been collected and updated into the geodatabase scheme to provide new digital data. A case study of data-sharing and utilization of geospatial information in the designed region have been conducted to evaluate the effect of the GIS based system to the regional planning development study. Consequently, the developed GIS system can provide an easy mechanism for delivering internet-based geospatial information services and provide faster and more intelligent ways for developing regional planning maps.

ACKNOWLEDGEMENTS

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THE ANALYSIS OF SERVICE DEMAND OF FERRY TRANSPORT ROUTE KENDARI – WAWONII POST REGIONAL DIVISION AT REGENCY KONAWE ISLANDS

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ABSTRACT: The need of transportation service is result of interaction between social activity and economy in spaces. The spread of activities and interaction pattern that complex raise transportation problems that is very diverse and there are many factors as determinant that should be considered. The increasing of passengers flow trough the sea and the ferry transport as a result of the rate of national development and equitable development results throughout all corners of the country, need of crossing between the island and the harbor are also increasing. Therefore, optimization of the sea transportation as a step anticipation of a possible surge in passenger absolutely done. The purpose of this research is to analyze the transport demand condition and analyze the difference of ferry transport demand condition before and after division into territories and then, to analyze the operational performance of ferry transport with route Kendari - Wawonii. The result of this research showed that the condition of demand transport crossroad Kendari - Wawonii after division into territories is higher than before the extraction, except the number of boat trips. Level of service on the ship at the third classification, the needs of crossroad transporter number after expansion is 2 transporter with frequency twice in a day, the level of dock service, parking area is still able to serve transport operations (PAOR = 4,6 %) but needs improvement on means of supporting the pier has been damaged. Terminal service levels has been unable to serve passengers operational where, the needs of the lounge at terminal need to be developed at month-25, because it is beyond the capacity of the lounge (TOR = 111,11%). Increasing model of the passengers number $Y = 7{,}693X^2 – 71{,}27X + 920{,}7$ with $R^2 = 0{,}671$, beside that the model of on increasing number of vehicle $Y = 1{,}280X^2 – 4{,}571X + 227{,}8$ with $R^2 = 0{,}617$. Based on the results of Chi-Square test showed that the model of on increasing number passengers and vehicle route Kendari-Wawonii are significant. Load Factor of ferry was 0.99 in month-18th.

Keywords : *Regional Division, Ferry, PAOR, TOR, Load Factor, Chi-Square.*

INTRODUCTION

According Jinca (2003), the increasing flow of passengers by sea and ferry transport as a result of the pace of national development and the equitable distribution of the fruits of development to all corners of the country, the need for inter-island crossing trajectories and between ports has increased as well, therefore the optimization of fleet as anticipatory measures of the possibility of a surge in passengers was to be conducted by the Department of Transportation. It is specifically to serve the journey through the waters between the islands, not only provided but also freight ships crossing the sustainability journey on the road.

As an archipelago, the ferry transport in Konawe Islands is a vital necessity in the post-expansion support regional development. Progress of ferry transport may encourage the smooth flow of goods and services as well as increasing the mobility of people, especially in remote areas.

Kendari ferry transport harbor - Wawonii (Konawe Islands) is one of the trajectories of the 8 track crossing port in Southeast Sulawesi, operated since 1996 for connecting the island with the city of Kendari - Wawonii. To support the operation of ferry transport, has built the infrastructure which is 1 (one) ports, one (1) track and navigable by 1 (one) fleet of ferries in normal conditions as well as two (2) up to 3 (three) fleet of ferries on certain conditions (homecoming season).

RESEARCH PURPOSES

The purposes of this study are:

1) To analyze the condition of the ferry transport demand Kendari - Wawonii before and after the regional divisions.
2) To analyze the different transport demand conditions between before and after the regional divisions at Kendari – Wawonii route.

[1] Lecturer on Technical Faculty, Civil Departement, Halu Oleo University – Kendari

3) To analyze operational performance ferry transport routes.

LITERATURE REVIEW

Transport and Regional Development

Morlok (2000) suggested that because of differences in the level of ownership of resources and the limited ability of the region to support the needs of the population of an area leads to the exchange of goods, people and services between regions. This exchange begins with the process of supply and demand. As a tool in the process of supply and demand needs to be delivered towards other areas necessary means of transport. Means of transport that allows to assist mobility in the form of public transportation.

Movement of people and goods from one place to another is always through certain lines. The place of origin and destination are connected to each other by a network (network) in the space. The network can be a network of roads, which is part of the transportation system. Transportation is important in a system, because without transport nexus between one place to another place not materialize either (Bintarto, 1982) in Mandaku (2010).

Ferry Transport

Transportation or travel is not only applicable on land but also in the waters or rivers. According to Law No. 17 of 2008 states that transport streams, lakes and boat transportation crossing is done in rivers, lakes, reservoirs, swamps, Anjir, the canal and the canal for the transport of passengers, goods and / or animals held by the company and river transportation lakes which are arranged in an integrated implementation of intra and inter-which is an integral national transportation arrangements as well as using a regular fixed routes and the route is not equipped with fixed and irregular.

Port

Ports are nodes of the chain of the transport system as well as a gateway, especially for sea transport in the framework of the traffic of goods, movement of passengers and animals. Thus the port has a role and functions in supporting economic growth. According to (Jinca, 2003) explains that the port as a haven of ships - ships should be a safe place, so that loading and unloading can take place to ensure the security of the goods to be unloaded or loaded at the port. of subsystems transportation, port is the node of the chain for the smooth sea and land transportation.

The port has several important roles. As for the role of the port according to (Nasution; 2008) are as follows:

1) To serve the needs of international trade of the region (hinterland) where the ports are located.
2) Assist the wheels of trade in the development of regional industry
3) Hold the increasing market share of traffic (traffic) is an international, both transshipment and goods (inland touthing).
4) Provide a transit facility for the rear area (hinterland) or local / neighboring countries.

Transport Performance

Jinca (2003) explains that there are four major benefits for society for transport infrastructure, namely:

1) Open the isolation region and the region
2) Increasing economic activity and support the smooth region
3) Facilitate access to technology and the utilization of social facilities
4) Increased mobility and social contact between people.

Service performance boats are things that must be considered, because many things that greatly affect the satisfaction and safety of the passengers. Suwarto (2009) says there are several things that affect performance on a port, among others, infrastructure, accessibility and human resources. For passenger satisfaction levels, aspects - aspects that are measured, namely the scheduled departure time, rates, security, convenience and availability of facilities.

THEORETICAL BASIS

Forecasting Movement

Transportation forecasting method were analyzed by quantitative methods that process quantitative data using a mathematical approach, the model used in forecasting demand for transportation services is a model compound interest with equation (Kamaluddin; 2003):

$$Pn = Po\,(1 + r)^n \tag{1}$$

$$r = \left(\frac{Pn}{Po} - 1\right)^{\frac{1}{n}} \tag{2}$$

Where:

Pn = Number of movements in the future
Po = Number of movement at the present time
r = Growth factor
n = Difference in time

The Concept of Ferry Transport Development

Ferry transport development concept includes several things:

1) Site Selection Port

Crossing the harbor site selection should consider the following matters:

a) The condition consists of Depth Coastal waters, Dominant Wave Height, Space Motion, Flow Speed Dominant, tidal, Shipping Channel;
b) the condition of the Mainland. consists of Accessibility, Land Status, Land Condition and supporting facilities;
c) the distance between the candidate port locations with potential regional suppliers of flow of passengers and goods;
d) common types of ports and basic facilities that could be provided, and
e) benefits to the area.

2) Operational Patterns include (Nasution, 2008):

a) Type of ship

Based on the condition of the waters of Indonesia, JICA-Japan International Cooperation Agency (1993) classifies ships crossing into 5 (five) types as presented in Table 1.

Table 1. Types of Ships Crossing

Types of Ships	GRT	LOA (m)	B (m)	A (m^2)	Capacity		FD (m)	SPD (knot)
					2-T	Passengers		
A	1000	70	14	686	27	600	3,5	16
B	500	47	11.5	378	15	500	2,6	14
C	500	39	10.5	278	11	300	2,2	11
D	500	39	10	273	11	300	2,4	14
E	150	30	8	168	7	100	1,5	11

Source: Nasution, 2008

Where:

GRT : *Gross Register Tonnage*
LOA : *Length Over All*
B :*Breath*
A :*Effective Loading Deck Space*, untukmengakomodasikendaraan
2-T : *Truck*, 2 Ton (25 *m²/ truck*)
FD : *Draught in Full Load*
SPD : *Speed*

b) Ship Frequency

Formula to calculate the frequency of ships crossing, are:

$$FK = \frac{\sum SUP\ d \quad / hari}{\sum SUP\ s \quad / hari} \tag{3}$$

where:

FK : Frequency Ship
SUP : Production Unit
d : demand
s : supply

c) Travel Time

$$Tsea = \frac{S}{SPD} \tag{4}$$

where:

SPD : Speed boat
s : mileage
Tsea : Travel Time

d) Total of Pier

Formula for calculating the amount of dock required, are:

$$D = \frac{FK}{\frac{Pelabuhan}{Tops}} x[Tsea + Tport] \tag{5}$$

where:

D: Total Pier
Tport: Time on the harbor
Tops: Operational time

e) Total of Fleet
Formula to calculate the number of required fleet is:

$$K = \frac{\sum Trip\ x \quad [Tsea + Tport]}{Tops} \tag{6}$$

where:
Σ trip : The number of boat trips
T_{sea} : Travel Time
T_{port} : Time on the harbor
T_{ops} : Operational time

The size and type of vessel be approached by two quantities based on the size of the request and based on the demands of safety and security of shipping which is a function of the condition of the waters along the shipping lanes. Type of ship approached from characters demand (passenger and freight) will be served.

To get the most optimal vessel size is by reviewing on two things: the amount of demand for transport and natural conditions in the shipping channel. Both of these are compared and the conditions that generate greater revenues used the basis for determining the type of ship.

a) Based on the current demand
By using the numbers traveling population projections and the flow of goods, it can be estimated ferry transportation needs.
b) Based on the natural conditions
It is the basis for selecting the type of vessel based on the natural conditions are the following types of waters, shipping distance, travel time and speed of currents.

Transportation system performance indicators should be monitored continuously in order to know the position of demand and supply system, when the demand is getting closer to the supply, the more critical the situation. The information needs to be collected include:

1) The growth of transport demand, as seen from the historical data for existing services, including the need to know the seasonal factor, whether the growing season, harvest season, rainy season, dry season or related factors with school holidays or during the day religious.
2) The level of use of infrastructure, to see the extent to which the existing load factor, whether it has reached a critical point. The critical point in the range of loading factor between 0.7 and 0.8, when it is the figure needs to be a breakthrough so there is no queue at peak currents transport.
3) The waiting time is one indicator of essential services, long waiting periods to indicate the occurrence of capacity problems in the provision of service.
4) Speed of service is another indicator related to the efficiency of the service system.
5) The length of the queue is another indicator of the transport service, when long queues, it means that the waiting time is long.

SERVICE INDICATORS IN PORT

Terminal Ports

Terminal or waiting room at ferry ports and serves as a waiting room for the arrival of passengers at the ferry ports so that the capacity of a terminal is used as it should be then used analysis Occupancy Ratio terminal. Terminal Occupancy Ratio is obtained by the equation:

$$TOR = \frac{\sum ST}{S} x100\% \tag{7}$$

where:
TOR = Terminal Occupancy Ratio
ΣST = number of seats occupied in the terminal / day
S = The number of seats available

Vehicle parking area

Vehicle parking area (Parking Area Occupancy Ratio) is to measure the capacity of parking in the port ferry transport is obtained by the equation:

$$PAOR = \frac{\sum AP}{ALP} x100\% \tag{8}$$

where:
PAOR = Parking Area Occupancy Ratio
ΣAP = Number of parking area / day
ALP = area parking lot
The value of the vast amount of parking is calculated by the equation:

$$\sum AP = JK \times SRP \quad (9)$$

where:
JK = Number of vehicles by type of vehicle
SRP = Unit Parking Space

Parking space unit (SRP) is a parking space for one vehicle or the size of the effective area for putting the vehicle, including free space and wide opening doors. SRP is a measure of the need for space to park a vehicle in a safe and comfortable with the amount of space that is as efficient as possible (Munawar; 2000).

Transport dock Crossing

A very important role in the mooring dock a ship that will dock at a port, the level of ability of a pier should always be measured in order to know the ability of a dock. In analyzing a dock that can be used as it should be then used BOR (Berth Occupancy Ratio). BOR is obtained by the equation:

$$BOR = \frac{\sum TK}{T} x100\% \quad (10)$$

where:
BOR = Berth Occupancy Ratio
ΣTK = Total Time Mooring whole Ships
T = Time Available / Day

The Testing of Statistics

To sharpen the analysis of the data examined, it will use two approaches statistics, the formula approach and non linear regression and chi-square formula. Chi-Square used to test the significance of differences in frequency. Chi - Square calculated by the equation:

$$X^2 = \sum \frac{(f_0 - f_e)^2}{f_e} \quad (11)$$

X^2 = Chi-square Value
f_0 = observation Frequency
f_e = expectations Frequency

The result of the equation X2 X2 4:11 compared with Table. The requirements are significant differences in outcome X2 is greater than X2 table, while the regression model approach is based on the basic equation as follows:

$$Y = A + BX \quad (12)$$

where,
Y = Dependent Variable
A = Constant
B = coefficient of Variables
X = Independent Variable

RESEARCH METHODS

Location and Time Research

This research is located in the Port Crossing Wawonii in Kendari. The timing of the research takes as long as four months, from November 2014 until February 2015.

Data Sources and Data Collection Techniques

Source of data required in this research are:

a) Primary data is data obtained directly in the field through observations in the form of the condition of the port infrastructure, accessibility conditions, conditions of human resources, service conditions include a mooring the vessel, the number of passengers and the number of vehicles.
b) secondary data, i.e. data obtained from the relevant agencies in accordance with the scope of research in the form of publications supporting data from the Central Bureau of Statistics (CBS) covers the geography and demographic conditions, as well as the Department of Transportation and Southeast Sulawesi province ASDP include passenger traffic in the period the last 5 years, the flow of vehicles in the last 5 years, the number of trips of vessels in the last 5 years, the speed of travel, means (the number and capacity of vessels) and port infrastructure crossings (main facilities and support facilities) as well as literature references scientific works.

The Variables of This Research

The variables were used in the study can be seen in table 2 below.

Table 2. Variables, Indicators and Parameter of Research.

Elements that are reviewed	Indicators	Parameter	Data Source	The Analysis Method	Formula
Demand for transport services	Trip of ship	- Number of passengers - Number of vehicles - The growth rate of boat trip	ASDP Kendari	Double Flower Model	1 dan 2
Frequency of ferry transport demand	Frequency of ship	- Number of vessels travel request - Total availability	data analysis result	quantitative statistical methods	3
	Total of ship	- Number of vessel trip - Time in the harbor - Time ship operational	data analysis result		6
	Level of service (BOR, TOR PAOR)	- The number of passengers - Number of vehicles - Berthing Time	data analysis result		7,8,9,10

Source: Design Research, 2015

RESULTS AND DISCUSSION

Transport Demand Crossing Before Regional Divisions

The Ferry transport demand conditions route Kendari- Wawoniibefore division were reviewed the amount (boat trip), the number of passengers and the number of vehicles during the five (5) years, namely 2008 to 2012. Data ferry transport demand growth during the five years last before the division presented in Table 3.

Table 3. Demand Ferry transport Development Conditions before Expansion

No	Years	Vessel Trip	Passenger	Vehicle
1	2008	339	13999	1968
2	2009	338	15327	2676
3	2010	334	13097	2102
4	2011	335	12159	2240
5	2012	337	13260	3202
Total		**1683**	**67842**	**12188**
Average		**337**	**13568**	**2438**

Source PT.ASDP Indonesia Ferry (Persero), 2015

Table 4.The Percentage of Demand Ferry transport Before Regional Divisions

No	Years	Vessel		Passenger		Vehicle	
		Trip	Growth Rate	Total	Growth Rate	Total	Growth Rate
1	2008	339	0.00	13999	0.00	1968	0.00
2	2009	338	-0.29	15327	9.49	2676	35.98
3	2010	334	-1.18	13097	-14.55	2102	-21.45
4	2011	335	0.30	12159	-7.16	2240	6.57
5	2012	337	0.60	13260	9.06	3202	42.95
Total			**-0.58**		**-3.17**		**64.04**
Average			**-0.12**		**-0.63**		**12.81**

Source: Data Analysis, 2015

Based on Table 3 shows that the demand for these vessels Kendari - Wawonii before the division experienced growth fluctuated from year to year where the demand for ships on average decreased by 0.12% and demand for vessels for 5 (five) years were highest in 2008 amounting to 339 trips per year, while demand for ships lowest occurred in 2010 amounted to 334 trips per year, as well as based on Table 4 shows that the demand for these passengers Kendari - Wawonii before division also experienced growth fluctuated from year to year where passenger demand on average

decreased by 0.63% and passenger demand for 5 (five) years were highest in 2009 amounted to 15 327 people per year, while passenger demand is lowest occurred in 2011 amounted to 12 159 people per year.

Based on Table 4 shows that the demand for these vehicles Kendari - Wawonii before division increased growth from 2008 to 2009 amounted to 35.98% and in 2010 to 2011 of 6:57% and from 2011 to 2012 amounted to 42.95%, while from 2009 to year 2010 decreased growth in demand for vehicles by 21:45% where vehicle demand on average increased by 12.81% and demand for vehicles during 5 (five) years were highest in 2012 amounted to 3202 units per year, while the lowest occurred vehicle demand in 2008 amounted to 1968 units per year.

Transport Demand AfterRegional Divisions

The ferry transport demand conditions Kendari - Wawonii be reviewed after the expansion is the number of (boat trip), the number of passengers and the number of vehicles during the two (2) years in which the conditions of the development of ferry transport demand for the development of distributed annually per month that in 2013 to 2014.

Total demand for vessels (boat trip) on average over the last 2 years at 30 trips per month (stagnant) while the number of passenger demand by an average of 1276 people per month and the amount of the average vehicle demand over a period of twoyearsLast amounted to 364 units per month.

Table 5. Growth Rate Request Transportation Crossing Expansion After the period of Januaryuntil December 2013

No	Month	Vessel		Passenger		Vehicle	
		Trip	Growth Rate	Total	Growth Rate	Total	Growth Rate
1	Januari	31	0.00	1040	0.00	270	0.00
2	Februari	28	-9.68	1005	-3.37	264	-2.22
3	Maret	31	10.71	336	-66.57	103	-60.98
4	April	27	-12.90	760	126.19	235	128.16
5	Juni	30	11.11	465	-38.82	148	-37.02
6	Juli	30	0.00	714	53.55	184	24.32
7	Agustus	31	3.33	1228	71.99	481	161.41
8	September	30	-3.23	676	-44.95	210	-56.34
9	Oktober	31	3.33	1080	59.76	361	71.90
10	November	30	-3.23	1250	15.74	376	4.16
11	Desember	31	0.00	1081	0.09	322	-10.80
Total		**330**	**-0.54**	**9635**	**173.62**	**2,954**	**222.58**
Average		**30**	**-0.05**	**98**	**15.78**	**269**	**20.23**

Table 6. Growth Rate Request Transportation Crossing Expansion After the period of January until November 2014

No	Month	Vessel		Passenger		Vehicle	
		Trip	Growth Rate	Total	Growth Rate	Total	Growth Rate
1	Januari	31	0.00	1313	0.00	404	0.00
2	Februari	28	-9.68	931	-29.09	327	-19.06
3	Maret	31	10.71	1224	31.47	355	8.56
4	April	30	-3.23	1263	3.19	338	-4.79
5	Mei	31	3.33	1121	-11.24	288	-14.79
6	Agustus	31	0.00	2759	146.12	682	136.81
7	September	30	-3.23	2670	-3.23	720	5.57
8	Oktober	31	3.33	2817	5.51	676	-6.11
9	November	30	-3.23	1796	-36.24	529	-21.75
Total		**273**	**-1.97**	**15894**	**106.48**	**4,319**	**84.44**
Average		**30**	**-0.22**	**200**	**11.83**	**480**	**9.38**

The ferry transport demand on the route Kendari - Wawonii after the division experienced growth fluctuated from month to month in which the vessel demand on average decreased by 0.05% in 2013 and 0.22% per month in 2014.

The route of passenger demand Kendari- Wawonii after the division experienced growth fluctuated from month to month where passenger demand on average increased by 15.78% per month in 2013 and amounted to 11.83% per month in 2014. The number of requests the highest passenger occurred in November in the year 2013 amounted to 1,250 people per month, and in October 2014 amounted to 2817 people per month while the lowest number of passenger demand that occurred in March of 2013 amounted to 336 people per month, and in February 2014 amounted to 931 persons per month.

Passengers data in August to December 2013 and from August to November 2014 showed a significant number of passengers compared to the previous months, it is caused by the activities of implementation of development projects in Konawe Islands impacting on the level of labor mobility from outside Konawe high enough Islands. In addition, from

August to November 2014 also occurred activities receipt Selection CCS category 2 (Candidate for Civil Servants). As for the month of May in 2013 and in June and July 2014, the process of docking ships carried as ship maintenance efforts, so that the data vessel trip, passenger demand and a vehicle can not be included as a sample because these data affect the normality of the data (data outliers) that may affect the accuracy of predictions of the model were made.

Transport Demand Model

Passenger Demand Model

Model development of passenger numbers after the Kendari-Wawonii polynomial division following the model where the model equation is: Y = 7,693X2 – 71,27X + 920.7.

Based on the coefficient of determination (R2) is then every month from January 2013 until November 2014, after the division of influence and the development occurred passenger numbers. The greatest effect 67.10%. The graph of the model number of passengers can be seen in Figure 1.

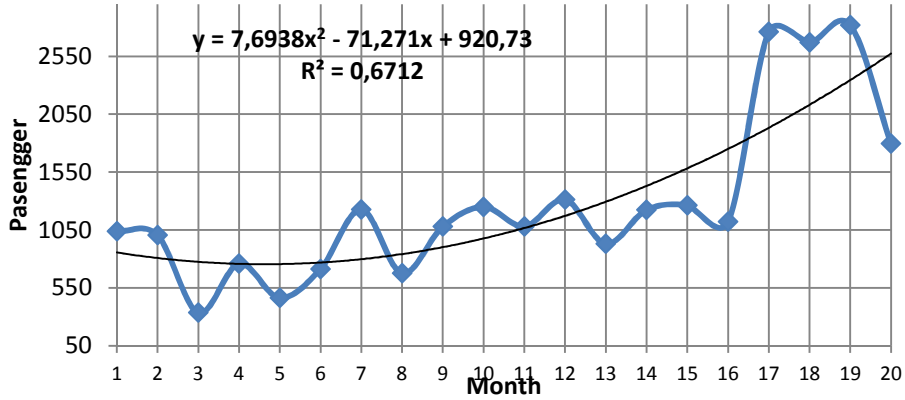

Figure 1. The PolynomialModelof Passengers Numbers Developments After Expansion

Demand Model Vehicles

The Model of the number of vehicles Kendari-Wawonii after splitting follow the polynomial equation is: Y = 1,280X2 - 4,571X + 227.8

Based on the coefficient of determination (R2) is then every month from January 2013 until November 2014, after the division of influence and there is a development of vehicles. The coefficient of determination are 61.70%. Graph of the model number of the vehicle can be seen in Figure 2.

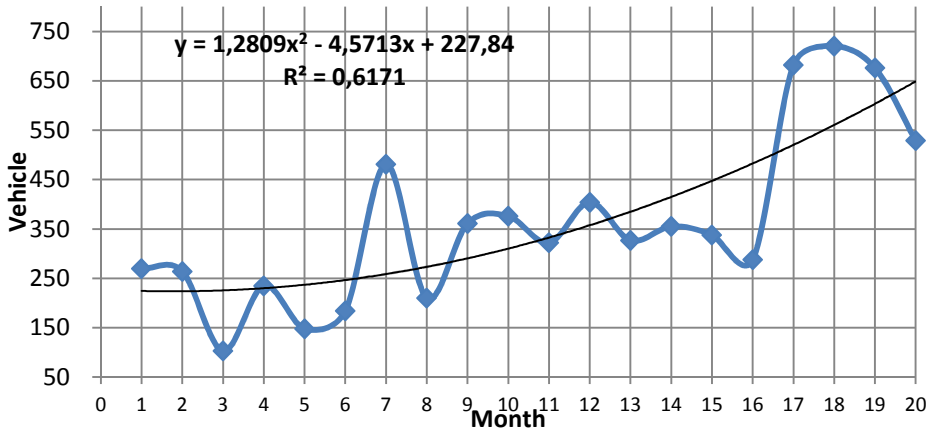

Figure 2. The Polynomial Modelof Total Vehicle Developments After Expansion

The recapitulation of the state of progress of the ferry transport demand Kendari - Wawonii both before expansion and after the expansion are described in Table 7.

Table 7. Comparison of Ferry Transport Demand Before and After Expansion.

No	Description	Before Expansion	After Expansion
1	Number vessel	28	30

	trip (trip / month)		
2	Number of passengers (person / months)	1131	1276
3	Number of vehicles (units / month)	204	364
4	Passenger demandModel	$Y = 59{,}85X^2 - 24109X + 2x10^8$; $R^2 = 0{,}395$	$Y = 7{,}693X^2 – 71{,}27X + 920{,}7$; $R^2 = 0{,}671$
5	Vehicle Demand Model	$Y = 87{,}14X^2 - 35011X + 4x10^8$ $R^2 = 0{,}512$	$Y = 1{,}280X^2 - 4{,}571X + 227{,}8$; $R^2 = 0{,}617$

Source: Data Analysis, 2015

To determine whether there is a difference between the conditions of ferry transport demand before and after the division were analyzed by chi-square technique. The hypothesis is:

Ho : There is no difference between the conditions of transport demand crossing before splitting withafter the Expansion.

Ha : There is a difference between the conditions of transport demand crossing before splitting withafter the Expansion.

Results of The recapitulation of ferry transport demand before and after the division presented in the table below.

Table 8. Summary of Transport Boat Trip Request crossing before and after Proliferation

Boat Trip Request	**f_0**	**fe**	**(fo-fe)**	**$(fo-fe)^2$**	**$X^2 = (fo-fe)^2 / fe$**
Before Expansion	28	29	-1	1	0.03
After Expansion	30	29	1	1	0.03
Total	**58**	**58**	**0**	**2**	**0.07**

Source: Data Analysis, 2015

Table 9. Summary of Passenger Travel Demand in Ferry transport before and afterRedistricting

Passengers traveling	**f_0**	**fe**	**(fo-fe)**	$(fo-fe)^2$	**$X^2 = (fo-fe)^2 / fe$**
Before Expansion	1131	1203.5	-72.5	5256.25	4.37
After Expansion	1276	1203.5	72.5	5256.25	4.37
Total	**2407**	**2407**	**0**	**10512.5**	**8.73**

Source: Data Analysis, 2015

Table 10. Summary of the Transport Vehicle Travel Demand crossing before and afterRedistricting

Mode Demand	**Fo**	**fe**	**(fo-fe)**	**$(fo-fe)^2$**	**$X^2 = (fo-fe)^2 / fe$**
Before Expansion	204	234	-30	900	3.85
After Expansion	264	234	30	900	3.85
Total	**468**	**468**	**0**	**1800**	**7.69**

Source: Data Analysis, 2015

Table 11. Comparison of the X^2_{result} with X^2_{table}

Type of Demand	**Hypothesis Decisions**	**X^2**	
		Calculation Results	**Tabel**
Boat Trip	Ho Received	0.07	3.841
Passengers	Ho Rejected	8.73	3.841
Vehicle	Ho Rejected	7.69	3.841

Source: Data Analysis, 2015

Pattern Operations and Services at the Port Crossing

Frequency

The number of passenger demand period November 2014 that a maximum of 144 people / day and total vehicle demand of 33 units per day where the number of passenger loading capacity of 72 people per day and the loading capacity of the vehicle by 6 units so that the frequency of ship 2 times / day.

Number of Ships

Known travel time (T_{sea}) for 210 minutes, time in port (T_{port}) for 35 minutes, the operational time (TOPS) for 258 minutes, so that the number of vessels by 2 fleet.

Number of Passengers

From the analysis of Load Factor (LF) at Table 12 can be concluded that at month 18, the capacity of ferry transport experiencing maximum points, so that in the 19th to be considered for addition vessel trip.

Table 12. Model-Based Development of Number of Passengers polynomial

Month	Total Daysin the Month	Passenger Data Preliminary	Monthly Data Model	Daily Data Model	**LF**
1	31	1040	857.12	27.65	0.38
2	28	1005	808.93	28.89	0.40
3	31	336	776.13	25.04	0.35
4	27	760	758.71	28.10	0.39
5	30	465	756.68	25.22	0.35
6	30	714	770.03	25.67	0.36
7	31	1228	798.77	25.77	0.36
8	30	676	842.89	28.10	0.39
9	31	1080	902.40	29.11	0.40
10	30	1250	977.30	32.58	0.45
11	31	1081	1067.58	34.44	0.48
12	31	1313	1173.25	37.85	0.53
13	28	931	1294.31	46.23	0.64
14	31	1224	1430.75	46.15	0.64
15	30	1263	1582.58	52.75	0.73
16	31	1121	1749.79	56.44	0.78
17	31	2759	1932.39	62.34	0.87
18	30	2670	2130.37	71.01	0.99
19	31	2817	2343.74	75.60	1.05
20	30	1796	2572.50	85.75	1.19

Note: The capacity is 72 people and unloading of ships passengers

a. Service Level Pier

Based on the survey results obtained by the amount of time the ship mooring average of 35 minutes and note the time available per day (T) for 258 minutes so that the level of service dock (BOR) is 13.57%. This means that the service dock at the port of Kendari-Wawonii these crossings are still available for the present.

b. Parking Service Level

The number of vehicles parking for passenger cars by 1 unit, 1 unit for trucks and motorcycles amounted to 16 units so that the vast amount of parking (AP) calculated by:

AP = The number of vehicles per type of vehicles x SRP
= (1 x 15) + (1 x 47.5) + (16 x 1.5)
= 86.5 m2

So the number of the parking area of 86.5 m2, which is known land area parking lot (ALP) by 25 mx 50 mx 25 m + 25 = 1875 m2, so the level of parking services (PAOR) was 4.6%. This means that the capacity is still very adequate parking space for use.

c. Level of Terminal Services

Passenger numbers by an average of 60 passengers per day, of which an unknown number of seats available (S) with 54 seats, so that the value of TOR is 111.11%. This means that the terminal services at the port of Kendari-Wawonii these crossings were not able to serve the number of passengers to the present. Based on the analysis in Table 13 concluded that up to 20 months of terminal capacity is still adequate space but need the addition of waiting for the passenger seat. Maximum capacity will be reached in month 25 and will exceed the capacity at month 26, so the addition of terminal space capacity has to be considered.

Table 13. Space Needs Passenger Terminal PolynomialModel

Month	Total Daysin the Month	Passenger Data Preliminary	Monthly Data Model	Daily Data Model	Space Needs (m^2)	LF
1	31	1040	857.12	27.65	22.40	0.21
2	28	1005	808.93	28.89	23.40	0.22
3	31	336	776.13	25.04	20.28	0.19
4	27	760	758.71	28.10	22.76	0.21
5	30	465	756.68	25.22	20.43	0.19
6	30	714	770.03	25.67	20.79	0.19
7	31	1228	798.77	25.77	20.87	0.19
8	30	676	842.89	28.10	22.76	0.21
9	31	1080	902.40	29.11	23.58	0.22
10	30	1250	977.30	32.58	26.39	0.24
11	31	1081	1067.58	34.44	27.89	0.26
12	31	1313	1173.25	37.85	30.66	0.28
13	28	931	1294.31	46.23	37.44	0.35
14	31	1224	1430.75	46.15	37.38	0.35
15	30	1263	1582.58	52.75	42.73	0.40
16	31	1121	1749.79	56.44	45.72	0.42
17	31	2759	1932.39	62.34	50.49	0.47
18	30	2670	2130.37	71.01	57.52	0.53
19	31	2817	2343.74	75.60	61.24	0.57
20	30	1796	2572.50	85.75	69.46	0.64

Source: Data Analysis, 2015

Note: The need for one person passenger space is 0,9m x 0,9m = 0.81m2

Spacious passenger terminal space available 9m x 12m = 108m2 with a number of 54 pieces waiting chair.

CONCLUSION

1) The condition of the ferry transport demand Kendari-Wawonii before the expansion and after the expansion are:
   a) the number of vessels on average trip per month after the split (30 trips / month) is greater than the number of trips before the expansion vessel (28 trips / month).
   b) the average number of passengers per month after the split (1276 people / month) is greater than the number of passengers before splitting (1131 person / month).
   c) the average number of vehicles per month after the split (364 units / month) is greater than the number of vehicles after the division (204 units / month).
   d) ferry transport demand model both the number of passengers and the number of vehicles before expansion and after expansion following the model polynomial because the R2 value of these models has the highest value.
2) There is a difference between these ferry transport demand conditions Kendari-Wawonii before division by division after
3) Operational performance ferry transport routes Kendari - Wawonii today are:
   a) based on the classification of passengers, vehicles, trip boats and dock capacity is still at the third classification where the number of daily passenger by 43 people / day less than 1000 people per day, the number of vehicles by 13 units / day less than the 250 units / today, the number of daily vessel trip for 1 trip / day less than the 6 trips / day and can dock capacity in the dock by ships weighing 157 GRT less than 500 GRT
   b) The needs of ferry transport fleet size after expansion is 2 fleet with a frequency of 2 times per day ie 1 fleet of Kendari and one fleet of Wawonii -Konawe Islands
   c) The level of service dock by BOR value of 13.57% means that the level of service on the pier in the harbor crossing is still allowed to serve aboard. Parking service levels based on the value PAOR of 4.6% means that the level of parking services in the harbor crossing is still allowed to serve the flow of vehicles while the level of service terminal based on the TOR of 111.11% means that the service level crossing terminal at the port currently does not allow for serve the flow of passengers.

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TRANSPORTATION PLANNING MODEL FOR EARTHQUAKE DISASTER DEBRIS - ISSUES RELATING TO TEMPORARY STORAGE SPACE IN METROPOLITAN TOKYO -

Y. Arai [1], A. Koizumi [2] and T. Inakazu [3]

ABSTRACT: Disposing of debris in the aftermath of the hugely destructive the Great East Japan Earthquake of March 2011 has been a significant long-term challenge. This is not least because experts expect a major earthquake to strike metropolitan Tokyo in the near future and are concerned that this would generate even more debris. It is important to transport debris quickly and efficiently from ravaged areas to temporary storage sites to safeguard public health. Firstly, this study estimated the volumes of debris that an epicentral earthquake would generate in Tokyo's 23 wards. The authors used five debris categories, including concrete and wood. This study calculated the space needed to temporarily store earthquake waste by debris type at parks, sports grounds, or other prospective sites, temporary storage, and quantified the volumes that each site could handle. The purpose of this calculation was to identify which localities have or lack spare temporary storage capacity. Secondly, this study constructed a transportation plan model extending from locations producing debris to temporary storage sites. The authors used linear programming (LP) to make formulations and regarded ton-kilometers as the objective function. Here, the goal was to identify the interdependence of municipalities. Finally, this study used the proposed model to simulate two transportation cases, the first covering separate transportation, and the second integrating overall regional transportation. In comparing both results, the authors estimated that integrated transportation generated ton-kilometers were around 12.9% lower than for individual transportation. This finding highlighted the importance of transporting earthquake debris regionally through collaboration between neighboring municipalities.

Keywords: Earthquake disaster, debris, temporary storage space, transportation planning, linear programming model.

INTRODUCTION

Disposing of debris in the aftermath of the hugely destructive the Great East Japan Earthquake of March 2011 has been a significant long-term challenge. The Great East Japan Earthquake wreaked huge destruction in Japan. Quickly disposing of the massive amount of debris from the disaster has been an important challenge in reconstructing affected areas. The three prefectures in the Tohoku region that the disaster hit hardest were Iwate, Miyagi, and Fukushima. They had around 28 million metric tons of debris at the end of February 2014: 5,740,000 metric tons for Iwate Prefecture, 18,800,000 metric tons for Miyagi Prefecture, and 3,490,000 metric tons for Fukushima Prefecture (MOE, 2014).

Experts expect a major earthquake to strike metropolitan Tokyo in the near future, resulting in an even greater disaster and generating even more debris. It would be essential to transport them to temporary storage sites before starting the disposal process. Although waste disposal is generally a task for municipalities, transportation covering wide areas through collaboration between neighboring authorities would become necessary because of the difficulty of any one municipality handling massive volumes of debris by itself. At such a time, there would be a need for regional transportation in cooperation with neighboring municipalities. Although it would be ideal in emergencies to transport debris to temporary storage sites that are as near as possible, an excessive volume of such debris could accumulate in parks or sports grounds depending on how this material is generated or depending on geographical differences, such as for locations with open spaces.

It would be useful to have full preparations in place to resolve the issue of massive volumes of debris that could result from an earthquake. This study accordingly focuses on transporting such material and proposes efficient transport plans that test optimization methodologies (Arai 2014). Such an approach could produce useful knowledge for formulating disaster plans and dealing with earthquake debris.

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EARTHQUAKE DEBRIS AMOUNTS AND ACCEPTABLE VOLUMES AT TEMPORARY STORAGE SITES

Estimating Debris by Town Block

This chapter considered building damage from the quake itself and fire and used formulas that the Cabinet Office announced to calculate the debris from those events, covering the numbers for complete and partial destruction. This study estimated the number of damaged buildings and the amount of debris by town block. The following presents the procedure we used to estimate the number of damaged buildings.

This study estimated the number of completely and partially destroyed buildings by multiplying the number of buildings by structural category (wooden and non-wooden) and age (old, relatively old, and new) by the complete and partial destruction rate. Data from materials that the Tokyo Disaster Management Council announced for the rates of completely and partially destroyed buildings was used in this analysis. This study also estimated the total number of fires by multiplying the number of buildings by the rates of fire from different causes.

Next, this chapter considered the composition of debris based on the number of damaged buildings mentioned above. The five debris categories were concrete, wood, metal, others (combustible), and others (non-combustible). This study then estimated the volume of earthquake debris by multiplying debris composition ratios for each building number estimate, referring to Table 1, which the Bureau of Environment of the Tokyo Metropolitan Government announced.

Figure 1 and Figure 2 show the number of damaged buildings and the estimated amounts of earthquake debris.

Table 1. Debris composition ratios. [unit: %].

Debris categories	Wooden buildings	Non-wooden buildings	Burned buildings
Concrete	47.5	85.1	58.9
Wood	20.4	0.5	5.1
Metal	1.4	7.0	1.7
Others (combustible)	3.8	0.9	1.0
Others (non-combustible)	26.9	6.4	33.4

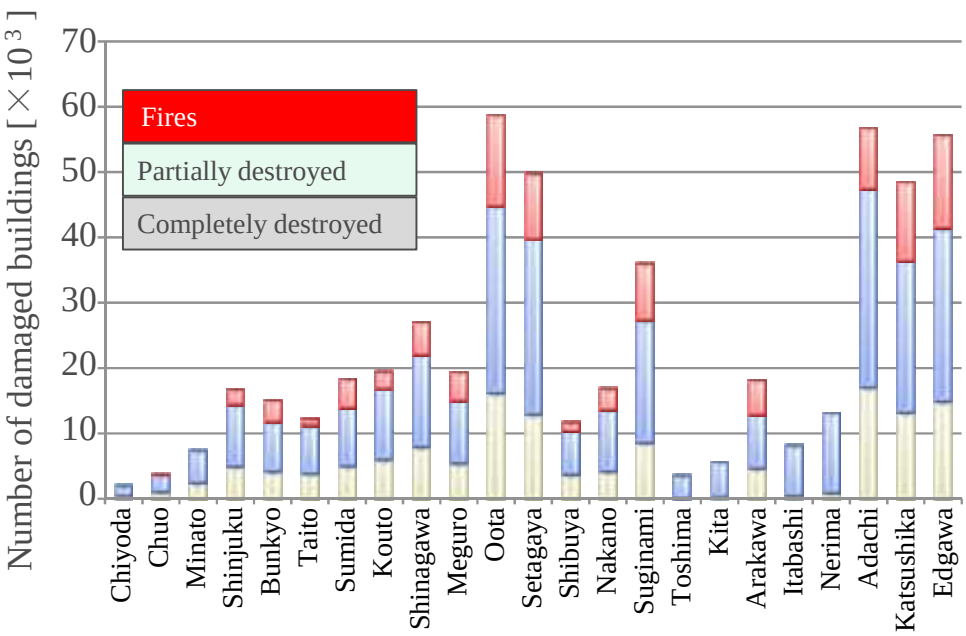

Figure 1. Number of damaged buildings.

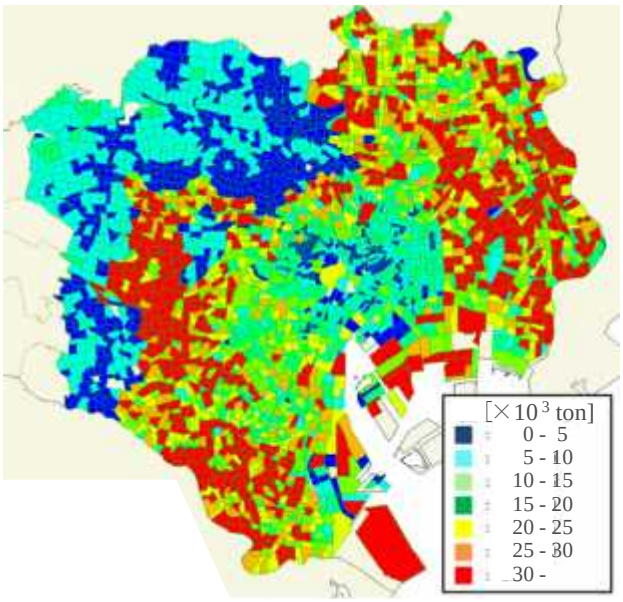

Figure 2. Estimated amounts of earthquake debris.

Calculating Acceptable Volumes at Temporary Storage Sites in Light of Debris Composition

To decide how much debris temporary storage sites could accept, we first needed to identify the space needed to amass earthquake debris. For example, wood and other lighter material take up more space than concrete and other heavier debris that is the same total weight. It is therefore desirable to increase the ratio of concrete debris for the total capacity of temporary storage locations in areas with more non-wooden structures while increasing ratios for wood debris in areas with dense concentrations of wooden structures. Therefore, for this study, due to different attributes between areas in terms of earthquake debris, this study adjusted debris compositions for temporary storage sites by ward. We provide calculation examples below. When converting results for earthquake debris generated throughout Area A, the debris ratios were 60% for concrete, 20% for wood, 5% for metals, 5% for combustible others and 10% for non-combustible others, creating a need to allocate space in prospective parks and sports grounds according to the above percentages. After each ward decided on spaces for temporary storage of each type of debris by applying such ratios, this study provided data converted back into weights based on allocation spaces as parameters for analyzing future models (restrictions on acceptable debris amounts at each temporary storage site).

Figure 3 shows the regional division of metropolitan Tokyo and the estimated amounts of earthquake debris compositions. Table 2 shows 1) the number of candidate temporary storage sites and 2) the ratio of the acceptable debris amount to the debris amount generated at each area.

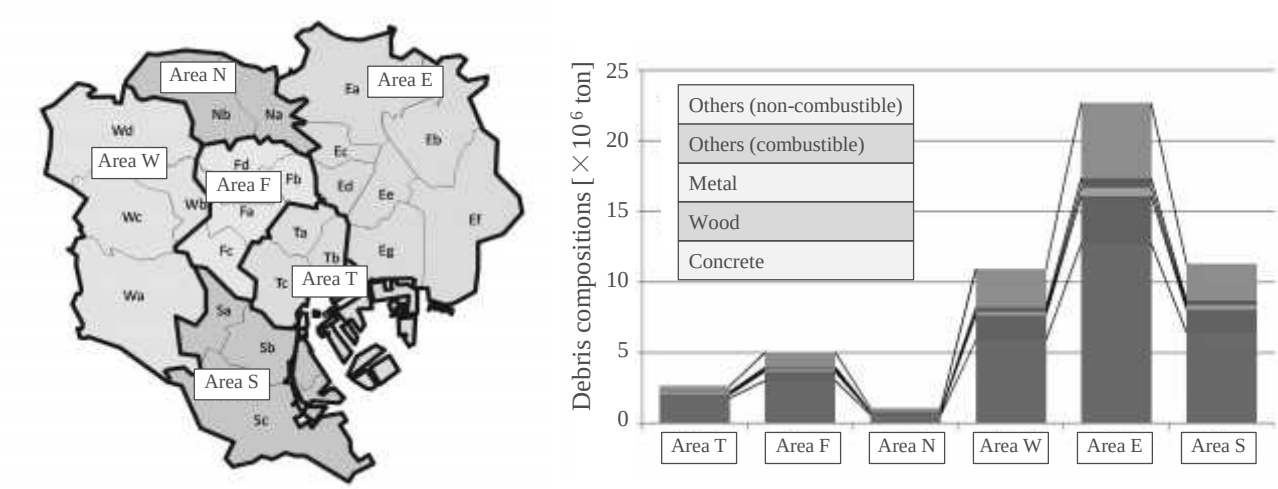

Figure 3. Regional divisions of metropolitan Tokyo (left) and their debris compositions (rigth).

Table 2. Number of candidate temporary storage sites and the ratio of the acceptable debris amount to the debris amount generated at each area.

	area T	area F	area N	area W	area E	area S
No. of candidate temporary storage sites	16	24	25	47	71	34
Ratio of the acceptable debris amount to the debris amount generated at each area	1.61	1.03	3.18	1.29	0.98	0.88

USING LINEAR PROGRAMMING TO CONSTRUCT TRANSPORTATION PLAN MODEL

This chapter constructed models for transportation networks and used linear programming (Dantizig 1963) to make formulations for effective transportation to temporary sites as primary collection points of earthquake debris from town block units. First, Figure 4 shows a transportation model from earthquake debris nodes to temporary storage locations. Here, i represents the site of a node where earthquake debris is produced, j is a temporary site, x_{ij} is the transportation tonnage from i to j, l_{ij} is the distance (kilometers) from i to j, w_i is the amount of earthquake debris generated at i, and q_i is acceptable tonnage at j.

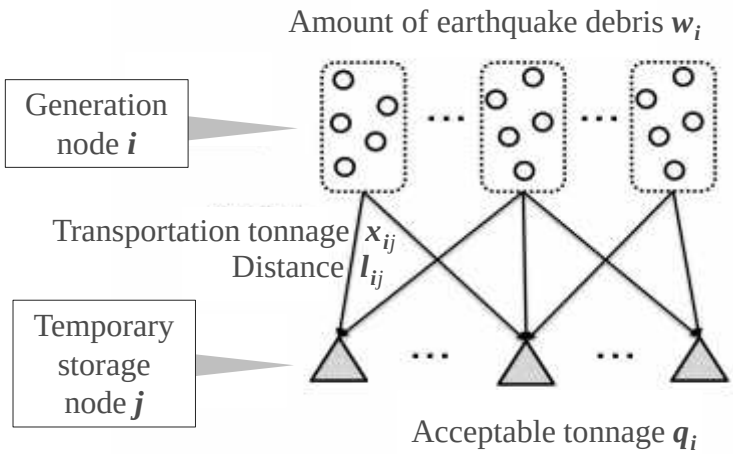

Figure 4. Transportation model from earthquake debris nodes to temporary storage sites.

In considering mathematical programming to minimize the index of the transportation volume by distance (ton-kilometers) while satisfying allowable constraints at temporary storage sites, the model constructed can be expressed according to the following formulations.

[Objective function]

$$\sum_i \sum_j x_{ij} l_{ij} \rightarrow \text{Minimize} \tag{1}$$

[Subject to]

$$\sum_j x_{ij} = w_i \tag{2}$$

$$\sum_i x_{ij} \leq q_j \tag{3}$$

$$x_{ij} \geq 0 \tag{4}$$

TRANSPORTATION PLAN AND COLLABORATIONS WITH NEIGHBORING MUNICIPALITIES

This chapter quantified improvements in transportation efficiencies through collaboration with neighboring municipalities. Below, this study created cases for individual transportation by area and integrated transportation for overall areas, comparing both cases (see Figure 5). We covered areas T, F, N, and E and excluded E and S areas, as the amounts of earthquake debris in the latter two areas exceeded acceptable amounts.

Resolving the optimization problem for minimizing the objective function of formula 1 under the constraints of formulas 2 to 4 led to the results shown in Figure 6. Note that the ton-kilometers shown here represent a total for calculations of the five debris categories that this study covers. This study integrated address blocks into one town and made transportation distances a straight line between two points.

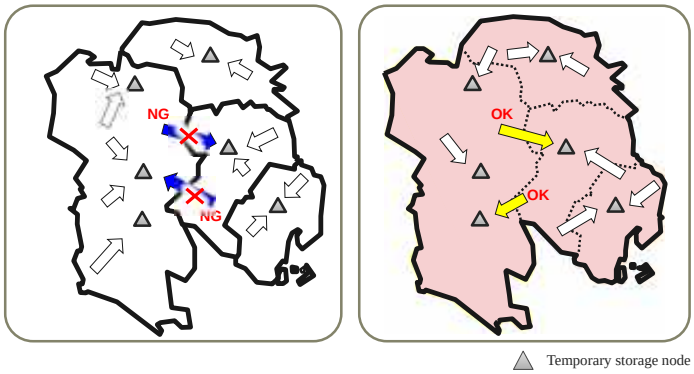

Figure 5. Comparison between individual transportation by area (left) and integrated transportation for overall areas (right).

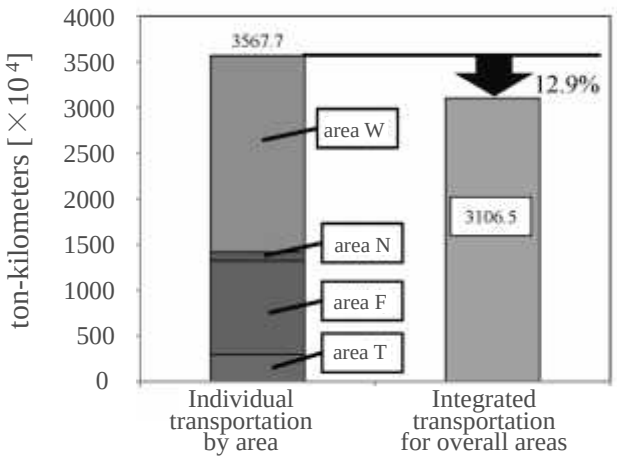

Figure 6. Ton-kilometers of individual transportation and integrated transportation.

Individual transportation distances in areas T, F, N, and W were 2,968,000 ton-kilometers, 10,266,000 ton-kilometers, 920,000 ton-kilometers and 21,523,000 ton-kilometers, respectively. The figure for area W was the biggest because it had more debris and is a larger location. At the same time, area N had the smallest ton-kilometer figure despite a relatively large size. This probably resulted from area N having little earthquake debris and sufficient parks in wards belonging to it, with parks being able to accommodate a lot of transportation, for high transportation efficiency. The individual transportation total for areas T, F, N, and W was 35,677,000 ton-kilometers. The total for integrated transportation, however, would be 31,065,000 ton-kilometers, or about 12.9% less.

The individual transportation result for area T is shown in Figure 7. We color-coded areas accepting debris. Although the transportation results in Figure 7 are for accepting concrete debris, we presented this graphic as a representative example for similar trends that was found in the other four debris categories. The transportation results for area T showed that Ward Ta took some debris from wards Tb and Tc. Also, Ward Tb took some debris from Ward Ta. This illustrates mutual assistance between wards Ta and Tb. Figure 7 shows the need for active collaborations with other wards to ensure efficient transportation.

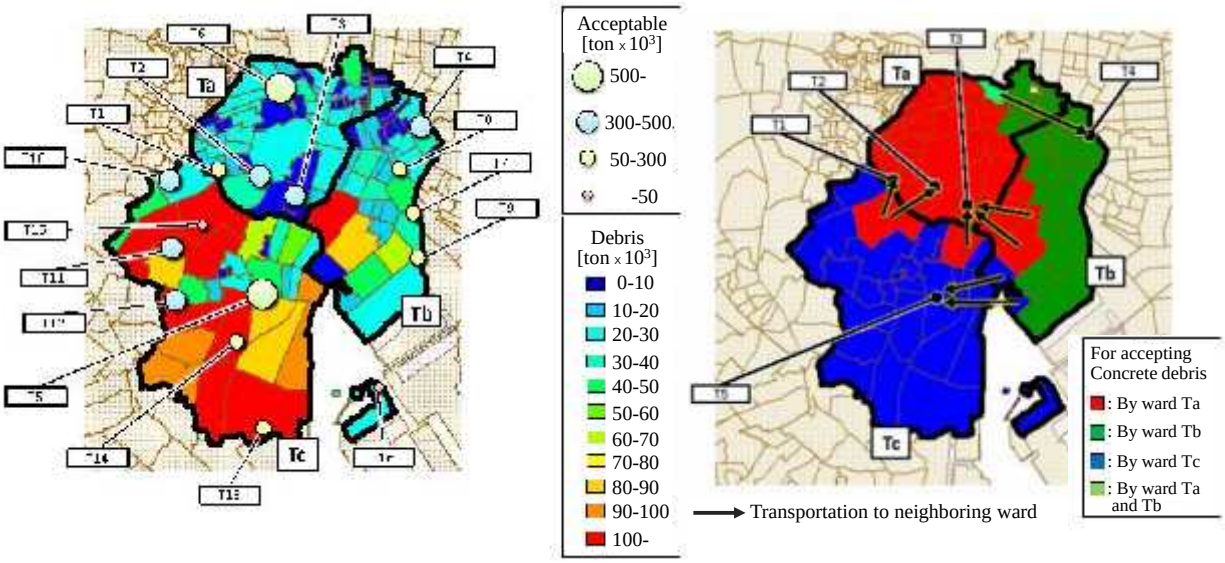

Figure 7. Acceptable /generated debris amounts (left) and individual transportation result (right) for area T.

Here, this study focused on transportation between wards and areas, expressed as a support relationship matrix. Table 3(a) and 3(b) show the support relationships for individual transportation and integrated transportation respectively. Each circle indicates support between wards. Table 3(a) and 3(b) both show results for concrete debris transportations (we omitted other debris types for the reason mentioned earlier). Table 3(a) shows that collaborations with other wards in area T were also active in all the areas. Although Table 3(b) also shows extensive transportation with other wards, collaboration with other wards was more active than in Table 3(a) after lifting the principal of disposing of debris locally and permitting transportations to neighboring areas. Transportation to other areas was extensive. We believe such transportation to other areas helped improve transportation efficiency.

The results above revealed that transportation of earthquake debris from where it originated to temporary storage locations is more efficient if it is beyond ward and area boundaries.

Table 3. Support relationships matrix for individual transportation (a; left) and integrated transportation (b; right).

		area T			area F				area N		area W			
		Ta	Tb	Tc	Fa	Fb	Fc	Fd	Na	Nb	Wa	Wb	Wc	Wd
area T	Ta		○	○										
	Tb	○												
	Tc		○											
area F	Fa					○	○	○						
	Fb							○						
	Fc				○									
	Fd													
area N	Na									○				
	Nb													
area W	Wa												○	
	Wb													
	Wc										○	○		
	Wd											○	○	

b ↑
a → ○
Ward-a accepts the debris form ward- b.

Neighboring ward

		area T			area F				area N		area W			
		Ta	Tb	Tc	Fa	Fb	Fc	Fd	Na	Nb	Wa	Wb	Wc	Wd
area T	Ta		○	○	○	○								
	Tb	○				○								
	Tc		○		○		○							
area F	Fa			○			○	○				○		
	Fb				○			○						
	Fc				○						○	○	○	
	Fd				○									
area N	Na					○		○		○				
	Nb							○						○
area W	Wa												○	
	Wb													
	Wc										○	○		○
	Wd											○		

b ↑
a → ○
Ward-a accepts the debris form ward- b.

Neighboring ward

CONCLUSION

Chapter 2 estimated the volumes of debris by town block that could result from an epicentral earthquake in Tokyo. Five debris categories, including concrete and wood, were used. This study calculated the space needed to temporarily store earthquake waste by debris type, and quantified the volumes that each site could handle. We thus identified which localities have or lack spare temporary storage capacity.

Chapter 3 constructed a transportation plan model extending from locations producing debris to temporary storage sites. This study used ton-kilometers as the objective function.

Chapter 4 used the proposed model to simulate two transportation cases, the first covering separate transportation, and the second integrating overall regional transportation. In comparing both results, it was clear that integrated transportation generated ton-kilometers were around 12.9% lower than for individual transportation. This finding highlighted the importance of transporting earthquake debris regionally through collaboration between neighboring municipalities.

ACKNOWLEDGEMENTS

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COMPOSITION ANALYSIS OF EMISSION TRUCK ON URBAN ROAD NETWORK IN MAKASSAR

Mukhtar Lutfie[1], Lawalenna Samang[2], Adi Sakti[3], Isran Ramli[4]

ABSTRACT: Trucks have a significant role in transporting build materials to the infrastructure development needs in the region South of Sulawesi. Thus, the operation of the truck on the urban road network for these activities, has brought the issue concerns the environmental impact of increased traffic accidents, improvement of urban transport congestion, increased emissions and vehicle noise. In the context of understanding the extent of the contribution of trucks to the issue of the problem, it needs to be done in depth study of phenomena related to the composition of the truck traffic on the existing road network, the contribution of emissions and noise caused, contributed to the decline in road traffic performance, and significant role in infrastructure development, and will eventually be able to do an assessment of the risk level operator of the truck on the urban road network. As a preliminary study on this issue, this paper focuses on analyzing the composition emission of the truck traffic on urban road network with a case study in the city of Makassar. This study has conducted a survey enumeration traffic volume for 13 hours from 07:00 to 22:00 on 36 major roads in the city of Makassar. The survey results were extracted to obtain emission composition trucks a day. Furthermore analysis to composition emission of the truck all day performed using statistical analysis approach to look at the composition emission of the phenomenon is different trucks all day interurban road sections surveyed. The results show that the composition of CO2 emissions is the biggest truck of the type of vehicle but the percentage is quite small 0.63% s / d 27.29%, where the frequency distribution level throughout the day is not the same for all arterial roads in the city of Makassar. The results in this study has provided a preliminary description of the composition emission of the trucks on the road network in the city of Makassar, and further studies can be used to estimate emissions and noise prediction trucks, trucks contribution to the urban traffic congestion, and the operation of the truck on the risk assessment of urban roads which will be carried on further studies in the future.

Keywords: Truck, urban road, composition of emissions, traffic, Makassar.

INTRODUCTION

As long as we live would need air to breathe. The air contained in the gas which is composed of 78% nitrogen, 20% oxygen, 0.93% argon, 0.03% carbon dioxide, and the remainder consisting of neon, helium, methane and hydrogen. Oxygen gas is an essential component for the survival of living beings, including humans. The composition of the air as it is normal and can support human life. However, due to human activities that are not environmentally friendly, often declining air quality. These changes can be physical properties and chemical. The chemical changes may be a reduction or addition of one of the chemical components contained in the air. Such conditions are commonly referred to as pollution (pollution) air.

The population of the world continues to increase each year, causing an increase in energy requirements. Today, almost all human energy needs derived from the conversion of fossil energy sources, such as vehicles that use fossil fuels as an energy source. Directly or indirectly, this resulted in a negative impact on the environment and health of living beings due to combustion of fossil fuels produces substances harmful contaminants. Air pollution, especially in large cities has led to the decline in air quality that disrupts the environment comfort even has caused health problems. Declining air quality is mainly caused by the use of fossil fuels uncontrolled and inefficient in transportation facilities are generally concentrated in large cities, in addition to industrial activities, household and forest fires. Research results in several major cities (Jakarta, Bandung, Semarang and Surabaya) showed that the motor vehicle is a major source of air pollution. Results of research in Jakarta showed that motor vehicle pollution contributes CO by 98.80%, 73.40% for NOx and HC by 88.90% (Environment, 1992).

Since the start of the industrial revolution in 1796 in France, the state of the global climate is gradually deteriorating. The rate of release of harmful gases in Atmospheric CO2 particularly exacerbated by the rate of economic growth are positively correlated to the rate of growth of the industry. This causes the average temperature at the Earth's surface rose by $1^{o}C$ in recent years.

In the last 100 years, CO2 emissions increased by 2.5% per year and is expected to increase 3 three-fold from its current state. The concentration of CO2 is also expected to increase to 500 to 600 ppmv in the mid 21st century These increases tend not to stop because it supported a sizeable forest depletion. Experts believe that the decline of forests accounted for 23% increase in CO2. Last predictions of the IPCC (Intergovernmental Panel on Climate Change)

Working Group 1 (IPCC Working Group 2; 2001) has said that in line with the rise in CO2 levels in the atmosphere of the earth, will be an increase in the average temperature of the surface of the state in 1990 amounted to 1.4 to 5 , 8°C until 2100 with different regional diversity.

Emissions of CH4 (methane) is the release of CH4 into the air originating, inter alia, of the natural gas is not burned, as the main element of natural gas is methane. Methane is one of the green house gases that cause global warming.

Makassar City is a city of education, tourism, industry and craft. Therefore Makassar particularly increased in the field of road transport. Moreover, if this happens at signalized intersections not, because that happens not only congestion but also lead to queues, delays, traffic accidents and even pollution occurs.

On that basis, it is necessary to do research to help solve the problems posed, including pollution. Pollution that occurs normally would cause damage to the building (rust, mold), causing respiratory distress and health, resulting in climate change and rising sea levels and global warming.

In the context of understanding the extent of the contribution of emission trucks on the issue of the problem, it needs to be done in-depth study of the analysis of the composition of truck emissions in the urban road network in the city of Makassar.

DATA ANALYSIS AND RESULTS

The Data

The location of this research study is located on the main road 34 which consists of a 61 way street in the city of Makassar is as follows:

Table 1 Location Study

No.	Street	Directions Road
1.	A.P. Pettarani	Sultan Alauddin
2.	A.P. Pettarani	Urip Sumaharjo
3.	Abd. Dg. Sirua	Perintis Kemerdekaan
4.	Abd. Dg. Sirua	A.P. Pettarani
5.	Adyaksa	Pengayoman
6.	Adyaksa	Hertasning
7.	Adyaksa Baru	Pengayoman
8.	Adyaksa Baru	Abd. Dg. Sirua
9.	Andi Tonro	Maapaodang dan Kumala
10.	Arif Rate	Sultan Hasanuddin
11.	Arif Rate	Cendrawasih
12.	Bandang	Tentara Pelajar
13.	Bandang	Veteran Utara
14.	Batua Raya	Abd. Dg. Sirua
15.	Batua Raya	Borong Raya
16.	Borong Raya	Batua Raya
17.	Borong Raya	Ujung Bori
18.	Boulevard	A.P. Pettarani
19.	Boulevard	Adyaksa Baru
20.	Cakalang	Yos Sudarso
21.	Cakalang	Tinumbu
22.	Cendrawasih	Dangko
23.	Cendrawasih	Haji Bau
24.	Tata Arah	Moh. Tahir
25.	Dg. Tata	Mallengkeri
26.	Dr. Ratulangi	Jend. Sudirman
27.	Dr. Ratulangi	Kakatua
28.	Gagak	Kakatua
29.	Gagak	Rajawali
30.	Gunung Bawakaraeng	Jend. Sudirman
31.	Haji Bau	Dr. Ratulangi
32.	Haji Bau	Penghibur
33.	Hertasning	A.P. Pettarani
34.	Hertasning	Hertasning Baru
35.	Hertasning Baru	Gowa
36.	Hertasning Baru	Hertasning
37.	Jend. Ahmad Yani	Jend. M. Yusuf

No.	Street	Directions Road
38.	Kakatua	Dr. Ratulangi
39.	Kakatua	Gagak
40.	Masjid Raya	Urip Sumaharjo
41.	Penghibur	Pasar Ikan
42.	Perintis Kemerdekaan	Daya
43.	Perintis Kemerdekaan	Urip Sumaharjo
44.	Rajawali	Penghibur
45.	Rajawali	Gagak
46.	Rapocini Raya	A.P. Pettarani
47.	Rapocini Raya	Veteran Selatan
48.	Sulawesi	Riburane
49.	Sultan Alauddin	Andi Tonro
50.	Sultan Alauddin	Sutan Hasanuddin (Gowa)
51.	Sutan Hasanuddin	Slamet Riyadi
52.	Sungai Saddang	Jend. Sudirman
53.	Sungai Saddang	Veteran Selatan
54.	Tentara Pelajar	Nusantara
55.	Tentara Pelajar	Bandang
56.	Urip Sumoharjo	Perintis Kemerdekaan
57.	Urip Sumoharjo	Gunung Bawakaraeng
58.	Veteran Selatan	Veteran Utara
59.	Veteran Selatan	Sultan Alauddin
60.	Veteran Utara	Masjid Raya
61.	Veteran Utara	Veteran Selatan

Analysis

Analysis composition the emissions of truck traffic on urban road network with a case study in the city of Makassar. This study was conducted a survey enumeration traffic volume for 13 hours from 07:00 to 22:00 on 36 main roads in the city of Makassar. The survey results were extracted to obtain the composition of the truck to the existing traffic volume. Further analysis composition the emissions of trucks throughout the day is done by using descriptive statistical analysis approach to look at the phenomenon of the composition of the truck all day whether the emissions differ between urban road sections surveyed.

The formula used as follows: $n / \varepsilon n \times 100$

where: n = number of vehicles at certain hours

εn = total number of vehicles at certain hours

Analysis of the composition of the vehicle taken based on the results of a survey that has been calculated amount of percentage and presented in tables and charts as below:

Table 2 IPCC emission factor A.P. Pettarani street

No.	Emission factors	Emission (g/ h)					
		NOx	CH4	NMVOC	CO	N2O	CO2
1.	Motorcycle	7116,0	3558,0	85362,5	426812,9	39,9	2596417,9
2.	Passenger cars (petrol)	49869,6	1658,4	124685,9	1080619,0	93,4	6068125,9
3.	Passenger cars (diesel)	1325,3	8,9398	30954,2	1325,3	17,8	326852,0
4.	Large buses	160,8	1,0850	3756,8	160,8	2,1	39669,5
5.	Bus Medium	129,6	0,8	3028,7	129,6	1,7	31981,1
6.	Taxi Petrol	917,6	30,5	2294,2	19883,8	1,7	111655,8
7.	Taxi Diesel	6,0	0,04	140,2	6,0	0,0	1481,3
8.	Large trucks	1072,6	6,5	214,6	965,1	3,2	79367,7
9.	Trucks Medium	2225,6	5,6	556,0	2225,6	22,5	411745,5
10.	Small Truck Gasoline	2446,0	69,7	4892,1	29004,1	3,9	255099,7
11.	Small Truck Diesel	549,9343	1,3914	137,3966	549,9343	5,5	101739,5
Emission Total (gram/jam)		65819,5048	5341,1638	256023,1261	1561682,6458	192,2	10024136,5
Emission Total (kg/jam)		65,8195	5,3412	256,0231	1561,6826	0,1	10024,1
Emission Total / day (kg)		1053,1	85,4	4096,3	24986,9	3,0	160386,1

Source: Processed Data, 2013

Table 3 IPCC emission factor (converted) A.P. Pettarani street

No.	Emission factors	Emission (g/ h)					
		NOx	CH4	NMVOC	CO	N2O	CO2
1	Motorcycle	23645,5	786,3	59119,4	512372,1	44,3	2877183,1
2.	Passenger cars (petrol)	49872,2	1658,5	124692,2	1080673,5	93,4	6068431,8
3.	Passenger cars (diesel)	1325,3	8,9	30955,7	1325,3	17,8	326868,5
4.	Large buses	129,8	0,8	3032,3	129,8	1,7	32018,9
5.	Bus Medium	135,5	0,9	3166,3	135,5	1,8	33434,6
6.	Taxi Petrol	994,4	33,0	2486,2	21547,9	1,8	121000,8
7.	Taxi Diesel	10,9	0,1	255,0	10,9	0,1	2692,6
8.	Large trucks	277,3	1,8	6477,2	277,3	3,7	68394,6
9.	Trucks Medium	1502,3	10,1	35088,4	1502,3	20,2	370506,2
10.	Small Truck Gasoline	3657,5	121,6	9144,6	79254,6	6,8	445047,7
11.	Small Truck Diesel	528,5	3,5	12345,2	528,5716	7,1	130355,7
Emission Total (g/h)		82079,6	2625,9	286763,0	1697758,3	199,2	10475934,8
Emission Total (kg/h)		82,0	2,6	286,7	1697,7	0,1	10475,9
Emission Total / day (kg)		1313,2	42,0	4588,2	27164,1	3,1	167614,9

Source: Processed Data, 2013

Table 4 IPCC Emission Factor A.P. Pettarani Street

No.	Emission Factors	NOx	NMVOC	CO	N2O	CO2
1.	Pollutant	1053,1648	4096,5724	24988,1642	3,0767	160394,1322
2.	IPCC (conversion)	1313,2751	4588,2091	27164,1333	3,1872	167614,9571

Source: Processed Data, 2013

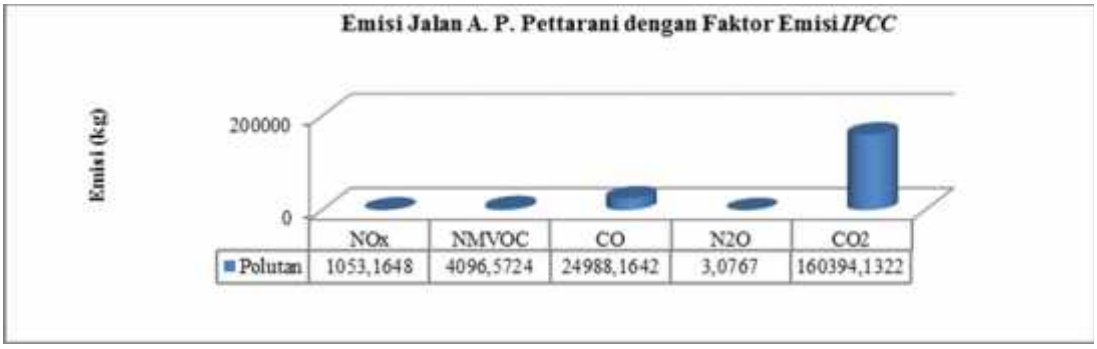

Figure 1 Chart Emissions Vehicle of IPCC Emission Factor A.P. Pettarani Street

Table 5 Percentage IPCC Emission Factor A.P. Pettarani Street

No.	Emission Factors	Emission (Persentage)					
		NOx	CH4	NMVOC	CO	N2O	CO2
1.	Motorcycle	10,81	66,62	33,34	27,33	20,79	25,90
2.	Passenger cars (petrol)	75,77	31,05	48,70	69,20	48,59	60,54
3.	Passenger cars (diesel)	2,01	0,17	12,09	0,08	9,30	3,26
4.	Large buses	0,24	0,02	1,47	0,01	1,13	0,40
5.	Bus Medium	0,20	0,02	1,18	0,01	0,91	0,32
6.	Taxi Petrol	1,39	0,57	0,90	1,27	0,89	1,11
7.	Taxi Diesel	0,01	0,00	0,05	0,00	0,04	0,01
8.	Large trucks	1,63	0,12	0,08	0,06	1,69	0,79
9.	Trucks Medium	3,38	0,11	0,22	0,14	11,71	4,11
10.	Small Truck Gasoline	3,72	1,31	1,91	1,86	2,04	2,54
11.	Small Truck Diesel	0,84	0,03	0,05	0,04	2,89	1,01
Persentage Total		100	100	100	100	100	100

Source: Processed Data, 2013

Table 6 Percentage emission trucks of IPCC emission factor AP. Pettarani street

No.	Emission Factors	Emissions/ Day (Kg)	Persentage	Percentage of Total Vehicles
1.	NOx	100,7082919	0,705	9,56
2.	CH4	1,332063354	0,009	1,56
3.	NMVOC	92,80349939	0,650	2,27
4.	CO	523,919004	3,667	2,10
5.	N2O	0,564369724	0,004	18,34
6.	CO2	13567,2436	94,965	8,46
Total		14286,571	100	

Source: Processed Data, 2013

Based on the above data it can be seen that the rate of emission composition trucks from other vehicles NOx 9.56%, 1.56% CH4, NMVOC 2.27%, 2.10% CO, N2O 18.34% and 1.01% CO2.

Table 7 Emission factor Indonesia A.P. Pettarani street

No.	Emission Factors	Emissions (Percentage)					
		CO	HC	Nox	PM10	CO2	SO2
1.	Motorcycle	526048,7	221691,9	10896,7	9017,9	3136565,6	300,5
2.	Passenger cars (petrol)	769977,7	76997,7	38498,8	192,4	5458458,7	500,4
3.	Passenger cars (diesel)	2777,1	198,3	3471,4	525,6	329342,8	436,4
4.	Bus	1805,7	213,4	1953,4	229,8	116712,2	152,6
5.	Truck	22163,9	4749,4	46702,6	3693,9	1109721,8	2163,6
6.	Car transport	41309,6	4868,9	2012,7	5,7	298694,7	27,7
Total Emissions (g/h)		1364083,0	308719,9	103535,9752	13665,7	10449496,0	3581,5
Total Emissions (kg/h)		1364,0	308,7	103,5360	13,6	10449,4	3,5
Total Emissions/day (kg)		21825,3	4939,5	1656,5756	218,6	167191,9	57,3

Source: Processed Data, 2013

Table 8 Percentage emission factor Indonesia AP. Pettarani street

No.	Emission Factors	Emissions (Percentage)					
		CO	HC	Nox	PM10	CO2	SO2
1.	Motorcycle	38,56	71,81	10,52	65,99	30,02	8,39
2.	Passenger cars (petrol)	56,45	24,94	37,18	1,41	52,24	13,97
3.	Passenger cars (diesel)	0,20	0,06	3,35	3,85	3,15	12,18
4.	Bus	0,13	0,07	1,89	1,68	1,12	4,26
5.	Truck	1,62	1,54	45,11	27,03	10,62	60,41
6.	Car transport	3,03	1,58	1,94	0,04	2,86	0,78
Total Percentage		100	100	100	100	100	100

Source: Processed Data, 2013

Table 9 Percentage truck emissions of emission factor Indonesia AP Pettarani street

No.	Emission Factors	Emission/day (Kg)	percentage	Percentage of Total Vehicles
3.	CO	354,6058304	1,86	1,62
4.	HC	75,98696366	0,40	1,54
5.	NOx	747,2051427	3,93	45,11
6.	PM10	59,10097174	0,31	27,03
7.	CO2	17754,68398	93,32	10,62
8.	SO2	34,61628345	0,18	60,41
Total		19026,1992	100	

Source: Processed Data, 2013

Based on the above data it can be seen that the rate of emission composition trucks from other vehicles CO 1.62%, 1.54% HC, NOx 45.11%, 27.03% PM10, SO2, CO2 10.62% and 60.41%.

Table 10 Percentage vehicle of pollutants/ emissions AP. Pettarani street

Emission Factors	NOx	CH4	NMVOC	CO	N2O	CO2	HC	PM10	SO2
≈ fuel	0,55%	0,04%	2,15%	13,11%	0,002%	84,14%			
≠ fuel	0,85%			11,14%		85,35%	2,52%	0,11%	0,03%

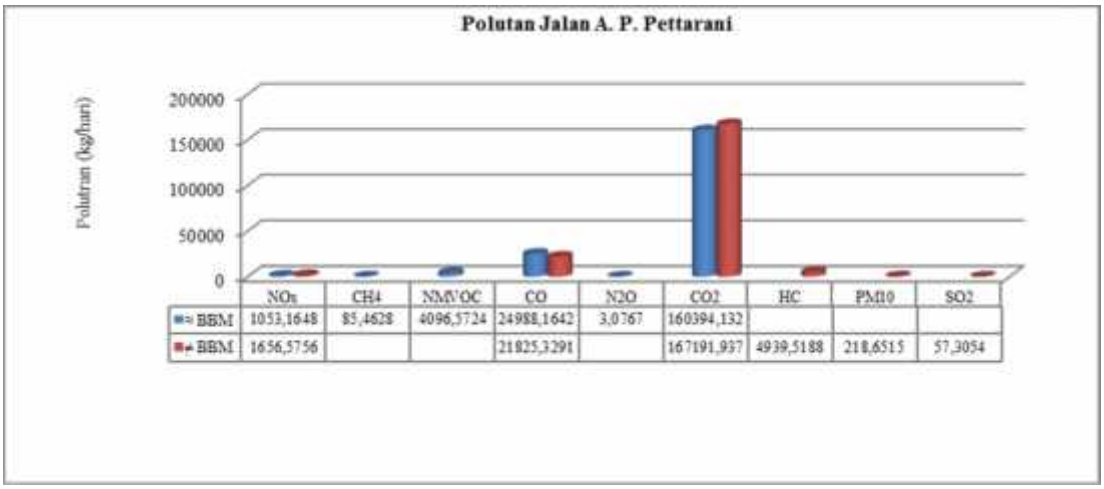

Figure 2 Chart Vehicle Emissions A.P. Pettarani Street

Table 11 Percentage emission trucks A.P. Pettarani street

Emission Factors	NOx	CH4	NMVOC	CO	N2O	CO2	HC	PM10	SO2
≈ fuel	9,56%	1,56%	2,27%	2,10%	18,34%	8,46%			
≠ fuel	45,11%			1,62%		10,62%	1,54%	27,03%	60,41%

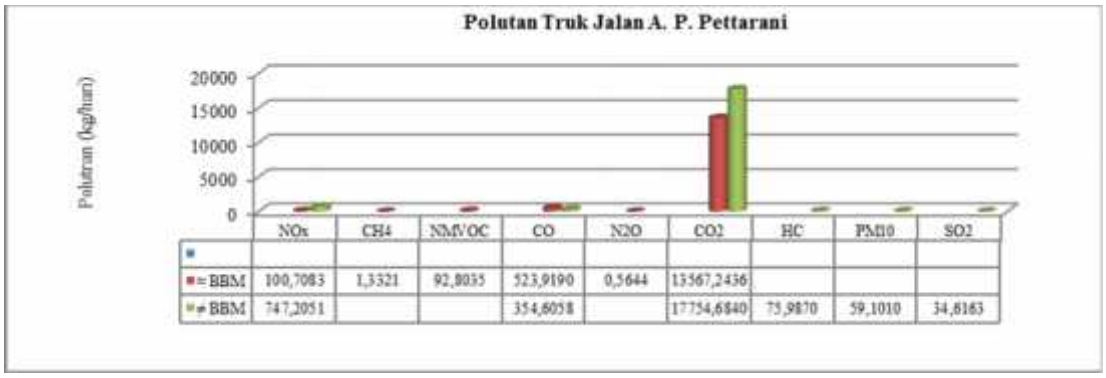

Figure 3 Chart Emission Trucks A.P. Pettarani Street

The Results

The results of data processing and analysis of the results can be seen in the following table:

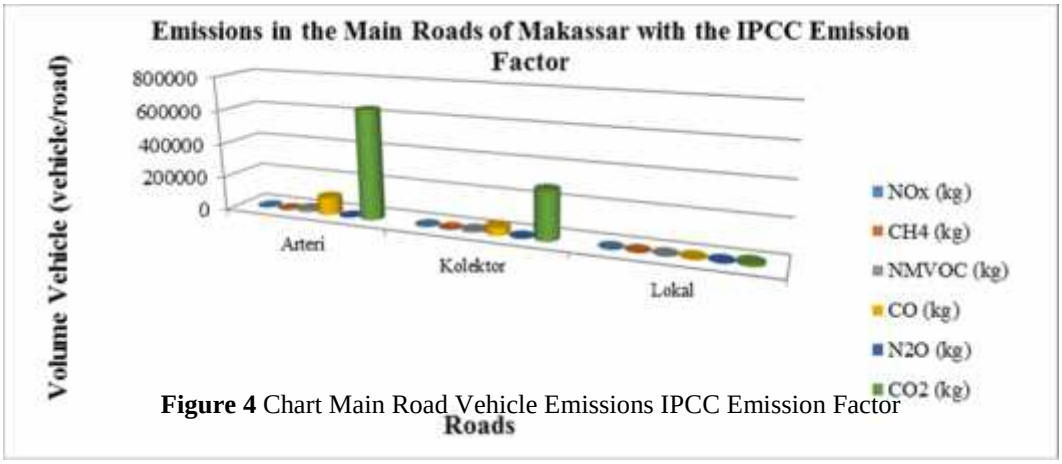

Figure 4 Chart Main Road Vehicle Emissions IPCC Emission Factor

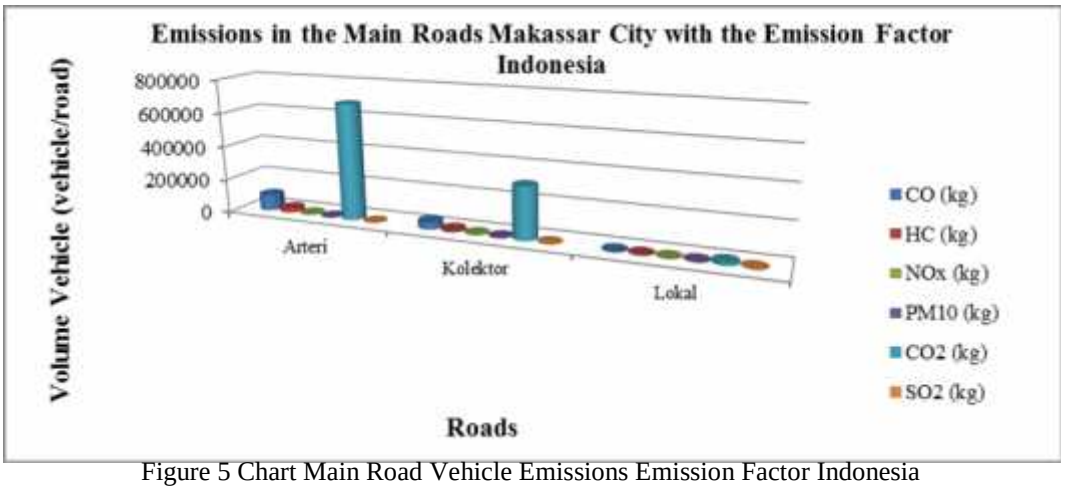

Figure 5 Chart Main Road Vehicle Emissions Emission Factor Indonesia

Table 12 Percentage of makassar truck emissions ≈fuel and ≠fuel

Street Name	NOx	CH4	NMVOC	CO	N2O	CO2	HC	PM10	SO2
A.P. Pettarani	9,56	1,56	2,27	2,10	18,34	8,46			
	45,11			1,62		10,62	1,54	27,03	60,41
Abd. Dg. Sirua	8,33	0,98	1,99	1,94	10,79	5,39			
	40,15			1,02		6,25	0,72	13,35	57,30
Adyaksa	6,87	0,84	1,76	1,76	10,14	4,92			
	38,17			0,95		5,73		12,22	53,94
AdyaksaBaru	2,22	0,18	0,25	0,20	3,39	1,34			
	9,94			0,20		1,84	0,17	3,77	19,90
AndiTonro	3,69	0,55	0,99	0,90	5,17	2,55			
	22,61			0,50		3,07	0,40	8,43	39,04
Arif Rate	4,32	0,87	1,32	1,18	6,94	3,43			
	27,20			0,71		4,26	0,67	14,38	44,52
Bandang	12,02	1,23	2,44	2,32	20,07	9,01			
	50,10			1,56		10,75	1,13	19,77	66,86
Batua Raya	8,23	0,90	1,79	1,73	13,47	6,06			
	40,47			1,07		7,13	0,77	14,40	57,30
Borong Raya	15,59	1,17	1,69	1,53	16,22	7,77			
	42,45			1,17		9,77	0,86	15,75	59,54
Boulevard	3,83	0,71	1,16	1,11	6,78	3,26			
	26,42			0,67		4,00	0,59	12,05	40,79
Bandang	12,02	1,23	2,44	2,32	20,07	9,01			
	50,10			1,56		10,75	1,13	19,77	66,86
Cakalang	31,85	3,11	4,76	4,11	43,51	22,38			
	70,52			3,84		27,29	3,01	41,18	83,73
Cendrawasih	3,63	0,31	0,55	0,50	6,26	2,52			
	18,95			0,39		3,09	0,30	6,19	32,10
Dg. Tata	8,65	0,65	1,24	1,16	14,85	6,09			
	36,94			0,93		7.37	0,67	12,58	53,41
Dr. Ratulangi	4,06	0,69	0,87	0,73	5,55	2,67			
	20,08			0,50		3,50	0,51	12,13	36,31
Street Name	NOx	CH4	NMVOC	CO	N2O	CO2	HC	PM10	SO2
Gagak	23,87	1,99	3,33	3,17	32,56	15,89			
	62,28			2,63		19,10	1,91	29,11	74,89
GunungBawakaraeng	3,73	1,10	1,67	1,48	4,78	2,99			
	27,81			0,74		3,59	0,73	16,09	46,43
Haji Bau	9,96	2,62	3,96	3,59	13,42	7,96			
	50,23			1,94		9,71	1,86	31,89	68,08
Hertasning	6,49	0,71	0,61	0,60	8,29	3,96			
	21,54			0,63		5,15	0,63	12,21	25,31
HertasningBaru	13,88	1,38	2,55	2,46	20,60	9,61			
	51,45			1,68		11,59	1,22	20,89	66,95
Jend. Ahmad Yani	0,72	0,16	0,22	0,19	1,39	0,63			

	5,82			0,13		0,82	0,14	3,47	11,36
Kakatua	9,35	0,70	1,05	0,93	14,90	6,19			
	35,12			0,92		7,88	0,72	13,85	51,72
Masjid Raya	11,38	2,05	3,63	3,71	12,86	7,91			
	49,78			1,80		9,02	1,39	22,91	62,05
Penghibur	4,45	0,73	1,01	0,90	8,82	3,82			
	26,15			0,71		4,94	0,71	15,05	41,35
PerintisKemerdekaan	7,83	0,80	1,56	1,42	16,76	6,85			
	40,30			1,13		8,33	0,88	16,46	58,15
Rajawali	20,48	1,96	3,22	2,92	31,00	14,69			
	59,98			2,48		17,87	1,93	30,31	74,94
Rapocini Raya	2,27	0,23	0,52	0,53	2,75	1,36			
	15,38			0,26		1,56	0,16	3,25	25,75
Sulawesi	5,93	1,65	2,25	2,04	9,27	5,07			
	36,91			1,19		6,23	1,25	24,83	54,53
Sultan Alauddin	9,83	1,17	2,03	1,86	15,53	7,09			
	42,70			1,26		8,69	1,00	18,55	60,70
SutanHasanuddin	3,70	0,92	1,38	1,28	5,14	2,94			
	26,14			0,69		3,60	0,66	13,83	41,65
Sungai Saddang	3,69	0,93	1,38	1,29	5,45	3,04			
	26,54			0,71		3,68	0,69	14,42	41,29
TentaraPelajar	16,38	2,90	4,18	3,63	27,32	13,49			
	58,60			2,69		16,88	2,59	40,07	75,30
UripSumoharjo	6,92	0,69	1,16	1,06	9,34	4,23			
	29,74			0,70		5,24	0,54	10,90	46,95
Veteran Selatan	8,38	1,19	1,38	1,22	12,52	5,82			
	31,99			1,00		7,38	1,00	20,32	46,54
Veteran Utara	10,74	1,46	2,82	2,77	16,21	8,03			
	49,11			1,57		9,31	1,16	20,18	64,53

Source: Processed Data, 2013

The results of this analysis showed that the composition of CO2 emissions is the biggest truck, but the percentage of the type of vehicle is quite small 0.63% s / d 27.29%, where the rate of frequency distribution throughout the day is not the same for all arterial roads in the city of Makassar. The main arterial roads show truck emission frequency greater than the secondary arterial roads.

CONCLUSION

Based on the results of data processing and analysis of the results, the conclusions from the results of this study show that the composition of truck emissions to the volume of traffic that is small enough, where the frequency distribution throughout the day is not the same for all arterial roads in the city of Makassar. The main arterial roads show truck emission frequency greater than the secondary arterial roads.

The results in this study provide a preliminary description of the estimation of emission composition trucks on the road network in the city of Makassar, and further can be used to study the noise prediction truck, truck contribution to urban traffic congestion, and the risk assessment of operating a truck on urban roads will be on advanced studies in the future.

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SPATIAL MODEL OF GRADUATE STUDENT TRAVEL IN MAKASSAR CITY

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ABSTRACT: Travel behavior to campus is the primary journey that has a large enough proportion of the trip, especially in Makassar that affect the performance of roads around campus. Travel behavior to campus is influenced by socio-economic variables and spatial variables. This study aims to: 1) To Analyzing the characteristics of the travel behavior of students to campus at public universities in Makassar; 2) Analyzing the regression model of residence location choice based socio-economic factors and spatial factors; 3) To analyze the spatial patterns of residential location of the college students in Makassar, in terms of geographical distribution and identify hotspots.Metode used method of regression and spatial location of the shelter anlisa students at the University of Hasanuddin Makassar. Regression analysis showed a significant association between living costs of students with the direct distance between the campus and residential location. The other thing is a significant relationship between the cost of transport to the Distance trip to campus. Furthermore, spatial analysis methods Hotspot and isolines for student residential location.

INTRODUCTION

Education is one of the basic human needs. In fulfillment of the need for education, especially formal education, then there was movement toward campus. The movement itself requires a mode of transport as a means to move. The phenomenon of the campus environment as an area with all forms of activity in it. Campus as an educational area will result in seizure and pull the trip. Travel is not limited to movement from one place to another, but in campus activities not only one type of activity being done. But the journey that followed occurred mobility other activities that should be done. Mode of transportation to work as a supporting mobility traveling.

Modal choice varies for each individual student. This variation can be attributed to a number of factors such as culture and different behaviors, different effects on trips based on environmental conditions, topography and geography, the influence of the availability of various modes of transport, nature jan network and road network conditions and government policies. Selection of a mode of transportation for a trip to campus is one example of regular travel behavior, because students generally travel to campus at around the same relative time each day, with the same intensity (following the lectures), and passes through the same route habit of traveling to campus is one of the most important predictor for routine travel behavior. Thus the habit very important factor in explaining the behavior of students using the mode of transportation to campus. According Tumlin (2004), one of the most effective ways that minimize travel costs and distance, Sertan to reduce transport needs is to provide shelter for the perpetrators be traveling at a location travel destination

In exploratory spatial data analysis the map has an important role to play. The map is the most established and conventional means of displaying areal data. There is a variety of ways ascribing continuous variable data to given areal units that are predefined. In practice, however, none is unproblematic. Mapping and geovisualisation is an important step to provoke questions, but exploratory data analysis requires highly interactive, dynamic data displays. Recent developments in spatial data analysis software provide an interactive environment that combines maps with statistical graphs, using the technology of dynamically linked windows.

The phenomenon of interest may be described in most general terms as interactions between populations of actors and opportunities distributed over some relevant geographic space. Such interactions may involve movements of individuals from one location to another, such as daily traffic flows in which case the relevant actors are individuals such as commuters (shoppers) and the relevant opportunities are their destinations such as jobs (or stores). Similarly, one may consider migration flows, in which case the relevant actors are migrants (individuals, family units, firms) and the relevant opportunities are their possible new locations.(Manfred M.)

According Mark P. The core of transportation are access to activity sites. This means that the population establishes proximity to places of work, recreation, socializing, shopping, and school. In transportation planning, individual passenger trips are often classified by the purpose of the trip and the trip origin. Common home based trip types include:

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[4] Lecturer, Civil Engineering Department, Hasanuddin University, Makassar, 90245, INDONESIA

- ***Work Trips***. Trips made to a person's place of employment such as a factory, a store, or an office.
- ***Shopping Trips***. Trips made to retail establishment regardless of the size or type of purchase. Trips made to a store "just to look" are shopping trips even though no purchase is made.
- ***Social or recreational trips***. Cultural trips made to recreational or entertainment facilities (e.g., church, civic meetings, concerts, sporting events). Travel to social activities (parties, visiting friends) would be included.
- ***Business Trips*.** Trips made in the course of performing a normal day's work. The origin of such trips is often the place of employment.
- ***School Trips***. Trips made by the students to an institution of learning.

Research Purposes
The purpose of this study are :

- To Analyzing the characteristics of travel behavior of students to campus at public universities in Makassar.
- Analyzing the regression model residence location choice based socio-economic factors and spatial factors.
- To analyze the spatial patterns of residential location of the college students in Makassar, in terms of geographical distribution and identify hotspots.

LITERATURE REVIEW

This phenomenon occurs when the congestion on some roads or region in Makassar, where one of the locations prone to congestion that is currently happening in the city of Makassar are on the roads around the Hasanuddin University and State University of Makassar. The location is centrally located and and on arterial roads. Increasing the number of students who study at the campus resulted in the increasingly congested roads and students who go to and from college many who use personal automobiles, motorcycles and public transport, it accounts for density and traffic congestion on the roads around the University of Hasanuddin and the State University of Makassar. Therefore, the need for proper planning to resolve transportation problems that occur around the Hasanuddin University and State University of Makassar.

The existence of domestic college campus in the city of Makassar affect the amount of traffic generation and attraction. It is often congestion during peak hours in the morning, afternoon and evening. It is caused by:

1. The number of public transport passengers in the front of the lower ascending campus.
2. The high number of pedestrian crossing on the road around the campus without the necessary bridge for pedestrian crossings.
3. The number of vehicles that enter and exit the campus.
4. Less optimal functioning traffic light around campus.
5. geometric condition of roads around campus is not adequate to accommodate the volume of traffic.

Spatial Analysis

GIS (Geographic Information System) is a powerful tool that allows to perform spatial analysis and graphic representation of large amounts of data. Analysis of demand for transport and transport planning is a process of intensive data processing. Understanding the needs of the transport depends on the analysis of travel behavior, which in turn depends on several factors among which are the socio-demographic characteristics, the characteristics of land use and transport system itself. This paper explores how GIS can be used to analyze a sample that represents the population of spatial data and temporal data varied**.**

Demand for transport are usually derived from the demand for primary travel activities. Understanding how spatial environmental impact in the choice of activities is very important to understand the travel behavior and associated impacts such as congestion and decrease air quality from vehicle emissions due to the increase in the volume of vehicle traffic. In addition, consideration of the factors in the socio-economic analysis of transportation demand allows institutions in the field of transport related to not only predict the future needs of the transportation demand, but also allows them to adjust programs and policies to affect the entire system changes such as the shift mode or increase the use of public transport .. There are two main levels of transportation needs analysis; namely 1) the method of aggregate and disaggregate method. Disaggregate analysis done using data collected at the individual level - the choices made on the mode used, participation activity, time of departure, vehicle ownership, income, residential location, school or workplace and several other variables. This data can be collected to understand the travel patterns of areas as small as urban level to the regional level is greater, for example. option for individuals to travel depending on the options available to them, such as the lack of public transit lines operating or transit stop in an area will deter and reduce the mobility. The program enables GIS spatial analysis and mapping to present visually from land use and transport infrastructure and identify potential markets or transport facilities and infrastructure gaps.

Spatial Data

Most of the data will be handled in a GIS spatial data, geographic data oriented. This data has a specific coordinate system as a reference base and has two important parts different from other data, the location information (spatial) and descriptive information (attributes) that is described as follows

- Opera-locations (spatial), associated with a good coordinates geographic coordinates (latitude and longitude) and the XYZ coordinates, including datum and projection information.
- Descriptive information (attributes) or nonspasial information, a location that has some information related to it. Examples of types of vegetation, population, area, zip code and so on.

Measuring Travel Distance

One of the most important parameters in measuring the spatial accessibility is the distance between the supply and the demand locations. Various measures of distance ranging from Euclidian distance to travel times can be identified in the literature. In their study on the accessibility to health services in Montreal, Apparicio et al. (2008) found a correlation of .992 between the shortest network distance and the shortest time distance.. Following this work, we selected the shortest network distance as our distance parameter (Travel-time should be used if distance is a poor measure of travel impedance e.g. if roads are unevenly distributed and travel speeds vary to great extent.

Regression Analysis.

Regression analysis is one of the analysis of the most popular and widely used. Regression analysis is widely used to make predictions and forecasts. This analysis is also used to understand any independent variables associated with the dependent variable, and to determine the forms of the relationship. Regression is composed of one independent variable (predictor) and the dependent variable (Response / Criterion) is called simple linear regression (bivariate regression), while the independent variable regression over a so-called multiple regression (Multiple regression / multivariate regression), which may consist of the two predictors (multiple regression) or more

Analisis Regresi Sederhana dan Regresi Berganda.

Simple regression analysis and regression

Regression analysis was used to determine whether a variable can be used to predict or forecast the other variables, if a dependent variable (the dependent variable) depend on the independent variable (independent variable), the relationship between the two variables is called a simple regression analysis.

Multiple linear regression analysis

In a statistical regression analysis is one method to determine the causal relationship between one variable with another variable. variabelY called independent variables, or variables, variable X (as is often depicted in the graph as abscissa, or X axis). These variables are known as variable that is affected, the dependent variable, dependent variable, or variable Y. Both of these variables can be a random variable (random), but the variable that is affected must always be random variables

Common forms of simple linear regression equation showing the relationship between the two variables: the variable X as independent variable and variable Y as the dependent variable is :

$$Y = a + b \quad (1)$$

$$b = \frac{n(\Sigma xy) - (\Sigma x).(\Sigma y)}{n(\Sigma x^2) - (\Sigma x)^2} \quad (2)$$

$$a = \frac{\Sigma y - b.(\Sigma x)}{n} \quad (3)$$

which showed that:
Yi = dependent variable
X1 =Independent variables
a = intercept (the dot is cut curve on the Y axis)
b = The slope of linear curve

RESEARCH METHODOLOGY

Research Location.

The location study performed at the University of Hasanuddin in Makassar as in Figure 1

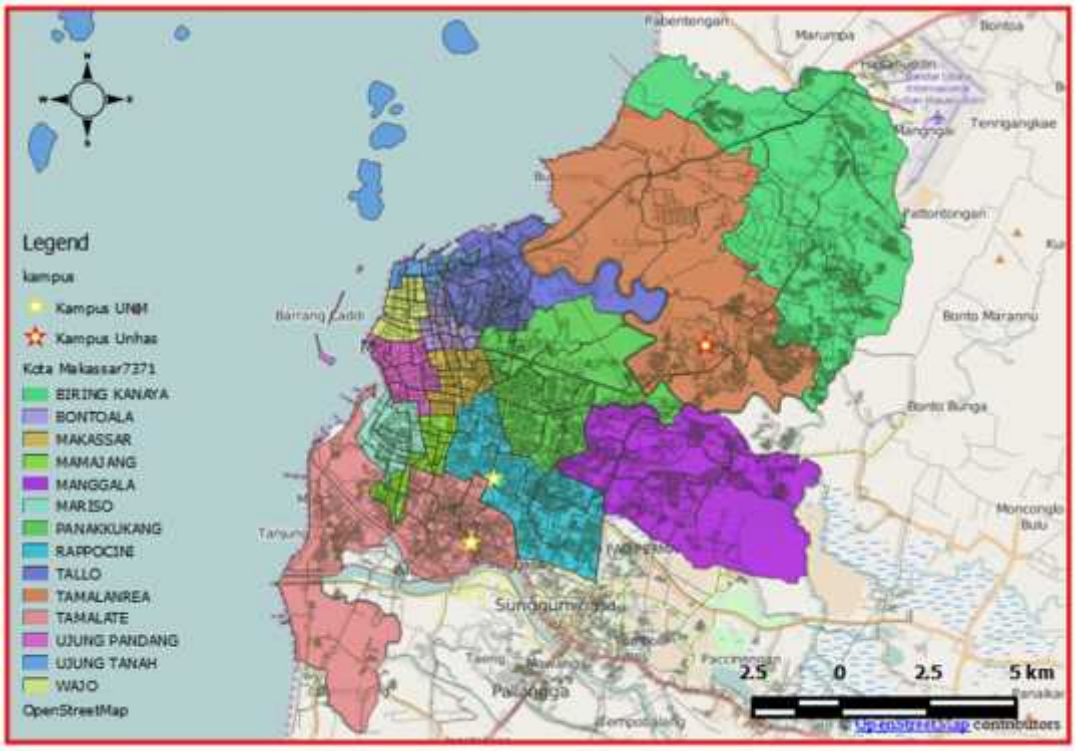
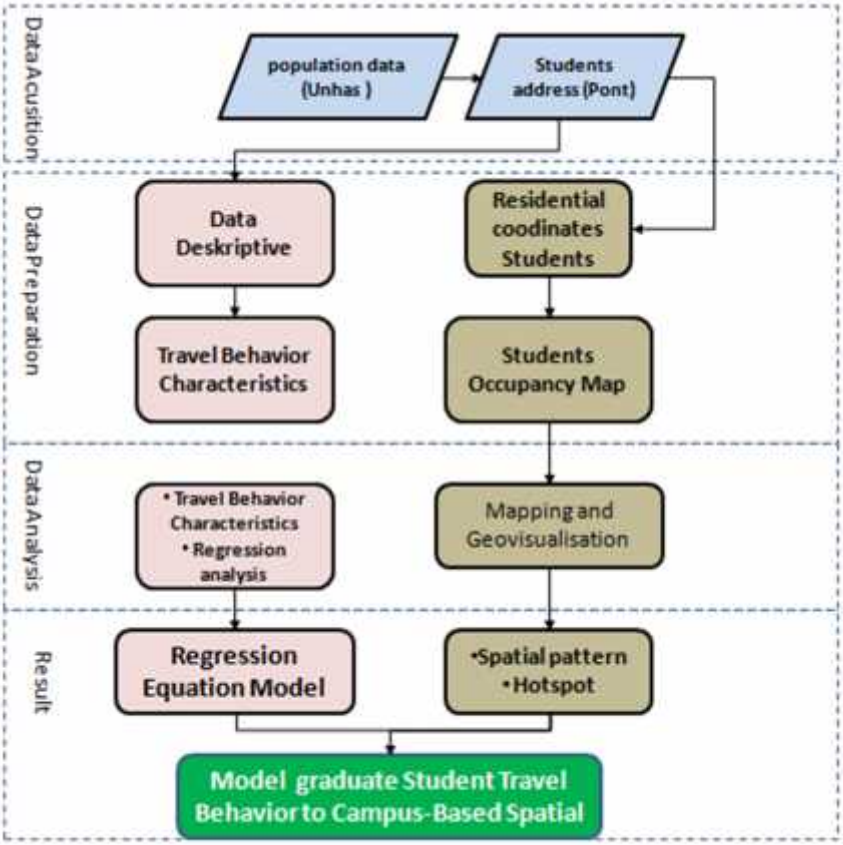


Figure 1: Map Location Research

Methodology

The methodology of this research is presented as Figure 4



Flowchart Metodology

RESULTS AND DISCUSSION

Regression Analysis

From the results of the analysis by the method of regression linear regression model between the dependent variable (Y) is the allowance and the independent variable (X) sort path with R2 values of 0:52. These results indicate that the cost allowance will be increased along with the increase in short path, as well as the relationship between student transportation costs will increase with distance Travel with R2 value of 0.64. Regression model are presented in Table 1 below.

Table 1: Regression Equation Model

NO	Regression Equation Model		R2	t Stat
1	Y=7.55+162X		0.52	7.22
	Y	Allowance		
	X	Short path		
2	Y=7.5544+59.501X		0.64	4.22
	Y	Transportation costs		
	x	Travel distance		

Spatial Analysis

Based on survey data Hasanuddin University students using the GPS device can be mapped locations such as a student residence in Figure 2 below :

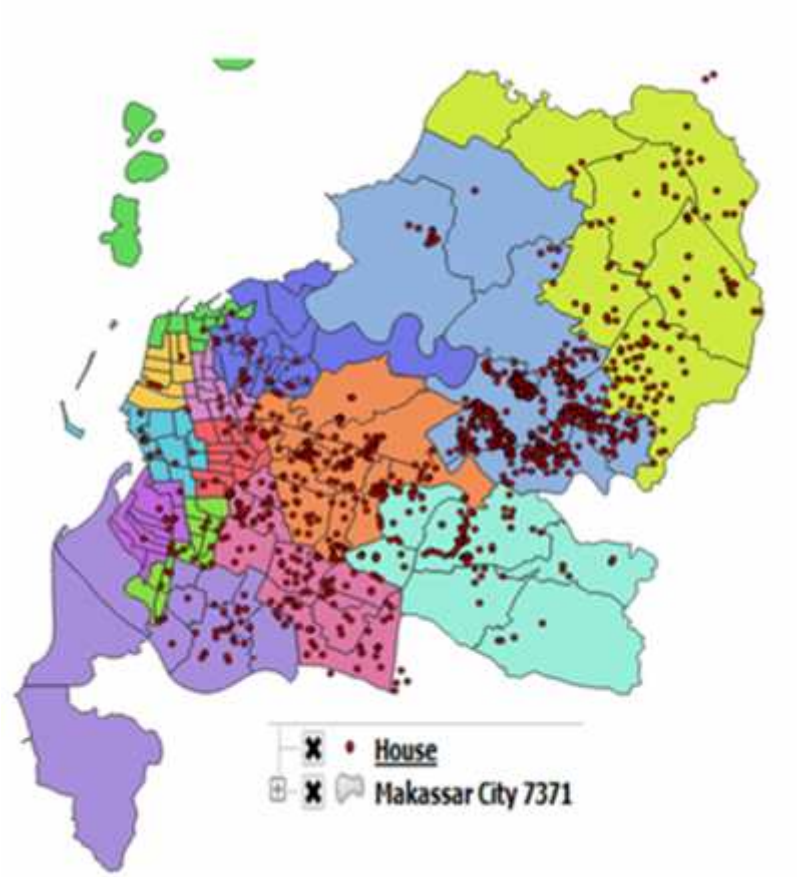


Figure 2: Map of location of residence Students

Based on survey data location Hasanuddin University student residence, then the map Heatmap shows that the location of most of the student housing is located near the campus as indicated on the heat map. The density of the location of student housing can be categorized in four classes: Zero, less dense, dense and very dense in Figure 3 below :

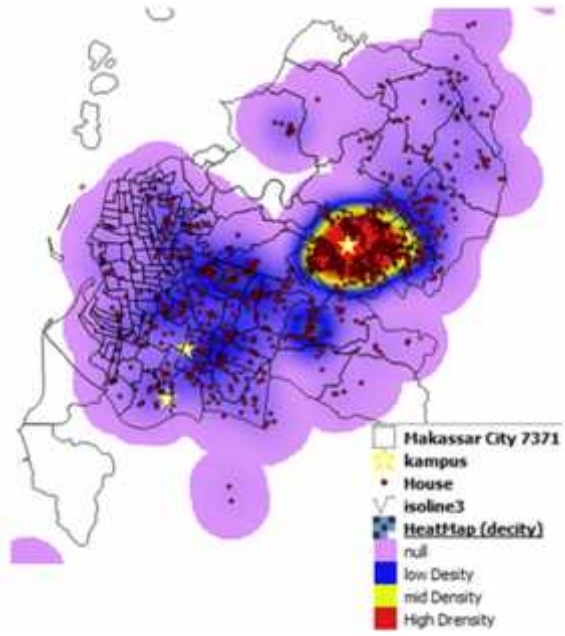


Figure 3: Figure 3: Map of the density of location of residence Students (Hotspot)

Student residential locations can also be analyzed mapped with map isolines (contour) as figure 4 below

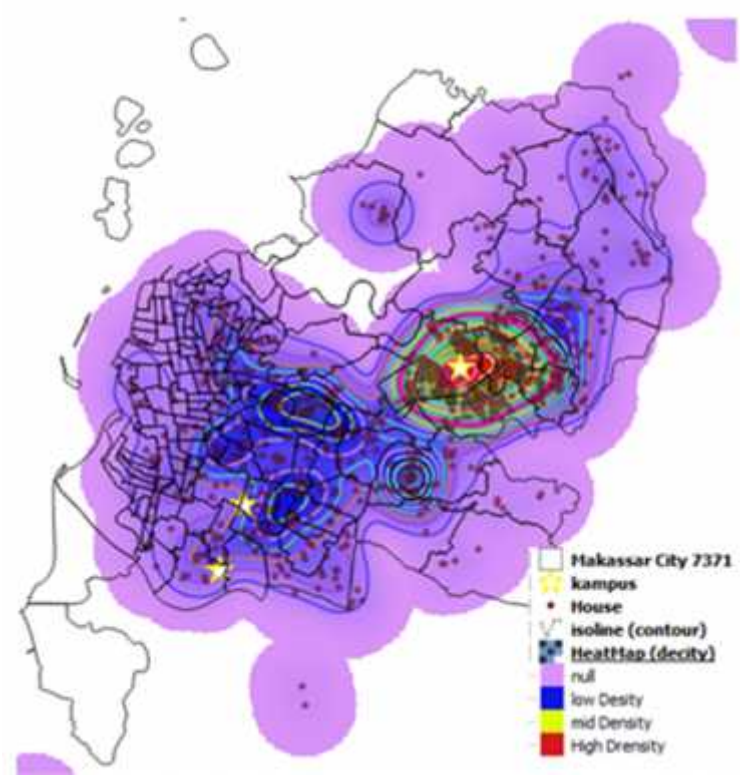


Figure 4: Map of the density of location of residence Students (isoline/contour)

CONCLUSION

From the results of the regression equation, found a significant association between living costs of students with the selection of the location of residence. The greater the distance between campuses with student residential location, then the greater the student living costs. Likewise, the cost of transport on the way to the campus there is a significant relationship with the travel distance. The greater the distance of travel, then the greater the transportation costs.

In this study also, it was found that the students concentrated on the residential locations around campus as shown on the map Hotspot. It is closely related to the level of accessibility and transportation costs and living expenses of students.

In order to make decisions in the field of transport and education, we need to have more detailed information about the characteristics of student travel and transportation facilities and infrastructure information. This method can provide a new perspective is needed for education policy makers to do with transportation planning, decision-makers and local authorities. With the help of GIS can assist transportation planners in analyzing and taking the right policy.

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SELECTION OF AIRPORTS LOCATION IN SMALL ISLANDS USING MULTI CRITERIA DECISSION MAKING (CASE STUDY: KEPULAUAN SERIBU REGION IN INDONESIA)

Doddy Ari Suryanto [1], Sakti Adji Adisasmita[2]

ABSTRACT: The main stages in determining the new airport location based on multi decision criteria were to determine the size of land required; the evaluation of factors that influence the airport location; the study of the possible alternatives area which can be built; soil survey/inspection location; environmental studies; evaluation of the locations that are considered potential; preparation of an outline plan, cost and revenue estimates; evaluation and final selection; and report and recommendations. The purpose of this study were to select the optimal location of airports and feasible in terms of technical viewed from soil investigation, aviation operation safety zone, land use, air transport operational, commercial, and environment aspects, and also preparing a strategy analysis, forecast growth in short, medium and long terms in facing the challenges ahead. The study was conducted by collecting primary and secondary data. The data was based on field survey; interviews; discussions with airport authority, official government, etc; and also from various agencies. There were several aspects that must be considered in the selection of airports on the islands were (1) environmental aspects, (2) the technical aspects, (3) economic, and (4) the social aspect, and (5) the utility aspect. Analytical tools used were: (1) regression analysis, (2) analysis factor of several aspect, and (3) Analytic Hierarchy Process (AHP). The findings and recommendations are, optimal airport location in small island should provide: The technical aspects of airport and availability of lands 39.6%, accessibility 27.4%, land structure of 15.4%, availability of utilities 10.5%, potential users (market) 3.8 %, and the wind speed 3.3%.

Keywords: *Airport Location, Small Island, Multicriteria, AHP*

INTRODUCTION

Soekarno-Hatta International Airport has over capacity. It is seen that at the time was built predicted number of passengers is 20 million passengers a year, and now serves more than 60 million passengers a year more than 300% increase from the initial prediction. Soekarno-Hatta has now ranked as the world's ninth most populous in the world's airports. Therefore, many complaints occur relating to the inconvenience at the airport which became the pride of the Indonesian nation today. In order to anticipate the surge of passengers at the airport, the government is conducting a review of the three locations are airports buffer Soekarnto-Hatta Airport, the airport terminal Kertajati in Java with a capacity of 5-6 million passengers per year, service Karawang planned with a capacity of 30 million passengers per year in the first stage, and service in the Kepulauan Seribu that still must be examined more deeply because it is in the islands.

In the reform era (1998) in Indonesia, a centralized system was replaced by a decentralized system, the government introduced a system of regional autonomy, which authorizes local governments to regulate and manage self-government in accordance with the aspirations of the local area.

In an effort to improve the relationship between air transport system of the area, many regencies in Indonesia wants to build an airport. Airport development requires substantial funds, so that the construction of the airport works well, in terms of effective and efficient, the necessary analysis of detailed feasibility studies and Panjang-term master plan, to obtain the optimal location of the airport within the context of an effective and efficient system of airports.

The factors that must be considered and taken into account in determining the location of the airport, especially in terms of (1) the physical aspects, (2) economic and social aspects, (3) operational aspects, (4) regional planning and development aspects, and environmental aspects. Airport development must be placed at appropriate locations in accordance with their capacity to provide optimal results (Shinya & Batari, 2011).

AIRPORT CONCEPT

According to Annex 14 of ICAO (International Civil Aviation Organization), airport defined as an area on land or water (including any buildings, installations and equipment) intended either as a whole or in part for the arrival, departure and movement of aircraft. Indonesian Government Regulation No. 71 Year 1996 on Airport Affairs article 1

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says: "Airport is used for landing and takeoff of aircraft, passengers boarding, and cargo or mail unloading, and equipped with facilities aviation safety and as a place of transfer between modes of transport ". (Ashford, N & Wright, 1992)

Feasibility studies and construction of airport development is basically doing a feasibility study and construction of airport development and draw up a basic plan development and airport development. In the feasibility study stage also performed the analysis and evaluation of alternative election airport location for the construction of the new airport.(Horonjeff, 1988)

Details of the required feasibility analysis in the feasibility study construction of airports are as follows (Permenhub No.11 / 2010 on the Order National Airport): (1) Feasibility Analysis and Integration with Regional Planning; (2) The Economic and Financial Feasibility Analysis; (3) Technical Feasibility Analysis Development; (4) Analysis of Operations and Flight Safety; (5) Exploitation Feasibility Analysis of Air Transport Services; and (6) Environmental Feasibility Analysis.

AIRPORT LOCATION

There were several factors to determine and select of the new airport location, used as a guide for future development. In general, the factors can also be used for the development of existing airports.

Airport location will be affected by factors (1) Location of the airport away from residential areas; (2) The availability of land for development; (3) The existence of another airfield; (4) Obstacles around (Surrounding Obstruction); (5) Facility to obtain land transport; (6) Cost of Construction; (7) Availability of utilities and (8) Proximity to request aeronautics.

Several previous studies illustrate aspects that determine the determination of the location of airports, namely (1) the economic aspect; (2) social and cultural aspects; (3) availability aspects of accessibility; (4) aspects of the status of the land and (5) environmental aspects (Janic Milan & Aura Reggiana, 2002).

The interesting thing about this airport selection analysis, that there are no default rules detailed in Indonesia and has not been widely studied by other researchers. It is important for the analysis because Indonesia is an archipelago with a lot of tourists area small islands, so that with one of its objectives is to support the development of the area and the most important thing is to support the larger airports in the vicinity, so the concept of the network air transportation can be performed well.

Airports in the islands have a high level of dependency on the climate around him, especially the wind conditions, characteristics and availability of land on the island, the wave conditions, topography and the most important is the accessibility plans of nearby islands and requests passengers to the airport (Athanasios Ballis, 2003).

Based on Table II PM Transportation No. 69 In 2013 it can be seen that kind of airports that are in isolated areas, such as Stone Island Airports Lasondre in North Sumatra, service Sipora Rokot in West Sumatra, service Oesman Rahadi in Ketapang in West Kalimantan, and others have a class 3C airport and serves as the airport feeder or aggregator.

There are several aspects that must be considered in the selection of airports on the islands are (1) environmental aspects, (2) the technical aspects, (3) economic, and (4) the social aspect, and (5) the utility aspect. The environmental aspects of a Kepulauan Seribu most important thing is the location is not included National Park Zone, low wind speeds. The technical aspects have seen from availability of land, soil bearing capacity. Economic aspects related to the tourism potential and the potential of the local movement of passengers and goods. The social aspect was influenced by the accessibility of facilities and logistics administration. Utility aspect was influenced by the availability of facilities such as electricity, clean water and etc (Putri et al, 2015).

AIRPORT FEASIBILITY STUDY

Determination of the airport optimal location to be viable and meet the data collection phase and policies, forecasts, consideration of new technologies, system requirements, an alternative system, and the implementation of development plans (Caroline et al, 2003).

Data collection and policy, including (1) the type of flights that will be served, commercial and general aviation, helicopter, military, (2) the maximum flight route will be served, (3) environmental effects that would arise, (4) land use and planning transportation from the government, (5) economic factors, and (6) the source of funds for the design and construction. Forecasting, covering traffic; routes to and from the area were to be developed for the airport, including passengers, aircraft. Considering of new technology, including aircraft and airport facilities (particularly for air navigation systems). The requirement System, including aircraft specifications, capacity and facility needs analysis (flight facilities, terminals, roads, utilities, and air navigation). The systems alternative, including the study of alternatives, and use selection system. The implementation of development plans must be based on government policy, environmental considerations, the overall project implementation schedule, financing, land acquisition, implementation schedule development, and design operational implementation (Morlok and Edward, 1998).

Based appendix IIIB PM Transportation No. 69 In 2013 it was stated that there are policies airports as opener isolated areas with few criteria and indicators, namely (1) located in the interior or mountains, islands, coastlines; (2) limited accessibility; and (3) low levels of public life (PT. Angkasa Pura, 2013).

ANALYTICAL HIERARCHY PROCESS

The working principle AHP is a simplification of a complex problem that is not structured, strategic and dynamic into its parts, and arrange in a hierarchy. The level of importance of each variable given a numerical value subjectively about the significance of these variables relative to the other variables (Saaty, 1994). Stages in the AHP method is as follows : (1) define the problem and determine the desired solution, (2) create a hierarchical structure that begins with the main objective, (3) perform pairwise comparisons using a Likert scale (1 - 9) that shows the comparison of the level of interest, (4) create a pairwise comparison matrix that illustrates the relative contribution or influence of each element of the destination or the level above criteria, (5) calculates eigen vectors of each pairwise comparison matrix which is the weight of each element for the determination priority elements at the lowest level of the hierarchy until it reaches the destination, (6) check the consistency hierarchy is determined by the value of consistency ratio (pairwise comparison) where the value of CR <1 (Rickard Bergqvist et al, 2014).

RESEARCH METHODS

Data collection consists of primary and secondary data, primary data obtained through surveys using questionnaires, interviews, and observations, secondary data collected from the central statistics bureau, department of transportation, and so on (Figure 1).

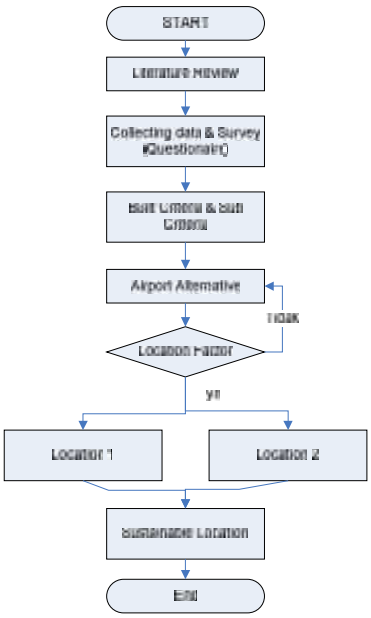

Fig. 1 Flow Research

Analysis tools used are (1) statistical analysis using factor analysis for the selection of the most influential factor for the selection of airports in the Kepulauan Seribu, (2) scoring method (Likert scale) that is used to see the weight of the factors that most influence the election airports, (3) Making Model Analytic Hierarchy Process is used to make decisions in choosing one of the best airports by scoring has been done.

RESULTS AND DISCUSSION

Based on standard factors used in airports in general elections by using factor analysis models obtained Analytical Hierarchy Process (AHP) as shown as Fig. 2.

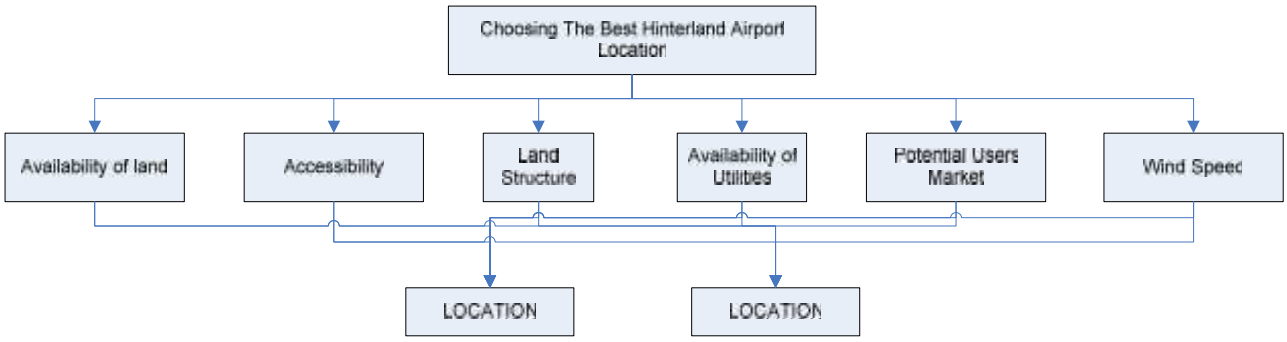

Fig. 2 AHP Model

Kepulauan Seribu is a group of islands located in the north of the Bay of Jakarta and Jakarta Java Sea. The most northern island is Sebira island located 100 miles from the mainland Jakarta Bay. Judging from the position of the city of Jakarta as the city of Bandar, then the position of Kepulauan Seribu is on the face of Jakarta. Location Kepulauan Seribu are geographically located between 06 ° 00'40 "and 05 ° 54'40" south latitude and 106 ° 40'45 "and 109 ° 01'19" east Longitude. In the western part, there are some small islands which have mostly been used as a place of settlements and partly used as a tourist resort. The total land area of Kepulauan Seribu regency is 897.71 ha and the Kepulauan Seribu waters reached 6997.50 km^2. There number of islands in the Kepulauan Seribu is 110 islands with composition (Figure 3).

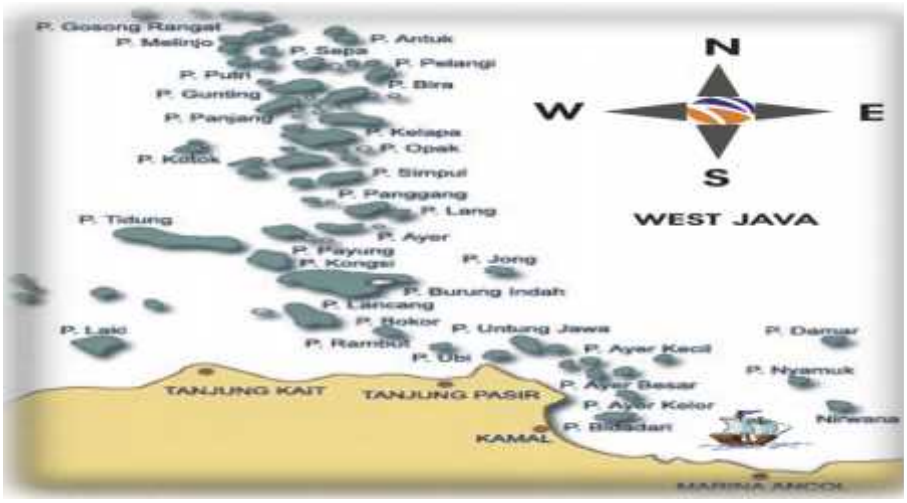

Fig. 3 The Islands in Kepulauan Seribu.

Kepulauan Seribu currently has a runway that has a long heritage taken care not owned by Pelita Air Airlines precisely in Panjang Island which has an area of 1.4 km^2. The condition of the runway has a length of 930 m. The runway was originally used by the owners of villas and cottages to transport from and to Jakarta by using a private plane.

Considering foreign tourist visits and the archipelago that come to the Kepulauan Seribu continued to increase in 2013 amounted to 1.49 million visitors and up 300% in 2014 (Table 1), and the central government's plan to make the Kepulauan Seribu airports into airport Soekarno-Hatta buffer, it is necessary to study the feasibility of the old airport and the alternate location of a new airport in the Kepulauan Seribu because of many factors that must be considered mainly related to the characteristics of the Kepulauan Seribu.

Table 1 Number of Tourist in Kepulauan Seribu

Tourist Type	2008	2009	2010	2011	2012	2013	2014
Foreigner Tourism	3.009	3.316	4.786	6.692	8.422	15.521	21.488
Local Tourism	19.734	137.910	226.234	592.306	651.237	1.482.949	3.009.151
Total	132.734	141.226	231.020	598.996	659.659	1.498.470	3.030.639

The initial plan was to look at a typical airfield in the Kepulauan Seribu is not used for commercial airfield, but only for transit tourists and distribution lines staple for people in North and South Kepulauan Seribu, but this time, the government plans to build a service class IIIC Kepulauan Seribu. Based on this background, the researchers will conduct analysis of alternative selection service locations Kepulauan Seribu, assuming at least two alternative options besides the airport in Panjang Island.

The Kepulauan Seribu Airport was planned to have classes IIIC serves as a tertiary scale collector airports. Under the rules, the airport must have a minimum of airside zone and the zone lanside. On the landside zone planned minimum ARFL (Aeroplane Reference Field Length) 1400 meters, with the best characteristics have wings span (Wing Span) of at least 24 m at a distance of outer main wheel (Mean Outer Gear) 6 m, with a shoulder width of 30 meters and 15 meters. The aircraft served is kind of turbo-prop or jet small as Fokker-27, Tetuko 234, Fokker-28, etc.

Passenger terminal for small aircraft has three spacious passenger terminal based SNI 03-7046-2004, namely 120 m^2, 240 m^2 and 600 m^2 both domestically and internationally by meeting the requirements of security and safety of flight operations. Terminal plan has several facilities such as, (1) an apron; (2) Cargo Terminal; (3) Terminal passengers; (4) Parking; (5) Ground Service Equipment and (6) Regulatory Flight (PK-KDP).

Kepulauan Seribu with 110 island has a land area that is allowed to be made with a class 3C airport there are two locations, there are 24 island has an area of over 10 hectares. The archipelago has an average rainfall of 100-400 mm and reaches an average of 10-20 days / month. The wind is blowing in the Kepulauan Seribu are usually influenced by the Monsoon winds. The wind speed in December to February range between 7-20 knots, while in other months ranged from 7-15 knots. Sea depths ranging from 0- 40 meters on the island of Pulau Pari umbrellas and more than 40 meters.

The soil has a Kepulauan Seribu sloping topography between 0-15% with a height of between 0-2 meters below sea level and affected by the tide. Of the 110 islands that there were only a few that have a type of tropical heat with temperatures between 26-27 degrees Celsius and humidity of 80%. In general, the condition of the soil depends on the density of vegetation, to islands that have dense vegetation and have a fairly thick layer of soil, then water the soil condition is good and fresh.

Based on several matters relating to the aspects that must be met for the planning of airport, there are several islands that can be used as airports, namely the Panjang Island and Pulau Tunda.

Results of a survey of experts found several aspects that influence the selection of airports in the Kepulauan Seribu is the environmental aspects such as wind speed, rainfall, technical aspects such as land requirements, the height of the sea surface, the characteristics of the soil and sea water, the condition of infrastructure and accessibility.

The results of calculations using AHP obtained Consistency value ratio (CR) of 0.037104 to the main criteria is the result of a survey questionnaire with PT. Angkasa Pura used to determine appropriate areas for the location of the Kepulauan Seribu airports.

The technical aspects of airport and availability of land 39.6%, accessibility 27.4%, land structure of 15.4%, availability of utilities 10.5%, potential users (market) 3.8 %, and the wind speed 3.3%.

Panjang Island has a condition of some aspects of airport planning, among others, examination of the environment of this island is not included in the zone of the national park, has a wind speed of 18 knots, rain spout average of 3000 mm / year. Viewed from a technical aspect, Panjang Island area 12.9 hectares with conditions coordinate position 05 ° 38'40 "S, 106 ° 33'45" BT with an average height of 3 meters above sea level, the type of soil on the land is coral sand the weathering of coral reef limestone with a thickness of between 1-5 meters. Conditions waters have a temperature of 29.4 degrees Celsius, salinity 30.1 ppm, the brightness of 6.6 meters and a flow of 0.15 cm/sec. Panjang Island is located in a collection of small islands adjacent to Hope Island is planned to be built warehouse storage (hap) for the needs of the surrounding island as the island is a collection of more than 50% of the islands in the Kepulauan Seribu. Access from the island farthest North Kepulauan Seribu can be taken approximately 2 hours by fast boat and 4 hours by boat ojeg or timber ships. Panjang Island is also located 20 minutes from Scout Island is the center of local government administration Kepulauan Seribu regency, so the whole ojeg and ship transport in the Kepulauan Seribu entering this Scout Island. The condition of the infrastructure around Panjang Island is supported by the small islands around it especially Scout Island, Panjang Island conditions thus supporting infrastructure was supported by the presence of Scout Island.

Tunda island has an area of 250 hectares with the trip will take approximately 2 hours by fast boat from the Port Karangantu in Serang City Banten, located in the Southern part of the Kepulauan Seribu with coordinates 50 48 '43 "latitude and 1060 16' 47" BT mean temperature The average of 29.4 degrees Celsius, the level of salinity 33 ppm, the brightness of 3-6 meters and the sea currents 0.09 cm / sec. Geological conditions consisting of limestone rocks (rock), sand and sediment originating from the island of Java and the Java Sea and metamorphic sedimentary rocks, limestone, clay stone that is known as the island made of frozen lava deposition. Flat topography of Tunda Island 0-3 m above sea level with the eastern part of 1-2 meters higher than the western part. Section morphology sandy beaches and mangrove vegetation in the east and south of the island. Land in Tunda Island is dominated by shrubs, only about 10 hectares of land used for residential areas and public facilities.

Tunda Island is located about 1 hour from Tanjung Pasir Banten region. Administratively, this island has to offer north in Banten Province, so that the location of their accessibility closer to the Soekarno-Hatta airport, so the level of

accessibility is very good. A source of electrical energy on the island is derived from diesel and solar power that is able to produce electricity 35 KW and operating costs Rp 3,500 - Rp. 6500 per night. Source of drinking water comes from riverse osmosis drinking water treatment.

CONCLUSION

Based on the above research, then by looking at the criteria of the many factors value that influence the selection of the location of airports like technical aspects of airport and availability of land 39.6%, accessibility 27.4%, land structure of 15.4%, availability of utilities 10.5%, potential users (market) 3.8 %, and the wind speed 3.3%, and destinations in the rise and airports in The Kepulauan Seribu then obtained a sustainable location for the Airport is in Panjang Island

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CHARACTERISTICS AND PATTERNS OF PEOPLE MOVEMENT OF USERS CITY TRANSPORT USERS IN FULLFILLING DEMANDS IN PALU

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ABSTRACT: One of the causes of the problems of urban transport is not well identificated characteristics and travel patterns of the population in fulfilling of everyday demands that are scattered in the space/area in urban areas. Characteristics and population movement patterns is formed of spatial and non-spatial pattern. Spatial patterns is carried out on the basis of travel activity in a particular location by observing the condition of the land use of an area/region, whereas non-spatial movement is a movement pattern that knows no boundaries of space/area. This study aims to identify the characteristics and trip population patterns that use city transport in the city of Palu in fullfiling their daily activities demans based on the family data characteristics; the average of monthly income, the average of monthly cost of transportation realesed and residential home building area (m^2), while the population movement patterns such as; age and gender, occupation, original travel destination, travel time and the type of vehicle used. Data collection techniques are done randomly by using of interviews home in 8 districts which are scattered in 45 villages with a total sample of 1,749 households collected of 348 city transport users. The analysis used is descriptive analysis. The research results showed that the characteristics and travel patterns of the population is dependent on the distribution pattern of land use in the region, the majority of travel demands of the population in Palu city leaded to the downtown area from various parts of the districts. The biggest movement was dominated by educational journey. Peak hours of travel time occurred during morning hours of 6:00 to 07:00 and movement patterns dominated by urban transportation which travell to the town center with a variety of travel destinations.

Keywords: *characteristics, movement patterns, public transportation and demands fulfillment.*

BACKGROUND

The movement occurred because of the fulfillment of demands. Fullfiling demands of an activity that is usually carried out every day, such as; fulfillment of economic, educational, entertainment, social, cultural and so on. One of the reasons for movements in urban areas is about the socio-economic tendencies, the high mobility of people as a result of the increase of people private vehicle. This case is supported by the development and improvement of the road infrastructure so that travel becomes increased and long, as a result of the growth in the number of vehicles will increase the volume of flow traffic which will ultimately have an impact on travel delays, congestion, waste fuel and various other environmental impacts.

Tamin (2000) states that the factors affect the movement is a socio-economic conditions such as the number of family members working and family income. Something that characterizes the family income as vehicle ownership, patterns of land use and development as well as the circuit avalaibilty. According to Warpani (1990) some factors determine of movement that can be applied in Indonesia are: family income, the number of vehicle ownership, distance from the center of city activity, mode of transport, utilisation of vehicle and time. Family movement is closely related to activities conducted by the family. Meanwhile activities are strongly influenced by family characteristics, such as: family income, ownership of vehicles as well as the structure and number of people per family. All these characteristics affect the level of family demands and of family movement frequency.

There are several reasons why a movement is conducted. Firstly is the fact that the movement is required by the human to fullfill their requirements for daily activities, both on a local scale in which they live and in the inter-regional scale. In the local scale condition, movements arise because of human activities are not always able to do in their own place where they live, whether in the office, at the factory or on the farm. While the larger area scale found the fact that the spatial separation between the potential of the resources with other resources. While utilizing a resource in an area will require other resources from other places in such a way would require movements of the resources from one place to another.

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The magnitude of the trip depends on the activities of the city, while the cause of the trip is the human desire to meet their requirements that are not obtained at the place where they live. Generation and attraction of the trip varies for each type of land use. The higher level of land use will be higher movement generated (Tamin, 2000: 60). Distribution of this movement shows where and from where the traffic moves in a region. Distribution patterns of traffic flow between zones of origin to destination zones are the result of two things happen simultaneously, namely the location and intensity of land use that will generate traffic flow and space divider, as well as the interaction between the two land uses will generate movement people and/or goods. The higher the intensity of land use, the higher the level of ability in attracting traffic, but if the distance is longer will reduce the attractive of the land use. The transport system can only reduce barriers to movement at inner space, but can not reduce the distance. Therefore, the amount of traffic movement between the two pieces of land use depends on the intensity of land use and the separation space (distance, time, cost, modes and routes) between two zonas. Because of this reason traffic flow between two pieces of land use has a positive correlation toward the land use intensity and a negative correlation toward distance (Tamin, 2000: 64).

Palu city as the capital of Central Sulawesi province, with a variety of activities will atrract movement of people and goods from various regions and hinterland regions to meet their various demands / activities, so public transportation (angkot) become the primary choice in traveling, especially for the captive users (no choice) other than public transportation.

Problem:

How to set a map related to characteristics and patterns of population movement of city transport users in fullfiling their travel demands in the Palu city?

Research Purposes

Answering problems that is being faced with by the public transportation users, by identifying the population movement characteristics and patterns under utilization of city transport in Palu city.

Benefits of Research

Results of this research are expected to be usefull input, as:

- Clausal considerations that can be used in forming a decision policy related to the urban transport service system in favor of the user community.
- Materials or scientific reference for further research in the field of urban transport.

LITERATURE REVIEW

Characteristics of Pattern Movement

The movement was formed as a result of the activities conducted out of the place of residence. This means that linkages among regions are crusial instruments in creating travel and distribution patterns of land use that greatly affects travel patterns of people (Tamin, 2000). Indonesian general dictionary defines the behavior as character, temperament, behavior, whereas in the large Indonesian dictionary behavior interprets as a response or reaction of individuals who manifest in movement (gesture), not just the body or speech. In the context of collective behavior is defined as those activities together in a certain way and follow certain patterns as well. So the travel behavior can be interpreted as a human behavior in traveling to their destination.

Spatial patterns of movement and Non Spatial In Transportation Systems

Movement patterns in the transport system consists of two patterns of movement, namely: (1) The pattern of spatial movement and (2) non-spatial pattern of movement (Tamin, 2000).

- Movement Spatial Patterns

  A movement pattern that is done on the basis of travel activity in a particular location by observing the condition of the land use of an area / region. Spatial movement within the region consists of:

  - ✓ travel patterns of people

    Travel patterns are influenced by work activity and residencial. Pattern of this trip has a spatial distribution, such as offices, housings and shops.
  - ✓ Travel goods Patterns

    Travel patterns are influenced by the activity of production and consumption marked by the movement of the distribution of production centers to consumption locations.

  Both are highly dependent on the distribution pattern of land use in the region.

  The opinion on the classification of movement patterns revealed by Chapin (1965) consists of five patterns of movement, namely:

  - ✓ Radial: the movement coming from suburban neighborhoods to get the CBD for a particular purpose.
  - ✓ Circumferential: Movements originating and aims in the suburbs.

- ✓ Through: the movement is just passing through town with the origin of the outer city.
- ✓ CBD (Central Business District): The movement that occurs only in the CBD.
- ✓ Sub Urban Activity Center (SAC): The movement that led to the SAC / suburban activity centers.

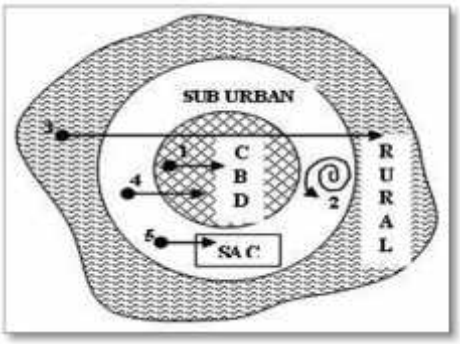


Fig.1. Classification System Movement
Source: Chapin, 1965 : 344

- Movement of Non-Spatial Patterns

  A movement pattern that knows no boundaries of space / area. This movement pattern is composed of:

  ✓ Type means of transport

  In traveling choose the type of transport is the most important action. Choosing transport moda usually consider several factors, including the purpose of travel, mileage, cost and comfort level.

  ✓ Time of movement

  The timing of the movement of people rely heavily on routine daily activities. Someone is doing activities can be done in the morning, noon, night depending on the purpose of the journey.

  ✓ Reason of movement

  The reason for the movement can be classified based on the purpose of the movement is related to the educational, social, cultural, economic, religious and so on.

Thus, the spatial pattern of movement and non-spatial carried out on the basis of the trip by taking into account the condition of the area of land use / urban. In forming the spatial pattern of this movement is reflected by the movement of people who use the mode of transportation within a specific time with a specific purpose.

METHODOLOGY

Location Research

Research studies conducted in Palu city with a population of 347 257 inhabitants who inhabit an area of 395.06 km^2 and consists of 8 districts with 45 villages.

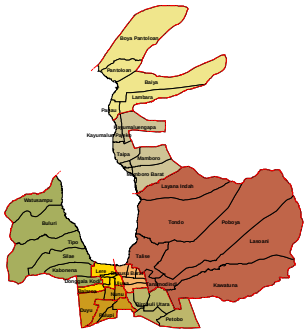
Palu city as a widening Result of 8 Regional District	Based on sub-district	Population (People)	Area (Km2)	Population Density (people/km^2)
	1. Ulujadi	25.057	40.24	623
	2. Palu Barat	58.306	8.28	7.042
	3. Tatanga	35.562	19.49	1.825
	4. Palu Selatan	64.113	27.38	2.342
	5. Palu Timur	67.385	7.71	8.740
	6. Mantikulore	57.044	208.80	273
	7. Palu Utara	21284	29.94	711
	8. Tawaeli	19.105	59.75	320
	Total	**347.257**	**395.06**	**881**

Fig. 2. Location Study Areas
Source: BPS of Palu 2014 Already Processed

Data Source

- Primary Data
  Primary data were obtained through questionnaires with home interview method (door to door), which is guided by a surveyor who is equipped with a questionnaire sheet related to the characteristics and patterns of population movement in the fulfillment of their requirements.

- Secondary Data
  Secondary data is data or information that has been collected by others and the researchers only serves as user data. The data is in the form of literature, the data of government agencies, magazines, journals, maps related to the study area, regulatory / government policies and from the internet.
  Secondary data is expected to be obtained from various agencies and institutions such as Bappeda, BPS, Department of Public Works, and the Department of Transportation and Information Palu.

Population , Sample and Survey Implementation
Population
The population consists of various professions / jobs, gender, age, status and income levels. In order to meet the target of observation diversity / people characteristics movement as public transportation users, the data collection was done in a balance and avoid any data that dominated the group specific data. The type and number of observations designed to fit the analysis plan. For that the data collection is done by observing the target respondents were proportionally divided into: residential areas, residential type on the user population living in 8 districts with 45 villages.

Samples
Determination of the number of samples in the household interview survey (home interview) based on the number of population in the study area or in other words samples taken is depending on the size of the entire population in the study area. According to Black (1981) Provisions number of samples taken in the Household survey interviews indicated according to the table below:

Table 1. Sample Minimum Recommended in Home Interview Survey are as follows:

The county	Recommended amount of Sample	Minimum sample
< 50.000	1 in 5	1 in 10
50.000 - 150.000	1 in 8	1 in 20
150.000 - 300.000	1 in 10	1 in 35
300.000 - 500.000	1 in 15	1 in 50
500.000 - 1.000.000	1 in 20	1 in 70
> 1.000.000	1 in 25	1 in 100

Source: Black, 1981

The calculation method of determining the number of samples
The total population of the city of Palu in 2013 amounted to 347 257 inhabitants and according to statistical sampling, sampling for household interviews are:
- Number of residents: 300000-500000 people,
- Samples are the recommended: 1 in 15 (6.67%) and the minimum sample: 1 in 50 (2.00%)
- In view of limited time and personnel available, then the sample is taken at least 1 in 50 (2.00%). then the sample size recommended is 347 257 x 2.00% = 6945.14 inhabitants or 1,736 households by assuming that each family there are four members of the family.
- To anticipate that the number of samples of 2.00% can be achieved because the data is defective (can not be used), then added to the tolerance of 0.75% of the total sample of 1736 households in order to obtain the data for 1749 households.

The sample size of 1749 households is obtained by dividing the 8 zones of the districts to 45 urban villages and sub zones of each of sub zone is further divided into sub zone RW. Based on this amount of the sample, is obtained 348 people who use public transportation in fulfilling of their demands.

Implementation of Survey
Implementation of the survey interview was conducted from house to house (home interview survey) by visiting households randomly selected (random sampling). Surveyor assigned attached with; identity cards, permits from the relevant agencies, and form to be filled (questionnaire).

Interviews were conducted with the head of the family or one of the family members over the age of 20 years or who knows for certain activities and trips conducted by each member of family. Moreover, surveyors also equipped with such equipment; clipboard, stationery, survey forms, raincoats and motorcycle. The survey was conducted two shifts per day ie morning between 08:00 to11:00 and in the afternoon between 15:30 to 17:30.

RESEARCH RESULT

The study result related to the characteristics and patterns of population movement transport users in the city of Palu city in fulfilling their needs, described as follows:

Characteristics of the City Transport Users in Palu

The results of general information of the household survey on the family characteristics as the transport users in the city of Palu city which is based on; income, the average monthly transportation cost and residential home building area (m^2), presented in accordance with the following picture:

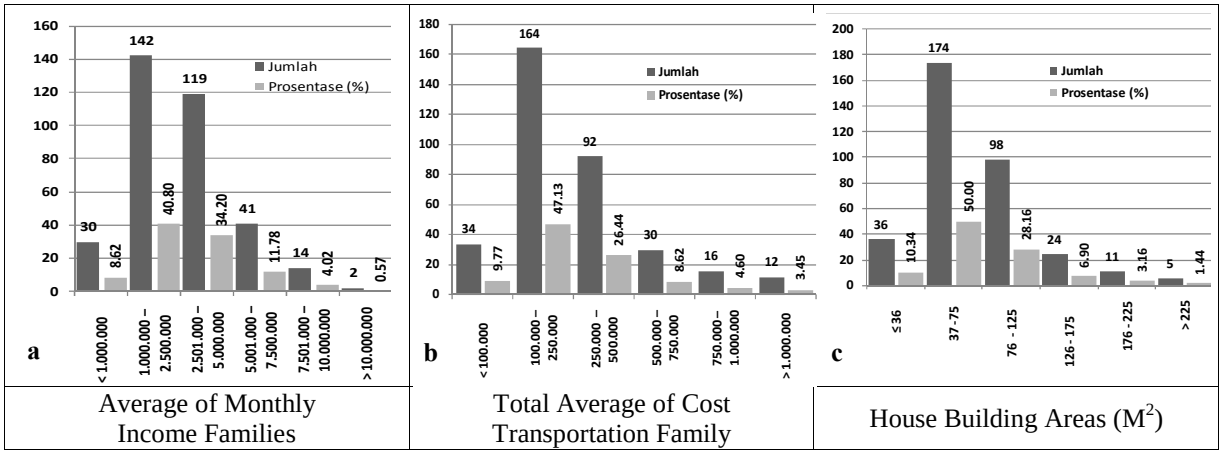


Fig. 3 Characteristics of the City Transport Users in Palu

On average Monthly Income Families

Total monthly family income of each household is the amount of revenue generated by all the family members who already work and has income. From the data shows that the largest city transport users with the amount of revenue Rp.1.000.000 to Rp.2.500.000 of 40.80%, followed by income Rp.2.501.000 to Rp.5.000.000 at 34.20%, further revenue Rp.5.001.000 to Rp.7.500.000 and ≤ Rp.1. 000.000 much as 11.78% and 8.62%, while the smallest city transport users with the level of income of Rp.7.501 million to Rp.10.000.000, 4.02% , and > Rp.10.000.000 of 0.57%

Total average of Transportasi costs/ Month

The amount of the average transport costs realese for transport costs per month issued every month to every household including users who use public transportation. Total transportation fee 100,000 to 250,000 or 47.13% , followed by 250,000 to Rp.500.000 or 26.44%, and as big as 9.77% ≤ 100,000, while expenditures amounted transport costs Rp.500.000 to Rp.750.000 and Rp.750.000 to Rp.1.000.000 amounted to 8.62% and 4.60%, and the expenses of transportation > Rp.1.000.000 is of 3.45%.

Area of House Resedential

Size of building homes in the town of Palu residents who use public transportation of his journey in fulfilling their needs was the largest types of 37 to 75 (m^2) or about 50.00 %, followed by the type of 76 to125 (m^2) or around of 28, 16 %, while the type ≤ 36 (m^2) and type 126 to 175 (m^2) or approximately 10.34 % and 6.90 %, while the smallest of travelers with housing type of 176 to 225 (m^2) or about 3.16%, and the type of > 225 (m^2) or in average of 1.41%.

From these results indicate that the characteristics of the families in the city transport users to travel in order to meet their daily needs is highly correlated with; (1) the average monthly income of a family member, (2) the average monthly cost of transportation realesed and (3) building area of resedential house.

Characteristics and Population Movement Patterns City Transport Users

An overview of the characteristics and movement patterns of the population using public transportation in fulfilling of their daily needs in the city of Palu, presented in accordance with picture 4,5 and 6 below:

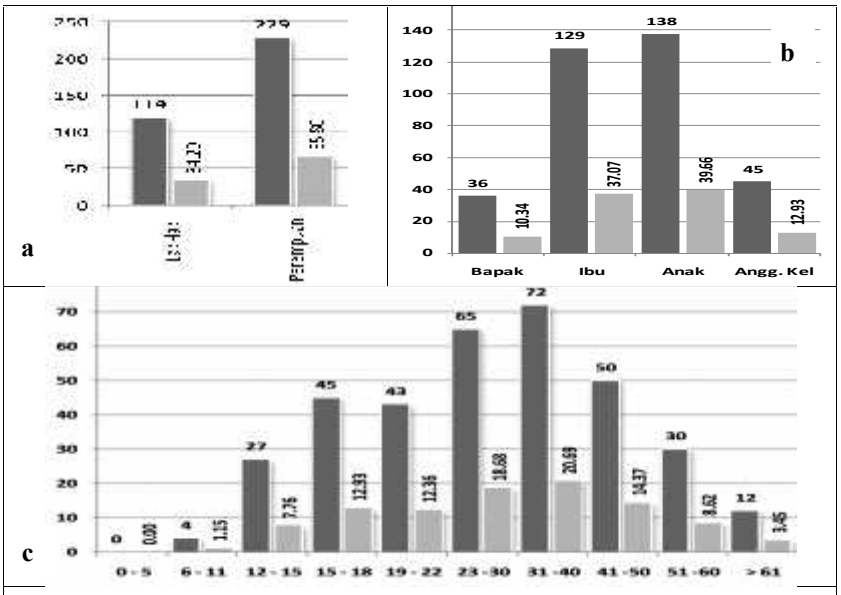


Fig. 4 Travelling Characteristics of City Transport Users

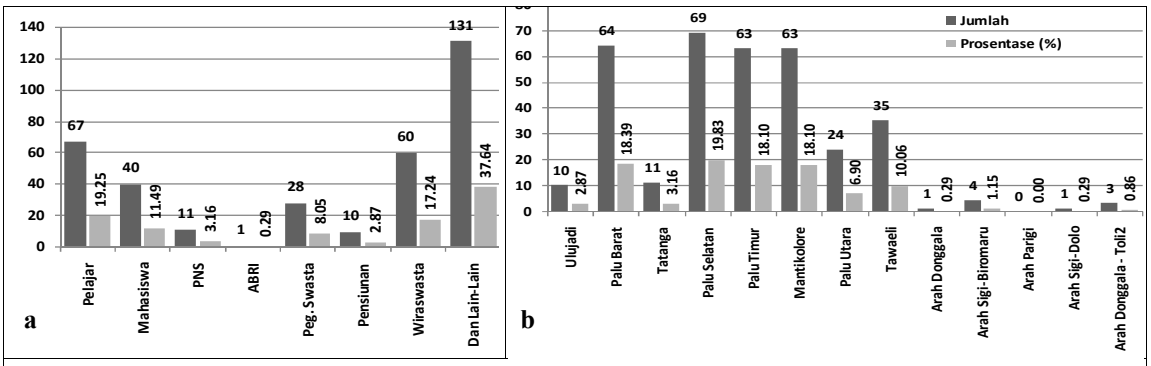


Fig. 5 Travellers Occupation Type of Travel and Destinations users

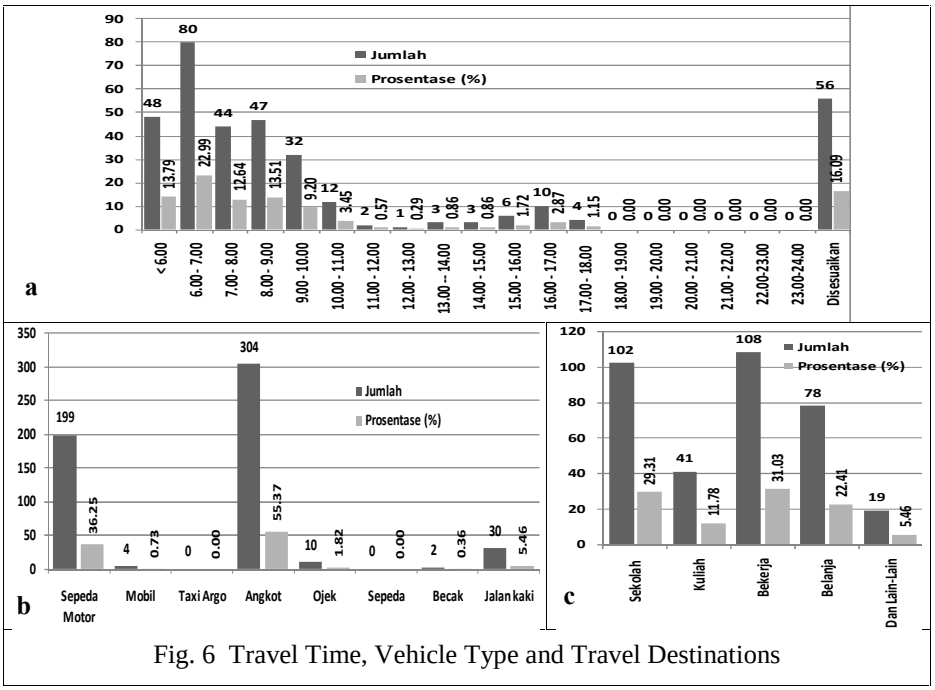


Fig. 6 Travel Time, Vehicle Type and Travel Destinations

Characteristics and patterns of trip of population transport users in the city of Palu city in fulfilling their everyday life needs are shown according Figure 4, 5 and 6, with the following explanation:

- The largest travel of city transport users in Palu city, on the dominance of the female sex 65.80%, the rest were men of 34.20%, (Fig. 4a).
- Based on membership in the family structure, the largest travel is a child of about 39.66%, followed by 37.07% of mothers, family members in the household 12.93% and the last are the fathers 10.34%, (Fig. 4b).
- Based on the age of the traveling public transportation users, most are aged of 31- 40 years, 23-30 years and 41-50 years with the percentage of each 20.69%, 18.68% and 14.37%, followed by age of; 15-18 years, 19-22 years and 51-60 years respectively or 12.93%, 12.36% and 8.62%, and the smallest is age of; 12-15 years, > 61 years old and 6-11 years respectively or 7.76%, 3.45% and 1.15%, (Fig. 4c).
- The other type of work that dominated by traveling is not include categories that have been set at 37.64,%, followed by students and entrepreneurs respectively 19.25% and 17.24%. The rest of 11.49% are students, Private employees 8,05%, 3,16% and the smallest civil servants are ABRI 0.29%, (Fig. 5a).
- The biggest destination of the trip is the South Palu, Palu West, East Palu and Mantikolore that are respectively of 19.83%, 18.39%, 18.10% and 18.10%, the number of trips is leading to; Tawaeli, North Palu, Tatanga and Ulujadi which are respectively of 10.06%, 6.90%, 3.16% and 2.87%, and the smallest leading out of city ie; Sigi Biromaru direction, Donggala-Tolitoli Direction and Sigi Dolo direction which each number of trips 1.15%, 0.86%, 0.29% and 0.29%, (Fig. 5b).
- The dominance journey duration time on the morning at 6-7 or 29.99% (traveling to schools and to offices), followed by a trip time adjusted 16.09% generally travel to college, to work place and to other activities locations, before 6 o'clock, 8 to 9 o'clock and 7 to 8 o'clock that respectively of 13.79 %, 13.51% and 12.64%, and the smallest journey performed on 11- 12 hours until the night, except in the early evening hours are 16:00 to 5:00 p.m. of 2, 87%, (Fig. 6a). Original-Destination matrix while traveling are shown in Table 2 and Fig. 7.
- To reach waiting locaton of public transportation, the traveler can use a variety of ways; directly in front of the house waiting for 55.37%, escorted by motorcycles 36.25%, 5.46% walking, wear a motorcycle driven by 1.82%, 0.73% driven car and use tricycles 0.36%, (Fig. 6b).
- Based on the purpose of the trip, the biggest is the education places (school and college) amounted to 41.09%, 31.03% followed by trips to work places, shopping trips 22.41% and others trips 5.46%, (Fig. 6c).

Matrix Origin Destination (OD) based on a survey of home interviews with 1749 households, it was known that about 348 respondents of city transport users which their spread can be seen in Table 2, while Fig. 7. shows the original of Home City Transport User Based on the divided district zone.

Table 2 Matrix Origin Destination (OD) Users City Transport in Palu City yealded from Home Interview Survey

	Ulujadi	Palu Barat	Tatanga	Palu Selatan	Palu Timur	Mantikolore	Palu Utara	Tawaeli	Arah Donggala	Arah Sigi-Biromaru	Arah Paringi	Arah Sigi-Dolo	Arah Dgl-Toli toli	Jumlah
Ulujadi	9	10	1	1	2	2	0	1	0	0	0	0	0	26
Palu Barat	1	29	1	3	4	6	0	2	0	1	0	1	0	48
Tatanga	0	9	7	2	4	1	0	0	0	1	0	0	0	24
Palu Selatan	0	2	0	33	13	12	1	1	0	2	0	0	0	64
Palu Timur	0	6	1	19	27	11	2	1	1	0	0	0	0	68
Mantikolore	0	5	1	6	8	20	2	1	0	0	0	0	0	43
Palu Utara	0	2	0	5	4	9	17	10	0	0	0	0	0	47
Tawaeli	0	1	0	0	1	2	2	19	0	0	0	0	3	28
Jumlah	10	64	11	69	63	63	24	34	1	4	0	1	3	

Source: Home Interview Survey Processed results (2015)

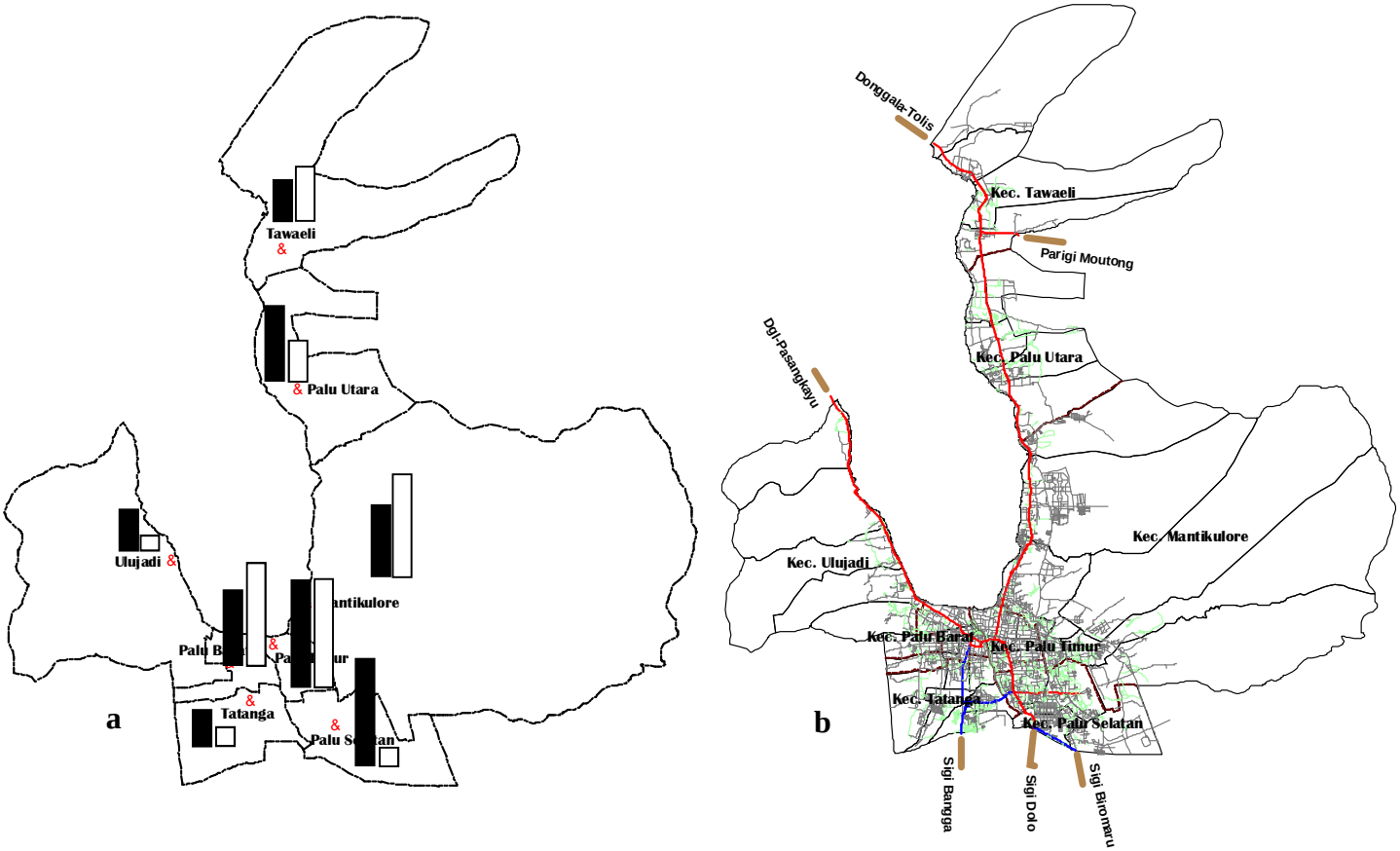


Fig. 7a and b. Originally User Objective Based on the Divided of City Transportation Zone District and Road Network Model in Palu

CLOSING STATEMENT

Conclusion

By knowing the characteristics and patterns of population movement in fulfilling of their demands, it can be concluded as follows:

- The system of urban transportation services in the city of Palu is still a door to door service (full deviation), which demonstrates the family characteristics as follows:
  - The level of family income
    The smaller the income level of a family more likely to use public transportation for travelling because they pay only once to the public transportation to reach their destination.
  - Total average transport costs realesed per month
    The smaller the monthly transportation cost used, it is more likely family members using public transportation in fulfilling of their journey.
  - Size of building homes
    The smaller the area of the house where they live the more likely family members using public transportation in fulfilling of their journey.

- The characteristics and patterns of population movement transport users in the city of Palu city:
  - Users dominated mostly by female.
  - Based on membership in the family structure, the largest travel shown by child, followed by mothers, family members in the household and the latter are fathers.
  - Based on the traveler's age, the largest is the age of 31- 40 years, followed by 23-30 years, 41-50 years, the next age; 15-18 years, 19-22 years and 51-60 years and the smallest is 12-15 years, > 61 years old and 6-11 years of age.
  - This type of work is dominated by travellers are other workers that do not include in this category, followed by students, entrepreneurs, students of university, private sector employees, civil servants and the smallest are armed forces.
  - The biggest purpose of the trip destination is South Palu, Palu West, East Palu and Mantikolore, the number of trips is leading to Tawaeli, North Palu, Tatanga and Ulujadi, and the smallest is leading out of town, namely; Sigi Biromaru direction, Donggala-Tolitoli Direction and Sigi Dolo direction.

- The dominance of the largest travel time is in the morning of at 6 to 7 (traveling schools and offices), followed by a customized travel time is generally the way to lectures and other activities that are not bound by time. And certain times until the afternoon hours are 4:00 p.m. to 5:00 p.m.
- To reach the waiting place of city transport, the traveler can use a variety of ways; waiting directly in front of the house, escorted by motorcycle, on foot, escorted wear motorcycle / car or use a rickshaw.
- Based on the purpose of the trip, the biggest is the education (school and college), followed by trips to work, shopping and travel and others.

5. 2 Suggestions

Based on the conclusion, a number of suggestion can be given as follows:

- City transport service system that still is door to door service (full deviation) are appropriately reviewed in order to provide a service of safe, comfortable and cheap against its inhabitants,
- Restructuring urban transportation route (route remains) should be done so that users get the assurance that waited, waited a long time, without any prior negotiation (straight up) and long travel time can be predicted,
- In addition to captive users (depending on public transport), the choice can attract users to switch to public transportation to continue disseminating and enforcing against illegal users of motor vehicles (without SIM) in schools such as junior and senior high scholl.
- With the implementation of regular city transportation, it can reduce the impact on urban transportation problems that have started to be felt in the town of Palu, such as; the emergence of congestion on certain road segments,
- By operating the city transport routes as expected, motorcycle operators who had been in certain road sections will switch to the actual operation and serves as a feeder transport. They will start and end the trip or serves as a feeder to public transportation.

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THE OPERATIONAL COST CHARACTERISTIC OF INTER CITY FREIGHT TRANSPORT IN SOUTH SULAWESI - INDONESIA

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ABSTRACT: Infrastructure development of freight transport system is vital in Master Plan for Acceleration and Expansion on Indonesia's Economic Development (Masterplan Percepatan dan Perluasan Pembangunan Ekonomi Indonesia/MP3EI). Based on this context, this study aims to investigate and analyze the operational cost of inter-city freight transport in South Sulawesi Province. The data survey using questionnaire interviews to the freight transport operators was carried out in order to collect information on characteristics of the freight transport including operator characteristic, vehicles characteristic, trip characteristic, and commodity characteristic. The survey was conducted on the freight transport operators travelling across Parepare - Makassar, one of freight transport main routes in South Sulawesi Province during one week. The survey data was analyzed using the descriptive statistic approach to understand the existing freight transport characteristic phenomenon. Furthermore, the operational cost analysis was conducted by developing relationship model between the cost and the distance traveled by freight transport using regression model approach. The analysis result shows that the operational cost of the freight transport is significantly determined by the operator salary cost, fuel cost, and vehicle maintenance cost. The freight transport operational cost model gives decent significance level in aspect of the transport modes used as well as the commodities transported. These results provide basis and expectation toward more advanced analysis on developing more comprehensive model of freight transport modes choice for freight transport trip in South Sulawesi Province in further studies.

Keywords: *regression model, operational cost, inter city freight transport, South Sulawesi*

INTRODUCTION

In order to implement the Master Plan for Acceleration and Expansion on Indonesia's Economic Development (MP3EI, 2011), construction and development of freight transport system hold vital position. Several studies of freight transport in Indonesia have been and still being conducted to understand the behavior and characteristic of infrastructure construction and development of the freight transport system including characteristic or behavior of the stakeholders in the freight transport system, freight transport model of needs, infrastructure budgeting concept of freight transport, et cetera. Most studies were conducted for freight transport behavior in Jawa, Sumatera, and Kalimantan. Among others were Ridwan (2012) for Jawa, Syahminan, dkk (2011a, 2011b, 2012) for Sumatera, and Mahmudah, dkk (2012) for Kalimantan.

Specifically, the research on freight transport behavior and characteristic in South Sulawesi Province is still rarely conducted and not yet profoundly explored. Previous research attempted to observe the characteristic of freight transport in South Sulawesi Province only focused on development of container port in South Sulawesi Province (Idrus dkk., 2012; Dewa dkk., 2012).

In order to explore the behavior and characteristic of the freight transport in South Sulawesi Province, the writer have attempted to depict the characteristic of freight transport operating in region Mamminasata Metropolitan Area, South Sulawesi Province (Hakzah, dkk., 2013a and 2013c) and also the characteristic of freight transport trip across Makassar - Parepare in northern of South Sulawesi Province (Hakzah, dkk., 2013). Moreover, the load value and cost characteristic of the freight transport based on the transport modes and commodities transported have also been successfully described (Hakzah, dkk., 2014).

As a follow-up of these research results, and contibutes to development of freight transport in South Sulawesi Province, this study aims to describe operational cost characteristic of freight transport, and models the relationship between freight transport operational cost against the distance traveled for the category transport modes and comodities transported in freight tranport trip across Makassar - Parepare as one of main axis in freight transport network in South Sulawesi Province.

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THE STUDY METHOD

This study used data collected from the interviews survey on characteristic of freight transport operating in Makassar-Parepare, as main axis in freight transport network in South Sulawesi. This survey results data was partially elaborated and analysed gradually as in Hakzah, dkk. (2013b; 2014). This freight transport survey covered interviews survey on many aspects of the existing freight transport system, carried out for 24 hours for 5 days started from 06.00am to 06.00am the next day. The survey used form that have been designed and tested before. The interview method on the vehicle operator was conducted by stopping the vehicle to roadside then interviewed the driver or the operator based on the question items in the questionnaire. As for the question items in the questionnaire, they covered the trip origin-destination, departure time, estimated cost and travel time, types and quantities of the commodities transported, and the freight transport vehicles types along with the the loads values, et cetera.

The data collected then described and analysed statistically against the phenomenon of the freight transport operational cost in aspect of the transport modes as well as the commodities transported. Furthermore, the study also developed model of freight transport operational cost against the distance traveled using the simple linear regression model approach.

THE RESULTS AND DISCUSSION

Characteristics of the Freight Transport Operational Cost

Based on the results of the freight transport survey carried out in Makassar - Parepare corridor and performed data compilation, then obtained representation of the freight transport operational cost characteristic is shown in Figure 1 from the viewpoint of transport modes used, and Figure 2 shows from the viewpoint of the commodities transported.

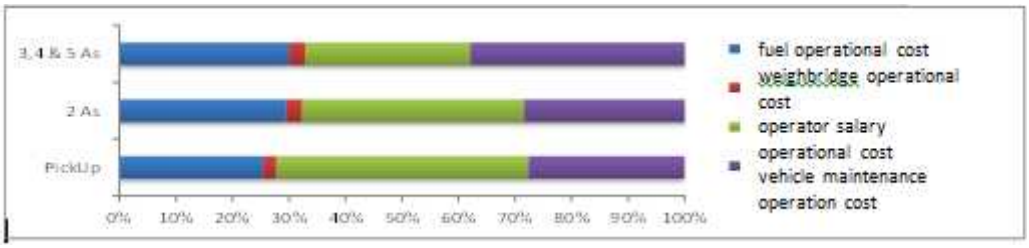


Figure 1. Model and degree of freedom with bar ends in rigid tip

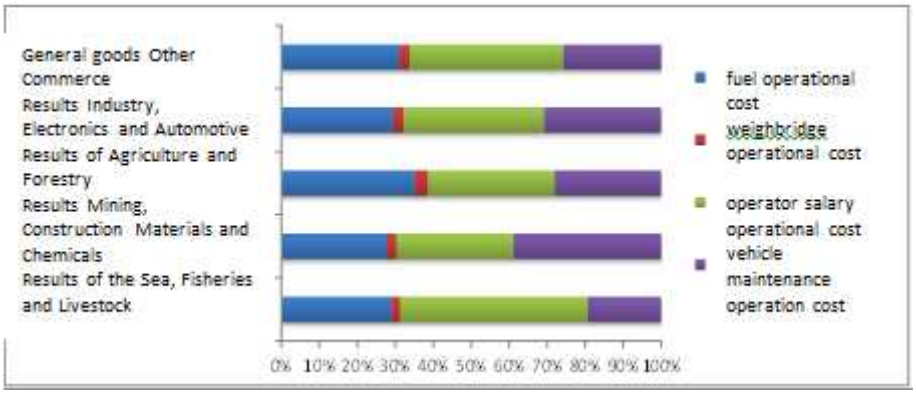


Figure 2. Model and degree of freedom with bar ends in rigid tip

Figure 1 and Figure 2 show that there are 3 (three) types of freight transport operational cost (namely: fuel operational cost, operator salary operational cost, and vehicle maintenance operation cost) which dominate the amount of transport operational cost of freight transport in South Sulawesi. As for additional cost, although not in big proportion, is in the form of payments in weighbridge. Based on types of transport modes used, Figure 1 shows that the operator salary cost has bigger portion than the fuel cost and maintenance cost. Similar thing happens for the type of 2-axles truck where the fuel cost and maintenance cost is relatively equal. However, for the 3-4-5-axles truck, the vehicle maintenance cost is more dominant than the fuel cost and operator salary cost where both are relatively have equal portions..

In Figure 2, it is shown that between those three dominant cost are alternately having high proportion from commodities transported viewpoint. For the marine products, fishery and livestock, industrial products, electronics and automotives, and others commerce goods, the operator salary cost is more dominant than other cost. However, for the commodities of agriculture and forestry, the fuel cost is more dominant. In case of mining, construction materials, and chemicals, the maintenance cost is more dominant than the other cost.

The Operational Cost Model of he Freight Transport Modes

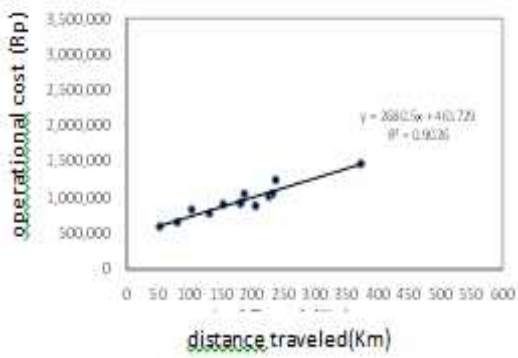
The modeling results of relationship between freight transport operational cost against the distance traveled using simple linear regression model based on the types of modes used is shown in Figure 3, and the parameter values of the regresion model are presented in Table 1.

Table 1. Parameter values of the regression model based on types of modes

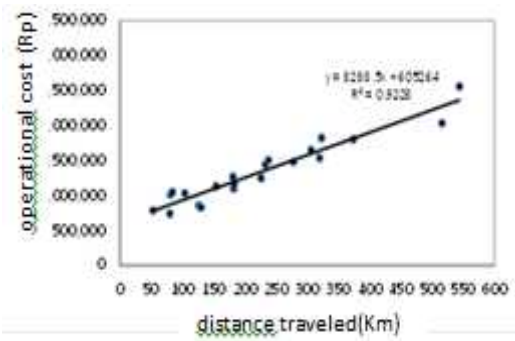
Model Category	Parameter Value		Determination Coefficient (R^2)
	M	N	
Mode Pickup	461.729	2.680,5	0,9026
Mode 2 Axles Truck	605.264	3.233,5	0,9223
Mode 3-4-5 Axles Truck	450.451	4.708,0	0,8811

Table 1 and Figure 3 show that the modeling results of relationship between freight transport operational cost and distance traveled for the types of transport modes using simple linear regression as presented in Table 1 have decent significance level where all models have value of determination coefficient (R2) near to 1 ranging from 0,8 - 0,9.

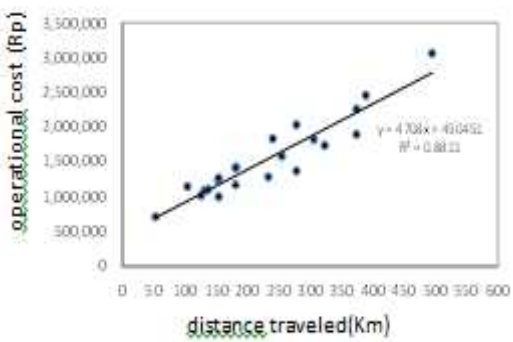
Furthermore, Table 1 and Figure 3 show that the model line of operational cost for 3-4-5 axles truck mode has higher slope than the model of two other modes. This is indicated by the higher parameter value of $_N$ for 3-4-5 axles truck mode than 2 axles truck mode and pickup mode. As for the two modes mentioned latest, the model line for 2 axles truck mode has higher slope than the model line of pickup mode. These indicate that the operational cost of freight transport using 3-4-5 axles truck mode is higher than the two other modes. Similary, the operational cost for 2 axles truck has higher operational cost than pickup mode. In other words, the more the machine and the container capacity of the mode used, the more the operational cost in operating the mode.



a. Pickup



b. 2 axles truck



c. 3-4-5 axles truck

Figure 3. Model and degree of freedom with bar ends in rigid tip

Operational Cost Model Against Distance Traveled Based on the Freight Transport Commodities

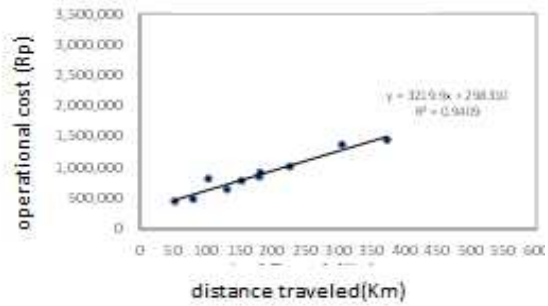
The modeling results of freight transport operational cost against distance traveled based on the commodities transported is shown in Figure 4 where the parameter values of the regression model is presented in Table 2.

Table 2. Parameter values of regression model based on type of commodities

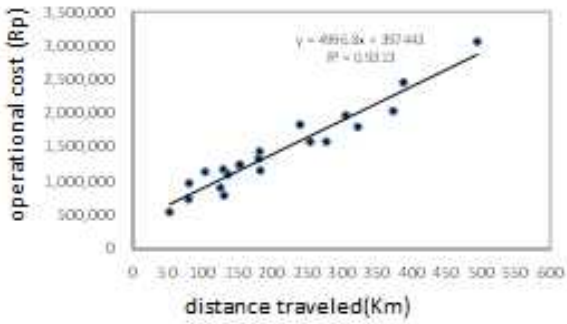
Model Category by Commodities	Parameter Value		Determination Coefficient (R^2)
	M	N	
Marine, fishery & livestock	298.310	3.219,9	0,9409
Mining, Construction & Chemical	397.443	4.996,8	0,9333
Agriculture and forestry	243.164	3.421,5	0,9095
Electronic, automotive & industrial	657.557	2.949,7	0,9349
Other commerce goods	239.014	4.329,5	0,9419

Table 2 and Figure 4 show that the modeling results of freight transport operational cost against the distance traveled for the transported commodities have the determination coefficient values (R^2) near to 1 indicating that the models have decent significance level by using simple linear regression model as presented in Table 2.

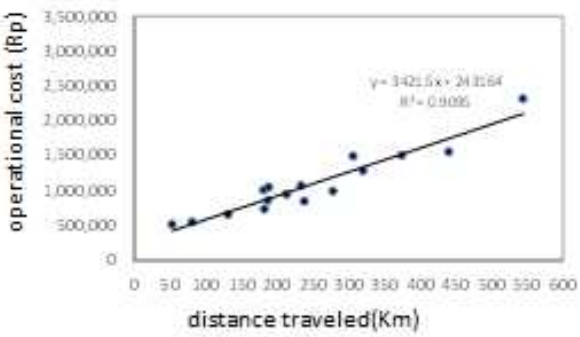
Furthermore, Table 2 and Figure 4 show that model of freight transport operational cost for mining, construction, and chemical has higher slope of line than the 4 other commodities. This is shown by higher value of $_N$ parameter of the model for mining, construction, and chemicals than value of $_N$ parameter of the other commodities. Consecutively followed by model line of commerce goods, agriculture and forestry, marine products, fishery and livestock, and electronics, automotive, and industrial products. The diversity of the line slope of the operational cost model indicates that the mining, construction materials and chemicals has the highest operational cost of all transported commodities, then followed by other commodities according to their height of the line slope of the model.



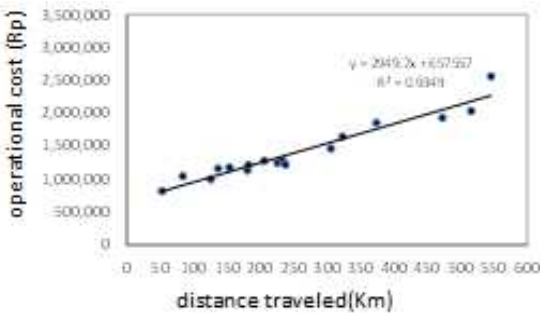
a. Marine products, fishery and livestock



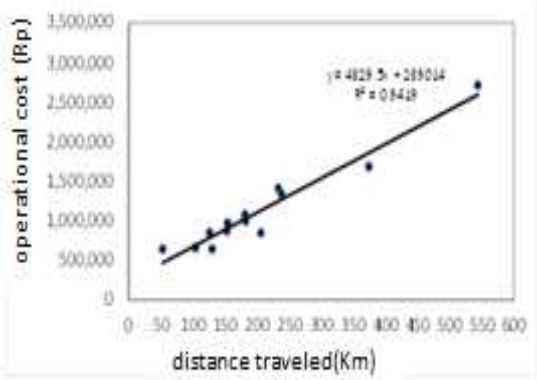
b. Mining, construction and chemical



c. Agriculture and forestry



d. Electronic, automotive and industrial products



e. Other commerce goods

Figure 4. Model and degree of freedom with bar ends in rigid tip

The the modeling of freight transport operational cost against distance traveled by its transport modes as well as by commodities transported as presented in Table 1, Table 2, Figure 3, and Figure 4 gives phenomenon that the operational cost of the freight transport are vastly varies depend in the transport modes used and commodities transported. This indicates a behavior exists that choice of transport modes is sensitive to the commodities transported which becomes consideration for the freight owner or even the sender in operating freight transport in South Sulawesi Province. Indication that this behavior in choosing the freight transport mode is existed needs to be elaborated further including to explore other factors influencing mode choice of the freight transport stakeholders in South Sulawesi.

CONCLUSION

This study has explored one of main characteristics of freight transport namely freight transport operational cost which travels between cities in South Sulawesi Province. By carrying out survey through interviews using questionnaires with the freight transport operators traveling across Parepare - Makassar, one of freight transport main routes in South Sulawesi Province for one week, the characteristics of freight transport including operator characteristic, vehicle characteristic, trip characteristic, and commodity characteristic has been well described. Furthermore, the operational cost of the freight transport against distance traveled by freight transport modeling has been conducted using regression model approach resulting indication of decent regression model significance.

Operator salary cost, fuel cost, and vehicle maintenance cost are 3 (three) components of freight transport operational cost which comprehensively dominant of total operational cost of freight transport in South Sulawesi Province. The amount of freight transport operational cost is quite sensitive to transport modes used and commodities transported. The results of modeling indicates that the more the capacity of transport mode, the more the operational cost of the freight transport, in which the 3-4-5 axles truck transport mode has the highest operational cost. By commodities, the highest freight transport operational cost is the mining, construction materials, and chemicals.

The modeling results of freight transport operational cost obtained in this study becomes one of basis in developing a model of freight transport mode choice in South Sulawesi Province in further studies.

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AGING SIMULATION OF HOT MIX ASPHALT MIXTURES UTILIZING RECLAIMED ASPHALT PAVEMENT

I. Rismawansyah [1], M. Ihsan [2]

ABSTRACT: The utilization of reclaimed asphalt pavement (RAP) has become more popular due to increasing concerns of sustainability, environmentally and economically. The research was to investigate the effect of simulated aging to the characteristics of two types of pavement layer, Asphalt concrete-Wearing Course (AC-WC) comprising considerable portion of reclaimed asphalt pavement or bituminous waste materials. Aging simulation includes Short Term Oven Aging (STOA) and Long Term Oven Aging (LTOA) method. The research included determination of the appropriate composition of RAP and new materials, determination of Optimum Asphalt Content (OAC) and production of specimens for the two type of asphalt pavement. Three types of specimen were control condition (non aged), STOA and LTOA specimens. All of specimens are tested to determine the Marshall Properties, effect of immersion and Indirect Tensile Strength (ITS) test. Indirect Tensile Strength, Retained Marshall Stability and Tensile Strength Ratio, have shown decreasing value while the Marshall Stability and Flow have shown incremental value.

Keywords: *Asphalt Recycling Pavement, Aging, Marshall Properties, Durability, Indirect Tensile Strength.*

INTRODUCTION

Background

Nowadays, the conservations of materials, environmental issue, and costs effective are three major issues in pavement construction, rehabilitation and maintenance. Especially for flexible pavement utilizing asphalt cement that comes from natural resources. Asphalt pavement recycling is one of procedure that eminently and realistically meets these concerns because of the economical in terms of energy and materials usage.

In asphalt pavement recycling, materials reclaimed from old pavement structures are reprocessed along with some new materials to produce asphalt mixtures meeting all normal specification requirements (Asphalt Institute, 1983). The asphalt contained in old pavements is an additional valuable resource. Because of factors such as oxidation, the aged asphalt may have lost some of its original properties, but when combined with new asphalt it can again serve as an effective binder. The reuse of aged asphalt may reduce the amount of new asphalt required for pavement reconstruction.

Asphalt pavement materials consists of aggregate and asphalt cement. Asphalt cements are composed of organic molecules that react with oxygen from environment. This reaction is called oxidation and it changes the structure and composition of asphalt molecules. Oxidation causes asphalt cement to harden. Because asphalt cement exists in thin films covering the aggregate, the oxidation reaction occurs at a faster rate. “Short term aging” is used to describe the aging that occurs in this stage of the pavement life. Oxidative hardening also occurs because of exposure to air and water. “Long term aging” happens at a relative slow rate in a pavement, although it occurs faster in warmer climates and during warmer seasons. Because of this hardening, old asphalt pavements are more susceptible to cracking. Improperly compacted asphalt pavements may exhibit premature oxidative hardening because of a higher percentage of interconnected air voids. This allows more air to penetrate into the asphalt mixture, increasing oxidative hardening (Asphalt Institute, 2001).

Research Objectives

1. To perform AC Wearing Course (AC-WC) hot recycling asphalt utilizing bituminous waste materials (RAP) and new materials (aggregates and asphalt cements) and to conduct performance tests by determine the Marshall characteristics, failure plane by Indirect Tensile Strength (ITS) test and durability of the mixture.
2. To determine the effects of aging to the hot recycling asphalt by using short and long term aging procedures which were developed at Oregon State University under the Strategic Highway Research Program (SHRP) A-003A project.

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LITERATURE REVIEW Asphalt

Recycling Pavement

Asphalt pavement recycling is not a new development. Used to called reconstruction, it has been in use for more than 5 decades. In asphalt pavement recycling, materials reclaimed from old pavement are reprocessed with addition some new materials to produce new asphalt mixtures meeting all normal specification requirements. The asphalt contained in old pavements is an additional valuable resource. Because of factors such as oxidation, the aged asphalt may have lost some of its original properties, but when combined with new asphalt it can again serve as an effective binder. The reuse of aged asphalt may reduce the amount of new asphalt required for pavement reconstruction. Recycling can correct many types of pavement distress. By reusing aggregate and asphalt from deteriorated pavements, the need for new materials is appreciably reduced and the overall cost of the improved pavement will be less. The addition of asphalt to existing granular base materials during recycling increases structural strength without adding to pavement thickness, thereby preserving roadway geometrics. Since the pavement thickness is not increased, drainage problems are avoided. Adding asphalt also helps waterproof the base and renders it less susceptible to moisture damage. (Asphalt Institute, 1983)

Asphalt Aging Behaviour

As stated in Asphalt Institute (2001) asphalt cements are composed of organic molecules, because of that they react with oxygen from the environment. This reaction is called oxidation and it changes the structure and composition of asphalt molecules. Oxidation causes asphalt cement to harden. In practice, some oxidative hardening occurs before the asphalt is placed. Because asphalt cement exists in thin films covering the aggregate, the oxidation reaction occurs at a faster rate. “Short term aging” is used to describe the aging that occurs in this stage of the asphalt’s life. Oxidative hardening also occurs during the life of the pavement because of exposure to air and water. “Long term aging” happens at a relative slow rate in a pavement, although it occurs faster in warmer climates and during warmer seasons. Because of this hardening, old asphalt pavements are more susceptible to cracking. (Whiteoak et. al., 2003)

According to Kliewer et. al. (1995), the development and validation of laboratory aging procedures to simulate short and long term aging for asphalt-aggregate mixtures was undertaken as part of the Strategic Highway Research Program (SHRP) project A-003A at Oregon State University (OSU). The laboratory procedure for short term aging is to heat loose mix in a forced draft oven for 4 hours at a temperature of 135°C. This short term oven aging (STOA) simulates the aging of the mixture during the construction process. The laboratory procedure for long term aging of the compacted mixture is to heat the compacted specimens in a forced draft oven at 85°C. This long term oven aging (LTOA) simulates the aging of the mixture in service. An aging period of 4 or 5 days is recommended to simulate project about 10 years old.

Asphalt Mixture Testing

Asphalt mixture testing conducted to determine the quality of AC Wearing Course must meet the requrements as stated on Indonesian Directorate General of Highway (2007). There are two types of mixture properties that will obtained in this research are volumetric and Marshall properties. According to Asphalt Institute (2001), the test are :

1. Volumetric Properties
   a. Bulk specific gravity of aggregate
   b. Effective specific gravity of aggregate
   c. Maximum specific gravities of mixtures
   d. Asphalt absoption
   e. Effective asphalt content
   f. Percent VMA in compacted paving mixture
   g. Calculation of air voids in compacted mixture
   h. Percent VFA in compacted paving mixture
2. Marshall Properties
   a. *Stability*, the Marshall Stability value is the maximum load before the specimen yields or fails. Marshall stability test provides the performance prediction measurement for the Marshall Mix design method. The primary use of Marshall stability is to evaluate the change in stability with increasing asphalt content to aid in selecting of the optimum asphalt content.
   b. *Flow*, Marshall Flow, measured in units equal to 0.25 mm, represents the total deformation of the specimen. The deformation is a measure of a decrease in the vertical diameter of the specimen.
   c. *Marshall Quotient* (MQ), is Marshall Stability divide by Flow. Mixes that have very low flow values and abnormally high Marshall stability values are considered too brittle and rigid for pavement service. Those with high flow values are considered too plastic and have tendency to distort easily under traffic loads.

Indirect Tensile Strength (ITS)

According to AASHTO T 283, HMA made from certain materials are sensitive to the presence of water in the pavement. Water will cause the binder to not adhere to the aggregate. Since the binder is the "glue" that holds the

pavement together, rapid failure of the pavement can be expected if the binder cannot adhere to the aggregate. This is often referred to as stripping. To help prevent stripping, additives such as hydrated lime or liquid anti-stripping chemicals may be required. Indirect Tensile Strength (ITS) test is a test method that can be used to determine if the materials may be subject to stripping and also to measure the effectiveness of additives. The specimens are tested for indirect tensile strength by loading the specimens at a constant rate and measuring the force required to break the specimens. The tensile strength of the conditioned specimens is compared to the control specimens to determine the tensile strength ratio (TSR).

METHODOLOGY

Research procedure consists of preparation, where Reclaimed Asphalt Pavement (RAP) materials were dried before used as material for specimens. Three types of materials that used in this study they are RAP, new aggregate and asphalt concrete have to be tested before used as composition of mixtures. Tests include extraction test for RAP, Los Angeles test, specific gravity test, sieve analysis, absorption to water test, sand equivalent test for new aggregate. Tests for bituminous material include Penetration test, flash point test, softening point test, loss on heating test, ductility test, penetration test after loss on heating and specific gravity. There are four types of specimens design in this research:

a. Specimen design for determining the composition of RAP and new aggregate without adding asphalt concrete. The variation of the specimens are : 100% RAP, 80% RAP : 20% new aggregate and 75% RAP : 25% new aggregate.
b. Specimen design for determining asphalt content design. The variation of added asphalt content is by 0.3% from 0.6% until 1.5% asphalt content addition to the chosen composition from 1.
c. Specimen design with asphalt content design as control specimens that use for testing on Marshall properties, ITS test.
d. Specimen design with condition Short Term Oven Aging (STOA) and Long Term Oven Aging (LTOA) for testing on Marshall properties, immersion, and ITS test.

Test specimens preparation were :

a. Number of specimens – At least three specimens were prepared for each combination of RAP, aggregates and asphalt content.
b. Preparation of RAP and aggregates – RAP and aggregates dried to constant weight at 105°C to 110°C.
c. Mixing and compaction temperature determination.
d. Mixture preparation – Each test specimen is weighed into separate pans for of each size fraction and percentage of RAP required to produce a batch that will result in a compacted specimen 63.5±1.27 mm in height. This will normally be about 1.2 kg.
e. Compaction of specimens – 75 blows are applied to compact the specimens to each side using compation hammer. After compacting, remove the specimens from mold and cool overnight until ready for testing.
f. For STOA specimens, the procedure is similar but after the specimens mixed, it must placed on oven for 4 hours at temperature of 135°C before compaction.
g. For LTOA specimens the procedures is similar with STOA procedures but after placed on oven for 4 hours at temperature of 135°C and compaction, the specimens must placed again in oven for 5 days at temperature of 85°C before ready for testing.

Specimens testing follows the procedures :

a. Marshall Properties Testing
1) Bulk specific gravity determination – This test is performed according to ASTM D2726.
2) Stability and flow tests – After the bulk specific gravity of the test specimens have been determined, the stability and flow tests are performed.
b. Indirect Tensile Strength testing
1) Height, dry height, submerged weight and surface dry weight of the specimens are measured and the air voids is determined.
2) Each specimen placed in water bath at 25° C and vacuum saturate for 45 minutes, then the wet conditioned specimens placed in a 60°C water bath for 48 hours.
3) The specimen is removed from the bath and then placed on its side between the bearing plates of the testing machine. A load is applied to the specimen by forcing the bearing plates together at a constant rate of 2 in. (50 mm) per minute.
4) The maximum load is recorded, and the load continued until the specimen cracks.

RESULTS AND DISCUSSION

Laboratory Results

1. Material Testing Result

Asphalt mixture consists of fresh aggregate and RAP. For new aggregate and asphalt cement, physical testing is conducted and the result is shown in Table 1.

Table 1. Material testing result

No	Test Type	Specification	Result	Unit
	Aggregate			
1.	Abrassion	Max. 40	24,4	%
2.	Bitumen adhesivity	> 95%	98	%
3.	Bulk Specific Gravity Coarse Aggregate	>2,50	2,729	gr/cc
	Bulk Specific Gravity Fine Aggregate	>2,50	2,868	gr/cc
4.	Coarse Aggregate absorption	<3	2,480	%
	Fine Aggregate absorption	<3	2,564	%
5.	Sand Equivalent	Min. 50	85	%
6.	Soundness Test	Max. 7%	1,57	
	Asphalt AC 60/70			
1.	Penetration at 25º C	60 - 79	62,2	0,1 mm
2.	Softening Point	48 – 58	48	º C
3.	Flashing Point	>200	347	º C
4.	25º C Ductility	>100	>100	cm
5.	Loss on heating (163ºC, 5 hours)	<0,4	0,050	%
6.	CCl_4 solutibility	99	99,317	%
7.	Penetration after loss on heating	75	82,0	% of original
8.	Specific Grafity	1	1,030	gr/cc

The result of material testing for aggregate and asphalt cement were passed all the specifications in AASHTO 1982. The existing asphalt content in the mixture of RAP is 5.9% and in the mixture with addition of new asphalt cement is 4.4%, it is decreasing from original asphalt content which was 6 to 7 percent because the effect of oxidation.

2. Composition determination of the mixture

The mixture that tested for the effect of aging are consists of RAP and new aggregate with the addition of asphalt cement. To determine the best proportion of RAP and new aggregate, this research has trial 3 different proportion (100% RAP, 80% RAP : 20% new aggregate, and 75% RAP : 25% new aggregate) without any asphalt cement addition to determine the Marshall Properties. The result of the test is shown in the Table 2.

Table 2. Composition determination of the mixture

Marshall Properties	Specification	Proportion (RAP : New Aggregate)		
		100 : 0	80 : 20	75 : 25
Density (gr/cc)	-	2.199	2.178	2.184
VMA (%)	min. 15	16.29	16.14	15.94
VFB (%)	min. 65	67.46	52.44	51.04
VITM (%)	3.5 - 5.5	5.30	7.68	7.81
Stability (kg)	min. 800	2687	2668	2630
Flow (mm)	min. 3	2.17	2.33	3.03
MQ (kg/mm)	min. 250	1239	1147	907

From Table 2., for 100% RAP composition is passing all the specification except flow and gradation. The mixture is brittle because aging process makes the flow become low and the gradation not meet the requirement for AC-WC gradation because during the service life, the asphalt pavement experiencing degradation by traffic loading and milling process. For 80% RAP :

20% fresh aggregate composition, there are some items that not passed the specification, those are flow, VITM and VFB. But for 75% RAP : 25% fresh aggregate composition, although there are some items that not passed the specification (VITM and VFB), considering the gradation of this mixture is the gradation of AC-WC mixture and the flow are meet the requirements comparing with others composition, hence the choosen composition for this reasearch is 75% RAP : 25% fresh aggregate. With the prediction that is if some percentage of new asphalt is added to this composition, will make the mixture pass all the specification especially VITM and VFB.

3. Determination of Optimum Asphalt Content (OAC)

The OAC of recycled AC-WC mixture was obtained by preparing chosen specimens with five variation of asphalt content addition with the interval of 0.3% and testing those specimens with Marshall Testing method. The analysis to determine the OAC is shown in Table 3.

Table 3. Optimum Asphalt Content determination

Marshall Properties	% Bitumen (Addition to 4.4% existing)				
	0.6%	0.9%	1.2%	1.5%	1.8%
Density (gr/cc)	2.200	2.216	2.219	2.218	2.233
VMA (%)	15.82	15.68	15.69	15.94	16.36
VFB (%)	59.68	64.77	69.19	71.85	77.97
VITM (%)	6.40	5.52	4.85	4.49	3.45
Stability (kg)	1748	2027	1789	1876	2059
Flow (mm)	3.17	3.70	3.47	3.77	4.13
MQ (kg/mm)	560	548	517	498	499

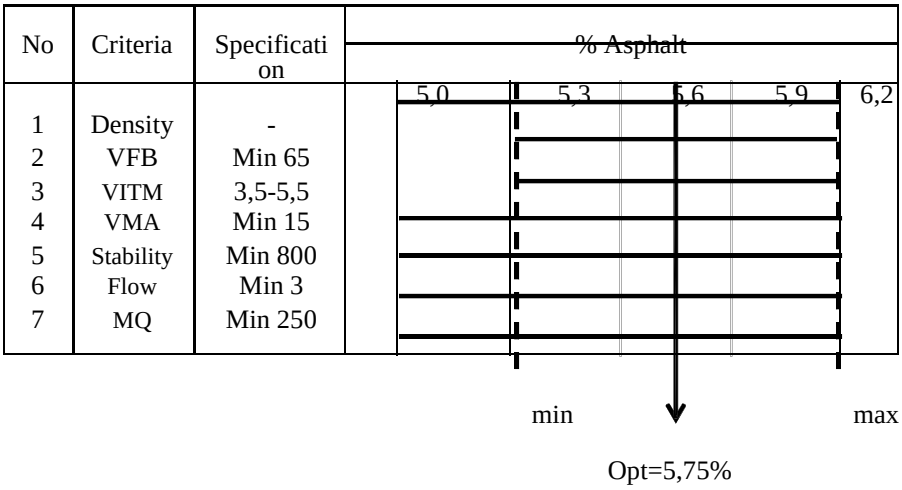

Fig. 1. Optimum Asphalt Conten Determination

From test result, the OAC of the mixture is 5.75% from the total of mixture. Therefore, to determine the effect of aging to the recycled asphalt pavement, 5.75% total asphalt content or 1.35% of asphalt addition to 75% RAP : 25% fresh aggregate mixture is used.

4. Specimens Testing Result In Optimum Asphalt Content Condition

To determine the effect of aging to the recycled AC-WC hot mixture, three specimens that are Control, STOA, and LTOA specimens are tested. The control specimen is the specimen that made by standard prochedure in Optimum Asphalt Content, the STOA and LTOA specimens is made by prochedure that adopted from Strategic Highway Research Program (SHRP) A-003A project. There are two major testing prochedure that conducted to all specimens in this research :

a. Marshall testing

There are two major result in Marshall testing, that are volumetric properties (density, Voids in Mineral Aggregate, Voids Filled with Bitumen, Voids In The Mix) and Marshall Properties (Stability, Flow, Marshall Quotient). The Marshall testing result for all specimens are shown in the Table 4. As control parameter, Indonesian Directorate General of Highway Specification year 2007 is used to determine the condition of the mixtures whether it confirms the requirements.

Table 4. Marshall Testing result for Control, STOA, and LTOA condition

Marshall Properties	Specification	Specimens		
		Control	STOA	LTOA
Density (gr/cc)		2.217	2.223	2.181
VMA (%)	min. 15	15.48	15.26	16.84
VFB (%)	min. 65	69.47	70.68	64.77
VITM (%)	3.5 - 5.5	4.73	4.48	6.26

Stability (kg)	min. 800	1930	1943	1977
Flow (mm)	min. 3	3.17	3.50	3.60
MQ (kg/mm)	min. 250	617	555	549

b. Indirect Tensile Strength (ITS)

ITS test is used to determine the potential crack and indirect tensile strengths of asphalt mixtures at low and intermediate pavement temperatures. There are two types of specimens in this research those are unconditioned and conditioned. For unconditioned, the specimens are tested in standard condition and for the conditioned, the specimens ares tested after placed in a 60°C distilled water bath for 48 hours. The result of the test is shown in Table 5. The major item that measured in this test is the maximum loading number (P max) for each specimen to determine the ITS value and Tensile Strength Ratio. Asphalt Institute SP-2 (1996) is used as minimum requirements for this test.

Table 5 ITS test result for Control, STOA, and LTOA condition

Specimens		P max (N)	ITS (kPa)	TSR (%)
Control	Unconditioned	56.33	5161	79.25
	Conditioned	44.67	4090	
STOA	Unconditioned	53.00	4723	76.38
	Conditioned	40.67	3608	
LTOA	Unconditioned	50.33	4517	66.04
	Conditioned	33.67	2983	

DISCUSSION

Effect of aging to Indirect Tensile Strength (ITS)

Indirect Tensile Strength (ITS) test is used to indicate crack potential of the mixture and evaluates susceptibility from effect of water. Figure 2 and Figure 3 shows the effect of aging to Indirect Tense Strength (ITS) and Tensile Strength Ratio (TSR) of the mixtures.

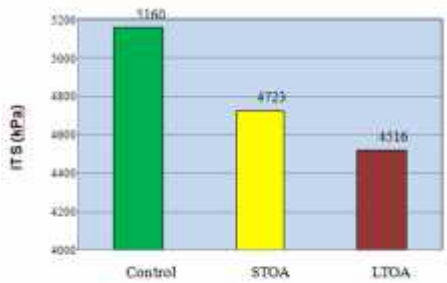

Figure 2. Effect of aging to ITS

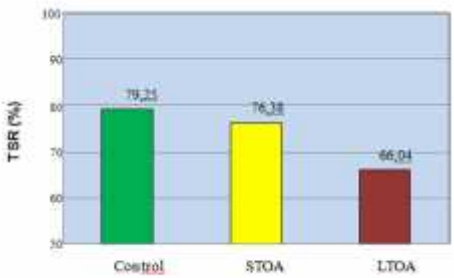

Figure 3. Effect of aging to TSR

The result of testing are ITS for Control specimen is 5160 kPa, for STOA is 4723 kPa and for LTOA specimen is 4516 kPa. It is indicates that effect of aging process will make the value of ITS decreasing because the mixture will be more brittle by the effect of aging or oxidation process.

Tensile Strength Ratio (TSR) is strength decreasing rate of the mixture that can effect the resistance of moisture damage and crack potential. TSR is obtained by dividing average tensile strength of the dry subset with average tensile strength of the conditioned subset. The acceptable resistance to moisture damage (TSR) based on Asphalt Institute SP-2 (1996) is 80%. From Figure 3., the TSR for Control specimen is 79.25%, for STOA specimen is 76.38% and for LTOA specimen is 66.04%. The results show that all the specimens cannot fullfill the requirement. Can be concluded that the effect of aging to the recycled AC-WC hotmix asphalt mixture make the resistance of moisture damage decreasing. Aging process in the mixture caused by asphalt cement oxidation will accelerate the asphalt cement hardening. Thus, the ability of asphalt cement as bonding agent weaker. As the result, the resistance of moisture damages become lower and crack potential will be higher.

CONCLUSIONS AND RECOMMENDATIONS

Conclusions

1. AC-WC asphalt hot mix recycled can be used as material for new AC-WC asphalt hot mix with addition of new aggregates and asphalt cements. Composition that selected for this research is 75% RAP : 25% new aggregates with Optimum Asphalt Content is 5.75% of total mixture. The properties of mixture in OAC condition are : density

2.217 gr/cc, VMA 15.48%, 69.47%, VITM 4.73%, stability 1930 kg, flow 3.17 mm, and MQ 617 kg/mm which all of those properties can fulfill the specification.

2. Effect of aging process to AC-WC asphalt hot mix recycled using short and long term aging procedures is The Indirect Tense Strength (ITS) for Control, STOA, and LTOA specimens are 5160 kPa, 4723 kPa and 4516 kPa respectively. The effect of aging process will make the value of ITS decreasing because the mixture will be more brittle by oxidation process. The Tensile Strength Ratio (TSR) for Control, STOA and LTOA specimens are 79.25%, 76.38% and 66.04% respectively. Can be concluded that the effect of aging to the recycled AC-WC hotmix asphalt mixture made the resistance of moisture damage become lower and crack potential will be higher.

Recommendations

Based on the result of the research, there are some recommendation for further research to make the better analysis in effect of aging to hot mix asphalt recycling :

1. The mixture of hot mix asphalt recycling using 75% RAP and 25% fresh aggregates with asphalt cement addition of 1.35% has no problem with the Tensile Stress Ratio (TSR) value that is very low. There are several methods to correct the poor TSR, those are : using different properties of asphalt cement, liquid anti-stripping additives, hydrated lime and change of aggregate or mix design.
2. Effect of aging process using short and long term aging procedures which were developed at Oregon State University under the Strategic Highway Research Program (SHRP) A-003A project must be validate with field experiment to determine exactly the effect of aging to the mixture by actual condition. It will become problem because to conduct the research needs long time and high budget.
3. To have better analysis in effect of aging to the hot mix asphalt recycling, it is recommended that the mixture have a simulation for loading test, for example Wheel Tracking Test because we will have not only effect of oxidation or weather to the mixture, but also the loading from vehicles to the pavement.

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DELAY CLAIM ANALYSIS ON HOSPITAL PROJECTS

Peter F.Kaming[1], Aditya P. Poernomo[2]

Abstract: The success of a construction project can be achieved when the works completed at the planned time, according to the allocated costs and meet the required quality. Delay of a project is one of the barriers that often happen. Delays causes construction claims, therefore delay analysisis needed and has an important rolein the decision-making process. The purpose of this study were: 1) to analyzethe relationship amongst causes of delay occurred in the hospitals projects; 2) to identify those activities that were delay according to consultants and contractors; and 3) to analyzethe frequency and type of claims in hospital construction projects. This study obtained a sample of 40 respondents consisting of consultants and contractors involved in the construction of the hospitals. Variables causing delays were obtained by using factor analysis is then performed multiple linear regression. Results of study showed that there were three significant factors that caused delays for hospital project, namely a) information factor, b) material factors, and c) equipment factors that cumulatively can explain 33.7% of delay. The study adopted independent sample t-test to analyze the dominant delay of the activities. Results of study showed that most often delay was mechanical electrical work, and there are 4 activities that has significant cause delay differentlyaccording to consultants and contractors namely: a)work of ceiling and partition, b) painting, c) power & lighting panels and d) the water pump & water heater. Analysis of the frequency and types of claims using the chi-square test obtained that the most common case wasan extra time claimat no additional cost.

Keywords: *Claim, projectdelays, hospital,*

INTRODUCTION

Background

The success of a construction project can be achieved when the work is completed at the planned time, according to the allocated costs and meet the required quality , in the achievement of business one of the barriers that often occured is delay project completion time . Building the hospital is building health facilities that have the function of planning complex and thus require different with building construction projects in general. In the world of construction delay claims management is a very important part to prepare for the possibility of a claim during the construction so as to avoid loss of material and productive time. Delays in construction claims analysis is one of the keys to success for today's construction projects which have an important role in the decision-making process.

Research Objectives

This research aims to:

1. Identify the factors causing delays in construction claims hospital
2. Identify activities that late in the construction of hospitals according to consultants and contractors
3. Perform factors analysis of the causing delays relationship with delays in the construction of hospitals
4. Perform analysis of the frequency and types of claims in hospital construction projects

LITERATURE REVIEW

Delay in Construction Projects

Understanding the delay according Ervianto (1998) is as execution time is not used in accordance with the plan of action, causing one or several activities to follow to be delayed or not completed on schedule as planned. Delay (delay) is when an activity or activities of construction projects have additional time, or not held in accordance with the expected plan, Callahan (in Suyatno, 2010).

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Claim Construction

The claim is generally defined as a request or petition, claim construction is a claim arising out of or in connection with the execution of a construction project between service users and service providers or between major service providers with sub-providers or suppliers of materials or between outsiders and users / providers who usually about additional demand of time, expense or other compensation (Yasin, 2008).

Factors Causing Delays in Construction Projects

In construction claims, delays can be described as a time for some parts of the construction project has been extended or not carried out due to unforeseen circumstances (see Andi, et.al., 2010) factors that may cause delays in construction labor among other factors, factors work equipment, information factors, material factors, factors kerakteristik location, managerial factors, and other factors

Type Delay in Construction Projects

According to the AIA (see Arditi and Patel, 1989) the delay that occurs during execution of construction work can be classified as follows:

1. Delays are excused with compensation (Excusable-compensable delay).This delay was not caused by the contractor but by the service user, the reasons for the delay may be a delay in providing access to the owner of the site, changes in the scope of work or construction details, and suspension or termination of temporary work by the owner.
2. Delays are excused without compensation (Excusable Non-compensable delay).This delay is caused by events beyond the control of the contractor and the owner of such extreme weather, fires, floods, strikes, and vandalism.
3. Delays are not forgiven (Non-compensable delay).This delay may occur due to inefficiency by the contractor, the owner may request compensation for the delay in the total duration of the project to the contractor. The reason for this delay may be less qualified jobs, technical, or material, delays in producing the design, and the failure to coordinate the work.
4. Delay joint (Concurrent delays).This delay is due to several factors overlap caused by the owner and the contractor, the solution required careful analysis to prove the share of each of the parties concerned.

Types of Project Delay Claim Construction

According to Yasin, (2008) types of claims that can be proposed when delays occurred on construction projects as follows:

1. Claims extra cost and time. Claims of this type usually demanding to requests for additional time and additional costs. Among several types of claims, these two types of claims that often arise due to delays in completion of the work
2. Claims indirect costs. Providers that late finishing a job for reasons of service users, requesting additional overhead costs by reason of this cost increase because the work has not been completed.
3. Claims extra time. Service providers are only given additional time at no additional implementation costs due to certain reasons.
4. Compensation claims another. In some cases, service providers other than getting extra time also obtain other compensation

METHODOLOGY

Types of Research

Form of research is a survey research, research conducted by distributing questionnaires to 40 respondents consisting of consultants, contractors and project owners. In this study, the information obtained by using questionnaires and interviews with the respondents. Location of the study are used to capture the data is in the area of Central Java in particular Klaten, Sukoharjo, and Winton.

Population and Sample

Respondents in this study is a contractor team consisting of project managers, site managers and quality control, consultants consisting of planners and consultants consultant and owner supervisors involved in hospital construction projects. sampling method in this study using the proportionate stratified random sampling.

Data Analysis Techniques

1. Factor analysis was done to reduce the dimension of data causes of claims delays in the construction of hospitals.
2. Test rankings mean scores and standard deviations were aimed to determine the activity of the dominant group delays, as well as the frequency of these types of claims are often made.
3. Test Independent sample t-test to compare the level of group delay consultant and contractor activities.

4. Multiple linear regression analysis was conducted to obtain the factors causing delay claims that have a significant effect on hospital construction projects.
5. The chi-square test to calculate the ratio of the frequency of delay claims made by consultants and contractors with experience of working between 1-5 years and> 5 years.

DATA ANALYSIS AND DISCUSSION

Factor Analysis

Results of sample adequacy test showed that KMO 0.702 > 0.5 and Bartlett Test(significant 0.00 < 0.05). This means that the analysis could be preceded due to the adequacy of the samples. (Source: Poenomo, 2015).

Table 1.Rotated Factors of Delays are Formed Based of Eigen Value > 1. (Source: Poenomo, 2015)

Rotation Matrix		
Factors Generated	**Varians %**	**Cumulative %**
Labor Factor (X1)	14.952	14.952
Factors working equipment (X2)	12.638	27.590
InformationFactor (X3)	11.195	38.785
Material Factors (X4)	9.835	48.620
Location Factor (X5)	8.221	56.841
Managerial Factor (X6)	8.134	64.974
Factors of Accidental work (X7)	6.854	71.828

Tabel 2. *Independent samples test*for Group Activities(Source: Poenomo, 2015)

	F	Sig.	Note	Rank
X1	2,591	0,116	not significant	15
X2	0,083	0,775	not significant	16
X3	0,115	0,737	not significant	9
X4	0,001	0,971	not significant	3
X5	0,706	0,406	tidak signifikan	14
X6	0,001	0,970	not significant	5
X7	4,019	0,052	not significant	11
X8	0,069	0,794	not significant	7
X9	5,206	0,028	significant	8
X10	3,404	0,073	not significant	13
X11	5,988	0,019	significant	17
X12	2,838	0,100	not significant	1
X13	7,259	0,010	significant	2
X14	0,067	0,797	not significant	6
X15	4,366	0,043	significant	4
X16	2,767	0,104	not significant	10
X17	0,060	0,808	not significant	12

Linear Regression Analysis of Factors Causing Delays in Construction of Hospitals

Based on the results of the analysis consists of 17 variables work there are 4 groups of work that has significant delay value comparison between the consultant and the contractor that ceiling and partition work, painting work, power and lighting panels work and work and work the water pump water heater.

Table 3. Independent Samples Test Delay Ceiling and Partition Work(Source: Poenomo, 2015)

		Levene's Test for Equality of Variances		t-test for Equality of Means		
		F	Sig.	t	df	Sig. (2-tailed)
X9	Equal variances assumed	5,206	,028	,562	38	,577
	Equal variances not assumed			,548	28,555	,588

Sig. variance results obtained probability 0.028 <0.05 so it can be concluded variant data is not the same. Based on table t test probability obtained 0.588> 0.05 so that we can conclude there is no delay equation ceiling and partition work by consultants and contractors

Differences delay that occurs on the job because of the ceiling and partition consultants assess employment ceiling and partition based planning schedule while contractors assess the ceiling and partition work related to other work inside ceiling and partitions.

Table 4.Independent Samples Test of Delay Painting Works.(Source: Poenomo, 2015)

		Levene's Test for Equality of Variances		t-test for Equality of Means		
		F	Sig.	t	df	Sig. (2-tailed)
X11	Equal variances assumed	5,988	,019	1,087	38	,284
	Equal variances not assumed			1,072	33,387	,291

Sig. variance results obtained probability 0.019 < 0.05 so it can be concluded variant data is not the same. Based on table t test was obtained probability 0.291> 0.05 so that we can conclude there is no delay equation painting work by consultants and contractors

Differences delay that occurs in a painting because the consultant assess a painting based planning schedule while judging a painting contractor requires a gradual process and related to building maintenance period.

Table 5. Independent Samples Test for Delay of Activity for Power Panels and Lighting Work (Source: Poenomo, 2015)

		Levene's Test for Equality of Variances		t-test for Equality of Means		
		F	Sig.	t	df	Sig. (2-tailed)
X13	Equal variances assumed	7,259	,010	2,549	38	,015
	Equal variances not assumed			2,503	31,592	,018

Sig. variance results obtained probability 0.010 <0.05 so it can be concluded variant data is not the same. Based on table t test was obtained result of probability 0.018 <0.05 so it can be concluded there is a delay equation ceiling and partition work by consultants and contractors

Differences delays on power and lighting panels work due to the consultant and contractor assess the work of the power panel and lighting can change due to changes in the needs of the equipment due to the owner.

Table 6.Independent Samples Test Delay Work Water Pump & Water Heater (Source: Poenomo, 2015)

		Levene's Test for Equality of Variances		t-test for Equality of Means		
		F	Sig.	t	df	Sig.(2-tailed)
X15	Equal variances assumed	4,366	,043	-1,515	38	,138
	Equal variances not assumed			-1,489	32,049	,146

Sig .variance results obtained probability 0.043 <0.05 so it can be concluded variant data is not the same . Based on table t test probability obtained 0146 > 0.05 so that we can conclude there is no delay equations water pump work and water heater according to consultants and contractors .

Differences delays on power and lighting panels work due to the consultant and contractor assess the work of the power panel and lighting can change due to changes in the needs of the equipment due to the owner .

Table 7. Summary of Regression Models of DelayFactors(Source: Poenomo, 2015)

Model	R	R Square	Adjusted R Square	Std. Error of the Estimate
1	**.591[a]**	**.350**	.207	.594

Tabel 9. ANOVA of Delay Factor (Source: Poenomo, 2015)

Model		**Sum of Squares**	**df**	**Mean Square**	**F**	**Sig.**
1	Regression	6.076	7	.868	2.458	**.039[b]**
	Residual	11.299	32	.353		
	Total	17.375	39			

Table 9. RegressionCoefficients of Factors Causing Delay Claim(Source: Poenomo, 2015).

Model		**Unstandardized Coefficients**		**Standardized Coefficients**	**t**	Sig.
		B	**Std. Error**	**Beta**		
1	(Constant)	.666	.940		.709	.484
	X1	-.104	.175	-.093	-.592	.558
	X2	.339	.163	.320	2.081	**.046**
	X3	.384	.184	.338	2.087	**.045**
	X4	-.485	.185	-.413	-2.625	**.013**
	X5	.012	.160	.012	.072	.943
	X6	.107	.182	.094	.587	.562
	X7	.012	.140	.012	.083	.934

Table 10.Goodness of Fit form Stepwise Method(Source: Poenomo, 2015).

Model	**R**	**R Square**	**Adjusted R Square**	**Std. Error of the Estimate**
1	,364[a]	,132	,109	,630
2	,497[b]	,247	,207	,594

Table 11. Stepwise Method of ANOVAfrom Factors Causing Delays (Source: Poenomo, 2015).

	Model	**Sum of Squares**	**df**	**Mean Square**	**F**	**Sig.**
1	Regression	2,298	1	2,298	5,792	,021[b]
	Residual	15,077	38	,397		
	Total	17,375	39			
2	Regression	4,298	2	2,149	6,081	,005[c]
	Residual	13,077	37	,353		

	Total	17,375	39			
	Regression	5,849	3	1,950	6,089	,002[d]
3	Residual	11,526	36	,320		
	Total	17,375	39			

Based on the comparison of the results of the stepwise regression method R square value on Predictor X3 for 0132, which means the information factors can explain 13.2% delays in hospital construction projects with significant value of F table X3 is 0.021 <0.05 can be expressed factors significantly influence the delay information.

R square value of the predictors for 0247 from X3 and X4, which means a factor of information and material factors can account for 24.7% delays in hospital construction projects with significant value of F table X4 is 0.005 <0.05 can be expressed and information material factors significantly influence the delay

R square value of the predictors X3, X4, and X2 by 0337, which means a factor information, material factors, and factors of work equipment can be explained by 33.7% delays in hospital construction projects with significant value of F table X2 is 0.002 <0.05 can be expressed factor information, material factors, and factors significantly influence the work equipment delays

Comparative analysis of claims consultants and contractors

Based on the result (see Table 12), it can be obtained information that the most frequent types of claims by both consultants and contractors are additional claims without additional cost to the average value of 3.24.

Tabel 12. Frequency of Claim Types byConsultant dan Contractor(Source: Poenomo, 2015).

	Type of Claim	Mean		
		Consultant	Contractor	Total
1	Claims for additional cost and time	2.80	3.00	2.90
2	Claims indirect costs	2.20	2.52	2.36
3	Claims for additional time at no extra charge	3.10	3.38	3.24
4	Other compensation claims	2.00	2.05	2.02

Table 13. Crosstab Frequency Delay Claim by Consultants And Contractors.(Source: Poenomo, 2015).

	Frekuency of delay claim					Pearson
	Low	Quite low	Medium	High	Total	Chi-Square
Consultant	1	10	5	3	19	
Contractor	0	5	15	1	21	0.035
Total	1	15	20	4	40	

Comparison of the frequency of claims delays between consultants and contractors gained significant value 0.035 <0.05 that the ratio of claims delays between consultants and contractors can be declared significant. it can be concluded that the contractor is more frequent in construction delay claim hospital compared consulting firm.

Table 14 Crosstab Frequency Delay Claim toResponents'working Experience.(Source: Poenomo, 2015).

	Delay Claims frequency					Pearson
	Low	Quite Low	Medium	High	Total	Chi-Square
1 - 5 years	1	12	3	1	17	
> 5 years	0	3	17	3	23	0. 001
Total	1	15	20	4	40	

Comparison of the frequency of claims delays based on experience working in a construction project obtained significant value 0.001 <0.05 so it can be declared significant. In the group with 1-5 years of work experience to make a claim on the dominant frequency is low enough levels of the control group with over 5 years experience in the dominant frequency in moderate claim so that we can conclude a long experience of working directly proportional to the frequency of the delay claim.

CONCLUSIONS AND RECOMMENDATIONS

Based on research carried out can be concluded as follows:1). There are 7 factors that cause delays in the labor factor, factor working equipment, information factors, material factors, factors location, Managerial factors, and factors of workplace accidents. Of the seven factors formed then performed multiple linear regression to obtain three significant factors for hospital construction project delays that factor information, material factors, and factors that can explain the work equipment by 33.7% delay.2). Analysis of independent sample t-test delay in the work group concluded in the construction of the hospital group activities most frequently delayed by consultants and contractors is the type of mechanical and electrical work. Valuation differences between consultants and contractors delay due to the perception of a consultant assessment is based on the planning schedule while contractors consider the implementation schedule and a change in the scope of work.3). Analysis of the frequency and types of claims using the chi-square test was obtained levels of the most common claim is a claim extra time at no additional cost to the average value of 3.24 and frequency of filing a claim is more often done by a contractor with experience of working in over 5 years.

Recommendation can be given as follows:1). For consulting firms and contractors regarding the factors that delays often occured in a particular hospital construction project which is expected to be better prepared especially for factors that significantly influence the delay.2). In particular the construction of the development of the hospital must be considered the dominant types of claims occur so as to make preparations, especially at the planning stage so as to avoid losses that may occur when subjected to the delay claims**.**

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AN EVALUATION OF RELATIONSHIP BETWEEN MOTIVATION, WORK ENVIRONMENT, LEADERSHIP, WORK DISCIPLINE, AND PERFORMANCEOF EMPLOYEE IN A CONSTRUCTION COMPANY

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ABSTRACT: Human resources is a major driver of the company. Company's objectives depend on the quality of human resources involved in its organization. This study aims at evaluating the relationship between motivation, work environment, leadership, work discipline, and the performance of employees in construction services provider at Jakarta. Respondents in this study were the entire workers with 49 employees. Data were collected by distributing questionnaires to the entire employees. Structural Equation Modeling (SEM) was applied to determine the relationship between the variables in the study. The results showed that: 1) leadership has a significant relationship with the discipline of working; 2) work discipline has a significant relationship with the employee's performance; and 3) despite there were some relationship between motivation and discipline of work, work environment, and leadership, but they do not have significant relationship. The study suggested that leader of the company should strengthen the work discipline in order to improve the employee performance.

Keywords: *motivation, work environment, leadership, work discipline, and employee performance*.

INTRODUCTION

Background

Competition in construction services provider in Indonesia is getting tighter. To be able to compete in the market of the construction industry, the company must provide adequate resources, such as equipment, technology and human as the main driver of the company. In order to support management activities within the company, the company must have competent employees in their respective fields, knowledgeable, necessary skills, and desire to perform better. Developing employee performance would have the impact on the progress of the company, so the company is being able to survive and compete. Therefore, efforts to improve the performance of employees are serious challenge to the management for company's success. In achieving the its objectives, the company depends on the quality of performance of human resources.

PT Asiana Technologies Lestary (ATL) is a company engaged as a provider in construction services. The company was founded in 2001 and is specialised engaged in CivilEngineering (Hydrolic Engineerig, Wetlands and Coastal), Mechanical (Mining and Manufacturing, Construction Tools and Equipment Transportation Lift), Electrical (Installation Control and Instrumentation) and Environmental Management (Piping Water/Waste). The service area of the company covered several major cities in Indonesia, among others: Jakarta, Padang, Bandung, Surabaya, Bali and Makassar. The company expects high employee performance. In line with the increase in performance of employees, the company's overall productivity would increase and finally the company can survive in the market competition in the construction industry in Indonesia.

Based on the observations of researcher who also one of the employees of ATL, a trend that occurs when employees are less likely to get positive support from the leadership of the company. As providing facilities in the work, awards, allowances and bonuses project, the company can lead to decreased employee performanceand job would not be able to accomplishedthe targets. In addition, lack of binding regulations for the employees, it would lead to constraints being applied to the problem of discipline, customs and culture of the company. Leadership provides a positive and significant influence on employee performance (Potu, 2013). Then the thought of how sustainable the overall factors would affect the performance of each employee.

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Research Objectives

The research have four aims: a) to determine the relationship between motivation, work environment, and the leadership of the working discipline inATL; b). to determine the relationship between motivation and performance of employees of ATL.c) todetermine the relationship between leadership and employee performance of ATL. D) to determine the relationship between the discipline of working with the performance of employees of ATL.

LITERATURE REVIEW

Employee Performance

Performance of the organization is the answer to the success or failure of the established organizational goals. Wirawan (2009) stated that the performance was the output generated by functions or indicators of a job or a profession within a certain time.Mangkunagara (2000) definedthat performance was the result of the quality and quantity of work achieved by an employee in carrying out their duties in accordance with the responsibilities assigned to him.

Work Discipline

According Simamora (see Reza, 2010) is a procedure that corrects discipline or punish subordinates for violating the rules or procedures. Labor discipline is a tool used by managers to communicate with employees so that they are willing to change a behavior as well as an effort to increase awareness and willingness of a person to comply with all company regulations and social norms in force (Riva, 2004).

Motivation

Motivation is the process that explains the intensity, direction, and persistence of an individual to achieve his goal. Motivation is an "impulse or desire that causes a person to do something" (Nawawi, 2006 in Reza, 2010). Motivation can be defined as a motivating factor derived from the human self, which will affect the way a person acts. Thus, work motivation will affect work performance.

Work Environment

Nitisemito defined that work environment is something that was around the workers and could affect him in performing tasks (see Pramono, 2011). In line with what Supardi proposed (see Potu, 2013), the work environment was about the state of the workplace, both physical and non-physical that could give the impression of a pleasant, secure, reassuring and suggests home to work and so forth.

Leadership

According to Robert, leadership is how a leader perform leadership functions and how it be seen by those who are trying to lead or those who may be observed from the outside (see Reza, 2010). Leadership style is the behavior and strategies, as a result of a combination of philosophy, skills, personality traits, attitudes, which are often applied to a head when he tried to influence theperformance of subordinates (Tampubolon, 2007).

Research Accomplished

Research by Pariaribo (2014) with the title Influence of Leadership Style and Work Motivation on Job Satisfaction And Its Impact on Employee Performance at the Local Planning Board Supiori, the results showed that the style of leadership and motivation positive and significant impact on job satisfaction and employee performance.Research by Potu (2013) with the title Leadership, Motivation and Work Environment Effect on Employee Performance at the Regional Office of Directorate General of State Assets Suluttenggo and North Maluku in Manado. The results showed that leadership, motivation, and work environment simultaneously positive and significant impact on employee performance.

Research by Sari, et al (2012) examined that the Influence of Leadership, Motivation, and Job Stress on Employee Performance in SyariahMandiri Bank Branch Office Makassar. The results showed that leadership, motivation and job stress jointly significant effect on the performance of employees.

Research by Pramono (2011) with the title Influence Compensation, Motivation, Work environment and Leadership on Employee Performance PT. MitraAdiPratama Semarang. The results showed that partially contained significantly and positively influence the compensation, motivation, work environment and leadership on employee performance.

Research by Octaviana and Ariefiantoro (2011) with the title Influence of Work Motivation, Leadership, and Work Environment on Employee Performance (Case Study On Employee Contractor PT. WinehPandawangi Semarang. The results showed that motivation, leadership, work environment have a significant effect on the performance of employees.Research by Reza (2010) with the title Influence of Leadership Style, Work Motivation and Discipline on

Employee Performance PT SinarSantosa Perkasa Banjarnegara. The results showed that the style of leadership, motivation and discipline significant effect on employee performance.

Table 1. Variables Research and Indicators

Variable	Symbol	Indicator	A	B	C	D
Employee Performance	Z	Length of time completing the work		√	√	√
		Quality standards of the company	√	√		
		Suitability of the work with the target company	√	√	√	√
		Recognition and rewards of leadership		√		
		Desire to improve labor productivity		√		
		The use of the work load			√	√
		Attendance rate			√	
		Quantity of work	√			√
		Initiative in working	√			√
		Organizational Commitment				√
Work Discipline	Y	Quality of work				√
		Work quantity				√
		Compensation is required				√
		Location workplace or				√
		Conservation: respect for the rule				√
Motivation	X1	Security	√		√	√
		Financial	√		√	
		Social	√		√	√
		Award	√	√	√	√
		Ownership of the assets of the company	√	√		
		The attention of the leadership		√		
		Opportunity to take the initiative	√	√		√
		Chance resolve the problem of work	√	√		
		Physiological: hunger, thirst, clothing, housing				√
Work Environment	X2	comfortable working facilities and adequate		√		
		The level of noise and pollution		√		
		The relationship between employees		√	√	
		Communication between employees		√		
		Opportunity to discuss with the leadership		√	√	
		Freedom of creativity and innovation			√	
		Job security program			√	
Leadership	X3	Disclosure Leadership		√	√	√
		Delivery of instruction work	√	√		√
		Understanding of the conflicts in the work	√	√		√
		The attitude of the leadership firmness	√	√		
		The attitude of the leadership of justice		√		
		Exemplary behavior	√		√	
		Objective	√		√	√
		Fostering creativity	√		√	√

Source: Research (A) PariariboNoack, 2014; (B) AgusPramono, 2011; (C) KestriaOctaviana Twilight and True Ariefiantoro, 2011; (D) Regina Aditya Reza (2010)

Hypothesis Developed

Four hypothesis were developed as followed: 1) motivation, work environment, and leadership have a real relationship (significant) with work discipline; 2) motivation to have a real relationship (significant) with employee performance; 3) leadership has a real relationship (significant) with employee performance; and 4) discipline of work have a real relationship (significant) with employee performance.

VariablesResearch

Variables were developed based on literatures as shown in Table 1. (Raja, 2015)

Framework

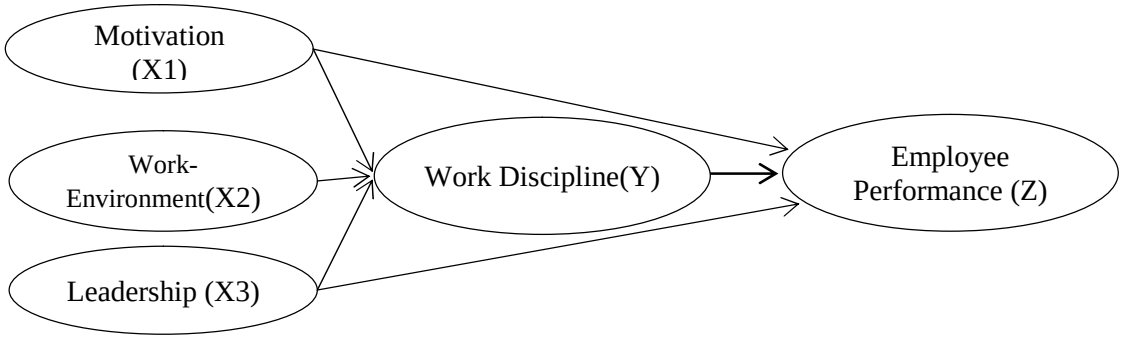

Fig. 1 The Framework

RESEARCHMETHODOLOGY

Respondents/Samples

The population is a mix of all the elements in the form of events, things, or people who have similar characteristics that became the center of attention of researchers, therefore regarded as a research universe (Ferdinand, 2006 in Reza, 2010). Inroads in this study were all employees in the company of ATL totaling 49 employees.

The sample is a subset of the population, made up of some members of the population (Ferdinand, 2006 in Reza, 2010). In this study did not use sampling techniques since the sample studied is the whole of the population is called the census. Referring to the opinion of Sugiono (see Pariaribo, 2014), that sometimes the problem of sampling altogether eliminated by incorporating the entire population as the respondents, namely all of the known population size is limited by considerations such as: a). limited number of population, which is less than 100 so still within reach of researchers; b). by using the whole population as a respondent, then the data more representative; and c). respondents were people who had clearly known.

Data Collection

Data collection methods used in this research is to use a questionnaire or a questionnaire addressed to the subject of research / respondent and through literature.The questionnaire is divided into three parts, namely: 1) The first part is filled by employees: the data respondent characteristics (gender, age, marital status, education last, long work). 2) The second part is filled by employees: the question of motivation, work environment, leadership. 3) The third part is filled by the direct supervisor of the employee who fills the first and second: the question of labor discipline and employee performance.

DATA ANALYSIS AND DISCUSSION

Data Analysis Three Pre-determined Techniques.

It consists of the following: 1) Testing the validity of the statement shows all the items in the questionnaire used in this study is valid because it has a value r calculated>rtable. 2) Reliability testing showed that all variables have Cronbach's Alpha value greater than 0.6 and it can be concluded that the questionnaires used in this study has a reliable measure.3). Factor Analysis was conducted to compact the variables. A sample was considered to be sufficient to further analyzed by factor analysis when the number of KMO and Bartlett's Test> 0.5 with sig. <0.05 (see Santoso, 2012). The test results showed all variables can be said to meet the requirements and adequate for further analysis. In addition, each of the indicators in the variable must have a value of MSA> 0.5 in order to be published in all the indicators are still unpredictable and can be analyzed further. By looking at the output of anti-image correlation, all indicators have MSA values> 0.5. After all the variables meet the specified requirements, do extraction and rotation to see how many factors formed and indicates the position of an indicator on a factor. Table 2 showed the result of the factors.

Table 2. Number of Factors Formed(Source: Raja,2015)

Major Factor	Total Factors	**Total Variance(%)**
Motivation (X1)	6	68,912
Working Environment (X2)	3	70,513
Leadership (X3)	3	75,650
Work Discipline (Y)	3	72,125
Employee Performance (Z)	3	77,224

SEM (Structural Equation Modeling) for the Study

In the SEM models are used, the variable motivation, work environment, leadership, work discipline, and the performance of the employee becomes latent variables. While the factors that formed became manifest variables.

1). Test data distribution

The level of trust that is used in this test was 99%, 1% significance level with a value of z = 2.58 (Standard Normal Distribution Table). Thus, a distribution considered normal if the number c.rskewness or numbers c.r.kurtois is between -2.58 and +2.58.From the test results, the data cannot be said to amount to 49 normal distribution, either individually or jointly as there are some values c.rskewness and c.r.kurtois that does not lie between -2.58 and +2.58. Because the data are not normally distributed, performed data outlier detection by looking mahalanobis d-squared value of the largest and smallest values p1 and p2, the data number 42 is the most outlier. Most data outliers removed and repeated testing data normality. As a result, normally distributed data were expressed either individually or jointly.

A model declared fit with existing data and meet the eligibility criteria if: 1) the value of chi-square (χ^2) count <χ^2 table with P> 0.05; 2) The value of GFI (Goodness of Fit Index) and AGFI (Absolute Goodness of Fit Index) is close to 1; and 3) the value of NFI (normed Fit Index), CFI (Comparative Fit Index), IFI (Incremental Fit Index) and RFI close to 1.From the test results indicate that the model cannot be declared fit models with existing data and meet the eligibility criteria. There are several actions that can be performed, the first increase the number of samples and the second action is to create a path diagram (Santoso, 2014). Because the samples used in this study were the whole of the population, it is not possible to increase the number of samples. Path diagram was chosen to complete the model.

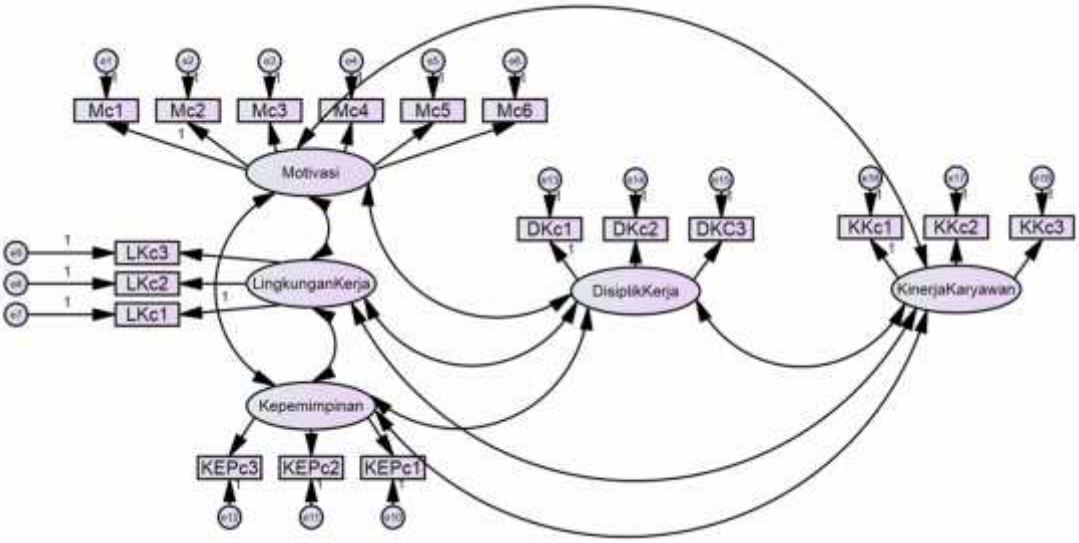

Fig. 2 Measurement Model (Source: Raja, 2015)

2). Path Diagram.

Path freely diagram can be interpreted as a model simplification process 'filled' with indicators, to be a model that will unify the existing indicators (Santoso, 2014).

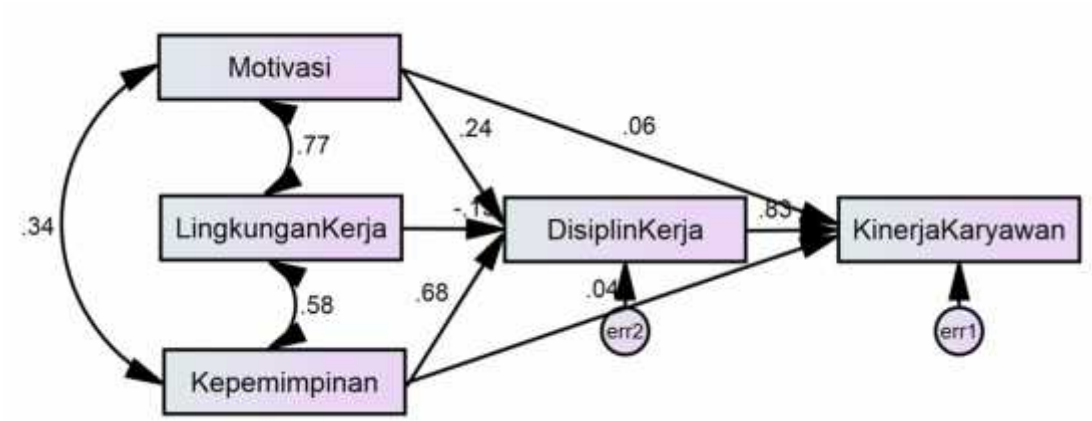

Fig. 3Final Model based on Results of Analysis of Composite model (Source: Raja,2015)

Table 3. Regression Weights Composite Model(Source: Raja,2015)

Variables Relationship			Estimate	S.E.	C.R.	P
Work Discipline	<---	Work Environment	-.176	.261	-.675	.500
Work Discipline	<---	Leadership	.598	.113	5.297	***
Work Discipline	<---	Motivation	.452	.311	1.450	.147
Employee Performance	<---	Work Discipline	.621	.070	8.858	***
Employee Performance	<---	Motivation	.084	.102	.824	.410
Employee Performance	<---	Leadership	.029	.061	.484	.628

Table 4. Regression Weights Standarized Composite Model(Source: Raja,2015)

Variables Relationship			Estimate
Work Discipline	<---	Work Environment	-.128
Work Discipline	<---	Leadership	.683
Work Discipline	<---	Motivation	.238
Employee Performance	<---	Work Discipline	.833
Employee Performance	<---	Motivation	.060
Employee Performance	<---	Leadership	.045

A variable declared to have relationships with other variable when the probability value (P) is smaller than 0.05. Leadership has a real and significant relationship with work discipline, P (0.000) <0.005, with a factor loading value of 0.683. Discipline work has a real relationship (significant) with employee performance, P (0.000) <0.05, with the value of the factor loading 0,833.

4.3Discussion

The results showed that motivation does not have a significant relationship with the discipline of work, so it can be interpreted that the motivation of employees both from outside and from inside the employees themselves do not affect the discipline of employees. In addition, it points out that the work environment does not have a significant relationship with the discipline of work, so it can be said that the circumstances surrounding the work place both physically and non-physically not establish discipline employees.

This study also showed that motivation does not have a significant relationship with the employee's performance, so it can be interpreted that the motivation of employees both from outside and from inside the employees themselves do not affect the performance of the employee. This is not in line with the results Pariaribo (2014), Pramono (2011), Octaviana and Ariefiantoro (2011), and Reza (2010) which stated that the motivation significant effect on employee performance.

The results showed that leadership does not have a significant relationship with the employee's performance, it can be concluded that the behavior and attitude of the leaders in the company does not affect the performance of the employee. This is not in line with the results of research conducted by Potu (2013), Sari et al (2012), Octaviana and Ariefiantoro (2011), which states that the leadership has a significant influence to employee performance.Based on the results of tests

that have been conducted, proving that leadership has a significant relationship with the discipline of work, and to see the loading factor (leadership ◊ work discipline) of 0.683> 0.5 can be said that the leadership was instrumental in shaping the discipline of employees. These results are consistent with the theory put forward Rival (2004), that the labor discipline is a tool used by managers to communicate with employees so that they are willing to change a behavior and an effort to raise awareness and willingness of a person to comply with all company regulations and norms prevailing social.

The results also show that the discipline of work has a significant relationship with the employee's performance, and to see the loading factor (labor discipline ◊ employee performance) amounted to 0.833> 0.5 can be said that the work discipline plays an important role in shaping the performance of employees. These results are consistent with the results of research Reza (2010) who also found that discipline significant effect on employee performance.This study also shows the relationship between the variables of motivation, work environment, and leadership. The relationship between motivation and work environment, work environment with leadership, motivation and leadership is a significant correlation (P <0.05), and the positive direction. So it can be interpreted that cozy feeling to the work environment affect employee motivation, if the circumstances surrounding the work place both physically and non-physically better and more comfortable, the motivation of employees will increase and vice versa. Attitudes and behavior of leaders in the company affect the working environment of employees, if the leader being open, caring, assertive, and impartially, the employee will feel comfortable with the workplace and have a good relationship with colleagues and partners of the company and vice versa. Similarly, the relationship between leadership and motivation, leadership attitude in the company will increase the motivation of employees and vice versa.

CONCLUSION AND RECOMMENDATION

Based on the above conclusions, can be given advice to top management of ATL, that in order to improve the performance of employees is necessary to improve labor discipline and to improve the discipline of work needs to be done to improve the attitude of the leadership of caring, open, firmly and impartially in the company. In addition, although this study showed no significant relationship between motivation and discipline and work environment with work discipline, adapted from previous studies on the performance of employees, management companies need to pay attention to how employees can be motivated by good, then to create and maintain work environment is conducive and comfortable for employees so that employees can increase performance and support companies to achieve goals and be able to compete in the construction industry market.

The results of this research and the limitations that can be used as a source of ideas and input for the development of research related to HR (Human Resources) in the field of construction services in the future, then the suggested extension of this research include: increasing the number of variables and also the number of samples sufficient to meet requirements for use with SEM analysis.

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CRITICAL SUCCESS FACTOR IN PARTNERSHIPS BETWEEN MAJOR STAKEHOLDERS IN LOW-COST APARTMENT DEVELOPMENT IN SURABAYA METROPOLITAN AREA

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ABSTRACT: The low-cost apartment development program is one of the reliable solutions to reduce housing backlog in East Java Province due to land scarcity and high price of land. But there were many problems facing the implementation of low cost apartment such as unconformity with regulation and lack of private sector involvement to develop these projects. Poor public decision making process is one of the reasons why private sector did not have interest to be involved in the partnership in low-cost apartment development. However, private sector is still highly expected to contribute in this program. The objective of this research is to identify critical success factor for public private partnership from stakeholder's perception. The secondary data of housing backlog were provided from East-Java Provincial Body of Planning and Development and the variables of critical success factors were obtained from literature review studies. The critical success factor variables to better understand the low-cost apartment implementation were then validated by 30 purposive sampling respondents from both government and private sectors. These chosen variables were then identified and analyzed using importance analysis and independent sample T-test.
The result shows that the critical success factors from government perception are public decision making process, and site availability. While the critical success factor from private sector perceptions are low income group's ability to pay, available financial market and favorable legal framework.

Keywords: *Critical Success Factor, Public Private Partnership, low-cost apartment, Surabaya Metropolitan Area.*

INTRODUCTION

One of procurement and partnership method is Public Private Partnership which has been applied in a lot of infrastructure projects. PPP plays an important role in bringing private sector competition to public monopolies in infrastructure development and service provision, and in merging the resources of both public and private sectors to better serve the public needs that otherwise would not be met (Zhang and Chen, 2013). PPP has been well practiced in many developed countries in Europe, America and Australia for delivering construction and building projects (Cheung et al, 2012). The critical success factor researches have been conducted in some research, such as in Australia, Hongkong and United Kingdom (Cheung et al, 2012). The same topics have been done in Libanon (Jamali, 2004) and Malaysia (Ismail and Ajija, 2013).

In Indonesia, PPP has contributed to the development of a lot of infrastructure project, such as transportation, tollroad, energy, shipping, oil and gas, and telecommunication (National Body of Planning and Development, 2014). Only a few private sector who are interested to participate by building partnership in housing project, especially in low-cost apartement development. Financial scheme regarding in investment return is the reason why private sector did not interest to involve in this partnership.

A number of previous research do not expecially examined the critical success factor of public private partnership in housing project. But several research has investigated that the dominant failure factor in public private partnership in low-cost housing project in the developing countries is related to deliver their intended purpose of enabling low-income groups to have access to sustainable homeownership (Trangkanont, 2012). Moreover, this paper is aimed to examine the critical success factor in Indonesia, focused on low-cost apartment development program in Surabaya Metropolitan Area. This paper focuses on the results obtained from a recent survey of PPP low-cost apartment projects in Surabaya Metropolitan Area concerning the factors that are considered critical to the success of PPP project implementation.

In particular, there are three objectives of this present study. First, it aims to examine the importance of the success factors as perceived by the overall respondents. Second, the study intends to identify the differences in perception concerning the importance of the success factors between the public and private sectors. The last objective is to show evidence concerning the importance of the factors of the two key parties involved in PPP– the public sector and private sector.

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LOW-COST APARTMENT IN INDONESIA

According to the Regulation of the Minister of Public Works No. 05/PRT/M/2007, low-cost apartment is the high-rise buildings, which is divided into sections and are functionally structured in horizontal and vertical directions and are units that each can be owned and used separately, equipped with common parts, common objects and common ground.

The land scarcity and the housing expensive price in Surabaya are kind of housing problem which cause of the housing backlog. These are the reason why the low-income people live in illegal slump area, in a suburb area or in greater Surabaya. Due to land scarcity and housing backlog problem, going up is the most effective solution to optimise the available land (Susilawati and Yakobus, 2010).

In Indonesia, low-cost apartment development is one of Government's programs in housing provision to reduce housing backlog and implement affordable housing program. The existing low-cost apartments (strata-title housing) were always built by the Ministry of Public Works and the Ministry of Public Housing in local government's asset land (Indonesian Act no 20/2011, article 17). Until 2014, 276 twin blocks include 21,644 units have been established and managed by East Java Province Government and local Government in East Java Province (Public Works Department, East Java Province, 2014). Some low-cost apartment projects in Surabaya Metropolitan Area have been adopting partnership between public and private. The private sector'e roles are land providing, management and investment (Public Works Department, East Java Province, 2014).

PUBLIC PRIVATE PARTNERSHIP

A PPP is defined as a partnership between the public sector and the private sector for the purpose of delivering a project or a service traditionally provided by the public sector (European Commission, 2003). This partnership has benefits: PPP have developed in part due to financial shortages in the public sector and PPPs have demonstrated the ability to harness additional financial resources and operating efficiencies inherent to the private sector (European Commission, 2003)

Some previous researchers mentioned that government has tried to make the private sector or investors involve to provide low-cost apartment. However, not many investors are interested due to the budget limitation, financial problem and the technical standard requirement (Susilawati and Yakobus, 2010; Susilawati et al, 2009). Eventhough there are some risks concerning PPP implementation in housing projects, the partnership in this fields is still possible to be adopted more extensively. Certain critical conditions must be satisfied for the PPP model to succeed (Wibowo, 2014).

CRITICAL SUCCESS FACTOR IN PUBLIC PRIVATE PARTNERSHIP

Rockart (1982) in Li et al (2005) defines Critical Success Factors (CSFs) as "those few key areas of activity in which favourable results are absolutely necessary for a manager to reach his/her goals. A large number of articles have defined and discussed PPP critical success factors (CSFs) extracted from case studies and research surveys (e.g. Chan et al., 2010; Zhang, 2005; Trangkanont and Charoenngam, 2013). Most articles derive CSFs that typically center on issues of appropriate risk allocation between contracting parties, strong and good private consortium, favorable legal framework, supportive political and business climates, fair and transparent procurement processes, and sound financial viability.

Ismail and Ajija (2013) mentioned the 5 most critical factors of PPP in Malaysia infrastructure projects . They are (1) good governance; 2) commitment and responsibility of public and private sectors; 3) favorable legal framework; 4) sound economic policy; and 5) available financial market. Whilst Li et al (2005) investigated the CSF of PPP in UK construction industries and classified the factors into 5 groups. They are effective procurement, project implementability, government guarantee, favorable economic condition and available financial market.

The PPP in housing projects researches were conducteds specifically in Thailand (Trangkanont and Charoenngam, 2013) which studied about critical failure factors in low-cost housing project in Thailand and Australia (Susilawati and Armitage, 2005) which investigated the barrier in entering partnership for affordable housing. The most critical factors in Thailand low-cost housing projects are policy pressure, procurement process and inappropriate contractors. Whereas the most barrier factors are negative outcomes, power and status differences, Organisational conflicts and Difficulty in merging values and cultures. In this paper, the critical factors of PPP in general, PPP in housing projects and pilot survey are combined.

The summary of CSFs for PPP housing projects:

Table 1. Critical Success Variables

No	Critical Success Factor	Criteria	Source
1	Stable macro-economic condition	Economic	Hardcastle (2005), Ismail (2013) Trangkanont (2014)
2	Favourable legal framework	Legal	Hardcastle (2005), Ismail (2013) Trangkanont (2014)

3	Sound economic policy	Economic	Hardcastle (2005), Ismail (2013) Trangkanont (2014)
4	Available financial market	Economic	Hardcastle (2005), Ismail (2013) Trangkanont (2014)
5	Multi-benefit objectives	Government	Hardcastle (2005), Ismail (2013)
6	Appropriate risk allocation and risk sharing	Implementation	Li (2005); Hardcastle (2005), Ismail (2013) Trangkanont (2014)
7	Commitment and responsibility of public and private sectors	Implementation	Hardcastle (2005), Ismail (2013) Trangkanont (2014)
8	Strong and good private consortium	Consortium	Hardcastle (2005), Ismail (2013) Trangkanont (2014)
9	Good governance	Government	Li (2005), Hardcastle (2005), Ismail (2013)
10	Shared authority between public and private sectors	Implementation	Li (2005), Hardcastle (2005), Ismail (2013)
11	Political decision making process	Legal	Li (2005) Hardcastle (2005), Ismail (2013) Trangkanont (2014)
12	Competitive procurement process	Implementation	Hardcastle (2005), Ismail (2013) Trangkanont (2014)
13	Government involvement by providing guarantee	Government	Li (2005), Hardcastle (2005), Ismail (2013)
14	Thorough and realistic assessment of the cost and benefits	Implementation	Hardcastle (2005), Ismail (2013)
15	Site availability	Implementation	Li (2005), survey
16	Low-income group's ability to pay	Implementation	Trangkanont (2014), survey

RESEARCH INSTRUMENT

The study adopted variables from the previous research that contribute to the success of Public-Private Partnership (PPP) projects, as shown in Table 1 with some reductions and adjustment to make it more realistic. The rationale for adopting similar success factors to those used in prior studies, particularly by Li (2003) and Hardcastle, Trangkanont and Ismail.

The survey was conducted during October 2014-February 2015 to low-cost apartment manager in Surabaya Metropolitan Area under the partnership agreement. There are 3 low-cost apartments, namely Graha Asri, Warugunung and Siwalan Kerto which each has certain scheme. In all, 32 completed questionnaires were returned out of the 32 distributed. Of these, 17 respondents came from public sector (Surabaya Municipality Government, East Java Province Government) and 15 from the private sector (investor, potential investor and management team). The effective return rate is 100% as questionnaires were compiled by purposive sampling of observation and interview.

DATA ANALYSIS

The relative importance of the CSFs was analysed by means of Likert rating scale questions in the survey instrument. The scale were very important (5), important (4), moderate (3), unimportant (2) and very unimportant (1). Statistical analysis undertaken include reliability test, importance analysis and Independent sample T-test. The cronbach alpha reliability for the factors is 0.812, which means the data collected for the critical factors are reliable.

FINDINGS AND DISCUSSION

Based on the overall respondents' results, the top five most critical factors, in descending order of importance are:

1. Political decision making process
2. Commitment and responsibility of public and private sectors
3. Appropriate risk allocation and risk sharing
4. Site availability
5. Good governance

Public sector result

1. Political decision making process

2. Site availability
3. Commitment and responsibility of public and private sectors
4. Appropriate risk allocation and risk sharing
5. Strong and good private consortium

Private sector result
1. Low-income group's ability to pay
2. Available financial market
3. Favorable legal framework
4. Appropriate risk allocation and risk sharing
5. Government involvement by providing guarantee

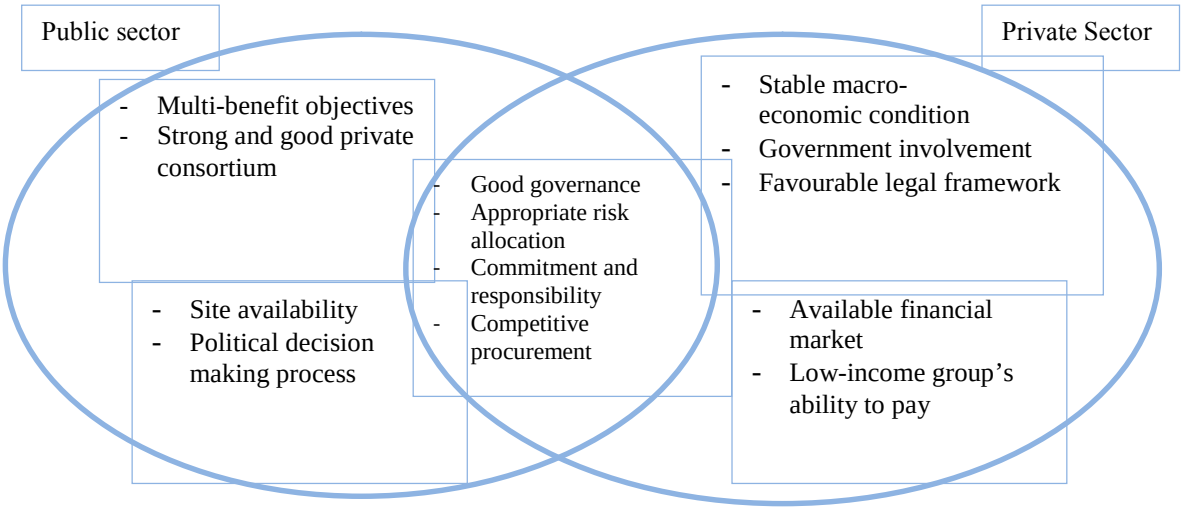

Figure 1. The highest rank critical success factor from public and private sector's perception

Discussion:

For overall respondents' answer, the political decision making was ranked first as a necessary factor of the success of PPP in low-cost apartment projects. The decision making process was significantly effected by political situation. Successful PPP implementation requires a stable political and social environment, which in turn relies on the stability and capability of the host government (Chan et al, 2010). The 'commitment and responsibility of both public and private sectors' is the second most important CSF, as perceived by the overall respondents. This is consistent with the argument by Chan et al. (2010), and Li et al. (2005) who claimed that commitment is one of the fundamental principles in partnership. As explained by previous research, the attitude of the stakeholders in a PPP project has an influence on the quality of outputs, such as stakeholder relationships and stakeholder management. Both parties have to manage the project seriously and share the authority and responsibility clearly.

Appropriate risk allocation and risk sharing was ranked as a third critical factor from all parties perception. The risk allocation and risk sharing analysis would consider whether the public sector should either accept the high risk cost, share the risks with the public sector, or retain the risk in the public sector (Li et al, 2005). The risk allocation analysis would suggest that certain risks should be retained by the public sector, transfered to the private sector or shared between the two sectors.

Site availability was ranked fourth as a necessary factor to ensure the success of PPP projects. It is common problem that it is not easy for government to provide land for low-cost apartment building due to the location requirement and limitation of land asset. The fifth place was good governance. This factor also refer to the consortium's ability to manage and operate the projects. Furthermore, Farhana (2010) and Wibowo (2014) argued that a well-defined legal framework is necessary for PPP projects to prevent corruption. The corruption problem has been the most failure factor in PPP projects

Different perception of critical success factor for low-cost apartment projects between public and private sector were specifically in site availability and low-income's group ability to pay. In this paper, these two factors would be the highlight as they were important factor that influence the successfull of PPP implementation in low-cost apartment projects. Site availability were placed in the forth position by government perception, after Political decision making, strong and good private consortium and commitment and responsibility of public and private sectors in the first, second and third rank respectively. According to Indonesian Act no 20/2011, the land should be provided by local government

and then Ministry government would build on it. Unfortunately, the land acquisition process was not easy and it has led the local government to provide the land asset for low-cost apartment projects. Furthermore, government would involve the private sector to provide the land. The other fact, the growth of low-cost apartment in a province depends on the local government's initative to propose the low-cost apartment projects in their location to the ministry government.

In contrast, private sectors did not see the site availability factor as a significant factor. They tend to consider the financial aspect, such as low-income's ability to pay the rental fee. Low-cost apartment is dedicated to the low-income, so that it is private sector's concern to make sure the investment return from the revenue in the form of rental fee. The low-income's ability to pay was ranked as a fifth critical factors from the public sector's view, after available financial market, appropriate risk allocation and risk sharing, commitment and responsibility of public and private sectors and favorable legal framework respectively.

According to private sector's answer in the questionnaire, the available financial market is the first rank. Many researchers Akintoye et al. 2001; Li et al. 2005; Zhang 2005 have found that project financing is a key factor for private sector investment in public infrastructure projects. The availability of an efficient and mature financial market with the benefits of low financing costs and a diversified range of financial products would be an incentive for private sector taking up PPP projects (Chan et al, 2010).

This paper also examines the difference from mean analysis using Independent sample T-test , as described as follows:

X_1	Stable macro-economic condition	Different
X_2	Sound economic policy	Different
X_3	Available financial market	Different
X_4	Low-income group's ability to pay	Different
X_5	Multi-benefit objectives	No differences
X_6	Good governance	Different
X_7	Government involvement by providing guarantee	Different
X_8	Strong and good private consortium	No differences
X_9	Appropriate risk allocation and risk sharing	No differences
X_{10}	Commitment and responsibility of public and private sectors	No differences
X_{11}	Shared authority between public and private sectors	Different
X_{12}	Competitive procurement process	Different
X_{13}	Thorough and realistic assessment of the cost and benefits	Different
X_{14}	Site availability	No differences
X_{15}	Political decision making process	No differences
X_{16}	Favourable legal framework	Different

CONCLUSION

This paper has presented the findings from a questionnaire survey conducted in Surabaya Metropolitan Area, East Java Province, Indonesia, looking at the 16 previously identified and modified CSFs for PPP. The statistical analysis was used to identify the CSFs. The findings showed that 16 variables are realiable which was shown by reliability analysis 0.812.

The differences in the ranking of the CSFs between stakeholders reflect the unique nature of PPP implementation in different perception of responsibility. Moreover, the different perception of critical factor in PPP low-cost apartment implementation, has described the different view of the essential factor for the successful of PPP from government and private sector. There are some essential variables for both parties, they are Good governance, Appropriate risk allocation, Commitment and responsibility and Competitive procurement. These variables are critical to be considered for the PPP housing implementation. Corresponding effective strategies based on those CSFs identified can also be generated for successfully delivering future PPP projects for accelerated excellence.

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TIME-PERFORMANCE PREDICTION OF CONSTRUCTION PROJECTS USING BELIEF NETWORKS

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ABSTRACT: Time performance of construction project is affected by various factors of uncertainty such as: labor productivity, construction equipment productivity and availability of construction materials. Infact, those uncertainty factors were interrelated. However, several existing time performance prediction model didn't consider the interrelated among uncertainty factors. This paper proposes a model to predict the time performance for building construction project using Belief Networks (BNs). BNs presented causal relationship among a set of uncertainty factors.To validate the model, two building construction projects in the city of Surabaya were used as a case study.The results showed that the proposed prediction model can predict accurately and it also provides the early warning for time delay of construction project.

Keywords: *building construction projects, time, prediction, probabilistic, uncertainty, Belief Networks.*

INTRODUCTION

In the implementation of construction projects, there are several projects that experienced delays in construction. According Proboyo (1999) the main causes of the time delay of the project for the contractor side, are mainly due to preparation of resources and the planning/scheduling of work. Based on research conducted by Odeyinka in Aibinu

& Jagboro (2002), seven of ten projects were surveyed that experienced delays in implementation. Generally, the uncertainty condition in construction projects is influenced by various factors, such as: the calculation of the work quantity, labor productivity, unit price and other factors. With uncertainty condition, the project time performance cannot be expressed deterministically.

Based on the initial survey carried out on the building construction projects in the city of Surabaya, it was found that the time delay for particular week was caused by the lack of materials supply such as concrete, weather conditions i.e. high rainfall which disrupted equipment and labor productivity, the increase in the materials price, unavailability of construction equipment, a change in the design and poor relationship between the consultant supervisor and contractor. The field phenomena requires measurement methods of project performance which take into account the relationship between the uncertainty factors affecting the project performance. These can improve the accuracy of performance prediction and provide early warning of delays in the construction project.

To accommodate time uncertainty of construction projects, this study use the BNs to predict the probability of time performance of construction project. BNs method is a probabilistic graphic that combines probability and graph theory to explain the variables that have a high degree of connectivity. In addition, BNs, also incorporates uncertainty via conditional probabilities and it is a right tools to provide proactive measures to stakeholders (Luu, et al, 2009).

LITERATURE REVIEW

Time Performance of Construction Project

In measuring project performance, quantitative and qualitative techniques can be used. The quantitative technique is much better than the quantitative one. Project time performance is the most important criteria in determining the success of a construction project measurement out of the five criteria for the success of the project (Khosravi & Afshari, 2011).

Based on the literature study, questionnaires and interviews (expert opinion), the most significant factor that affects the time performance on the implementation of the construction project by the contractor: 1) The increase of the

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materials price (Luu, et al, 2009; Marzouk& El-Rasas, 2013; Doloi, et al, 2012; Issa, 2012; Fugar& Agyakwah-Baah, 2010), 2) weather factors (Luu, et al, 2009; Marzouk& El-Rasas, 2013; Doloi, et al, 2012; Pourrostam & Ismail, 2011; Fugar& Agyakwah-Baah, 2010), 3) Labor availability (Marzouk& El-Rasas, 2013; Wong & Vimonsatit, 2012; Pourrostam & Ismail, 2011; Fugar& Agyakwah-Baah, 2010), 4) The accuracy of the construction method (Luu, et al, 2009; Doloi, et al, 2012; Pourrostam & Ismail, 2011; Fugar& Agyakwah-Baah, 2010), 5) Replacement of new workers (Andi & Wijaya,2003), 6) delayed of materials delivery (Marzouk& El-Rasas, 2013; Doloi, et al, 2012; Issa, 2012; Wong & Vimonsatit, 2012; Pourrostam & Ismail, 2011; Fugar& Agyakwah-Baah, 2010), 7) availability of construction materials (Luu, et al, 2009; Marzouk& El-Rasas, 2013; Wong & Vimonsatit, 2012; Pourrostam & Ismail, 2011; Fugar& Agyakwah-Baah, 2010), 8) availability of construction equipment (Luu, et al, 2009; Marzouk& El-Rasas, 2013; Pourrostam & Ismail, 2011; Fugar& Agyakwah-Baah; Andi & Wijaya,2003), 9) labor productivity (Marzouk& El-Rasas, 2013; Doloi, et al, 2012; Pourrostam & Ismail, 2011), 10) equipment productivity (Marzouk& El-Rasas, 2013)

Belief Networks (BNs)

Belief Network is a probabilistic graphical model that represents a set of variables and probabilistic freedom. In the modeling, a Bayesian network use Directed Acyclic Graph (DAG), where each node represents a variable and arc (edge) symbolizes the condition of dependence between variables. This dependence is measured by the conditional probability for each node with its parent node (Langseth & Portinale, 2007). BNs are based on conditional probability theory which was called Bayes' rule (Luu, et al, 2009). Bayes' rule can be expressed as follows:

$$P(B|A) = \frac{P(A|B) * P(B)}{P(A)} \qquad (1)$$

Where P(A) is the probability of A, P(A|B) is the probability of A given that B has occurred. Simple structure of Belief Network is presented in Fig 1.

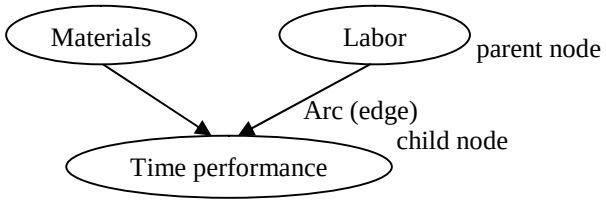

Fig. 1: A Simple Structure of Belief Network

In a Bayesian model, each node has a conditional probability table (CPT) associated with the node (Motidyang, 2007; Wong, et al, 2004). The conditional probability represent likelihoods based on prior information or past experiences (Sahely& Bagley, 2001). In each parent variable and possible state variable, there is a line in the CPT which represents the probability that a child node is composed of several state (McCabe, et al, 1998;Olmus & Erbas, 2004).The amount of probability distribution needed to fill the CPT grows exponentially with the number of parent nodes associated with the table. CPT is usually determined based on expert opinion and experimental data (Langseth & Portinale, 2007; Sahely& Bagley, 2001).

METHOD

Build Model with BNs

The model was built by the following steps: 1) Identify uncertainties that affected the time performance of construction projects, through the literature study, interview of construction experts, and questionnaires 2) Determine the uncertainty factor that most affects the project performance, through a questionnaire, literature study, interviews construction expert, 3) Determine the relationship between uncertainty, through a questionnaire with a dependency matrix, 4) Building a graphical structure models, 5) Calculate the conditional probability value for each factor in the model.

Calculating Prediction of Project Time Performance

Steps to calculate performance prediction of project time are 1) Determine the probability of the factors that affect the project performance through prior input and fill CPT with secondary data, questionnaires and interviews construction expert, 2) Calculating the probability prediction of project time performance from time to time, Bayes

software, 3) Calculating prediction of project time performance, by prior calculating the Expected (E) value. The formula used to calculate prediction of project time performance, namely (Fahirah et al, 2015):

$$E_n\ (\%) = \text{BNs probability}_n \times \text{Plan}_n(\%) \quad (2)$$

$$\text{BNs Prediction}_n\ (\%) = \text{Plan}_n\ (\%) \pm E_n\ (\%) \quad (3)$$

n = time period (weeks)

Calculating Accuracy of Prediction Model

The accuracy of prediction models of project time performance were calculated using statistical formulas Absolute Percentage Error (APE).

$$APE = \left|\frac{yt - \hat{y}t}{yt}\right| x\ 100 \quad (4)$$

where $\hat{y}_t$ = prediction value in week t,

y_t = actual value in week t,

t = prediction period (weeks).

If the APE value ≤ 30, the accurate prediction models to be used (Makridakis,et al, 1998). Prediction method is presented in Fig 2 (Fahirah et al, 2015).

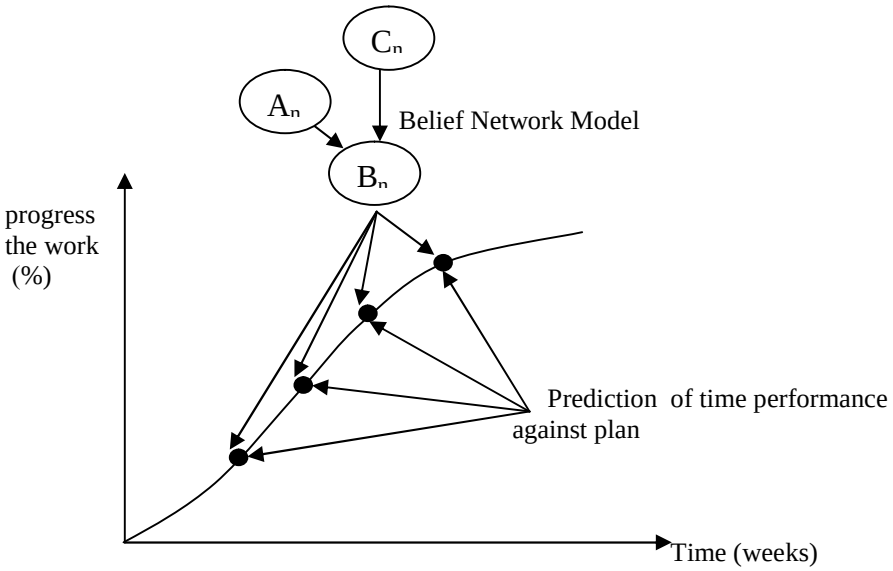

Fig. 2: Prediction process of project time performance using the BNs model

RESULTS AND DISCUSSION

Model Structure

Prediction models structures are built based on the literature study, questionnaire, interview of construction expert which can be presented in Fig 3. Based on the prediction model the factors that directly affect the performance of construction project are labor productivity, the productivity of construction equipment and the availability of construction materials.

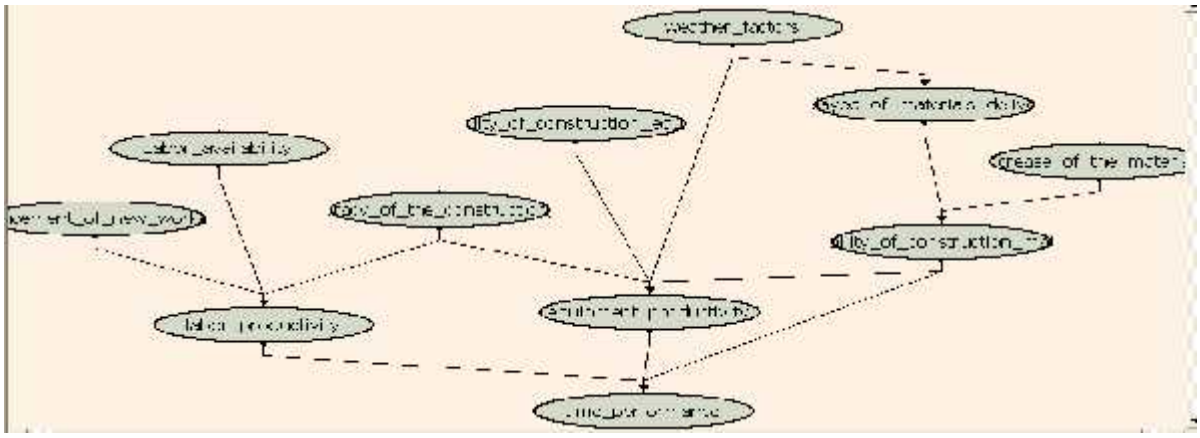

Fig. 3: Structural model of the relationship between the factors affecting the time performance of construction project

Implementation of Predictions Model

The model is implemented into case studies building X and Y of construction project as a model validation in prediction of construction project performance. A brief description of the building construction project is presented in Table 1.

Table 1 A brief description of the construction project

Description	Project X	Project Y
Project Type	Building	Building
Project Owner	Government	Government
Project Scope	12 floors	11 floors + a rooftop
Period of implementation based contract	27 weeks	17 weeks
Physical Contract Value	IDR. 58.580.000.000,-	IDR. 32.705.654.000,-

Source: Secondary Data, 2015

Stage of Belief Networks

Based on the relationship between the factors that affect performance in Fig 3, each node factor (prior) and CPT filled with a probability value based on primary data (questionnaire) and secondary data. Example of data filling of probability (prior) to factor accuracy of construction method with state "proper = 0.8" and "improper = 0.2". The probabilities results of the time performance simulation are presented in Fig 4 and 5.

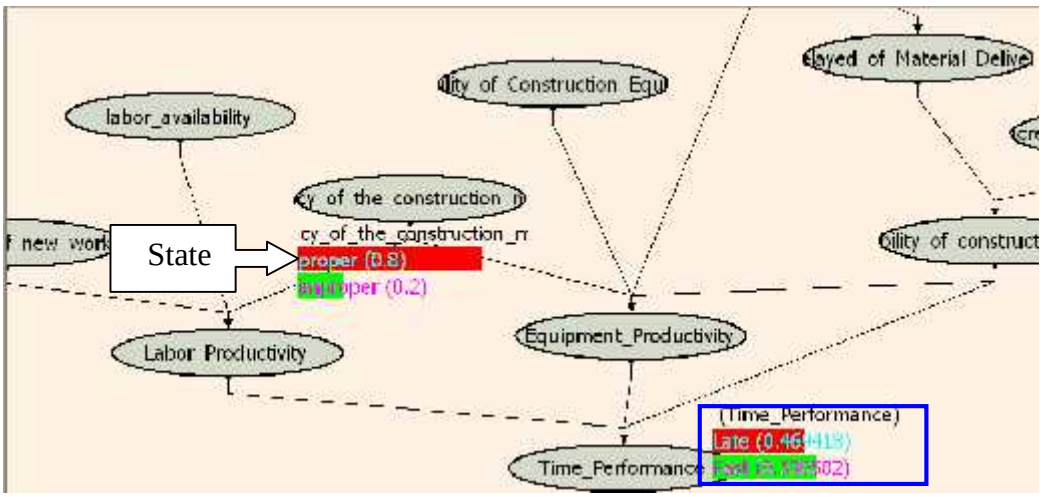

Fig. 4 Filling probability (prior) factor state in the accuracy of the construction method in Construction Project of Building X using the Software Belief Network (Source: primary and secondary data, data analysis, 2015)

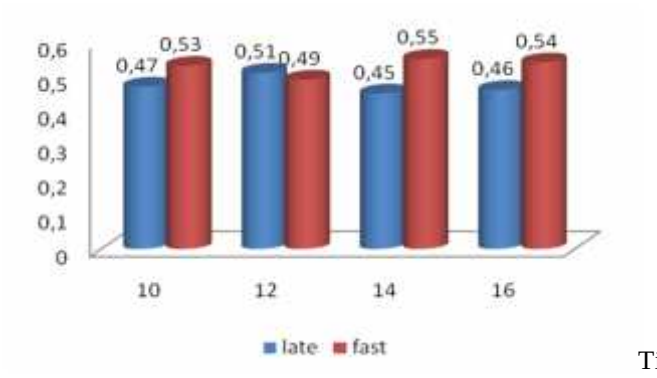

Fig. 5 Probability Results of Time Performance of Construction Project of Building X
(Source: Data Analysis, 2015)

Stage of Time Performance Prediction of Construction Project

Based on the results of the output probability BN presented in Figure 5, the next step is to calculate the predicted value for 10 to 16 week by using the formula (2) and (3). Time performance prediction calculation results in an "S" curve for a building construction project is presented in Fig 6.

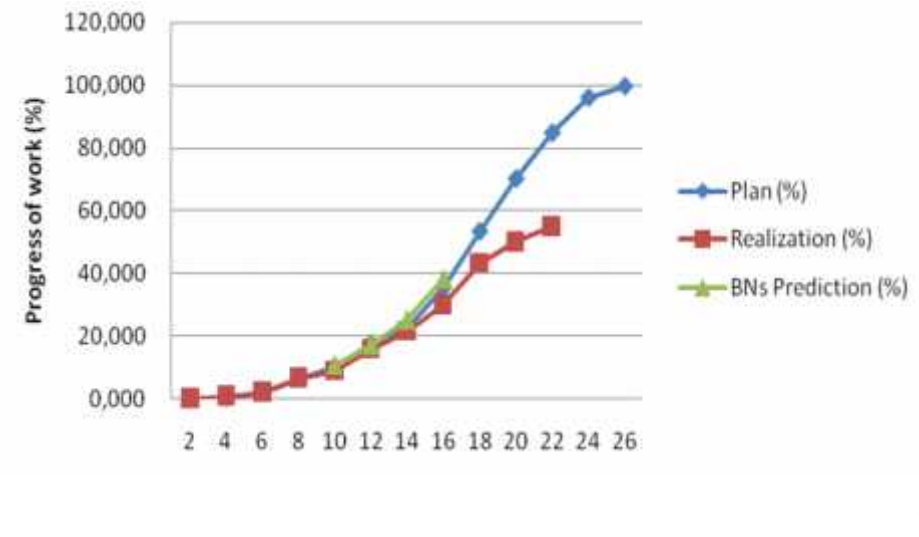

Fig. 6 Predicted Results of Time Performance of Construction Project Building X with "S" curve
(Source: Data Analysis, 2015)

Accuracy of Prediction Model

Accuracy measurement of the model can be calculated using statistical formulas of APE, eqn (4). The validation results of the model is accurate/feasible because APE values obtained for the two construction projects meet the eligibility standards/accuracy APE value ≤ 30. The complete value of the average APE for both building construction projects are presented in table 2.

Table 2 Results of the average APE for Projects of Building X and Y

Time (Weeks)	Project of Building X	Project of Building Y	*APE* mean
10	20,039	7,889	13,964
12	7,041	3,361	5,201
14	16,062	4,925	10,494
16	26,896	20,744	23,820
APE mean	17,510	9,230	13,370

Source: Data Analysis, 2015

CONCLUSION

Factors that directly affecting the performance of the building construction project were labor productivity, the productivity of construction equipment and the availability of construction materials.

Relationship model between the factors affecting the performance of a construction project using the BNs can predict time performance of building construction project accurately or feasibly, therefore the models built with BNs can be used as an alternative measurement of the construction project performance that can provide early warning and minimize the time delay of the building construction project.

For further research, to increase the accuracy of the model it is important to add or subtract other variables that affect the time performance of construction projects, and add or subtract or even change the relationship model between the uncertainty factors.

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PROJECTASSESSMENTS AND DESIGNS MONITORING AND EVALUTION (M&E) EMERGENCY PROCUREMENT IN CENTRAL SULAWESI

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ABSTRACT: Monitoring and evaluation project procurement emergency in natural disasters can be even more difficult for disaster-related projects in Central Sulawesi: project assessments and designs monitoring emergency procurement, quality standar are often not established, and the necessary data might be hard to collect. But good monitoring and evaluation not only improve project outcomes for stakeholders. This will usually be when there is no further threat to loss of life, damage to major infrastructure or when a state of emergency is lifted. goverment are permitted to forgo routine procurement procedures where it is justifiable on the grounds that the usual procedures would prevent the agency delivering the goods or services in time to bring effective relief Identifying a control group is challenging under ordinary circumstances; for a post-disaster project, it may be considerably harder. This may be due to the fact, for instance, that the project takes place only in a specific location (where the disaster occurred), that those who participate have special characteristics. This paper selecting the control group can be accomplished using methodologies that fall into two broad categories: experimental designs (randomized) and quasi-experimental design (nonrandomized).estimate of the counterfactual by using random assignment to create a control group (experimental design),to control for pre- and post-program procurement and development in participants, and to establish program impacts, relevant data collected at baseline and follow-up (including sufficient time frame to allow for program impacts);cost-benefit or cost effectiveness analysis to measure project efficiency; and qualitative techniques to allow for the triangulation of findings. The results showed there were differences between districts in the province of Central Sulawesi in enforcing quality of procurement of goods and services , which are caused by the quality of personnel , understanding of the regulation of procurement , training systems , Correspondence between the procurement procurement priorities , the data discrepancy with the applicable standards and regulations and inaccuracy choose contractor.

Keywords: *Tender, Evaluation, Monitoring. mitigation, construction*

INTRODUCTION

In the project portfolio organization found three different kinds of projects: project compliance and emergencies (to be done), the project operational, and strategic projects. Project compliance are generally projects that need to meet the requirements necessary to be operating in a region, because that project to be done. Emergency projects, such as building infrastructure, building construction after the disaster hit, must meet the criteria in the fulfillment of an emergency project (Clifford, 2007).

The construction project is a series of activities to produce a physical facility (building) by using the available resources, namely Man (Labor), Machines (Equipment), Materials, Methods, Money (Fund), which took place in certain period of time with the design and specifications previously agreed. The success of a construction project, is determined by management's ability to process the available resources by choosing the proper construction service providers according to regulations in force (Sumargono,2004).

Often the selection of appropriate methods of procurement that is not going to cause serious problems in the implementation of construction projects. Selection of contractors who conducted openly often makes the selection process to be too long, but with the chosen method of direct appointment would cause other problems in the aspects of the audit. Similarly, if the price fixed price contract will make a substantial risk to the contractor is selected but the condition of the contract cost plus the owners will feel uncomfortable (Gunawan, 2007).

Competition auction model with both open and limited most widely used to choose service users primarily for public projects (Mangitung,2010). Users and providers of goods and services need to understand construction and have the same perception in implementing regulations relating to procurement of goods and services especially construction construction which substantially emergency or disaster mitigation based.

Based on the basic thoughts of the background and issues that have been raised, then this study is expected to answer and reveal a model of evaluation and monitoring of procurement of goods and services that are more specific is limited to the procurement of goods and services based on the construction of the building as a result of the disaster mitigation flood with a scope in Central Sulawesi

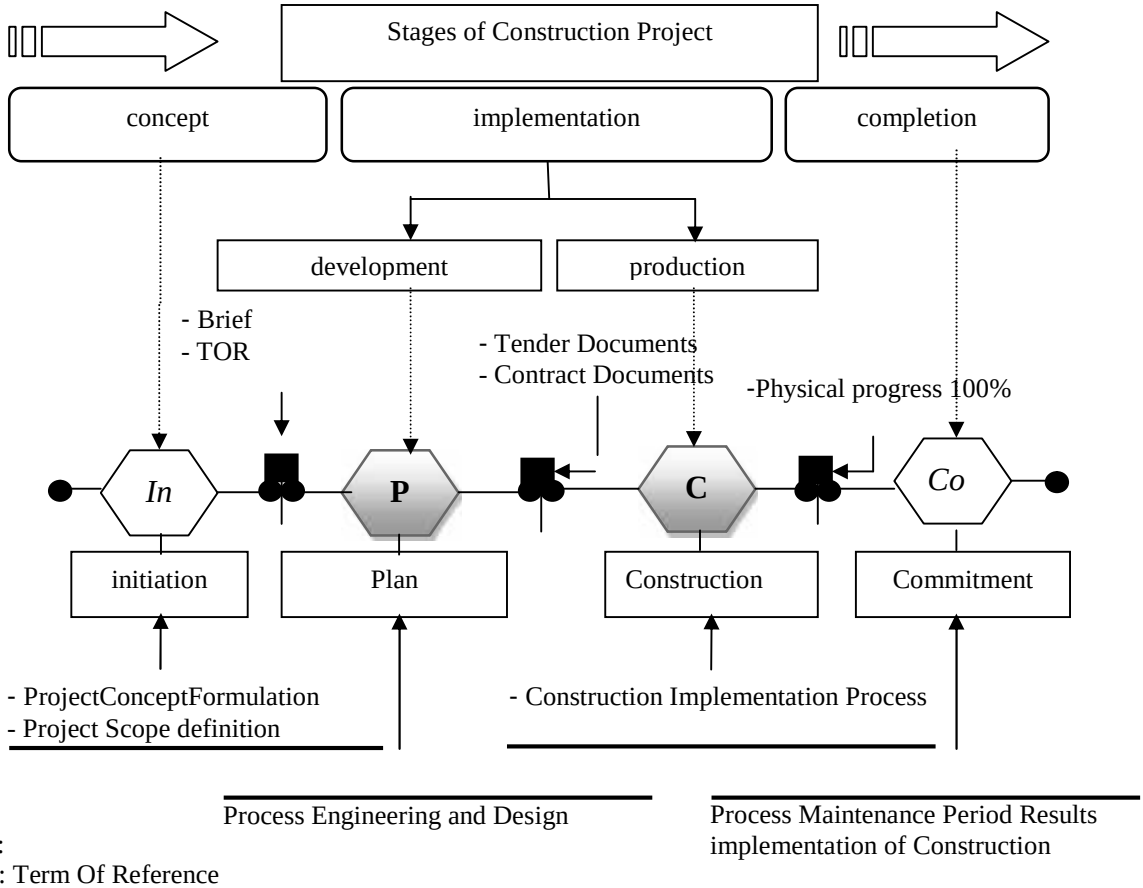
For more direct research to be carried out, then formulated in the form of research questions to guide and basic studies in this research, namely: :

a. How obstacles and irregularities that often occur in the standard implementation of the procurement of goods and services to the building construction mitigation based implementation of the application process begin the process of planning, preparation and the price index contract stage
b. What factors are considered in each subsystem enforcement of procurement of goods and services the construction of the building with the implementation of mitigation based application process
c. How themonitoringandevaluation ofthe implementation ofthe modelregulationson procurementof goodsandservicesthe construction ofbuildingswiththe implementation ofmitigationbasedapplicationprocess.

THEORY

Variety of construction procurement methods can not be released to the context of the stages of construction projects that each stage has its unique characteristics in its management, so that the phases of estimation and management requires a combination of various components of resources in order to Achieve success , these components are expertise *competences,tool/techiques*, *fund sources*. (Mangitung,2010)

More Clearly stages Construction Management Services can generally be assumed that the three (3) main stage. The stages consist of initiation, implementation and completion. In the implementation phase there are two (2) phases namely, the stage of development and production phases,(*Potter*) 1995, Morris 2005, Mangitung 2010)



Note:
TOR: Term Of Reference

Image 2.1. Stages of Construction Project (Mangitung,2010)

The Basic Principles contained in the Presidential Regulation 70 of 2012, Article 3, among others:

a. Efficient, means the procurement of goods / services to be undertaken using funds and limited resources to achieve the goals set in the shortest possible time and be accountable;
b. Effectively, it means that the procurement of goods / services must comply with the requirements that have been established and can provide maximum benefits in accordance with the target set;

c. Open and competitive, meaning the procurement of goods / services should be open to the provider of goods / services that meet the requirements and carried through healthy competition among providers of goods / services that are equivalent and qualified / specific criteria based on the rules and procedures are clear and transparent
d. Transparent, means all the provisions and information on procurement of goods / services, including administration of procurement technical requirements, evaluation procedures, the results of the evaluation, determination of prospective suppliers of goods / services, are open to participants of goods / service providers who are interested as well as for society at large;
e. Fair / non-discriminatory, mean giving equal treatment to all prospective suppliers of goods / services and do not lead to benefit a particular party, or the manner and for any reason;
f. Accountable, meansmustreach the targetof physical, financial andbenefits forthe convenience of thepublicadministrationandpublic servicesin accordancewiththe principlesand regulations oftheprocurement of goods/services.

Definition and Construction Procurement Process

The majority of contracts for construction work to let through a competitive bidding when the design is sufficiently advanced to tender for compilation (Thomas,dan Cheung 2004). In Many construction industry, a sizeable proportion of the business through bidding (Liu,Wang,Lai,2003)

Procument according to the Oxford dictionary consice Electronic Dictionary Version 9 comes from the verb meaning Procure gets to Obtain effort by effort or manage to Obtain manage to get a shutter that is certainly in the context of the construction industry, can be interpreted manage with an effort to produce products such as buildings or infrastructure building, Road, building water, etc. (Mangitung, 2010)

The auctionisthe definition ofa seriesof activitiestoprovidegoods/servicesby creatinghealthy competitionamongproviders of goods/services that areequivalentand qualified, based onspecificmethods andprocedures that have beenappliedandfollowedby therelatedpartiesareabidingprinciplethatelectedproviderbest. (Ervianto, 2005).

while (Masterman,2002) define the procurement of the following services, *Procument system is the organisational structure adopted by client for the implementation, and at times eventual operation of the a project.*

The system can also be carried out negotiations for the completion of the project if the public project shall be implemented by direct appointment, because it is urgent. Infrastructure for instance due to natural disasters. Further selection service providers either through negotiation or competition generally begins with the selection based on competency assessment that belongs to the service provider, service provider evaluation known through pre-qualification system.(Mangitung,2010)

Definition of Mitigation

Many of the meaning or definition of the disaster, which generally reflects the characteristics of the interference pattern of human life, the impact on the structure sosioal, damage to the aspects of the system of government, and other buildings as well as the needs of society as a result of the disaster.

W Nick Carter in his book "Disaster Management" provides a definition of disaster by Concise Oxford Dictionary as " sudden or great misfortune, calamity whereas by Webster Dictionary, a disaster is defined as *"sudden calamitous event producing great material demage, loss, and distress"Definition of a disaster or disaster according to Wikipedia*: *disaster is the impact of a natural or man-made hazards that negatively effects society or environment*. (Nurjanah,2011)

Definition of Disaster in Law Number 2007 paragraph of article 1 paragraph 1 is an event or series of events that threaten and disrupt the lives and livelihoods caused by both natural factors and / or non-natural and human factors that result in human fatalities, environmental damage, loss of property, and the psychological impact (UU 24/2007)

The basic concepts of monitoring and evaluation

Basically, monitoring and evaluation is an effort to determine whether an activity or project implemented in a manner consistent with the plan or program has been set. Practically, monitoring and evaluation is needed to determine whether the resources have been used appropriately and according to plan, whether the activity of the process carried out in accordance with the manner required, and whether the planned targets or targets can be achieved. Therefore, monitoring and evaluation becomes a very important position in the implementation of a plan for several reasons, among others, to determine the extent to which a program can be implemented and what interventions are needed to increase the impact of the program.

Monitoring and evaluation should be done based on destination . Success of a monitoring and evaluation based on the ability to assess the success and failure of specific management objectives . Therefore , management objectives or activities should be made realistic , specific , measurable and clearly written . The purpose of monitoring and evaluation

can also be affected by the final goal of the activity , the need to obtain complete data , monitoring methods and the availability of resources . Well- defined goals will strengthen the purpose and efficiency of the monitoring program . Furthermore , well-defined objectives that will provide a yardstick or benchmark for measuring the effectiveness of management activities . Monitoring and evaluation activities can form the direct supervision of an activity by a person who is responsible . This surveillance can be performed using format control to the entire process with a sampling approach(Bennett, 2003).

METHODS AND ANALYSIS

Research Design

The method is based on observations carried out research on the views of service users and service providers in Central Sulawesi and the surrounding areas relating to the procurement of construction services especially construction building-based mitigation. Descriptive statistical analyzes were performed with the program package was applied from decision making. Research methods outline basic three important parts (Boediono, 2008), namely: (i) the research procedures, (ii) research techniques, and (iii) research methods. In this regard, the research methods used in this study were divided into seven (7) phases of activity

1th Phase is a preparatory stage, which consists of: (i) the observation of facts to examine the quality of the performance problems of procurement of goods and services based mitigation based on data from national and provincial institutional and relevant research results; (ii) to formulate the problem and research objectives; (iii) carry out regularly scheduled seminars; and (iv) developing the research instrument.

2th Phase is a literature review stage, which consists of: (i) reviewing the results of other relevant studies; (ii) identify the factors, variables and parameters as the indicators within each sub-system of the implementation of quality standards; (iii) develop a framework of logic-based models of hierarchical systemic and comprehensive; (iv) carry out pilot surveys

3th Phase is the implementation of the stages of identification performance quality standards-based procurement mitigas, which consists of: (i) determination of the price index; (ii) identification of difficulties and understand the quality standards; (iii) identification of constraints and implementation of quality standard deviation

4th Phase, is the identification and verification stage consists of: (i) verification of the implementation of quality standards factor; (ii) verification of indicator variables and parameters as well as the factors influencing the implementation of quality standards; (iii) carry out surveys to-2 to collect expert opinion to verify the influence of variables; (iv) carry out the variables indicative of the dominant factors affecting the implementation of quality standards.

5th Phase, is the stage of selection and grouping variables, which consists of: (i) carry out surveys to-3 to collect expert opinion to verify the degree of influence of variables enforcement of quality standards; (ii) validation of the grouping variable with factor analysis (factor analysis); (iii) formulating indicators and parameters for measuring the inclination of logic variables influence the enforcement of quality standards factor

6th Phaseis the stage ofweightingfactorsandvariables, consisting of: (i) carry outsurveystocollectall4expert opinionon the level ofinterestcomparisonsbetween variablestofactoramongthe factorstobe reviewedand theapplication ofquality standardssystem

7th Phaseis the stage ofmakinga modelof monitoringandevaluation, whichconsists of: (i) formulatelogicmodels; (ii) establish the methodof measurementmonitoirngindicatorvariable; (iii) formulatean evaluationmethodforassessing theapplication ofratingvariablestofactorquality standards; (iv) create amodel ofmonitoring andevaluationin the form ofsoftwarewithDelphiprogramminglanguage; (v) the applicationandverification ofthe modelonseveralproject-based procurementmitigation; (vi) develop a programimprovementandrefinement ofsolutionsforthe implementation ofquality standards; and(vii) and thefinalization ofthe model

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Research Instruments

Monitoring and evaluation of the implementation of quality standards-based procurement mitigation conducted with a systemic approach (input-process-output-outcome-impact) factors that need to be specified, along with indicator variables and parameters in each system parts (subsystems) enforcement. Factor in this study is intended circumstances or events that influence the occurrence of something or something that actively contribute to a settlement, the results and processes. The variable in this study is intended as something that has a variation or something that can change that reflects the character of the factors. The indicators referred to in this study something that can be used as a tool to measure, provide guidance and information on the variable. The parameters defined in this study is a measure that limits or performance variable.

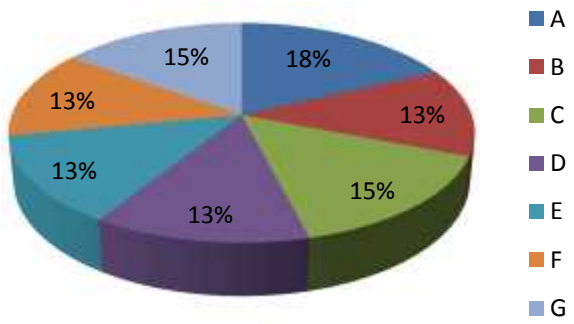
RESULTS AND DISCUSSION

Constraints implementation of quality standards in the procurement of goods and services based construction building mitigation

Starting from the difficulty of understanding the substance of procurement quality standards are exacerbated by the lack of socialization conducted an advisory institution and defective access to information services, then the implementation is always faced with the constraints of implementation and procurement quality standard deviations (Jessica S and Søreide T, (2006) . as noted by Jessica S and Søreide T, (2006) that developing countries (including Indonesia) still face many obstacles in implementing proper procurement quality standards and true in the field, such as limited human resources, local policy is very influential, achievement of cost limitations procurement and purchasing quality facilities and infrastructure.The same thing also expressed by Smith N (2008) and Morgan J, Bowles G, K & Carter (2012) that the dissemination and implementation of quality standards is determined workability procurement standard book itself, the quality of human resources that supported the reliability of procurement and quality monitoring tools availability of service providers (contractors) which meets the technical requirements as well as coordination among components of the organization in the field. Morgan S, Fisher S, & Herrmann J (2013), Mutava C, (2014); (Jessica S and Søreide T, (2006), stated in a technical study to evaluate the achievement of quality system implementation of procurement of goods and services contingency.mentioned some of the obstacles or challenges to achieve the quality of the procurement of goods and services are: (i) the limited quality of human resources (ii) the substance of procurement elusive quality standards and applied; (Iii) very dominant influence local policies; (iv) a disaster mitigation project service users are less precise; (v) no internal coordination service users on the quality management of procurement; and (vi) lack of service providers (contractors) qualified and (vii) lack of availability of the institution as a controller monitoring and evaluation of the quality of procurement. Another expert opinion completes the statement once declared by Jessica S and Søreide T, (2006) .; Morgan S, Fisher S, & Herrmann J (2013) and Sri EW, Wijaya K & Martoyo (2013) about the obstacles to implementation of quality systems of procurement, are: (i) coordination less (ii) the low participation of providers (iii) lack of monitoring and evaluation (iii) sanctions are not applied inconsistent.

Based on the results of a study of 60 respondents on the implementation of quality standards of procurement constraints on disaster mitigation projects, as shown in Figure 1, found several problems that often occur in the field, are: (i) the limited quality of human resources (16% of respondents); (Ii) the substance of procurement elusive quality standards and applied (15% of respondents); (Iii) influence local policy is dominant (15% of respondents); (Iv) a disaster mitigation project service users are less precise (13% of respondents); (V) no internal coordination service users on the quality of procurement management (13% of respondents); (Vi) lack of service providers (contractors) qualified (12% of respondents); and (vii) lack of availability of the institution as a controller monitoring and evaluation of the quality of procurement (10% of respondents). Results of these studies strongly support the opinion N Smith (2008) and Morgan J, Bowles G, K & Carter (2012) which states there are 4 (four) factors affecting the implementation of input procurement quality standards, namely human resources, procurement quality monitoring tools, quality Emergency tendering procedures and materials procurement quality standards. If the observed distribution of respondents per district, the analysis results obtained showed almost 100% of respondents in Toli-Toli declare a major obstacle in achieving the quality of procurement is the limited quality of my human resources, procurement quality control, monitoring and evaluation of the limited reliability and quality of the provision of independent, as well as nearly 75% of respondents respectively in Morowali, Tojo Una-Una and Toli-Toli says the same thing with the respondents in Work Units (SKPD) Central Sulawesi.

The condition was inversely related to the conditions in Moutong Parigi district, Donggala which showed that only about 40% of the respondents believe the same thing with the other districts, as shown in Figure 1. This phenomenon illustrates that the magnitude of the gap between stakeholders faced the Procurement of Goods and Services in addressing obstacles or challenges the implementation of quality standards of procurement between Moutong Parigi district, Donggala with another district. These problems stem from differences in the quantity and quality of available resources (human, tool monitoring, service providers (contractors), institutional and budget), so that the central government as a builder should make efforts to monitor and evaluate the implementation of the procurement quality standards in order to obtain uniformity of quality procurement of goods and services and disaster mitigation project policy perfection procurement proportionate.



limitations the quality of human resources
the substance of procurement elusive quality
standards and are applied

very dominant influence local policies

a disaster mitigation project service users are less precise
no internal coordination service users to

quality management procurement;

lack of service providers (contractors) qualified

lack of availability of the institution as a controller
monitoring and evaluation of the quality of procurement.

Figure 1. Constraints implementation of quality standards in the procurement of goods and services based construction building mitigation

The study of 60 respondents (experts Procurement) on the implementation constraints on increasing quality standards of procurement Procurement as shown in Figure 4.27, found several problems that often occur in the field, are: (i) the limitations of the human resources quality control of procurement quality (35.5% of respondents); (ii) lack of accountability monitoring quality control of procurement (29.6% of respondents); (iii) lack of quality service providers (contractors) (12.0% of respondents); (iv) difficulty understanding the substance of procurement quality standards (11.4% of respondents); and (v) the lack of coordination between the service user and service provider quality management procurement (9.3% of respondents).

If the observed distribution of respondents per district, the results of the analysis showed nearly 90% of respondents in Toli-Toli declare a major obstacle in achieving quality Procurement is the limited quality of human resources quality control of procurement, the limited number and reliability monitoring tools and limited monitoring audit institutions quality independent procurement; as well as nearly 80% of respondents respectively in Morowali, Tojo Una-Una and Toli-Toli says the same thing with the respondents in Toli-Toli. The phenomenon is quite different from the situation in Moutong Parigi district, Donggala which showed around 65% of respondents believe the same thing with the other districts, From the description of this discussion it can be concluded that the implementation of quality constraints dominant on improving procurement Procurement is the limited quality of human resources low accountability and quality control monitoring of procurement. When compared with the procurement of goods and services based construction building mitigation, increased Procurement more demanding precision to maintain the strength of the existing structural and functional to the increased traffic load repetitions and influence runoff water volume due to the growth of the function of surrounding space and budget constraints, so requires improving the quality of human resources and support accountability procurement quality monitoring tools (CPPR, 2012).

Based on the results of a study of 60 respondents (experts Procurement) on the implementation constraints on the maintenance of quality standards of procurement Procurement as shown in Figure 1, found several problems that often occur in the field, are: (i) lack of accountability monitoring quality controllers procurement (39.3% of respondents); (Ii) limitations of the quality of service providers (contractors) (16.1% of respondents); (Iii) lack of quality human resources quality control of procurement (13.2% of respondents); (Iv) the lack of coordination between the service user and service provider quality management procurement (12.7% of respondents); and (v) difficulty understanding the substance of procurement quality standards (11.3% of respondents). If the observed distribution of respondents per district, the results of the analysis showed nearly 75% of respondents in Toli-Toli declare a major obstacle in achieving the procurement of quality goods and services-based building construction mitigation is limited quality service providers (contractors) and accountability monitoring and evaluation (the amount, reliability and quality monitoring

audit institution independent procurement); as well as nearly 60% of respondents respectively in Morowali, Tojo Una-Una and Toli-Toli says the same thing with the respondents in Toli-Toli. The phenomenon is quite different from the situation in Moutong Parigi district, Donggala which showed around 35% of respondents believe the same thing with the other districts, as shown in Figure 4.30. From the description of this discussion it can be concluded that the implementation of quality constraints on the maintenance of the dominant procurement Procurement is limited accountability for monitoring and evaluation of quality control and quality limitations procurement service providers (contractors) are used. Procurement of building construction quality based mitigation requires the use of appropriate materials procurement stacking so long can still be maintained until it reaches the maximum service life (CPPR, 2012).

implementation of quality standards Procurement	Perception of Respondents (%)		
	Sourcesofbaseline	Monitoring	Evaluation
• Limitations of Human Resource Quality	15,7	35,5	13,2
• limited funding	37,6	29,6	39,3
• limited time service users	12,0	8,1	16,1
• substanceprocurement quality standards	13,4	11,4	11,3
• Procurement priorities	15,1	9,3	12,7
• Risks and Challenges	6,4	6,1	7,4
Total	100,0	100,0	100,0

Table .1 identification of constraints on the implementation of the procurement quality standards Disaster Mitigation Project,

Aberration achievement of the quality of procurement of goods and services

Technical study ever conducted by the CPPR MEP UGM (2012) on the performance quality of the goods and services mentioned that there are 6 (six) deviation implementation of procurement quality achievement, namely: (i) Intervention by certain parties (ii) inadequate regulation (iii) Not all personal / unit want on the monitor (iv) absence of further implications of the results of monitoring and evaluation (v) the performance appraisal system is not adequate (vi) Low reward or punishment standards accepted obscurity compared to the achievement of the results obtained

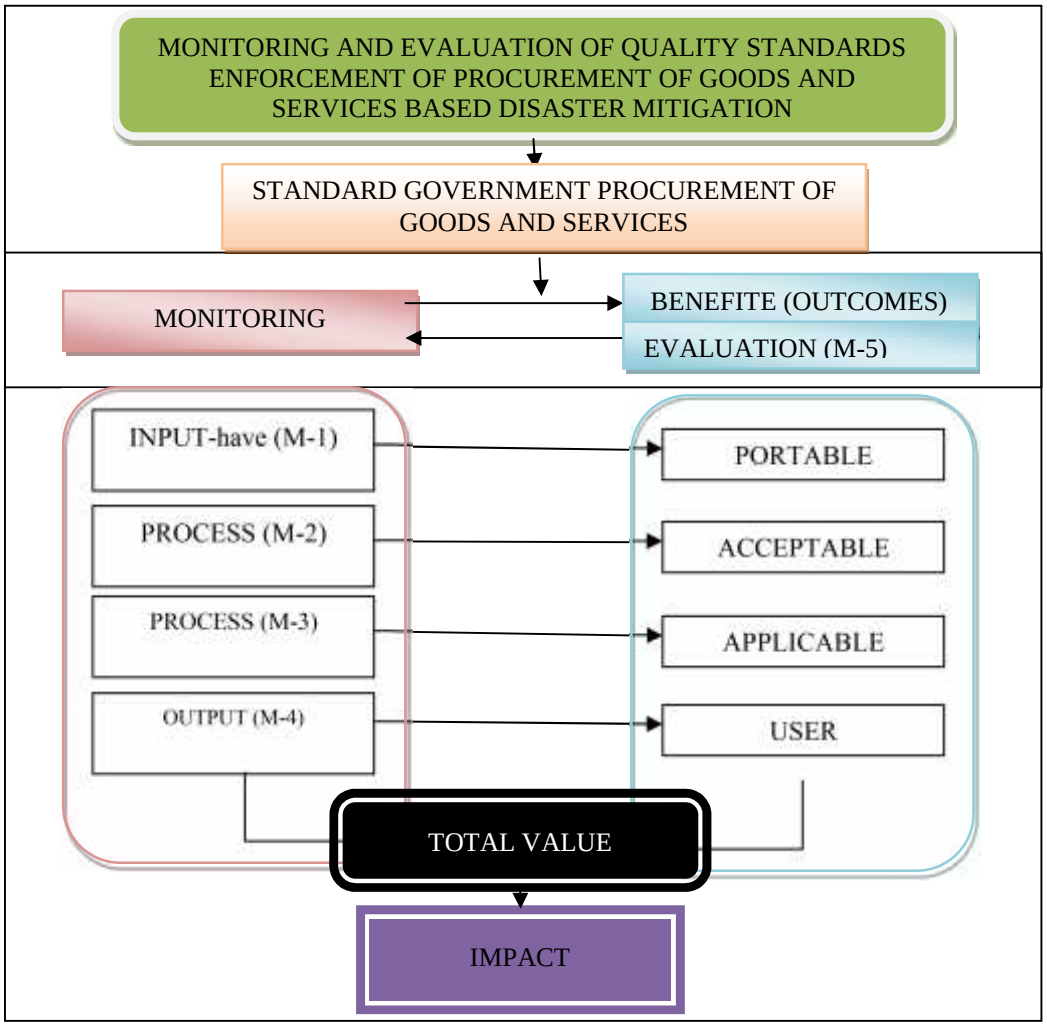


Figure 1. Concept of Monitoring and Evaluation Model Entry Standard Procurement Documents Based Mitigation

Implementation of the model is done to monitor and evaluate the implementation of quality standards of goods and services based on disaster mitigation. Implementation of the model starts with the interview survey respondents who have specific experience in the implementation of the procurement of goods / services based disaster mitigation, which supported the minimum educational civil engineer with a background in career services are appropriate and adequate, and comes from: BPPD office (Regional Natural Disaster Management Agency), the Office of Public Works, the Office of Human Settlements, the Office of Water Resources, and the Office of Procurement Services Unit. At every institution is taken 1 to 2 respondents because the respondents will be divided equally follow a normal distribution according to the selected study sites. In this study, been some research location that is central Sulawesi and 6 (six) other places are districts in Central Sulawesi (Sigi regency, Donggala, Parigi Moutong, Tojo Una-Una, Toli-Toli, and Morowali). The choice of location is based on a purposive sampling method, where the election because a particular purpose. So the number of respondents is 60 people who have competence in the application of the procurement of goods / services

Table 2, Evaluation Category Entry Standard Procurement Documents Based Disaster Mitigation

Value (%)	Monitoring Category	Evaluation Category			
		Portable	*Acceptable*	*Applicable*	*User Friendly*
<25	Not	Not	Not	Not	Not
25 – 50	Less	Less	Less	Less	Less
50 – 75	Enough	Enough	Enough	Enough	Enough
>75	Good	Good	Good	Good	Good

Table 3 Results of Monitoring and Evaluation Standards Enforcement Procurement Document-Based Disaster Mitigation

No.	Monitoring and Evaluation	BPPD	Office CK	Office BM	Office SDA	ULP	Mean
1.	Have (M-1)	6,00%	3,60%	4,80%	4,90%	5,63%	4,91%
	a. How to recognize	1,18%	0,63%	0,70%	0,88%	1,00%	0,85%
	b. how to get	2,50%	1,63%	3,13%	2,00%	3,13%	2,40%
	c. How to archive	2,32%	1,35%	0,97%	2,02%	1,50%	1,56%
2.	Understanding (M-2)	15,00%	13,88%	14,00%	13,88%	11,63%	13,63%
	a. understanding of the	3,75%	3,75%	2,75%	2,25%	3,00%	3,04%
	language	11,25%	10,13%	11,25%	11,63%	8,63%	10,59%
	b. understanding of substance						
3.	Applying (M-3)						
	a. physical documents	42,81%	35,56%	31,94%	36,56%	31,56%	35,47%
	b. executive competence	3,24%	3,75%	1,25%	2,49%	2,50%	2,49%
	c. data management	10,31%	10,31%	4,68%	4,69%	8,43%	6,30%
	d. utilization tool	3,75%	2,50%	2,38%	4,12%	2,50%	2,97%
	e. utilization of materials	8,88%	7,25%	6,88%	8,88%	5,63%	7,40%
	f. methods of implementation	9,13%	5,00%	10,00%	9,13%	5,00%	7,31%
		7,50%	6,75%	6,75%	7,25%	7,50%	7,14%
4.	Supervise (M-4)	11,63%	11,25%	13,88%	7,63%	11,00%	10,88%
	a. achievement uniformity	4,13%	3,75%	4,38%	2,63%	4,00%	3,73%
	b. Achievement of quality assurance	7,50%	7,50%	9,50%	5,00%	7,00%	7,15%
	Value Monitoring	75,44%	64,29%	64,61%	62,96%	59,81%	65,22%
5.	Evaluating (M-5)						
	a. Portable						
	b. Acceptable	60,00%	36,00%	48,00%	49,00%	56,25%	49,11%
	c. Applicable	75,00%	69,38%	70,00%	69,38%	58,13%	68,14%
	d. User Friendly	77,84%	64,66%	58,07%	66,48%	57,39%	64,49%
		77,50%	75,00%	92,50%	50,83%	73,33%	72,51%

In general, monitoring results in the category of "good enough" with 59.81% which is the lowest in the Procurement Services Unit (ULP) and the highest on the Regional Disaster Management Agency (BPPD) by monitoring the value of 75.44%, followed by the Department of Resources Water Resources (SDA) (62.96%), the Department of Highways (BM) (64.61%), and the Department of Human Settlements (CK) (64.26%). This suggests that the Regional Disaster Management Agency (BPPD) and the Department of Human Settlements (CK) as the central unit in charge of managing the procurement of goods and services had been monitoring the implementation of the procurement documents properly in accordance duties and functions, while the Procurement Services Unit (ULP) better position ourselves as a procurement official controls the extent of the procurement documents may be owned, understood, and applied by field practitioners in the field of consultancy and the procurement of goods / services. Results if the data derived from the logic models show that the level of fairness of the analysis is quite good because it is not much different from the results of the monitoring assessment among stakeholders. Thus it can be said that the level of achievement of the reach of uniformity in the quality assurance of goods / services procurement of emergency in the region of Central Sulawesi and the surrounding areas in both categories

Evaluation of the implementation of the procurement documents reviewed in 4 (four) aspects of assessment, namely: (1) portable (2) acceptable (3) applicable and (4) user friendly. Regional Disaster Management Agency (BPPD) and Procurement Services Unit (ULP) assess fairly portable document pengadan (respectively above 50.0%, ie 60.0% and 56.25%) due to first get the book printed documents pengadan than Department of Human Settlements, Department of Highways and Department of Water Resources. The third unit is more dominant in the technical manuals rather than the procurement documents. In general, the fifth unit is working well enough to understand the language and the technical substance of the procurement documents as acceptable average value is 68.14%, which means that means that the content of the discussion document pengadan easy to learn and understand. Similarly applicable aspects and user friendly. Results of evaluation of these two aspects that are in good enough category, with each average is 64.49% and 72.51%. In general, the results of an evaluation of any aspect of being in good enough category (50.0% - 75.0%) except for portable aspect, which is located in the unfavorable category (25.0% - 50.0%). This indicates that the socialization of the procurement documents (how to identify, acquire, and archive) further

improved in all units of the relevant stakeholders penyelenggaan procurement of goods and services based on mitigation, including universities, so that in the future the community created a standard minded procurement of goods and services based mitigation. In addition it should be considered a book pengadan uniform documents so as not to give rise to many interpretations that make a gap for abuse of authority

CONCLUSIONS

Procurement statutes and rules usually are effective during the recovery period. While some supplies and services may meet provincial / district standards for emergency or urgency during the recovery phase, many do not. Personnel who have been working on the emergency may have grown accustomed to a procurement process that now must be adjusted to accommodate previously existing requirements for competition and compliance with other procurement policies.

Some things that can be concluded from the modeling logic monitoring and evaluation of the implementation of the procurement documents based on disaster mitigation (emergency) is as follows:

1. The logical model is built with a systemic approach, hierarchical, and comprehensive will facilitate in monitoring the factors and variables that affect the implementation dominant pengadan documents in the field.
2. The results of the monitoring and evaluation of the implementation of the procurement documents (procedures for the procurement of goods / services) based hazard mitigation (floods) in Central Sulawesi and the surrounding area, indicate that aspect acceptable, applicable, and user friendly is at good enough category (50.0% - 75.0%), but less good aspects of the portable category (25.0% - 50.0%). This indicates rbahwa socialization pengadan documents (way to know, acquire, and archive) further improved in all units procurement stakeholders, including universities, so that in the future created minded community standards in the procurement of goods / services construction based on disaster mitigation (flooding). In addition it should be considered to make the books pengadan documents in the form of a paperback book, so it is easy to carry to the field

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MANAGING RISK IN THE ROAD MAINTENANCE WORK OF PERFORMANCE BASED CONTRACT

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ABSTRACT: Risk Management has been one of the key success factors of any construction project success. A valuable output in managing the project risk is the risk response which determines the effectiveness of the risk management process. This work is a part of a larger research study for risk management under performance based contracts (PBC) for road maintenance work in Indonesia. These performance-based maintenance contracts transfer the long-term responsibility for design, construction and maintenance works from government agencies to contractors for a lump sum payment. However, this paper will focus on the choice of the risk response strategies by contractors under PBC. A survey questionnaires adopted and distributed to the contractors who have been involved in the pilot project on PBC throughout Indonesia. The findings of the study revealed that the tendency of the industry to accept most of the non-technical risks while trying to control and mitigate most of the technical risks where they can have more impact on the risk exposure.

Key words: *Contractors, risk management, risk response, performance based contracts, road maintenance.*

INTRODUCTION

Risk Management has been one of the key success factors of any construction project success. A valuable output in managing the project risk is the risk response which determines the effectiveness of the risk management process. PBC are considered a fairly new project delivery system that put the contractor to a longer term commitment than in other traditional contract that are widely used for road construction works (Gruneberg, Hughes & Ancell, 2007). As, PBC transfer the long-term responsibility for maintenance works from government agencies to contractors for a lump sum payment, it is clearly shifting increased risks on to the contractor and away from the client. Moreover, contractors often take on projects with high complex, variable and risky nature in terms of dollar value, weather conditions, public competition and other issue like traffic control. These long-term commitments can also extend beyond construction to the maintenance phase, sometimes committing the contractor to a warranty period to enforce the performance goals for the required maintenance period that is considered as the warranty period in the contract (FHWA, 2009). This raises important questions concerning the management of risks under PBC from a contractor's point of view.

This work is a part of a larger research study for risk management under performance based contracts (PBC) for road maintenance work in Indonesia under PBCs that includes the risk management process for contractors. The full research study covered all the risk management process including the risk planning, identification, assessment and analysis, response planning and control. This paper, however, will only focus on on the choice of the risk response strategies by contractors under PBC This was determined through several interviews including a shortlisted questionnaire for the associated risks where the respondents were asked to choose between several risk response strategies for each risk selected.

LITERATURE REVIEW

As this research discusses risks in the project context, a formal definition from "A Guide to the Project Management Body of Knowledge" is used (PMI 2013). Then risk is defined as "Project risk is an uncertain event or condition that, if it occurs, has a positive or negative effect on one or more project objectives such as scope, schedule, cost, and quality". Project Risk Management includes the processes of conducting risk management planning, identification, analysis, response planning, and controlling risk on a project. The objectives of project risk management are to increase the likelihood and impact of positive events, and decrease the likelihood and impact of negative events in the project. In recent years, there has been increased interest in framing the risk management process and adopting new and effective risk management tools and techniques for the construction industry in general and especially in road construction and maintenance projects.

The authors conducted a literature review of the risk management processes and risk response strategies to explore the different methods and techniques used in the construction industry in general and specifically in the road construction and maintenance works.

Risk Management Process

There is a plenty research on the risk management process, but to date, little empirical research work has been carried out on the management of risk under PBC. One of the most complete studies is by Williams (1995) that included 241 references about risk management and uncertainty where the oldest dated back to 1959. The PMBOK fifth edition (2013) defines the risk management process in six main phases: (1) risk management planning, (2) risk identification, (3) qualitative/quantitative risk analysis, (4) risk response planning, and (5) risk monitoring and (6) controlling risk, which are as follows:

1. Plan Risk Management—The process of defining how to conduct risk management activities for a project.
2. Identify Risks—The process of determining which risks may affect the project and documenting their characteristics.
3. Perform Qualitative Risk Analysis—The process of prioritizing risks for further analysis or action by assessing and combining their probability of occurrence and impact.
4. Perform Quantitative Risk Analysis—The process of numerically analyzing the effect of identified risks on overall project objectives.
5. Plan Risk Responses—The process of developing options and actions to enhance opportunities and to reduce threats to project objectives.
6. Control Risks—The process of implementing risk response plans, tracking identified risks, monitoring residual risks, identifying new risks, and evaluating risk process effectiveness throughout the project.

Others have a more detailed approach to the risk management process such as Chapman (1997) and then Chapman and Ward (2003) who break the risk management process into a nine-phase approach: (1) define, (2) focus, (3) plan, (4) structure, (5) ownership, (6) estimate, (7) evaluate, (8) plan, and (9) manage. Some have simplified the risk management process into only 3 phases: (1) risk identification, (2) risk analysis, and (3) risk response (Buchan, 1994). Ashley, et al.(2006) introduced a guide to risk assessment and allocation for highway construction management which adopted the Caltrans Project Risk Management Handbook. It consist of six primary steps in project risk management: (1) Identification, (2) Mitigation, (3) Assessment, (4) Allocation, (5) Analysis, and (6) Tracking and updating. Regardless of the different stages or the cycles of the risk management process, "identification and assessment will be worthless unless responses can be developed and implemented which really matter in addressing identified risks" (Hillson, 1999).

Risk Response Strategies

Risk responses is the process of developing options and actions to enhance opportunities and to reduce threats to project objectives (PMI, 2013). It can be considered one of the main risk management process outputs along with risk planning. This is what defines a successful risk management process implementation since it provides contractors with the backup plans, contingencies, and corrective actions to be taken in case of the risk occurrence or a preventive action to prevent the materialization of the risk in the first place. This depends on the adopted risk response strategy, whether to reduce the exposure of the risk or prevent it. It is worth noting that the risk response strategy is the larger aspect of the risk response stage, which leads to risk response techniques to be developed according to the project's complexity and conditions. Several authors have identified different risk response strategies. The PMBOK fifth edition (2013) listed the following strategies: (1) avoid (extending schedule, reducing scope, shutting down the project), (2) transfer (financial risk exposure, insurance, warranties, guarantees), (3) mitigate (taking early actions, adopting less complex processes, conduction of more tests, choosing more stable suppliers, prototyping, redundancy), or (4) accept (passive acceptance: no action except to document the strategy and leave it to the project team to deal with it; active acceptance: establishing contingency reserve for money, time, and resources). Hillson (1999) had a different simplified approach, also listing four different strategies without differentiating between active and passive acceptance: (1) avoid (seeking to eliminate uncertainty), (2) transfer (seeking to transfer ownership and/or liability to a third party), (3) mitigate (seeking to reduce the size of the risk exposure to below an acceptable threshold), or (4) accept (recognizing residual risks and devising responses to control and monitor them). No matter which response strategy is adopted, the aim is to adopt a proactive and effective manner to respond to the risk occurrence.

Risk Response Under Performance Based Contracts

A project delivery system is defined as a method for procurement by which the clients' transfer or share risks to other project entities. Risk response strategies can be very different from one delivery system to another depending on the risk exposure and the uncertainty associated with the project. The more uncertainty associated with the project, the more detailed and confrontational the response must be (Mills, 2001). Under PBCs, there are different factors that assert this fact and make the contractor change strategy based on the delivery system characteristics which are not limited to (Gruneberg et al., 2007; Hughes & Kabiri, 2013):

1. Longer commitment and liability period towards the project

2. More risks allocated towards the contractor
3. Longer time exposure to several risks including the maintenance period
4. Fitness for purpose, and associated insurance risks
5. Long-term cost increase and inflation through the performance period
6. Resource availability issues

In other hand, There are risks associated with PBC on client sides of the contract. For example, the risk regarding government ability to continue to pay led to a higher price being offered by contractors in one (unidentified) PBC projects in Brazil, to the extent that all bids were in excess of available funds (Zietlow 2005).

RESEARCH METHODOLOGY

In order to carry out an assessment of contractors' attitudes towards risk management of PBC, this research involved several activities that align with the risk management process including risk identification, risk assessment, risk analysis, and finally, the risk response which is the main focus for this paper (Figure 1). First, A literature review was conducted along with using a risk breakdown structure (RBS) to identify the different risks associated with the construction phase.

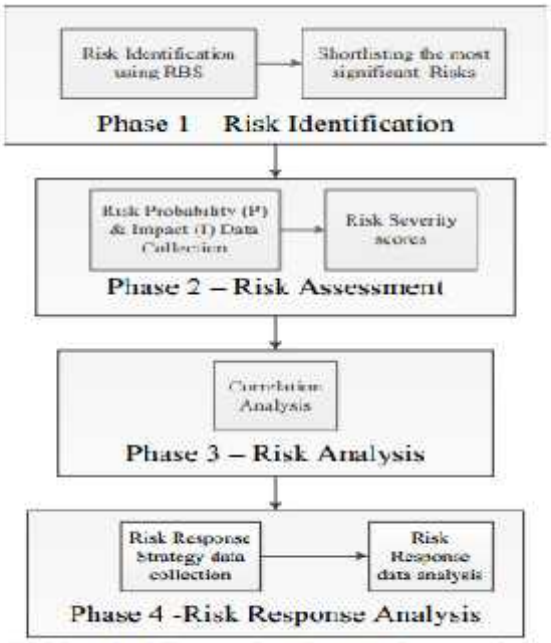

Figure 1: Methodology for full research study, while this paper focuses on Phase 4

The second phase, open-ended interviews were conducted with different expert professionals who combined have more than 8 years of road construction and maintenance works experience. Those professionals participants held several different positions including but not limited to; road and bridge department director, bidding and estimating department head, risk manager and project manager. Participants were asked to share their expertise to shortlist the risks that were identified for construction and maintenance. The risks identified for the construction phase were shortlisted to the most significant ones as shown in Table 1.

The third phase was the risk assessment where the probability (P) and impact (I) values for each risk were collected to assess the severity (S) of each risk along with the risk response strategy chosen which will be explained in detail later in this paper. The P and I data were collected from 10 different contractors through different interviews where they were asked to provide the P and I values for each risk and their response strategy for that risk. Last phase, a correlation analysis was conducted to study the association between the different risks throughout the construction and maintenance phases.

Table 1. Shortlist Of The Most Significant Risks Associated With The Construction Phase

Level/Process	Construction Risks
Design	Specifications of the tender
	documents are unclear
	Still Raw Design
	Error Estimation
Contract	Scope of work not
	Defined
	Unclear Contract
	Clause
Performance	Performance Standards
	Bonding Capacity
	Price Fluctuation
	Weather Changes
	Working Methods Unclear
	Design Changes
	Material Specification Changes
	Fulfillment Performance Standards
	Application of Penalty
Maintenance	Maintenance Period

Analysis of the risk response is the main purpose of this paper which involved 2 main elements: (1) Data collection for risk response strategies , and (2) Analysis of risk response strategies.

Risk Response Strategy Data Collection

Data of the risk response strategies were collected from the 10 respondents after defining and explaining the different possible strategies:

1) Risk Avoidance: This strategy can be achieved by trying to eliminate the risks or the uncertainty. This can be done by adopting a different approach in the project (e.g. not bidding on the high-risk portion of the project) or what many researchers refer to as a complete risk transfer to a third party.
2) Risk Mitigation: This strategy aims towards reducing the overall risk exposure but not totally eliminating or transferring the risk. This is done by reducing the risk severity by reducing one or both of its components represented in the probability or the impact of the risk.
3) Risk Acceptance: This strategy recognizes the risks and monitors them through the project and builds in some (underestimated) contingency to account for it if happens for a minimum loss encounter.
4) Risk Transfer: Transferring responsibility for managing a risk factor to another organization entity better able to mitigate the risk factor .

The above mentioned strategies were explained to the respondents in a manner where the avoidance strategy is considered the best and the most aggressive approach to eliminate the risk completely.

The mitigation strategy is the next choice which is usually selected to reduce exposure and the last resort is to accept the risk unless it can be addressed by any other strategy. Finally, the combinations of the risk response strategies are there to give the experts the choice if they see that the risk is very incidental or situational to the project nature.

Risk Response Data Analysis

Data of the risk response strategies for the maintenance construction risks showed in Table 1 were collected in table format where the respondents were asked to fill their choice of risk response strategy according to those six strategies proposed and explained above. Every respondent chose one of the strategies for each risk. After the collection, the risk response strategies were tabulated and graph into a column frequency diagram for analysis.

RESULTS AND DISCUSSION

The raw data for risk response strategies is shown in Table 2. The results from each respondents yielded four sets of risk response strategies: Acceptance, Avoidance, Mitigation, and Transfer.

Table 2 Risk response strategy raw data

Risk Name	Contr-1	Contr-2	Contr-3	Contr-4	Contr-5	Contr-6	Contr-7	Contr-8	Contr-9	Contr-10
Specifications of the tender documents are unclear	Acceptance	Acceptance	Acceptance	Acceptance	Acceptance	Acceptance	Acceptance	Acceptance	Acceptance	Acceptance
Still Raw Design	Acceptance	Mitigation	Mitigation	Mitigation	Mitigation	Mitigation	Acceptance	Mitigation	Acceptance	Acceptance
Error Estimation	Acceptance	Mitigation	Mitigation	Transfer	Mitigation	Mitigation	Mitigation	Mitigation	Mitigation	Mitigation
Scope of work not Defined	Avoidance	Avoidance	Avoidance	Avoidance	Avoidance	Avoidance	Avoidance	Avoidance	Avoidance	Mitigation
Unclear Contract Clause	Mitigation	Mitigation	Mitigation	Mitigation	Mitigation	Mitigation	Avoidance	Mitigation	Avoidance	Avoidance
Performance Standards	Mitigation	Mitigation	Mitigation	Mitigation	Mitigation	Mitigation	Acceptance	Mitigation	Acceptance	Acceptance
Bonding Capacity	Acceptance	Acceptance	Acceptance	Acceptance	Acceptance	Acceptance	Acceptance	Acceptance	Acceptance	Acceptance
Price Fluctuation	Mitigation	Mitigation	Mitigation	Mitigation	Mitigation	Mitigation	Acceptance	Mitigation	Acceptance	Acceptance
Weather Changes	Transfer	Transfer	Transfer	Transfer	Transfer	Transfer	Transfer	Transfer	Transfer	Transfer
Working Methods Unclear	Mitigation	Mitigation	Mitigation	Mitigation	Mitigation	Mitigation	Transfer	Mitigation	Transfer	Transfer
Design Changes	Mitigation	Mitigation	Mitigation	Mitigation	Mitigation	Mitigation	Acceptance	Mitigation	Acceptance	Acceptance
Material Specification Changes	Transfer	Acceptance	Transfer	Transfer	Transfer	Transfer	Acceptance	Transfer	Acceptance	Acceptance
Fulfillment Performance Standards	Mitigation	Mitigation	Mitigation	Mitigation	Mitigation	Mitigation	Mitigation	Mitigation	Mitigation	Mitigation
Application of Penalty	Transfer	Transfer	Transfer	Transfer	Transfer	Transfer	Mitigation	Transfer	Mitigation	Mitigation
Maintenance Period	Mitigation	Mitigation	Mitigation	Mitigation	Mitigation	Mitigation	Acceptance	Mitigation	Acceptance	Acceptance

Based on the above collected data, the risk response strategies was divided into the construction cycle phases as summarized in Table 3. Risks most commonly “Acceptance” by contractors include: Specification of the tender documents are unclear and Bonding Capacity. Risks that contractors “Mitigation” include: Fulfillment Performance Standards and Error Estimation . Also, more than half the respondents chose the “Avoidance” response for: Scope of work not defined, Unclear Contract Clause. Finally, most of the respondents had “Transfer” risk response strategy for three risks: Weather Changes, Material Specification Change and Application of Penalty.

In next analysis, the results were further examined as shown in Table 4 where it describes the number of times each response strategy was selected along with the percentage of the selection.

Table 3 Risk response strategies for the construction phase

Risk Name	Acceptance	Avoidance	Mitigation	Transfer	No. Response	Total Responses
Specification of the tender documents are unclear	10	0	0	0	0	10
Still Raw Design	4	0	6	0	0	10
Error Estimation	1	0	8	1	0	10
Scope of work not defined	0	9	1	0	0	10
Unclear Contract Clause	0	3	7	0	0	10
Performance Standards	3	0	7	0	0	10
Bonding Capacity	10	0	0	0	0	10
Price Fluctuation	3	0	7	0	0	10
Weather Changes	0	0	0	10	0	10
Working Methods Unclear	0	0	7	3	0	10
Design Changes	3	0	7	0	0	10
Material Specification Changes	4	0	0	6	0	10
Fulfillment Performance Standards	0	0	10	0	0	10
Application of Penalty	0	0	3	7	0	10
Maintenance Period	3	0	7	0	0	10

Table 4 Risk response strategy frequency for all risks

Risk Name	Acceptance	Avoidance	Mitigation	Transfer	No. Response	Total Response
Total No. of Response Selection	**41**	**12**	70	27	0	150
% of Response Selection	27,3 %	8%	46,7 %	18%	0%	100%

According to Tables 3 and 4, it is clear that contractors favor the acceptance and mitigation strategies in dealing with most of the risks associated With theconstruction phase. The acceptance and mitigation strategies singled and combined account for about 74% of the total strategies. A closer look at those two strategies will find that for most of the risks originating from financial or force majeure, contractors were adopting the acceptance strategy as in the case of risk of Specification of the tender documents are unclear,Fulfillment Performance Standards , Bonding Capacity, and Weather Changes. Strategy for technical risks leans towards a more mitigating strategy. This can be attributed to the competitive nature of the construction industry along with the recent financial crisis that has pushed contractors to accept more financial risks and try to reduce the severity of most of the technical risks that they can influence by adopting mitigation strategies. Another finding is that the Avoidance-Transfer strategy did not have much frequency of use since these are two strategies that are fairly divergent from each other no matter what the project circumstances and conditions.

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CONCLUSION

Study clearly show that contractors favor the acceptance and mitigation strategies in dealing with most of the risks associated with the construction phase. Moreover, a closer look at those two strategies will find that for most of the risks originating from financial or force majeure, contractors were adopting the mitigation strategy as in the case of risk of investment in innovation, bonding capacity, and weather changes. Strategy for technical risks leans towards a more accepting strategy. This can be attributed to the competitive nature of the construction industry along with the recent financial crisis that has pushed contractors to accept more financial risks and try to reduce the severity of most of the technical risks that they can influence by adopting mitigation strategies.

This study can help PBC contractors by underlining their risk strategies attitudes, which can be a step forward towards providing a more sophisticated and advanced risk response strategies that confront the industry risk in a more aggressive manner. It can also contribute to future development on how to deal with those newly identified risks that materialize only under PBC delivery system. Finally, this research can be a solid step towards a holistic approach in managing risk under Performace Based Contract.

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LABORATORY FLUME EXPERIMENTS ON LOG ACCUMULATION AT A BRIDGE DURING A FLOOD EVENT

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ABSTRACT: Log accumulation due to a flood event was found at a few bridges along the Nayoshi River in Tsuwano Town, Shimane Prefecture, Japan. Laboratory flume experiments simulated the log accumulation at a model bridge. The experiments showed that the log accumulation generated backwater rise at the model bridge. The backwater rise increased with the number of log pieces trapped by the model bridge. The concept of loss coefficient due to the log accumulation was introduced for the discussion of the backwater rise. It is found that the loss coefficients can be determined by the accumulated wood pieces.

Keywords: Flood, woody debris, driftwood, woody debris accumulation, woody debris jam, loss coefficient.

INTRODUCTION

A heavy rain hit Tsuwano Town in Shimane Prefecture, Japan on July 28, 2013 (Figure 1). It caused landslides and debris flows in the mountain area of the Nayoshi river basin in Tsuwano Town. These resulted in flood with significant amount of woody debris and sediment in the Nayoshi river. A few bridges without a pier caused an accumulation or a jam of woody debris and then changed the river course, as shown in Figure 2.

Figure 1. Study area in Japan.

Figure 2. Woody debris accumulation at the bridge and the new river course.

Field investigations into the characteristics of the woody debris accumulation have been conducted on the 26 to 27 October and 1 to 4 November, 2013 (Rusyda et. 2014a). Based on the result of the field investigations, two types of the woody debris accumulation can be distinguished; one is the accumulation of woody debris at the bridges and the other is the accumulation of woody debris on the field along the river. From the viewpoint of flood defense, it is important to know the characteristics of woody debris accumulation at the bridges.

Laboratory flume experiments on woody debris in the stream have been made in the previous works (e.g. Braudrick et al 1997; Braudrick et al 2001; Bocchiola et al. 2006; Bocchiola et al. 2008; Schmocker and Hager, 2011; Rusyda et. 2004a & b). For example, Braudrick et al (2001) discussed the wood transport mechanism. Schmocker and Hager

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(2011) showed that the characteristics of the bridge and the woody debris had significant effect on woody debris accumulation during the flood event in Switzerland on 2005. Rusyda et al (2013a & b) discussed the volume of woody debris trapped by obstacles, such as bridge piers and riparian trees, during flood events. Rusyda et al (2014a & b) also made flume experiments using logs as a model of woody debris and showed that log accumulation can be determined by the 'shaded area' of the pier to river flow. However, more studies are needed in order to obtain deeper understanding of the log accumulation at the bridges. The present study is an extension of the previous works (Rusyda et al, 2014a & b). The purpose of the present study is to investigate the characteristics of log accumulation at a model bridge without a pier in the laboratory flume. In the experiments the behavior of log accumulation at a model bridge are examined using logs as a model of woody debris.

REVIEW OF THE FIELD INVESTIGATIONS INTO THE WOODY DEBRIS ACCUMULATION IN THE NAYOSHI RIVER (Rusyda et al, 2014a)

The Nayoshi River is located in Tsuwano Town, Shimane Prefecture, Japan (Figure 3). The river basin area of Nayoshi River is 17.5 km^2 and the length of the main river is 8.70 km. In this river reach, there are four bridges without piers (Figure 3). The river bed slope is about 1.4° (i=2.4/100) at Point A and Point B (Figure 4).

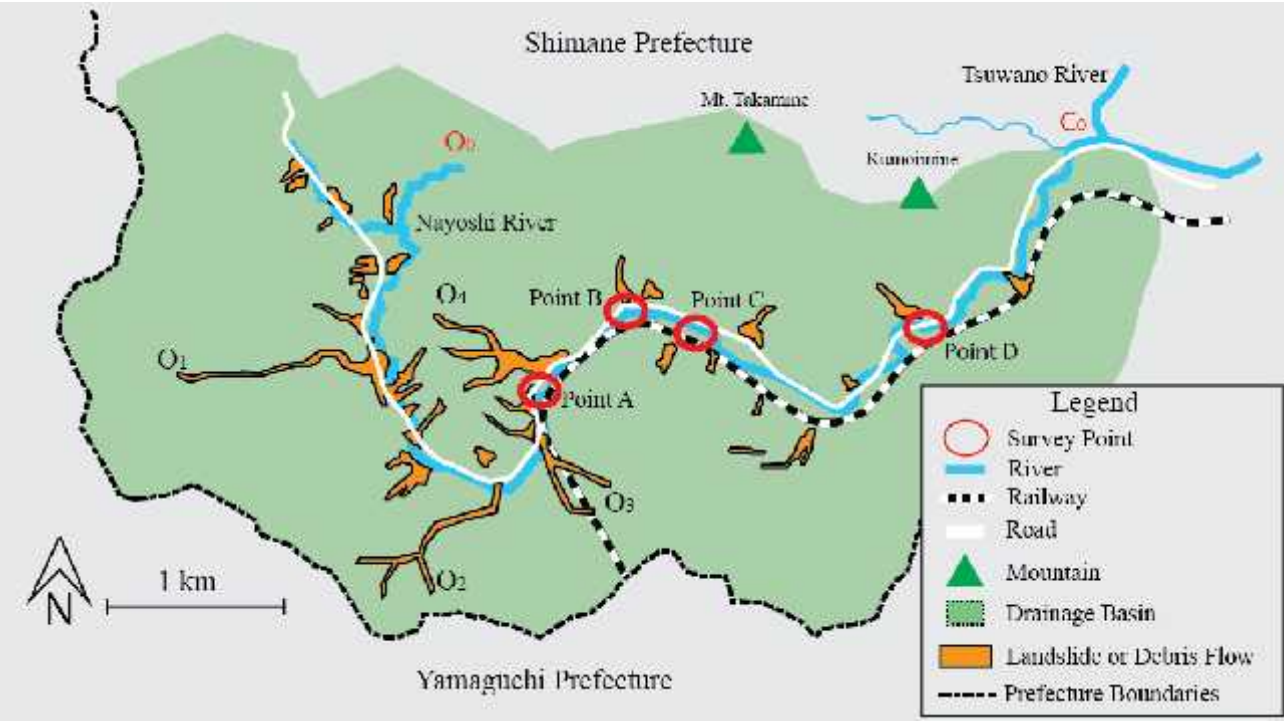

Figure 3. Landslides and debris flows in the Nayoshi River basin.

On July 28, 2013, heavy rainfall occurred in Tsuwano Town. Figure 5 shows the rainfall condition at Nayoshi Rainfall Station. The total accumulation of the rainfall was 411 mm from 2 a.m. to 10 a.m. and the largest hourly rainfall was 92 mm from 5 a.m. to 6 a.m. in the early morning. This heavy rain caused many landslides and debris flows. The landslides and debris flows carried a significant amount of wood and sediment into flood flow of the Nayoshi River.

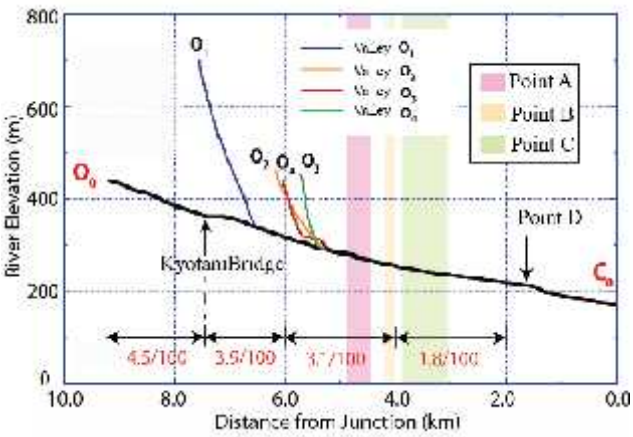

Figure 4. The longitudinal profile of the Nayoshi River.

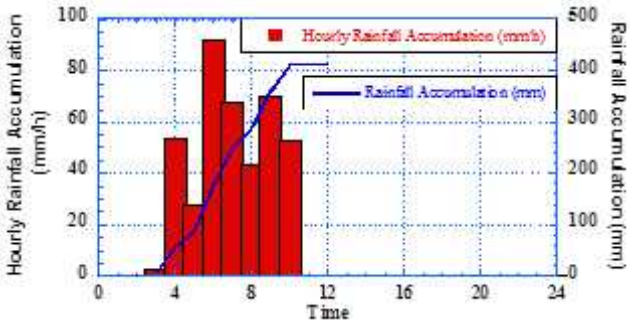

Figure 5. Rainfall Condition

During the flood event, woody debris was trapped by a few bridges in the Nayoshi River. Figures 6 shows the situation of woody debris accumulation at the bridge. Figure 7 indicates the scale of the bridge and the woody debris accumulation.

Figure 6. View of woody debris accumulation at the bridge at Point A from the upstream side.

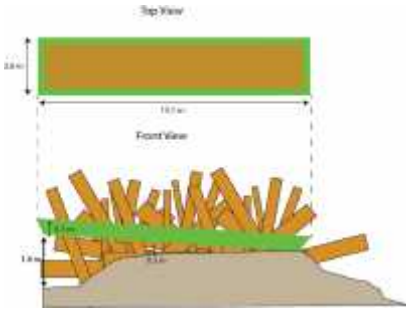

Figure 7. Scale of the bridge at Point A.

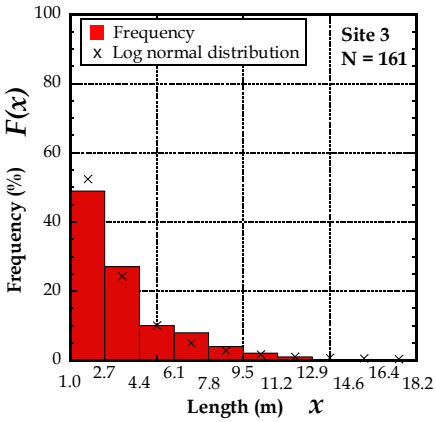

Figure 8. Frequency distribution of length of each wood pieces at point C.

Figure 8 and 9 show frequency distributions of length and diameter of each woody debris piece along the Nayoshi River, respectively. It is found that individual wood pieces have the largest frequency of length between 1 m and 3 m and of diameter between 12 cm to 16 cm. The average length of the individual wood pieces at point B was around 3.4 m, while the average length at point C was around 3.5 cm. The average diameter of the individual wood pieces at point B was around 15 cm, while the average diameter at point C was around 18 cm (Table 1).

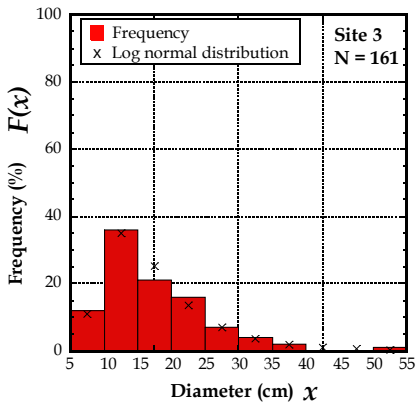

Figure 9. Frequency distribution of diameter of each wood pieces at point C.

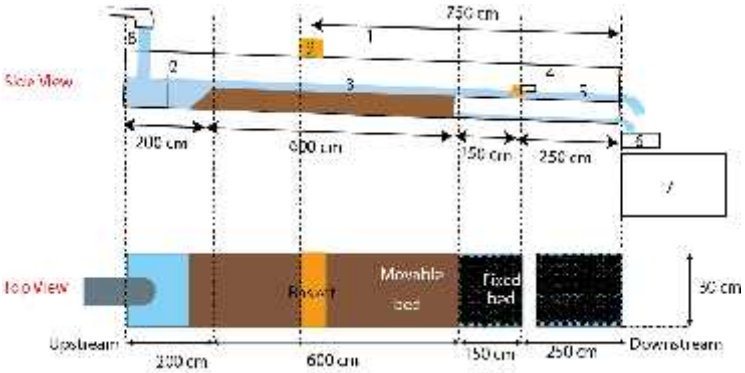

Figure 10. Laboratory flume: (1) Flume; (2)Filter; (3) Movable bed; (4) Bridge and Wood Models; (5) Fixed Bed; (6) Trap; (7) Water Tank; (8) Flow Discharge; (9) Basket of model log

Table 1. Statistical characteristics of individual wood pieces

	Point B (N=125 wood pieces)		Point C (N=125 wood pieces)	
	Length (m)	Diameter (cm)	Length (m)	Diameter (cm)
Maximum	18	33	12.7	53
Minimum	1.1	6	1.1	6
Mean	3.4	15	3.5	18

LABORATORY FLUME EXPERIMENTS

Laboratory Flume

A rectangular flume was used in the experiments, as shown in Figure 10. It is 12 m long and 30 cm wide. The both sides of the flume was smooth glass. A model bridge without piers was installed at the station 2.5 m upstream from the downstream end. The flume was set at the slope of 0.6/100.

Sediment grains were used as the movable bed material. Their grain density was 2.65 g/cm³. The diameter of the sediment grains was d_{50} = 3.6 mm. The sediment grains was laid in the movable bed part with a thickness of 10 cm. The length of the movable bed part was 6 m.

Model Bridge

Referring to the bridge in Nayoshi River, we made a model bridge for the laboratory experiments. Figure 11 shows the model bridge in the flume. The length, width and thickness of the model bridge was 30 cm, 5.2 cm and 1 cm, respectively. The clearance between the model bridge and flume bed was 5.03 cm. The model bridge was set 2.5 m from the downstream end. The dimension of the bridge is shown in Figure 12.

Figure 11. Model Bridge

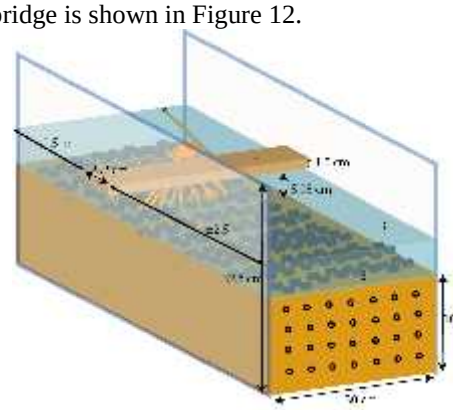

Figure 12. 3d image of the bridge in the flume experiment fixed bed part: (1) Flume; (2)Fixed bed; (3) Surface Flow; (4) Bridge Models; (5) Trapped wood models; (6) Hole for the groundwater; (7) Water Tank; (8) Flow Discharge; (9) Basket of wood model pieces

Figure 13. Basket and water flow direction

Model Wood

Cylindrical wood pieces were used as model logs; their length was 7.0 cm and their diameter was 2.0 mm. The reduced size of the model log was 2/100 (Table 2). The prototype of the model log was based on the field investigations of Rusyda et al (2014a). In order to maintain the similarity between the model in the laboratory and the prototype in the field, the model log conditions were determined. The woody debris deposited in the field has branches and root. However, for simplicity, the present experiments used cylindrical wood pieces as model of woody debris . This satisfies the condition of L>>D. In order to maintain the waterlogged condition, the log pieces were soaked in the water for 10 minutes and then were put in a basket. The density of the wood was 0.65 g/cm³.

Table 2. Experimental condition

No	Measurement (m)	Model Measurement (cm)	Scale
Bridge Length at point A	13.1	30	2/100
Bridge Width at point A	2.6	5.2	2/100
Bridge Length at point B	10.8	30	3/100
Bridge Width at point B	1.9	5.2	3/100
Mean of Wood Length at Point B	3.4	7	2/100
Mean of Wood Diameter at Point B	0.15	0.2	1/100
Mean of Wood Length at Point C	3.5	7	2/100
Mean of Wood Diameter at Point C	0.18	0.2	1/100

In order to drop the wood pieces into the flume channel, the basket was mounted on the top of the flume sides 5 m upstream from the model bridge; it was 56.4 cm long, 16 cm wide and 26 cm high (Figure 13). The basket consisted of 4 sections which were filled by 200 pieces respectively.

The wood pieces were released on the flow surface by opening the bottom of each section in the basket at the same time. This method was modelled after the woody debris inflowed by landslides on valley slopes.

Test Procedure

Four video cameras were installed at three positions: the first video camera near the downstream end was measure the time for flow discharge measurement; the second one on the top of the flume was to record the movement of the wood; the third and fourth ones at the right-hand side of the flume was to record the movement of the wood and water flow.

First, water flow was supplied at the upstream end of the flume. The steady flow discharge of around 300 cm^2/s was set up. Several buckets were prepared at the downstream end in order to measure the flow discharge. Steady and uniform flow condition was satisfied in around 60 seconds from the inflow of water at the upstream end. Second, the model wood pieces were released on the flow surface at the station 7.5 m from the downstream end. Some wood pieces were transported to the downstream end by the water flow and the others were trapped at the model bridge. Third, flow discharge and depth were measured immediately after the wood pieces trapped at the model bridge. Finally, wood pieces trapped at the bridge were counted after the stoppage of supplied water.

Table 3 shows the experimental conditions. There were eleven runs. The duration of each run was around 15 to 20 minutes.

Table 3. Experimental condition

No	Discharge (cm^2/s)	Number of released wood pieces
1	296	600
2	301	400
3	298	800
4	294	600
5	300	600
6	300	400
7	298	800
8	299	400
9	299	200
10	301	800
11	300	600

Table 4. Experimental results

No	Discharge (cm^2/s)	Number of released wood pieces	Number of trapped pieces
1	296	600	398
2	301	400	3
3	298	800	422
4	294	600	541
5	300	600	425
6	300	400	0
7	298	800	723
8	299	400	158
9	299	200	29
10	301	800	400
11	300	600	565

RESULTS AND DISCUSSION

Log Accumulation

The situation of log accumulation at the model bridge is illustrated in Figure 14. The result of experiments are described in Table 4. Moreover, Figure 15 shows the relationship between the number of trapped wood pieces and the released wood pieces. It is found that the number of trapped wood pieces increases with the number of the released wood pieces. However, trapping wood pieces at the model bridge requires sufficient number of released pieces. The critical condition for trapping wood pieces by the model bridge was around 200 pieces.

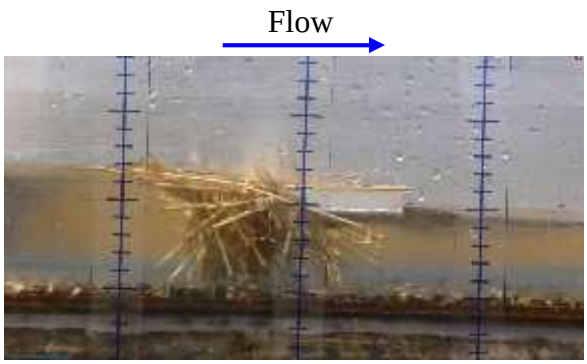

Figure 14. Side view of log accumulation at a model bridge during water flow.

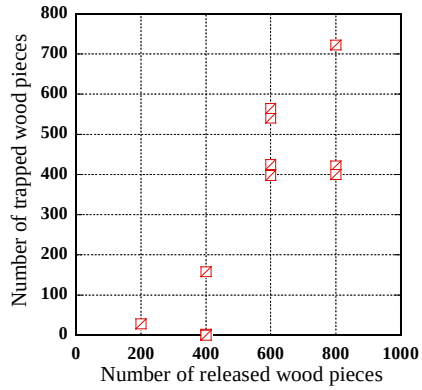

Figure 15. Relationship between the number of released and trapped wood pieces

Backwater Rise

Backwater rise indicates a water level increase due to obstruction of the flow. In this experiment, significant backwater rise occurred at the upstream side when water surface contacted with the bridge and then caused the log accumulation. Figures 16 and 17 shows longitudinal profiles of water surface level with and without log accumulation at the model bridge. The log accumulation at the model bridge generated a significant backwater rise. However, approximately same water level can be found at the downstream side from the model bridge.

Figure 16 Backwater phenomena process before and after log accumulation

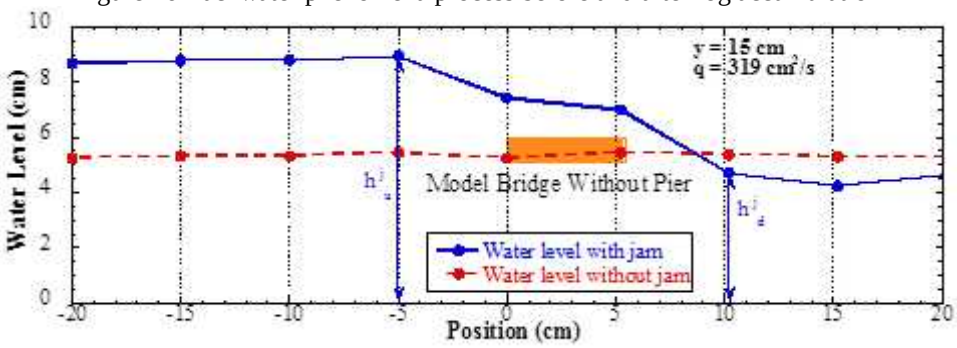

Figure 17 Water height condition at a model bridge with log accumulation and without log accumulation

In order to calculate backwater rise, the following equation are used:

$$\Delta h_{ud}^{j} = h_{u}^{j} - h_{d}^{j} \tag{1}$$

where h_{u}^{j} = upstream water depth with log accumulation; h_{d}^{j} = downstream water depth with log accumulation. Figure 18 presents backwater rise based on Equation (1) versus the number of wood pieces trapped by a model bridge. It is found that the backwater rise increases with the number of trapped wood pieces.

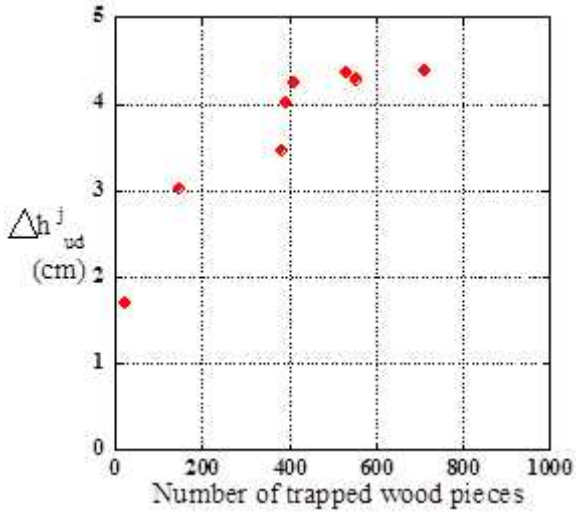

Figure 18. Relationship between the backwater rise with log accumulation and the number of wood pieces trapped by a model bridge.

Loss Coefficient

Head loss due to obstructions in a river can be expressed by 'loss coefficients'. 'Loss coefficients' due to log accumulation and a model bridge can be defined by the following formula;

$$f_d^j \equiv \Delta E_{ud}^j / \left((v_d^j)^2 / 2g\right) \quad (2) \qquad\qquad f_d^n \equiv \Delta E_{ud}^n / \left((v_d^n)^2 / 2g\right) \quad (3)$$

where f^j_d = loss coefficient with log accumulation; f^n_d = loss coefficient without log accumulation; v^j_d = downstream velocity with log accumulation; v^n_d = downstream velocity without log accumulation; ΔE^j_{ud} = energy loss between the upstream and downstream station of the model bridge with log accumulation; ΔE^n_{ud} = energy loss between the upstream and downstream station of the model bridge without log accumulation.

Figure 19 shows loss coefficients versus number of trapped wood pieces at the model bridge. It is found that number of trapped wood pieces determines the loss coefficients. Therefore, it becomes important to predict the volume of wood pieces trapped at a bridge from the viewpoint of flood defense.

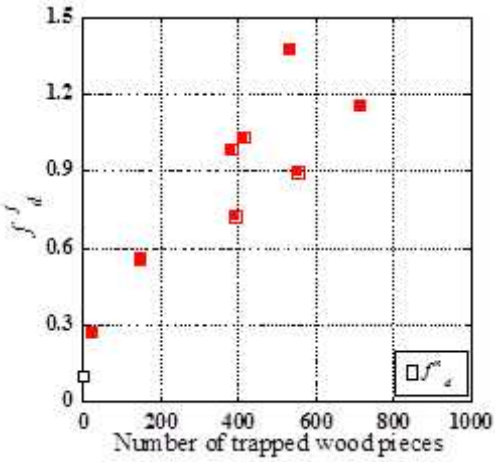

Figure 19 Relationship between the loss coefficient and the number of wood pieces trapped by a model bridge.

CONCLUSION

The woody debris accumulation at the bridge without a pier occurred during the flood event in the Nayoshi River, Tsuwano Town, Japan. This phenomena was simulated by using the log pieces as a model of woody debris in the laboratory flume. The results obtained in this study are as follows:

1. The experiments showed that the log accumulation at the model bridge caused the backwater rise. The backwater rise increases with the number of log pieces trapped at the model bridge.
2. 'Loss coefficient' due to the log accumulation at the model bridge was introduced for the discussion of the backwater rise. It is found that 'loss coefficient' can be determined by the number of log pieces trapped at the model

bridge. Number of trapped wood pieces increase with the number of released wood pieces. Thus, indirectly the number of released wood pieces affected 'loss coefficient'.

3. Since the trapped wood pieces determines the backwater rise and loss coefficient, it becomes important to predict the volume of wood pieces trapped at a bridge from the viewpoint of flood defense.

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NUMERICAL SIMULATIONS OF BAROCLINIC FLOWS IN THE ARIAKE SEA JAPAN AFTER HEAVY RAIN FELL OCCUR USING THREE-DIMENSIONAL OF SALINITY STRATIFICATION ANALYSIS

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ABSTRACT: IPCC had reported in the 5th assessment report (AR5) that there is no doubt for global warming during the past and till the end of this century. Many investigations as an assessment for the impact of global warming on a coastal region have been conducted, for example, on sea level rise, sea water temperature increase, sea water acidification, and so on. However, baroclinic structure in a coastal region due to change of rainfall pattern has not been discussed, yet, except for the effect on marine ecosystem. In this research, the Delft3D model in a coastal area was used as the numerical model to makes three-dimensional hydrodynamic model analysis. In the other hand, the numerical experiment condition, the river discharge data was used and observed start from January 1, 2012. Firstly, the preliminary numerical simulation was carried out for approximately 6 months (till 5th July). Next, we conducted the numerical analysis to compare an effect of runoff pattern on the baroclinic flow for 20 days from 6th July to 26th July, 2012. Finally, we modification river discharge data by using multiplier factor α (1.0, 1,2 1,5 2.0, and 0.8) to make several case condition and makes comparison for each case. From the comparison among numerical experiments for several extreme floods considering as a projection of climate change, change of rainfall pattern can change the baroclinic structure in a coastal region of the Ariake Sea. This result suggests that regime change of marine environment and ecosystem can change due to climate change.

Keywords: *Baroclinic flow, Rain fell, Freshwater, Salinity.*

INTRODUCTION

IPCC had reported in the 5th assessment report (AR5) [IPCC, 2013] that there is no doubt for global warming during the past and till the end of this century. Thus, we have to take account of global warming as a fundamental condition to consider the countermeasures as an adaptation to the future situation. In the report we can see the upward trend of yearly precipitation in the mid latitude region including Japan in high confidence after 1951. Many investigations as an assessment for the impact of global warming on a coastal region have been conducted, for example, on sea level rise, sea water temperature increase, sea water acidification, and so on. However, an effect on regions of freshwater influence (ROFIs) [Simpson and Sharples, 2012] or baroclinic structure in a coastal region due to change of rainfall pattern has not been discussed, yet, except for the effect on marine ecosystem.

In 11th to 14th July, 2012, record-breaking heavy rain, namely the Northern Kyushu Heavy Rain fell on the northern part of Kyushu in Japan, causing floods and landslides, especially in Kumamoto, Oita, Fukuoka, and Saga Prefectures. Some rivers reached the highest discharge above the past largest recorded one. Thus, we attempt to investigate the effect of global warming on freshwater dynamics in a coastal region in the Ariake Sea by three-dimensional numerical experiments in several cases with river runoff in the Northern Kyushu Heavy Rain Event in 2012 and some modified runoff as a projection of climate change.

METHODOLOGY.

Numerical model.

In this study, we used the numerical model that Yano *et al*. (2009) had developed by applying the Delft3D [Deltares, 2008] which was a generalized three-dimensional hydrodynamic model in a coastal area. As shown in Fig.1 computational domain was the range combining the Ariake Sea and the Yatsushiro Sea, which was also one of the largest semi-enclosed bays in Japan. We applied the linear orthogonal coordinate system of 10" interval resolution (Δx=250m) horizontally and the 10 layers σ-coordinate system vertically

Open boundary was set as a line connecting Akune to Kabashima. 40 tide components were given as an open boundary condition, based on the known harmonic constant of both edges. The harmonic constants (amplitude and lag)

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only for the four major tidal components (M2, S2, K1, and O1 tide) were adjusted to measurement result of tide at several tide gauges [Yano *et al*., 2009]. Further the moving wall boundary model for a tidal flat area [Deltares, 2008] was adapted. We omitted description in detail for the calculation accuracy of the present model here, because it was fully confirmed by our previous studies on stratification, the tide and tidal currents [*e.g*., Yano *et al*., 2009]

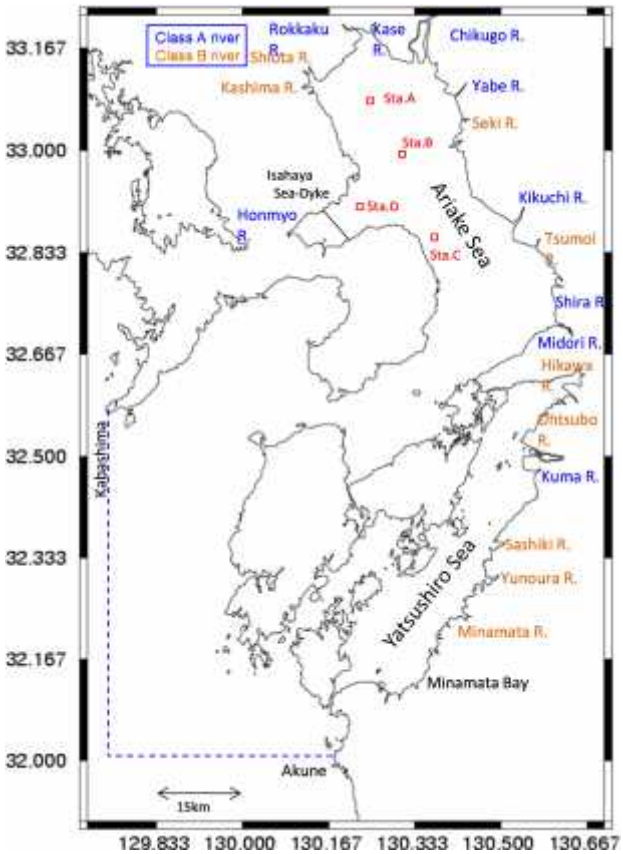

Fig.1 Computational Domain

We considered hourly river discharge from eight A-class rivers (the Chikugo R., the Yabe R., the Kase R., the Rokkaku R., the Kikuchi R., the Shira R., the Midori R., and the Kuma R.) and nine B-class rivers (the Shiota R., the Kajima R., the Seki R., the Tsuboi R., the Hikawa R., the Ohtsubo R., the Sashiki R., the Yunoura R., and the Minamata R.) for simulation of the baroclinic flows due to freshwater inflow (see Fig.1).

Numerical Experiment Condition

In the present paper numerical simulation was carried out using observed river discharge data from January 1, 2012. Firstly, the preliminary numerical simulation was carried out for approximately 6 months (till 5th July). Next, we conducted the numerical analysis to compare an effect of runoff pattern on the baroclinic flow for 20 days from 6th July to 26th July, 2012.

Fig.2 shows temporal variation of tide at Oura (near mouth of Isahaya Bay) and river discharge of A-class rivers in the Ariake Sea from 6th July, 2012 to 26th Jul, 2012. We can see two large floods on 12th July and 13-14th July. The first flood has a total peak discharge of about 9,000m^3/s, and the second one has about 15,000m^3/s.

Table 1 Case for numerical Experiments condition

Case No. (*i*)	Factor
1	1.0
2	1.2
3	1.5
4	2.0
5	0.8

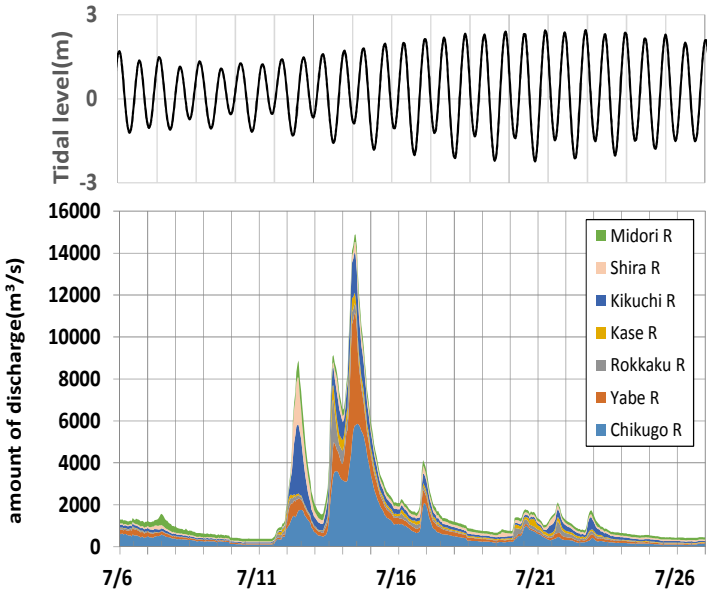

Fig.2 Tide and total runoff from A-class rivers during a period from 6th-26th July, 2012

In the present study this flood (Case-1) is considered as a basic condition of flood event after global warming. Factor is used as a multiplier factor as shown in the equation (1).

$$Q_i(t) = \sim_i Q_1(t) \qquad (1)$$

where, i: case number, t: time, Q: river discharge

Numerical experiments in five cases with various value of ~ as shown in Table 1 were conducted to investigate on effects of flood discharge on the baroclinic flows in the Ariake Sea due to changing the all river discharge. Especially, case-4 was the most extreme case with twice discharge of the Northern Kyushu Heavy Rain event. On the other hand, case-5 was the smallest flood case in this series.

RESULT AND DISCUSSION

Horizontal Salinity Distribution Pattern

Fig.3 shows the computation result of horizontal salinity distribution in the surface layer (top layer) of the Ariake Sea in both cases (case-1 and case-4) at 11:00 on 14th July, when passed one day after the largest flood on 13th July.

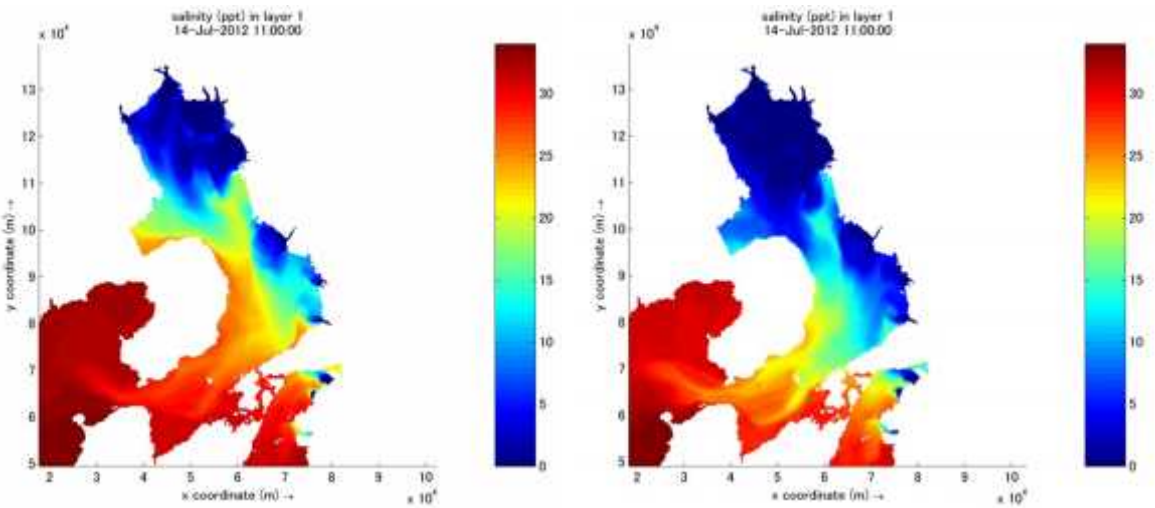

Fig.3 Computation result of horizontal salinity distribution in the surface layer in case-1 vs case-4 at 11:00 14th July, 2012.

This timing was low tide, when freshwater from rivers could most widely spread in the bay area. We could see that riverine freshwater had distributed only in the northern region (head of the bay) where the Chikugo River and the Yabe River were located, and freshwater didn't reach to the southern region, and case-4 shows the same distribution in the northern region also, but freshwater could reach in Isahaya Bay. On the other hand, Figs.4 shows ones in both cases at 12:00 on 15th July (at low tide after two tidal cycles (25hours)). From the result in case-4, we could see that the riverine freshwater could reach to Isahaya Bay. From this figure could be seen, freshwater in case-1 has reached to Isahaya Bay. From a comparison both of them there was a significant difference of freshwater spreading in the southern region of the bay could be seen.

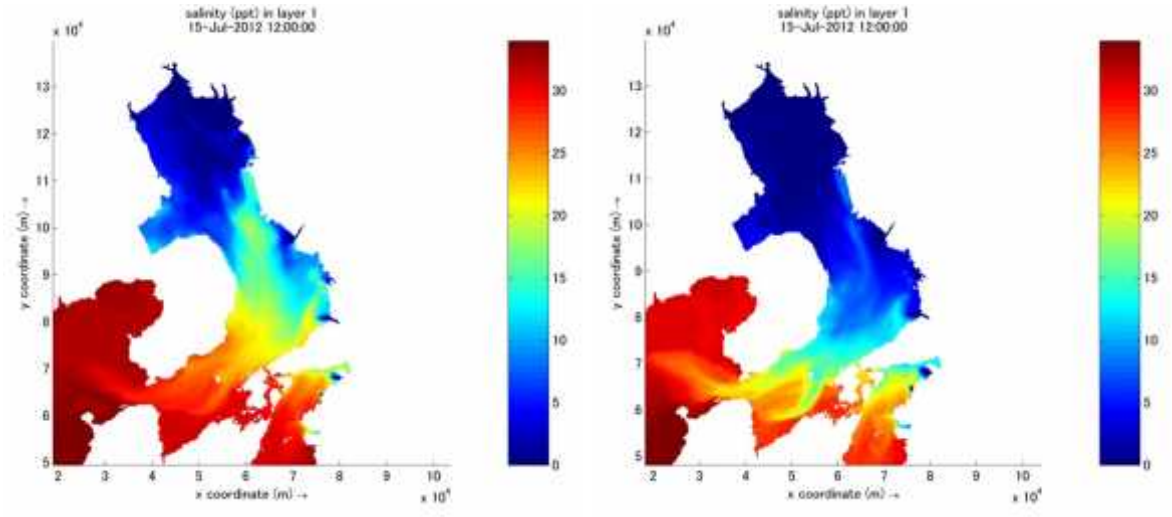

Fig.4 Computation result of horizontal salinity distribution in the surface layer in case-1 vs case-4 at 12:00 15th July, 2012

Vertical Structure of Salinity Stratification

Fig.5 and 6 show computation results of salinity isopleth in case-1 and 4 at Sta. A (see Fig.1) located in the head of the bay and at Sta. D (Fig.1) in Isahaya Bay, respectively. In case-1 the salinity stratification developed during neap tide and it was weakened in spring tide. On the other hand, in case-4 stratification is weakened due to large amount of freshwater input during neap tide strong stratification kept during spring tide. Generally, stratification will develop in neap tide, in which tidal current is weak, and disappear in spring tide with stronger tidal current in the Ariake Sea. Thus, change of rainfall pattern can make aquatic environment shifted to a different regime.

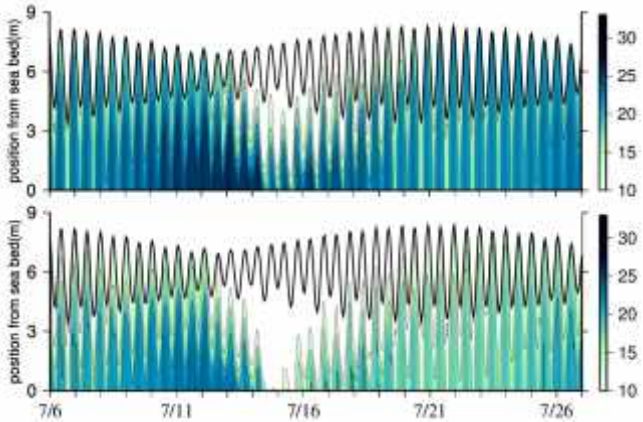

Fig.5 Computation results of salinity isopleth in case-1 (upper) and 4 (lower) at Sta. A.

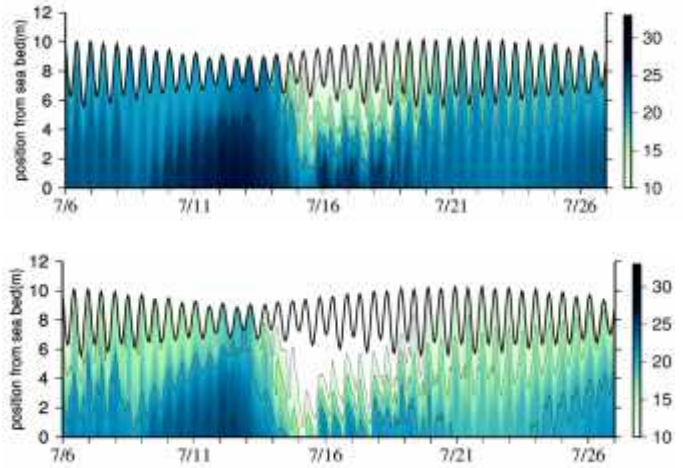

Fig.6 Computation results of salinity isopleth in case-1 (upper) and 4 (lower) at Sta. D.

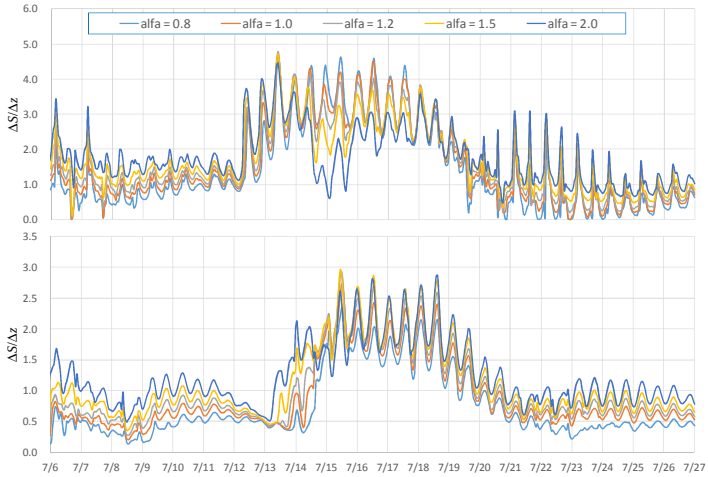

Fig.7 Computation results of temporal change of $\Delta S/\Delta z$ at Sta. A (upper) and Sta. D (lower).

Finally, Fig.7 shows the temporal change of vertical salinity gradient $\Delta S/\Delta z$ [1/m] in all five cases at Sta. A and Sta. D. Here, ΔS is defined as a difference between salinity at surface layer and one at bottom layer, and Δz is water depth h. $\Delta S/\Delta z$ can demonstrate the strength of stratification. Well mixed condition can be seen during the flood in case-3 and case-4, which are large discharge cases, at Sta. A in the head of the bay, but only strong stratification is shown at Sta. D in Isahaya Bay. This implies that an impact of large freshwater input can be characterized locally.

From these result we can evaluate that the effect of runoff change due to climate change can affect not only stratification dynamics but also anoxic water dynamics. Further study by ecosystem model for the effect on marine ecosystem should be conducted for understanding the overall effect of runoff change.

CONCLUSIONS

From the comparison among numerical experiments for several extreme floods considering as a projection of climate change, change of rainfall pattern can change the baroclinic structure in a coastal region of the Ariake Sea. This result suggests that regime change of marine environment and ecosystem can change due to climate change. Thus, further detailed assessment of impact on water quality and ecosystems is necessary in order to evaluate the necessity of adaptation to the change.

ACKNOWLEDGEMENTS

We would like to say thank you for everyone that has contributed on this research till complete, and on this moment also I want to say special thanks to Mr. Nishimura and Mr. Kitagawa that already support to running simulation on this research, best regards both of you.

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RAINFALL-RUNOFF MODELING OF BENGAWAN SOLO CATCHMENT AREA WITH TANK MODEL USING IFAS

Hary Puji Astuti [1], Umboro Lasminto [2] and Andi Patriadi [3]

ABSTRACT: A flood forecasting by rainfall data or discharge has not been implied well in most of areas in Indonesia. It is caused by the existence of equipment so data will be uncompleted. Needed a modeling is to know runoff on each catchment area. Along with the development of satellite technology, there is technology named Geographic Information System (GIS) which come with its application and program. A program is to make hydrology modeling using distributed and tank model called Integrated Flood Analysis System (IFAS). In this case, model will be made for Bengawan Solo Catchment Area. The purpose of model is to know how big the goodness of IFAS modeling when it will compare with measurement data. Problem of this case is how to make model of Bengawan Solo catchment area in IFAS, parameter calibration of surface, subsurface, aquifer and river course. Next, it also will find the best model according to discharge from AWLR (Automatic Water Level Recording) of Bengawan Solo using three types of tests : Root Mean Square Error (RMSE), Nash and Volume Error (VE). Watershed model of IFAS will be made in five type (default, 1, 2, 3 and 4) which each model will have different rate of parameter. Determination of parameter is taken by analysis of impact of each parameter to the graph shape. Changes of parameter will impact to the hydrograph. According to simulation model and three tests before, the best model is model 4. Model 4 has RMSE 3.097, Nash 0.955, and VE 0.0000442%. This model said as the best is under the limit of RMSE (gain to zero), Nash (gain to one) and VE (under 5%). Using three tests before, parameter of model 4 will be validated to the same location but different period. Validation of model in different period will result performance which is not as good as before. In other hand, the value of test is still under the edge. This model has rate value for RMSE 6.88, Nash 1.106 and VE 0.28%.

Keywords: *Bengawan Solo catchment area, IFAS, modeling, rainfall, runoff, satellite*

INTRODUCTION

Flood is hard unpredictable nature disaster if there is no early warning system. It always causes many damages when it happens. Bengawan Solo River is the longest river in Java and causes flood every year to cover some lands in its catchment area.

Flood prediction is related to hydrology process and influenced by meteorological factor. A process of rainfall-runoff needs time influenced by characteristic of catchment area, location of rainfall and land-use. Flood prediction and warning system use time lag which used to count change rainfall into runoff or surface runoff.

[1] Shanti (2009) said that cellular automata method that NDVI (Normalized Difference Vegetation Index) can predict flood in Bengawan Solo river. It assumes some variables which influence rainfall and duration. [2] Maulana (2009) made rainfall runoff modeling of Delluwang catchment area using SWMM (Storm Water Management Model). SWMM is a model which can analyze water quantity and quality connected with drainage. And [3] Lasminto (2011) made modeling and flood prediction of Bengawan Solo catchment area with satellite data. There was error between observed data model 1%, 10% and 20% at different period.

Needed rainfall data is dominant in each catchment area. unfortunately, some rainfall data and land-use are missed because it is still hard to get them. Nowadays, the government installs new equipment called AWLR (*Automatic Water Lever Recorder).* The main purpose is to increase the measurement of runoff accuracy on the river.

There is a modeling using *tank model* and *distributed model* in free software named IFAS (*Integrated Flood Analysis System).* This software uses satellite rainfall data and ground rainfall data. It can calculate river's discharge so known flood forecasting and its inundation area according to the history.

The main problem is the accuracy of IFAS in comparing with manual calculation. It also needs parameter from AWLR or water surface level. Parameter will calibrate the result. Calibration result will show how it closes with IFAS prediction value

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IFAS (INTEGRATED FLOOD ANALYSIS SYSTEM)

IFAS is application software developed by ICHARM to make runoff modeling and flood prediction. The purpose is to solve unavailable rainfall data and discharge. It helps because using observation satellite based rainfall data. This program has some positive aspects :

1. Huge area coverage for satellite data, e.g.: rainfall data, topography, land-use data and geology data. All of data can be downloaded from available website.
2. Available parameter in hydrology and GIS data to be combined to simplify user in analyzing and using hydrology modeling to catchment area
3. IFAS can show output in graphic and digital maps. Users can identify the flood risk by visual observing simulation result

Besides modeling process in IFAS shown in picture below :

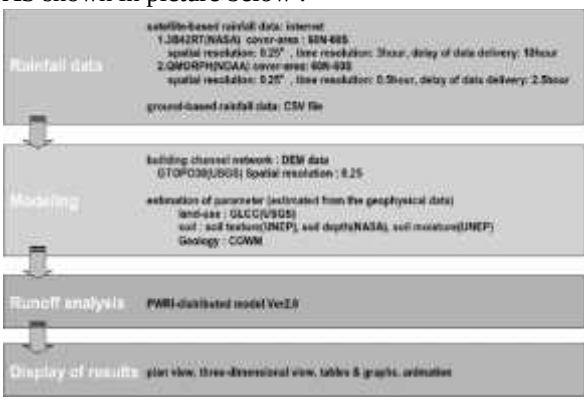

Fig 1. Diagram of IFAS

IFAS uses distributed and tank model as main concept. The river flow is like a tank. There are two flows, vertical and horizontal. There are two parts in vertical flow : surface tank and ground water tank. In surface tank, there is a sub-layer called subsurface, but sometimes it is seldom used. In this research, sub-surface will become a part of parameter which will be optimized. The purpose is to make the result getting closer with actual data. In the picture below, there is illustration showing tank model.

TANK AND DISTRIBUTED MODEL

Tank model is simple model which shows hydrology process with four vertical tanks. The main purpose of this concept is to know the physical characteristic of catchment area. All of precipitation assumes to enter the first tank. Precipitation is located before the first tank and evaporation is taken from process of first tank. If there is no water in first tank, evaporation will be taken from second tank; if there is no water in second tank too, evaporation will be taken from third tank; and so on. Output from the highest tank is calculated as surface runoff, and output from the second tank is calculated as middle flow and so on [4] (Sugawara, 1978).

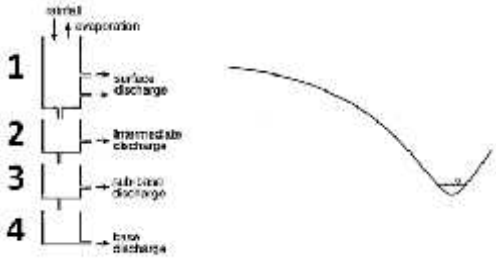

Fig 2. Simply 2D Tank Model

Surface tank illustrate not only surface runoff which has middle to fast velocity but also percolation into groundwater tank. In this process, evapotranspiration is rejected so groundwater tank stimulates groundwater flow (confined and unconfined). River flow is illustrated between river channel tank using kinematic wave (look at fig 3).

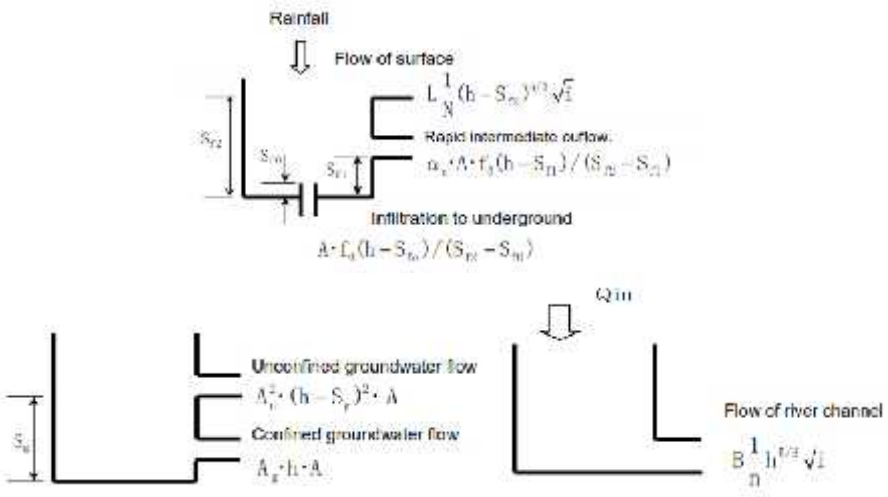

Fig 3. Tank hydrological model in each layer

This model has advantages in stability. It is easy and applicable, connected with GIS and continued to be researched with some innovation [5] (ICHARM, 2012).

Rainfall runoff modeling uses distributed model because all data components connected with basin condition, globally land-use data and soil type. Picture below stimulates DAS to some grids which needed rainfall data and surface runoff on each grid. This model is used to make modeling by using sample. Surface goes into direction of outlet cell on each grid of catchment area.

Fig 4. Distributed hydrological model on gridded catchment area

FEASIBILITY MODEL TEST

Root Mean Square Error (RMSE) and Nash are used to fix the criteria which represent model. RMSE represents mean square deviation for output of model to measurement data. Best value for RMSE is closed to zero. The other method is Nash. It compares runoff model square deviation and actual runoff. The result of simulation succeeds if model closes one (1) (Drogue, et al, 1999) [6]

$$\text{RMSE} = \sqrt{\frac{1}{n}\sum_{i=1}^{1}(Q_{obs} - Q_{model})^2} \quad (1)$$

$$\text{Nash} = 1 - \frac{\sum(Qmodel - Qobs)^2}{\sum(Qobs - Qobsrata)^2} \quad (2)$$

$$\text{VE} = \left|\frac{\sum_{i=1}^{n} Qobs - \sum_{i=1}^{n} Qmodel}{\sum_{i=1}^{n} Qobs}\right| \times 100\% \quad (3)$$

Qobs = observed discharge (m^3/s)
Qmodel = model discharge (m^3/s)

MODELING OF BENGAWAN SOLO CATCHMENT AREA WITH IFAS

There are 30 parameters which separated into four parts. Before making modeling with IFAS, ensured that make parameter analyses to know how parameter influencing result.

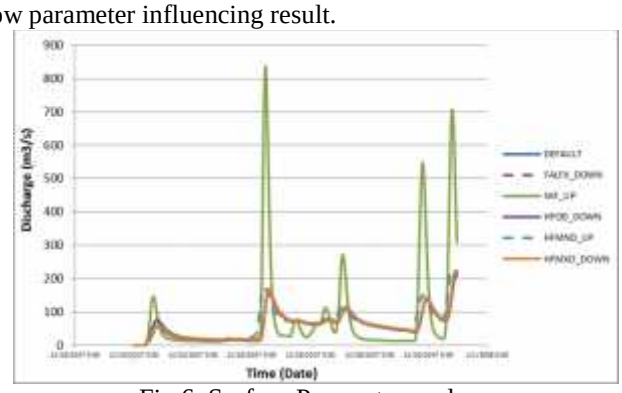

Fig 6. Surface Parameter analyze

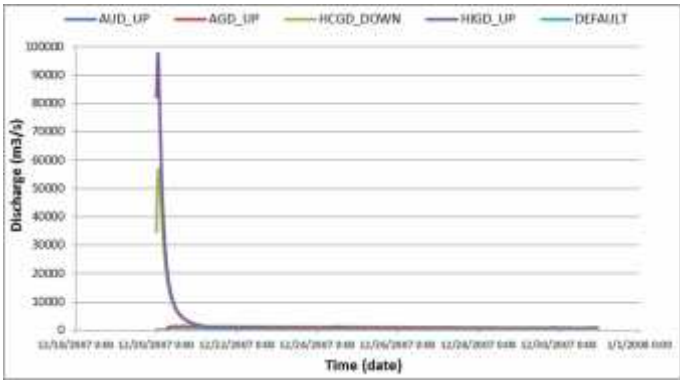

Fig 7. Aquifer parameter analyze

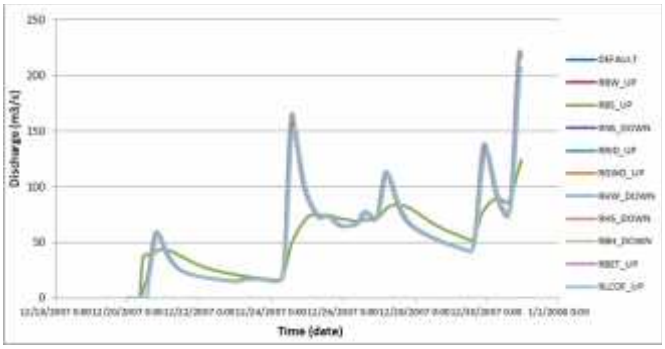

Fig 8. River course parameter analyze

According to Fig 6, SKF and HFMND influence the result significantly. Increasing of SKF not only increase peak volume but also make big difference between valley and peak of wave. Increasing of HFMND gives time lag for peak volume. Increasing of these parameters will accelerate peak volume time. Another word we can say, rise part from wave will be shorter than before.

According to Fig 7, there are some parameters which give changes in making wave. Decreasing of HCGD influences the rise of base-flow significantly. Increasing of HIGD will rise base-flow, so it makes set-part longer. It makes time for peak volume coming faster.

According to Fig 9, parameters which influence in shaping wave are RBS, RNS, RLCOF. River course parameter is according to actual physical data. Increasing of RBS will decrease in shaping wave. But increasing RLCOF will move peak volume time. The time friction will increase rise part so peak time will be longer. Decreasing of RNS will decrease peak volume too.

Modeling of Bengawan Solo catchment area with IFAS starts by fixing location and downloads some data. Used rainfall data is period between December, 20-30 2007. Elevation and topography data are downloaded from GTOPO30 on E100N40 column for Indonesia

In this research, we will make five models in IFAS. Each model has its parameter value.

- *Model default*, is with parameter default.

- *Model 1*, is made with parameter value three times larger than parameter default for surface and aquifer. Sub surface is made two times larger than parameter default.
- *Model 2*, is made with parameter which is between parameter model 1 and parameter model 3.
- *Model 3*, is made with most of parameter surface and aquifer bigger than parameter model 1. Parameter subsurface is same as parameter model 1 but parameter river course is made smaller than parameter model 2.
- *Model 4*, is made with default river course parameter. Most of parameters value are as same as parameter model 3 but SKF, SNF, HIGD, RBW and RBS are made smaller.

RESULT OF MODELING

In fig 9 – fig 14, five model has illustrated in graph which compare with measurement discharge and rainfall data.

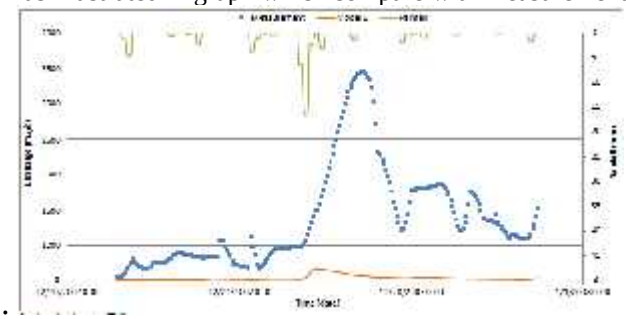

Fig 9. Comparison graph observed discharge and model default

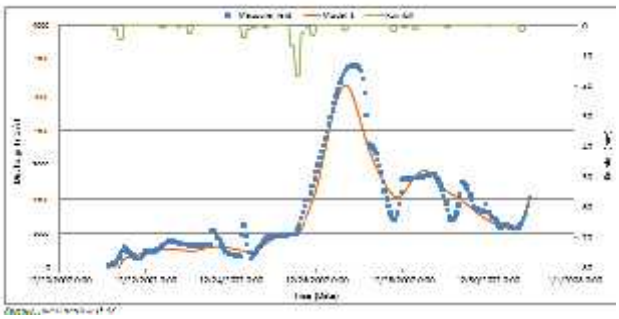

Fig 10. Comparison graph observed discharge and model 1

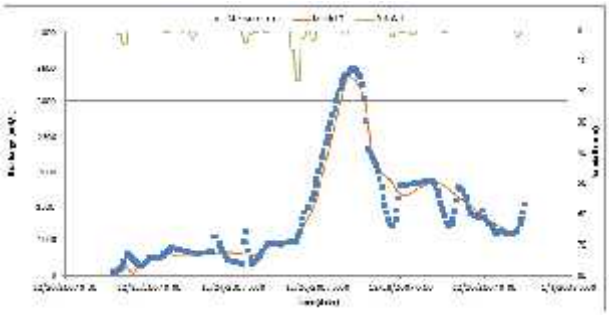

Fig 11. Comparison graph observed discharge and model 2

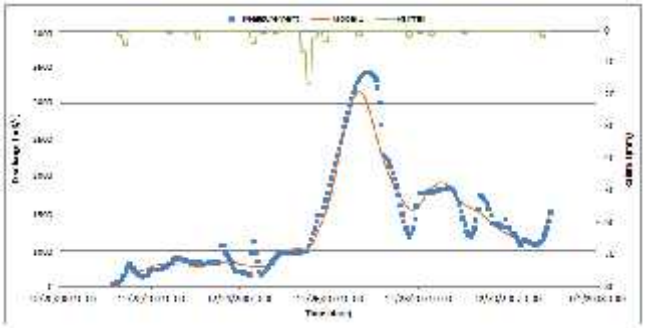

Fig 12 Comparison graph observed discharge and model 3

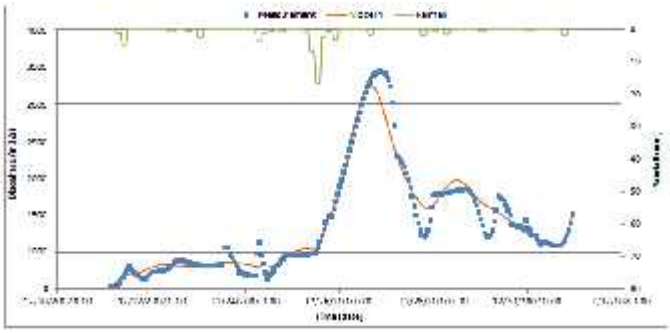

Fig 13. Comparison graph observed discharge and model 4

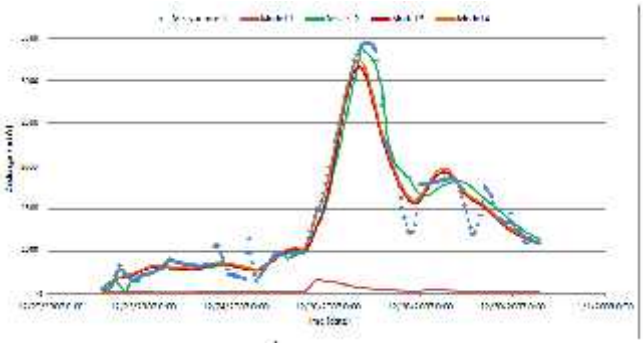

Fig 14.

According to Fig 14 above, peak volume of model closes peak volume of observed discharge but wave shaping model is not as varied as wave shaping observed discharge. Wave shaping is flatter than shape of model 3. On model 3, peak and valley of wave shapes significantly following measured discharge. Peak volume of model 3 is smaller than model 2.

Comparison hydrograph between model 3 and model 4 does not have many differences. On model 4, peak volume is bigger than model 3. The most influenced factor is SKF. On model 2, parameter SKF is higher than model default and model 1 so it influences increasing of volume on model 2. Furthermore, SKF value of model 3 is bigger than model 2. Peak volume of model 3 decreases but other side of wave increases. The rise of this parameter will give influences for shaping valley of wave on model 3.

Besides that, comparison among model 1, 2, 3, 4 and observed discharge has lag time of peak volume. The time friction of each model is caused by calibration of parameter value. The most influenced factor for time lag is HFMND. If HFMND is less, time of peak volume will be faster.

Significant changes happen from model default to model 1. Value increases in some aspects discharge, base-flow, wave and peak discharge. According to feasibility test : RMSE and Nash, we can get conclusion in table below

Table 1 Recapitulation Feasibility Test For Model

Model	VE (%)	Keterangan	RMSE (%)	Nash
Default	22.771	Oke, <5%	21.908	-1.265
Model 1	0.083	Oke, <5%	3.555	0.940
Model 2	0.004	Oke, <5%	3.447	0.944
Model 3	0.045	Oke, <5%	3.373	0.946
Model 4	0.0000442	Oke, <5%	3.097	0.955

The best performance model is model 4 and its parameter value. Model 4 is the best model because VE is less than 5%. Although VE value of model 4 is not the smallest.

Model 4 which has 3.097% in RMSE is the smallest value. Although the value is about 3%, it is still far from ideal condition. For Nash, model 4 has the best and the smallest value. Its value is 0.955 which is the nearest among other models.

Validation Model

Validation model is needed to do to know how much the parameters influence model [7]. Model 4 which is the best performance has to be validated by changing some components. In IFAS, the component is rainfall-runoff period which changes into 2008, 1-11 March. In fig 15 below, it shows the differences discharge of measurement and model

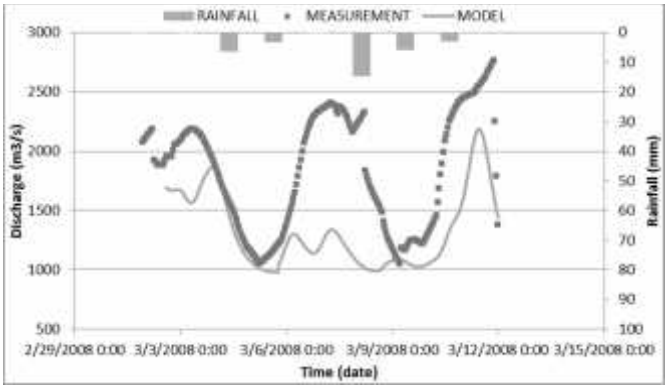

Fig 15. Validation model

As reviewed previously, validation model needs to be tested with same method. This study assumed that three methods have represented actual value.

$$\text{RMSE} = \sqrt{\frac{1}{n}\sum_{i=1}^{1}(Q_{obs} - Q_{model})^2}$$

$$\text{RMSE}_{val_model} = \left(\sqrt{\frac{1}{243} \times 115043653}\right) \times 100\% = 6{,}88\%$$

$$\text{Nash} = 1 - \frac{\sum(\text{Qmodel} - \text{Qobs})^2}{\sum(\text{Qobs} - \text{Qobsrata})^2}$$

$$\text{Nash}_{val_model} = 1 - \frac{115043653}{5{,}066189{,}990} = 1{,}016$$

$$\text{VE} = \left|\frac{\sum_{i=1}^{n} Qobs - \sum_{i=1}^{n} Qmodel}{\sum_{i=1}^{n} Qobs}\right| \times 100\%$$

$$\text{VE}_{val_model} = \left|\frac{44685{,}262 - 321073{,}070}{44685{,}262}\right| \times 100\% = 0{,}28\%$$

According to validation model, all of validation model tests show good result. Validation model tests have value 1.016 ≈ 1 for Nash, 0.28% < 5% for VE and 6.88% for RMSE.

CONCLUSIONS

Each parameter has their influences to shape the wave and create discharge in IFAS model. According to feasibility test model (three kind of tests), the best performance model is model 4. As described previously, this study uses validation model to ensure the result with the same kind of test. They shows good enough validation value in other period for model 4.

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THE GEOGRAPHY OF DILI AND THE FLOOD CONTROL PROBLEMS

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ABSTRACT: Dili, East Timor's capital, is a rather flat city, but with a very small area, lying nestled between the mountains and the sea. According to the latest estimates, the city has a number of inhabitants which exceeds 230,000 and a strong population growth is expected to take hold in the coming years. With this population growth, the city also getting bigger, new areas were sealed by the construction of new access roads but especially for house building to the growing population. Thus the rainwater that falls on Dili and, in particular, the waters that come down from the adjacent mountains, while the building construction makes the city with less infiltration capacity, the floods in the city are becoming a recurring fact, during the rainy season, a period with hazards consequences for the population, economy and environment. Under a Cooperation between the Government of East Timor and the Government of Victoria was developed between 2010 and 2012 one Master Plan Drainage in Dili, whose main conclusions and recommendations were approved by the Timorese Government. Among those recommendations was proposed a phased planning interventions in the drainage of rainwater, in order to avoid constant flooding. The plan extends to 2025 and is now in Phase 2, which will end in 2017. This phase comprises the design and construction of the works considered first priority. From 2017 to 2025 will be held the Phase 3 with the design and construction works of medium term. In the already elapsed so far, in Phase 2, the East Timor Government hired a company - Águas de Portugal - that won the support project to the National Directorate of Basic Sanitation, the implementation of a significant amount of drainage projects in Dili. These Detailed Engineering Projects include the rehabilitation of 19 urban Channels of the 36 existing in Dili, 5 of their 6 Rivers and the construction of two retention basins. The purpose of this presentation is to inform the 2nd MICCE, about the detailed engineering designs already prepared, projects still in progress and the projects that need to be performed in the future, for the complete resolution of a permanent drainage plan and the implementation of this plan. We consider as priority works the following (Regularization of 2 Rivers and the intervention in 16 channels are already contracted and its budget is about 75MUSD to open this year and should last for about 2 years running. In the 2nd MICCE the purpose of this study is to present the current situation of floods in Dili and the technical solutions found and its reasons for drafting this DED, especially the 2 Rivers to be primarily intervened with the construction of concrete walls in most of the extent to stabilize, with a conventional construction, but also with the use of so-called Berlin Wall in individual areas of higher population density along the riverbanks. A hydraulic model will also be presented, as well as the solutions provided to the mouths of rivers and main drainage channels[4].

Keywords : *Channels, Dili, Flood, Hydraulic Model, Retention Basins, Rivers.*

INTRODUCTION

In 2010, the Constitutional Government of the Democratic Republic of Timor-Leste (designated as Government) cooperated with the Government of Victoria, Australia, began the master plan for Dili's Sanitation and Drainage, named by "Dili Sanitation and Drainage Master Plan" (DSDMP).

The Master Plan Phase 1 was concluded in 2012 and, in October of the same year, was presented to the Council of Ministers and received Government approval.

In that stage of the Masterplan, it was concluded that the frequent floods in Dili are mainly due to the following factors:

- Insufficient capacity of the drainage system (undersized network);
- Sediments deposition along the channels;
- Drainage channels clogging/obstruction due to accumulation of debris and litter;
- Kankung plantation in drainage channels and river beds;
- Blockages caused by pedestrian/roads passage over the channels, reducing their cross-section;
- High rate of urban development resulting of 30% population increase since 2004;

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[4] This presentation is based on a technical study carried out in 2014/2015 to the government of East Timor by AdP - Águas de Portugal (wwwadp.pt), as Contractor, in collaboration with Cenor Consulting Engineers (www.cenor.pt), as main Sub-Contractor

In addition to the current situation diagnosis, the DSDMP Phase 1 established strategies for mitigation of floods and defined methodologies and criteria to be considered in the catchment areas and urban drainage network analysis.

Phase 1 recommended some structural improvements to the drainage system and indicates the following technical solutions as relevant to alleviate the existing flooding problem:

- Existing channels widening/deepening/re-sloping;
- Construction of retardation basins in the down town area;
- Modification and/or rehabilitation of existing retardation basins in the upper catchments;
- Construction of relief channels;
- Construction of sediment traps/litter traps;
- Construction of roadside drainage systems with sufficient inlet pitch from the road to the drains;
- Construction of new diversion channels to divert flow from the existing channels that are flooded most;
- Construction of sufficient manholes in the underground drainage conduits to allow maintenance work in the future.
- River system improvements to increase the existing river conveyance.

The Government requested the DED (Detailed Engineering Design) of part of the Díli drainage system under the scope of the Masterplan, initially estimated on the 2013-2025 Investment Plan. They are the following:

- Almost all the city drainage channels (they are 36 in total)
- 2 Retention Basins (RB1 and RB 3) in a total of 5 that was study in phase 1;
- Rivers Maloa, Kuluhun. Santana, Becora and Bemori training works including Maloa and Becora upstream basins refurbishment. Only Comoro river (the must large and important in the city is outside this study.

DATA SOURCES

- Dili Sanitation and Drainage Master Plan – Phase 1, prepared by Melbourne Water, Yana Valley Water, SKM, June 2012, provided by NDBS in March 2014;
- Dili Sanitation and Drainage Master Plan – Phase 1, drainage network records (in editable format) obtained during Phase 1, provided by the Government in May 2014;
- Drainage network records, provided in May 2014;
- Data from sub-hourly rainfall of tree udographers stations, provided in April 2014:
  - EH Dili's Airport (data for the period from 29.05.2012 to 27.03.2014).
  - EH Dili's Benamauk (data for the period from 29.05.2012 to 27.03.2014).
  - WTP Bemos (data for the period from 09.07.2012 to 31.03.2014)
- Port of Díli Tide Data, provided by Aportil in April 2014.
- Extracts of Dili's road drainage network design, regarding recent works of DNEPcc, provided between June and July 2014.
- LiDAR orthophotomaps and detailed topographic mappings, prepared in April and May 2014 by the Consultants.
- Fieldworks carried out by the consultant in April and May 2014.

The following data would have been in the best interests of the study, however, was not made available:

- Initial study called "*Dili Sanitation and Drainage Master Plan (DSDMP)*" in editable format, including all written and drawn pieces, as well as the utilized data.
- Previous studies related to the drainage network of Dili.
- Records of maximum floods levels reached in several places in the city.
- Urban intervention projects and studies that may be important to this study.

Furthermore, some meetings were held with some Sucos (a division of a District, or group of Villages) Leaders, in order to gather information about specific problems and to get their own opinion about possible routes' drainage network.

DATABASE UPDATING

In order to develop the study and DEDs based on reliable information, it was made an update of the database, including precipitation data to determine the peak flow to be considered in the design, topography and existing network registration.

The updating of the database was a lengthy process due mainly to the following factors:

- Lack of networks registration;
- Information dispersed by various entities
- Garbage accumulated in the channels and access difficulties to allow successful mapping;
- Lack of “as-built” drawings of the recently completed, or in construction stage, drainage works.

The topographic information was obtained by LiDAR (Light Detection And Ranging) survey, supplemented with photogrammetric survey.

The registration of the existing network was obtained from the authorities and supplemented by information collected in the field.

The rainfall data were obtained from the Government. The design flow rates were determined based on these data as it's described later.

LIDAR SURVEY

Topographic information was developed using Airborne Laser Scanning (ALS) - LiDAR method. Data was obtained from a fixed wing aircraft on April 2014.

The ground check survey allowed an assessment of the accuracy of the ALS LiDAR data.

The contours supplied were compiled from a rigorous triangulation of the supplied data. They are intended to provide a visual representation of the terrain data.

PHOTOGRAMMETRIC SURVEY

In order to complement the information collected by LiDAR and assign characteristics of continuity that increase the degree of detail in the establishment of the profiles, as well as facilitate the "altimetric" legibility of the information, it was considered that it would be appropriate to acquire additional information by photogrammetric methods from the LIDAR data, aerial photography and ground control points.

Pursuing this goal, and based on aerial triangulation used for the production of orthophotos supplementary cartographic information is being acquired for various channels and major water courses. We would like to emphasize that the accuracy of the information collected may be affected by obstacles of various kinds, such as the presence of vegetation, shadows caused by adjacent elements to channels, water levels and the existence of sediment / debris and photo interpretation compatible with GSD.

Given these constraints, field crews have been mobilized to collect additional information through expeditious methods. In the following section, filed surveys are described.

FIELD SURVEYS – GENERAL

The field surveys had, at first, four main goals:

- Reconnaissance works of the areas to be served and the existing drainage network.
- Identification of sections to be studied.
- Channels characteristics identification (cross-sections dimensions) and collection of relevant information regarding maintenance and operation related problems.
- Channels alternative courses analysis and evaluation of possible solutions to be implemented.

The network surveys proved to be essential for the development of the studies, given the inadequacy of existing records and the finding, in the field, the ongoing drainage works unknown to the Government.

SURVEY CARRIED OUT

For the analysis of a drainage network operation it's paramount to be based on the updated registers.

With regard to the Government drainage network registration, it was found that the information it had was insufficient because:

- in most cases does not present data of the channels and drains cross sections dimensions;
- has no altimetry information;
- has inaccuracies regarding connections between channels and also about the location of the drainage network sections;
- does not identify all existing channels.

On the other hand, the LiDAR survey doesn't have a degree of detail allowing the identification of all channels cross-section.

Thus, in order to clarify these information gaps, it was necessary to conduct extensive field reconnaissance, so that information can be properly included in the drainage network modelling.

Manholes opening/inspection campaign was also made to obtain information about the manhole depth and check the inlet/outlet pipes diameter.

This campaign was hindered by the fact that, in many situations, the manholes covers are:

- covered with asphalt pavement, therefore, it is not possible to open;
- blocked or damaged, therefore, it is not possible to open;

The information gathered in the field survey, in the LiDAR topography and aerial photograph, allowed to correct some shortcomings in network register and, also, to detect specific conditions of the drainage network that should be corrected, regardless the works that may be proposed in this study.

Some examples of those conditions are presented below and, since they are common along the drainage network, we consider that they should be subject of correction.

Those conditions refer to:

a) Structural problems:
   - Cross-section reduction in channels downstream sections
   - Overlapping jurisdiction regarding the drainage network
   - Buildings on top of the channels
   - Lack of rules regarding the use of public space by the different services (water, electricity and telecommunications):
   - Lack of maintenance and conservation of the existing structures:

Figure 1 - Channel D9

Figure 2 - Channel D3.4

Regarding the drainage network maintenance, we emphasize that DNSB began in January 2014 a program for channel cleaning that ended in August of the same year.

This work was essential to prepare drainage network for the next rainy season and allow better performance, however, there are still some problems which can only be overcome with the implementation of a maintenance and operation program of the drainage system that ensure and/or provide:

- The continuity of the cleaning works of channels and sediment retention chambers;
- Replacement of existing waste retention systems by others with operation and effective maintenance
- Eradication of sewage discharges directly to the storm water drainage channels.
- River mouths desilting while the rivers training works of the rivers are not implemented;

Figure 3 - Channel D16

Figure 4 - Channel D9

Figure 5 - Channel D6

Figure 6 - Channel D11

Figure 7 - Maloa river mouth

During the field survey it was possible to verify that, in Dili, were being executed numerous road drainage works.

Since is not reasonable that the drainage network modeling do not include this information, several contacts have been made with Roads Directorate in order to get the “as built” drawings of those works and also DEDs of other works that are envisaged to be implemented in the short term.

Figure 8 - Collapse of Channel D10 sea outlet Structure

Figure 9 - Becora river. Sediments retention weir with foundation severely undermined

Figure 10 Collapse of Channel D16 sea outlet structure

Figure 11 - Channel D11

Figure 12 - Channel D10.1

Figure 13 - Channel D35.1

EXISTING DRAINAGE SYSTEM - OVERVIEW

In general, the Dili drainage system is located mainly in the lower parts of rivers Comoro, Maloa, Kuluhun, Santana, Bemori e Becora catchments, within low and flat urbanized areas.

The drainage system comprises, essentially, the following main components:

- Six major rivers that flow through the city and discharge into the coastline (Comoro, Maloa, Kuluhun, Santana, Bemori e Becora);
- Urban drainage consisting of large trapezoidal and rectangular open channels and some underground pipes/box culverts. The channels discharge the runoff through concrete outfalls at the shoreline or at the rivers.

Dili's drainage network is classified into four levels:

Level 1 – rivers

Level 2 – channels with more than 2 meters width

Level 3 – channels with over 1 and up to 2 meters width

Level 4 – channels with less than 1 meter width Dili's major drainage network consists of the six rivers and about 70km of masonry or concrete channels with over 1 and up to 5 meters width trapezoidal or rectangular.

The secondary drainage is performed by level 4 channels and, in general, occurs the following:

- The consolidated urban areas are served by a small section masonry or concrete channels network connecting to the major drainage network.
- There are no drainage networks in the unconsolidated urban areas, the stormwater run directly on the ground surface or in small ditches that normally are silted or collapsed.

In addition, were recently built and are still under construction, roads drainage infrastructures in Dili urban area, consisting of closed channels under sidewalks or channels in the open, in both sides of the streets, which collect the stormwater and connect to the main drainage channels.

HIDROGRAFIC NETWORK

Maloa and Kuluhun River

The Maloa and Kuluhun rivers have similar features and their total length can be divided into two distinctive reaches:

- The mountainous or upstream reach, coming from elevation 846 and 860m, and an average slope of about 16% and 18 5%, in Maloa and Kuluhun rivers respectively.
- The urban, or downstream reach, coming from elevation 16 and 30 down to the ocean, resulting in a much smaller average slope of just about 0,7% and 1%, in Maloa and Kuluhun rivers respectively.

It is along these latter reaches that the training works must be performed and it's also along these reaches that occurs an intense urban occupation of the river banks.

The present downstream reach, with a total length of about 2,3 km, has, basically, a rectangular cross-section, with depths varying between about 3 m and 2 m. On the Kuluhun River downstream reach, with a total length of about 2,8 km, the depths vary between 3 m and 1.3 m.

The longitudinal slope decreases gradually from upstream to downstream, starting at about 1,7% in Maloa river and 2 % in Kuluhun river and ending down at virtually zero (almost horizontal bed).

The section width varies between 15 m and 7 m in both rivers, more as function of urban encroachment than as a consequence of natural movements of the rivers bed and banks.

The present river bed conveyance is also limited further by the existence of five bridges in Maloa river and six bridges in Kuluhun river. Some of these bridges have a very narrow span, sometimes with intermediate piers that increase flow obstruction, and, most important of all features, the structure of their trays reduces the flow depth available for the floods.

URBAN DRAINAGE NETWORK

As already mentioned, the urban drainage network is composed of a vast network of open or closed channels along the Dili city and that can be distinguished into three categories:

- The main channels that connect to the receiving environment (ocean or rivers);
- The secondary channels, smaller, and connected to the main channels;
- Tertiary channels even smaller, and connected to the secondary channels

The actual design of the system result of a drainage infrastructure development process extending from earlier drainage systems in older urban areas.

Currently, the existing infrastructures are in general in poor condition, both structurally as hydraulic operation point of view, resulting from the construction seniority and aggravated by lack of maintenance and access.

The Dili drainage basin can be subdivided, at a macro scale, in three different areas defined by:

- West Area, limited by Comoro and Maloa rivers
- Central Area, between Maloa and Kuluhun rivers
- East area, at east of the Kuluhun river

Central Area

The Central Area, in particular, is served by more conventional drainage system, also consists of collectors and box-culverts beyond the usual channels in the open. The main channels in this area of town are the channels D1 to D11. It is a substantially flat area.

The roads drainage is made in most cases by kerbs inlets connected to the main drain although, in some cases, there is also a network of small ground or masonry channels.

This is an area, structured, densely urbanized and although most of the buildings are consolidated, there are some areas in Caicoli and Mascarenhas districts where existing constructions do not seem to be definitive.

Figure 14 - Channel D2. Construction of the road drainage system in Belarmino Lobo street

The consolidated areas are served by a network of closed channels with kerbs inlets along the roads which, in some cases, are in poor condition, or small section open channels of stone masonry or concrete, which have connection to the main channel.

The change of the open sections for buried pipes and some links between main channels are equipped with litter trap systems, however some doubts remain about the effectiveness of the operation / maintenance.

Figure 15 - Channel D3

Figure 16 - Channel D3

Figure 17 - Channel D6

Figure 18 - Channel D7

Channels D3, D4, D5 and D6 discharges into the ocean are made directly under Avenida dos Direitos Humanos road wall;. Channels D1, D2, D8, D9 discharges into the ocean are made are made by concrete structures extend in to the sandy beach. All of them are under the tide influence.

The main problems detected in the Central Zone were:

- Uncertainties of the network registers;
- Existing drains manholes find themselves in completely paved streets, which means it is not possible to access them for maintenance operation or to identify the diameter and ground level of the inlet/outlet pipes;

- Lack of open channel network maintenance
- D1 channel outlet section of the sea outlet structure.
- The sea outlet structures condition.

POTENTIAL RISC OF FLODING

In accordance with DSDMP Phase 1, the existing drainage system has been designed for a return period of 5 years, so, for rainfall with higher return period, is expected to occur floods in Dili.

To aggravate the limitations of this undersized network, there are other factors that decrease the channels flow capacity, worsening the flood scenario. These factors are:

- Sediment deposition and waste / solid material accumulation along the channels
- Channels cross section downstream reduction;
- kangkung" plantations in rivers bed and channels for which are made landfills in the riverbed;
- Flow section bottleneck caused by culverts and pedestrian bridges;
- The increase in impermeable areas as a result of the urban growth of areas located further in the catchment upstream.

HYDROLOGYCAL STUDY – RATIONALE

Since there are no flow gauging stations along de channels and rivers in analysis or anywhere near them, the evaluation of flood discharges on this channel had to be made indirectly, by studying the precipitations that cause them.
So, intense precipitations that occur in the Dili area are studied, leading to the establishment of intensity-duration-frequency curves.

Then, a full physical characterization of the catchment areas is presented.
Based on the intensity-duration-frequency curves and on the catchment areas characteristics, precipitation hyetographs are constructed.

Finally, flood hydrographs are derives from the precipitation hyetographs by using the using the Soil Conservation Service unit hydrograph.

ANALYSIS OF PRECIPITATIONS IN THE DILI AREA

Daily precipitations

There are three main rainfall measurement stations around the Dili area, with several decades of records (enough to make statistical analysis).
There are a few very recent ones, mainly in the Dilli urban area, but their number of recording years is not yet sufficiently large to allow a meaningful statistical analysis.
The three "historical stations" referred above are:

- Dili airport, at altitude 4,20.
- Dare, at altitude 492,00.
- Aileu, at altitude 930,00.

This trio of stations is very suitable to understand how intense precipitations vary with the altitude, as it will be seen further ahead in the present report.

The full records of these stations, both for historical and recent years, are not very easy to obtain or retrieve, for administrative reasons.

So, the consultant team did its best, within the short time available, to retrieve the records that have been used in previous studies, as well as the statistical analysis presented in those same studies.

The first of such studies was, of course, the Dili Sanitation and Drainage Masterplan – Phase 1 (2012), where the following sample of annual maximum daily precipitations in Dili station was used (going from 1977 to 2010):

Table 1 Maximum Daily Rainfall Depth Collected at Dili Airport Station

Year	Maximum Daily Depth (P), mm	Year	Maximum Daily Depth (P), mm
1977	80	1989	40
1978	113	1990	51
1979	58	1991	46
1980	85	2003	54.2
1981	102	2004	127
1982	58	2005	113
1983	61	2006	69.4
1984	78	2007	69.4
1985	72	2008	81.6
1986	52	2009	34.6
1987	45	2010	140
1988	60		

The authors of the Phase 1 Masterplan adjusted a Log Pearson type III statistical law to this sample and obtained the following results from it:

Table 2 Maximum annual daily rainfall for various return periods in Dili station

Return Period (years)	Average Daily Rainfall (mm)
2	71,0
5	96,0
10	115,0
25	140,0
50	158,0
100	**177,0**

In Masterplan Phase 1, no data were used for Dare and Aileu stations.

In 1974, just before Portugal granted independence to its overseas colonies, a synthetic collection of the main hydrological studies made up until then for those colonies was published.

Included in that collection, there is a paper written by J.M. Rocha Faria (1974) and entitled “Análise da Frequência dos Maiores Valores Anuais da Quantidade de Precipitação Diária em Alguns Locais do Ultramar” (Frequency Analysis of the Maximum Values of Daily Precipitation in Some Overseas Places). For Dili and Dare stations, the following samples were used:

Table 3 Maximum annual values of daily precipitation in Dili and Dare

ANO	Dili 8°34’S 125°34’E	Dare 8°36’S 125°34’E
1966	101,3	86,9
1965	69,4	120,7
1964	57,2	226,3
1963	76,1	200,0
1962	47,2	88,3
1961	75,2	91,2
1960	58,0	104,0
1959	106,1	138,0
1958	143,5	90,0
1957	78,3	115,0
1956	131,3	128,0
1955	66,3	91,1
1954		150,9
1953		119,4

The author of the referred paper adjusted Gumbel statistical law to these samples and obtained de following results from them:

Table 4 Maximum annual daily rainfall for various return periods in Dili and Dare stations

Return Period (years)	Average Daily Rainfall (mm)	
	Dili	Dare
5	114,8	166,9
25	167,0	238,7
100	210,0	298,0
1000	281,0	395,7

Finally, in 2004, Mónica Calçada elaborated an MSc thesis in Lisbon Instituto Superior Técnico (IST), entitled "Análise de Cheias e Delimitação de Zonas Inundáveis em Timor Leste" (Flood Analysis and Evaluation of Flooding Risk Areas in East Timor), where she used daily precipitation records for Dili, Dare and Aileu station in the 1951-1974 period.

The sample values she used could not be retrieved in time to be displayed in the present report, but results obtained with the adjustment of statistical Gumbel laws were the following:

Table 5 – Maximum annual daily rainfall for various return periods in Dili, Dare and Aileu stations

Return Period (years)	Maximum Daily Precipitation (mm)			
	Dili		Dare	Aileu
	8 h – 8 h	0 h – 24 h		
50	159,2	193,7	218,05	273,22
100	**176,5**	**209,4**	**240,46**	**310,64**
250	199,2	230,0	269,96	359,90

There is the interesting detail that this author obtained slightly different values for Dili, when she used precipitations measured from 8 h in a day to 8 h in the next and when the measurement was made from 0 h to 24 h.

Here, it must be highlighted that maximum 24 hours duration precipitation and maximum daily precipitation are, statistically, not exactly the same thing, given that, for the former, the highest 24 h consecutive period must be found for each sample year, while for daily precipitations the recording 24 h period is exactly the same every year.

As a consequence, 24 h duration precipitations for a given return period tend to be about 10% higher than the daily ones.

With this in mind, looking at the values highlighted, and taking into consideration some margin of uncertainty, it was considered reasonable to take the following values for the 100 year return period 24 h duration precipitations:

- Dili: 200 mm.
- Dare: 250 mm.
- Ailleu: 300 mm.

Since these three satiations have altitudes close to zero, 500 m.a.s.l and 1000 m.a.s.l, the following equation was derived, to obtain the 100 year return period 24 h duration precipitation (P100_24h) for a catchment area with altitude ALT:

$$P_{100_24h} = 200 + \frac{ALT}{10}$$

However, since the catchment areas that contribute for this channel are predominantly located at sea level the altitude considered as equal to zero.

Once this value of precipitation has been obtained for the 100 year return periods, values for other return periods can be obtained from the following ratios, deducted from other values:

$$\frac{P_{50}}{P_{100}} = 0{,}89\,;\ \frac{P_{25}}{P_{100}} = 0{,}79\,;\ \frac{P_{10}}{P_{100}} = 0{,}65$$

Sub daily precipitations

The majority of Timor rainfall measurement stations only have records of daily precipitations. In the Portuguese colonial period, sub daily durations were recorded only in Dili, but the values are not easy to retrieve.

Nevertheless, the catchment areas that contribute for this channel all have sub daily concentration times, meaning that it is not feasible to study floods with daily precipitations only.

Fortunately, the authors of the Masterplan – Phase 1 installed three new udographers in the Dili area in 2012 in the following locations:

- Dili Airport.

- Benamauk.
- Bemos (Water Treatment Plant).

The Consultant team managed to obtain time continuous rainfall records in these three stations going from May 2012 to May 2014 (2 full years).

Just two years is not sufficient for a statistical analysis. Therefore, an alternative method was used to deduct sub daily precipitations from daily precipitations with the same return period.

It was verified that precipitation records are virtually equal in all three stations, which is not surprising, because they are very close to each other and they all stand at altitudes near to the sea level. So, we took the Dili Airport records and we elaborated the following graphic representation of accumulated precipitations, starting in May 29th 2012 and ending in May 6th 2014:

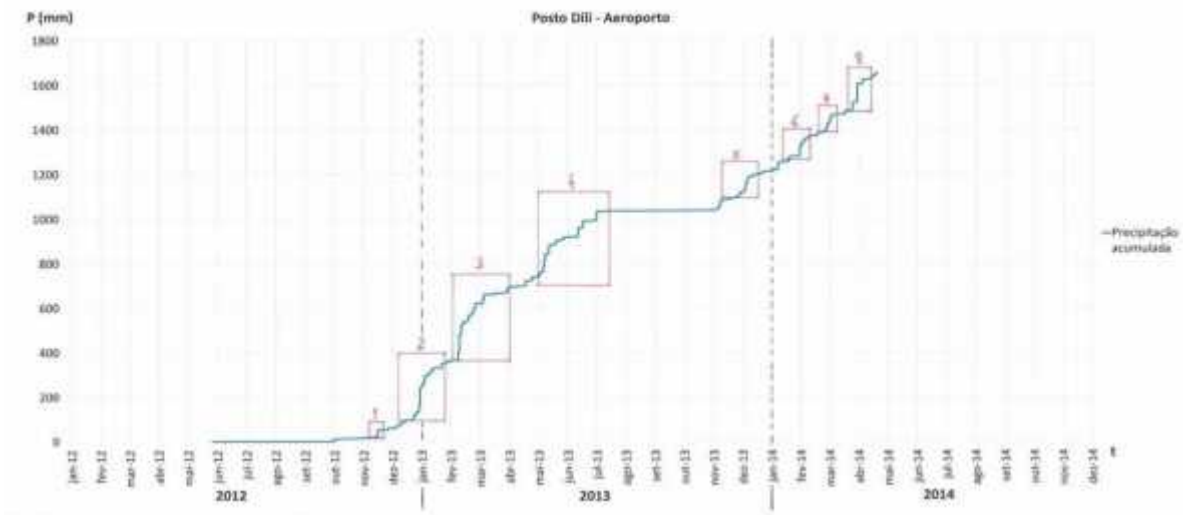


Figure 19 – Accumulated precipitations in Dili Airport Station

The flat reaches of this graphic represent periods of low or absent precipitation, while the steep reaches represent high precipitation periods.

The steeper de reaches are, the more intense the precipitations were. So, eight zones of high precipitations were selected within the graphic.

Each of these eight zones comprehends several individual precipitation events, in a total of 18, as indicated in appendices 1 through 8, each of the events having been represented also in appendices 9 through 25.

For each individual event, the following analysis has been made:

Evaluation of the maximum continuous precipitations with 1/2, 1, 3, 6, 12 and 24 hours.

Evaluation of the ratios Pt/P24 of the maximum precipitation with duration t and the maximum precipitation with 24 h duration.

Looking at the graphics in appendix 1, the following results and conclusions have been obtained:

Daily precipitation is, usually, concentrated within a period that seldom last more than 3 hours in a row with higher intensity at the beginning of the event. After that, it normally doesn't rain again before another 24 hours (another, separate, event).

The values of the Pt/P24 ratios vary very little from event to event, the average values being:

- P0,5/P24 = 0,39; P1/P24 = 0,62
- P3/P24 = 0,82; P6/P24 = 0,87
- P12/P24 = 0,90; P24/P24 = 1,00

It was also considered interesting to see what were the absolute maximum values of the precipitations with each duration, from all the 24 events. The results were as follows:

- P0,50max = 30,00 mm; P1max = 52,20 mm;
- P3max = 79 mm; P6max = 80,00 mm;
- P12max = 80,00 mm; P24max = 100 mm.

From these values, the following ratio values were obtained:

- P0,5/P24 = 0,30; P1/P24 = 0,52
- P3/P24 = 0,79; P6/P24 = 0,80
- P12/P24 = 0,80; P24/P24 = 1,00

Intensity-Duration-Frequency Curves

The two sets of Pt/P24 values obtained in the previous sections are not too different from each other, which certainly confirm its general applicability.

It was decided to take the second set and, with it, the adimensional intensity-duration presented in Figure 20, was derived:

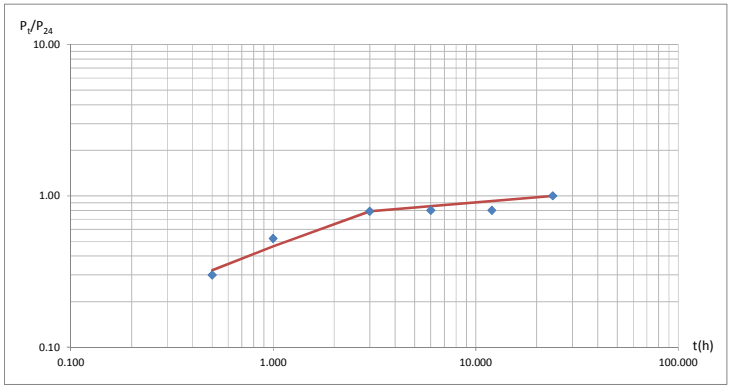


Figure 20 - Adimensional intensity-duration lines

These lines are described by the following equations:

- Pt/P24 = 0,6992 t0,1126 for 2,7 h < t < 24 h
- Pt/P24 = 0,4641 t0,5255 for t ≤ 2,7 h

These equations are also valid for P24 = 1 mm, if Pt/P24 is replaced by Pt.

So, knowing that for Dili, more or less at sea level, the 100 year return period 24 h duration precipitation is 200 mm (see section 5.2.1), we just have to multiply the previous equations by 200 to obtain the intensity-duration curves for T = 100 years at zero altitude:

- $P_t = 139{,}820 \times t^{0{,}1126}$ for 2,7 h< t < 24 h.
- $P_t = 92{,}818 \times t^{0{,}5255}$ for t < 2,7 h

For return periods other than 100 years, it was admitted that it is enough to multiply the values of Pt obtained by means of the previous equations by the following ratios, established in section 5.2.1:

$$\frac{P_{t50}}{P_{t100}} = 0{,}89\,;\ \frac{P_{t25}}{P_{t100}} = 0{,}79\,;\ \frac{P_{t10}}{P_{t100}} = 0{,}65\,.$$

CATCHEMENT AREAS CHARACTERIZATION

River System

The characterization of the catchment area of each river in analysis was performed following the same methodology.

In general, two catchment areas were considered for each river:

- The first corresponding to the total drainage area dominated at the river outlet;
- The second corresponding to the catchment area dominated by the upstream section of the river training works.

To determine the flood flows for each catchment area, it was necessary to estimate the corresponding concentration times. This was made applying the formulas of the Soil Conservation Service (SCS), NERC, Temez, Giandotti and Kirpich.

<u>SCS formula, NERC formula, Temez formula, Formula of Giandotti and Formula of Kirpich.</u>

The values obtained for each of the above parameters concerning each of the two catchment areas are presented in Table 6 below:

Table 6 Values of the parameters for the concentration time formulas

River	Catchment area	A (km²)	L (km)	S (%)	$S_{10\%-85\%L}$ (m/km)	hm (m)	J (m/m)
Maloa	Maloa 1	11,59	6,87	12,31	91,96	258,00	0,12
	Maloa 2	8,83	5,24	15,76	139,16	326,79	0,16
Kuluhun	Kuluhun 1	6,88	7,28	11,81	83,42	207,65	0,12

	Kuluhun 2	4,19	4,45	18,67	185,62	309,93	0,19
Bemori	BM2+BM1	10,40	6,93	12,33	125,00	338,54	0,12
	BM2	8,14	5,50	14,96	162,86	403,19	0,15
Becora	BC3+BC2 +BC1	18,63	9,03	9,77	86,43	370,34	0,98
	BC3+BC2	11,67	5,16	15,70	145,74	496,81	0,16
	BC3	9.59	4,86	15,65	154,32	522.60	0,16
Santana	SA	0,71	2,19	10,62	47,83	50,71	0,11

The curve number (CN) values were calculated based on the values presented in the tables edited by the SCS, considering the land use, the hydrological soil group (HSG) and the wet antecedent moisture conditions (AMC).
In the concerned study area, soils fall into the type "B" (shallow loess, sandy loam) Hydrological Soil Group and the land use is either "forest" or "urban/mixed" occupation.

The SCS CN depends on the soil Antecedent Moisture Conditions (AMC). AMC-III corresponds to a saturated soil, AMC-I to a dry soil and AMC-II to an intermediate situation.

In the present study AMC-III was considered for the 100 years return period and AMC-I for the 10 years return period. For the intermediate return periods (25 and 50 years) the CN number was linearly interpolated from the CN values obtained for the 100 and 10 years return period. The results are shown in the table below.

Table 7 Curve Numbers for each of the land use and return period considered

Ground occupation	CN			
	T=100	T=50	T=25	T=10
Urban/Mixed	94	89	87	85
Forest	82	74	69	66

For each catchment, the corresponding CNs for each return period was obtained by weighing the areas of each kind of land use. The results are presented in the table below.

Table 8 Curve Numbers considered

River	**Catchment**	**CN**			
		T=100	**T=50**	**T=25**	**T=10**
Maloa	Maloa 1	85	78	74	71
	Maloa 2	85	77	73	70
Kuluhun	Kuluhun 1	85	77	73	70
	Kuluhun 2	82	74	69	66
Bemori	BM2+BM1	83	76	71	68
	BM2	82	74	69	66
Becora	BC3+BC2+BC1	86	80	76	73
	BC3+BC2	82	74	69	66
	BC3	82	74	69	66
Santana	SA	84	77	72	70

Although calculation of concentration times by each of the methods described above were made, the SCS concentration times were adopted for the two catchment areas in study.

The table below shows the concentration times adopted for the return periods considered rounded to the next higher half decimal point.

Table 9 Concentration times considered

River	**Catchment**	**Concentration time (ct) (h)**			
		T=100	**T=50**	**T=25**	**T=10**
Maloa	Maloa 1	1,55	1,94	2,18	2,37
	Maloa 2	1,10	1,42	1,60	1,73
Kuluhun	Kuluhun 1	1,60	2,10	2,35	2,60
	Kuluhun 2	0,95	1,25	1,45	1,50
Bemori	BM2+BM1	1,60	2,00	2,33	2,58

	BM2	1,25	1,58	1,83	2,00
Becora	BC3+BC2+BC1	2,08	2,50	2,83	3,08
	BC3+BC2	1,17	1,50	1,75	1,83
	BC3	1,08	1,42	1,67	1,75
Santana	SA	0,67	0,83	0,92	1,00

Urban Drainage Network

The drainage network catchment areas are outlined in Drawing 2.

The characterization of all urban watersheds was performed following the same methodology.

It was established the area of each catchment and sub-catchments.

To determine the flood flows for each catchment area, it was necessary to estimate the corresponding concentration times.

The time of concentration (tc, in minutes) of the different sub-catchments was estimated applying the KIRPICH formulae, published on "California Culverts Practice" (1956), expressed as:

$$t_c = 57 \, (L^3/H)^{0,385}$$

Since the values of the concentration time of each catchment were similar, in order to standardize the calculations, it was adopted a concentration time of 15 min for all the catchment areas of all the channels. This value corresponds to a conservative value adapted to the various catchments.

ESTABLISHMENT OF PRECIPITATION HYETOGRAPHS

In order to define the precipitation hyetographs for 10, 25, 50 and 100 years return period the following information was taken into account:

- the intensity-duration curves general equations for return period 100 years.

$P_t = 139{,}820 \times t^{0,1126}$ para / for 2,7 h< t < 24 h.

$P_t = 92{,}818 \times t^{0,5255}$ para / for t < 2,7 h.

- the ratios determined for conversion of these curves for return periods other than 100 years in section 00:

$$\frac{P_{t50}}{P_{t100}} = 0{,}89 \; ; \; \frac{P_{t25}}{P_{t100}} = 0{,}79 \; ; \; \frac{P_{t10}}{P_{t100}} = 0{,}65$$

- the daily precipitation is, usually, concentrated within a period that seldom last more than 3 hours in a row with higher intensity at the beginning of the event as referred in 0.

Given this, the rainfall duration considered to compose the referred Hyetographs was of 3 hours and the distribution of the incremental precipitation blocks was made to be decreasing overtime.

The following procedures were adopted to compose the hyetograph:

- Applying the relation 0,133 x ct, where ct is the concentration time[5], to determine the duration of each precipitation block. This relation led to a precipitation block with 1.197 minutes duration for urban catchments and varying between 5 and 16 minutes in rivers catchments.
- To standardize the hyetographs the duration of each precipitation block was defined as 2 min for all urban catchment areas, corresponding to 0,133 x 15min.
- Regarding the rivers, the duration of each precipitation block was defined as 5 min for all the catchment areas.
- Each precipitation block, for a given return period, corresponds to the difference between the precipitation with a determined duration t and the precipitation with duration t-Δt.

The disposal of blocks was made, as showed in the Figure 21, by the following method:

- Place the biggest intensity block at the beginning of the hyetograph (with 2 min duration for urban network and 5 min duration for rivers).
- Distribution of the remaining precipitation with 3 hours duration in blocks also with same duration (2 or 5 min) arranged in descending order of precipitation intensity.

5 Nunes Correia, in "Alguns procedimentos adoptados pelo *Soil Conservation Service* para a determinação de caudais de cheia em pequenas bacias naturais e urbanas", defines that the precipitation block to consider in the construction of the Hyetograph should have a duration no bigger than approximately 0,133ct.

All of the precipitations were corrected in accordance with their duration and the area of the catchment, as proposed by Chow et al (1988)[6].

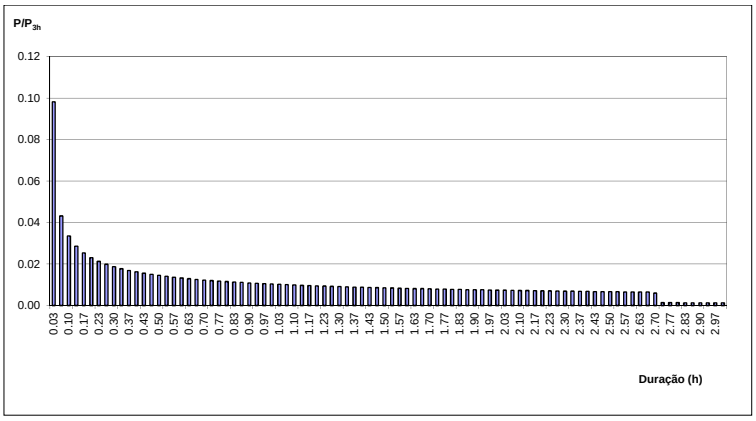
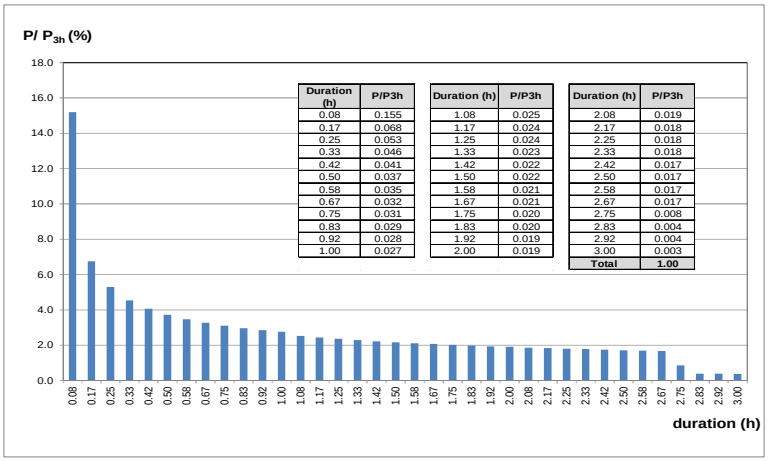


Figure 21 - Adopted distribution of the precipitation blocks (urban catchments)



Duration (h)	P/P3h
0.08	0.155
0.17	0.068
0.25	0.053
0.33	0.046
0.42	0.041
0.50	0.037
0.58	0.035
0.67	0.032
0.75	0.031
0.83	0.029
0.92	0.028
1.00	0.027

Duration (h)	P/P3h
1.08	0.025
1.17	0.024
1.25	0.024
1.33	0.023
1.42	0.022
1.50	0.022
1.58	0.021
1.67	0.021
1.75	0.020
1.83	0.020
1.92	0.019
2.00	0.019

Duration (h)	P/P3h
2.08	0.019
2.17	0.018
2.25	0.018
2.33	0.018
2.42	0.017
2.50	0.017
2.58	0.017
2.67	0.017
2.75	0.008
2.83	0.004
2.92	0.004
3.00	0.003
Total	1.00

Figure 22 – Adopted distribution of the precipitation blocks (rivers catchments)

CALCULATIONS OF FLOOD HYDROGRAPHS

Considering the precipitation hyetographs obtained as described in chapter 0, the subsequent flood hydrograph for the catchment areas were calculated.

The flood hydrographs for this catchment area were determined by convolution of their respective hyetographs, with the unit synthetic hydrograph of the Soil Conservation Service (SCS).

This was performed using the HEC-HMS model developed by the U.S. Army Corps of Engineers.

The unit hydrograph of the SCS is characterized by the lag time, given by the expression:

Lag = 0,6 x ct

Where, Lag is the lag time and ct is the concentration time of the catchment area.

The lag time for each catchment area was determined considering the concentration time values defined in next point.

This lag time and the Curve Numbers (CN), obtained as described in next point, were considered in the model.

The results of the model for each of the catchment areas and return periods were the flood hydrographs that can be analysed in more detail in the DED's reports.

URBAN DRAINAGE NETWORK INTERVENTIONS - FUNDAMENTALS

As previously stated, the detailed design projects relate only to the channel's identified as priorities in the DSDMP's Phase 1 report.

6 This correction takes in to account the fact that the precipitation intensity with a given duration is not uniform in the entire catchment area and this non-uniformity is more relevant as larger is the area of the catchment and as smaller is the duration of the precipitation.

For each channel, the following steps were taken while developing the detailed design projects:

- Data collection and updating the existing database;
- Define the boundaries of the drainage sub catchments;
- Design the solution;
- Size the solution;

In the following subchapters, each one of these steps is described.

CATCHMENT DEFINITION

To assess hydraulic performance in the existing drainage network and develop adequate hydraulic solutions, drainage sub catchments have been defined as well as their inflow nodes.

Drawing 2 shows the defined drainage sub catchments and the correspondent inflow nodes.

The boundaries of each sub catchment were defined combining topographic, rivers and cadastral data, complemented with information gathered during field surveys.

Each sub catchment has its own properties that have direct influence over the volume of runoff produced. The most conditioning properties are:

- Total area of sub catchment;
- Length of overland flow path;
- Average surface slope.

URBAN DRAINAGE DESIGN SOLUTIONS

General Fundamentals

Looking at Dili's ground surface, it's notable how the mountains rush over the urban area with a very steep slope, promoting runoff from the mountain areas to the urban area. So, the first requirement of Dili's drainage network to avoid flooding is the interception of runoff coming from the mountains before it gets to the urban area.

Figure 23 - Dili's Ground Surface Model

Although these works are not provided in DSDMP's Phase 1 report, or in this contract, it was considered essential that hill side interceptor channels are built, namely in Suco Mascarenhas, so they can collect runoff from the mountains and drain it to the rivers.

Figure 24 - Suco Mascarenhas's Hill Side Interceptor Channel Location

In the detailed design projects, it was assumed that runoff from the mountains is collected and doesn't reach the urban area.

Despite Dili's drainage network, as previously stated, is undersized (it's sized for rainfalls less than 5 years return period only, according to DSDMP's Phase 1 report), the proposed solutions incorporate the existing network as much as possible.

Fundamentally, the detailed design projects define major channels that will shape the drainage network and drain runoff along these channels to the rivers or sea, and the links between the existing drainage network and the new drainage infrastructures.

After a diagnosis of the drainage system and the identification of critical points, coordination meetings with DNSB were held in which the following assumptions were set:

- The sizing flows estimated in Phase 2, for the same return period, are higher than the hydrological assessment in Phase 1;
- While designing the solutions, a 25 years return period was adopted, despite DSDMP's Phase 1 report refers to return periods of less than 10 years and although international literature recommends return periods between 10 and 20 years. It was a prudent basis given the characteristics of the urban area and poor maintenance;
- Urban space limitations (narrow streets and private land) forced us to adopt more expensive underground solutions in some places.
- The existence of severe constraints, like already built infrastructure that cannot be intervened, forced us to divert flows to other channels, overburdening them. This is the case, namely, of the diversion from channels D5 and D6 to channels D3 and D4.
- The construction of new sea outlet structures is also necessary, because the existent structures are silted up.
- Avoid or eliminate, as much as possible, section channels crossing private land.

To simplify the description, the following three areas have been distinguished, as shown in the figure below, according to the king of urban occupation and solutions to implement:

- Central Dili;
- West Dili;
- East Dili.

Figure 25 - Dili's Areas

There are, however, some measures that shall be applied no matter the area, namely:

- Keep in operation, as much as possible, the existing drainage infrastructures;
- Cleaning and clearing the existing drainage infrastructures;
- Improve flow conditions in junctions where channels meet;
- Re-slope invert slop sections;
- Rebuild hydraulic culverts and walkways that reduce channel's cross section;
- Replace poorly maintained channel's sections.

Central Dili

Central Dili is a flat area with a high rate of population and public and governmental buildings. It's also the most critical area in terms of flooding and where the biggest space constraints are, due to the high rate of construction.

As previously stated, the drainage solutions were developed assuming that runoff coming from mountains doesn't reach urban areas.

According to site investigations and interviews with locals, downtown urban floods have a strong contribution coming from the overtopping of Kuluhun's banks. During heavy rains, river flow spills over the riverbanks and

It was assumed that Kuluhun's river overspill near Suco Mascarenhas is solved after river training works that have been considered priority.

The following channels are inside Central Dili

- D1, D2 and branches, D3 and branches, D4, D5, D6, D7 and branches, D8, D9 and branches, D10 and branches, D11.

From the referred major channels, only Channel D7 discharges into a river (Kuluhun River). All the others discharge into the sea.

Figure 26 - Central Dili's channels that discharge into the sea

Aside the space constraints in this area, due to the intense construction, there are also constraints to intervene in the most recently built channels. The major channels with constraints are:

- Channel D5, for it's been built during the developing of detailed design projects;
- Channel D6, for it's recently built;
- Channels D9 and D10, due to space constraints that prevent their widening.

This means that from the ten available sea discharges, only six (D1, D2, D3, D4, D8 and D11) can be enlarged.

Channel D3 is directly linked to channels D5, D6 and D10. It's also linked to channel D9, indirectly, through channel D10.

Channels D8/D11 and D1/D2 are located in the ends of Central Dili and work as independent channels, as they aren't connected to the other channels. This means that Central Dili area as to be conveyed mostly through channels D3 and D4.

This fact strongly affects the final solution because it reduces sea discharges to two outlets and the flow must be conveyed that way. Channels D3 and D4 become, for that matter, the "backbone" of the drainage network, as well as their branches. Their branches will convey flow diverted from channels with restrictions to channels D3 and D4.

The final solution includes the following interventions:

- Channel D1: remains as an independent channel with its own discharge;
- Channel D2: DNSB requested its extension to 15 de Outubro Street. This extension allows to intercept flow that is currently conveyed to channel D9. Branches D2.1 to D2.3 collect flow diverted from channel D7;
- Channel D3: Along with channel D4, it'll be part of the drainage network "backbone". DNSB requested its extension along Bispo de Medeiros Av. up to the roundabout southwest of the stadium. Its branches collect flow diverted from channels D10 and D9;
- Channel D4: along with channel D3, it'll be part of the drainage network "backbone". DNSB Its branches collect flow diverted from channels D6 and D3;
- Channel D5: none intervention planned beside the adaption of the link to channel D3 because of the modifications this channel will suffer;
- Channel D6: diversion nodes to channel's D4 branches will be built and it'll be necessary to re-slope a section next to Caicoli's retention basin (RB1). It's important to remember that even diverting all the upstream flow, the underground sections of channel D6 don't have enough capacity to convey the tributary flows;
- Channel D7: only improvement works are planned and diversion nodes to channel's D2 branches will built. The flow diversion assures that the channel has enough capacity to convey the tributary channels;
- Channel D8: it'll be extended so it can receive channel's D11 flow, upstream the crossing with D. Aleixo Corte Real Street;
- Channel D9: because of space constraints, it can't be enlarged. For that reason, all flow from upstream Jacinto Cândido Street will be diverted to channel's D10 branches with link to channel D3;
- Channel D10: because of space constraints, it can't be enlarged. For that reason, all flow from upstream Jacinto Cândido Street will be diverted to channel's D3 branches;
- Channel D11: it'll be divided in two parts: upstream and downstream the crossing between D. Aleixo Corte Real Street and Abílio Monteiro Street. Downstream the crossing, only improvement works and a few re-slopes will be made. Upstream the crossing, a box-culvert will be build, side by side with the existing channel.

Along with the construction of underground sections will occur the construction of street inlets and links to the box-culvert, as well as diversion chambers to link the existing open channels to the new box-culverts. All box-culverts will have manholes for maintenance.

West Dili

Although it has a smaller rate of building occupation than Central Dili, West Dili has the same flat areas and runoff coming from the mountains problems.

According to site investigations and interviews with locals, downtown urban floods have a strong contribution coming from overtopping of Maloa's banks. During heavy rains, due to Maloa dam siltation, river flow spills over the riverbanks and spreads through the streets, worsening the flooding.

It was assumed that Maloa's river overspill was solved after river training works that have been considered priority.

It was also assumed that runoff from mountains doesn't reach the urban area because it's planned that some flow goes to channel D12A (external to the contract) and some flow to retention basin RB3.

The following channels are inside West Dili:

- D12, D14 and branches, D15 and branches, D18, D19.

Channel D12 is an open channel that goes along the hillside, between buildings and vegetation, and discharges into Maloa River. It'll be kept in operation as an open channel but it must be enlarged, deepen and reshaped.

Channel D14 is a large open channel that goes from an agricultural area towards the ocean, bordered by buildings/constructions downstream Almirante Américo Tomás Av. This channel will suffer some improvement interventions, as well as branches D14.1.1 and D14B.

Channel D14.1 will suffer major interventions both in terms of layout and structure. The current layout is very sinuous and that hinders the flow conditions in the channel, turning the flow more turbulent. The new layout is less sinuous but requires the demolition of some existing buildings/constructions. In terms of structure, the channel will have some open trapezoidal shape sections and some box-culvert sections.

Similar to channel D14, channel D15 is also a large open channel, bordered by buildings/constructions. This channel will only suffer some improvement interventions, as well as branch D15.1.

Channel D18 and D19 are currently independent channels with individual discharges into the sea.

Channel's D19 discharge is poorly maintained and is fully silted, so a new one has to be built. The existing channel D19 will be kept in operation but a new box-culvert will be built side by side. This new channel will convey diverted flow from channel D18.

Flow diversion to channel D19 will allow channel D18 to be kept in operations only with some improvement interventions.

It's important to highlight the role of two channels that are supposed to collect runoff from the mountains, namely channel D12 and RB3's inflow channel, both external to the contract. It was assumed that these two channels will convey all runoff from the hillside of Desa Bairo Pite.

Retention Basin RB3, in Desa Bairo Pite, is also a key component of the drainage network since much of the runoff from hillside of Desa Bairo Pite will be conveyed to it. The outlet discharges into a new channel (D16A as it was called) that will be built and linked to the sea. This channel is also external to the contract.

Along with the construction of underground sections will occur the construction of street inlets and links to the box-culvert, as well as diversion chambers to link the existing open channels to the new box-culverts. All box-culverts will have manholes for maintenance.

East Dili

Similar to West Dili, East Dili also has a smaller rate of building occupation than Central Dili and the same problem with flat areas, although a little more steeper.

Channels D21 to D32 are inside East Dili area.

They are all independent from each other.

Except channels D21 and D22 that discharge into the sea, all the others discharge into rivers.

Because of runoff from mountains, a hillside interceptor channel shall be built in Desa Bidau Santana hillside. This channel is supposed to discharge into the sea through channel's D21 discharge. This channel is external to the current contract.

The final solution includes the following interventions:

- Channels D22 to D24 will only suffer improvement works;
- Channels D25, D27 and D31 will have box-culvert sections side by side with the existing channels;
- Channels D26, and D32 will be enlarged;
- Channel D28 will be built as a rectangular shape open channel;
- Channel D29 will be extended and the existing sections will be improved and kept in operation;
- Channel D30 will be extended, enlarged and kept in operation. Branch D30.1 will also be enlarged;
- Channels D31 and D32 will have box-culvert sections to increase flow capacity;

- Channels D33 and D34 will be enlarged in the downstream sections and the existing sections will be improved and kept in operation;
- Channels D35 and D35.1 will be enlarged in the downstream sections and the existing sections will be improved and kept in operation.

PROPOSED SOLUTIONS

For hydraulic assessment and performance, United States Environmental Protection Agency's software for hydraulic modeling SWMM – Storm Water Management Model has been used.

Based on this software and on internal process calculations, both the current network performance and the network performance after the implementation of the proposed interventions will be simulated.

Besides the network layout, the model building requires the following data:

- Junctions elevation;
- Channels cross section;
- Manning's roughness coefficient;
- Inflows;
- Boundary discharge conditions (tidal variations);
- Other nodes relevant to the network.

The elevation data was collected via LIDAR surveys and the channel cross sections via LIDAR/network surveys.

The Manning-Strickler formula was applied for checking the hydraulic flow conditions:

$$Q = KSR^{2/3} i^{1/2}$$

The drainage channels have different coverings and have generally poor maintenance. To simulate these differences, the following Manning's Coefficient has been considered:

- Open channels in natural ground, n= 0.07;
- Open channels with concret covering, n= 0.035;
- Underground pipe culverts/box-culverts, n=0.015.

The inflow hydrograph is another input to the hydraulic modelling. The inflow hydrograph in each node was determined in the HEC-HMS modelling, for further information see the Hydrological Study.

Under the conditions of downstream, discharge into ocean, tidal level variations was considered. The time series was defined based on the tide timetables provided by the Port of Dili, adjusted to LIDAR survey.

In the maritime hydrographic datum, the referred high tide level is 2,30. Since the LIDAR survey used in the present design is located 1,9 m above the hydrographic datum, the elevation used in the calculation of backwater curve was 2,30 - 1,90 = 0,40 (cross-section 2+873,42).

RIVER TRAINING WORKS – FUNDAMENTALS

The hydraulic analysis of the present situation for each river[7], conclude:

- The section width varies along the rivers more as function of urban encroachment than as a consequence of natural movements of the rivers bed and banks;
- The existing bridges limit the already insufficiency of the present rivers bed conveyance;
- At the river's mouth the capacity is very small because the available bed slope becomes virtually flat;
- The current cross section of the rivers bed will have to be significantly increased to reach a centennial peak discharge capacity;
- In some sections, a significant widening of the river bed is not socially feasible, because of the intense urban occupation of the rivers banks;
- The main intervention will have to be the deepening of the rivers bed.

GENERAL CONCEPT

As mentioned above, all the bridges that exist along the rivers reach's to be trained will have to be, either replaced, or somehow adapted to fit the new rivers training works. Nevertheless, the new bridges are not included in the present design and, so, they will have to be the object of separate designs, to be produced separately.

In the present design, the river training works have been conceived and dimensioned as if no bridges existed at all. This implies that the new bridges must not interfere with river flow in any way.

The constraints to which the new bridges designs will have to obey are given in detail in the DED's reports.

[7] A more detailed analysis for each river can be consulted on the DEDs report.

As to the river training works themselves, it was concluded that the current cross section of the rivers bed will have to be significantly increased to reach a centennial peak discharge capacity. Theoretically, that could be achieved in one or both of two following ways: widening or deepening of the rivers bed.

With the exception of Becora river, significant widening of the rivers bed is not socially feasible, because of the intense urban occupation of the rivers banks.

So, the main intervention will have to be the deepening of the river bed where the needed excavation depths range from:

- zero upstream to a maximum value of about 4 m downstream, in Maloa river;
- zero upstream to a maximum value of about 2,5 m downstream, in Kuluhun river;
- 1 m upstream to a maximum value of about 3,5 m downstream, in Becora/Benamauk river;
- 1,5 m upstream to a maximum value of about 2,5 m downstream, in Bemori river;
- 3,5 m upstream to a maximum value of about 4,5 m downstream, in Santana river.

The new longitudinal profiles have longitudinal slopes becoming progressively flatter from upstream to downstream.

The slope specified for the end reaches is 0,2 % on Maloa river and 0,5 % on Kuluhun and Santana rivers. These slopes are meant to eliminate the fluvial delta that exists today in the river's mouth and that is responsible for bank overtopping in the lower reach.

As to the river width, it was decided to establish a constant value all along the intervened reach which have sufficient size to avoid excessive excavations and not involving the definitive destruction of a large number of urban houses as being excessive.

For each river, the establish width value were the following:

- Maloa – 12 m
- Kuluhun – 10 m
- Becora/Benamauk – 12 e 15 m
- Bemori – 12 m
- Santana – 20 m

It must also be stressed that these widths must be strictly respected all along the intervened reaches, no matter what the implications will be for the urban occupation of the side banks.

The main reason for this is that the rivers carry a considerable amount of sediment load during major floods, which has the tendency to settle down as a consequence of progressively decreasing slopes and velocities when the rivers approaches its mouth. In its turn, sediment settling causes the rivers bed to rise, leading to bank overtopping.

This is particularly important in the Maloa, Kuluhun and Santana rivers final reaches, which will extend their river bed onto the ocean itself. To avoid precocious sediment settling at the river mouth, bringing back the existing delta shaped clogging of that mouth, it is essential that the width and, therefore, the high flow velocities, are kept until well inside the ocean, where the sea currents will disperse the sediments in a harmless manner.

Because of this, sea outlet structures will have to be built, with massive side guiding walls, resistant to sea storms agitation.

BANKS RETAINING WALLS

The constant width of the Maloa, Kuluhun, Bemori e Santana river training works (excluding the sea outlet) will be ensured by vertical concrete walls (Figure 27) that will totally replace the existing masonry or concrete walls.

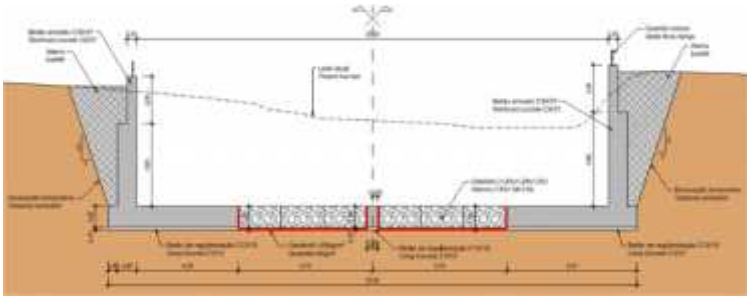


Figure 27 – Classic concrete retaining walls

Concerning the Kuluhun river, two types of walls will be used:

- Classic concrete retaining walls, with forward foundation aprons (instead of backward), meant to reduce lateral excavations.
- Special Berlin wall type retaining structures, which do not require any lateral excavations at all (Figure 28).

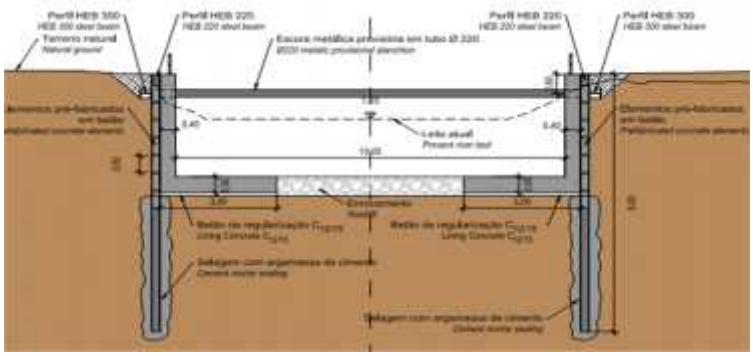


Figure 28 – Special Berlin wall type retaining structures

The second type is considerably more expensive than the first. So, where to apply one or the other was discussed in due time with the Timorese authorities.

The zone where the Berlin wall type solution will be applied is illustrated by Photos 88 and 89 and the objective is to keep the integrity of the blue colored building.

There will be two types of bank retaining walls along the Becora/Benamauk trained river bed reaches:

- Gabion walls along the left bank of a reach;
- Vertical concrete walls along both banks of the constant 15 m wide reach.

These will totally replace the existing masonry, concrete or gabion walls.

It will be impossible to keep any of the existing walls because their foundations are too superficial to cope with the intended deepening of the rivers bed.

In all the intervened reaches, where the classical walls solution will be applied, three different types of situations will occur:

There are no buildings within the new width, plus the side excavation widths.

There are no buildings within the new width, but they exist within the side excavation widths.

Buildings exist within the new width.

In the first situation, no buildings demolitions or resettlements will have to be carried out.

In the second situation, buildings will have to be demolished, but their space will be recovered in the end, and, so, it will be possible to rebuild them, if it is considered adequate (temporary resettlement).

In the third case, houses will be permanently demolished, calling for permanent resettlement of the inhabitants.

This has been recognized and accepted by the Timorese authorities. Anyway, there are places where a few marginal houses and retaining walls are already semi destroyed, due to previous floods.

It must be emphasized that the structural dimensioning of the concrete walls, classic or Berlin type, has been carried out in the present designed in only an approximate way. So, final geotechnical and reinforced concrete design must be carried out by the contractor itself, in a design/build procedure, or by means of an intermediate complementary design.

The same applies to all singularities, like openings in the walls for urban drainage pipes or channels. It must be stressed, though, that earthfill access ramps built into the river bed, like the one illustrated by Photo 89, must be totally eliminated.

The trained river bed will be partially lined with smooth concrete slabs protruding from the toe of the retaining walls on each bank – see Figure 28.

This central strip will be lined with rockfill. Every 2,5 m, this strip will be crossed by concrete beams, that will hold the walls toes against each other.

The pervious rockfill will limit hydraulic under pressures beneath the walls and its high roughness will prevent flow velocity to reach excessive values.

SEA OUTLET STRUCTURES

The river hydraulic design of the Maloa, Kuluhun and Sanatana rivers sea outlet structures is final and must be strictly respected by the contractor, in what concerns hydraulic length, width, bottom slope and wetted perimeter lining.

Namely, it must be emphasized that the whole sea outlet reach must have its bed fully lined with smooth cyclopean concrete, to compensate for velocity reduction caused by high tides, thus facilitating the expulsion of the sediment bed load into the deeper ocean, to avoid clogging of the outlet.

As to the structural design, it must be stressed that it has not been carried out within the scope of the present project, the works having been predefined just by means of mere similarity with other works of the same type in other regions.

The reason for this situation is that the design contract did foresee neither the time, nor the budget, to make the sea agitation studies and to design the structure to withstand them.

The solution is based on submersible coffins of precast concrete staves filled with rockfill is presented. Nevertheless, a different solution proposed by the contractor may be accepted, as long as its capability to withstand a 100 years return period sea storm is demonstrated.

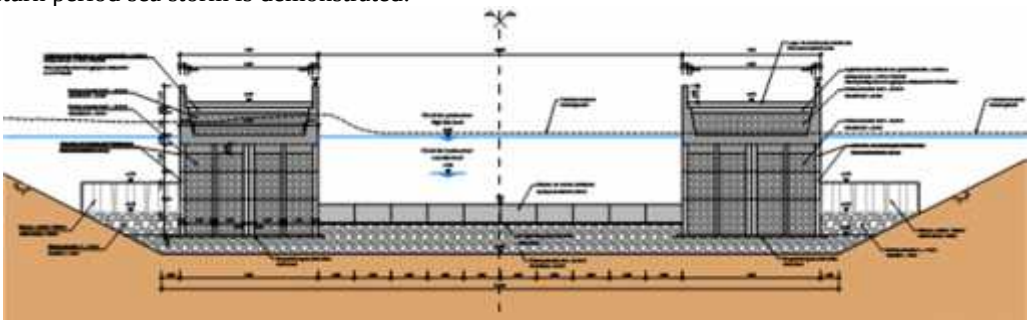


Figure 29 – Sea outlet structures

ESTABLISHMENT OF PRIORITIES

Given the high costs estimated for the works proposed in the detailed design projects and the expected costs for the construction of other necessary infrastructures, yes external to this contract, a justified hierarchy of priorities was established, so that investments can be distributed along time, putting the most effective and urgent up front. So, we propose the following sequence of priorities:

1. Complete Kuluhun river training works (including bridges replacement where needed).
2. Slope interceptor channel.
3. Demolition and desilting of Maloa and Becora retention weirs.
4. Complete Maloa river training works (including bridges replacement where needed).
5. Central zone urban drainage channels (Group 1 Channels).
6. Complete Santana/Becora/Becosi river training works (including bridges replacement where needed).
7. Remaining urban drainage network (Group 2 Channels).
8. Retardation basins.

The choice of Kuluhun training works as first priority results from the fact that, according to our site investigations and interviews, many of the downtown urban flooding situations have a very strong contribution coming overtopping of the banks of this river.

Similarly, the slope interceptor channel is very important, not only to ease the otherwise unsolvable problems of Suco Mascarenhas, but also to strongly reduce the amount of water that is conveyed to the downtown area, coming from the mountainside above Dili.

Demolition and desilting of the existing weirs is also considered a priority, because it is relatively inexpensive, and because:

- Becora dam is structurally unstable and its sudden rupture during a flood can cost lives.
- Maloa dam is less unstable, but it is causing right bank spillage that is being diverted right into the Caicoli urban area.
- Both dams have a flood dampening of less than 5%, which does not compensate the risk and the spillage effects.

Maloa river training works, although expensive, would be next priority, because, without cutting its overtopping, improvement of urban channels will not be sufficient.

Santana, Becora and Bemori rivers have not been given an high degree of priority, because the needed investment is quite high and because it does not affect the central core area of the city.

Retardation basins have been considered last priority, because it will imply important resettlement and land acquisition costs.

PROPOSALS FOR THE NEXT STEPS

The interventions proposed in the detailed design projects will not eliminate the occurrence of floods, but only reduce their impact and likelihood of occurrence.

However, in what concerns the urban drainage network, as previously stated, the increase in capacity for rainfall with less than 25 years return period is not in itself enough to minimize the occurrence of floods.

While developing Phase 2, some new important projects have been identified and should be developed so that their execution could accompany or precede the works already proposed in the detailed design projects. Those additional projects include:

- Bemori and Becora/Benamauk bridges replacement;
- New Drainage structures
  - Hillside interceptor channels in the mountains overlooking the Bidau - Santana and Bairro Pite areas.

- Enlargement/building of other drainage channels foreseen in DSDMP's Phase 1 (for instance, D12A, RC4, TS, D36, D16A, D17C, D20/airport);
- Other retention basins foreseen in DSDMP's Phase 1 (for instance, RB4 and RB5);
- New retention basins in de West area of Dili;

- River Manleuana diversion directly to Rio Comoro;
- Study erosion control works, to be built in the upper parts of the Kuluhum, Maloa, Benamauk and Bemori rivers catchment areas (as a matter of fact, although the lower river reaches training works designs have been made in a way that reduces sediment settling as much as possible, the need to build complementary erosion control works in the future was recognized, in order to reduce the bed load that enters the lower trained reaches).

It's clear that these works should be designed together with the hydraulic solution for river training and channels drainage. Also, when launching the tender, the works' scope should be as much detailed as possible.

✓ ***Complementary Works***

Despite the detailed design projects already developed, it's essential to take other measures, behaviours and tasks that contribute, in the long run, for a good hydraulic performance, operation and maintenance of the existing and new infrastructures. Some of those measures are:

- Create a single and updated cadastral register of the drainage network;
- Cleaning and desilting of drainage systems to allow thorough survey of the existing issues;
- Prepare or require *as-built* drawings of every drainage work as they are being built.
- Ensure the implementation of the drainage network operation and maintenance procedures.

The DEDs identified the main critical points of the drainage system and improvement measures are presented in detail.

However, given the difficulty of establishing effective coordination with all entities involved in the urban area of Dili it was not possible to define some concepts.

✓ ***Coordination with other Public Authorities***

Thus, there is a set of questions remaining still to be studied with the Urban Planning and the Territorial Planning.

- Road cross-section type.
- Definition of roadsides features: sidewalks and location of sinks / collectors or ditches / channels
- Drainage design of the future allotments and housing to be built along roadways to collect flows.
- Drainage design of the future allotments and housing to be built along roadways to collect flows.

✓ ***Coordination with dealers***

On the other hand, contacts with service providers' entities should be established to remove or divert existent infrastructures in order to eliminate:

- Obstructions caused by infrastructure that occupy the channel cross-section section.
- Obstructions caused by pedestrian / road passes over the channels that occupy part of channels cross section.
- Obstructions caused by infrastructure that occupy hydraulic passages cross section.

Details concerning reinforcements/replacement of bridges, containment works (walls), structures of large manifolds and maritime outflow points are not included in this project.

The same is applied to new designs, those identified during Phase 2, such as the slope of interceptors, new channels and collectors for drainage areas not initially foreseen.

In spite of not included in the Phase 2 Masterplan's scope, this phase includes a fully detailed hydraulic definition where the most complex structural works (sea outlet structure, new bridges and containing walls) will be launched for tender as design/build nature, which means that the Contractor will be responsible for the detailed structural dimensioning, following scrumptiously the hydraulic design defined by the Consultant Team.

WARNINGS AND RECOMMENDATIONS

Some warnings and recommendations must be weighted and summarized at the end of the present report, although many of them have also been given along the several chapters above.

Such warnings and recommendations concern the following features:

A. Topography

The present design was carried out upon a general LIDAR survey, complemented with local classical surveys of the bridges. The datum of these surveys is 1,9 m above the hydrographic datum. Given that the mean sea level is 1,40 in this latter datum, one concludes that the mean sea level in the LiDAR survey datum is 1,40 - 1,90 = - 0,50, which is a little bit odd (usually, it should be zero). Furthermore, because of trees and the housing along the river, the LIDAR survey may diverge a bit from reality here and there. So, complementary detail topographical should be made by the contractor during construction.

B. Bridges

Bridges are not part of the present design and they will have to be the object of separate (but not independent) designs. The constraints to be superimposed to those designs are given in the DEDs reports. Nevertheless, it must be noted that the construction of the training works and the new bridges must be simultaneous, or at least, very closely coordinated. The reason is that the construction of the training works implies demolition of the existing bridges. This could be accepted temporarily, but, then again, when the new bridges would be built, they would imply the demolition of parts of the training walls, which, obviously, would not be a good management of the available resources.

C. Sea Outlet Structure

Bathymetry: The present design was made using the bathymetry in the navigation map Aus 901. This might not be detailed enough for a final design. So the carrying out of a dedicated and detailed bathymetrical survey of this restricted area is highly advisable.

Sea Agitation Studies: Due to lack of time and budget, no sea storm agitation studies were made within the scope of the present deign. So, they must be performed within the design/build contract to be launched.

Structural Design: Verification of structural design of the overall sea outlet structure should, therefore, be included in the design/build contract too, as a function of the results of the agitation studies.

D. Hydraulic Design

In what concerns river hydraulics, the present design is considered final and, basically, no changes should be introduced in it by the contractor, without previous acceptance of the Timorese authorities, who, in their turn, are strongly advised to ask for the Consultant opinion before making a decision. Parameters that should not be changed without careful consideration include cross-section depth and width, bottom slope, curvature radius of the bends, lining characteristics and rockfill grading.

E. Network Maintenance and Cleaning

The regular maintenance and cleaning of the drainage network (open channels and underground collectors) is crucial to reduce the frequency and level of floods in the city and to ensure the proper functioning of the drainage system.

Thus, simultaneously with the channels execution works, the DNSB should implement a Maintenance and Clearing General Plan of the drainage network, on a monthly basis or lower, using the suitable human resources and proper equipment. After heavy rainfall, the system should be cleared immediately.

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Modelation Software:

HEC-HMS (Hydrologic Engineering Centers - Hydrologic Modeling System), US Army Corps of Engineers

HEC-RAS (Hydrologic Engineering Centers - River Analysis System), US Army Corps of Engineers

SWMM (Storm Water Management Model), EPA (Environmental Protection Agency)

SLOPING BREAKWATER WITH TAPERED CHANNEL AS A WAVE ENERGY CONVERTER

Asrini[1], A. Thaha [2] and F. Maricar [2]

ABSTRACT: The aim of this study is to determine the effect of wave steepness (Hi/L) and tapered channel parameter (shape and relative width) against the overtopping discharge on the sloping breakwater that be able captured for electricity power purpose. The wave tapered channel is considered to enhance the overtopping discharge capacity. Research has been conducted by 2D physical model simulation with scale 1:20 at The Coastal Engineering Laboratory, Civil Engineering Department, Hasanuddin University. Sloping breakwater models in three types of tapered channel, namely linear (LN), concave (NL1) and convex (NL2) were simulated by the various wave height and period. The experimental result showed that the breakwater with linear wall tapered channel gives the greatest overtopping discharge compared to 2 other models.

Keywords: *tapered channel, overtopping discharge, sloping breakwater, wave energy.*

INTRODUCTION

The main purpose of the coastal structures (seawall, dike, revetment, breakwater etc.) is to protect land and properties from the wave attack, still storm waves cause overtopping of these structures and endanger lives and coastal infrastructures (Mozahedy, 2010). Breakwater consists of obstacles in front of it to enhance the wave energy dissipation and to reduce wave loadings and related damages. In contrast, wave energy be able to use to converted be electrical energy. Overtopping depends not only on wave parameters such as wave height, wave period, wavelength, water level, but also on geometric layout and material properties of the structure (Soliman, 2003).

Wave-overtopping refers to the process where the waves run up a structure's face and, if the crest level of the structure is lower than the wave run-up height, overtop the structure (Kofoed, 2002). However many researchers have been trying to develop methods and formulas in order to predict overtopping discharges of coastal structures on certain possible conditions. These prediction formulas are widely varied and results in wide variations. Some previous studies have classified wave energy devices, according to their capture method shape and method of front-end converter movement (Taware et al, 2010). Overtopping devices are water reservoir, which collect water through wave overtopping and generate potential hydraulic head. This hydraulic head drives a turbine to produce electricity.

This research will be focused on method of catching the overtopping wave on the sloping breakwater with wave energy converging guiding walls. Wave energy converging guiding walls is located in sloping-front of the breakwater that have a function to guide sea waves go to the reservoir. This study was conducted with a calculated amount of water volume on reservoir in which the energy of incoming waves is stored as potential energy. Then, the captured water runs through turbines for electricity production.

LITERATURE

Based on the small-amplitude wave theory, the total energy contained in wind waves is the sum of kinetic energy and potential energy (Triatmodjo, 1996). The wave kinetic energy is energy caused by the velocity of the water particles due to the wave motion. While the potential energy is the energy generated by the movement of the water level due to the wave exists.

The kinetic energy of a single wave per unit width can be written as:

$$E_k = \frac{°gH^2L}{16} \tag{1}$$

Where: g = gravitational acceleration (9.81 m/s^2); p = the density of sea-water (assumed in this study as 1000 kg / m^3); H = wave height (m); L = wavelength (m).

And potential energy can be written as :

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$$E_p = \frac{\rho g H^2 L}{16} \tag{2}$$

P wave is a given wave's power at the structure toe and can be calculated using the standard power equation, as follows :

$$P_{wave} = EC_g \tag{3}$$

Where: E = energy density per meter width and Cg = the group velocity (m/s)

The potential power of the overtopping water per meter (P overtopping) can therefore be expressed as follows (Kofoed, 2002) :

$$P_{overtopping} = qg\rho R_c \tag{4}$$

Where: q = the overtopping rate (m^3/s/m); g = gravitational acceleration (9.81 m/s^2); p = the density of sea-water (assumed in this study as 1000 kg / m^3) and Rc = height crest freeboard.

EXPERIMENTAL METHODS

Experimental Set Up

The experiments were carried out in a wave flume in Civil Engineering Hydraulic System Laboratorium, Hasanuddin University, Gowa, Indonesia. A wave flume is 15 m long, 0.3 m wide, and 0.45 m high. Wave flume have been completed with a wave generator (type flap) to generate regular wave, stroke as wave height controller, and pulley as wave period controller. In the physical experiment, three sloping breakwater models with slope 1:2 are applied on water depth is 25 cm. Models are made from acrilyc with a thickness 4 mm. The model layout is shown in Fig.1. Each of models is conducted with 5 variation if wave height (Hi around of 3 cm – 7 cm) and wave period (T around of 0,8 s-1,2 s). Wave height and period is measured in manual with nine rulers on front of the models. The breakwater models is completed with a reservoir to stored the captured water as result of overtopping wave through run-up mechanism on wave energy centralized walls. The water that goes into the reservoir will make a difference of height between the reservoir water level to the sea water level will be used to turn turbines. In the reservoir, there is a bar with measure for measuring water height captured. Measurement of water height in the reservoir to do every 20 seconds for 2 minutes. Figure 1 shows experimental setup.

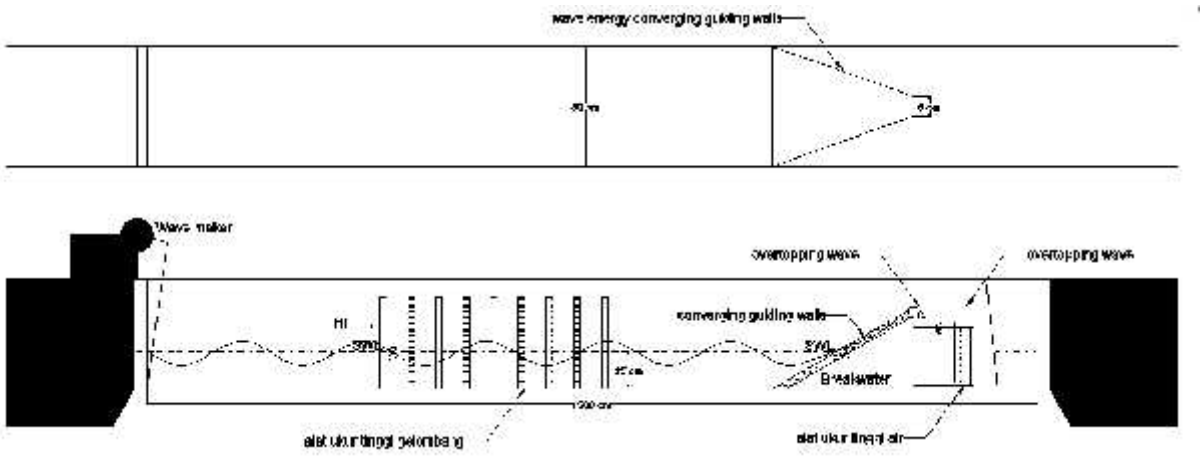


Fig.1 Experimental setup

Experimental Instrument and Measurement

In this section, laboratory experiments are carried out to measure the overtopping discharge from three sloping breakwater models with different of shapes of wave energy converging guiding walls. Figure 2 shows the shape of wave energy guiding walls on three models such as linear (LN), concave (NL1), and convex (NL2). Each of model is tested by variation of wave height (Hi) and wave periode (T).

Before the measurement, particularly it is need calibration on wave maker by controlled of stroke and pulley. Its function is to get wave height and period has been determined. Model test is placed in wave flume. After all component is ready, simulation starts with generated wave in wave flume. Wave height and period are measured at 9 point of measurement and are noted in the table of measurement. Water level on the reservoir also is measured in accordance with the time allowed on this research.

Fig.2 The wave energy converging guiding walls models

RESULT AND DISCUSSION

Effect of wave height

By analyzing the result of wave propagation model tests, the impact of wave height (Hi) on overtopping discharge (q) can be obtained. The relationship between wave height and overtopping discharge in various shape of guiding walls in depth 25 cm on this research can be seen in Fig. 3. It can be seen from those figure that overtopping discharge increase as wave height goes by.

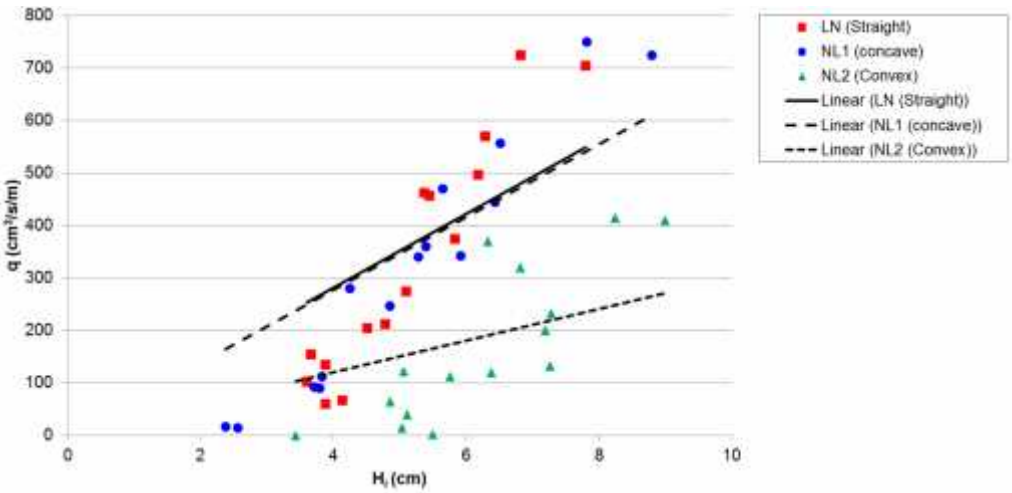


Fig.3 Relationship between wave height and overtopping discharge.

From the graph in the Figure 3 shows that sloping breakwater with linear converging guiding walls captures overtopping discharge greatest than two other models. Overtopping discharge from breakwater with linear converging guiding walls (LN) is around 60 – 745 $cm^3/s/m$. Overtopping discharge from breakwater with concave converging guiding walls (NL1) is around 15 – 745 $cm^3/s/m$. Overtopping discharge from breakwater with convex converging guiding walls (NL2) is around 2.5 – 415 $cm^3/s/m$. It is due to convex converging guiding walls reflected incoming wave greatest than captured incoming waves. It is showed at Figure 4.

At the figure 4 shows that the value of reflected coffisient from breakwater with convex converging guiding walls (NL2) greater than two other models. Reflected coeffisient (Cr) for breakwater with convex converging guiding walls is around 25% -67 %, reflected coefficient for concave converging guiding walls is around 15% - 70 %, and Cr for

breakwater model with linear converging guiding walls is around 16% - 67%. Figure 4 also shows that reflected coeffisien (Kr) linearly decresed with wave stepness (H/L).

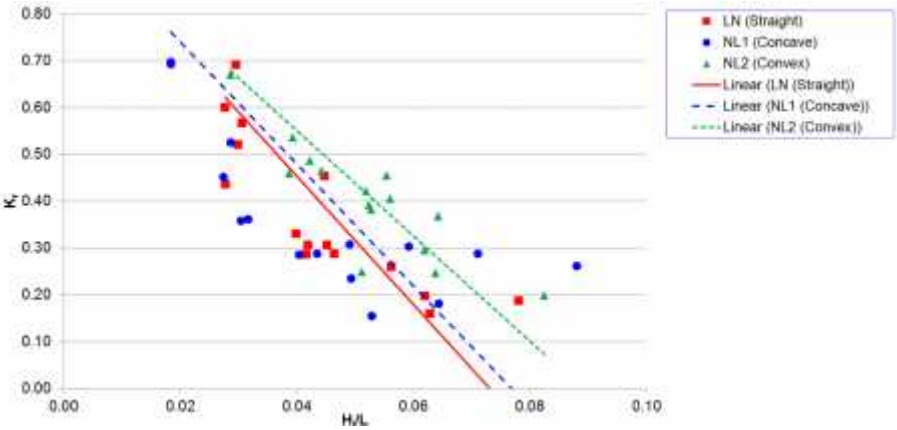
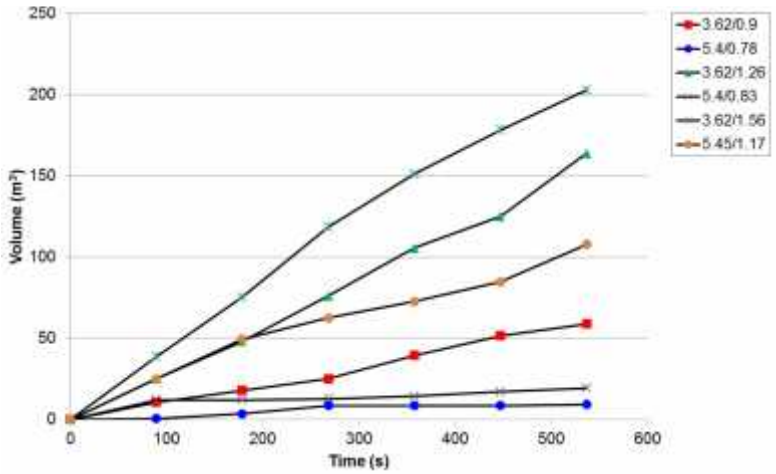


Fig.4 Relationship between wave steepness (H/L) and reflected coeffisien (Kr).

Effect of wave steepness

Figure 5 presents a comparison of the water volume captured in the reservoir for 3 types of wave energy converging guiding walls in variation of T/H. Figure 5 shows that the higher and the longer the wave length, the greater the water volume entering the reservoir. From 3 types of converging guiding walls in sloping breakwater being examined shows that linear converging guiding walls captures greatest water volume compared to 2 other walls. For example, the maximum water volume captured in 200 seconds for linear (LN) is around 121.16 m^3, for concave (NL1) is around 114.48 m^3, for convex (NL2) is around 62.64 m^3. It is due to the convex converging guiding walls reflected waves greater than the 2 other walls. From Figure 6, it can be seen that incoming wave are reflected by converging guiding walls back to the sea. It is due to overtopping discharge that can be captured only a little.



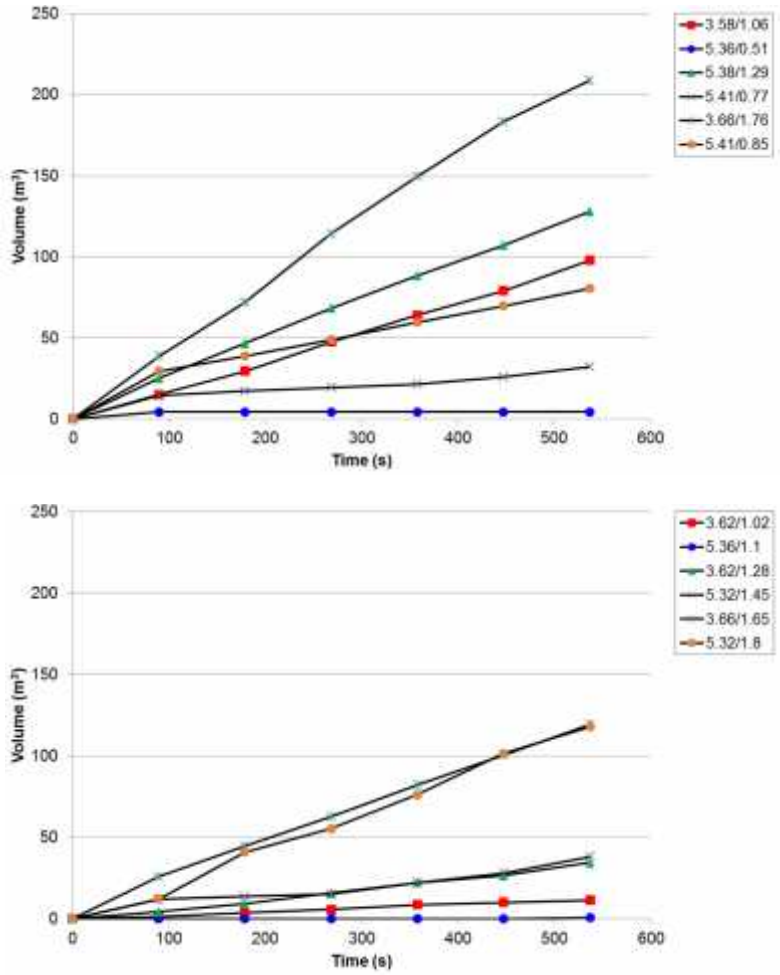


Fig.5 Reservoir production for 3 types of converging guiding walls (LN, NL1. NL2) in various T and H.

Fig.6 Reflected waves in convex converging guiding walls (NL 2)

The model test simulation with prototype trantition sea wave height (H) around 0.78 – 1.56 m and wave stepness (H/L) around 0.3 - 0.8 produces average discharge value (Q) for linear converging guiding walls (LN) 0.175 m^3/s, for concave converging guiding walls (NL1) 0.171 m^3/s, for convex converging guiding walls in sloping breakwater (NL1) 0.100 m^3/s. Fig. 7 shows the relationship between wave steepness (H/L) and wave power in the reservoir. The wave power in the reservoir linearly increased with wave steepness.

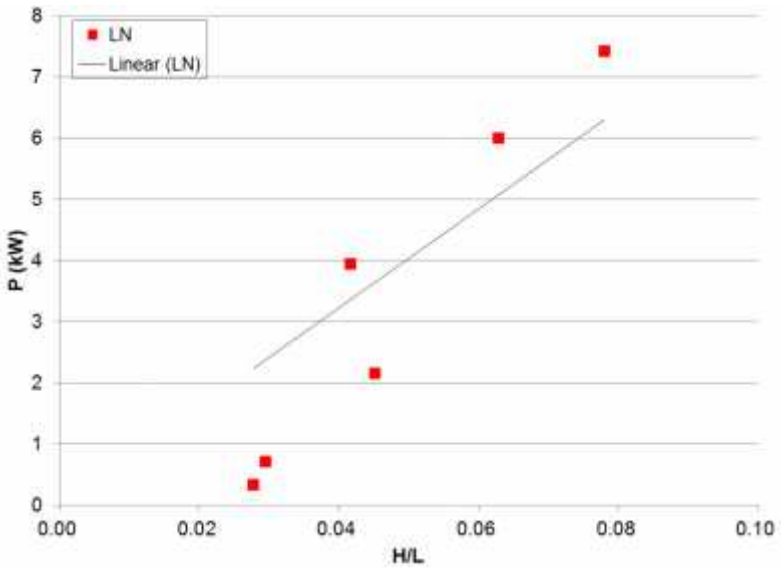


Fig.7Relationship between wave steepness (H/L) and wave power (P overtopping).

CONCLUDING REMARKS

From the analysis and discussion above, it is nececcary recommended these considerations as follows :

1. Sloping breakwater with linear converging guiding walls (LN) captured sea wave greatest than concave walls (NL1) and convex walls (NL2).
2. The maximum overtopping discharge is influenced by the shape of converging guiding walls, wave height and period.
3. The greater the wave steepness was, the greater the overtopping discharge became and reflected coffisient decreased.

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COMBINATION OF ECOLOGICAL AND HIDRAULICAL IN RIVER MANAGEMENT (CASE STUDY AT LAWO RIVER SOUTH SULAWESI)

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ABSTRACT: Ecohydraullic is a concept that combines ecological and hydraulical aspect in managing river environment. The ecological issues in this research is the vegetation grown in the river bank as flood retention, while hydraulical aspect refers to the flows of water in river bank to reduce flooding. The aims of the research were to obtain the diameter of vegetationin the river bank and the river management techniques that can reduce the velocity of flow. There are five stages in this study are hydrology analysis, hydraulics analysis, land use analysis, flooding analysis and ecohydraulic analysis. Hydrology analysis is used to obtain 50 annual flood discharge while the hydraulic analysis used to find the flood water level. Land use analysis to obtain the potential land use along the river and the flooding analysis describe the the risk of flooding in the riverbank. Ecohydraulic analysis is used to obtain the optimal width of the banks, the diameter of which is suitable for the management of vegetation along the river as well as the high floodwaters. The results of the analysis suggests that the diameter vegetation of riverbank between 10 cm and 20 cm and width of riverbanks are 100 meters - 150 meters. With the ecohydraulic concept, the river management techniques can reduce the water level along the river and the velocity of flow. Without banks arrangement, flood water level is more than 2.6 meters and the presence of structuring and vegetation in the banks the water level to 0.7 meters - 2.5 meters . The flow velocity can be reduced between 10 % - 76 % . Based on this research, it is known that the arrangement of riverbanks can provide benefits in flood control measures . This arrangement is the basis in determining the demarcation line of the river .

Keywords: *Ecohydraullic, riverbank, flooding*

INTRODUCTION

Most of rivers in Indonesia often suffer flooding or overflowing. Excessive water volume or amount of discharge water during the rainy season can cause flooding. Overflow of river water as a result of high rainfall will generate surface water. A small portion of rainwater percolate into the ground, while most of the flows on the surface of the ground and to get to the river. Floods are an inevitable function of the hydrologic cycle, and flood cycles were originally seen as blessings because they sustained riverine ecosystems and the floodplain economies dependent on them. (Tarlock, 2012)

The other side, flood with great discharge is considered as disaster. Flooding causes loss of material and sometimes caused of human deaths. Floods that inundated houses, fields of agriculture and other infrastructure can cause a much harm to the the public. Furthermore, floods lead economic losses of both personal and community. With these considerations, the government has to be able to manage the river and flood control.

A variety of well-established measures are used to mitigate floods. They can divided in two methods are structural and non structural. Structural methods such as storage reservoir, detention basins, levees and flood walls, channel modification, land treatment, emergency flood fighting, floodproofing, stormwater management. While non structural methods are flood forecasting, floodplain regulation, coastal zone management, evacuation, relocation, flood insurance, and land acquisition. (Changnon, et.al. 1983).

Flood plain is an area of land that is prone to flooding. In the dry season, this area is not inundated and can be used by humans as a rice field or plantation. While at large flow condition, this area inundated. Also, floodplain as referred to riparian. Specifically, Government of Indonesia regulate the floodplain in Government Regulation No. 38 of 2011 about River. The regulation adjust that borders the river as a buffer space between the river ecosystem and the mainland, so that the river functions and human activities are not mutually disturbed. Definition of riparian areas is a virtual line in left and right riverbed, while the riverbanks is the space between the edge of the riverbed and foot embankment located in the left and right of the riverbed. So, in Indonesia floodplain regulation covered the quality of riparian and riverbanks.

River morphology illustrates integration between abiotic characteristics (physical, hydrology, hydraulics, sediment, etc.) and biotic characteristics (biological or ecological) area which it passes. Factors that influence the morphology of

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the river not only abiotic and biotic factors but also human activities in the basin (socio anthropogenic). Influence of human intervention can result changes in river morphology faster than the natural influence abiotic and biotic.

Waryono (2008) illustrates the river morphology in figure 1.

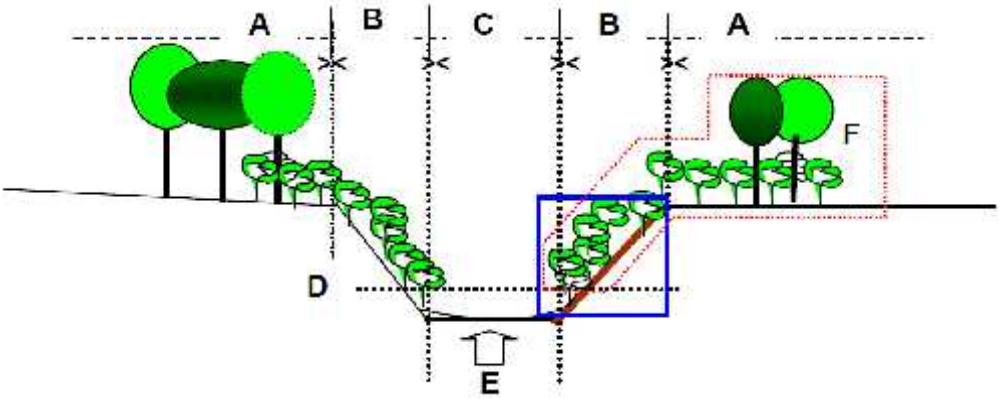

Figure 1. River Morfologi (Waryono, 2008)

River morphology includes not only the body but also the surrounding areas of the river. A is the area along the river which is a barrier between the river`body with the surrounding areas. Area B is a river edge that is limiting the flow area. Area C is water bodies and D shows the water level. The vegetation that grows on the banks of the river and also called riparian vegetation.

Ecohydrullic concept can be developed with eco-engineering approach or the utilization of ecological components to repair the physical structure of the river basin. Ecohydraullic intended to preserve the ecological components in the environment of the river in hydraulic engineering. Application of concept on the river as the protection of river bank erosion is the manufacture of riparian buffer strips or planting vegetation on the riverbanks. With the vegetation planted on the riverbank also cool river water creates a good environment for the growth of various types of aquatic animals. Theoretical foundation of eco-hydraulic engineering, namely vegetation with plant canopy will reduce the velocity of the water to the ground. By reducing the speed of the water in the river downstream flooding problems in the area can be reduced and the natural conditions of the river can be maintained. (Maryono,2005).

Based from the problems, the aims of the research were to obtain the diameter of vegetationin the river bank and the river management techniques that can reduce the velocity of flow.

MATERIAL AND METHODS

This research is located on the River Lawo Soppeng Regency which is part of the regional unit of Walanae River - Cenranae. Administratively the study area is located in South Sulawesi Province. The watershed area is 17 104,45 ha. There are five stages in this study are hydrology analysis, hydraulics analysis, land use analysis, flooding analysis and ecohydraulic analysis. Hydrology analysis is used to obtain 50 annual flood discharge while the hydraulic analysis used to find the flood water level. Land use analysis to obtain the potential land use along the river and the flooding analysis describe the the risk of flooding in the riverbank. Ecohydraulic analysis is used to obtain the optimal width of the banks, the diameter of which is suitable for the management of vegetation along the river as well as the high floodwaters.

RESULT AND DISCUSSION

Hydrology Analysis

Nakayashu hydrograph as result of this analysis in three locations for 50-year return period are presented in Figure 2.

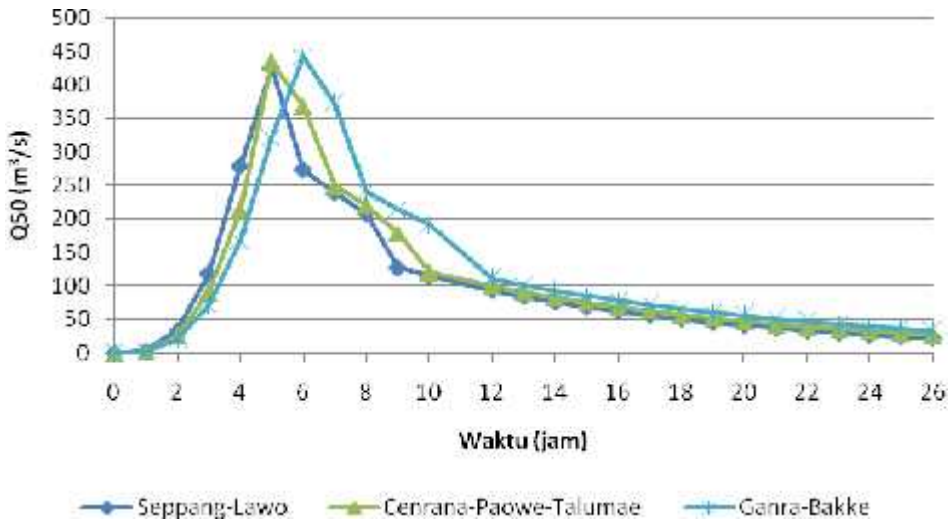

Figure 2. Hydrograph Nakayashu

Nakayashu hydrograph shows the time spent from the beginning of the rain until the flood peak is 4 hours at the local and regional Seppang-Lawo, Cenrana-Paowe-Talumae. Peak discharge in the first region is 425,432 m^3 / s. While the second area is 433,795 m^3/s. In the area Ganra-Bakke flood peak occurred with peak discharge of 441,692 m^3/s.

The analysis shows that downstream areas suffer greater peak discharge, this is due to the accumulation of river flow. The maximum discharge calculation results for the three regions as shown in Table 1.

Table 1. Maximum Discharge at Lawo River

Period (Year)	Qmax (m^3/s)		
	Seppang - Lawo	Cenrana-Paowe-Talumae	Ganra – Bakke
2	256.326	257.458	295.549
5	320.418	279.918	322.612
10	359.269	366.255	362.496
20	393.876	395.121	391.820
25	404.493	412.358	409.150
50	425.432	433.705	441.692

Hydraullic Analysis

The characteristics of the cross section in the river upstream, midstream and downstream varies greatly. (Table 2)

Table 2. Characteristics Of The Cross Section

Lokasi	Lebar dasar (meter)			Lebar Atas (meter)		
	Max	Min	Rata-rata	Max	Min	Rata-rata
Seppang	29.6	12.8	20.9	49.6	26.1	35.7
Lawo	31.2	6.6	17.9	49.1	14.4	30.2
Cenrana	48.0	10.3	28.0	58.2	31.0	41.4
Paowe	37.2	11.2	21.2	48.3	21.1	32.0
Talumae	44.0	5.9	24.1	52.0	13.2	33.3
Ganra	26.8	2.2	18.2	53.9	7.4	24.6
Bakke	16.0	11.2	13.7	53.9	30.3	36.0

Based on hydraulics analysis obtained by the variation of the capacity of the river as in Figure 3.

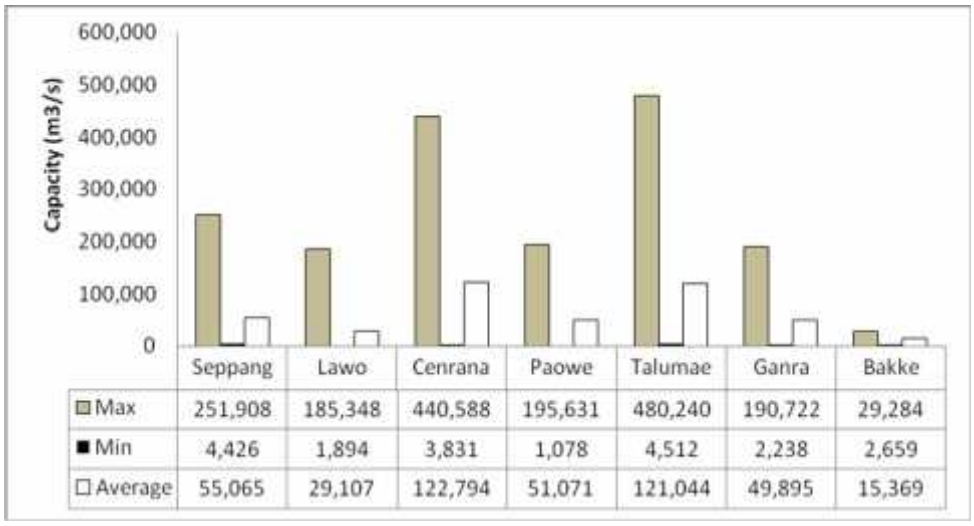

Figure 3. River Capasity

Land Use Analysis

Riverbanks is an area that receives the threat of flooding, but with its strategic location and the nearest water source, the banks used by the community. Conditions of land use on river banks along the river Lawo largely covered by paddy field. On the right side of the river, there is a paddy field as much as 43% were on the left side of the paddy field as much as 30%.

Flooding Analysis

Description of flood events at each location shows that Lawo River suffered major flooding, especially in areas downstream. The threat can not be reduced even with the efforts to normalize the river and constructing embankment partially. Analysis of flood load indicate the magnitude of the threat of flooding in Seppang, Lawo, Cenrana and Paowe is less than 2 meters, was Talumae suffered flooding threat greater than 2 meters.

Ecohydraullic Analysis.

Ecohydraullic analysis results that flood plains planted with trees in diameter 10 cm - 20 cm can reduce the threat of flooding. The width of the banks varies between 100 meters to 150 meters. The design width of riverbanks and vegetation diameter and can accommodate a minimum of 50 annual discharge plan can be seen in Table 3.

Table 3. Ecohydraullic Analysis In Lawo River

Location	Width of banks (meters)		Diameter of vegetation (cm)
	Left	Right	
Seppang	150	150	10
Lawo	150	150	10
Cenrana	150	0	10
Paowe	120	120	20
Talumae	100	100	10
Ganra	0	120	10
Bakke	150	150	10

Ecohydraullic analysis indicate that the inundation water at various points decrease significantly, so that flood damage can be reduced. Ecohydraullic is one of the efforts to reduce flood losses through made area immune to flooding (floodproofing) for certain property and processing floodplains (Linsley et al, 1996). Results of the analysis are presented in Table 4.

Table 4. Inundation Level Reduction In The River Due To The Concept Ecohydraullic

Location	Inundation high (meter)				. Inundation level reduction (meter)
	Without ecohydraullic		ecohydraullic		
	Left	Right	Left	Right	

Seppang	2.885	1.635	1.821	0,571	1.064
Lawo	3.975	2.895	1.456	0,376	2.519
Cenrana	2.626	0.000	1.689	0.000	0.937
Paowe	4.724	3.124	2.491	0.891	2.233
Talumae	3.595	3.505	1.901	1.811	1.694
Ganra	6.555	8.975	0.000	0.734	8.241
Bakke	11.251	10.971	2.221	1.941	9.030

Decreasing of inundation level is due to the magnification of the cross section of the river or riverbanks flooded areas. It is based on the theory that the width of the riverbanks causing an increase in the capacity of the river so that the water level happens to be lower. By vegetation planting in riverbanks, the flow of water can be retained on the banks so that the streamflow be lower. It means that ecohydraullic requires flood-resistant areas to minimize the threat of flooding in downstream areas.

Ecohydraullic analysis also verify that velocity of streamflow can be reduce. The velocity reduction at manu location varies between 10% to 76% . (Table 5)

Table 5. Reduction Of Streamflow Velocity

Location	Width of banks (meters)		Diameter of vegetation (cm)	V (m/s)		V reduction (%)
	Left	Right		Without ecohydraullic	ecohydraullic	
Seppang	150	150	10	2.344	0.897	62
Lawo	150	150	10	1.550	0.703	55
Cenrana	150	0	10	3.751	0.899	76
Paowe	120	120	20	1.901	0.845	56
Talumae	100	100	10	2.448	0.699	71
Ganra	0	120	10	1.707	1.542	10
Bakke	150	150	10	1.602	0.621	61

Decreasing water velocity by vegetation on the banks due to the increased interaction region (width of the banks) and the losses kinetic energy due to friction between the face . Its supported by Sadeghi et al (2010) that the presence of vegetation on the riverbanks can cause differences in velocity water in water bodies and on the riverbanks. With the vegetation on the riverbanks, the velocity of water in the river banks is much smaller when compared to velocity the water in the river. With the vegetation, then there is momentum transfer lateral, shear forces and energy loss and increased flow resistance. Similarly, Sun et al. (2010) resulted that the vegetation on the riverbanks greatly affect the pattern of streamflow, decrease speed and increase friction between the flow and basic channels (increasing the value of the drag coefficient).

Ecohydraullic analysis that generate optimal width of the banks is also a base or reference in determining the border line of the river. It should be considered to enhance Government Regulation No. 38 of 2011 about the river. The policy regulate that border line of small river outside urban areas (without levee) at least 50 meters from the edge of the left and right of the riverbed along the river channel.

CONCLUSIONS

The diameter vegetation of riverbank between 10 cm and 20 cm and width of riverbanks are 100 meters - 150 meters. With the ecohydraulic concept, the river management techniques can reduce the water level along the river and the velocity of flow. Without banks arrangement, flood water level is more than 2.6 meters and the presence of structuring and vegetation in the banks the water level to 0.7 meters - 2.5 meters . The flow velocity can be reduced between 10 % - 76 % . Based on this research, it is known that the arrangement of riverbanks can provide benefits in flood control measures . This arrangement is the basis in determining the demarcation line of the river .

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HYDRODYNAMIC MODELING OF JENEBERANG ESTUARY

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ABSTRACT: Jeneberang Estuary, located south of Makassar, Indonesia, is one of the largest and most important river in Sulawesi, with anthropogenic changes, mainly related to flood control and water supply. A hydrodynamic model of the Jeneberang estuary was established by applying the ECOMSED modeling. The hydrodynamic model is derived from the hydrostatic assumption and Boussinesq approximation. A high-resolution computational grid was generated covering the Jeneberang estuary. The model was run for 30 days driven by tidal forcing at the ocean boundary and river hydrograph at the upstream. The observed tidal data and hydrograph were accessible for the set up of the model. Hydrodynamic simulations have been performed and computed water levels were compared to observations of existing water level gauges along the estuary. An extensive model calibration was needed to adjust model configurations and uncertain parameters to optimize the matching of the model with observations. For the period of a neap-spring-neap cycle, the model settings determined in the calibration process are verified satisfactions with respect to water level measurements. Good agreement was found between model results and observed temporal and spatial variations in water elevation and currents, in the Jeneberang Estuary. The configuration of the hydrodynamic model of the Jeneberang estuary will be applied as a basis for further hydrodynamical investigations of historical topographic states and for morphodynamical computations.

Keywords: *Hydrodinamics model, ECOMSED, grid, estuary*

INTRODUCTION

Jeneberang Estuary is located in the southern of Makassar, Indonesia. The Jeneberang River flowing from east to west across the province, is one of the largest and most important river in Sulawesi (Fig.1). Between these river mouths, the coastal of Jeneberang estuary had developed. The coastal area included Barombong Beach at the south to the Tanjung Bunga at north. length. Estuary Jeneberang coast is very dynamic, naturally, accumulation sediment from fluvial, coastal, marine processes and human activities. An active delta system which has been formed in humid tropical environment under condition of relatively tidal amplitude, low wave-energy, and fluvial input.

Originating from Mt. Bawakaraeng (2,833 m), it flows to the Makassar Strait. The river is 90 km long with a catchment area of 727 km2. There are two reservoirs, which have constructed in the catchment, the Bili-bili reservoir located on the Jeneberang River, holds 375,000,000 m3 water flow and the Jenelanta Reservoir located on the tributary Jenelanta River. In the rainy season (October-April), floods are normally caused by rainstorms, and often short floods are seasoned. There is a productive land for paddy rice and the area under irrigation area is 17,400 ha. The Jeneberang river mouth had formed a delta, which divided the river into two river mouths, the north river mouth and the south river mouth (Fig. 2).

Improvements Lower Jeneberang River work covered dredging, excavation, embankment and revetment constructions for a 9.6 km section between the mouth of the Jeneberang River and the Sungguminasa Bridge. The project was to conduct river improvements for the Jeneberang River, to provide flood protection of Makassar. [2]. Development activities that have been conducted in the Jeneberangan to support the growth of Makassar include: River Improvement (1993), Long Storage / Closure North Estuary (1994), Jeneberang Rubber Dam (1999), and Bili-Bili Multi Purpose Dam (1999). Beside to reduce flood damage, The Bili-billy Dam also stabilize supplies of raw water by installing a raw water transmission main to the Somba Opu water treatment plant, leading to the response to water demand from Makassar and the surrounding area (JICA, 2001). Jeneberang Estuary is in the western part of the southern coast of the island of South Sulawesi, which consists of tidal areas, fluvial and river delta areas that are low slope (<2%) and is covered by alluvium and coastal sediments.

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The scope of this study is to implement and improve the hydrodynamic numerical modeling system for estuarine zone. The model used is fully integrated three dimensional (3-D), time dependent, hydrodynamic, and sediment transport. This module of ECOMSED (Estuarine Coastal and Ocean Modeling System with Sediment), developed by HydroQual, Inc., (2002), has been successfully applied to the coastal and estuarine waters. The development of ECOMSED was originated in the mid 1980's with the creation of the Princeton Ocean Model (Blumberg and Mellor, 1987). Some recent applications of the module include Chesapeake Bay (Blumberg and Goodrich, 1990), Delaware Bay and Delaware River (Galperin and Mellor, 1990), the Gulf Stream Region (Ezer and Mellor, 1992). Mandang and Yanagi (2008) well reproduced the tides and tidal currents with the use of a horizontally two-dimensional barotropic hydrodynamical model ECOMSED which was applied to the Estuary of Mahakam, East Kalimantan, Indonesia.

The objective of this study is to identify the circulation pattern of water flow in the Jeneberang Estuary.

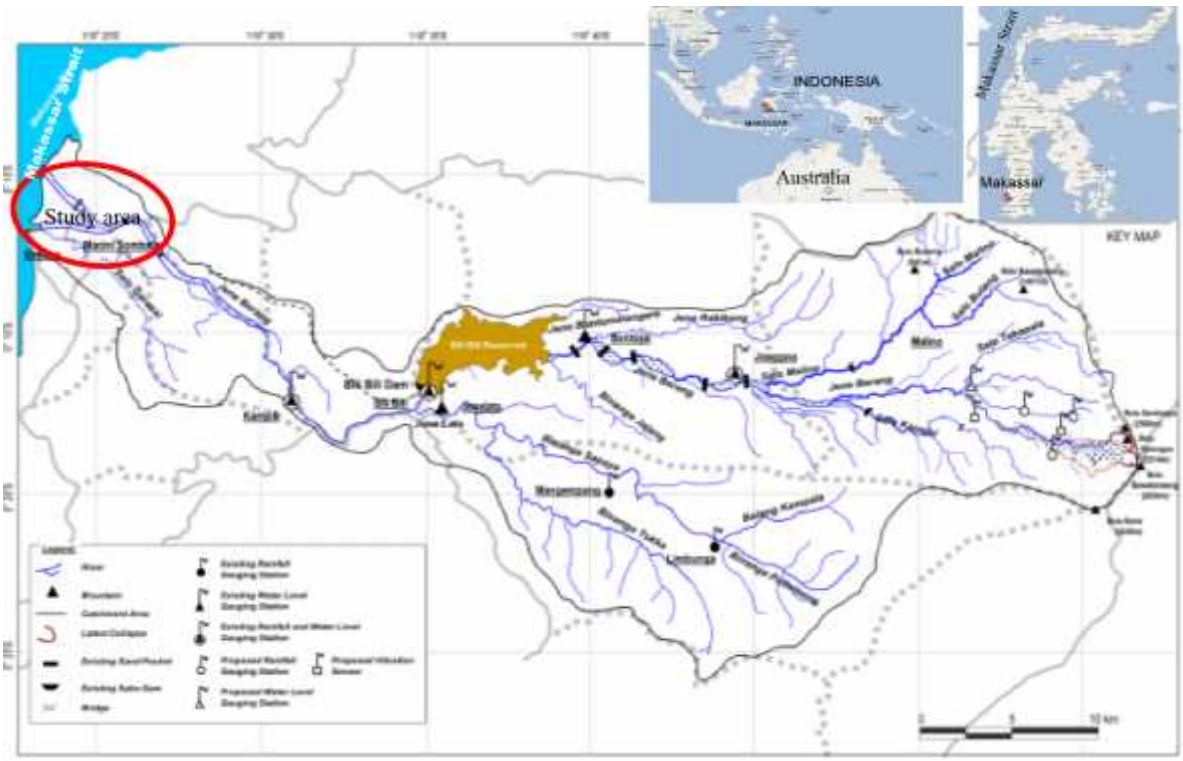

Fig. 1 Jeneberang River watershed

Fig. 2 Study area (Google earth image 2015)

MODEL DESCRIPTION

The hydrodynamic module, ECOM described here is a three-dimensional, timedependent model developed by Blumberg and Mellor (1987). This module of ECOMSED, has a long history of successful applications to oceanic, coastal and estuarine waters. The equations which form the basis of the circulation model describe the velocity and surface elevation fields. Two simplifying approximations are used (Blumberg and Mellor (1987).; first, it is assumed that the weight of the fluid identically balances the pressure (hydrostatic assumption), and second, density differences are neglected unless the differences are multiplied by gravity (Boussinesq approximation). The governing equations of the hydrodynamic component in the model are the continuity and momentum equations. The hydrodynamic part of the model uses the following basic equations. The basic equations for the three-dimensional mode are:

The continuity equations:

$$\frac{\partial U}{\partial x}+\frac{\partial V}{\partial y}+\frac{\partial W}{\partial z}=0 \tag{1}$$

The momentum equations:

$$\frac{\partial U}{\partial t}+U\frac{\partial U}{\partial x}+V\frac{\partial U}{\partial y}-fV=-\frac{1}{\rho_0}\frac{\partial P}{\partial x}+\frac{\partial}{\partial z}\left(K_m\frac{\partial U}{\partial z}\right)+F_x \tag{2}$$

$$\frac{\partial V}{\partial t}+U\frac{\partial V}{\partial x}+V\frac{\partial V}{\partial y}+W\frac{\partial V}{\partial z}+fU=-\frac{1}{\rho_0}\frac{\partial P}{\partial y}+\frac{\partial}{\partial z}\left(K_M\frac{\partial V}{\partial z}\right)+F_y \tag{3}$$

$$\rho g=-\frac{\partial P}{\partial z} \tag{4}$$

where (U,V,W) are the eastward (z), northward (y), and upward (z) components of the current with the ρ_0 reference density, the in situ density, g the gravitational acceleration, P the pressure, K_M the vertical eddy diffusivity of turbulent momentum mixing. A dynamic boundary condition evaluated at the sea surface $z=\eta$ will indicate the relation between sea surface elevation h and vertical velocity at the sea surface W_η as,

$$\frac{\partial \eta}{\partial t}+\frac{\partial \eta}{\partial x}U_\eta+\frac{\partial \eta}{\partial y}V_\eta=W_\eta \tag{5}$$

DESIGN AND RESULT

Model Grid

Raw bathymetry data contain in XYZ format, obatained in two different field, topographic surveys (December-2015) and Bathymetry data of the ocean domain were obtained from DISHIDROS (Indonesian Navy Hydrographic Department) of Indonesian Navy. The model domain in this study covers the area of Jeneberang, (Fig. 3).

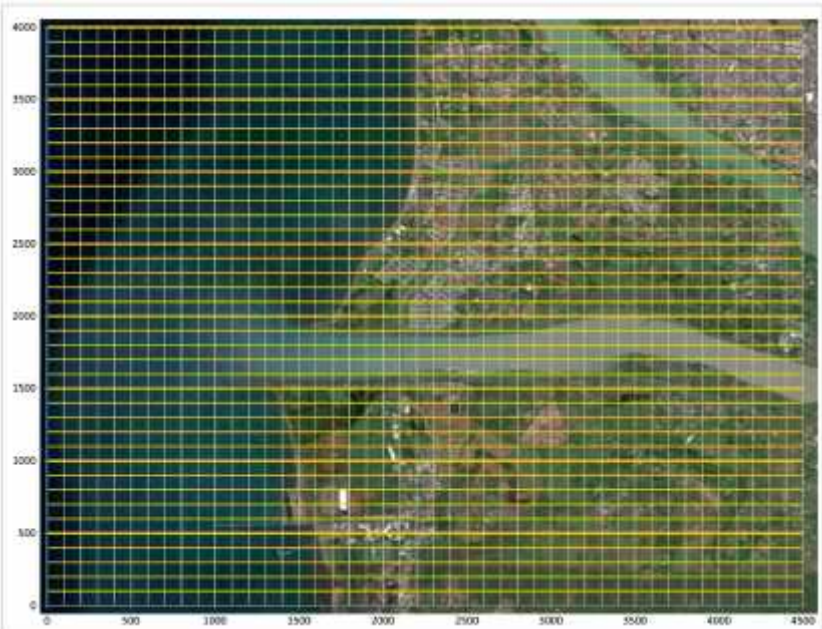

Fig. 3 Grid model of Jeneberang Estuary

The spatial extent of data is approximately 4000m × 4500 m. The data points contains elevation of sea bed, stretching 4500 m seaward and river floodplain. The elevation generally ranges from 0 m up to -10 m. Based on the elevation data collected, a cartesian grid for the model was set up with with 45 × 40 grid cells, sized 100 m × 100 m.

The x axis is pointing to the easts (ocean to landwards). A cell size of 100 m is coarse compared to sizes of the geomorphological features of the estuary, which are are at the order of 100 m. However, for preliminary calculation it would suffice. First the XYZ file is transformed into a point-figure file with elevation as the property of each point. The result grid is saved as an ASCII file whose file name to be declared in the variable model.

Hydrodynamic boundaries

The data needed to model the Jeneberang Estuary are the bathymetry, the tidal forcing at downstream open boundaries, the river discharges at the upstream boundaries and the wind forcing. The boundary condition along the open boundary, was given as the tidal elevation.

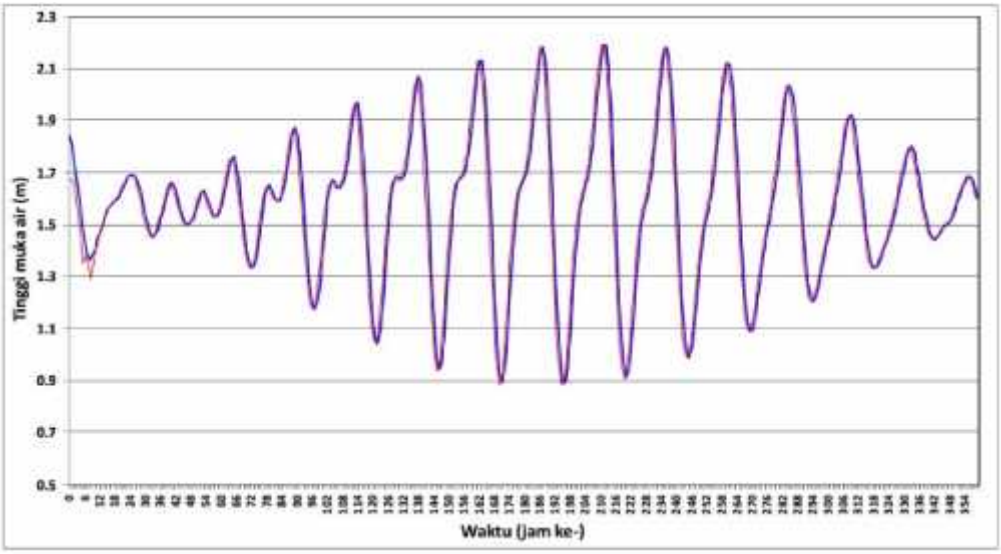

Fig. 4 Water elevation data at the Jeneberang mouth

The river discharge at the upstream boundary in Desember 2013 was given at the Jeneberang Rubber Dam. The averaged river discharge data of the Jeneberang Rubber Dam (2003–2013) obtained from the Balai Sungai Jeneberang Popengan, are shown in Fig. 5. As shown in the figure 5, the maximum and averaged daily discharge of Jeneberang River in the period of 2003 to 2013 was relatively constant in between 99.39 - 100.01 m3/s; while the the minimum daily discharge was decreasing with time and attained the lowest level in 2011, which was 75.22 m3/s.

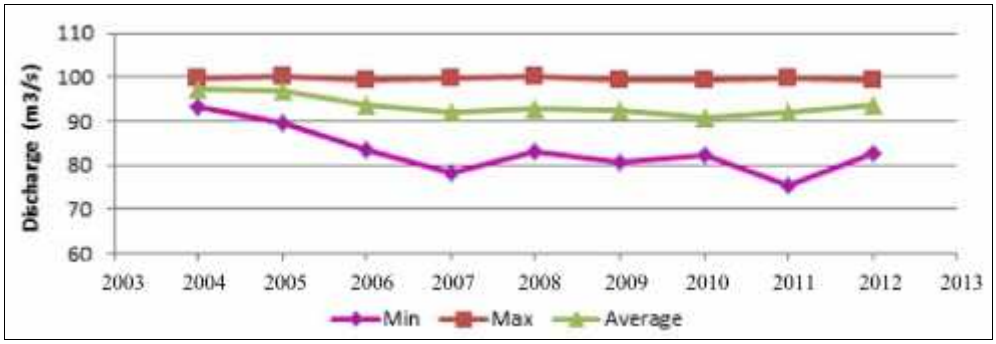

Fig. 5 River discharge data of the Jeneberang river

By introducing a river discharge, we have to set up the upstream flow boundary and accounting for the influence of the inflow water to the system.

Calibration

The first part of the calibration is to demonstrate the model's capability for reproducing the time variable water elevations in the ocean and rivers. Water level calibration is fundamentally important because it demonstrates that the model bathymetry, geometry and hydraulics are configured correctly. The hydrodynamic was calibrated with available

data from 1 December to 31 December 2013. The hydrodynamic model was calibrated with water levels data in the Jenebereng mouth. The calibration of the model is based on the comparison of modelled and observed temporal series of free surface elevation at AWLR stations. The calibration consists in adjusting the Manning coefficient in order to represent the tidal signal correctly. The comparison of the tidal part of the observed and modelled elevation is rather easy if its based on harmonic decomposition.

Figure 4 shows the time series of the observed and modelled elevation for the AWLR gauge station. The spring-neap tidal cycle is clearly visible and well represented by the model. A constant value of 0.022 was used in the ocean domain. Then, the applied Manning coefficient decreases linearly to a value of 0.017 at the upstream part of the estuary. This variation takes into account in a simple way the gradual transition between marine and riverine environments. The value of 0.017 was also chosen for the Manning coefficient in the riverine part. The ability of the hydrodynamic model to simulate hydrodynamic processes in the Jeneberang Estuary was assessed by comparing model output with observed data.

The vertical eddy diffusivity coefficients of momentum are obtained through the 2.5 level turbulence closure scheme developed by Mellor and Yamada (1982). The horizontal mixing coefficients used in the calibration were adjusted using a non-dimensional parameter, which ranged from 0.1 - 0.2 in the rivers and ocean.

A minimum bottom friction coefficient of 0.0024 (nondimensional) and bottom roughness coefficient of 0.003 meters were used throughout the model domain for the calibration.

Hydrodynamic Result

The hydrodynamic model realistically reproduces observed water levels with measurements in the Jeneberang Rivers mouth. Based on the good model calibration to observed water level and velocity data, the overall water circulation in the model domain is well characterized. Figure 4 shows the water level verification of simulation results with observation data obtained from AWLR Station. Figure 4 shows that eastward component of current velocity (positive value) has an appropriate phase with observation data in almost every condition.

Fig. 6. Spring tide flow condition

Fig. 7. Neap tide flow condition

Fig. 8.Low tide flow

Fluctuations in sea level simulation results will be followed by periodic movement of water masses (Hatayama et al., 1996) as shown in Figure 4. Verify water level (elevation) is done by comparing simulation results with the results of measurements of field data. In general, the results of simulation models can represent quite well the pattern of water level in the waters of Makassar Coast (Figure 4)

The three plots show the pattern of current and direction, when the spring tide, neap and at low tide. Mass flow pattern of water in spring tide conditions against the tide dominated by the flow moving toward the north turn and then flows east toward the shoreline (Figure 6). Model also shows the tide into the Jeneberang River during peak tide. At neap tide, the flow pattern is seen moving from south to north, the flow around the mouth of the river was also seen there are currents move to north. While at low tide, current pattern showed similar results to areas far from the mouth of the river.

At the time of low tide, the current pattern of moving from the river into the sea, greater velocity around the river mouth due to the discharge of the river flow (Figure 8). In addition, the flow around the river mouth is also appear to have currents moving north, with a pattern away from the shoreline.Tidal flow patterns on the neap tides and low tide condition showing the flow is dominated by a movement towards the north (Figure 7).

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NUMERICAL MODELING OF HIGH VELOCITY FLOW IN THE COMPOUND CHANNEL

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ABSTRACT:This paper deals with the numerical simulation of high-velocity flow through a compound channel. In the compound channel, in addition to primary flow, there are secondary flow from the overbank to the main channel so that the flow cann't be modelled only in one dimension but should be modeled in two dimensions. HIVEL2D is a free-surface, depth-averaged, two-dimensional model designed specifically for flow fields that contain super-critical and sub-critical regimes as well as the transitions between the regimes. The models provides numerically stable solutions of advection-dominated flow fields containing oblique shocks Reviews such as standing waves and hydraulic jumps. Numerical models calibrated with a physical models that are measured in the laboratory. Numerical model of HIVEL2D can simulate secondary flow and hydraulic jump phenomenon that describe changes in flow from super-critical to sub-critical flows. Model is also able in describing water level profile both in the side and main stilling basins. The magnitude and direction of primary and secondary velocity can be seen clearly. Numerical model can be used as a tool for designing a stilling basin to reduce the cost and time.

Keywords: *High-velocity flow, compound channel, HIVEL2D, numerical model, physical* model.

INTRODUCTION

Compound stilling basin is a stilling basin that has bed elevation difference between in the main channel and in the river banks. In this stilling basin occurs secondary flow significantly from the river banks to the main channel that cannot be ignored. Hydraulic jump is a phenomenon in the stilling basin where there is a change of supercritical flow into sub-critical flow. Supercritical flow occurs due to water flowing through a spillway with a high bottom slope. This flow has a high velocity that is not able to be simulated by using an ordinary mathematical model. Therefore, to model the flow in stilling basin required a mathematical model capable of simulating the two-dimensional with shallow water and high velocity.

Surface Water Modeling System (SMS) is a computer program created to simulate an open channel flow, sediment transport or any other material which is transported by two-dimensional flow of an average depth. SMS software consists of several basic modules are often used, which are RMA2, SED2D, and RMA4. RMA2 is module 2-dimensional hydrodynamic numerical model of the average depth by using the finite element method. This module calculates the water surface elevation and velocity horizontal direction to open channel flow of sub-critical. In turbulent flow, RMA2 calculate the finite element solution of the Reynolds by using the Navier-Stokes equations. Friction is calculated using Manning or Chezy equation and coefficients Eddy used to define the characteristics of turbulent. This software can analyze both flows in steady and unsteady conditions. RMA2 work assuming hydrostatic conditions so that the acceleration in the vertical direction is ignored. Therefore, this program is two dimensions in the horizontal direction. So, RMA2 cannot be used in problems where vortices, vibration or vertical acceleration is the main thing.

From the description above it is clear that the RMA2 modules can only be used in subcritical flow and not for critical or supercritical flow. In fact, research on the design and construction of water frequently encountered not only subcritical flow but also critical and supercritical flow. Even in a rapidly changing flow category, there are the so-called local phenomena, namely a change of flow from supercritical to subcritical or otherwise in fairly short distance. This phenomenon occurs because of a change in cross-sectional shape or change the channel bed drastically in a short distance. The first is a local phenomenon of waterfall, which changes the flow of subcritical to supercritical. This phenomenon is characterized by the increase in velocity and a decrease in the water surface. An example of this phenomenon is the flow of the waterfall. While the second phenomenon called hydraulic jump, which is a change from supercritical flow into subcritical flow. This change occurs due to a reduction in the flow of energy in order to prevent scouring in downstream. An example of this phenomenon is the flow of the dissipation of energy in the stilling basin.

The purpose of this study is to apply computer programs SMS with HIVEL2D module for simulating critical and supercritical flow with depth average 2-dimensional approach in the compound stilling basin.

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LITERATURE REVIEW

HIVEL2D is a 2 dimensional depth average numerical model that design specially to simulate flow in subcritical and supercritical regime include in transition regime. HIVEL2D (Stockstill and Berger 1994) was designed to simulate flow in typical channel with high velocity. The model used finite element to solved 2 dimensional equation in shallow water. Model does not involve Coriolis or wind effect where it is not essential in the high velocity flow. Vertical integration of mass and momentum conservation equations for incompressible flow with the assumption that the vertical acceleration is negligible compared with the results in the general equation is referred to as shallow flow equation.Dependent variables of two-dimensional fluid motion is defined by the flow depth, h, volumetric discharge per unit width in the x-direction, p, and volumetric flow per unit width in the y direction, q. This variable is a function of the independent variables in two-dimensional space x and y, and the dimension of time t. Shallow flow equation in conservative form are given as follows (Abbott 1979):

$$\frac{\partial}{\partial} + \frac{\partial F_x}{\partial} + \frac{\partial F_y}{\partial} + H = 0 \tag{1}$$

Where:

$$Q = \begin{pmatrix} h \\ p \\ q \end{pmatrix} \qquad F_x = \begin{bmatrix} p \\ \frac{p^2}{h} + \frac{1}{2}gh^2 - \frac{h\sigma_x}{\rho} \\ \frac{p}{h} - \frac{h\sigma_y}{\rho} \end{bmatrix} \quad F_x = \begin{bmatrix} q \\ \frac{p}{h} - \frac{h\sigma_x}{\rho} \\ \frac{q^2}{h} + \frac{1}{2}gh^2 - \frac{h\sigma_y}{\rho} \end{bmatrix} \tag{2}$$

$$\mathrm{H} = \begin{bmatrix} 0 \\ \mathrm{gh}\frac{\partial}{\partial}\frac{z_b}{x} + \mathrm{g}\frac{n^2 p\sqrt{p^2+q^2}}{C_b^2 h^{\frac{7}{3}}} \\ \mathrm{gh}\frac{\partial}{\partial}\frac{z_b}{y} + \mathrm{g}\frac{n^2 q\sqrt{p^2+q^2}}{C_b^2 h^{\frac{7}{3}}} \end{bmatrix} \tag{3}$$

p=u.h, wherw u is depth average velocity in x direction

q= v.h, where v is depth average velocity in y direction

g= gravitational acceleration

σ_x , σ_x , σ_y , σ_y = Reynolds stress, first index indicate direction and second index indicate normal axis direction at surface where the stress work.

ρ = water density

C= unit conversion coefficient

Reynolds is determined using Boussinesq approach related to stress to gradient in the flow, so that:

$$\sigma_x = 2\rho v_t \frac{\partial}{\partial} \tag{4}$$

$$\sigma_x = \sigma_y = \rho . v_t \left(\frac{\partial}{\partial} + \frac{\partial}{\partial}\right) \tag{5}$$

$$\sigma_y = 2\rho v_t \frac{\partial}{\partial} \tag{6}$$

v_t is Eddy viscosity, that varies with space and is calculated empirically as a function of local flow variables (Rodi) by the following equation:

$$v_t = \frac{C.n\sqrt{8g}\sqrt{p^2+q^2}}{C_b h^{1/6}} \tag{7}$$

Where, C is a coefficient in the range from 0.1 to 1.

Differential equation solved numerically using the method of finite element. Approach to the finite element method used is of Petrov-Galerkin formulation that incorporates a combination of component tests Galerkin and non-Galerkin function to control oscillation caused by convection. In this model, combinations of linear basic function with triangular and rectangular shapes are used to represent p, g and h. Each time step calculated using iteration method of Newton-Raphson. The slope can be assumed geometrically mild and steep. Constant boundary condition of combination subcritical and supercritical flow can be used during simulation both as inflow or outflow. Friction between water and channel bed is represented by Manning coefficient. Users can determine the coefficients α generate first-order or second-order backward temporal differences. Petrov-Galerkin approach in which the weighting function tests are all characteristics used. Shock detection mechanism based on the variation of the energy per element used to call βSH.

Turbulent Eddy viscosity is calculated based on simple parameters CSM and CSH using a user-selected velocity, depth and roughness.

ANALYSIS AND RESULT

To study the flow in the stilling basin required a physical model of the channel which has a length of 5 meters and a width of 0.3 meters. Elevation upstream channel is + 0.5, while the elevation of the channel stilling basin is +0.00 meters. The elevation difference led to a flow at high speed as the flow in spillway. To create a compound stilling basin, then the elevation of the side channel is raised as high as 10 cm of elevation of the middle so it has a + 0.1 meter elevation. The dimension of the channel used in this study as show below.

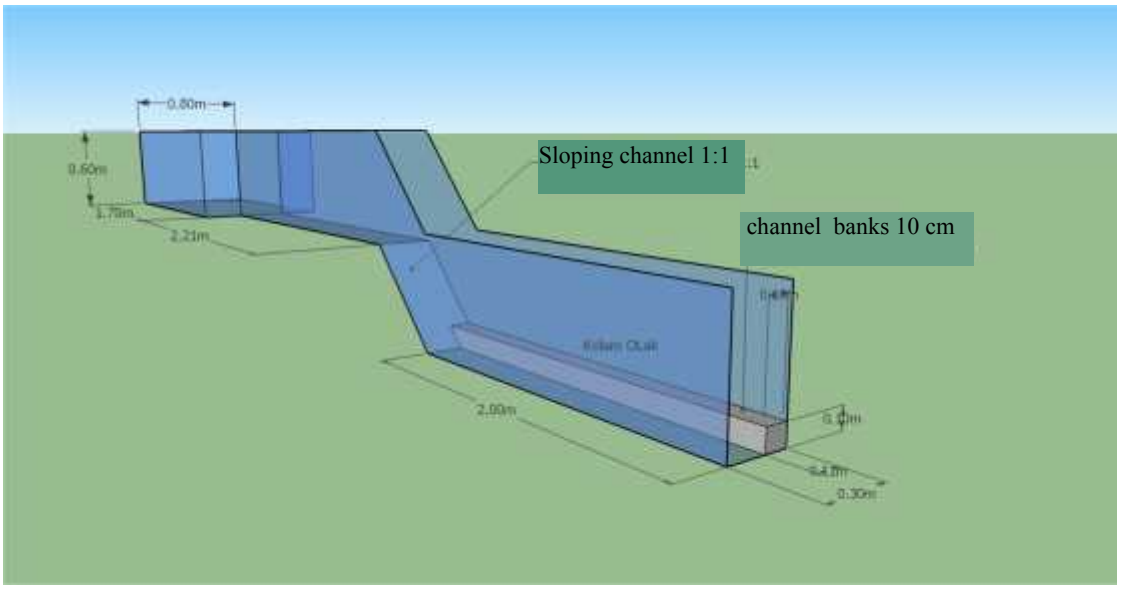

Fig. 1 Dimension of the experiment channel

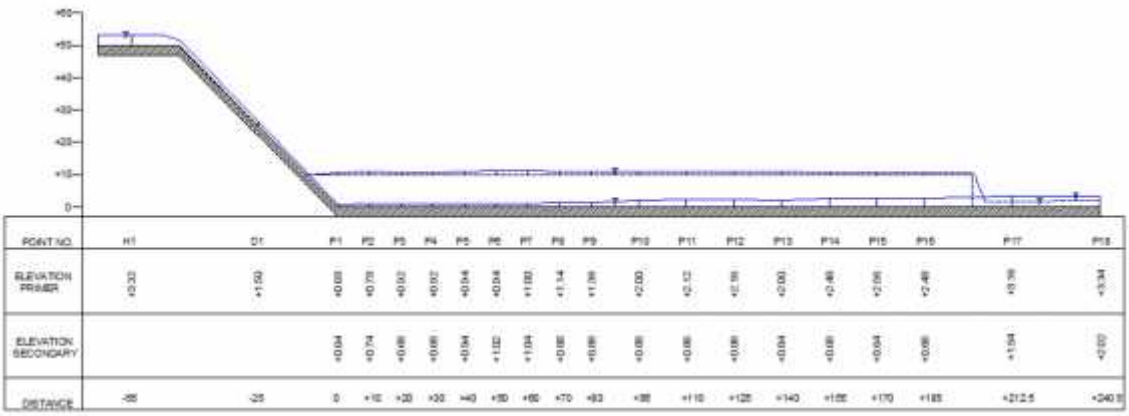

Fig. 2 Water level profile at side and main sections resulted by measurement of physical model

Water level in the channel longitudinal cross section measured at points as shown in Figure 2. The height of the water surface at each point of measurement used to describe the water surface profile. Water surface profile measurements performed in the center and the edge of the channel cross section. Shallow flow conditions of high velocity flow in physical model simulation can be seen in the picture below.

Fig. 3 Photo of supercritical flow of physical model channel with steep slope.

On the basis of physical models, numerical models are made with geometry as shown in Figure 3. The size of the mesh geometry is 1:25 x 1:25 cm. For the simulation, the boundary type used is an inflow string that is supercritical flow with the discharge of 0.0053 m^3/s and the outflow boundary string type is the outflow of supercritical flow. While the control of HIVEL2D models are as follows:

- Turbulence coefficient :
- Normal (smooth) = 0.1
- Near jump (rough) = 0.5
- Gravitational acceleration = 9.81.
- Conversion coefficient for Manning = 1.
- Maximum number iteration/time step = 10
- Froude number convergence = 0.005
- Computational time step 0.0001
- Number of time step = 500
- Save output every 10 time step
- Temporal derivative is second order backward.
- Material property use Manning coefficient of 0.015.

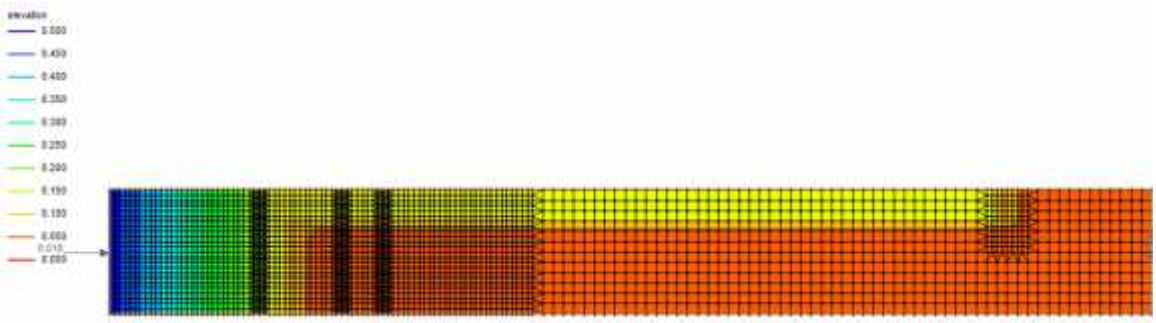
Fig. 4 Geometry of the numerical model

The results of numerical simulation of the model is the water surface elevation, velocity and direction of flow are presented in the following pictures.

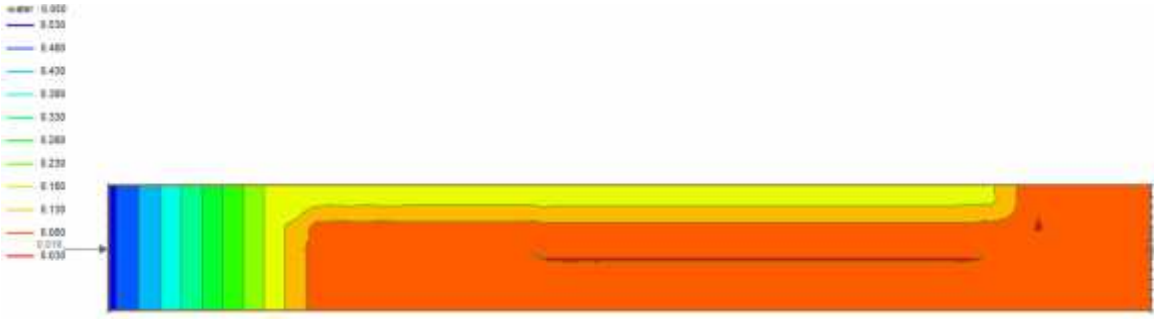

Fig. 5 Water level elevation resulted by numerical model

Figure 5 shows a plan view of the water surface elevation where the water level decreased to follow the slope in the steep sloping channel, while in the flat channel, the water level are almost the same. Here hydraulic jump does not occur because of the velocity on the flat channel is still high and there is no energy dissipation.

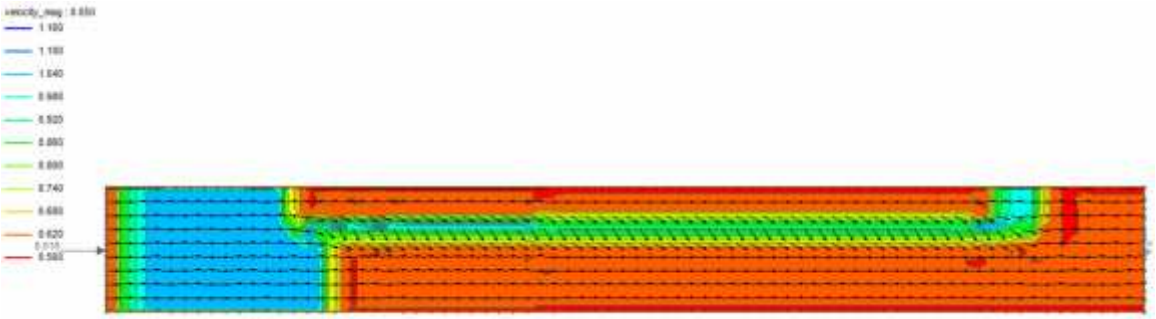

Fig. 6 Velocity profile in 2 dimension resulted by numerical model

Figure 6 shows the flow velocity distribution of the results of numerical modeling. Direction of the primary and the secondary flows can be seen clearly in the picture. Dominant primary flow occurs in the middle of the channel section; while in side section, the primary and secondary flows equally as dominant. Secondary flow velocity is quite large due the drop elevation from the side channel to the main channel.

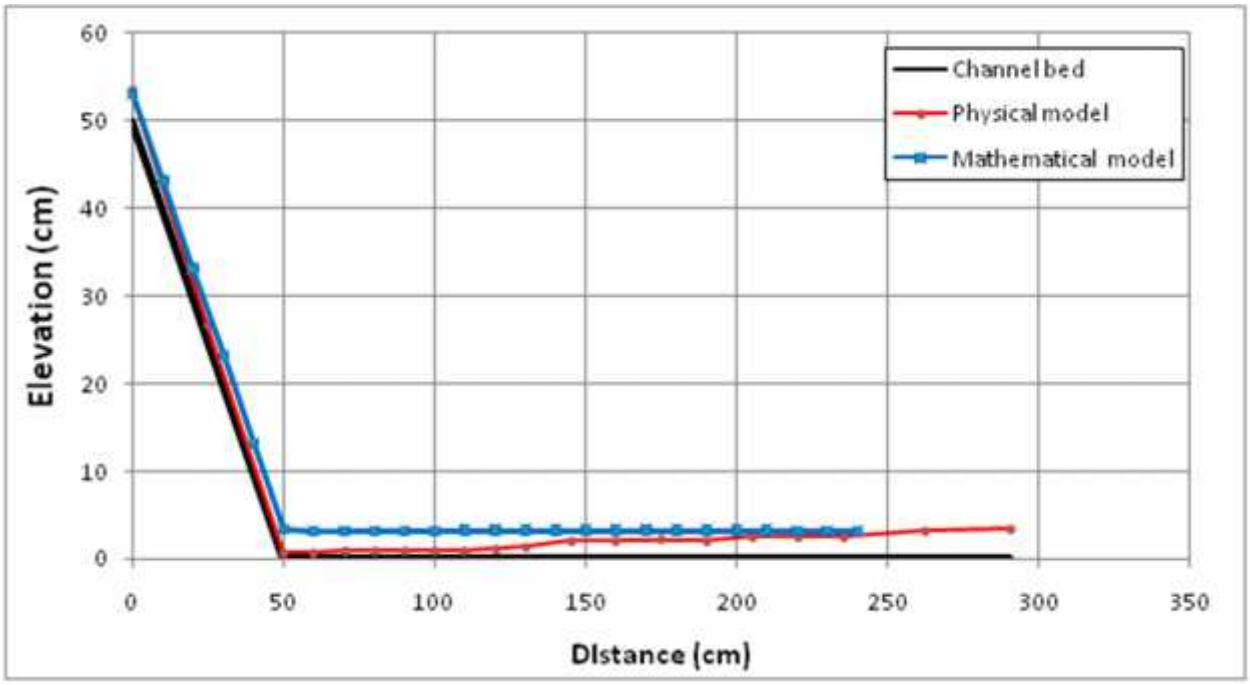

Fig. 7 Comparison of water level profile between main channel and channel bank of numerical model

Water surface profile results from physical modeling and numerical models are presented in Figure 7. Water level resulted by numerical model is higher than that resulted by physical model. But both elevations close each other in the downstream channel. Numerical models produce water level is almost the same in all parts of the flat channel, whereas the physical model, the water level at the beginning of the flat channel is low and increases in the downstream direction. Measurement of water level in the channel with a very shallow flow will be difficult to obtain results with good accuracy. In the numerical model simulations of shallow flow is also very difficult to obtain results with a small difference between numerical models and physical models. Therefore, the percentage difference between the results of numerical models and physical models is quite high. Water level profile in the channel banks are presented in Figure 8. Water levels generated by numerical models closer to the physical model in the channel banks compared with in the main channel. Water level generated by the numerical model is closer to the physical model in the downstream channel.

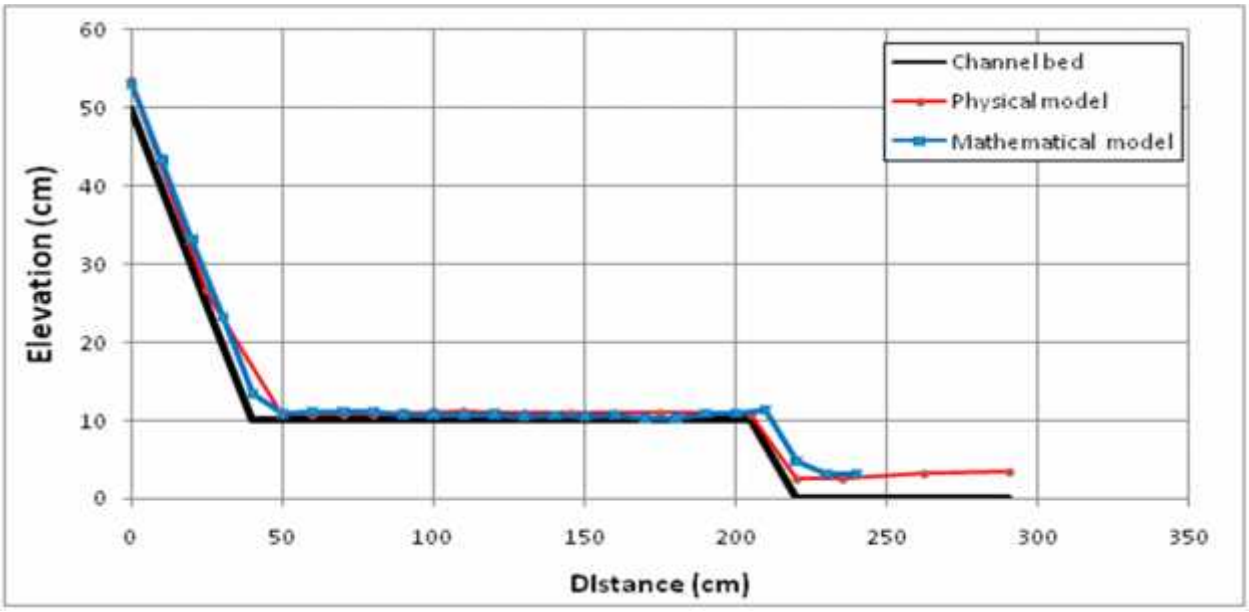

Fig. 8 Comparison of water level profile in the channel banks between numerical and physical models.

To describe the ability of the HIVEL2D model to simulate hydraulic jump, then a model with a larger dimension created. Hydraulic jump occurs when there is a change from supercritical flow into subcritical flow. To simulate the hydraulic jump, the downstream boundary condition is defined as subcritical flow with input of certain water level elevation. Figure 9 shows the process of hydraulic jump phenomena from time to time resulted by HIVEL2D model.

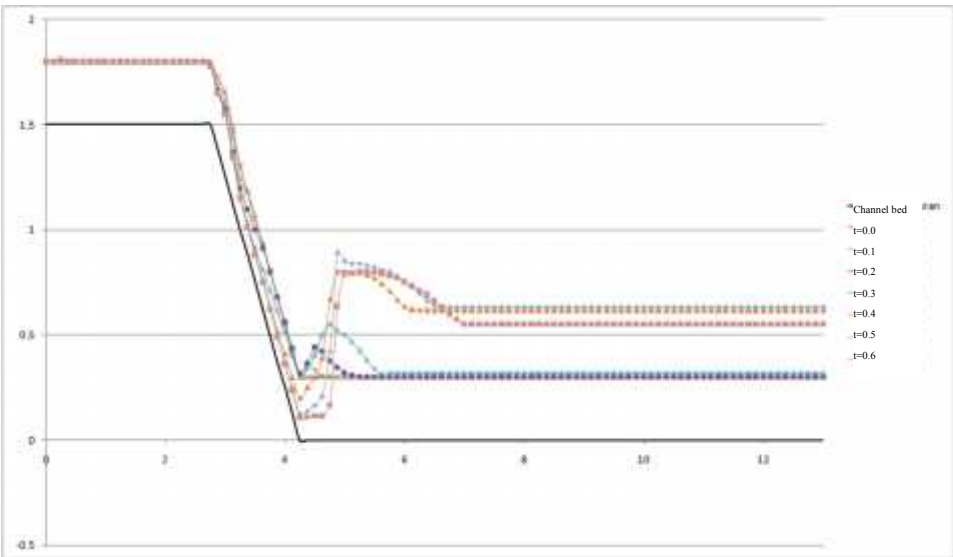

Fig. 9 Hydraulic jump profile resulted by numerical model simulation

CONCLUSION

Two dimensional model of HIVEL2D was able to simulate high velocity flow through compound stilling basin. Model can simulate both supercritical flow and subcritical flows that occur in the stilling basin. The secondary flow from side to main stilling basin can be described in the 2 dimension mathematical model. Flow velocity in the side channel is higher than in the main channel.

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BED LOAD TRANSPORT STUDY IN CHANNEL WITH NON UNIFORM BED MATERIAL

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ABSTRACT: In river improvement, the quantities of bed load transport need to be considered, because they could change the river bed configuration. Bed load transport analysis of non uniform sediment material needs the information of fractional bed materials. The study based on experimental and secondary data obtained from literatures, in which six hundred fifteen of fractional data were used to analyzed fractional bed load transport. The analysis of bed load transport by size fraction was based on three main parameters i.e., the channel slope, proportion of bed load and proportion of bed material in each size fraction. The fractional of bed load transport was observed to depend on the flow discharges, channel slope and bed materials. The result of the analysis showed also that the channel slope, proportion of bed load and proportion of bed material in each size fraction influenced the bed load transport for each fraction. An equation of fractional bed load transport was proposed in this study. Verification of the proposed equation with data from literature showed in good agreement.

Keywords: *non uniform sediment, channel slope, bed material and bed load in each size fraction.*

PREFACE

On various aspects of engineering work in Civil Engineering Hydro information about sediment transport would be necessary. Non uniform Sediment Transport in all places in a water flow will cause changes in bed configuration of this flow. To analyze the morphological processes that may affect the existence of a water structure, we need a formula quantity of sediment transport is accurate and easy to use. The theories developed for the analysis of sediment transport in rivers is generally based on shear stress (drag/tractive force), which is relatively limited practical application. This experiment uses a criticism discharge approach which is rarely done in previous studies. Reality on the ground the riverbed material is generally not uniform so that the grouping in a fraction needed to analyze the bed load transport.

The research conducted here is aimed at the quantity of bed load transport per fraction which is between the other fractions together transported by water flow. These parameters are considered as factor of bed load transport per fraction of sediment is a flow unit discharge, the discharge unit of criticism and the proportion of the bed material fraction, the ratio of relative grain size of the bed material, the proportion of bed load transport fraction concerned and channel slope. For the validity of the research used data from research at the Laboratory Centre for Science and Engineering, University of Gadjah Mada therewith previous research data obtained from literature.

This research is expected to be useful to get bed load transport formulation that uses parameters that are easily measured in the field, so that the formulation can be used in the engineering process with the gradation variation of the river with minimal effort or cost.

Various studies on the basis of the quantification of bed load transport was started by Du Boys (1879), suggests the assumption that the sediment moving in layers with thickness t, the movement of grain caused by shear force (Graf, 1984). Other formulas to estimate sediment transport base like Schoklistsch (1914), Gilbert (1914), Scafternak (1922), Donat (1929) and so (Graf, 1984).

There are three basic approaches in bed load transport (Graf, 1984) namely: 1). Type DuBoys formulation, by considering the relationship of shear stress. 2). Type Schoklitsch formulation, by considering the relationship of water discharge. 3). Einstein formula type, based on statistical considerations lift force. But in fact the formulation of bed load transport derived from the three forms (Garde and Raju, 1985) the following:

$$q_B = A(^2{}_0 - ^2{}_{0c})\,;\; q_B = A(U - U_{cr})\,;\; q_B = A(q - q_c) \tag{1}$$

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With q_B bed load transport, τ_0 shear stress, q flow discharge, U the flow velocity. Where subscript c or cr criticism condition and A is a constant that represents other variables, such as the channel slope, grain diameter, density and others.

Einstein advocated the need to classify the non uniform sediment within some range (grouping with fractions). Classification (range / fraction) particle size commonly used is the classification according to "The subcommittee on Sediment Terminology of the AGU (American Geophysical Union)" or the Wentworth scale (Peter R. Wilcock, 2006).

RESEARCH METHODOLOGY

Laboratory studies were performed using the existing facilities at the Laboratory of Hydraulics and Hydrology, Engineering Science Study Center Gadjah Mada University. Measurements were taken in Sediment Transport Flume length of 10 m and a width of 0.6 m and setting the channel bottom slope and flow rate is electronically governed in accordance with usefulness to some extent. Bed material used is non-cohesive material consisting of Materials 1, 2 and 3. All three describe the condition of the non uniform rough material until smooth. The material was taken directly from Krasak River.

Laboratory measurements done for 3 types of bed material with variations in discharge and channel slope. The results of flow and sediment transport analysis for each variation of slope and discharge obtained flow and sediment transport parameters for each grain as in Table 1. From the flow test results can be seen the flow of what happens is subcritical flow (Fr <1), except in running KISIQ1 and K1S3Q1 value of Froude number (Fr)> 1 (supercritical flow).

Table 1. The main parameters flow and sediment transport

No.	Running	Slope	q (m^3/s)	h (cm)	Froude	U (m/s)	Re	transport (kg/s)
				Material 1, D_{50} = 10,21 mm				
1	K1S1Q1	0.01	0.0086	2.5	1.154	0.5714	15771.8071	0.0437
2	K1S1Q2	0.01	0.0105	4	0.699	0.4377	18479.4540	0.0218
3	K1S1Q3	0.01	0.0153	5	0.730	0.5114	26217.0198	0.1090
4	K1S1Q4	0.01	0.0173	5.5	0.713	0.5235	29107.4196	0.0455
5	K1S1Q5	0.01	0.0270	7	0.774	0.6417	43564.9166	0.1002
1	K1S2Q1	0.015	0.0066	3	0.680	0.3686	12026.2433	0.0088
2	K1S2Q2	0.015	0.0115	3.5	0.932	0.5463	20482.3966	0.0157
3	K1S2Q3	0.015	0.0182	5	0.868	0.6081	31176.3500	0.0553
4	K1S2Q4	0.015	0.0221	5.5	0.912	0.6701	37256.5537	0.1273
5	K1S2Q5	0.015	0.0260	6.2	0.896	0.6985	42929.3664	0.0902
1	K1S3Q1	0.02	0.0066	2.3	1.012	0.4808	12286.8740	0.0134
2	K1S3Q2	0.02	0.0086	3.4	0.727	0.4201	15346.8183	0.0269
3	K1S3Q3	0.02	0.0144	4.5	0.801	0.5324	24919.9085	0.0520
3	K1S3Q4	0.02	0.0202	5.1	0.932	0.6595	34384.3291	0.1246
5	K1S3Q5	0.02	0.0260	6	0.941	0.7218	43167.8628	0.2289
				Material 2, D_{50} = 0.981 mm				
1	K2S1Q1	0.005	0.0134	4.5	0.747378	0.4966	23242.8403	0.0111
2	K2S1Q2	0.005	0.0163	5	0.776252	0.5437	27870.1299	0.0258
3	K2S1Q3	0.005	0.0182	5.5	0.752662	0.5529	30737.2464	0.0381
4	K2S1Q4	0.005	0.0231	6.2	0.795589	0.6205	38134.4339	0.0197
5	K2S1Q5	0.005	0.0250	7	0.718768	0.5956	40437.4111	0.0761
1	K2S2Q1	0.0031	0.0153	5.5	0.633	0.4649	25847.7660	0.0046
2	K2S2Q2	0.0031	0.0192	6.5	0.617	0.4926	31480.3041	0.0131
3	K2S2Q3	0.0031	0.0231	7.2	0.636	0.5343	37109.3147	0.0235

4	K2S2Q4	0.0031	0.0260	7.9	0.623	0.5482	41003.7747	0.0425
5	K2S2Q5	0.0031	0.0279	8.3	0.621	0.5606	43596.8868	0.0359
Material 3, D_{50} = 0.68 mm								
1	K3S1Q1	0.0007	0.0192	7.6	0.487935	0.4213	30559.3378	0.0060
2	K3S1Q2	0.0007	0.0221	8.2	0.501142	0.4495	34623.2371	0.0052
3	K3S1Q3	0.0007	0.0279	9.5	0.507364	0.4898	42272.4244	0.0123
4	K3S1Q4	0.0007	0.0308	10	0.503449	0.5036	45854.1622	0.0130
5	K3S1Q5	0.0007	0.0347	11	0.505978	0.5256	50604.2129	0.0172
1	K3S2Q1	0.0005	0.0192	8.2	0.435373	0.3905	30079.3482	0.0051
2	K3S2Q2	0.0005	0.0221	9	0.435831	0.4095	33913.0168	0.0058
3	K3S2Q3	0.0005	0.0279	9.7	0.491753	0.4797	42059.4651	0.0101
4	K3S2Q4	0.0005	0.0308	11	0.482027	0.4892	45514.5017	0.0108
5	K3S2Q5	0.0005	0.0347	11	0.505978	0.5256	50604.2129	0.0140
1	K3S3Q1	0.0003	0.0192	8.1	0.443460	0.3953	30158.2966	0.0025
2	K3S3Q2	0.0003	0.0221	8.7	0.458567	0.4236	34175.9084	0.0038
3	K3S3Q3	0.0003	0.0279	10	0.469791	0.4653	41744.0191	0.0073
4	K3S3Q4	0.0003	0.0308	11	0.482027	0.4892	45514.5017	0.0092

Description:
Running KnSnQn:
Kn = n-th variation of grain
Sn = n-th variation of the slope
Qn = n-th variation of discharge
q = measured flow discharge (m^3 / s)
h = flow depth (m)

Froude number = $\frac{U}{\sqrt{g \cdot h}}$

U = flow velocity (m / s)
R_e = Reynolds Numbers

Core activities of this research are the measurement of bed load transport at various flow conditions and channel bottom slope. Measurement of bed load transport is to measure the volume of bed load transported across a stream per unit time look. Measurements made on the bed materials and certain bottom slope, with each parameter made five variations.

In this study the bed load transport discharge is obtained by measuring the weight of sediment transport is arrested with bags of sediment catcher at the end of the channel during the experiment divided by the long trial.

RESULTS AND DISCUSSION

Inquiry results showed the three materials are relatively the same densities that are 2.6 kg/m3. Those are due three forming materials are the same materials. Similarly, the porosity value. Specific gravity (γ) (kg/m^3) for material 1 γ = 2.67, material 2, γ = 2.66 dan material 3, γ = 2.66., Porosity for material 1, e = 0.52, material 2, e = 0.63 and material 3, e = 0.69.

Results by sieve analysis obtained grain distribution as shown in the graph analysis of grain in Figure 1.

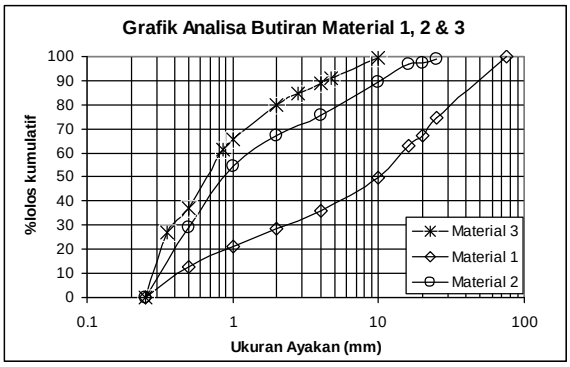


Figure 1. Graphic analysis of the original material granules.

Table 2. Original material parameters 1, 2 and 3

No	Description	Diameter 1 (m)	Diameter 2 (m)	Diameter 3 (m)
1	D_{16}	0.0006	0.0004	0.0003
2	D_{50}	0.0101	0.0009	0.0007
3	D_{84}	0.0380	0.0075	0.0028

The criticism discharge approach is felt more practical and easier if applied to the actual conditions in the field.

To optimize the results of the research, included data from previous research, i.e. research data Suparmo (1997), Yanuar (2002) and Shubhi (2002). The collection of measurement data (running) with data from the study (running) the previous produce 15 tilt (slope) channels. Each slope is composed of 9 fraction granules.

General formulation of bed load transport which is used in this research is

$$q_{bi} = A(q - q_{ci}) \tag{2}$$

The parameters that need to know are q_{ci} and A. Mode was used with regression analysis. In the analysis used two methods:

1. The first method is the use of regression analysis to obtain constants A and original bed material criticism discharge per fraction q_{ci} with reference equation $q_{bi} = A(q - q_{ci})$.
2. The second method is the way regression analysis is to obtain parameter equation using the formula B and Bathurst (1987) in order to get the original bed material criticism discharge per fraction q_{ci} with reference equation $q_{bi} = B(q - q_{ci})$.

The steps to the first method of analysis as follows:

1. With the help of Microsoft Excel graph depicted the relationship (trend line) q with q_{bi} using a type of simple linear regression equation is a straight line equation y = ax + b for each faction in each of the existing slope. Determine the regression equation q_{bi} = a (q) + b.

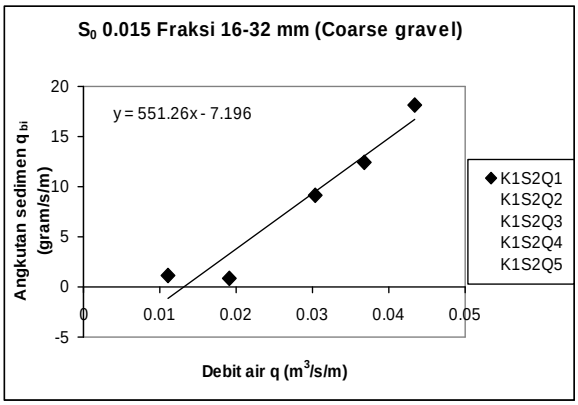


Figure 2. The relationship between water discharge q with sediment transport per fraction q_{bi}

2. Based on the formulation of bed load transport $q_{bi} = A(q - q_{ci})$, if q_{bi} = 0 with the assumption that when criticism conditions ,sediment transport does not happen then:

   0 = A (q - q_{ci})
   0 = (q - q_{ci})

   q = q_{ci} ; because q_{bi} = 0, then could determine its value q_{ci} namely q_{ci} = q. In other words if qbi = 0 regression equation 0 = a (q) + b, then the obtained value of q = qci = - b / a. The things that need to be considered is not always the regression equation analysis, can yield a positive value q_{ci} or the value q_{ci} <q. This is due inaccurate at the time of measurement, and any others that require more in-depth research.
3. Within a fraction often consists of more than one regression equation or difference tendency of some characteristic patterns of material transport that is completely different. This is partly due to differences in the proportion of original material before the transport, although still in a fraction of the same. In this analysis needs to take the average value of q, a, b, and q_{ci} and take one representative grain diameter values (D_i) representative of the respective fractions.
4. General sediment transport equation shows, if the grain has been moving, "the difference between water discharge to the criticism discharge per fraction" (excess water discharge) that occur will be proportional to bed load transport based on the following $q_{b\,i} = (q - q_{c\,i})$.

   However, by looking at the condition of the river / channel varies, the magnitude of sediment transport is strongly influenced by other variables, such as the slope of the river, grain diameter, density, etc., then the equation becomes:

   $q_{b\,i} = A(q - q_{c\,i})$

   Where A is taken from the average value of “a” in the fraction concerned.
5. To create a single formula $q_{b\,i} = A(q - q_{c\,i})$, created a tendency / trend line between the constants A with representative grain diameter (D_i) the slope of all channels is obtained the equation:

   $$A = -5.7913\, D_i + 0.2991 \quad (3)$$

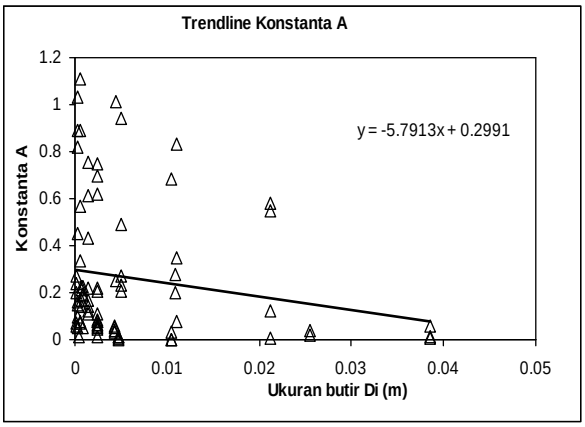


Figure 3. A constant trend graph

From Figure 3. above, the constant A that is obtained does not have a specific pattern, and not explaining that the constant A is a function of the dominant variables that need to be known. Therefore another approach is needed alternative, namely that the second method.

From the discussion above, there is a relationship between the constants A and size fraction. Figure 3 show the relationship followed the original bed material size distribution. In some studies found that the effect of relative grain size is greater than the absolute size of the granules (Wilcock, 1987; Wilcock and Southard, 1989; Ashworth and Ferguson, 1989). Therefore it is concluded that the coefficient is a function of the bed load transport proportion each size fraction (Wilcock and Southard, 1989; Shih and Komar, 1990).

The process of forming an equation of bed load transport per fraction is necessary to understand the basic theory for selecting independent variables. In evaluating the free variables are selected as the prediction of sediment transport equation, the first option is a regression procedure. Analysis of bed load transport equations form per fraction on this study, based on three elements of bed load transport equation. The first is Schoklitsch (1962) the total bed load transport equation (equation (4)), which is only used one measure the diameter D_{40} and the neglect of the proportion of bed load transport and the proportion of the original bed material per faction, but included channel slope.

$$q_{sb} = \frac{2.5}{\circ / \circ} S^{3/2}(q - q_c) \quad (4)$$

The second equation is the bed load transport equation Shih and Komar (1990) (equation (5)) where the proportion of bed load transport per fraction is used to obtain bed load transport per fraction.

$$q_{si}(D_i, ²) = q_s(²) f(D_{i,}²) \quad (5)$$

where q_s is total unit bed load transport and f (D_i, τ) the frequency curve for the distribution of bed load transport. Shih and Komar (1990) consider the effect of size distribution of bed load transport, but influence of the original bed material size distribution and channel slope is negligible.

The third equation is the bed load transport equation Wilcock and Southard (1989) (equation (6)) where the proportion of bed load transport and the proportion of original bed material, both used to calculate bed load transport per fraction.

$$q_{bi} = (P_i / f_i) q_b \quad (6)$$

where q_{bi} bed load transport for fraction i, q_b total bed load transport, P_i the proportion of bed load transport per fraction, f_i proportions of bulk non uniform original bed materials per fraction.

Based on the discussion above, the equation of bed load transport per fraction becomes:

$$q_{bi} = f_{bli} B (q - q_{ci}) \quad (7)$$

where, B (q - q_{ci}) is total bed load transport, B is a coefficient and a function of (f_{bmi}, D_i / D_r and S_0)

Based on the equation (7), and to complete the equation (2), can use relationship between bed load transport and the difference between water discharge and criticism discharge, based on the water discharge unit for each size fraction, and refers to the relationship:

$$q_{bi} = B (q - q_{ci}) \quad (8)$$

where q_{bi} bed load transport per fraction, q water discharge unit, q_{ci} unit water discharge criticism for the size fraction i, B coefficients (non-dimensional) is a function of (f_{bmi}, f_{bli}, D_i / D_r and S_0), f_{bmi} proportion of original bed material in size fraction i, f_{bli} proportion of bed load transport on the size fraction i, S_0 slope and D_i / D_r relative grain size of the original bed material ratio.

Table 3. Results of the relationship between q_{bi} and (q-q_{ci})

size fraction **(mm)**	**Bed Material**	***Running***	***Channel Slope***	**Equation parameter**
			S_0	**B**
16-32 mm (*Coarse gravel*)	Ukur1	K1S2Q1	0.015	0.13860
	Ukur1	K1S2Q3	0.015	0.13860
	Ukur1	K1S2Q4	0.015	0.13860
	Ukur1	K1S2Q5	0.015	0.13860
8-16 mm (*Medium gravel*)	Ukur1	K1S2Q1	0.015	0.13100
	Ukur1	K1S2Q3	0.015	0.13100
	Ukur1	K1S2Q5	0.015	0.13100
4-8 mm (*Fine gravel*)	Ukur1	K1S2Q1	0.015	0.22000
	Ukur1	K1S2Q3	0.015	0.22000
	Ukur1	K1S2Q5	0.015	0.22000
2-4 mm (*Very fine granule*)	Ukur1	K1S2Q1	0.015	0.14470
	Ukur1	K1S2Q2	0.015	0.14470
	Ukur1	K1S2Q3	0.015	0.14470
	Ukur1	K1S2Q5	0.015	0.14470
1-2 mm (*Very coarse sand*)	Ukur1	K1S2Q1	0.015	0.16890
	Ukur1	K1S2Q2	0.015	0.16890
	Ukur1	K1S2Q3	0.015	0.16890
	Ukur1	K1S2Q5	0.015	0.16890
0.5-1 mm (*Coarse sand*)	Ukur1	K1S2Q1	0.015	0.18390
	Ukur1	K1S2Q2	0.015	0.18390
	Ukur1	K1S2Q3	0.015	0.18390
	Ukur1	K1S2Q5	0.015	0.18390
8-16 mm	Ukur2	K2S2Q1	0.0031	0.0031

(***Medium gravel***)	Ukur2	K2S2Q2	0.0031	0.0031
	Ukur2	K2S2Q3	0.0031	0.0031
	Ukur2	K2S2Q5	0.0031	0.0031
4-8 mm (*Fine gravel*)	Ukur2	K2S2Q1	0.0031	0.0332
	Ukur2	K2S2Q2	0.0031	0.0332
	Ukur2	K2S2Q3	0.0031	0.0332
	Ukur2	K2S2Q4	0.0031	0.0332
	Ukur2	K2S2Q5	0.0031	0.0332
2-4 mm (*Very fine granule*)	Ukur2	K2S2Q1	0.0031	0.0342
	Ukur2	K2S2Q2	0.0031	0.0342
	Ukur2	K2S2Q3	0.0031	0.0342
	Ukur2	K2S2Q4	0.0031	0.0342
1-2 mm (*Very coarse sand*)	Ukur2	K2S2Q1	0.0031	0.0809
	Ukur2	K2S2Q2	0.0031	0.0809
	Ukur2	K2S2Q3	0.0031	0.0809
	Ukur2	K2S2Q4	0.0031	0.0809
0.5-1 mm (*Coarse sand*)	Ukur2	K2S2Q1	0.0031	0.1745
	Ukur2	K2S2Q2	0.0031	0.1745
	Ukur2	K2S2Q3	0.0031	0.1745
	Ukur2	K2S2Q4	0.0031	0.1745
	Ukur2	K2S2Q5	0.0031	0.1745

Reference Schoklitsch (1962) shows that there are power relations between bed load transport and Slope. Therefore, the relationship between the parameters of equation B for each size fraction and the slope is illustrated in Figure 4. The results shown in Table 4. and refers to the relationship:

$$B = C\, S_0^{\,d} \tag{9}$$

Where, C coefficients; S_0 Slope and d Exponent.

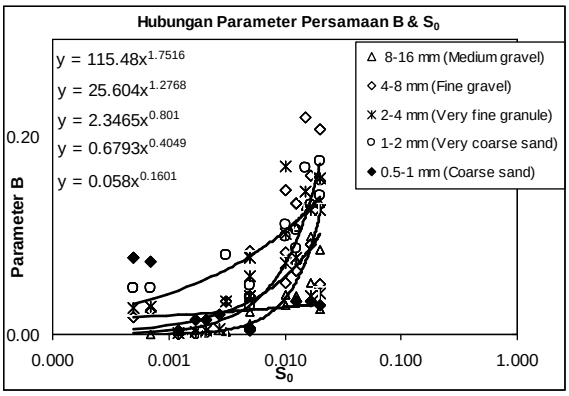


Figure 4. Relations between parameters of the equation B for each fraction and slope.

Table 4. Parameter equation $B = C\, S_0^{\,d}$ For each fraction.

Ukuran Fraksi	Parameter persamaan		D_i
	C	**d**	**(m)**
8-16 mm (*Medium gravel*)	115.48	1.7516	0.01131
4-8 mm (*Fine gravel*)	25.604	1.2768	0.00566
2-4 mm (*Very fine granule*)	2.3465	0.801	0.00283
1-2 mm (*Very coarse sand*)	0.6793	0.4049	0.00141
0.5-1 mm (*Coarse sand*)	0.058	0.1601	0.00071

Figure 5 shows the relationship between coefficient C and the size fraction. Figure 5 also shows the relationship between the exponent d and the size fraction. Both images show a relationship to the size fraction. The effect of relative grain size larger than the absolute size of the grains. It is concluded that the two parameters are more dependent on the proportion of the original bed material in each size fraction.

From the discussion above, the parameters of the equation B (equation 9.) Becomes

$$B = f\,(f_{bmi}, f_{bli}, D_i/D_r \text{ dan } S_0) \tag{10}$$

Figure 6. Shows the relationship between the parameters of equation B, Slope S_0 and the proportion of the original bed material in each size fraction f_{bmi} and figure 7 shows the relationship between the parameters of equation B, the slope S_0 and relative grain size ratio D_i/D_r. While Figure 8 shows the relationship between the parameters of equation B, Slope S_0 and the proportion of bed load transport in each size fraction f_{bli}. Value f_{bmi} and f_{bli} for each size fraction is calculated with the standard normal distribution (Gaussian distribution).

From Figure 6, 7 and 8 the proportion of original bed material either or the proportion of bed load transport in each size fraction, there is the typical relationship between the Slope and the parameter equation B. Therefore, based on Figure 6, 7 and 8 performed research data processing (Multiple Linear Regression Analysis) in order to obtain relations for the parameters (B, S_0 , f_{bmi}, f_{bli} dan D_i/D_r) The results of analysis to get parameter equation:

$$B = 23.699\left(f_{bli}\right)^{-0.302}\left(\frac{D_i}{D_r}\right)^{-0.607}\left(f_{bmi}\right)^{-0.202}\left(S_0\right)^{1.294}$$

With $R^2 = 0.799$ (11)

With the boundary condition, $0.0012 \le S_0 \le 0.02$, $0.125 \le f_{bmi} \le 0.895$, $0.28 \le \frac{D_i}{D_r} \le 12.364$ and $0.401 \le f_{bli} \le 0.996$.

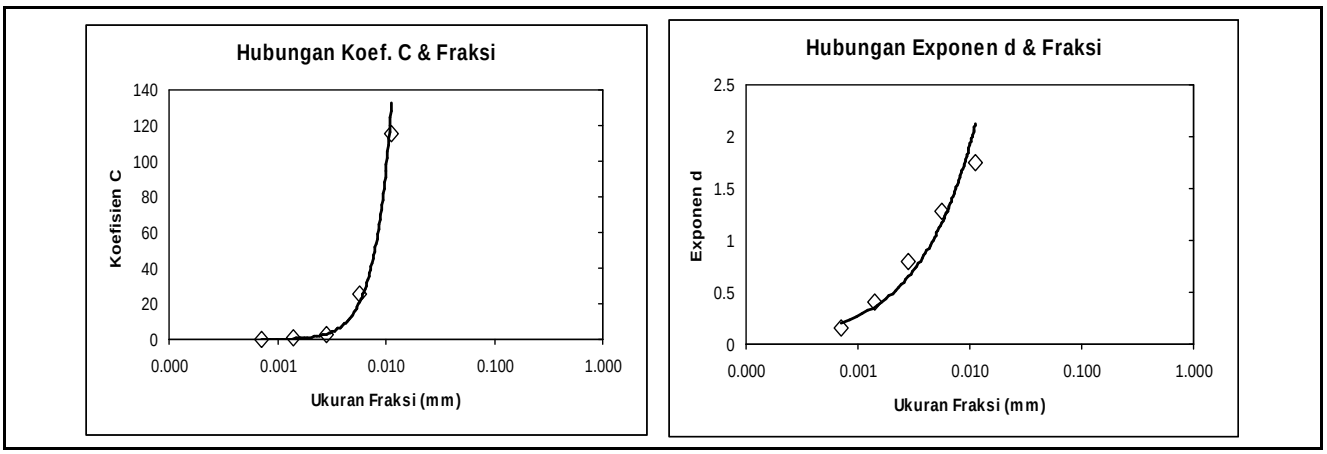


Figure 5. The relationship between the coefficient C, the exponent d and the size fraction

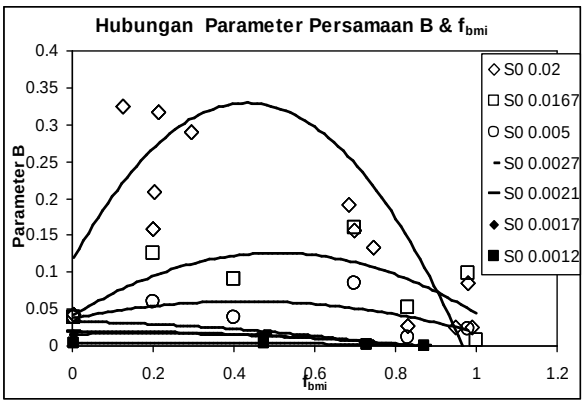


Figure 6. Relations between parameters of the equation B, f_{bmi} and Slope S_0

Figure 6. Shows the relationship between the parameters of equation B, Slope S_0 and the proportion of the original bed material in each size fraction f_{bmi}. From the picture shows the smaller the S_0, the more flat curve near the abscissa. Similarly, on the contrary the greater the S_0, the curve with a peak curve away from the abscissa. The value equation parameter B as the more likely the greater the amount of S_0. Vice versa, the smaller the S_0 values, tend to the smaller parameter of equation B. The value equation parameter B reaches a maximum value on the value of the proportion of bed material / original f_{bmi} for certain, then drops to a minimum value on the value of certain f_{bmi}.

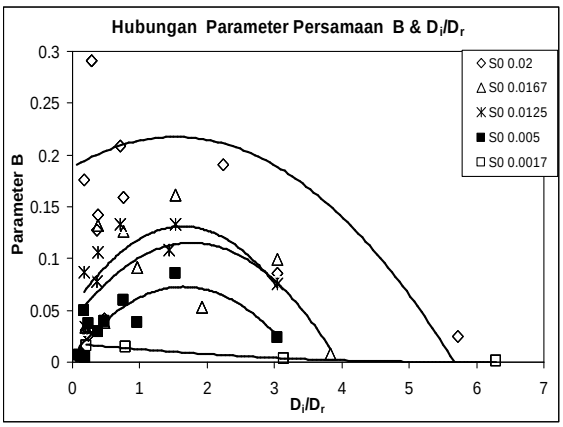


Figure 7. Relations between parameters of the equation B, Slope S_0 and D_i/D_r.

Figure 7. Shows the relationship between the parameters equation B, Slope S_0 and relative grain size ratio D_i/D_r. From the picture shows the smaller the S_0, the more flat curve near the abscissa. Similarly, on the contrary the greater the S_0, the curve with a peak curve away from the abscissa. The value equation parameter B as the more likely the greater the amount of S_0. Vice versa, the smaller the S_0 values tend to the smaller B parameter equation. The value equation parameter B reaches a maximum value on the value of the relative grain size ratio D_i/D_r certain then decreases until a minimum value on the value of D_i/D_r certain also.

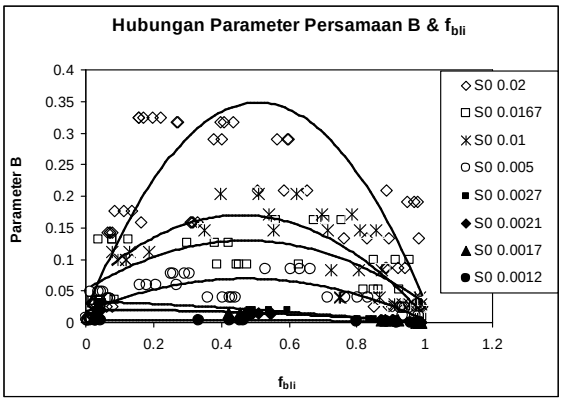


Figure 8. Relations between parameters of the equation B, Slope S_0 and f_{bli}.

Figure 8. Shows the relationship between the parameters of equation B, Slope S_0 and the proportion of bed load transport in each size fraction f_{bli}. From the picture shows the smaller S_0, the more flat curve near the abscissa. Similarly, on the contrary the greater the S_0, the curve with a peak curve away from the abscissa. The value equation parameter B as the more likely the greater the amount of S_0. Vice versa, the smaller the S_0 values tend to the smaller B parameter equation. The value equation parameter B reaches a maximum value on the proportion of bed load transport in each size fraction specific f_{bli}, then decreases until a minimum value on the value of certain f_{bli}.

Based on the reference Schoklitsch (1962) equation (4), for total bed load transport; Shih and Komar (1990) equation (5) and Wilcock and Southard (1989) equation (6) formulation for bed load transport per fraction, the formulation of bed load transport for the size fraction of the proposed equation (8) becomes

$q_{bi} = B(q - q_{ci})$ ..... (12)

where qbi bed load transport for size fraction i ($m^3/s/m$), q the unit water discharge, q_{ci} unit water discharge criticism for size fraction i can be obtained by using equation Bathurst;

$$\frac{q_{cr}}{g^{1/2} D_r^{3/2}} = 0.21 S_0^{-1.12} ; \frac{q_{ci}}{q_{cr}} = \left(\frac{d_i}{D_r}\right)^b ; \; b = 1.5\left(\frac{D_{84}}{D_{16}}\right)^{-1} \tag{13}$$

B is the parameter equation = f(f_{bli}, f_{bmi}, D_i/D_r dan S_0).

$$B = 23.699\left(f_{bli}\right)^{-0.302}\left(D_i/D_r\right)^{-0.607}\left(f_{bmi}\right)^{-0.202}\left(S_0\right)^{1.294}$$

f_{bmi} is the proportion of the original bed material per size fraction i (%), f_{bli} is the proportion of bed load transport per size fraction i (%) obtained from the Gaussian distribution or Rosin. S_0 is the Slope. D_i/D_r ratio relative size of the original bed material grains.

Verification

To test the validity of equation (12) proposed, it is necessary to process the verification of the equation. Verification using the data included in the boundary condition (range data), namely: $0.0012 \le S_0 \le 0.02$; $0.125 \le f_{bmi} \le 0.895$; $0.28 \le D_i / D_r \le 12.364$; $0.401 \le f_{bli} \le 0.996$.

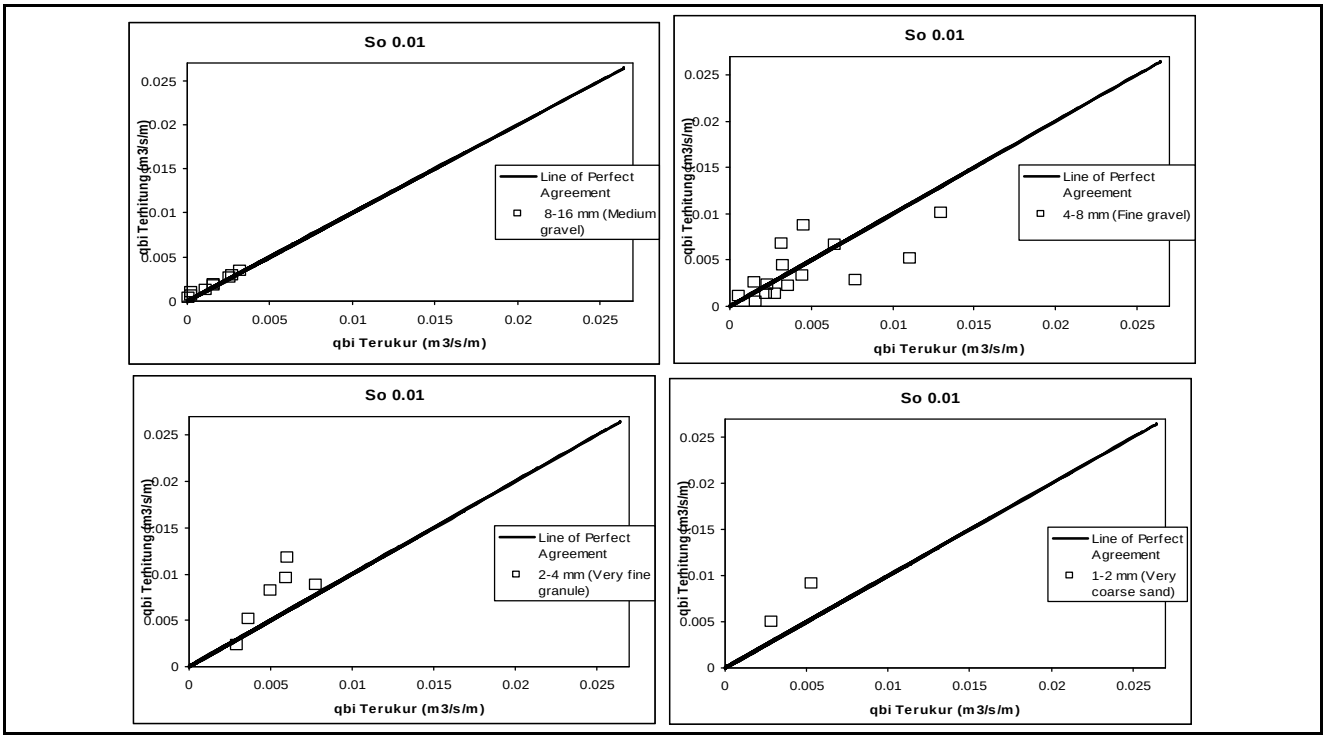
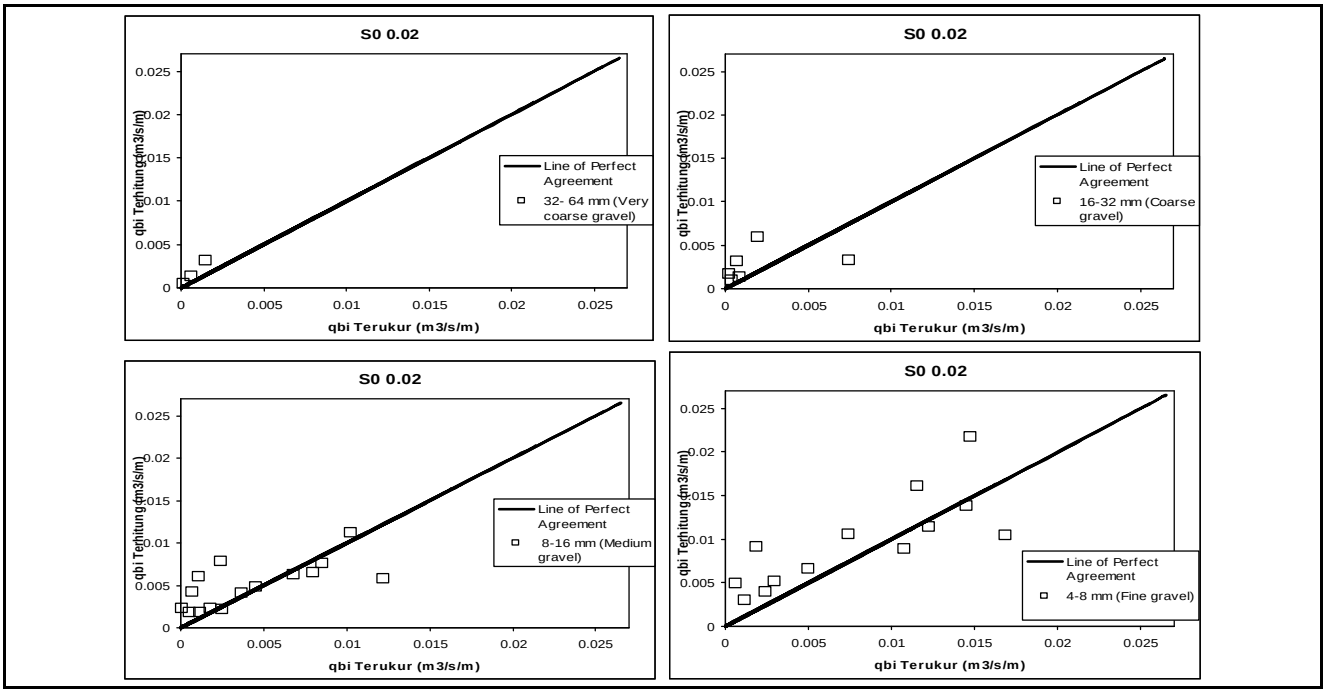


Figure 9. Comparison q_{bi} of Measured with q_{bi} of calculated with Equation (12) proposed, a fraction of S_0 0.01.

In the figure 9. S_0 0.01 indicates that a large fraction (Medium gravel), comparison q_{bi} of measured and q_{bi} of calculated on the line of perfect agreement, but in fractions smaller, comparison q_{bi} of measured and q_{bi} of calculated relatively more scattered.



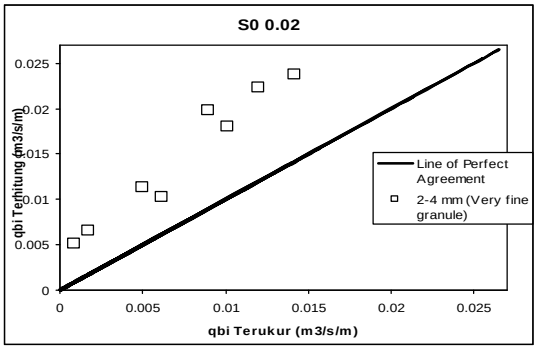


Figure 10. Comparison q_{bi} of Measured with q_{bi} of calculated with Equation (12) proposed, a fraction of S_0 0.02.

In the figure 10. S0 0.002 indicates that a large fraction (Very coarse gravel), comparison q_{bi} of measured and q_{bi} of calculated relative to the line of perfect agreement, but in fractions of a smaller, comparison q_{bi} of measured and q_{bi} of calculated relatively more scattered and on top line of perfect agreement (Very fine granule).

Subsequent verification using data Shubhi (2002), i.e. data not used in the analysis process.

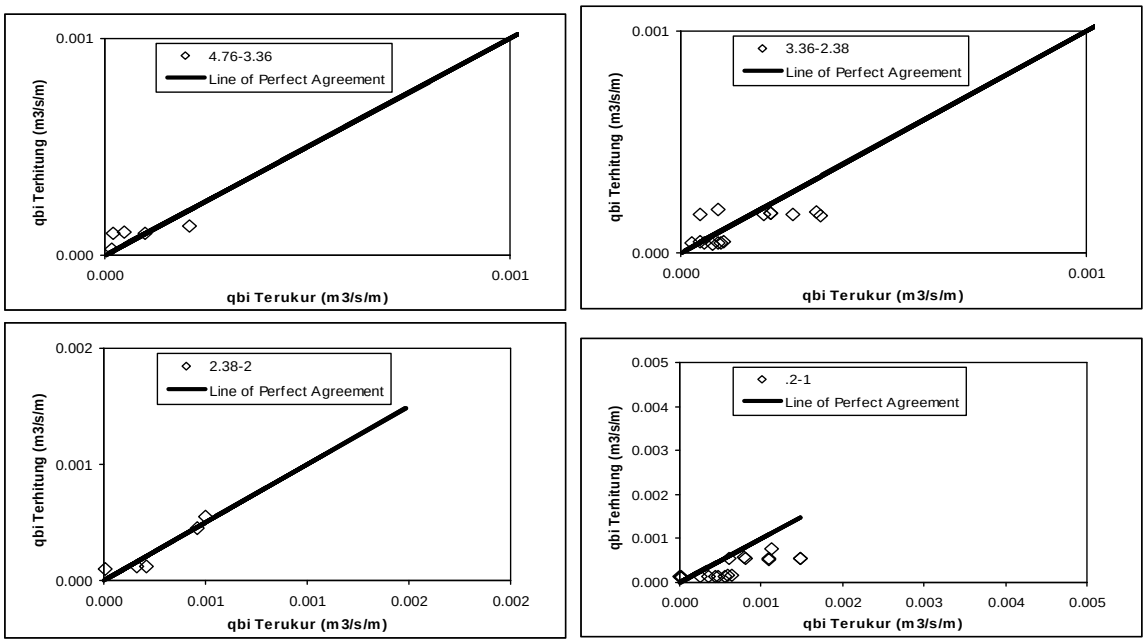


Figure 11. Comparison q_{bi} of Measured with q_{bi} of calculated with Equation (12) proposed, a fraction of S_0 0.005 and S_0 0.00175 (Data Shubhi).

In the figure 11. Shubhi data S_0 0.005 and S_0 0.00175 showed that in all the fractions, comparison q_{bi} of measured and q_{bi} of calculated relative on the line of perfect agreement, but there are relative spread.

CONCLUSIONS AND SUGGESTIONS

Conclusion

Based on observation and discussion, we can conclude several things, among others, as follows:

1. Based on data collected included data of previous research, the selection of functional form simple straight line regression equation showed good results in accordance with the formulation of bed load transport.
2. The discussion for each size fraction is concentrated on three important parameters (dominant), namely: the original bed material, size distribution of bed load transport (fraction size distribution of the non uniform material are, f_{bmi}, f_{bli} and D_i / D_r) and channel slope S_0.
3. In the discussion above the bed load transport depends on the original bed material size distribution and size distribution of bed load transport. This is due to the influence of relative grain size larger than the effect of absolute size of the grain, therefore the non uniform proportion of each size fraction is a very important role in determining the bed load transport in channels and Natural River.

4. In general, for the proportion of the original bed material in each size fraction, the value of B parameter equation becomes greater with increasing slope of the line and becomes smaller with the decrease in channel slope. In other words, the larger the slope, the larger also the bed load transport and vice versa.
5. From the verification results show that there is difficulty to predict bed load transport per fraction in small quantities compared to the per-fraction bed load transport in a relatively larger amount.
6. Parameter Equation (B) of bed load transport for the size fraction is proposed to be: B = Parameter Persamaan = f(D_i/D_r, f_{bmi}, f_{bli} dan S_0)

$$B = 23.699\left(f_{bli}\right)^{-0.302}\left(\frac{D_i}{D_r}\right)^{-0.607}\left(f_{bmi}\right)^{-0.202}\left(S_0\right)^{1.294}$$

Suggestion

1. In the process of grouping the size fractions from various kinds of study data needed careful, precision and patience.
2. It required testing to equations obtained through a more in-depth process of verification, particularly with field data per fraction and type of data is more numerous and varied, and data that are beyond the range that applies to this equation.
3. It should be studied and researched other parameters are quite dominant in bed load transport obtained by other means such as through the process of dimensional analysis.
4. It Need to do further research with more data, varied, accurate with a larger boundary condition for the resulting formula is more valid anymore.

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SANITATION AND BEHAVIORAL STUDY ON ENVIRONMENTAL HEALTH RISK IN THE COASTAL AREA AS A SPATIAL REFERENCE IN DISTRICT KONAWE

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ABSTRACT: Sanitation is one of the most important challenges for developing countries. According to WHO, diarrheal diseases kill one child in this world every 15 seconds, because access to sanitation is still too low. This poses a major environmental health problem, as well as detrimental to economic growth and human resource potential on a national scale. The lack of awareness and community involvement in the management of sanitation, particularly the use and maintenance of sanitation facilities and infrastructure in their own neighborhoods, inadequate sanitation facilities will adversely affect the health and environmental conditions. The focus of this study is to reveal the sanitation and behavior on environmental health risk of the community living in the coastal and river banks area.This study employed a quantitative approach ingathering data through interview and observation. The main sample area consists of 5 districts in coastal area. This sample was choosen randomly and proportionally based on the total neighborhoods in the selected villagesthat have been determined as the surveyed area. The number of samples in each village is 40 respondents.The survey result shows that in the riverbank and in the lowland area, the frequency and schedule of waste management is inadequate (behind the schedule, waste burning practice, unseperating waste). In addition, the level of pollution due to Waste Water Disposal System (SPAL) is still quite high; the majority of septic tank are unstandardized and more than half of the sample households are still practicing open defacation and is under the risk of contaminated of polluted water.

Keywords: *Sanitation, Environment, Waste management, Coastal and lowland area.*

INTRODUCTION

Sanitation is one of the most important challenges for developing countries. According to WHO, diarrheal diseases kills a child in the world every 15 seconds, due to limited access to sanitation. This poses a major environmental health problem, as well as detrimental to economic growth and human resource potential on a national scale. Inadequacy of sanitation facilities will adversely affect the health and environmental conditions. As a consequence, the government encourages the requirement that although to date coverage of sanitation services in urban and rural areas is not adequate. One of sanitation services are inadequate wastewater treatment settlements especially for low-income people in the densely populated neighborhoods, slums and urban sanitation prone.

Environmental health is the healthy level of an environment that includes a housee, sewerage, water supply and so on, all of which categorized into the sanitation sector. When sanitation is decent, the people were in a state of peace, happy which is in opposite case when bad sanitary conditions, it can be a picture that the communities in poor condition or deficiencies in material things.

Bathing and washing clothes in the river and defecate in the open spaces makes people easily exposed to disease, contaminate the ground water surface and degrades soil and shelter women and children at risk. And some people who defecate in the river because they do not have sewers for household waste water and waste water from the shower. It is also still a lot of people defecate found in the yard-yard or in gardens. This happens because the rural communities generally have relatively low educational level and in terms of weak economic factors. With relatively low education they do not understand or do not have insight into the knowledge of environmental health awareness, so in this case they were ignored by the negative impacts.

LITERATURE REVIEW

Sanitation is intentional behavior in the acculturation of clean living human beings for the purpose of preventing direct contact with dirt and other hazardous waste material in expectation this effort will be maintain and improve human health. Sanitation made to ensure the realization of the conditions that meet health requirements. While some other definition focuses on breaking the chain of transmission of germs from resources and environmental control.

Environmental conditions (sanitation) of villages and cities in Indonesia are generally bad. The behavior of people who are still common even "primitive" in treating the environment by throwing garbage and waste carelessly that can

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lead to disease spread to various places. Many houses in the village community are built without having their toilet and defecated in rivers and lakes. (Ahmad, 2014)

Coastal and marine areas of Indonesia's rich and diverse natural resources that have been utilized by Indonesia as one of the main sources of foodstuffs, especially animal protein since centuries. Coastal areas is a transitional area between land and sea, when viewed from the shoreline of coastal areas have two kinds of limits on the boundary parallel to the coastline and boundaries perpendicular to the shoreline. One of the purposes of the management of a coastal region is to control erosion (abrasion) of the beach. (Dahuri 2008 in Ahmad, 2014)

Behavior

Behavior is an activity or activites of the concenrned of organisms (living things). Therefore, from a biological standpoint all living organisms ranging from plants, animals up to human beings behave, because they have their respective activities. So it is with human behavior, in essence is the act or activity of man himself which has a very wide stretch include: walking, talking, crying, laughing, working, college, writing, reading, and so on. From this description it can be concluded that the intended behavior of (human) are all activities or human activities, both of which can be observed directly, or which cannot be observed by outsiders (Notoatmodjo in Ahmad, 2014).

Sanitation Infrastructure

a. Household toilet

Latrine is human waste disposal facilities consisting of a squat or a seat with a goose neck or without neck goose (pit) which is equipped with a septic and water to clean it. The terms of healthy latrines that do not pollute sources of drinking water (the distance between the source of drinking water with minimal hole of 10 meters), odorless, dirt in which cannot be touched by insects and rodents, they do not pollute the soil in the surrounding, easy to clean and safe to use, equipped wall and protective roof, lighting and adequate ventilation, water-resistant flooring and spacious room is adequate, available water, soap, and cleaning tool.

According to Nur (2011) the level of economic and social custom to the latrine ownership is considered capable but people bad habit in most areas of the latrine ownership. To prevent or at least reduce fecal contamination in the environment, the disposal of human waste must be managed properly, that should be done in a particular place or a healthy latrine. (Ahmad, 2014)

b. Waste Water Treatment Plant (WWTP)

Waste water which is not managed properly can cause adverse effects to living things and their environment. Some of the adverse impact is health problems, as well as environmental degradation.

c. Final Disposal (TPA)

Final Disposal (TPA) is a place where the waste reaches the last stage in its management since it began arising at the source, collection, transfer / transport, treatment and disposal. TPA is a place where garbage in isolation safely so as not to cause any disturbance to the surrounding environment. Therefore it is required the provision of facilities and the correct treatment so that security can be achieved by properly.

d. Drainage

Drainage function are as follows:

1) Liberates an area (especially dense settlement) of water logging, erosion, and flooding
2) Minimizes the risk of environmental health; free from malaria (mosquitoes) and other diseases
3) Protects humidity
4) Optimizes ad minimizes damagee to the soil structure and other buildings

e. Clean Water and Drinking Water

Water is the foremost requirement in everyday life. Humans, animals and plants need water for life. Water can also be used as solvents, cleaners and other uses such as household, industrial and other businesses. Water for industrial purposes serves as engine coolant, and cleaning. In addition, water also serves to businesses of agriculture, fisheries, sport, firefighters and others.

METHODOLOGY

Research was carried out in coastal areas Konawe. Primary sampling units (Primary Sampling) are 16 villages / wards. This sampling proportionately and randomly selected based on the total RT in all RW in Village / Sub predetermined becomes the survey area. The number of samples per Village / Sub as many as 40 respondents. Thus the overall number of respondent’s villages / wards was 640 respondents. Respondents were a housewife or married with children, and aged between 18 until 65 years. The instrument used in this study was a questionnaire study and observation sheet. The data which have been collected later was input and analyzed descriptively using SPSS 20.0. Then it is presented in tabular form.

Data collection is an important first phase, because of the completeness and quality of data and information will determine the sharpness of analysis and ultimately determine the quality of the plan which is also to be prepared. The scope of activities includes:

a. Collection of Secondary Data
   These activities include the collection of secondary data including document development policy and spatial policy, basic and thematic maps, aerial photos, satellite imagery, a study / research related to themes and research area.
b. Pre-Field Preparation (instruments of design)
   Several survey instruments include a checklist, a questionnaire about the data to be collected, the questionnaire contains a list of topical questions, interviews, unstructured interviews, and a field measurement sheets.
c. Field survey activities include:
   1) A review of the study area (site-visit), to get a directly visual experience above the location of the research;
   2) The on-site introduction of neighbourhood conditions;
   3) Documentation of the area condition in form of photographs, slides, and sketches;
   4) Field measurement;
   5) Interviews and questionnaires;
   6) Community participation through discussions, workshops.

RESULTS AND DISCUSSION

The dominant age level of respondents is ≥ 45 years; the proportion reached approximately 25.8%. The next group which is also quite dominant age group 31-35 years was almost covers 17%. Most respondents work as a housewife which is about 96.6% of the total respondents. Where the majority of respondents education is only at the level of elementary school which is about 33.5%. While others graduated from junior high school which is about 30.4%. The smallest percentage of respondents is a university education, only about 3.8%.

Based on data obtained indicate that the source of environmental health risks includes; risks related to sources of clean water which consists of the use of unprotected water sources, use of unprotected water sources, water shortages experienced by households. Furthermore, the source of water-related risks of domestic waste, consisting of suspected septic tank safe, pollution due to discharge of septic tank contents, ownership Sewers Waste (SPAL). Sources of risk are associated with the waste that consists of waste management, waste transportation frequency, timeliness of waste transportation, waste management local. Other risks that sources of unhealthy behaviors that provide opportunities risk exposure, i.e. behavior Hand washing, defecation Gratuitous, pollution in water storage containers and handling.

Table 1. Risk Area of Solid Waste

Waste Management	**Respondent**	**%**
Collected by informal collectors who recycle	0	0
Collected and disposed of to the polls	27	4,22
Burnt	277	43,28
Dumped into the hole and covered with soil	14	2,19
Dumped into the hole but not covered with soil	100	15,63
Discharged into the river / times / sea / lake	67	10,47
Left alone to rot	19	2,97
Dumped into vacant land / garden / forest and left to rot	116	18,13
Others	20	3,13
Unknown	0	0
Total	**640**	**100**

Table 1 above shows that the frequency of waste transportation and timeliness of transporting waste is inadequate and not timely reach a percentage of 100%. As for waste management by burning is 43.28% and local waste processing by means of discharged into the river / times / sea amounted to 10.47%.

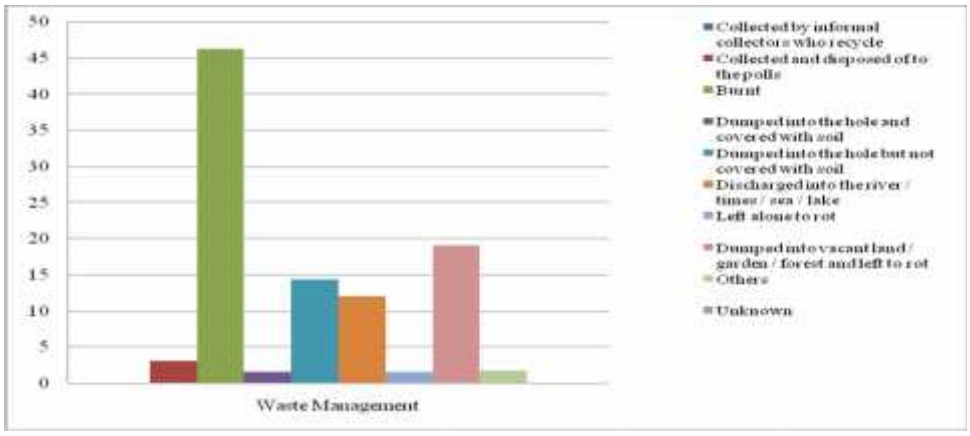

Fig. 1 Graph Percentage of Waste Management

Waste management as seemingly in Figure 1 graphs the percentage of waste management, shows that the largest waste management practices are burned with a total of 46.3% of respondents, banished to the polls conducted by the respondents own a total of 3.1%, dumped into the hole and covered with soil 1.6%, dumped into the hole but not covered with soil 14.4% and thrown into the river / times / sea / lake 12.1%, was left to rot 1.6% of respondents, dumped into vacant lots and left alone to rot 19.1% of respondents, other 1.8% of respondents.

In general, the details of how the disposal of the above then simplified by two broad categories, namely: 1) the service recipient rubbish and 2) non-service recipient garbage. For people who are not recipients of non-garbage services, they aren't sorting garbage before disposal it. As for the people who are recipients of the services are always throwing themselves turns trash garbage to landfills.

Table 2 Risk Area of Domestic Wastewater

Variabel	Category	Respondent	%
Safe suspected septic tank	not safe	175	27,34
	suspect safe	465	72,66
Pollution due to discharge of the contents of septic tanks	not safe	6	0,94
	Yes, safe	0	0,00
Pollution due Waste Water Disposal System	not safe	234	36,56
	Yes, safe	406	63,44

Table 2 shows that the whole several villages / villages in the coastal areas with a total of 92.7% of respondents to the pollution due to discharge of the contents of the septic tank (not safe). To the level of pollution because waste water disposal system is still quite high at 36.56% despite a safe at 63.44%. Globally for safe suspected septic tank variables still worrying only is at 72.66%, of which 27.34% are unsafe.

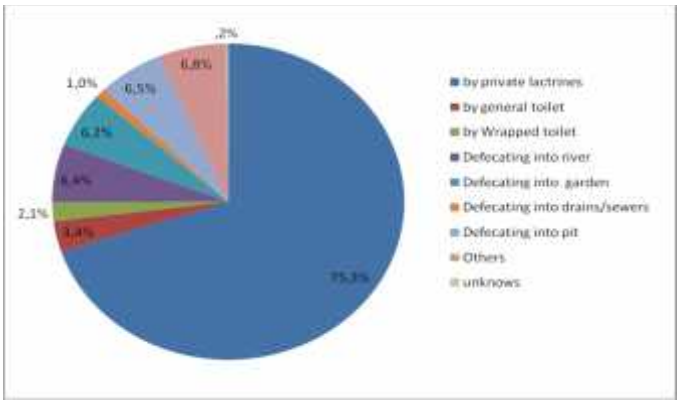

Fig. 2 Graph Percentage of defecation place

Figure 2 is the percentage of defectaion places. The figure shows that those who go to lactrines is 75.3%, defecating into rivers 6.4%, other 6.8% (still defecate in anywhere), toilet /WC General is 3, 4% of respondents. In addition to the pit of 6.5% and to 2.1% of wrapped toilet and into the garden /yard 6.2% as well as into sewers /ditches /drains of 1.0%.

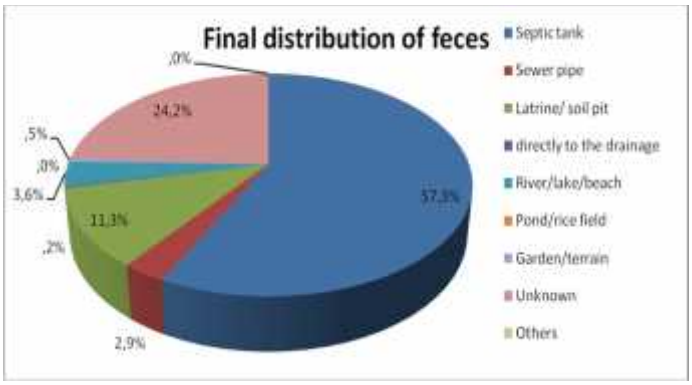

Fig. 3 Graph Percentage of The Final Distribution of Feces

The fig. 3 charts the final distribution of feces for disposal of septic tank to pretty much that is 57.3%, 2.9% sewer pipes, latrine / pit soil 11.3%, 0.2% directly to the drainage, river / lake / beach 3.6%, a pond / rice field 0.0%, garden / terrain 0.5% and 24.2% unknown.

Environmental Drainage / Sewer Neighborhood Home and Flooding

Table 3 Risk Area of Puddle

Kategori	Responden	%
There is water (flooding)	338	52,81
No puddles	302	47,19

Table 3 areas at risk of water logging shows that 52.81% (338 households) of total respondents around their house there are no puddles. While there are 47.19% (302 households) of total respondents around the house there is water logging or flooding. This situation is common for the area on the riverbank or to the characteristics of its land area is rather low when compared to its neighbors.

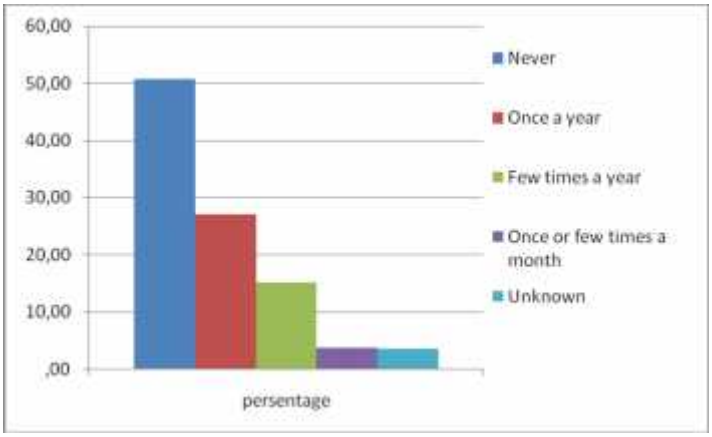

Fig.4 Graph Percentage of Households Who Ever Experiencing Flooding

The chart 4 above illustrates that the entire Village/Sub the total area of the survey respondents indicated there was 50.78% who have never experienced flooding, 27.03% which experienced flooding once a year, 15.16% a few times a year, 3.59% once or more in a month and 3.44% unknown.

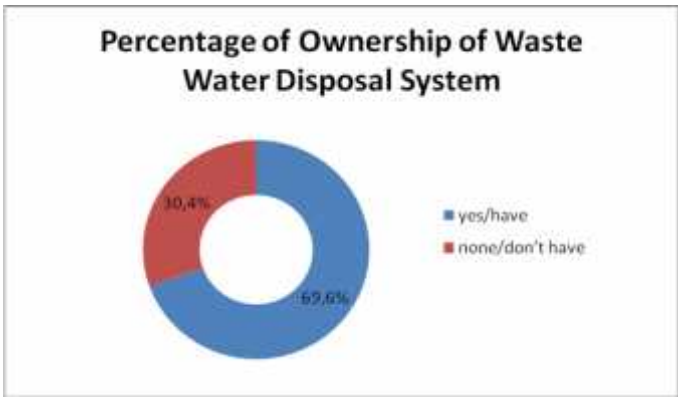

Fig. 5 Graph Percentage of Ownership of Waste Water Disposal System

Based on the chart 5 for the percentage of ownership Waste Water Disposal System, there are 69.6% of respondents who have Waste Water Disposal System and 30.4% of respondents who do not have Waste Water Disposal System. Waste Water Disposal System's ownership in the survey area is good, because it has begun the public awareness of the importance of ownership of Waste Water Disposal System.

Household Drinking Water Management

Table 4 Water Resources Based on Risk Area

Variabel	category	Responden	%
Protected water sources	No, the risk of contaminated water sources	321	50,16
	Yes, water resources are protected	319	49,84
The use of unprotected water sources	not Safe	262	40,94
	Yes, Safe	378	59,06
water scarcity	Face water shortages	90	14,06
	Never experienced	550	85,94

Based on Table 4 on risk areas of water sources, it appears that for all the Village / Village using improved water sources are not contaminated with the percentage of 50.16% (in the form of a water source tap water taps, public taps, public hydrants, artesian well hand pumps, wells dig protected, rainwater and water refills) and are at risk of contaminated 49.84%. While those using unprotected water sources that are not securing 40.94%, 59.06% safe. At all villages / wards, by 85.94% of the total percentage of never experiencing water scarcity and only 14.06% had ever experienced shortages.

Fig. 6 Household drinking water sources
Source: Observation 2015

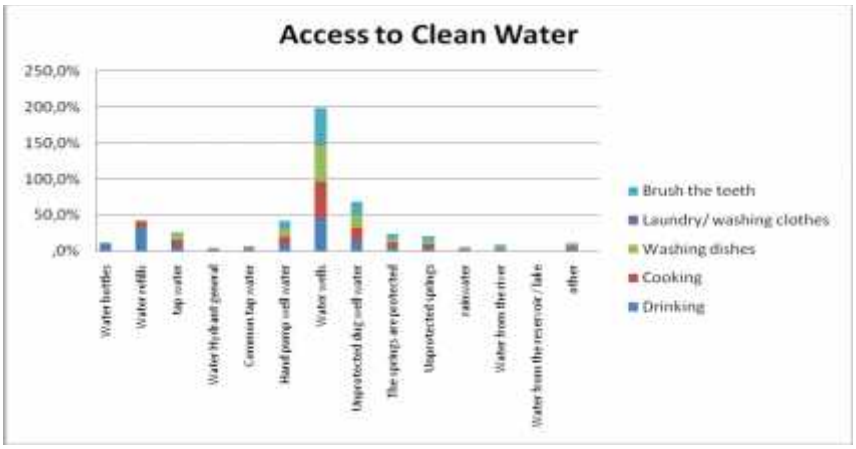

Fig. 7 Graphic of Access to Clean Water

Fig. 7 graphic of Access to Clean Water above illustrates the respondent uses for drinking water for 38.6% of the refill water, then water 34.1% protected dug wells, water wells are not protected 12.5% and there are those who use the water the river as a source of drinking water by 3.2%. The graph above also shows the use of water to cook, a lot of use of water from unprotected dug wells amounted to 49.3% of the total percentage of respondents, then from unprotected dug wells amounted to 20.7%, 12.3% hand pump, tap water taps 5,7%, 5.2% water refills, and some even use the river water by 3.4%. To wash the dishes (46.6%), washing clothes (45.2%) and brush teeth (45.7%) use water from unprotected dug wells as the main source of water. The use of unprotected wells for washing dishes (20.5%), washing clothes (19.8%), and brush teeth (19.8%). Hand pump well water for washing dishes (13.9%), washing clothes (13.9%), and brush teeth (14,15). While the use of tap water taps for washing dishes (9.5%), laundry (9.5%) and brush teeth (10%). The use of river water for washing dishes (6.1%), laundry (7.5%) and brush teeth (6.6%). While access to water from the reservoir water for washing dishes (0.9%), laundry (1.1%), brush teeth (1,1); Rainwater to wash the dishes (0.7%), laundry (0.9%) and brush teeth (0.7%); Public tap water for washing dishes (0.2%), laundry (0.2%) and brush teeth (0.2%); refill water for dish washing (0.5%), laundry (0.5%), brush teeth (0.5%); from unprotected springs to wash the dishes (0.5%), laundry (0.5%), brush teeth (0.5%); Others for washing dishes, washing clothes and brush all 0.2%.

Hygiene and Sanitation Behavior

Table 5 Risk Area based on Behavior Hygiene and Sanitation

Variabel	Category	Responden	%
CTPS in five critical time	Not	476	74,38
	Yes	164	25,63
Are the floors and walls of the pit are free of feces?	Not	232	36,25
	Yes	408	63,75
Do latrines free from cockroaches and flies?	Not	250	39,06
	Yes	390	60,94
flushes functioning.	Not	319	49,84
	Yes, work	321	50,16
Is there soap seen in or near the latrine?	not	319	49,84
	yes	321	50,16
Pollution in water storage containers and handling	Yes, polluted	138	21,56
	not, polluted	502	78,44
Defecation behavior	yes, BABS	309	48,28
	not	331	51,72

The table above about hygiene and sanitation behaviors seen that 74.38% of respondents do not make a habit of CTPS in five critical time such as: Before eating, after eating, after defecation, after cleaning baby's feces, before feeding a child, before preparing food and after handling animals and before prayer. Many respondents did CTPS is only done after a meal alone. While doing CTPS only 164 respondents (25.63%). For hygiene behavior latrine floor and walls are free of fecal there 408 respondents (63.75%) and were still dirty not free from feces of 232 respondents (36.25%). Then for latrines free from cockroaches and flies there are 390 respondents (60.94%) and that is not free there are 250 respondents (39.06%).

While flush functioning owned by 321 respondents (50.16%) and do not work owned by 319 respondents (49.84%). For soap visible in latrines or no latrines near the 321 respondents who had (50.16%) and do not have the 319

respondents (49.84%). This shows that more than half of the respondents own awareness of proper behavior PHBS. Pollution in water storage containers and handling uncontaminated by 502 respondents (78.44%) and there were 138 contaminated respondents (21.56%). While the behavior of defecation, of 331 respondents (51.72%) had not done defecation and 309 respondents (48.28%) who still do defecation behavior. This shows that more than half of the respondents own awareness about defecation in proper behavior. Especially with the incessant program of the Department of Health through the Division of Environmental Health in the form of Community-based Total Sanitation program.

Fig. 8 Latrines used by coastal communities
Source: observation, 2015

Sanitary Risk Index

Variable	Category	%
Water resources protected	No, the risk of contaminated water sources	50,3
	Yes, water resources are protected	49,7
The use of unprotected water sources.	yes	39,7
	not	60,3
Water scarcity	yes	8,9
	not	91,1
Suspected septic tank safe	not	24,2
	yes	75,8
Pollution due to discharge of the contents of septic tanks	yes	100,0
	not	,0
Pollution due Waste Water Disposal System	yes	34,7
	not	65,3
Waste management	not	96,9
	yes	3,1
Frequency waste transportation	inadequate	100,0
Timeliness waste transportation	Not on time	,0
Processing of local trash	unprocessed	95,9
	be treated	4,1
The existence of puddles	yes	53,3
	not	46,7
CTPS in five critical time	Not	78,0
	Yes	22,0
Are the floors and walls of the pit are free of feces?	Not	34,3
	Yes	65,7
Do latrines free from cockroaches and flies?	Not	37,6
	Yes	62,4
flush functioning.	Not	49,9
	Yes, work	50,1
Is there soap seen in or near the latrine?	not	49,9
	yes	50,1
Pollution in water storage containers and handling	Yes, polluted	17,5
	not, polluted	82,5
defecation behavior	yes, defecation	48,1
	not	51,9

Environmental health risk assessment is one way to provide an assessment environmental health of hazard exposure opportunities, especially in coastal areas of Konawe. Determination of the risk, is done by assigning weights to the

elements of danger and hazard exposure opportunities. The weights given to each variable component hazard and exposure opportunities danger then added to get the value of Sanitary Risk Index (IRS). The calculation results obtained under sanitary and environmental behavior in coastal areas Konawe explained that the major environmental health risks in coastal areas is included in the moderate risk level.

CONCLUSION

The conclusion from this study is that there are environmental health risks in coastal areas of Konawe include covering risks related to sources of clean water which consists of the use of unprotected water sources, use unprotected water sources, water shortages experienced by households. Furthermore, the source of water-related risks of domestic waste, consistt of suspected septic tank safe, pollution due to discharge of septic tank contents, ownership Waste Water Disposal System. Sources of risk associated with the waste consists of waste management, waste transportation frequency, timeliness of waste transportation, waste management local. Other risks that sources of unhealthy behaviors that provide opportunities risk exposure, i.e. behavior Hand washing, defecation Gratuitous, pollution in water storage containers and handling. Thus, the greater the risk of environmental health in coastal areas of Konawe is risk moderate level.

It is suggested to local governments Konawe particularly the Department of Health, it is necessary to strengthen health systems, especially in coastal areas, including the fulfillment of the need for clean water through a clean water supply or through brackish water treatment program. In addition, there is a need to improve toilet facilities, especially public toilet facilities which are already available. Moreover, environmental health risk assessment in Konawe especially coastal areas would be the basis for making decisions in formulating policy, service planning, and allocation of funds.

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BED LOAD TRANSPORT AT DOWNSTREAM SIDE OF PALU RIVER AROUND FOURTH BRIDGE

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ABSTRACT: Bed load transport research was found in a thousand times, but, part of them was done in a laboratory, built on statistical data and field work research. Palu river has a characteristic for fast stream, and carried sediment such as, suspended sediment, surface sediment, and bed load transport sediment. Most of the Palu River research activity was done in the laboratory, for this research was done directly in downstream of Palu River a round of fourth bridge. Bagnold equation is the equation which has token data from the river directed, and combine between those data with the Bagnold equation for calculating bed load transport in downstream Palu River around fourth bridge result that bed load transport was higher than founded sample.

Keywords : *Palu river, Bedload Transport, Bagnold Equation.*

INTRODUCTION

Bed load transport was occurred in three modes of transport, namely; rolling, sliding and saltation, may occur simultaneously in bed load transport.. The different modes of transportation are closely related and it is difficult, if not impossible, to separate them completely. Few bed load discharge equations were available when the systematic regulation of rivers began (A. Recking. 2010). To know the position of bed sampler it is better to ensure the bed load is on the river bed first. Based on the experience, shallow water has no high dune at the surface of the river bed. Somehow, this location chosen to laid bed sampler.

Some researches were done to predict the sediment transport at the downstream of Palu River. These researches were done using material of bed and move to laboratory research. Values of sediment transport were founded by sediment transport formula. Bed load transport measured indirectly to the river hasn't done yet.

The higher overall accuracy of a formula does not guarantee that the formula is superior to the others under all flow and sediment conditions. The accuracy rating of a formula may vary depending on bed slope, grain size diameter, and other hydraulic and sediment data. This indicates that we cannot predict bed load transport, discharge with any degree of reliability without an adequate number of observations (Arman Haddadchi,2013).

Bagnold (1966) discussion about moving bed load sediment in Van Rijn (1993), note that moving of bed load is because of force per unit area. Such as bed shear stress, gravity component, tangential stress and normal stress. Bed load prediction is of primary importance for river engineering, fluvial geomorphology, eco-hydrology, environmental surveys and management, and hazard prediction.

This study shows how to calculate the bed load transport using bed load catcher and calculated using Bagnold equation. Improving the knowledge about the bed load transport at bridge 4,might be represented trough graphics or figures, such as, figure of comparisons and figure of correlations.

According to Van Rijn (1993) The transport of bed material particles by flowing water can be in the form of bed load and suspended load. Depending on bed size of material, and the flow condition. Although in natural condition, there is no sharp division between the bed load transport and suspended load, it is necessary to define layer with bed load transport for mathematical representation.

This study also developed acknowledgment between parameter of bed load material, volume and weight of bed load material. The difference between two bed load values such as bed load values using Bagnold formula and bed load value from the river observation.

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STUDY AREA

Palu River is the river which has medium sediment concentrate inside the water. Research location placed at 0°54'32.70"S and 119°52'8.47"E. This location is chosen because it has a uniform flow and uniform grain bed load, and it can be seen visually. This location has a distance of ±2.96 Km. The average depth of the river at this location ± 1.96 m. General overview of the study site can be seen in Figure1below:

Fig. 1 Palu River

This research was conducted on the river bed around the fourth bridge downstream of Palu River. First able, determine the location which is the current flow around bridge 4 downstream of Palu River. (Fig.1).

METHODS

This study is a field study comparing bed load transport that was obtained by using the tool catcher bed load with the calculated bed load transport using Bagnold equation. Fieldwork was conducted by making Riverbed granules catcher. The device is placed on the bottom of the river for 3 hours, from 8 am to 11 am.From the observation before the study is done within 8 to11 hours and flow of water is similar to the uniform, is not influenced by the tide.

The use of transport in the form of a boat and basic sediment catcher tool is needed in this study. The components such as a boat anchor , buoy, water flow rate measuring device is also in need. The bed load material obtained from the riverbed using a shovel and the sediment catcher tool in meticulous in the laboratory to get the value of the characteristics and parameters of the rock.

Fig 2. River Grain bed load sediment Catcher

The observations were done also to determine bed load grain catcher in located, it is done by crossing the river and feel and measure using bamboo to determine whether the place is flat and deserves to be placed sediment grains catcher. To know bed load transport directly from the river, it is necessary to build the tools to catch bed load transport directly from the river (Fig.2).

Fig 3. Research Boat to transporting grains catcher

Other equipment that was made for this research is the research boat. This research boat made with a hole in the middle of the boat to take down the sediment grain catcher.

Fig 4. Transporting grains catcher by research boat

Research boat also built in order to transporting sediment catcher (Fig 3, Fig. 4).

Transporting equipment to the research study site in the river is the main activities when start researching. The data is taken directly from the river, the river flow rate, measuring the width of the river, and the river bed material

Fig 5. Flow and depth of the river at research location

Having arrived at the location, a tool for granular bed load catcher was placed at a location that has been set. To ensure that the tool does not move, the device uses a rope and anchor. Then, marking the location of the device in the river using floating equipment.

Fig 6. Floating tools to remarks the grains sediment catcher position

Fig 7. Grains found by grains sediment catcher

Measurement the velocity of flow was done in order to know the velocity of current flow (Fig. 5). Sediment catcher below the surface water was signed by the buoys tool (Fig.6).

RESULT AND DISCUSSION

After three hours of instrument is placed in the bottom of the river, the grains catcher was lifted, then brought to the research camp to remove and dried,separated, gravel, sand and other useless material. The result of river bed load transport using field research can be seen below :

Table 1. River bed load transport

Var	$\tau_b = \rho g z i$		v	$q_{b\,B}$	Sediment Catcher Result
	ρ	z			
	Kg/m^3	m	m/s	Kg. m/s	Kg. m/s
J_4Ra_1	1.65	-0.48	0.87	1.78E-04	1.53E-05
J_4Ra_2	1.64	-0.52	1.08	2.40E-04	6.60E-05
J_4Ra_3	1.64	-0.36	1.08	1.66E-04	6.57E-05
J_4Ra_4	1.62	-0.34	1.23	1.83E-04	3.12E-05
J_4Ra_5	1.66	-0.43	1.23	2.23E-04	1.10E-05
J_4Ra_6	1.65	-0.94	1.23	4.92E-04	2.95E-05
J_4Ra_7	1.63	-0.83	0.87	3.14E-04	2.04E-05
J_4Ra_8	1.63	-0.92	0.82	3.27E-04	1.46E-05
J_4Ra_9	1.64	-1.00	0.87	3.74E-04	1.41E-05
J_4Ra_{10}	1.65	-1.50	0.93	5.89E-04	2.06E-05
J_4Ra_{11}	1.64	-2.00	0.87	7.48E-04	1.46E-05
J_4Ra_{12}	1.64	-2.20	0.87	8.23E-04	3.86E-05
J_4Ra_{13}	1.64	-1.00	1.08	4.62E-04	3.93E-05

In other hand, the study is to take direct using bed load grain shovel. And measure the depth of the river. The result of river bed depth in cross section area can be seen below :

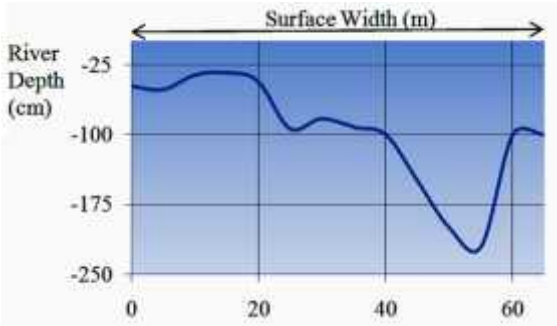
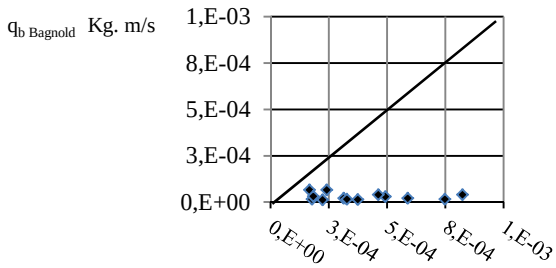


Fig 8. River bed cross-section around bridge 4

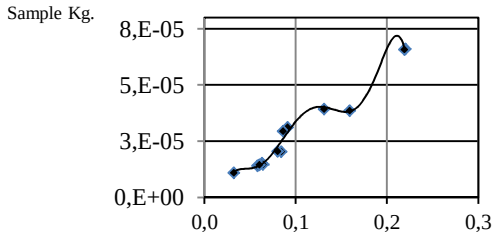
- $q_{b\ river}$ = sample weight x current velocity (1)
- $q_{b\ Bagnold} = \{e_b(\rho_s - \rho)^{-1}\}\left\{(\tau_b \bar{u})\left(g \cos\beta\,(\tan\phi - \tan\beta)\right)^{-1}\right\}$ (2)



Sediment Catcher Result, Kg. m/s

Fig 9. Comparison $q_{b\ river}$vs $q_{bSediment\ Catcher}$

River depth measurement results in cross section result from research sites were depicted in graphic form (Fig. 8). In Figure 9 above shows that the sediment transport of sediment in the river catches were higher than the calculation of sediment transport using Bagnold equation.



River Data , $q_{b\ River}$, Kg. m/s

Fig 10. Correlation W_{sample}vs$Q_{b\ Bagnold}$

Thirteenth granular sampling sites designed to collect sediment granular, From January 15th to February 18th in 2012 working for granular data collection, from February 23th to April 18th in 2013 working for desk work to calculate bed load transport.

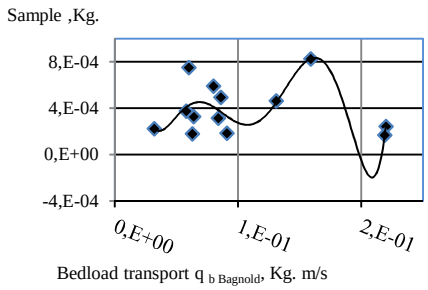


Fig 11. Correlation $w_{granular}$vs$Q_{b\ River}$

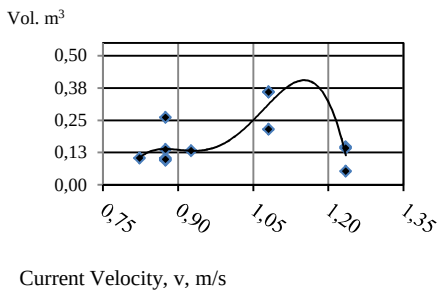


Fig 12. Correlation Volume of Granular and current velocity

The granules present in the sediment catcher, are removed and the drying, separated from other materials (Fig. 7).

CONCLUSIONS

Value bed load transports which are calculated using the Bagnold equation were higher than value of transport sediment were calculated using river data observation.

The correlation between sample weights (W) and Bagnold Bedload values has a good coefficient of correlation, it was yield:

$y = -167.4x^6 + 117.0x^5 - 31.47x^4 + 4.101x^3 - 0.268x^2 + 0.008x - 9E\text{-}05$, with coefficient correlation, $R^2 = 0.989$, this equation represented the value of "y" as the weight of the sample and the value of "x" as a bed load transport using Bagnold formula.

The correlation between sample weights (W) and bedload values which result of sediment catcher has correlation:$y = -230.3x^4 + 908.2x^3 - 1333.x^2 + 864.2x - 208.7$ with coefficient correlation, $R^2 = 0.670$, this equation represented the value of "y" as weight of the sample and the value of "x" as a bed load transport using Sediment Catcher.

Correlation Volume of Granular and current velocity yield: $y = 7396.x^6 - 5175.x^5 + 1400.x^4 - 185.4x^3 + 12.48x^2 - 0.398x + 0.005$, with coefficient correlation, $R^2 = 0.539$, this equation represented the value of "y" as the volume of granular and value of "x" as current velocity.

The difference in the value of the coefficient of correlation can be the difference in the parameters used in this study. Moreover, the value of the correlation coefficient can be managed again by adding other parameters such as grain size, and form factors. Can be done to further research.

ACKNOWLEDGMENT

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IN-SITU MEASUREMENT ON PHYSICAL PROCESS DURING DEVELOPMENT OF SALINITY STRATIFICATION DUE TO FLOOD EVENTS IN THE NORTHERN ARIAKE SEA, JAPAN

Shinichiro YANO[1], Yohei KITAGAWA[2], Camilla RANLUND[3], Akira TAI[4], Sangyeob KIM[5]and Akihide TADA[6]

ABSTRACT: A concept of the "regions of freshwater influence (ROFIs)" was proposed by Simpson (1997). Generally, ROFIs are considered as one of key factors to determine conditions of marine environment and ecosystem in coastal areas. We focus on freshwater dynamics in the gulf type ROFI of the Ariake Sea, Japan. To understand salinity stratification due to freshwater input from multiple rivers through flood events, *in-situ* measurement for 25hours on tidal current, CTD, turbulence micro-structure, and suspended solid (SS) grain size property was conducted at a single station in the northern Ariake Sea, Japan, twice (in neap and spring tides) in July, 2014. From the results of the measurement, the followings were seen: 1) Dumping process of vertical eddy viscosity due to stratification dynamics was measured; 2) Turbidity due to suspended solids transported by freshwater and one due to resuspention of bottom sediment were distinguished in the measurement result; and 3) Reducing process of DO in the bottom layer through the stratification development in neap tide and disappearing process of anoxic water during well mixing in spring tide were clearly measured.

Keywords: *Ariake Sea, stratification, baroclinic flow, ROFIs.*

INTRODUCTION

Recently, especially after 2000, severe deterioration of aquatic environment has been recognized in the Ariake Sea, which is one of the largest semi-enclosed bays in Japan, such as Tokyo Bay, Ise Bay, and Yatsushiro Sea. Some researchers point out the construction of Isahaya Sea Dyke (see Fig. 1) as the most significant reason for this problem. That is a reason why they assert the necessities of longterm observations, which must continue for five years, with opening two drainage gates in the dyke in order to clarify the dominate reason. Due to contradictive court decisions it is as of yet not certain that the measurements with open gates will be possible.

Many researchers have been trying to investigate the principal reason. In a coastal area off Shimabara Peninsula where the path of the ebb current towards the mouth of the Ariake Sea from Isahaya Bay can be seen, significant reduction of tidal current has been observed due to the decrease of the volume of Isahaya Bay due to the dyke (Nishinokubi *et al*., 2004). Meanwhile, some recent numerical investigations (*e.g*., Manda and Matsuoka, 2006; Tai *et al.*, 2006)have shown that the dyke does not significantly contribute to the decrease of the tidal current in the innermost part of the Ariake Sea with the exception of the Isahaya Bay, whose surface area was reduced to 2/3 by the dyke. Tai and Yano (2008) have clarified by using the harmonic analysis of tidal data for a long period that the closing can show nonlinear influence on tides and reduce the variation of tide amplification due to the change of M2 tidal amplitude with 18.6 years lunar nodal cycle. The construction of Isahaya Sea Dyke cannot be considered as the main factor of reduction of tide in the Ariake Sea, but decrease of tide in the outer ocean can cause it significantly (Tai *et al*, 2006). We can deduce from these results that the closing doesn't influence on barotropic tide and tidal current to a large degree.

On the other hand, dynamics of baroclinic structure according to river discharge and meteorological condition in the bay has not been examined in detail, although static baroclinic structure has already been investigated such as mean residence time and estuary circulation. Especially, there is no long-term data except a regular monthly survey for salinity, water temperature, and so on, in a few layers at the fixed points by local prefectural governments. Furthermore, it is not possible to acquire dataset demonstrating the dynamic process of density stratification before the closing. Hence, impact assessment of the dyke can have no choice but to depend on numerical analysis realistically. Yano and Nishimura (2014) evaluated physical influences on salinity stratification due to existing of the dyke by using 3-dimensional hydrodynamic model DELFT3D. They indicated that the stratification can significantly occur after large flushing by floods in spring tide while the effects to enhance stratification are restricted in neap tide. This means that the

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influence of the dyke is not strong for hypothesis that vertical mixing has been suppressed due to reduction of tidal currents by the dyke, and strengthened stratification causes developing anoxic water in a bottom layer. Besides, the result suggests that the main factor is the influence of operation of drainage gates of the dyke or change of the position of freshwater inflow from rivers which are located around inside of control pond separated by the dyke. However, it is a fact that there is a skeptical view, which such simulations unable to express actual phenomenon accurately. Consequently, theyneed to conduct*in-situ* measurement on the changes of dynamic baroclinic structure in details and impact assessment by using high precision numerical analysis possible to reproduce the measurement. In this investigation, we focused on flood events in rainy season and conducted measurements twice in the northern Ariake Sea during neap tide and spring tide respectively. In the present paper, we investigate the process of developing salinity stratification from the results obtained during the two measurements.

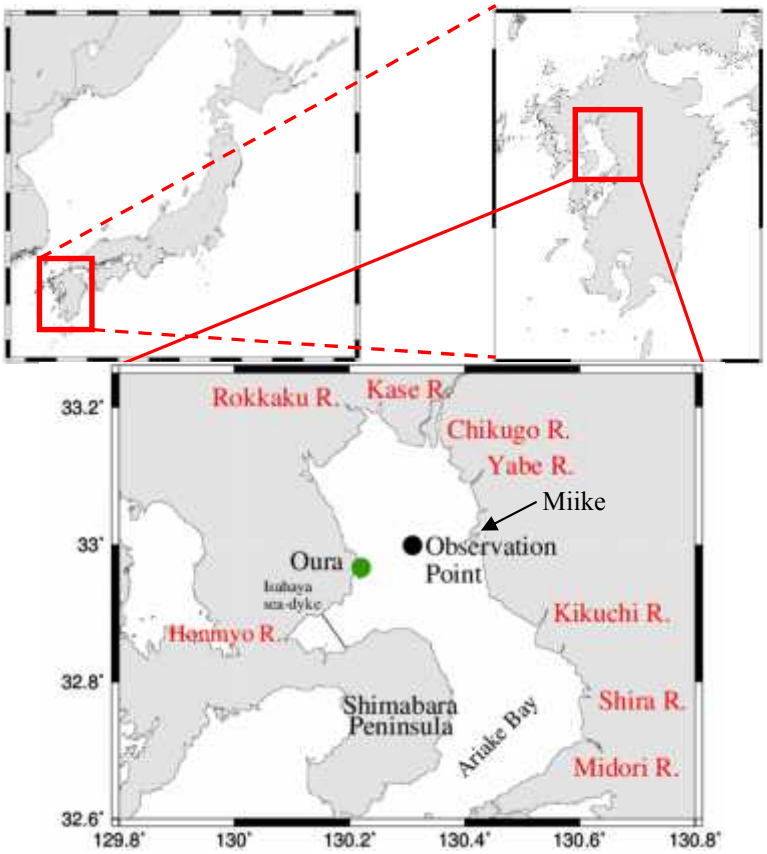

Fig.1 Location of the Ariake Sea, observation point and A-class rivers in the bay.

METHODOLOGY.

In-situ Measurement

Fig.1 shows the observation point (32°59’54”N, 130°18’25”E). The observation point is located in the center of cross-section which connects Oura with Miike (Fig.1), as an entrance of northern area of the bay. The observation point was selected from a reason why the point is susceptible to freshwater supplied from surrounding A-class rivers including the Chikugo River which is the largest river in Kyushu. This point is suitable to see the effects of freshwater due to seven A-class rivers (Chikugo River, Yabe River, Kase River, Rokkaku River, Kikuchi River, Shira River, and Midori River). In the Northern hemisphere, freshwater discharged from river can flow with the shore on the right-hand side due to influence of the Coriolis force, which causes horizontal circulation to flow in a counterclockwise direction. In case that a large scale flood occurs from the Chikugo River, freshwater can be transported directly into Isahaya Bay [Saita *et al*., (2009)].

The first measurement was conducted in neap tide (from 18:00 [JST] on 3 July, 2014 to 18:00 [JST] on 4 July, 2014), and the second one in spring tide (from 14:00 [JST] on 11 July, 2014 to 15:00 [JST] on 12 July, 2014). The observation ship was the Kakuyo-maru (155ton) belonging to Faculty of Fisheries, Nagasaki University.

Observations are conducted as follows. Firstly, in CTD observation we measured water temperature, salinity, dissolved oxygen concentration (DO), chlorophyll-*a* and turbidity every one hour by using multi-parameter water quality meter (Rinko-Profiler, JFE Advantech Co. Ltd., Japan). Second, vertical distribution of flow velocity and flow direction measurement was obtained by ADCP (M9-ADCP, YSI/Nanotech, USA) during overall period of measurement. The ADCP was mounted on a small floating boat. Third, vertical distribution of suspended solid (SS) concentration and

grain size distribution of SS was measured by LISST-100X (LISST) (Sequoia Scientific, Inc., USA). The LISST can store grain size data from 2.5-500μm, divided into 32 ranges in log-scale. The devise was pulled down by humans from the sea surface to the sea bottom at a rate of approximately 0.2m/s. Fourth, vertical profile of turbulence micro-structure was carried out by TURBOMAP-9 (JFE Advantech Co. Ltd., Japan). This device also freely fell from the sea surface to the sea bottom by gravity. Dissipation rate of turbulent energy£ and vertical eddy diffusivity/viscosity ¬$_{tz}$ were estimated from the measurement result of shear of horizontal velocity d*u*/dz. In the present paper, we focus on only the relationship between the process of stratification development due to flood events and river discharge, and also the variation of turbidity, DO, and ¬$_{tz}$. In addition, water density calculated from salinity and water temperature.

RESULT AND DISCUSSION

The Freshwater Inflow during the Measurement

Fig.2 shows the temporal variation of total discharge of all A-class rivers from 13 June, 2014 to 12 July, 2014 including twice observations (3-4 July and 11-12 July). To take the influence of watershed area in downstream from the discharge observation station in each river into account, the river discharge was modified by a factor evaluated from a ratio between the total river basin area and that at the observation station of river discharge, which is located in the upper reach where tide cannot influence. A large scale flood event occurred on 3 July and the total peak discharge reached to approximately 4,500m^3/s. After some days, the discharge again reached to almost same level due to the typhoon. In both flood events, the discharge from the Chikugo River, which is the largest river in the Ariake Sea, accounted for around half of the total freshwater inflow. This is the typical freshwater inflow condition in the Ariake Sea.

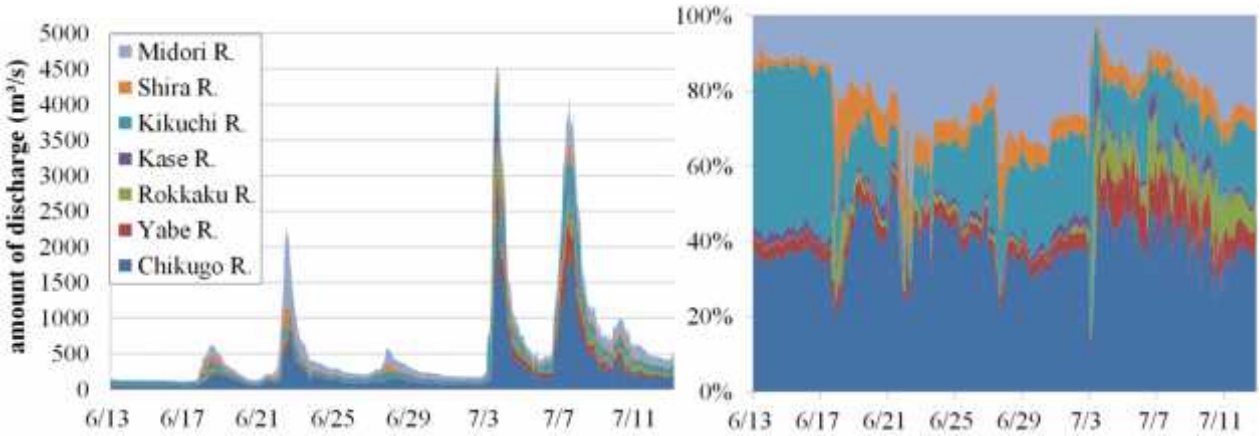

Fig.2 The temporal variation of total discharge of A-class rivers flowing into the Ariake Sea (left) and the ratio of A-class river discharge to total one (right) in the period from 13 June, 2014 to 12 July, 2014.

The Relationship between Density Stratification and Vertical Structure of Turbulence, Turbidity and Dissolved Oxygen Concentration

Fig.3 shows tempo-vertical variation of sea water density, vertical eddyviscosity ¬$_{tz}$, turbidity and DO for both of the first and the second observation. From density variations for the first measurement, it can be seen that density stratification was not formed at the beginning of the first measurement, because freshwater due to the flood on 3 July was not reached. This indicates that the density stratification was not maintained for 10 days by the flood events with total discharge of approx. 2,000m^3/s, which occurred before 10 days prior to the first observation (22 June). Meanwhile, the density declined around the surface from the middle to the end of the first observation period (after 6:00 on 4 July). The peak discharge from the Chikugo River was 2,000m^3/s at 14:00 on 3 July. On the assumption that the freshwater from the river was observed at the observation point, it takes approximately one day to travel from the river discharge observation point in the Chikugo River to the observation point in the Ariake Sea. Also, we can see the initial process of developing density stratification. On the other hand, from density variation for the second measurement, strong stratification had already been formed at the beginning and was maintained until the end of observation. The stratification was caused by flood on 7 July. However, density in the surface layer gradually increased with time. This indicates the mixing process due to strong tidal current in spring tide at the phase of the end of stratification. During low tide the density decreased while it increased during high tide. It is expected to get new information in connection with influence baroclinic structure caused by Isahaya Sea Dyke for generation of red tide and fate of anoxic water.

Fig.4 shows the temporal change of the potential energy¤ [J/m^3]to mix for complete vertical homogeneity in water column, which was proposed by Simpson *et al*. (1978) as an index of stratification, in two measurements.¤ can be calculated from the following equation:

$$\phi = \frac{1}{h}\int_{-h}^{0}(\rho - \bar{\rho})gz \tag{1}$$

where,ρ: water density, $\bar{\rho}$: depth averaged water density, *h*: water depth, *z*: vertical coordinate, and *g*: gravitational acceleration. We can see the development process of stratification in the first measurement clearly, and also sustainment of it in the second one.

In addition, Fig.3shows a relationship between stratification and eddy viscosity¬$_{tz}$. Vertical profile of¬$_{tz}$shows damping around pycnocline, especially in the second measurement which shows strong stratification. Also, we can see two different sources of suspended solids (SS) from the measurement results. Namely, resuspention from bottom sediments and transport with freshwater from rivers are distinguished. Finally, Fig.3shows a relationship between development of stratification and anoxic water dynamics. Low level of DO was seen in the bottom layer during developing stratification due to freshwater transport after 6:00 on 4 July from the first result. While, low DO layer was maintained in overall period of measurement from the second result. Thus, we can expect that these data may be used for validation of numerical model.

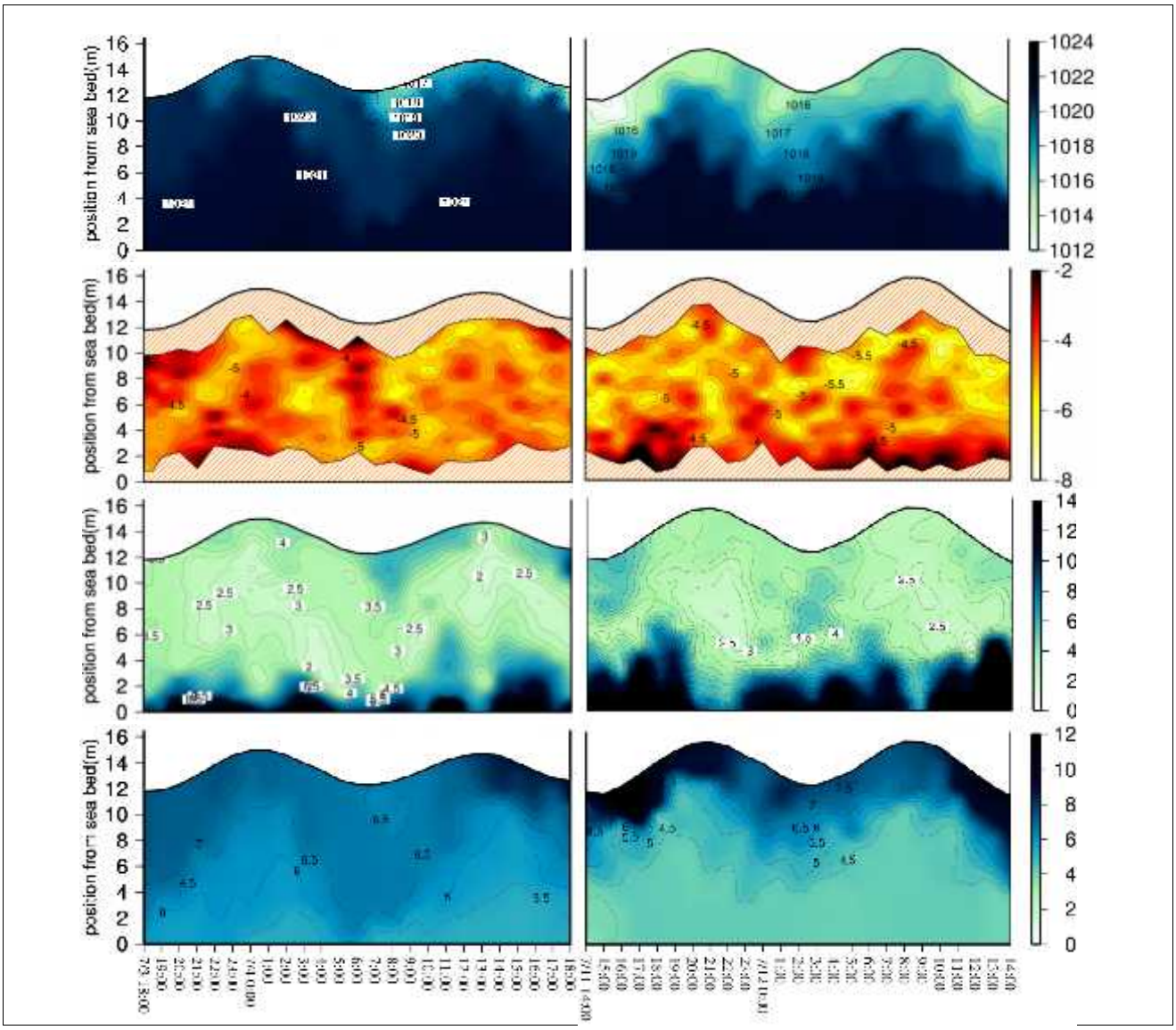

Fig.3Measurement results of isopleth in the 1st measurement (left) and the 2nd one (right).
From top to bottom the followings are described: Sea water density [kg/m^3],
vertical eddy viscosity [log-scale, m^2/s], turbidity [ppm] and DO [mg/L]

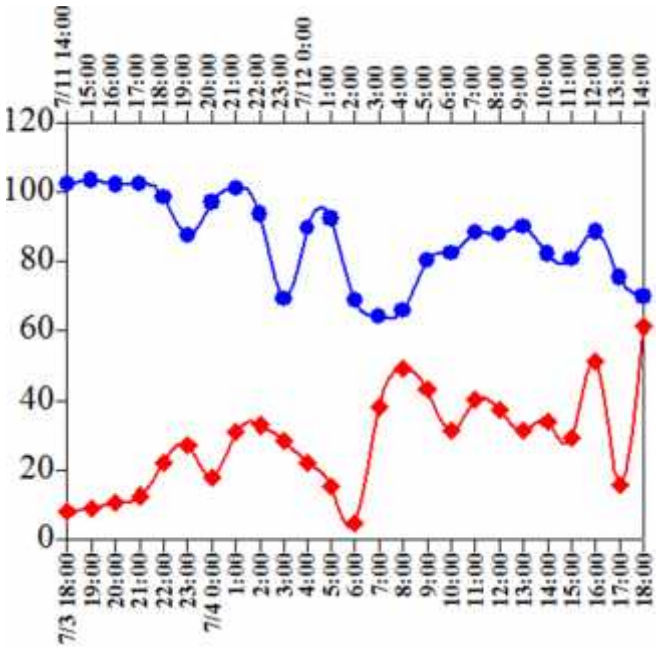

Fig.4Temporal variation of the index of stratification¤ [J/m^3].
[Red: the first measurement. Blue: the second one.]

CONCLUSIONS

Based on the results of twice *in-situ* measurements in the Ariake Sea, measured sub-tidal development and decay process of saline-freshwater stratification during rainy season shows the time scale of stratification depends on river discharge in a flood. From the measurement results we can see the followings:

1) Dumping process of vertical eddy viscosity due to stratification dynamics was measured;

2) Turbidity due to suspended solids transported by riverine freshwater and one due to resuspention of bottom sediment were distinguished in the measurement result; and

3) Reducing process of DO in the bottom layer through the stratification development in neap tide and disappearing process of anoxic water by well mixing in spring tide were clearly measured.

We also plan to investigate the relationship, not covered in this paper, between flow field and density stratification fate, as well as, blooming of phytoplankton and the fine sediment transport by freshwater originating from rivers. In addition, we have conducted similar surveys during a no-flood season (in winter, 2014), since it seems to exhibit characteristics different from a flood season. Furthermore, we have to evaluate quantitatively the impact of flood conditions on the stratification, namely, ROFIs in the Ariake Sea. We will also combine observation data and numerical simulation results to quantitatively evaluate the impact of Isahaya Sea Dyke on the baroclinic structure and suggest necessary measures towards regeneration of marine environment in the Ariake Sea.

ACKNOWLEDGEMENTS

The study was supported by JSPS KAKENHI Grant Number 24360200 and the research fund in FY2014 from Okumuragumi Public Trust. We also appreciate the help we received from students in Environmental Fluid Dynamics Laboratory in Kyushu University and Hydraulics Laboratory in Nagasaki University, and crews of the Kakuyo-maru, Nagasaki University.

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THE EFFECT OF LAND USE CHANGE ON FLOOD PEAK IN THE INTEGRATED AREA OF CAMPUS UII

Ruzardi [1]

ABSTRACT: Flood incident have direct correlation with land utility in the catchment area. Yogyakarta in their update thriving toward the north where this area including conservation area of rain. Similarly, in the integrated area of Islamic University of Indonesia since in 1992 occupy the area of Kaliurang Km 14,4 street with the area of about 35 Ha. This area is located between two tributaries, they are Pelang river and Kladuan river. The Islamic University of Indonesia is located next to upper course of Yogyakarta, and located at water catchment areas of rain which flowing into the city of Yogyakarta, then its needed to be researched the extent to the the impact of the transformation landuse on transformation to the top of the flood. The data required in this study was obtained through secondary data collection. Data obtained from the rainfall station PSDA Yogyakarta, land utilization data obtained from the office of BAPPEDA District of Sleman and PPTiK from the Gajah Mada University. Data analysis improvement started by revising on the rain data which missing, then count the scatter by using the methods Gumble and discharge flood analysis by using the rational method. Transformation landuse on the area of the Pelang river in 1996 to 2013 for the resident area was very significant, it reachs 75,1%, while in the cathment area of Kladuan river reach 26,4%. The road access has significant increased in the Pelang river. Then, for rice-fields have a significant decreased, it is 214 Ha. This transformastion directly affected against the peak of the flood. The results of research presented that the increasement the peak of the flood of about 12% for Kladuan river and 26% for pelang river.

Keywords: Rainfall, land use and flood peak

INTRODUCTION

The floods disaster which going on lately felt to be more often. The various causes of floods have been made research, one of the cause of flooding this can be by climate change with the increasing of rainfaall or tranformation of landuse. For example, the flood draft which formerly by the return period 25 years now in it can be in 10 years have already occured, it also accoured to the top of other return period.

In the planning flood hydraulic stucture the design used as the main in the determination of stucture dimensions. Design of flood using probability consept or probability of flood occured, where it is containing the understanding the value which obtained might be equal or even less. The concept of flood design not apply in the possibility of an tranformation of landuse which will give dirrect impacts to the flood rising.

There has never been any sufficient evidence that prove the peak of flood incidence that more short caused by increase of rainfall, but the rising of floods caused by the transformation of landuse. The rapid transformation of land use of a layer water pass impermeable strata which are often certainly directly influence at the top of flood, although these effects shown in behave which are different in several big cities. This occurence have been prove by researchers they are: Sorman (1977), Huff (1977), Laociga et al. (1996), and Ruzardi (2002). Boyd, M.J., Bufill, M.C. & Knee, R.M. (1994).

Islamic University of Indonesia started to be occupied in 1990. Active there are more than 18,000 people (Harsoyo, 2015). About 14,000 people occupying the campus building. There is no data about how many people live nearby. Physical evidence showed alteration of the area which changed rapidly. From the review of topography campus area it is located at enough higher elevation than the Yogyakarta which is about 320m above the sea surface. The transformation of land utilization in the campus area will give impact to the top of flood. Therefore, the purposed of study to apprised how much change the top of flooding caused by transformation of the area in Islamic University of Indonesia (see Figure 1). Enda O'connell et.all (2007) concluded that the prediction models and approach described rainfall-runoff modelling to predict lan use management impact on flooding is relatively undeveloped.

Yogyakarta flowing by three main rivers they are Code river, Gajahwong river, and Winongo river. Those rivers use as well natural drainage, besides, the soil conditions in Yogyakarta containing sand quite high to support infiltration and reduce runoff. Islamic University of Indonesia located in the area.

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Islamic University of Indonesia located over upstream Yogyakarta city, and also situated as a catchment area for the Yogyakarta city, then with the overview above, the case which will be observed are: What the transformation of land use in the area of Kladuan river and Pelang river nearby the Campus? And how many the increasement of the peak flood in this area? Based on the description above, this study is expected to provide answers about the land use change in flood peak.

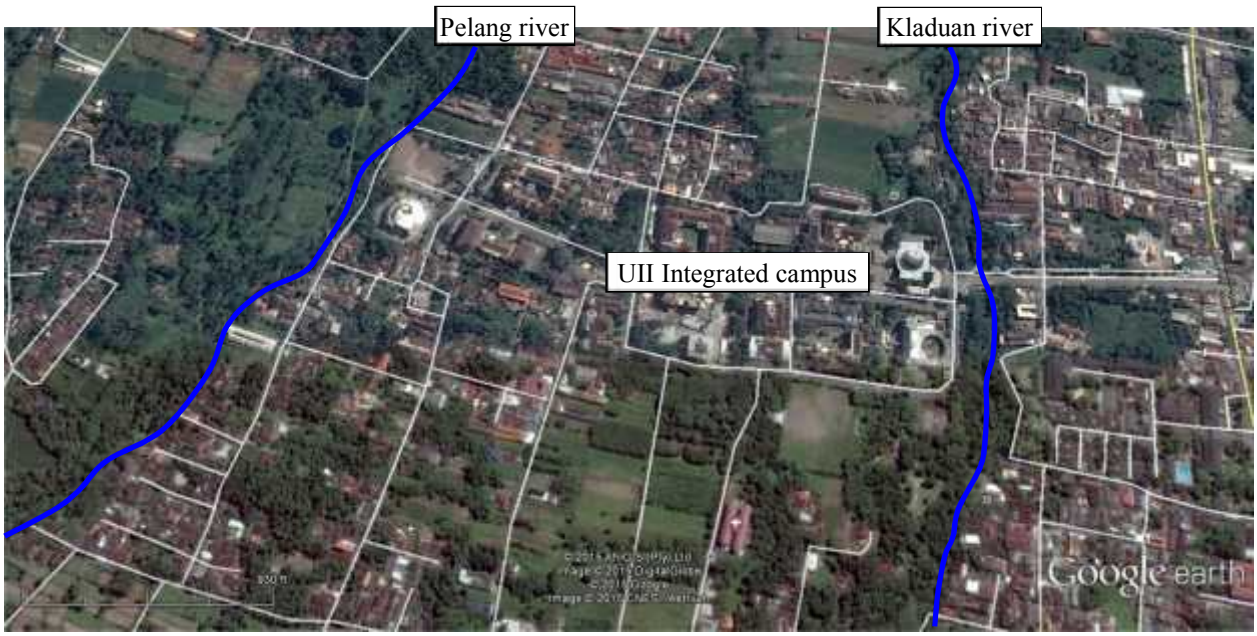

Fig. 1 Integrated area campus UII

METHODOLOGY

Rainfall data was obtained from PSDA Yogyakarta, there are four station in around the study areas. The rainfall intensity generated from Mononobe. Statistical testing with least squared method used for determined worthiness of the rain data.

Land use data collected from the villages and sub-districts in the basic map form. The data transformed in to digital maps data which using GIS software accordance to form river flow areas Klanduan and Pelang. The lan use data verificatied by topographical maps and aerial photographs from google earth.

Peak flood counted by using Rational mehod, by the formula:

$$Q = \quad .Cs.C.I.A \tag{1}$$

Q is the discharge of the peak flood of A is the catchment area, β the coefficient of the rain spread, Cs the coefficient of storage, C the runoff coeficient; I rain intensity. The case the peak flood calculated on the condition in 1996 and 2013. The difference in the peak flood is the increasement in the peak flood on that period.

Based on the field observation, it shows that the rain spread in the rain area, but the fact said that rain has a certain center with the depth value or maximum rain the area. The more widely the reviews area the more restrained the rain or in the other means the more far from the central point the rain intesity will be more lower. Empirical formula which has known to count the coefficient of the spread of rain (β) there it is:

$$A = \frac{1970}{-0{,}12} - 3960 + 1920 \tag{2}$$

There are possibility that the part of rainfall suspended in the depresson on the ground or on the hinderance, more further there shall a correction rational formula using it coefficient of storage (Cs), there it is:

$$C_S = \frac{2xt_c}{2xt_c + t_{cc}} \tag{3}$$

In the case: tc is time of concentration or the reference time is the length time of the water flows from the farthest point (inlet time) to the observation point (outlet time). Those time above consist of the length time of water flows on the ground (overland flow) which flows to the close channel plus by the length time of water flows in the channels to the obsevation point (outlet time). In simply form, it resulted the calculation of the length time of water flows (inlet flows) (tcs) plus the length time of water flow (outlet flow) (tcc).

In the empirical formula tc count by the California Division of Higways's formula:

$$t_c = \left[\frac{0{,}87L^3}{H}\right]^{0{,}385} \tag{4}$$

L: length of runoff water flows in the river (km), and H: the difference high of upstream elevation to the downstream (m).

The transformation of land utilization showed in the percentage of impermeable of strata which is identical equal to the runoff coefficiency (C). The runoff coefficient is the comparison between rain runoff on the surface to the precipitation falling. The run off coefficient directly connected with the layer which cover the land, such us: types of soil, topography, coarseness of land, the plants, and the landuse. The value of runoff coefficient could be count from the general value. The value C which consist of variouse land use could be counted by using formula:

$$c = \frac{\sum_{j=1}^{n} C_j A_j}{\sum_{j=1}^{n} A_j} \tag{5}$$

The method which will be used in determining the average rain in the area is by Polygons Thiessen. The measurement of rain that held at some stations totaled then multiplied by broad every a Polygon, then divided by the number of widely reviewed rerata produce rain in the area. The results it used to calculate the intensity of the rain with a method of Mononobe this as follows:

$$I_t = \frac{R_{24}}{2}\left(\frac{2}{t}\right)^{\frac{2}{3}} \tag{6}$$

I_t: intensity of rainfall for duration t (mm/hours), t: the duration of rainfall (hour), R_{24}: maximum rainfall (mm) in 24 hours. Data collection rain water derived from the BPSDA province of Yogyakarta. There are 4 rain stations which are located around river basin areas Kladuan and Pelang they are Kemput station, Prumpung station, Plataran station and Bronggang station. The rain do the processing of data to qualified to be worn in the analysis

The method of distribution Gumbel a maximum value type I (Extreem Value I/EV I) used to an analysis of the frequency of rain. The common equation EV I to count the frequency is analysis:

$$x_{Tr} = \bar{x} + KS_x \tag{7}$$

Where: X_{tr}: = the rain for the period of T_r, $\bar{x}$ = the maximum rainfall average observation in year, S_x = deviation standard, K = the frequency factors.

The maximum value gumbel according to Chow (1988) is a function of the distribution has a possibility that shaped the distribution of exponents double with two parameters. A form common namely equation:

$$F(x) = \exp\left[-\exp\left(-\frac{x-u}{\sim}\right)\right] \quad \text{untuk } -\infty \le x \le \infty \tag{8}$$

Parameter α and *u* using the equation:

$$\sim = \frac{\sqrt{6}}{®} s \tag{9}$$

$$u = \bar{x} - 0.5772\sim \tag{10}$$

RESULT AND DISCUSSION

Rainfall data that have been processed are presented in Table 1 below.

Table 1. Rainfall data used in the study area

Year	Maximum daily rainfall (mm)			
	1. Sta. Kemput	2. Sta. Prumpung	3. Sta. Plataran	4. Sta. Bronggang
2004	71	73	0	84
2005	161	29	112	162
2006	145	161	51	111
2007	10	44	1,2	201
2008	53	45	45	60
2009	0	40	10	53
2010	5	4	0	331
2011	2	136	127	0
2012	0	113	0	72
2013	11	90	86	36

The analysis land use shows that the eastern area of the campus (cathment area in Klanduan river), there are more residental areas, while in the western part of the campus area (sub cathment area in Pelang river) plantations and ricefields. This area reflect the eastern part and more describe the infested area by impermeable strata, while western region in the form of land more seeping the water, but if we reviewed overall the results showing that the western part more transform the eastern part. This result can be seen in Table 2 and Table 3, below.

The transformation of landuse in the sub cathment area of Pelang river for the resident area was very significant, it reachs 75,1%, while, in the sub cathment area of Kladuan river reach 26,4%. The results of this suggests that can be accepted because it is located on the edge of the eastern main road which have started to solid by settlement (can be seen a picture on the appendix). Along with the growth of the center of significant settlement road access and also with a sharp increase growth is approaching 100 percent. However the visual observation in the field this road more done by settlers own project.

Land use change is the most widely plantations and subsequently irrigated land. The result show that the irrigated land was reduced by 17.5 % to catchmant Kladuan river area and 26.0 % for cathment area Pelang river. In the period 1996 to 2013 for a total change / reduction reached 214 hectares of irrigated land, the reduction is of course also have a significant impact on the reduction of rice production. Assuming rice production of 6.5 tons / ha and can be harvested two times a year , then for 17 years this has been reduced rice production amounted to 2,782 tons . The production cuts have not taken into account the reduced harvest of the crops.

Table 2. Land use change catchmant area Kladuan river

Land use	Area, year 1996 (km²)	Area, year 2013 (km²)	Transformation %
Settlement	3,9328 (36,7%)	4,9730 (46,4%)	26,4
Irrigation rice fields	5,9556 (55,6%)	4,9122 (45,8%)	-17,5
Plantations	0,5612 (5,2%)	0,3479 (3,2%)	-38,0
Bare land	0,0111 (0,1%)	0,0250 (0,2%)	127,3
Road	0,2593 (2,4%)	0,4619 (4,3%)	78,1
Total	10,72	10,72	

Table 3. Land use change cathment area Pelang river

Land use	Area, year 1996 (km²)	Area, year 2013 (km²)	Transformation %
Settlement	1,440 (25,5%)	2,521 (44,7%)	75,1
Irrigation rice fields	3,788 (67,1%)	2,689 (47,6%)	-29,0
Plantations	0,214 (3,8%)	0,126 (2,2%)	-41,1
Bare land	0,095 (1,7%)	0,095 (1,7%)	0
Road	0,107 (1.9%)	0,213 (3,8%)	99,0
Total	5.64	5,64	

The runoff coefficient (C) is a dimensionlees coefficient relating the amount of runoff to the amount of the precipitation received (McCuen, R.H. 1998). It is a larger value for areas with low infilration and high runoff (pavement and steep gradient), and lower for permeable areas well vegetated areas (forest, flat land). It is important for flood control channel contruction and for poosible floood zone hazard delineation. A high runoff coefficient may indicate flash flooding areas during storms as ater moves fast overland on its way to a river channel or a valey floor. In the study for period 1996 to 2013 for a total area of about 16,13 sq km, found that the changes of runoff coefficient of 0,35 became 0,42. This case shows the value changes by about 7% for 17 years for developed regions, with an average annual change of about 0.41%. This condition is still not considered by the construction of recharge wells at some of the housing , which is believed to actual conditions on the ground will be smaller than the value obtained this

Results of the flood peak indicates that there has been a rise for the entire return period of flood (see Table 4 for Kladuan River and Figure 2 for Pelang river). The results have been like this can be ascertained formerly is a result of the change of land use that tends to rise from the review of the water impermeable layer. The mean increase in peak floods in the period 1996 to 2013 are 12 % for Kladuan catchment area and 26 % for Pelang catchment area. Nevertheless compared with the research results to other areas the result for sub catchment Code river is not dangerous.

Table 4. The Parameter Rasional and Peak flood in Kladuan cathment area in 1996/2013

Return period (T)	Coefficient of rainfall spread (β)	Coefficient of storage (Cs)	Coefficient *of run off* (1996/2003) (C)	Intensity of rainfall (mm/jam) (I)	Catchment area (Km^2) (A)	Discharge (1996/2013) (m^3/detik) (Q)
2	0,9764	0,6878	0,379/0,426	12,102	10,720	6,500/7,291
5	0,9764	0,6878	0,379/0,426	19,730	10,720	10,596/11,886
10	0,9764	0,6878	0,379/0,426	24,926	10,720	13,387/15,017
25	0,9764	0,6878	0,379/0,426	31,492	10,720	16,913/18,972
50	0,9764	0,6878	0,379/0,426	36,319	10,720	19,506/21,881
100	0,9764	0,6878	0,379/0,426	41,042	10,720	22,042/24,726

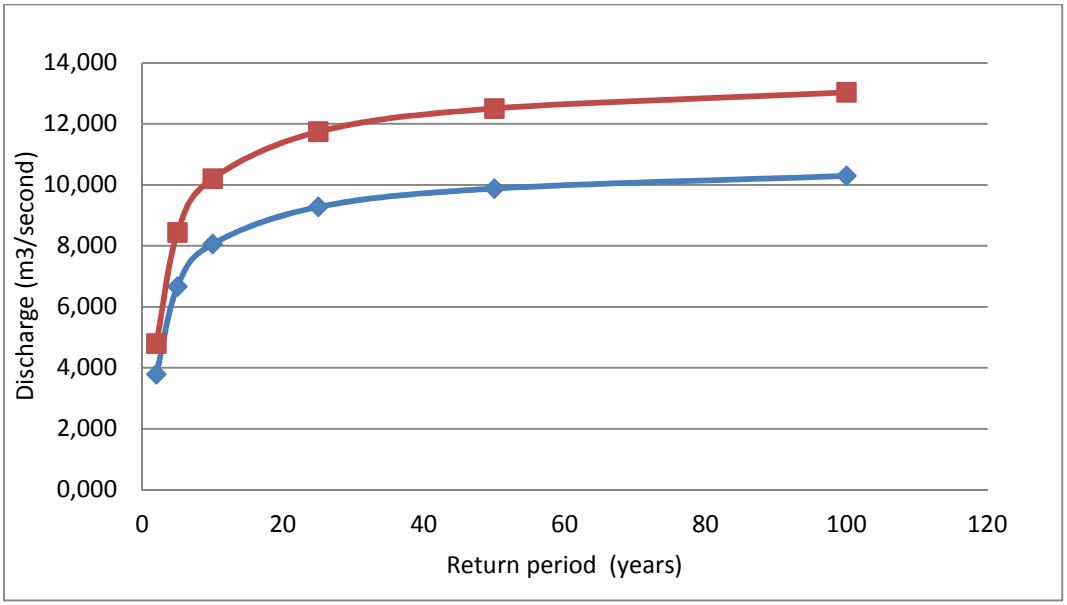

Fig. 2 The correlation of flood discharge with return period on Pelang river

CONCLUSION AND RECOMMENDATION

The area Islamic University of Indonesia began quite speed in the 1996 until now is still on going, and the affects of the development, caused the transformation of the land utilization in the cacthment area of Pelang, and change the area where it could absorb much water no being less than before. Based on the data result and the discussion which have been explained on the chapters before, it could be concluded that, there is correlation between land use change and flood peak. The transformation of the land use in the cathment area in the Pelang river quite fast, it could be seen Table 2 and Figure 2.

In this study does not include the effect of absorbing wells in the field, so that it should be smaller obtained from the calculation. For the future study it would need to take into account these conditions.

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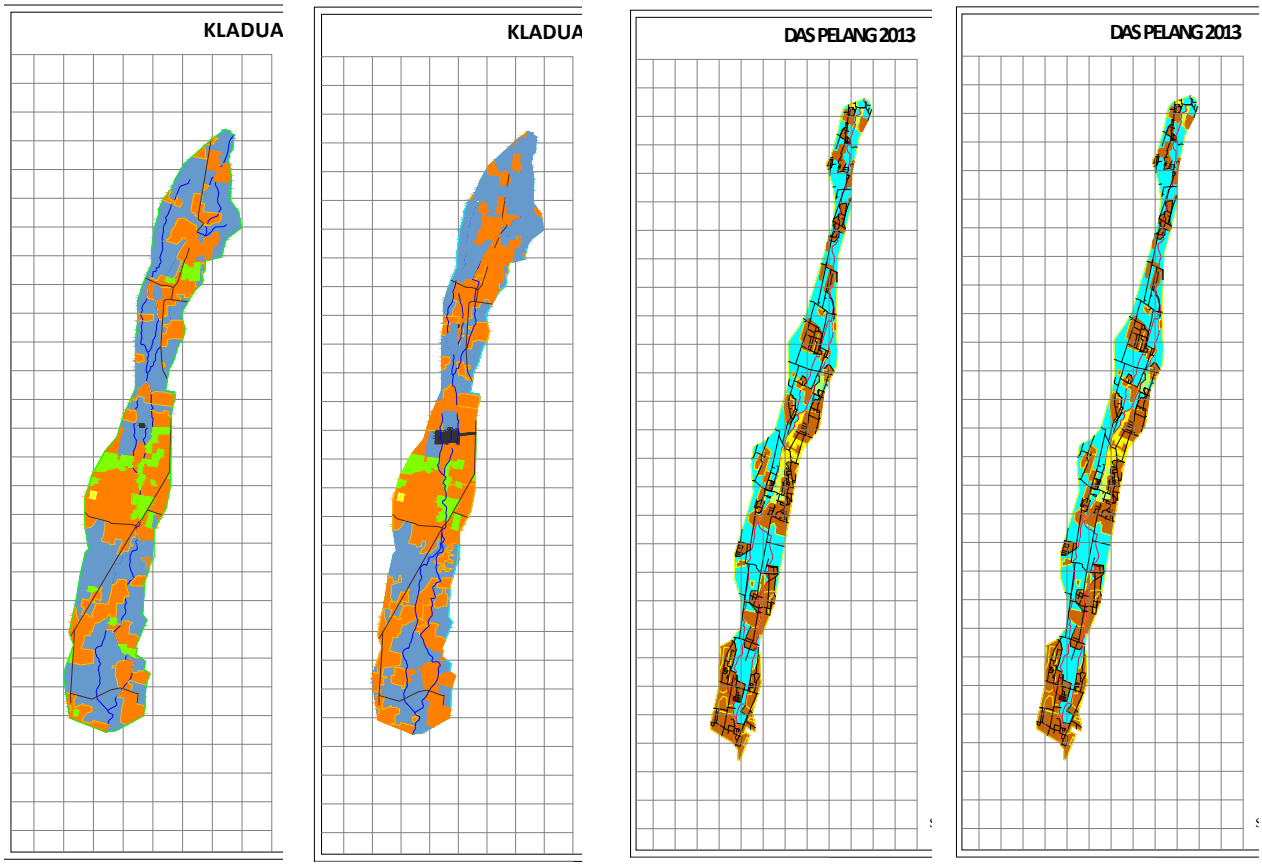

Fig. The profile land use Kladuan and Pelang catchment area

DESIGN STRATEGY APPROACH FOR FLOODING AREA AS A MITIGATION APPROACH, CASE STUDY: GORONTALO CITY

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ABSTRACT: Gorontalo City is the capital of Gorontalo Province, which located in Sulawesi Island. The city is located on the lowland between a lake and a sea. Its geographic location make Gorontalo City is disaster-prone, especially landslide and flood that strike almost every year. Those disasters occurred mainly because of the city planning and urban design approaches are not responsive to the geographic condition of Gorontalo City. Those disasters also overshadow the charm and potentials of Gorontalo City. To prevent recurrent flood and landslide, Gorontalo City should be planned and designed with green design approach based on its physical characteristic. This paper will elaborate some issues related to identify and analyzing problems and potentials of Gorontalo City, strategies and development models to solve the problems of Gorontalo City and to develop Gorontalo City as sustainable and resilient city.

Keywords: *Gorontalo City, Flooding Area, Mitigation Approach, Design Strategy.*

PREFACE

Background

A city should be planned and built with sustainability principles, which balancing the aspects of environment, social, and economy. Among these three, environmental aspect of a city is the major factor that should adapt the existing social and economy condition. But the environmental aspect should also be supported by the projected social and economy condition. The environmental aspect in planning should be structured and outlined in the city planning vision and applied through city planning principles and design strategies. These three aspects should be considered equal to gain a balanced development of a city. For example, if the city planning only focus on the economy aspect, there will be an imbalance development, that lead the city into unsustainable condition.

A sustainable city development and planning through balancing the environmental, social, and economical aspects should adapt and adjust the physical aspect of the city. Actually, this approach is strongly related to – and somehow is a part of – mitigation planning of a city. Balancing these three aspects are aiming to prevent the negative impact of unsustainable development. Punter (1991) gave an overview how sustainable planning synergy in shape, image, and activities will provide a sustainable place (space) of the city. Otherwise if the city is designed unsustainable, the city will be unsustainable.

Gorontalo City now is facing the result of unsustainable city planning and development. The city has a high-complexity problem in adapting rapid urban development to its physical and natural feature, which is vulnerable to disasters, especially flood and landslide.

Fig. 1: Floods and landslides in Gorontalo City

The topography of Gorontalo City, which is situated in valley, passed many rivers, and has a lake; and uncertain weathers are the primary causes of floods that often hit the city. The uncontrolled land conversion of irrigated agriculture are into urban area is worsening the impact of the floods. The issues mentioned above are correlated with the city planning and regulation regarding the city adaptation for increasing activities. In short, the city planning and development cannot maintain the citizen safety and comfort when the activities increasing. Whereas, Gorontalo City is

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the Capital of Gorontalo Province – it is should be a productive, sustainable, safe, and comfortable city and be example for other cities in Gorontalo Province.

Environment's physical features, such as rivers, lakes, and topography actually can be a great potential and give positive value for urban development when the urban planning consider the sustainable aspects mentioned before. Those features, mainly river and lake, can be a tourist destination, source of water, transportation, source of food, source of energy, irrigation, etc. Like many other Indonesian cities, the development of Gorontalo City seems partially see those potentials. It can be assumed that the economic and social aspect in urban planning and development is overpowering the environmental aspect, which resulted the negative condition in urban environment.

The complexity that Gorontalo City is facing has forced its government to make efforts through structural and non-structural action. Some alternative actions are revitalization of flood-prone areas, river and lake area restoration and improvement. There are also more extensive action, such as spreading public activities center in order to reduce the density concentrated in few urban areas so the impact of coming disaster can be minimalized.

Gorontalo City urban development complexity with a variety of activities in few urban areas requires a structured, synoptic spatial planning that can described in the vision and urban planning of Gorontalo City. This plan should also be implemented through urban development principles and strategies. While Gorontalo City continues to grow – demographically, economically, politically, socially, and culturally, today spatial planning in fact has not been able to create a sustainable city development.

The new spatial planning should direct Gorontalo City development from monocentric approach to polycentric approach supported by inter-urban-area synergy. This planning is expected to accommodate the city's growth and make it adaptable to its physical situation. The core action of the planning is creating more city centers that can be a planning-and-development model for other areas in Gorontalo City. In order to be feasibly executed, the planning should consider the cultural-social factor, economical factor, and ecological factor of Gorontalo City and its surrounding regencies/cities. The ecological factor, especially availability of land, is the important factor in this spatial planning.

Issues in Gorontalo City

Some potential problems regarding to the city of Gorontalo rightnow are:

1. Synergy and structure of urban areas that have not been structured and integrated regarding to mitigation and adaptation to the environment. This problem is including the undefined city order that led to unordered synergy with surrounding regencies/cities.
2. Existing activity centers is still not optimized. It affects to social and economic value in Gorontalo City citizen. There are some areas in the city that built and developed without considering the environment's carrying capacity. For example, some conserved agricultural area that supposed to be a catchment area is transformed into residential area.
3. Spatial planning integrity between Gorontalo City and its surrounding regencies/cities is still not synchronized. It is also partially responding the problems of Gorontalo City, have less strategic plan to increase the efficiency and effectivity to be a more productive and sustainable city.
4. The current development has affected directly the social condition of Gorontalo City citizen, especially who live in poorly developed areas in the city, like Limboto Lake and Botu Area. It shows that not only environment quality has been degraded by current development, but also social quality. For example, this degradation will be a social disaster, if current development is not changed.
5. Recurrent floods that stroke Gorontalo City is affecting the image of this city, disturbing citizen activities, and indirectly lessening the pride of its citizen.

THEORY AND CASE STUDY

Some theories regarding spatial and urban planning is considered to support the analysis for spatial planning of Gorontalo City and its surrounding area. Case study is also studied, particularly city which has the similar problem with Gorontalo City (esp. flood mitigation) and applied the approach of studied theory in its city planning.

City Planning

A city should be planned integrally with a balanced consideration among social, economic, and environmental aspects. Every aspect is affecting each other and should related positively to obtain a sustainable development. The city planning also should considering the elements of city. Shirvani (1985) specifically describe eight elements of city planning that should be considered in the process: 1) land use, 2) building for and massing, 3) infrastructure, 4) pedestrian walk, 5) supporting facility, 6) signage, 7) preservation, including environment preservation. These eight elements, if grouped hierarchically through sustainable development approach, will be three aspects: environment, social, and economy.

Planning on Waterfront Area (Sea, Lake, and River)

According to Breen & Rigby (1994), a city that has a water potential physical features, such as river, lake, sea, etc., which geographically forms a transition area between a land and water area called as waterfront area. This area should be designed with specific strategy and considering its environment, social, and economic aspects.

Similar Case Study

- Canal development in Vietnam as flood mitigation.
  Many cities in Vietnam flooded in rainy season while some places have already have canals. The causes are uncontrollable density in residential area, poor networks of rivers, and not well maintained canals. The solutions implemented are increasing the number of canals, rejuvenating existing canals, and land use reformation. Building that violate the land use regulation, mostly slums, in riverside area is destructed and the activities are moved into new well-designed area. The riverside area is transformed into green space that functioned as basin riparian area.

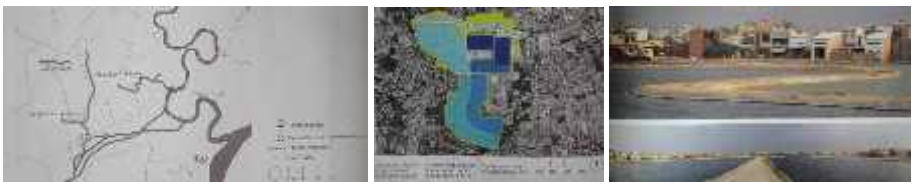

Fig.2: Spatial Planning in Vietnam

- City pond development in Taoyuan Metropolitan Area, Taiwan as water catchment area to solve the flood and shortage of water.
  Taoyuan is a city located in valley and river basin area. In rainy season, flood strikes regularly because the river is overflowing. While in dry season, the area will be dry that resulted a poor agriculture productivity. The solution implemented is building the ponds as water catchment area. The ponds are located in several spots in the city and connected by rivers and canals.

Fig. 3: Ponds in Taouyuan, located in urban area and agricultural area as a mitigation approach

IDENTIFICATION AND ANALYSIS OF GORONTALO CITY AND ITS SURROUNDING AREA

Location and Administration Border

Gorontalo City is a Capital of Gorontalo province. Its northern neighbor is Kecamatan (District) Tapa of Kabupaten (Regency) Bone Bolango; eastern neighbor is Kecamatan Kabila and Kecamatan Tilongkabila of Kabupaten Bone Bolango; western neighbor is Kecamatan Telaga, Kecamatan Batudaa, and Kecamatan Batudaa Pantai of Kabupaten Gorontalo; and its southern area is contiguous with Tomini Bay.

Fig. 4: Administrative map of Gorontalo City

Gorontalo City is the most populated region in Gorontalo Province, where its population number is 40% of the Gorontalo Province population. The residential and business centers development in this city is increasing rapidly resulting the higher density of Gorontalo City.

Fig. 5: Images of Gorontalo City

Topography

Topography of Gorontalo City is consisted of mainly lowland and basin (oval-bowl alike) surrounded by mountains and hills. This topography resulting the catchment area of 500.000 ha (50 km x 100 km) that able to accelerate and decelerate the climate activity. On the other hand, the river streamflow in lowland area is slower than streamflow in slopes. In southern part of this city, there are some hills and mountains. While the area that adjacent to Tomini Bay is mainly a flat area. The 0-8% flat area in northern area is the location of Kecamatan Kota Utara, the city's main activity district (urban area)

Fig. 6: Topography of Gorontalo City and its surrounding.

Hydrogeology

Gorontalo City has three large rivers (Bone River, Bolango River, and Tamalate River), a lake (Tomini Lake), and coast areas (next to Tomini Bay) that connected each other. The topography of Gorontalo City that is basin makes the city as potential area to be a water pool area.

Spatial Structure

Regional Spatial Planning of Gorontalo City stated that Gorontalo City is consisted of City Service Center as primary civic activity center, City Service Subcenter and Neighborhood Center as secondary civic activity center.

Fig. 7: Spatial structure plan of Gorontalo City

Gorontalo City spatial structure according to the Regional Spatial Planning is consisted of elements that classified into its structure (primary and secondary), function (economy, education, industry), scale (city, regional, national, international), for (office, market, open space, residential), and location.

Disaster-prone Areas

1. Flood-prone Area

   Flood-prone area is identified as area that often or highly potential to be stroke by flood. It covers estuaries, floodplain, alluvial mainly in riverside. The main causes of flood are high rainfall, uncontrolled land coverage in upstream area, lesser streamflow caused by sedimentation, and the topography.

2. Landslide-prone Area

Landslide-prone areas in Gorontalo City can be seen in the map below (based on analysis in Urban Masterplan of Gorontalo City).

Fig. 8: Disaster-prone area of Gorontalo City

DEVELOPMENT ANALYSIS IN GORONTALO CITY AND ITS SURROUNDING RELATED TO FLOOD ISSUE

Analysis of Planning and Existing Development

The two hills that flank the city give an image of 'welcoming gate' for someone who encounter Gorontalo City from Tomini Bay. While the water physical features – passed by three large rivers, and in between bay (Tomini Bay) and a lake (Limboto Lake) – give Gorontalo City 'a water city' image and characteristic. These features also should be a main consideration of Gorontalo City planning and development – perceptually and environmentally. In addition, managing the impacts of rainfall is closely related with these features too. The image below shows how rainfall and Gorontalo City topography and hydrogeology affect the condition of the city.

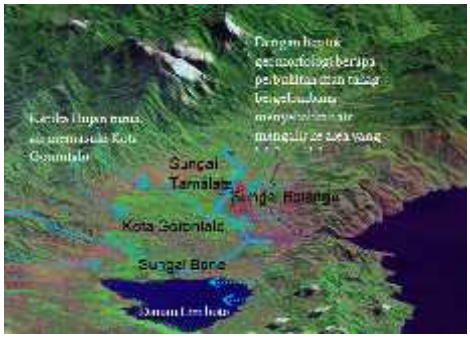

Fig. 9: Topography and Hydrogeology of Gorontalo City

Gorontalo City location in basin area forces a particular planning that ought to consider its unique environmental features: 1) Limboto Lake, 2) Bone River, 3) Bolango River, 4) riparian basin, and 5) slopes. If one of these features is not considered well in the planning process, it is not possible that disaster will strike often in the future. For example, 1) if Limboto Lake's shallowing is still going on – less than 15 m – water from this lake will be overflowing and affecting nearby land; 2) if the Bone River and Bolango River get narrower each year, the river stream will flood its neighbor area, especially in the confluence area; 3) if riparian basin is converted into other land use, the water that originating from hills and slopes will be directly flooding the city. The image below shows the condition – that has already occurred in Gorontalo City – where Limboto Lake contain its water and wet-rice agriculture area (as riparian) is converted into residential area.

Fig. 10: Climate is a significant factor for Gorontalo City

Nowadays, hills and slopes is used as residential and plantation area. Often, this condition is resulting a landslide when rain falls or earthquake strikes, because the ground is not absorbing the flowing water sufficiently. Land use in this area should be reformed into water catchment area (e.g. forest) and have soil reinforcement in several slopes.

Analysis of Spatial Data Development

Essentially, the existing spatial planning of Gorontalo City has considering its environment condition, but not the growing population caused by urbanization and increasing birthrate. This growing population needs more residential and commercial areas. The most demanded land uses in Gorontalo City are residential, offices, and commercials. This demand should not violate the existing spatial planning because the carrying capacity is not sufficient.

Fig. 11: Spatial planning of Gorontalo City 2010-2030

Unfortunately, the reality is different. In Kecamatan Dumbo Raya and Kecamatan Hulonthalangi, estuaries as well as confluence areas, some riverside is used as high density housing (see images below)

Fig. 12: Uncontrolled construction in riverside area.

In Bolango River, shallowing and narrowing is going on. This phenomenon lead to decreasing flowing and carrying capacity of the river. The cause is mainly irresponsible sand-mining, where the miners do not recover mine area and they live in riverside area (convert the green spaces into residential area).

Fig. 13: Uncontrolled sand mining in Bolango River in Botu

Some Limboto lakeside area of Kecamatan Kota Barat is used as residential area – even some buildings violating lake protection line. Some tidal parts of Limboto Lake is also used as agriculture area, which lead to shallowing of the lake.

Fig. 14: Uncontrolled housing development in Limboto Lake

Analysis of Agriculture Area

Agriculture in Gorontalo City cannot relied on extensification approach due to limit its potential area. Whereas some agriculture areas are converted into residential area, infrastructure, and city facilities. In Kecamatan Kota Utara and Kecamatan Dungingi the new road construction greatly increases the number of housings and commercial building. Tuloli (2013) found that land use conversion – from paddy fields to be built area – in 2006-2010 was massive, nearly 100 ha, especially in Kecamatan Kota Utara and Kecamatan Kota Timur. This conversion led to decreasing number of water catchment areas, recurring floods in Gorontalo City, and decreasing number of agricultural crops.

Analysis of Topography

Gorontalo City topography is mainly basin, consisted of three features: 1) hills, 2) slopes, and 3) plain. This kind of land surface is landslide-prone and the rivers in this city is easily covered by the landslide. Based on the topography, this city is divided into five areas: 1) plain area – slopes percentage 0-8% – located in every *kecamatan*, mostly in Kecamatan Kota Utara; 2) moderately sloping area – slope percentage 8-15%; 3) strongly sloping area – slope percentage 15-25%; 4) steep area – slope percentage 25-40%; and 5) very steep area – slope percentage >40%, located in Kecamatan Dumbo Raya, Kecamatan Hulonthalangi, and Kecamatan Kota Barat. Satellite images below show the topography of Gorontalo City and its surrounding.

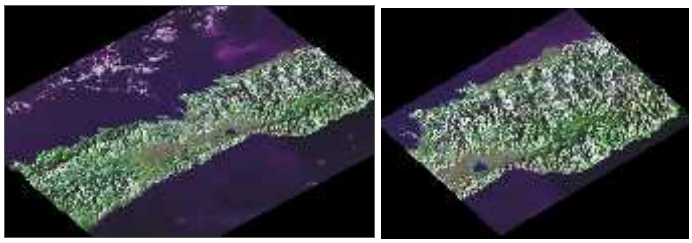

Fig. 15: Satellite images of Kota Gorontalo

PLANNING AND DEVELOPMENT STRATEGY

The basic principle used in the proposed planning and development strategy is sustainable development principle that balancing the economic, social, and environmental aspects and making environmental aspect as a core aspect in the process.

Problems in Gorontalo City today is making the city into unsustainable city. The strategy consider the problems as a result of *mismanaged* potentials (e.g. geographical feature) and use undeveloped potentials (citizen, cultural/social values) as generator for rejuvenating the city to be a sustainable city. The strategy will also aim to establish Gorontalo City as a disaster-responsive city.

The strategy is divided into four core actions: 1) land use and border planning, 2) bodies of water conservation planning, 3) infrastructure and transportation system planning, and 4) mitigation planning.

Land Use and Border Planning

1. Gorontalo City spatial planning will be focus on water stream area: Limboto Lake, Bone River, Bolango River, Tamalate River, and paddy field conserved area.
2. Strengthening the spatial identity and administrative borderline of Gorontalo City through green belt system, based on topography and disaster aspect. There are three areas that planned to be a *green border:* 1) Limboto Lake area, administrative borderline with Kabupaten Gorontalo; 2) Bone River area in Kecamatan Dungingi, administrative borderline with Kabupaten Gorontalo; 3) Bolango River area in Kecamatan Kota Timur and Kecamatan Dumbo Raya, administrative borderline with Kabupaten Bone Bolango.
3. Riverfront development through land use converting into office/commercial areas, green spaces, and public spaces as core action to regulate the river area. Increasing the water catchment capacity of Limboto Lake area through an integrated sustainable planning: mixed use development that also functioned as *environmental controller*.
4. Integrating Limboto Lake planning – as main water catchment area for Gorontalo City – to Gorontalo City spatial planning.
5. Utilizing paddy fields increasing the number of riparian areas as water catchment areas through conservation and regulation.

Bodies of Water Conservation Planning

1. River normalization and conservation by connecting Limboto Lake, Bone River, ponds, swamps, and coastal area and converting those areas into a green corridor.
2. Increasing the number of ponds as water catchment area in the overflown water area or confluence, especially in conserved paddy field.
3. Utilizing the bordering waterfront areas as linkages between Gorontalo City and its surrounding regencies.

4. Integrating the water management system through constructing dams and canals as flood prevention facilities.

Infrastructure and Transportation System Planning

1. Considering the land use planning to evaluate the class of existing roads and to develop new transportation network. Number of roads in conserved paddy field should be minimized to prevent further land use conversion.
2. Utilizing rivers, lake, water catchment areas, and sea as transportation network. This transportation network planning should also consider the land use of adjacent area to support Gorontalo's City tourism and cultural activities.
3. Developing water management infrastructure in Limboto Lake area to maintain water quality in this area – regarding its land use as mixed use area.
4. Creating green hub in green belt border area that functioned as interchange transportation hub. This hub will be transportation management controller and a welcoming area that enhancing the image of Gorontalo City as green city. Like most of green belt designed, this hub also designed as waterfront area.

Mitigation Planning

1. Mitigation-based planning in flood-prone areas, especially 1) Limboto Lake area; 2) Bone River area; 3) Bolango River area; 4) Tamalate River area; 5) conserved paddy field.
2. Mitigation-based planning in landslide-prone areas, especially hills and slopes in Kecamatan Kota Barat, Kota Hulonthalangi, and several other *kecamatans*, by reducing the number of residential and commercial (esp. office) area and increasing the number of green spaces in these areas.
3. Mitigation-based planning in flood-prone and landslide-prone areas, especially areas in Kecamatan Kota Barat and Kecamatan Kota Timur, by converting these areas into open spaces or conserved forests. Some slopes in these areas need soil reinforcement through retaining wall in order to prevent landslides heading to the river.
4. Mitigation-based planning regarding the earthquake, tsunami, or tidal flood through building heights regulation and coastal zone demarcation. This demarcation used to prevent the residential area construction in coastal zone, so the impact of tsunami or tidal flood will be minimalized.

ZONING DEVELOPMENT MODEL

There are five zones in Gorontalo City that will be development models based on analysis and planning strategy explained before. Those five zones are 1) Limboto Lake Conservation and Development Zone, including some areas in Kecamatan Kota Barat and Kecamatan Dungingi that have mutual influences with Limboto Lake Area; 2) Gorontalo City Center in Kelurahan Botu, a borderland with Kabupaten Bone Bolango; 3) River Stream Bottleneck Zone, consisted of some areas in Kecamatan Hulonthalangi, Kecamatan Dumbo Raya, Kecamatan Kota Selatan, and Kecamatan Kota Timur; 4) Borderland Riverfront Zone, 5) Conserved Paddy Field Zone, consisted of some areas in Kecamatan Kota Utara, Kecamatan Kota Tengah, and Kecamatan Kota Timur. These five zone are planned and designed to make resilient as well as grand vision of Gorontalo City is realized.

Fig. 16: 5 Zoning development model map

Limboto Lake Conservation and Development Zone

Limboto Lake is planned and designed according to land use management principle, water management principle, and infrastructure management principle. This zone is planned to be a mixed use development area in order to distribute populations, business activities, and development of Gorontalo City and to reduce the density in overcapacity *kecamatan*s, such as Kecamatan Kota Tengah. The arch-like waterfront design is based on theory of shoreline development. In some tidal areas, the waterfront surface will absorb the tidal water so the waterfront area still can be used.

In the middle of the lake, there will be a gigantic alphabet art installation that glow in the nights. This iconic installation will be a new landmark for Gorontalo City that can be seen from up in the air (airplane or drone).

Fig. 17: Limboto Lake Conservation and Development Zone

Gorontalo City Center in Kelurahan Botu

As a flood-prone area caused by narrowing of Bone River, this zone will be rejuvenated integrally with water management system. This zone will be center for Gorontalo City civic activities. It is planned to contain green spaces, public open spaces, municipal masjid, and new residential projects. The development will not only cover the riverside areas, but also in central of Botu and its neighbors. Nonetheless, Bone River is still the main focus of development by designing it interestingly and unformally. The design of riverfront area is aiming to cultivate 'interaction' between citizen of Gorontalo City and the river. This 'interaction' is expected to nurture the citizen responsibility over the river and make it an integral part of their life.

Fig. 18: Gorontalo City Center in Kelurahan Botu

River Stream Bottleneck Zone

This zone is important part of water management system because it is a confluence as well as estuary of River Bone and River Bolango. In this zone, there will be waterponds to contain overflowing water and to process this water so citizen of Gorontalo City is able to use it.

Fig. 19: River Stream Bottleneck Zone

Borderland Riverfront Zone

This zone is a borderland of Gorontalo City with Kabupaten Gorontalo. Some of its area have been used as municipal offices while there is a lack of public spaces here. Increasing the number of green spaces and waterfront areas that combined with existing municipal offices is the core development strategy in this zone. Some ponds also constructed to contain overflowing water from Bone River.

Conserved Paddy Field Zone

This zone is a paddy field area that managed to have an additional as water catchment area: some defunct fields are refunctioned and new waterponds are constructed. New waterponds will be a water source for nearby paddy field in dry season. This zone also expected to be main grain-producing area of Gorontalo City.

Fig. 20: Conserved Paddy Field Zone

CONCLUSION

Gorontalo City as a Capital of Gorontalo Province has a great opportunity to be a metropolitan city through managing its potentials and detailed and holistic planning process. Some disasters that strike Gorontalo City – especially floods – are happened because of unresponsive spatial planning to its environment condition (topograpy and hydrology) and rapid urbanization. Located in basin area between Limboto Lake and Tomini Bay and passed by some River, Gorontalo City is a confluence as well as an estuary. To prevent recurrent floods and landslides, there are four strategies to be implemented: 1) land use and border planning, 2) bodies of water conservation planning, 3) infrastructure and transportation system planning, and 4) mitigation planning. The cooperation between government and citizen/private sector is necessary to realize strategies above through zoning development models. This synoptic and integrated planning is needed to build Gorontalo City as a sustainable and resilient city.

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MODEL OF WATER PUMP POWERED BY WINDMILL DESIGN FOR IRRIGATION PURPOSED

Benny Syahputra[1],

ABSTRACT: Setro's Reservoir in Gedangan village of Rembang district Central Java Province has an area of ± 10 ha with a water depth of ± 6 meters. The volume of water of the reservoir is abundant even in the dry season, but the elevation of water is lower than the elevation of the rice filed. Therefore, water pump is requires to be able to irrigate the rice field. Using engine water pump to raise the water will take much money, and the cost requires for planting and maintaining of the rice plant will very high. According to data released by Balai Lingkungan Penelitian Pertanian or the Institute for Environmental and Agricultural Research (IEAR), 2010, the annual wind speed in the rice field area of Gedangan village mnimum is 2.3 meter per second (m/s), and maximum speed is 9.2 m/s. Those wind speed is possible for windmill to power the water pump. The aim of this study is to design a water pump power by wind to to the purpose of rice field irrigation based on the appropriate technology. The method used is the combination between design and experiment work. Experiment work aims to find the relation between independent variables such as wind speed and the speed of windmill with various diameter of the blade, and dependent variables namely the volume of water which can be resulted by the water pump. The study resulted the size of windmill's blade as follows: the minimum area of windmill's blade, $Le_{min} = 1/7 \times \pi \times r^2$; and the minimum length of tail's blade of windmill, $Pt_{min} = Ds - Pa_{min} = (2 \times r) - Pa_{min}$; as well as the length of blade's tail, $Pe = 2 \times r = Ds$. Those size of the windmill's blade can make the water pump possible to pump the water as much as 0.07 liter per second, and the pump can be operated on minimum windmill speed 2.5 m/s.

Keywords: *Reservoir, water pump, windmill, irrigate, rice-field*

INTRODUCTION

Farming is an inseparable activity with irrigation. Water is a vital necessity not only for human life but also for the growth of vegetation until can be harvested. In the reany season vegetation can be easy care since much of water to irrigate. In the dry season, the condition become change, water will become a rare and valuable comodity. That situation take place in Gedangan village of Rembang district. Farmers are not face with problem of water when start planting since they start in the reany season, but when in the second season of planting, they face with problem of shortage of water. Volume of water is decrease in the midle of second season of planting since the change of climate from reany to dry seseason. In this sason, rice field can be totaly damage because there is no water to irrigate.

The farmers in Gedangan village must be take an extra effort to get water to make their plant keep growing. For that effort, they usually have to rent a water pump to pump water from Setro reservoir to their rice field. Setro reservoir were build in 1970 and was officially used for irrigation by Ir. Sutami, Minister of Public Works of Republic of Indonesia at that time. The first main purpose of Setro reservoir was to use as water resources for drinking water around Rembang district., yet, by the growing of population, was needed to get new and better of water resources.

The new water resources in Pamotan and Sale (two subdistrict of Rembang district) was discovered in 1990. By discovering those water resources, the function of Setro reservoir as a raw material of water were stopped since have no more fulfill the requirement of quality, continuity, quantity, as well as financial ability of the district to operate.

In the year 2005 the government rebuild Setro reservoir to be able to be functioned as sources of water irrigation for the rice filed sorounding. The purpose of rebuilding of Stero reservoir was to provide its capacity of water than the old one (BAPPEDA REMBANG 2010). Note: BAPPEDA in English is stand for *The Council of Local of Planning and Development*. The old reservior have area only 10 hectares and around 6 meters depth of water or only can provide 600,000 cum of water. The volume of water in the new reservoir is very abundant and will not empty in the dry season. However, another problem are faced with this new reservoir, namely the elevation of surface water is lower than the elevation of rice field, cause require water pump to be able to hoist the water into the rice filed, and this mean also require extra cost to maitain the rice field until can be able to harvested. Those situation is difficult for farmers in Gedangan village, one side if they hire water pump to get the water a lot of money must be spent, in the other side if not hire water pump rice field cannot be irrigated and fail to harvest. So, both hire water pump or not the farmers will suffer of loss, and that situation need to be solved.

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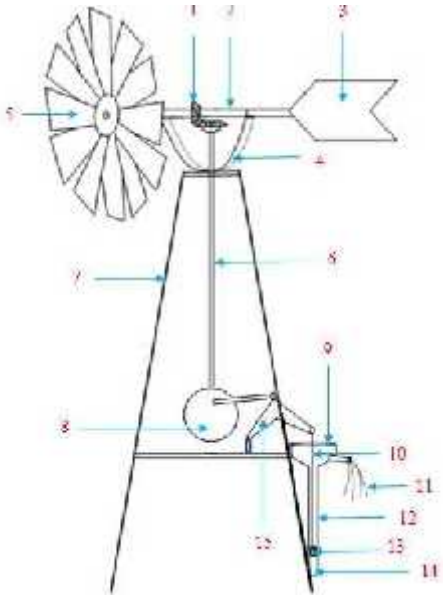
According to the applied technology, there are three types of water pump which are not used engine and of course fuel less. Those type of pump are air energy, wind energy, and solar energy pump. Data from IEAR mention that average yearly wind velocity in rice field area of Gedangan village minimum is 2,3 m/s and maximum is 9,2 m/s, and possible to power a water pump using wind energy. To make wind energy water pump need make a simple model and applicable, so that farmers also can make it, and before apllying, some test should be done to know the level of success.

RESEARCH METHODOLOGY

Methodology used in this research was combination between design and experimental of windmill. Experiment was conducted in the rice field located close to the Setro's reservoir in Gedangan village. Experiment time was taken 24 hours in a whole day. Measurement of wind speed and rotation speed of windmiil are done every 3 hours during experiment.

Design of windmill

Figure 1 shows the design of windmill features with water pump.



Note:
1. Gear / gear on
2. shaft
3. Tail
4. Bearing / Bearing
5. Mill
6. As
7. Tower / Tower
8. Gear / gear down
9. Storage
10. Stalk pump
11. Water
12. PVC Pipe
13. Rubber valve
14. Plastic valve
15. Crank

Fig. 1. Design of windmill

RESULTS AND DISCUSSION

Design of Tower

Tower where windmill is placed made as strong as possible using square pipe made from iron plate. Height of the tower is 1.5 meters. Detail of the tower can be seen in Figure 2.

Tower is made using four pillars in order stable and strong enough to support weight and movement of windmill. Field test result found that in designing tower with four pillars four parts, width, height, foundation, and material to be used must be thoroughly take into attrention in order save and running well.Those conditions according to the Alamsyah (2007) teory which be descriped in his thesis entitled "Take advantage of two blade wind turbin as the main power of alternator on wind electrical power". .

Experiment shows that tower with 60 cm width can fallen cause by storm. therefore, if the tower will be build in a big size have to be designed with sufficient wide. Instead of steel plate, material for tower can be changed using wood or bamboo according to the local condition to reduce the cost.

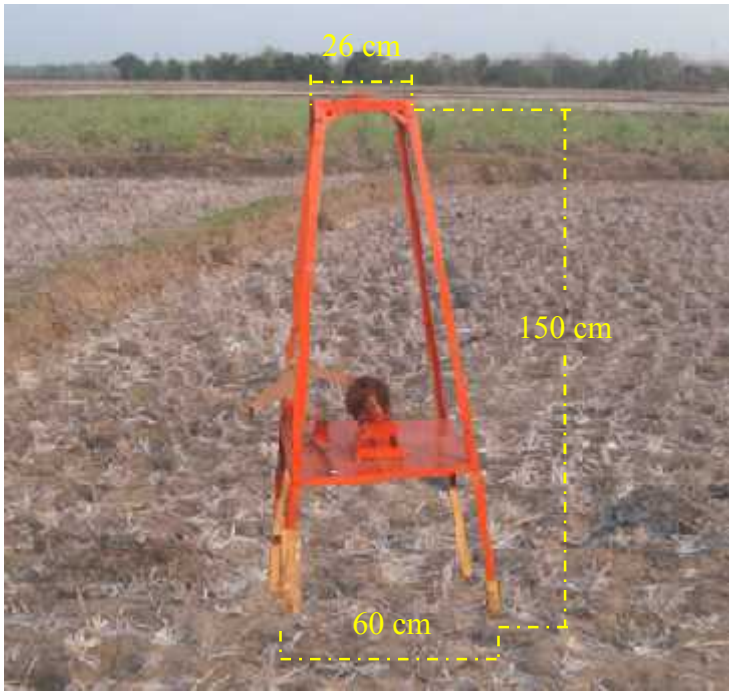


Fig. 2 Tower of Windmill

Design Tail of Windmill

The tail of windmill is a component consists of holder and leaf and has function to keep wheel and wheel direction removal in balance. Wheel direction removal is used to change the position of wheel from one position to another in direction of 360° so that wheel can rotate according to the gust of wind. This design enable the wheel can constantly rotate as long as wind blow eventhough direction of wind change. The length and area of tail must perfectly balance, in order windmill can running well without problems.

Based on the field experiment, the following formula, which derived based on the length of the spoke (r), can be used to design the length and area of tail:

a. Area of tail leaf (Le)

$$(Le) = \pi \cdot r^2 \cdot n \qquad (1)$$

where:

Le	= Tail leaf area (cm^2)
π	= 3.14
n	= coefficient (1/2, 1/3, 1/4, 1/5, 1/6, 1/7)
r	= spoke of the blade (cm)

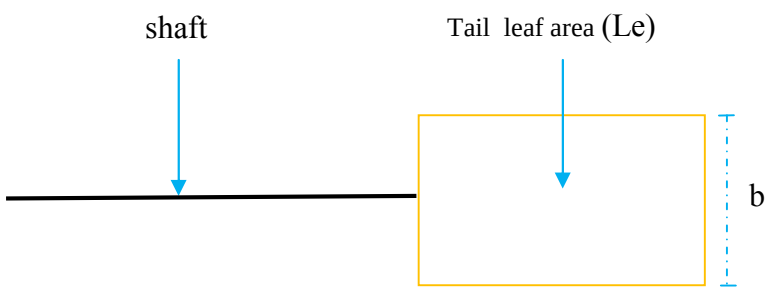


Fig. 3 Sketch area of leaf tail of windmills

The main factor to design tail leaf is calculate the minimum area. In the formula (1) above, there is coefficoent (n) in form of number ½; 1/3; ¼; 1/5; 1/6; and 1/7. This number is used to find coefficient of tail leaf minimum area, from the area of circle which is formed by spoke of the blade. The result shows that coefficient 1/7 is the minimum coefficient to design tail leaf area. The bigger of the coefficient, the stronger of windmill balance because the wider area of tail leaf. Yet, the wider of the tail leaf, the higher the making cost. Therefore, need to determine the minimum area of tail leaf in order get a good windmill with lower cost.

From the above coefficients and minimum tail leaf area, the following formulas can be derived:

$$\mathbf{Le_{min} = 1/7 \cdot \pi \cdot r^2} \qquad (2)$$

where:

Le_{min} = minimal tail leaf area (cm2)

1/7 = coefficient chosen

The above formula is used to design tail leaf in form of rectangular where the comparison between length and wide is 2 : 1.

Based on that formula and referring to Figure 3 above, the minimum of tail leaf area can be calculated, and the results given in Table 1.

Tabel 1. Size of tail leaf of windmills

No	Spoke (r) (cm)	Minimum Size of Tail Leaf $Le_{min} = 1/7 \cdot \pi \cdot r^2$ (cm^2)	Length of Tail Leaf side a (cm)	Length of Tail Leaf side b (cm)
1	20	179	18,9	9,46
2	25	280	23,6	11,8
3	30	403	28,3	14,1
4	35	549	33,1	16,5
5	40	717	37,8	18,9
6	45	907	42,5	21,2
7	50	1120	47,3	23,6
8	55	1355	52,1	26,1
9	60	1613	56,7	28,3
10	65	1893	61,5	30,7

Source: Analysis of the experimnet results, 2012

From Table 1, minimum tail leaf area and minimum length of side a and b can be known. The length of side a is the length of minimum tail leaf and denote as Pa_{min}, and use to determine the length of the tail of windmill Pe. The bigger windmill have radius of shaft 65 cm, length of side a = 61,5 cm, length of side b = 30,7 cm, and totally have an area of 1893 cm^2.

b. Length of tail (Pe)

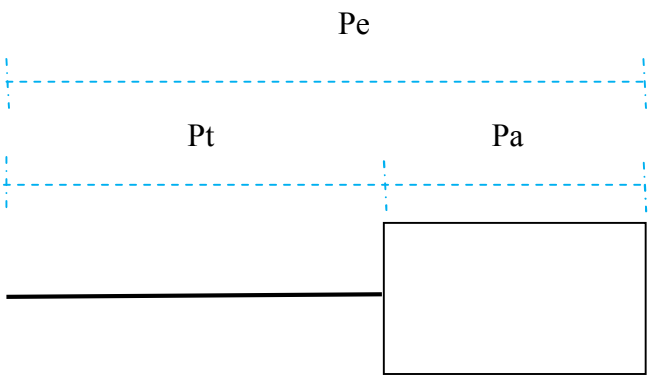


Fig. 4 Length of tail of windmill

$$\mathbf{Pe = Pt + Pa} \quad (3)$$

Where:
Pe = length of tail (cm)
Pt = length of stalk tail (cm)
Pa = length of tail leaf (cm)

To determine the length of the shaft tail (Pt) the following formula can be used:

$$\mathbf{Pt = Ds \times n = (2 \times r) \times n} \quad (4)$$

Where:
Ds = diameter of the blade (cm)
r = radius of the blade length (cm)
n = coefficient (1/4; 1/3; ½; 3/4, and 1)

Factor to design length of stalk (Pt) is the length of spoke of blade.. In the formula to calculate the length of tail shaft there is coefficient 'n' which consist of the numerical ¼; 1/3; ½; ¾; and 1. This numerical is taken to find the minimum length of tail shaft from blade diameter that is formed by the spoke of windmill. Experiment shows that coefficient n = 1 is the coefficient which correspond to design the length of tail shaft. It can be concluded that the lebgth of tail shaft is two time of the blade spoke or similar to blade diameter, and cn be derived the new formula to calculate the langth of stalk (Pt):

$$\mathbf{Pt = (2 \times r) = Ds} \quad (5)$$

The similar way is also done in determining the length of tail shaft. The results of the experiments find that the minimum length of tail shaft is blade diamater or two time the spoke of blade minus minimum length of tail leaf, where the form of the shaft is rectangular with length to wide is 2 : 1. Those minimum length of tail shaft can be written as a formula as follow:

$$\mathbf{(Pt_{min}) = Ds - Pa_{min} = (2 \times r) - Pa_{min}} \quad (6)$$

The length of windmill tail (Pe) is the sum of the minimum length of tail shaft (Pt_{min}) and the ;length of minimum tail leaf (Pa_{min}), where Pa_{min} find from Le_{min}. From that description can be written formula for the langth of windmill tail:

$$\mathbf{Pe = Pt_{min} + Pa_{min} = (Ds - Pa_{min}) + Pa_{min} = Ds.} \quad (7)$$

Therefore, the length of tail is two fold of the length of the spoke of the blade or similar to the blade, and can be wriiten as:

$$\mathbf{Pe = 2 \times r = Ds} \quad (8)$$

Based on the formula no. 9 above, can be known that the length of tail similar to two fold of the spoke of the blade. Therefore, for windmill which have length of spoke 65 cm, the length of windmill is 130 cm.

Design of the tail of windmill based on the calculation can be shown in Figure 5.

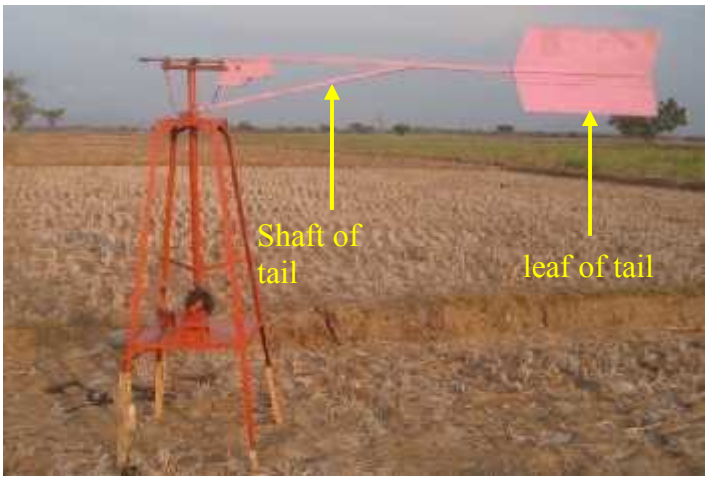


Fig. 5 Installation of Tails

Windmill assembly

Assembly process of windmill is the process to join together all pump components into one unit. The first step to be done is to inspect all components which will assembly and preparing the tools. The right step will make process of assembly easy and quikly as well as ensure the successfulness. The components consist of: windmills, exles, gear, pump, tail, and tower. Complete assembly of windwill shown in the Figure 6.

The result of Windmill design

Wind speed

The field measurement of wind speed was conducted using 'digital anemometer' and given in unit of meter per second (m/s), and the results were given in Table 2.

Fig. 6 Prototype of water pump power by windmill

Table 2. The results of field measurement of wind speed

No	hour	Wid velocity (m/s)
1	06.00	2,5
2	09.00	4,7
3	12.00	6,9
4	15.00	8,1
5	18.00	2,7
6	21.00	2,1
7	00.00	1,46
8	03.00	1,52

Source: Analysis of the result, 2012

Table 2 shows that average wind speed is 3,75 m/s. Maximum wind speed occur at 3.00 PM and minimum occur at 3.00 AM. The average Wind speed in the night is under 2,5 m/s, so that the water pump using windmill power only effective to use in the day, since the pomp is designed to use at the minimum wind speed 2,5 m/s.

The speed revolution of Windmill and the discharge

The measurement of windmill revolution was conducted by using digital tachometer in unit revolution per minute (rpm), while the measurement of water flow in the pump was using bucket or measurement cup. Test was performed to ten blades diameter which have been designed.

The steps on measuring the windmill revolution is by setting up special paper to detect the revolution of windmill's axle. When windmill start to rotate, press the test button on Tachometer, then directed infra-red light to the marking axle, so that the revolution of the axle will expose to the digital screen. The results of speed revolution of the widnmill's axle and the results of discharge measurement are given in Appendix 1.

CONCLUSIONS

Based on the srtudy of modeling of water pump and wind power, and the results of the test, can be drawn some conclusions as follow:

1. Some new formulas were found:
   a. Formula to calculate minimum area of the tail: $Le_{min} = 1/7 \cdot \pi \cdot r^2$.
   b. Formula to calculate minimum length of the tail-stem: $Pt_{min} = Ds – Pa_{min} = (2 \cdot r) - Pa_{min}$
   c. Formula to calculate the length of the tail: $Pe = 2 \cdot r = Ds$

2. The highest discharge of water flow is 0,07 liters per second for 1 meter diameter of widnmill blade.

3. The pump can start works at minimum speed of 2,5 m/s.

4. The results of ANOVA computer program test shows that the average discharge is influenced by diameter of blade and the revolution speed of windmill. The R value is adjusted to 0.772, means that the variability of water discharge which can be influenced by a variable diameter of blade and a wind speed is 77.2%.

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Appendix 1. The results of measurement of round shaft speed Windmills and Water Discharge based on blade diameter 40 cm to130 cm

			Blade Diameter																			
			40cm		50 cm		60 cm		70 cm		80 cm		90 cm		100 cm		110 cm		120 cm		130 cm	
no	w		Kp (Rpm)	Q (ltr/s)	Kp (Rpm)	Q (ltr/s)	Kp (Rpm)	Q (ltr/s)	Kp (Rpm)	Q (ltr/s)	Kp (Rpm)	Q (ltr/s)	Kp (Rpm)	Q (ltr/s)	Kp (Rpm)	Q (ltr/s)	Kp (Rpm)	Q (ltr/s)	Kp (Rpm)	Q (ltr/s)	Kp (Rpm)	Q (ltr/s)
1	06.00	2,5	0	0	0	0	0	0	0	0	47	0.012	49	0.013	53	0.016	51	0.015	48	0.012	44	0.010
2	09.00	4,7	0	0	0	0	66	0.022	70	0.024	76	0.027	81	0.030	85	0.033	78	0.028	73	0.025	69	0.024
3	12.00	6,9	0	0	0	0	149	0.049	155	0.052	162	0.054	169	0.057	168	0.056	156	0.052	150	0.049	143	0.047
4	15.00	8,1	0	0	0	0	180	0.061	184	0.062	192	0.064	198	0.066	214	0.07	205	0.068	193	0.064	188	0.063
5	18.00	2,7	0	0	0	0	0	0	48	0.012	54	0.016	62	0.020	69	0.024	65	0.022	60	0.019	57	0.018
6	21.00	2,1	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7	00.00	1,46	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
8	03.00	1,52	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0

Source : Result of the analysis, 2012

Note:
w = testing time ()
Ka = wind speed (m/s)
Kp = wheel shaft revolution speed (rpm)
Q = water discharge (ltr/s)

ENVIRONMENTAL MANAGEMENT EFFORTS DEVELOPMENT OF PIER INPULAU MARABATUANSOUTH KALIMANTAN

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ABSTRACT: Seaport development planning includes environmental aspects, which will follow the provisions of the Law of the Republic of Indonesia Number 32 of 2009 on the Protection and Environmental Management. Therefore, the importance of environmental management in the construction of the pier inPulauMarabatuanthat the activities carried out, the implementation is done with the environment (pay attention to aspects of environmental balance) and carried out on an ongoing basis. The purpose of this paper is to implement prevention and monitoring of adverse impacts and improve/develop the positive impact brought about by the development of sea ports in PulauMarabatuan, sub-district of Pulau Sembilan, district of Kotabaru, province of South Kalimantan. This study used qualitative research methods. Qualitative research method is a procedure that produces descriptive data in the form of documents, written, verbal information from the image/photo. Collecting data using interviews, observation and document review. The impact in the management of the construction phase of development of this port is dominated by socio-economic problems, such as labor, land compensation affected by the designation of the port area, the new economic activities such as boarding houses, shops, public unrest, disruption of social order while another is that the loss of vegetation, air quality deterioration, increased noise, vibration onset, and disruption of marine ecosystems. In terms of environmental management efforts used 3 pieces of approaches: technology approach, socio-economic and cultural approaches and institutional approach.

Keywords*: environmental management efforts, construction of the port.*

INTRODUCTION

The development of the territory of a region often has an impact on the increase in various sectors of activity. One of the matters relating to the development of the region is the growing population of well servicing needs economic, social and transportation. The level of ease of achievement in a region will give impetus to the increasing mobilization of the population itself.

One of the needs in transportation facilities and infrastructure is the ability to support the development of economic activity in sea transport. Marine transportation requires a halt, which is the port. The port is one of the gates and facilitates relations between regions, islands or among continents and nations to promote regional development. With these functions, the development of ports should be accountable in social and economic, technical and environmental.

Therefore, the development of ports in Indonesia will continue to be implemented in order to support freight/passenger transport, containers, general cargo, cruise pioneer, local cruise or cruise the people, either in the form of new construction or improvement of existing facilities.

In order to support the construction of the port in PulauMarabatuan, Kotabaru, South Kalimantan to support the activities of sea transport, passenger transport, containers, general cargo, cruise pioneer, local cruise or cruise people, either in the form of new construction or improvement of existing capacity, required a study activity which is able to provide a more comprehensive picture of the environmental impact assessment study and management of the impact of the construction of a port on the island.

LITERATURE STUDY

Environmental management programs undertaken aimed at suppressing/minimize negative impacts and maximize the positive impact on the environment. The approach taken may include technological approaches, socio-economic-cultural and institutional, such as:

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Technology Approach

Environmental impact management approach to technology intended to seek alternative of appropriate technologies that can be applied to minimize the negative impact on the environment.

Socio-Economic and Cultural Approach

Management of the environmental impact of socio-economic and cultural approaches pursued include:

1. Prioritize the local workforce (local) According to Reviews their ability to be involved in construction work
2. Compensation/restitution of land owned by a resident for the purposes of construction of the port with the principle of mutual benefit both parties (no land acquisition)
3. Help public facilities to the communities surrounding the port construction site in accordance with the capabilities of the proponent
4. Establish harmonious social interaction with community around in order to prevent social conflicts
5. Respect local customs prevailing in the community around the project.

Institutional Approach

Institutional approach is the institutional mechanism that will be pursued by initiator in anticipating significant impacts such as:

1. Cooperation with the agency concerned and related to environmental management;
2. Supervision of the work for environmental management by the competent authority;
3. Reporting the results of environmental management on a regular basis to the parties concerned.

RESEARCH METHODOLOGY

Research Methodology is based on the background, goals and objectives, namely:

Preparation

Preparation is done immediately is readiness in terms of equipment, administration and personnel.

Collection Methods and Data Analysis

Data Collection

Primary and secondary data collection is two (2) activities to be carried out in this study. Secondary data were obtained from the research/previous studies that have been done as well as from literature as well as information from the data from relevant agencies. While the primary data that is collected includes several environmental parameters expected to be affected by the impact of development activities port facilities. Primary data obtained from the results of observations, measurements and interviews and questionnaires.

Data Analysis

The data obtained from either the primary or secondary survey will be analyzed to formulate the current environmental conditions, and predict the impact that occurs after development activities.

Data analysis was done in various ways, such as:

1. Mathematical models, his formal method of estimating the magnitude of the impact on environmental components due to project activities using mathematical formulas, so the magnitude of the impact can be defined quantitative and behavioral effects can be traced.
2. Environmental Quality Standards, in the non-formal methods of forecasting significant impacts also taken through a comparison between the results of field measurements with Environmental Quality Standards (BML) as determined by the applicable regulations and environmental quality standards which have been agreed upon projected results.
3. Analog, this formal method based on analogy or comparing the environmental conditions and problems arise as a result of similar activities in different places.
4. Assessment experts/experts in their field, this informal method is used if encountered difficulties in collecting field data. Forecast impact produced by this method relies heavily on the knowledge and experience of the judge.

Sampling Method

Environmental components observed physical-chemical environment, flora and fauna components, socio-economic components and public health components.

Components of physical chemistry

Climate

Data analysis was performed by mathematical calculations based on empirical formulas that exist and descriptively.

Air Quality

To determine ambient air quality sampling will take place. Some of the factors that determine the results of the analysis, including:

• Wind direction
• Wind speed (m/s)
• Time and duration of sampling
• Air pressure (mmHg)
• Air temperature (^{0}C)
• The humidity (%)
• Pattern of diffused contaminants

Noise

Measurement of the noise level can be done in two ways:

• Simple way, carried by two people one to see the time and gave a downbeat readings instantaneous noise levels per five seconds within 10 minutes. The second person record instantaneous readout noise levels of sound level meter (SLM).
• Direct way, with an integrating sound level meter that has a measurement facility L_{TM5}, namely L_{eq} with every 5 seconds measuring time, measured for 10 (ten) minutes. With this facility $L_{Aeq,T}$ is obtained by directly reading on the sound level meter.

Hydrology

The approach taken in this study is a deterministic method by reviewing the results of observations, measurements of sampling directly in the field as well as literature studies and the results of previous investigations. Environmental components were analyzed, namely: a) The quantity of surface water (rivers), b) The quantity of groundwater, c) Seawater, including: ocean currents, tides and ocean waves.

Flora and Fauna

1. Terrestrial Flora and Fauna, Observe and identify environmental conditions in the vicinity of the land on the mainland species of flora and fauna that exist in the study area.
2. Aquatic Flora and Fauna, Observe and identify the type of flora that lives in waters in the study area affected by the construction of facilities at the port, especially mangrove communities and other plant species. Observe and care for the environment waters around port facilities, which will be built and directly affected from the surrounding waters.
3. Components of Social, Economic and Cultural Rights, analysis of the components of the social, economic and cultural done with existing secondary data processing quantitatively and qualitatively.
4. Components of Public Health, data analysis was performed by descriptive analysis.

Impact Forecast Method

In estimating the impact of the construction of facilities at the port's activities on the environment, the methods used are:

Formal Method

Formal method is a method of forecasting the impact of using a mathematical formula, such as forecasts of run-off water discharge, noise calculations, etc.

Non-Formal Methods

The non-formal method that will be used is the method of analogy, comparison of environmental quality standards and professional judgment.

Forecast Impacts

In this method, it will be done:

1. List of component test environment that exists around the location of the forecasted activities affected.
2. List of the activities test plan activities that potentially have an impact.

Method of Environmental Management

Environmental management efforts are organized based on the narrative form by outlining the following:

a. Source Impact
b. Impact Type
c. Impact Indicators
d. ImplementatedManagement

e. Location Management
f. Time Management
g. Executors and Supervisors

RESULT and DISSCUSSION

Description of Environmental Components

Physical Condition

1. Climatology, temperatures in the southern region of Kotabaru district ranged from 21.3 °C - 35.4 °C. Based on rainfall data showed that the average rainfall is highest in June (317.12 mm) and the lowest occurred in September (67.9 mm).
2. Oceanography
   - Tidal, tidal area has a mixed type skew double daily (mixed semidiurnal tide prevailing), which in one day happen twice tide and low tide twice, but high and different periods. Ups and downs in this area has a stable pairs is equal to 1.71 meters.
   - Wind and Waves, Pulau Marabatuan waters are a relatively open area of the attack in the ocean waves. Due to the wave hit, causing this area there are some beaches that have been abraded, especially the southern part of the coast side and southeast, while the other direction is relatively stable.
   - Flow, From the results showed that at high tide, from the east (Makassar Strait) flows moving into coastal waters and continues to move toward the north (Strait Sea) with maximum speed reaching 0.280 and upon reaching the highest tide speed is 0.183 m/s, whereas at the moment ebb, flow moves from north to south and from the west (Java Sea) moving eastward into the waters of the Strait of Makassar with smaller speed that is a maximum of only 0.200 m/s and low tide only 0.091 m/s.
3. Utilization and land cover, From the analysis of the data obtained imagery and land cover maps (land utilization) in the location area of study is the largest land is upland and coconut mixture for each 85.678 ha or 35.26% and 84.678 ha or 35.09. For settlement only as much as 17.367 ha or 7.20%.

Water Quality

1. Marine Water Quality, For data sampling parameters (brightness, pH, temperature, salinity and DO) in locations marine waters by relatively normal pH levels, i.e. in the range of 7 to 7.38. Temperature values are also within the normal range, i.e. 26 °C - 29 °C. Salinity values between 30-32 ppt and DO also normal value is between 6 to 7.4 mg/l, as shown in Table 3.3 below. As for the test parameters of physics, chemistry and biology for the shallow marine waters. Examination of the sea water quality is generally still qualified by LH Decree No. 51 of 2004 on Sea Water Quality Standard for marine life.
2. Well Water Quality, From the measurement results show relatively acidic acid-base level that is within the range of 3.20 to 4.79. Rated normal temperatures between 25 °C - 26 °C, and DO value was relatively low at 3-4 mg/l, whereas one sample point again with a more complete parameter shows still qualify for daily consumption.

Air Quality and Noise

Air quality

1. Sulfur Dioxide (SO_2), Test results Sulfur Dioxide in the neighborhood showed values ranging from 12.60 to 13.99 mg/Nm3. Based on these results show is still below Ambient Air Quality Standards based on Government Regulation No. 41 Year 1999 on the National Ambient Air Quality Standards.
2. Nitrogen Dioxide (NO_2), Examination Nitrogen Dioxide in the neighborhood showed values ranging from 27.30 to 33.91 g/Nm3. Based on these results show is still below Ambient Air Quality Standards based on Government Regulation No. 41 Year 1999 on the National Ambient Air Quality Standards.
3. Carbon Monoxide (CO), Test results Carbon Monoxide in the neighborhood showed values ranging from 205.80 to 261.53 mg/Nm3. Based on these results show is still below Ambient Air Quality Standards based on Government Regulation No. 41 Year 1999 on the National Ambient Air Quality Standards.
4. Particles (Dust), Results of the examination of particles in the neighborhood showed values ranging from 13.71 to 18.04 mg/Nm3. Based on these results show is still below Ambient Air Quality Standards based on Government Regulation No. 41 Year 1999 on the National Ambient Air Quality Standards.
5. Lead (Pb), Lead examination results in the neighborhood showed values ranging from 0:28 to 12:52 mg/Nm3. Based on these results show is still below Ambient Air Quality Standards based on Government Regulation No. 41 Year 1999 on the National Ambient Air Quality Standards.

Noise

The results of measurements of noise levels at locations surrounding environment demonstrates the value 53.6 to 57.8 dBA. Based on these results show the intensity of the noise still qualify by decree LH No. KEP-48/MENLH/ 11/1996.

Flora and Fauna

Flora and Fauna Mainland

1. Flora Land, coastal vegetation that is often found in PulauMarabatuan is WaruLaut (HibicusTiliaceus), PandanLaut (Pandarus Tectorius), Coconut (CocusNucifera) and Grass Run (Spinifex sp.), Coconut (Cocosnucifera), BanyanLaut (FicusBengalensis), Wild Banana (Musa sp.), Ketapang(TerminaliaCatappa), Breadfruit (Artocarpus Altilis) and Kapok (CeibaPentandra) and some plants/ trees.
2. Land Fauna, most of the land location of action plans, is a target ecosystems, with fauna that live in the built and natural ecosystems as follows: cats, birds, chickens, snakes, lizards, geckos lizards, dragonflies, butterflies, flies, mosquitoes, crickets and locusts and others.

Aquatic Flora and Fauna

Aquatic Flora

Types of sea grass obtained by observation in PulauMarabatuan is the kind of CymodoceaeSerrulata and EnhalusAcaroides.

Aquatic Fauna

From the observation of the entire transect line shows the percentage of live coral reef closures (living cover) with the most excellent category at transect 1 with presentation of non Acropora coral Acropora and 70.6%, 12.6% abiotic, dead scleractinia 9.6% and the other 12.6%. While closing the reef on transect 3 at 8% with broken coral condition.

The fish targets contained waters around Pulau Marabatuan include Red grouper, grouper and snapper Lumpur.

Population, Socio-Cultural and Economic

Population

Based on data from two villages in PulauMarabatuan, the total population of the total population of 3443 inhabitants with as many as 995. The number of households is the most populated village of TanjungNyiur the number of 1,998 inhabitants, while the Central Village just 1,445. When compared to the size of the island, the population density of the two villages of 14.27 inhabitants/ha.

Economy

Residents of PulauMarabatuan nearly 95% who depend on fishing.

Health Care

Health care on the island is quite good when viewed from the existing infrastructure, namely: general practitioners and dental respectively 1 and 6 paramedical aides.

Environmental Management

The following is a description of environmental management will be done to tackle work-related Marabatuan Port Kotabaru District Pulau Sembilan District of South Kalimantan Province.

Pre-Construction Phase

Initial Survey and Clearance

- *Socio-Economic and Cultural Approach*
  - Provide outreach to community problems port development plan of the location
  - Provide counseling, mentoring, training and coaching to local residents on a variety of skills to work and strive estimated is needed in port development on PulauMarabatuan through job training and education, also prioritize local residents to fill job opportunities.
  - Provide information and guidance to the community around social cohesion activities through associations and LKMD on Purposes and benefits for the citizens after the operation of the port.
- *Institutional Approach*
  - Tocoordinate with local village officials to limit the placement of the pin housing, especially against citizens directly adjacent to the project site.

Land Acquisition

- *Socio-Economic and Cultural Approach*
  - Find new land in lieu of land affected communities land for allotment of land for roads and ports inPulauMarabatuan.
  - Provide fair and decent compensation for loss of agricultural lands/plots earmarked for port location and land port road.
- *Institutional Approach*

  To coordinate with local village officials for the setting and payment of compensation for land residents who serve as harbor landside facilitie onshore.

Construction Phase

Mobilization of Labor

- *Socio-Economic and Cultural Approach*
  - Provide ample opportunities to residents living around the location of the port construction project site inPulauMarabatuan to be accepted as a construction labor and attempted informal sector.
  - Do not give the difference in treatment between local labor and labor from outside locations.
  - Provide occupational accident insurance to workers.
  - Provide labor costs in accordance with the standards of the Provincial Minimum Wage (UMP), which applies in the province of South Kalimantan.
- *Institutional Approach*
  - To coordinate with local village officials for socialization to the residents who have a negative perception
  - To coordinate with the Department of Labor District. Kotabaru and Indonesia Workers Union to supervise the work and initiator, which employs a workforce.

Mobilization of Equipment and Materials

- *Technology Approach*
  - Using the vessel unfit for use and in accordance with the tonnage capabilities for lifting heavy equipment and materials.
  - Requiring ships carrying heavy equipment for demobilization is ship that has engine airworthiness of harbormaster.
  - Making stock file (material accumulation area) in the port construction project site locations.
  - Constantly doing the watering at the project site during the demolition of heavy equipment from ship to land/workshop heavy equipment.
- *Socio-Economic and Cultural Approach*

  Dissemination of the ordinance unload the heavy equipment of the ship to the mainland.
- *Institutional Approach*

  To coordinate with the Department of Transportation District. Kotabaru traffic police to supervise and direct the transport vehicle.

Land Reclamation and Maturation

- *Technology Approach*
  - Implement appropriate methods of reclamation work.
  - Choose the design and construction of the gabion wall retaining reclaimed beach with precise construction designs.
  - Conducting surveillance and maintenance ofurug material vehicles regularly in order to remain normal condition of the vehicle and its exhaust gases still meet quality standards.
  - Limiting the transportation of cargo vehicles within the limits of optimal urug.
  - Avoid material spills during transportation by closing roads/material construction with tarpaulins/plastic.
  - Constantly doing the watering on the main road.
  - Installing traffic signs.
  - Flush regularly.
- *Approach of Social, Economic and Cultural Rights*
  - Conduct outreach to workers about the use of safety devices work in accordance with the working procedures or SOP.
  - The proponent register workers at the PT. Jamsostek (social security) and employs a workforce in accordance with labor laws.
  - Requiring to all workers to meet the safety regulations set by the company.
- *Institutional Approach*
  - To coordinate with the Transportation Department and Public Works,Traffic Police of Kotabaru District with the District of Pulau Sembilan to oversee and direct the transport vehicle.
  - To coordinate with the Department of Labor Kotabaru District province.

Construction of Pier and Trestle

- *Technology Approach*
  - Accommodate the interests of the fishermen, especially fishermen in Pulau Marabatuan in the dock is by way of moorings in the dock make this, which is equipped with ascending and descending stairs hand-side of the pier.
  - Make signs such as navigation channel buoy for ships that will come out and go in the harbor of PulauMarabatuan.
  - Installation of steel pile is done without damaging/breaking the basic composition of the coral in the beach.
  - Avoid spillage of material/grease that can disrupt marine ecosystems.
  - Installation of signs/information about their activities.

- All trucks transporting material should be equipped with pliers tub cover made from tarpaulin.
- Flush water on the road.

➢ *Approach of Social, Economic and Cultural Rights*
- To disseminate to the workers on the use of safety devices work in accordance with the working procedures or SOP.
- The proponent register workers at the PT. Jamsostek (social security) and employs a workforce in accordance with labor laws.
- Requiring to all workers to meet the safety regulations set by the company.

➢ *Institutional Approach*
- Coordination with the Department of Marine & Fisheries Kotabaru district of South Kalimantan province.
- To coordinate with the Department of Labor Kotabaru District provionce.
- Coordination with accident insurance (Jamsostek).

Development Land Side Facility

➢ *Technology Approach*
- Selection of transport vehicle (truck) from the year of manufacture is relatively new;
- Maintenance of the transport vehicle engine periodically;
- Restrictions on the transport vehicle speed raw material/adjuvant;
- Conduct regular watering at the project site locations.
- Planting trees around the project site.

➢ *Approach of Social, Economic and Cultural Rights*
- Treatment is free for members of the community around the project site that is affected.
- Conduct outreach/dissemination to the workers on the use of safety devices work in accordance with the working procedures or SOP.
- Requiring to all workers to meet the safety regulations set by the company/contractor.

➢ *Institutional Approach*

Coordination with the Department of Health and other relevant agencies in the District of Pulau Sembilan, Kotabaru regency, South Kalimantan province.

Mechanical and electrical work

➢ *Technology Approach*
- Managing the waste material/ litter as a result of these activities properly.
- Avoid chemical spills (oil, gasoline, etc.) to spill into the sea.
- Provision of bins around the location of the project site.
- Implementation of labor regulations on matters relating to K3.
- Placing the machines into the enclosed area in order to localize the source of the noise.

➢ *Institutional Approach*

Coordination with village officials and government districts of Pulau Sembilan, Kotabaru district.

Heavy Equipment Demobilization

➢ *Technology Approach*
- Requiring ships carrying heavy equipment for demobilization is ship that has Kelaikan engine of harbormaster.
- Using the vessel unfit for use and in accordance with the tonnage capabilities for lifting heavy equipment.

➢ *Approach of Social, Economic and Cultural Rights*
- Inform the community.
- Dissemination of the procedure for unloading heavy equipment from the mainland towards the ship.

➢ *Institutional Approach*

Coordination with village officials and government districts of Pulau Sembilan, Kota baru District.

Demobilization of Labor

➢ *Socio-Economic and Cultural Approach*

Provide training and short courses for local workers who will decide his contract.

➢ *Institutional Approach*
- To coordinate with village officials for socialization to the residents who have a negative perception due to layoffs.
- To coordinate with the police party to always remain vigilant, when things happen that would disrupt the social order.

Operational Phase

Mobilization of Labor

➢ *Socio-Economic and Cultural Approach*
- Provide ample opportunity for residents to be accepted as labor for trying to port operations informal sector.

- Admission and establishment of labor adapted to the needs and skills possessed.
- Do not give the difference in treatment between local labor and labor from outside locations.
- Provide occupational accident insurance to workers.
- Provide labor costs in accordance with the standards of the Provincial Minimum Wage (UMP) which applies in the province of South Kalimantan.

➢ *Institutional Approach*

- To coordinate with village officials to cooperate District of nine islands in the selection for port operations.
- To coordinate with the Department of Labor and Kotabaru District All Indonesian Workers Union (SPSI) to supervise the work and initiator which employs a workforce.

Operation of the Port

➢ *Technology Approach*

- Creation of emission reduction tool (Electrostatic precipitator).
- Preparation of liquid waste treatment building.
- Provision of sorting garbage containers in every corner of the place and gathering activities of the people.
- Make signs prohibiting littering.
- Make signs traffic signs.
- Conduct tree planting.
- Provide Reception facility (shelter waste / used oil) from the ship.
- guarding the port location for 24 hours by security.
- Maintenance of heavy equipment used.
- Completing a fire extinguisher.

➢ *Approach of Social, Economic and Cultural Rights*

- Dissemination of any activity related to port operations to the community in Pulau.
- Provide the location for Pulau Marabatuan and surrounding communities to strive informal sector.
- Facilitate permit activity for the community in PulauMarabatuan and its surrounding to do some business.

➢ *Institutional Approach*

- Coordinating with the village or sub-district government officials of Pulau Sembilan.
- Coordination with BLHD Kotabaru district South Kalimantan province.
- Coordination with the Department of Labor office Kotabaru district South Kalimantan province.
- Coordination with the district forestry offices Kotabaru, South Kalimantan.

CONCLUSION

Based on the analysis, it can be concluded that the impact of port construction work in Pulau Marabatuan, Pulau Sembilan sub-district Kotabaru district, South Kalimantan province, which comprises the pre-construction phase, construction, post-construction and operational phase can be managed in three approaches: technological approaches, socio-economic approach and culture, and institutional approaches.

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FLOOD PREDICTION TO PREVENT FLOODING

(Case Study : Pasangkayu River)

M Lukman [1], S Pallu [2], F Maricar [3], A Thah [4]

ABSTRACT: Pasangkayu river originates at Central Sulawesi (upstream) and flow through West Sulawesi Province, then to Makassar straight has its catchment area of 1,000.80 km^2 and 196 km length. Geographycaly , hydrology and climatology condition shown that Pasangkayu watershed has the tipical tropic condition. The flood control structures sometimes built not by an accurate analysis, but used to be construct when flooding over, as a response of the flooding. To prevent flooding, it must be started with hydrologic analysis, to estimate the peak flow with several methods at several different return periods. This study covered 30 km length of Pasangkayu river from downstream as the most effected flooding area, with 3 rainfall station , named ; Lalundu, Palolo and Kulawi.(year 1990 – 2010)
Based on average maximum daily rainfall ,,Log Pearson Type III Distribution method has been used to analyse the frequency of rainfall at each return period , and the recapitulation of flood design discharge using Nakayasu Method shows in Table 1.

Table 1. Flood Discharge using Nakayasu Method

Return Period (year)	Flood Discharge (m^3/s)
2	166
5	206
10	231
20	251
25	259
50	284

Hydraullic analysis using HEC-RAS , Mathematic Model , steady flow, one dimendional analysis with open network.

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Surface runoff hydrograph resulting from the analysis shown as on Figure 1.

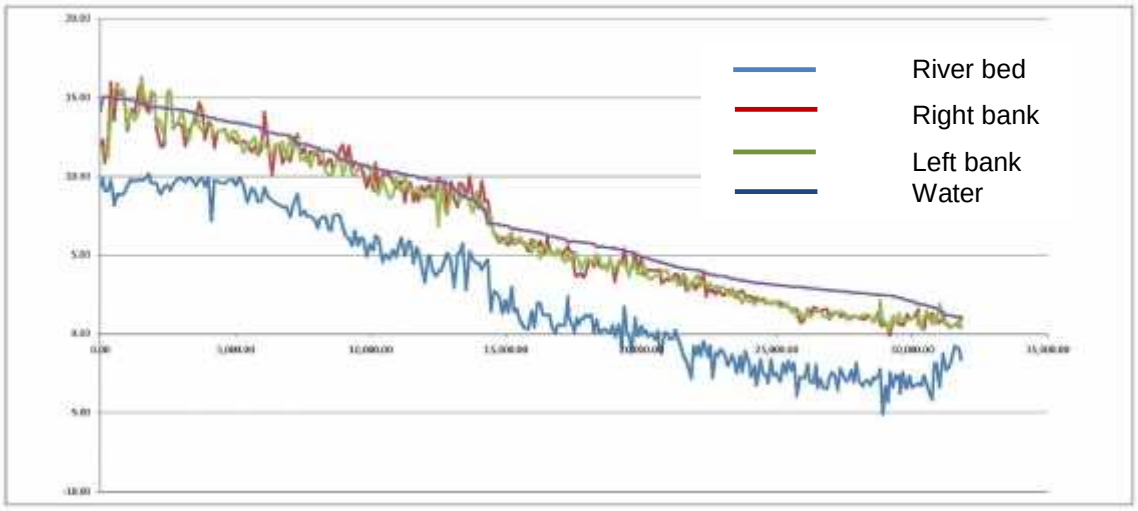

Figure 1. Water Surface

The result shows that even with 2 years return period, the capacity of Pasangkayu river not affordable for the discharge (166 m^3/s). Based on the result the decision maker could make decision , to construct flood control structure to prevent flooding at a certain years of return period.

Keywords : *Flooding, Flood Contro*

INTRODUCTION

Pasangkayu river originates at Central Sulawesi (upstream) and flow through West Sulawesi Province, then to Makassar straight has its catchment area of 1,000.80 km^2 and 196 km length. Geographycaly , hydrology and climatology condition shown that Pasangkayu watershed has the tipical tropic condition.

The design for flood control structures used to be with very limited topography data , such as 10 – 25 meters from each side of the river bank, and about 30 km length. With the very limited topography data, the analysis just show the water surface, and could not give any information about the area affected by flooding..

To prevent flooding, it must be started with hydrologic analysis, to estimate the peak flow with several methods at several different return periods.

There are 3 rainfall stations namely ; Lalundu, Kulawi and Palolo in that region as shown in Table 2

Table 2 : Annual Maximum Rainfall of Station Lalundu, Kulawi and Palolo

Year	Station		
	Lalundu	Palolo	Kulawi
1990	60.0	46.0	74.0
1991	92.0	90.5	93.5
1992	49.7	31.5	67.9
1993	71.3	66.7	75.8
1994	63.4	56.3	70.5
1995	44.3	23.6	65.0
1996	52.4	36.0	68.7
1997	75.9	93.7	58.1
1998	59.0	41.7	76.2
1999	40.6	22.6	58.5
2000	53.9	22.0	85.7
2001	58.5	47.0	70.0
2002	67.8	56.2	79.3
2003	93.2	68.0	118.3
2004	56.3	43.8	68.8
2005	83.9	92.0	75.8
2006	43.6	31.2	56.0
2007	51.2	36.0	66.3
2008	67.5	35.7	99.3
2009	65.0	37.8	92.2
2010	53.5	25.4	81.6

Source: BWS Sulawesi III & Calculation Result

Methodology of Study

This study started with hydrologic analysis , then discharge analysis as follows :

1. .Annual maximum rainfall
2. Regional daily maximum rainfall using Thiessen method
3. Probable rainfall using Log Pearson type III for different return periods
4. Examine the suitable distribution, using Smirnov-Kolmogorov , and Chi-Square method (Soewarno,1995)
5. Analysis of discharge will started with the data of rainfall intensity, catchment area and the length of the river
6. Hourly Rainfall intensities calculated with Mononobe fomula
7. Calculate hourly effective rainfall for different return periods (Fernandez Bonifacio and Salas D.Jose, 1999)
8. Discharge will be calculated using Nakayasu Sintetic Unit Hdrograph
9. Hydraullic analysis using HEC-RAS , mathematic Model , steady flow, one dimendional analysis with open network (Hydrologoc Engineering Centre, 2010)

RESULT AND DISCUSSION

This study covered 30 km length of Pasangkayu river from downstream as the most effected flooding area, with 3 rainfall station , named ; Lalundu, Palolo and Kulawi

Based on average maximum daily rainfall ,,Log Pearson Type III Distribution method has been used to analyse the frequency of rainfall at each return period, in figure 2 shown the Nakayasu Sintetic Unit Hdrograph, and in Figure 3 Shown the Hidrograph Planning Discharge of Flooding and in Table 3 shown the discharge of Flooding Hidrograph

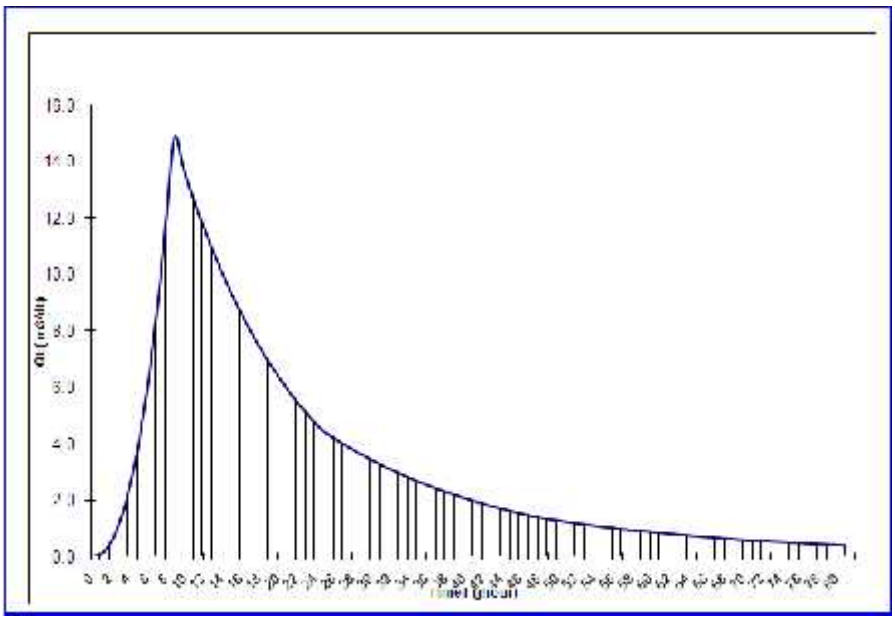

Figure 2. Nakayasu Sinthetic Unit Hydrograph

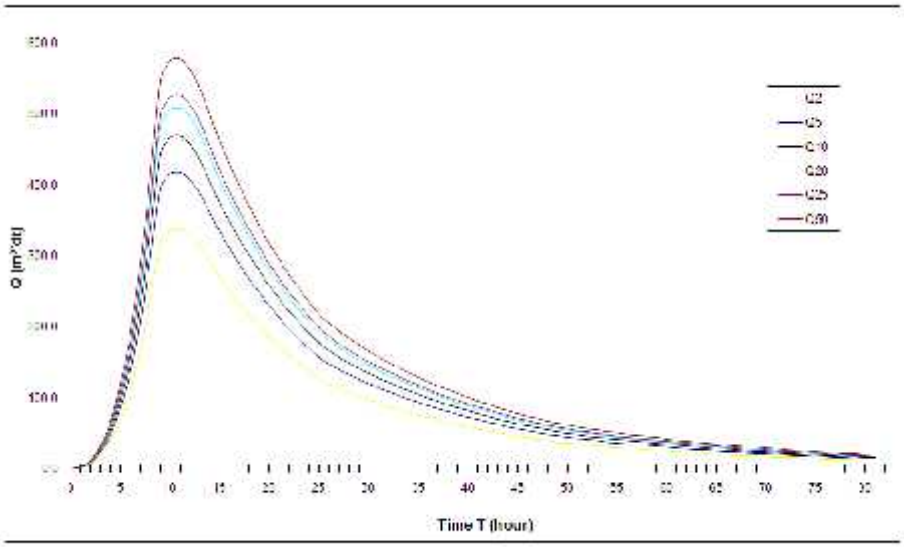

Figure 3. Hidrograph Planning Discharge of Flooding

Table 3 Discharge of Flooding Hidrograph

T (jam)	Q2 (m³/dt)	Q5 (m³/dt)	Q10 (m³/dt)	Q20 (m³/dt)	Q25 (m³/dt)	Q50 (m³/dt)
74.00.	35.66.	44.13	49.57	53.80	55.61	61.05
75.00.	34.86.	43.14	48.45	52.59	54.36	59.68
76.00.	34.08.	42.17	47.36	51.41	53.14	58.34
77.00.	33.31.	41.22	46.30	50.25	51.95	57.03
78.00.	32.56.	40.29	45.26	49.12	50.78	55.75
79.00.	31.83.	39.39	44.24	48.02	49.64	54.49
80.00.	31.12.	38.50	43.25	46.94	48.52	53.27
81.00.	30.42.	37.63	42.27	45.88	47.43	52.07

and the recapitulation of flood design discharge using Nakayasu Method shown in Table. 4

Table 4 Flood Discharge using Nakayasu Method

Return Period (year)	Flood Discharge (m^3/s)
2	166
5	206
10	231
20	251
25	259
50	284

Hydraullic analysis using HEC-RAS , Mathematic Model , steady flow, one dimendional analysis with open network. Surface runoff hydrograph resulting from the analysis shown as on Figure 3

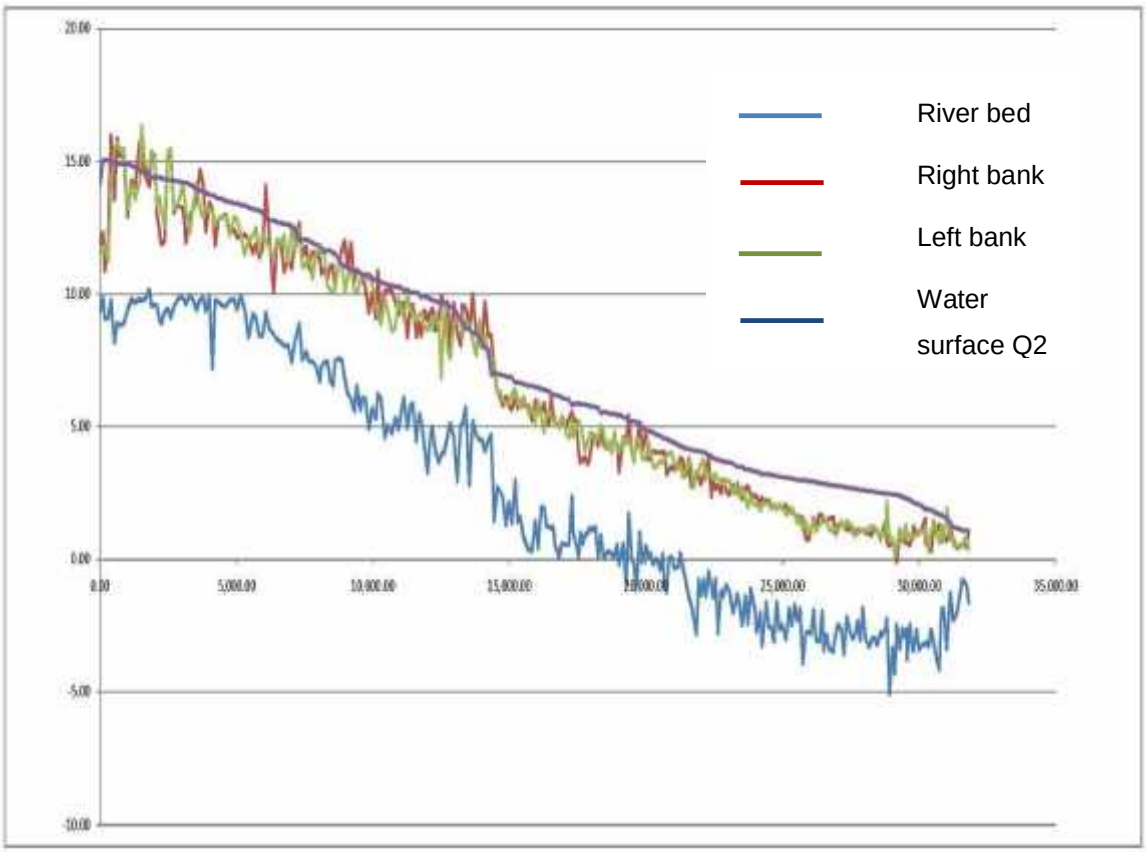

Figure 3 **Water Surface of Pasangkayu River for 2 year Return Period**

The result shows that even with 2 years return period, the capacity of Pasangkayu river not affordable for the discharge (166 m^3/s). This result just show about 30 km length of the river of 190 km length,

CONCLUSION AND RECOMMENDATION

Conclusion

Based on the result, the decision maker could make decision , to construct flood control structure to prevent flooding at a certain years of return period. As the river has its length about

Recommendation

We suggest to the management of river basin to provide more topography survey in the study/design stage, because with the very limited topography data, the result just shown the water profile in the river, at the specific reach In other hand , with more topography data, all the flooding area and the height of flooding could be analysis

ACKNOWLEDGEMENTS

The authors would like to appreciate the chief and all the staff of Main Office of Pompengan Jeneberang River Basin (BBWS-PJ), and the chairman and all the members of HATHI South Sulawesi for all the support and encouragement.

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FLOODS ANALYSIS IN JEMBER URBAN DRAINAGE SYSTEM

E. Hidayah [1], Sriwahyuni [1] and W. Yunarni [1]

ABSTRACT: Recently flood innundation in the Jember urban area shows that an increase in the number of point locations, and the height of puddle. Therefore, traffic congestions, and community activities due to flood innundation in Jember urban area can be disrupted. The purpose of this study was to analyze the causes of flooding in Jember urban area using SWMM. The flood events in three city districts of Jember was evaluated by assessing the intensity of the rainfall, the slope of the land surface, drainage channels, land use, and channel capacity. Based on measurements of the topography (drainage area, slope, and flow direction) was made schematization of the network system consisting the location of raingauges, the sub catchments, junctions, conduits, and outlets. Each object properties required input parameters, such as: elevations, areas, manning roughness values on oveland flow and channels, and the dimensions of the channel. The reliability of the model parameters was determined by calibrating the model, which compares the water level in the channel of the observations with the results of the model output. Furthermore, the causes of the flooding were analyzed through simulation models. The analysis showed that there were 111 noodes flooding. The causes of flooding ie: the channel capacities were not able to accommodate desaign rainfall for 1, 2, 5 or 10 year return period, there were the contribution of water from irrigation drainage, there were the channel bottlenecks, inlet holes were not allow water to enter the drainage channels, and there were sedimentation on the channel.

Keywords: drainage system, urban floods, design precipitation, rainfall-runoff, SWMM.

INTRODUCTION

Lately, the expansion of the Jember urban area growing faster, it is characterized by the increasing number of housing by developers and private housing. The growing number of housing caused a land conversion of agricultural land into residential. This condition will trigger a flood due to the increased amount of surface runoff. In topography, the Jember urban area is possible to drain the precipitation into the Bedadung river and its tributaries, but in fact it is hard to avoid flooding. Rainfall event that only a few moments with a relatively high intensity will cause inundation in some locations.This leads so-called pluvial flooding which is saturation of drainage infrastructure and there are overland flow along streets (Maksimovic et al., 2009). This condition would disrupt community activities, triggering damage to roads, and smooth transportation that will ultimately have an impact on the increased costs of road maintenance and vehicle operating costs. The floodwaters for each area of the city have different causes. Therefore, before any treatment for flooding needs to know the factors that cause a flood in advance.

SWMM is hydrology-hydrolic model that is developed by the United States Environmental Protection Agency (EPA) (Meierdiercks, 2010). This model has been successfully applied such as: for urban flood forecasting in the city of Dongguan in southern China (Jiang, Chen and, Wang, 2014), to solve the problem of water and to reduce losses due to rain (Weilin, Changhong, Xiaowen, Zhaoli, 2014). In addition, it has been widely successful SWMM used to calibrate and to validate such as to Jinan, a typical piedmont city in China (Yu, Huang and Wu, 2014). In order to realize effective and scientific management of urban storm flood, the establishment of rainstorm and flood model has important significance for the city to find out the causes of develop urban waterlogging countermeasures (Chen, 2010), (Zhao, Tong, Wang, 2009). This paper used SWMM to simulate floods with a different design scenario in Jember urban area, to identify the flooding spots and find out a variety of possible reasons, so as to advance the corresponding measures on this basis and provide scientific basis for urban flood control and drainage construction.

SWMM

The swmm is hydrology and hydraulic model that is widely used in urban storm flood, combined sewers, drains and other drainage systems analysis and design since its initial development in the world (Jorge et.al, 2005), (Rosman, 2005). Swmm consists of many functional program blocks, besides a coordinating executive block. The blocks can be overlaid and run sequentially or can be run separately with the linking or interfacing data being transferred between the blocks (Eric et. al., 2006). In SWMM, the whole study area is divided into subcatchments, and precipitation is averaged on the subcatchment. The surface runoff is produced on subcatchments also, and three methods can be used to produce the surface runoff. The surface runoff in a subcatchment is regarded to fully enter the junction related to it to be

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transported in the storm sewer conduits to the area outlet, so there is only pipeline runoff flow and no surface runoff routing is considered.

THE STUDY AREA

Jember urban area is located at latitude 8°4'30"S to 8°13'0"S and longitude 113°38'30"E to 113°45'30"E which is cleaved by Bedadung river with tributaries: Rembangan, Antirogo and Jompo. Its total area is approximately 108,98 km^2, and consist of third district: Patrang 36,99 km^2, Sumbersari 37,05 km^2 dan Kaliwates 24,94 km^2 (**FIG. 1**). Population densities in the Patrang District, Sumbersari district, and Kaliwates district are 2553.96 persons / km2, 3408.34 personss/km2 and and 4485.20 persons/km2 respectively.

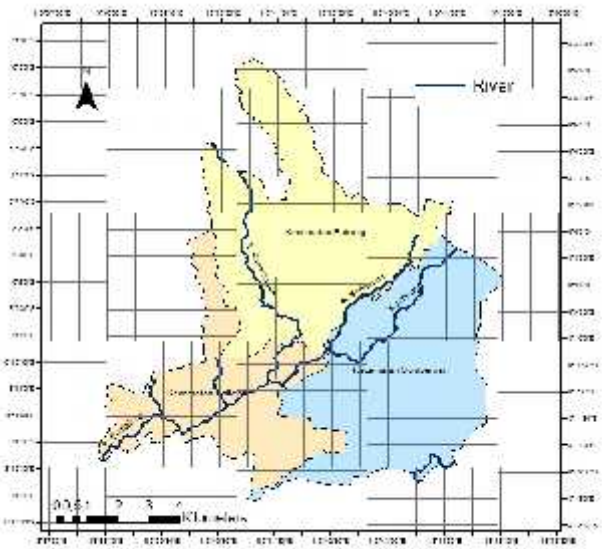

FIG. 1 Location of the study area.

THE THOPOGRAPHY

An average elevation in urban area is 80 m above sea-level. Based on DEM Aster GDEM 30, these area consist of two slope. The upstream portion has a steep slope that is almost entirely made up of natural terrain, with average slopes close to 7.43%, and these urban area are almost completely flat and their ground elevation is not so flat and their elevation ranges fluctuate between 5-10 ma.sl (above sea level) (Fig .2). According to the measurements results of surface topography of project, the steep slope is generally the case in the Patrang district to range between 0-10%, and the flat slope occurred in the Sumbersari and Kaliwates district with 0-3% and 0-2% respectively.

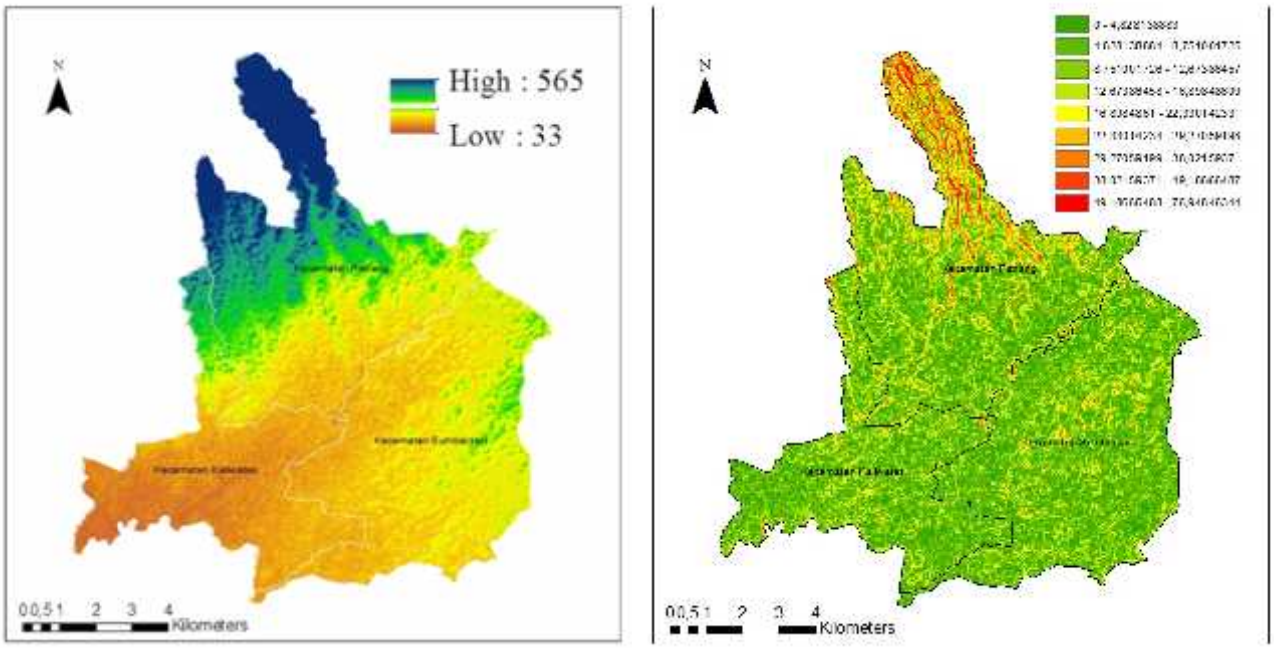

FIG. 2. Digital Elevation Model and Slope Map

LANDUSE INTERPRESTION AND CURVE NUMBER COEFICIENT CALCULATION

The main types of land use accoding to land cover that developed by google Earth (http://www.google.co.id/net/place/Jember,+Kab.Jember, 2015) include green space (evergreen broadleaf forest, cropland, and cropland natural vegetation mosaic) and urban and built up. The classified result is shown in FIG. 4. that the area proportions of different land cover classes. It is seen that the most of upper reaches of the study area is green space. In the low reaches, the land cover is dominated by urban and built up. Also, the curve numbers for each class are listed in Table 1. The curve number values of green space, and urban and built up were 55-91, and 68-98.

Table 1. Land cover and curve numbe values for each class

Class Number	Class description	Proportion %			Curve Number value
		Patrang	Sumbersari	Kaliwates	
1	green space	77.71	23.7	40.73	55-91
2	urban and built up	22.29	76.3	59.27	68-98

THE DRAINAGE NETWORK

Drainage network in the area is relatively complicated; the construction combines of sewage, irigation drainages and rainwater drainage canals on municipal roads. Drainage network system in all three districts can be grouped into 16 networks (FIG. 3). Five nerworks are located in Patrang District these are Jalan Srikoyo Drainage (JSRD), Rembangan River- Jalan Srikoyo Drainage (RR-JSRD), Kemuning-Rembangan River Drainage (KRRD), Polo Kemuning River Drainage (PKRD), and South Patrang–Polo River Drainage (SPPRD). Seven nerworks are located in Sumbersari district these are Kotok Irrigation–Ajung River Drainage (KIARD), Kotok Irrigation –Cakol River Drainage (KICRD), Jalan Mastrip Drainage (JMD), Jalan Kalimantan Drainage (JKD), Jalan Jawa Drainage (JJD), Jalan Sumatra Drainage (JSUD), Cakol River–Bedadung River (CRBRD), Four nerworks are located in Kaliwates District, these are Bedadung River–Jompo River Drainage (BRJRD), Jompo River–Argopuro River Drainage (JRARD), Argopuro River–Semanggir Irrigation Drainage (ARSID), and Bedadung South River Drainage (BSRD).

JRSD drainage network is divided to 115 the sub catchments, that is a contribution from residential area and drainage above and the outlets are the Baratan and Patrang rivers. RRJSRD network is divided to 120 the sub catchments that is a contribution from road drainage and irrigation, and outlet is the Patrang River. RRD network is divided to 124 the subcatchments that is a contribution from road drainages, the Branjangan canal and irrigation drainage, and outled is the Rembangan Rivers. PKRD network is divided to 61 the subcatchments that is a contribution from road drainage, residential area, and irrigation drainage, and the outlet are the Polo and Kemuning River. SPPRD network is divided to 142 the subcatchments that is a contribution from residential area, and irrigation drainage, and the outlet is the Jompo River. KIARD network is divided to 129 the subcatchments that is a contribution from Sriwijaya residential area, and Dam Konto overflow, and the outlet is the Ajung river. KICRD network is divided to 80 the subcatchments that is a contribution from residential area, and irrigation drainage, and the outlet is the Cakol River. JMD network is divided to 31 the subcatchments that is a contribution from residential area, and irrigation drainage, and the outlet is the Antirogo River. JKD network is divided to 68 the subcatchments that is a contribution from of road and campus drainage, and the outlet is the Bedadung River. JJD network is divided to 32 the subcatchments that is a contribution from drainage channels from urban areas, campuses and irrigation, and the outlet is the Antirogo River. JSUD network is divided to 39 the subcatchments that is a contribution from residential area which is outflowed to the Antirogo River. CRBRD network is a contribution from irrigation drainage and residential area of Jl. Karimata, and the outed is the Cakol river. BRJRD network is divided to 131 the subcatchments is a contribution from residential area and the outlet to the Jompo River and partly derived from Jln A Yani, and the outlet is the Bedadung River. JRARD network is divided to 117 the subcatchments that is a contribution from irrigation at the norhtern railway and a residential area, and the outlet is the Bedadung river. ARSID network is divided to 393 the subcatchments that is combination of residential area, and road drainage and the outlet is the Semangir canel. BSRD network is divided to 129 the subcatchments is a collection of residential area and the outlet is the Bedadung River.

The drainage network in Patrang district consists mostly of a network of natural ditches and drains, with the condition. The drainage network in Sumbersari district and Kaliwates district consists mostly of a network of artificial drains, including short stretches of closed conduits. Natural channels in pristine condition or overgrown shrubs or gravel, curved, and walled sand hollow, shallow, and irregularly shaped, with a range of manning rougness values between 0.028 to 0.125. Meanwhile, the condition of the artificial channel is protected by masonry with finishing or without finishing, or concrete with a range of manning rougness values between 0.016 to 0.033

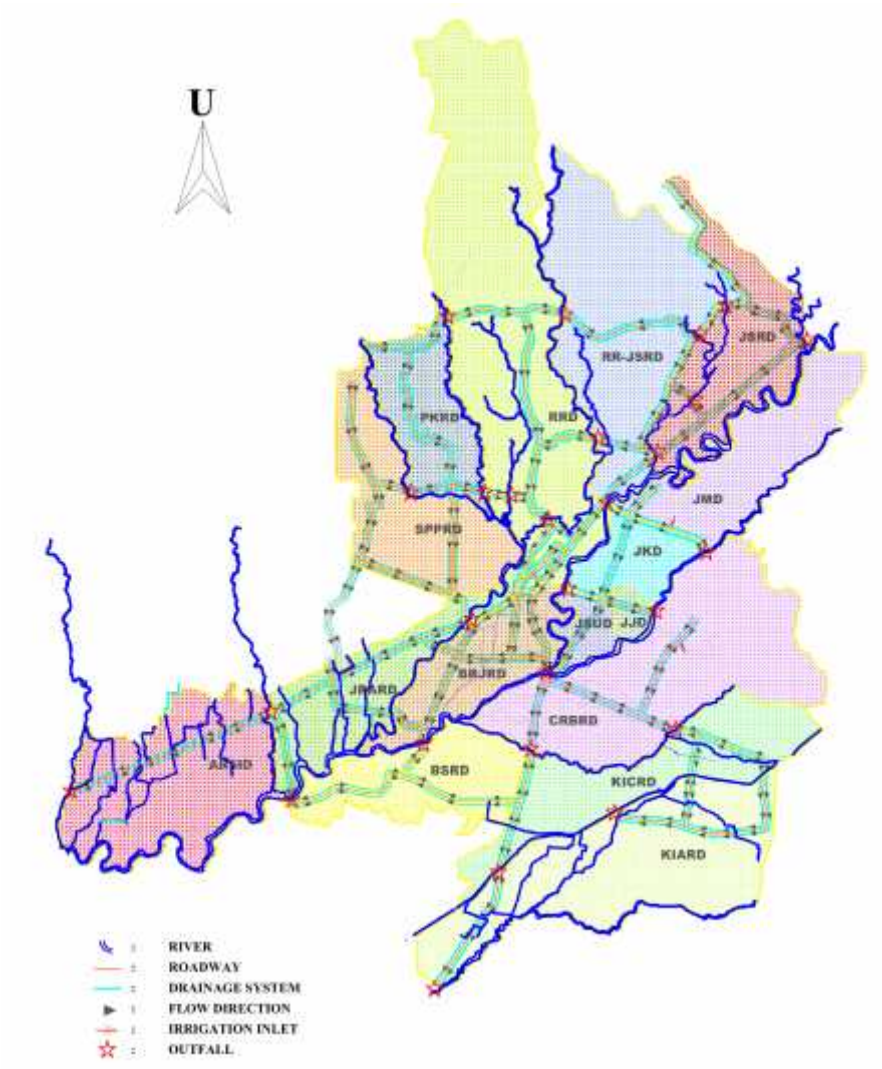

FIG. 3. Drainage network

RAINFALL INPUT AND FORCASTING

Rainfall Input

Rainfall data input is used in this study comes from the measurements of five raingauges (Semanggir, Jember, Wirolegi, Kasmaran, and Sembah) that is recorded from 2004 to 2014. Most of the rainfall is concentrated in the September to April. According to records of the fifth raingauges, a maximum rainfall depth of 135 mm occurred in 2006, while minimum rainfall depth of 54 mm occurred in 2012.

Rainfall Forcasting

Based on Smirnov-Kolmogorov goodness-of-fit test, rainfall distribution for Sumbersari districts and Patrang district were Log Pearson III distribution, while the Kaliwates District was Normal distribution. Design of rainfall for 1, 2, 5, and 10 year return period rainfall for Patrang district was 65.16 mm, 82.27 mm, 102.27 mm and 117.27 mm respectively; Sumbersari in consecutive sub-district is 74.19 mm, 91.32 mm, 100.29 mm and 104.13 mm; and District Kaliwates respectively is 71.87 mm, 100.60 mm, 119.46 mm and 129.32 mm. This indicates that, the sequence of the desaign rainfall depth from high to low in a sequence is Kaliwates districts, Patrang districts, and Sumbersari districts.

MODEL CALIBRATION

In order to ensure the reasonableness of the model, it is necessary to compare the calculated results with the actual flooding spots. The calibration was done by incrementally adjusting the manning parameter. This was determined at16 nodes based on event rainfall depth to compare with the actual flooding area. The results show that the error at Patrang,

Sumbersari and Kaliwates district respectively between 3-8%, and 6-8%. It indicates that the flooding results simulated by the model and the actual situation are relatively consistent, so that the simulation results are basically reliable.

RESULTS AND DISCUSSION

Flood Simulation

The simulated surface runoff, infiltration and precipitation of three districts are shown that the increase of the return period, the total precipitation, the surface runoff, the runoff coefficient, and the infiltration, this is because the studied area has a high pervious area rate. The increase of the highest infiltration value in Patrang District, this is because the studied area has a high procentase of green space particularly in RRJSRD, but there is a drainage network that the decrease of infiltration in BSRD network at Kaliwates district, because this area has a high population density. Infiltration will increase for the green space areas, otherwise in residential areas will decrease.

Junction overflow will cause inundation around the junction area, and is an important index for urban flood forecasting. From the results it was found that with the increase of the return period, the overflow junction number, the overflow volume and overflow duration increase steadily. When the return period is 1 year, 2, 5,and 10 rainfall are overflow. The node flooding is shown in FIG .4 as the red circle, that are found 111noods flooding. The noods flooding with the most number is in Patrang district as many as 46 nodes with high floods ranging between 5-30cm. Futhermore, the second highest noods flooding is in the Kaliwates district as much as 37 nodes with high floods ranging between 10-30cm. The latter is in Sumbersari district as much as 28 points with a high range of flooding between 10-50cm.

The numbers of nodes flooding in Jember urban area, in the analysis of hydrology were caused by the contribution of discharge from irrigation drainage, or overflow dam to the urbanized drainage network, and high population density in urban area. While the analysis of hydraulics were caused by the channel capacity cannot accommodate the appropriate channel function, the slope of the line is too flat in FIG.5, the sedimentation or waste which reduces the capacity of the channel and a small part due to bottleneck. In general, the location of the noodes flooding in the Patrang district more than other districts, although a slope of this district is greater than other districts. This is due to the capacity of the natural channel in the Patrang district not be able to accommodate the desaign flood. Flooding in Kaliwates District was caused by many things, namely the existence of bottlenecks, curves, channel capacity does not accommodate etc. Flooding in the Sumbersari district higher than both other regions due to the channel capacity cannot accommodate, the slope of the line is too flat, there is the sedimentation of the channel, the addition there are a contribution of discharge from irrigation drainage. Based on the fact the field, this is caused by the land conversion of irrigated area into a residential area and a network system that has not been followed by changes in the drainage network system.

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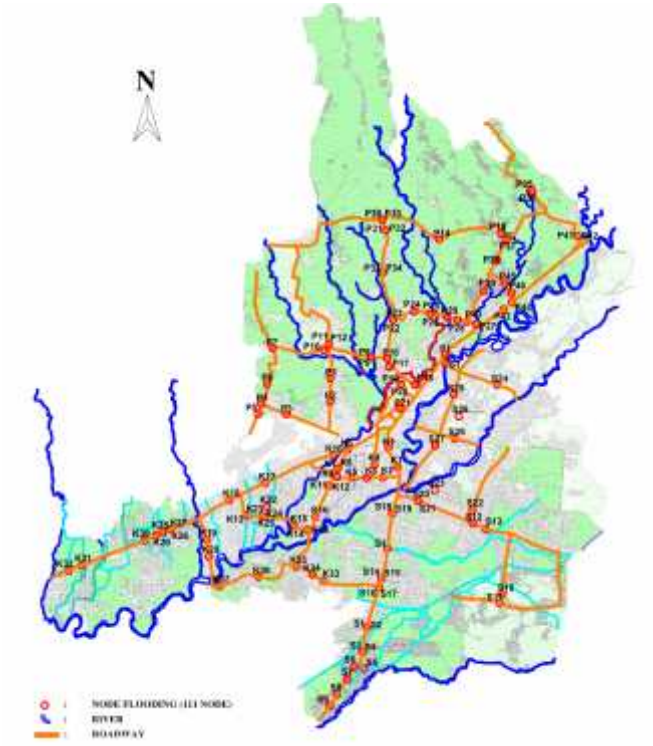
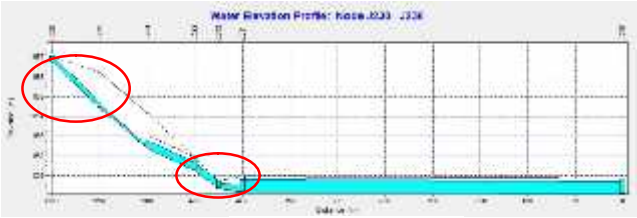
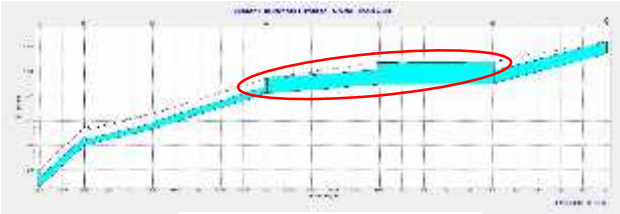


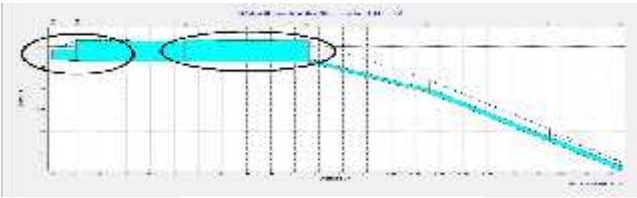
FIG. 4 Location of Nodes Flooding



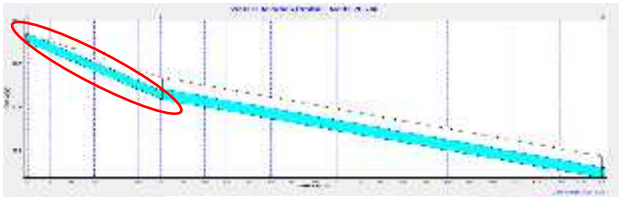
A.1 Flooding in



A.2 Flooding in RR-JSRD



A.3 Flooding in RRD



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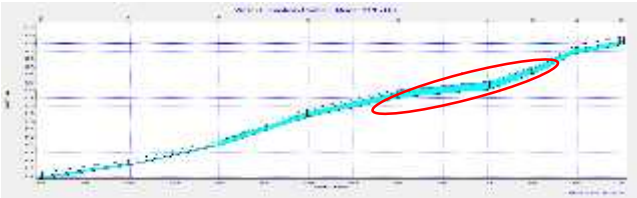
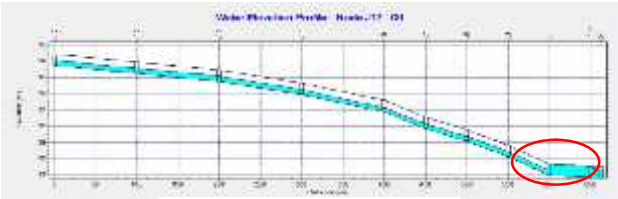


A.5 Flooding in SPPRD



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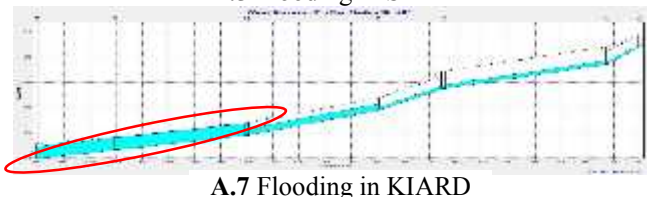
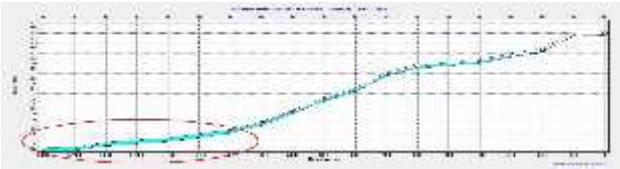
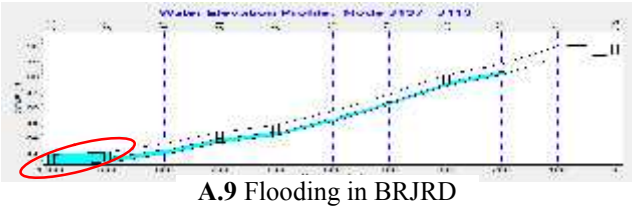


A.7 Flooding in KIARD



A.8 Flooding in CRBRD



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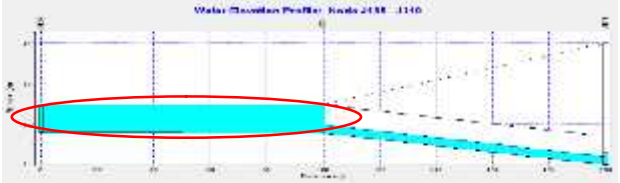
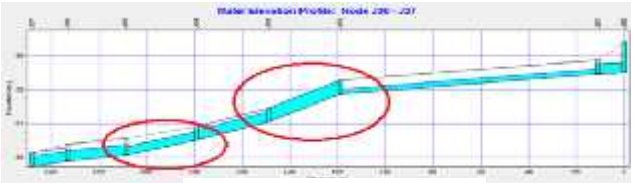
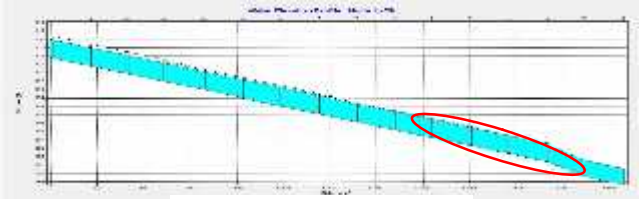


A.10 Flooding in JRARD

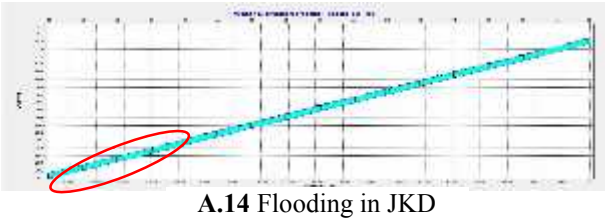


A.11 Flooding in ARSID

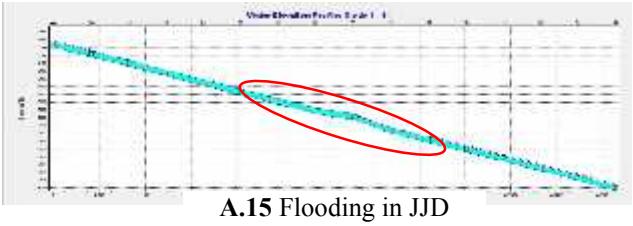
A.12 Flooding in BSRD



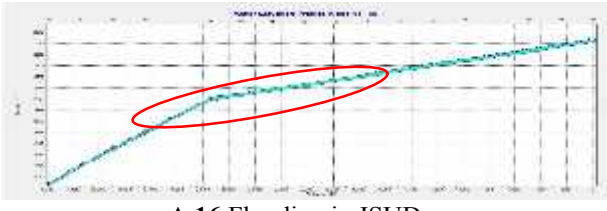
A.13 Flooding in JMD



A.14 Flooding in JKD



A.15 Flooding in JJD



A.16 Flooding in JSUD

FIG. 5. Result of running for 1 year time periode raifall

CONCLUSIONS

The SWMM models have been used to demonstrate the simulation of flow and water surface depth at Jember urban drainage system. The SWMM model was set up based on the DEM, thopography measurement, digital map and underground cannel network, and parameters derived based on the properties of the subcatchments and the storm sewer conduits. The parameter calibration shows the parameter robustness.

The urban flood processes were simulated with different return period design precipitation, and the results show that with the 1, 2, 5, and 10 year return period rainfall, the studied area will be inundated at 111 noods flooding. The numbers of nodes flooding in Jember urban area, in the analysis of hydrology were caused by the contribution of discharge from irrigation drainage, or overflow dam to the urbanized drainage network, and high population density in urban area. While the analysis of hydraulics were caused by the channel capacity cannot accommodate the appropriate channel function, the slope of the line was too flat, the sedimentation or waste which reduced the capacity of the channel and a small part due to bottleneck.

ACKNOWLEDGEMENTS

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ROOT STRENGTH OF UNDERSTORY VEGETATION FOR EROSION CONTROL ON FOREST SLOPES OF KELARA WATERSHED, INDONESIA

Hasnawir[1], Laura Sanchez-Castillo[2], Tetsuya Kubota[3] and Andang Suryana Soma[4]

ABSTRACT: Soil erosion is one of the most serious environmental issues in Indonesia. The watershed soil erosion is one of the biggest concerns when our objective is to circumvent erosional processes. This demand us an increasing on the demand of the application of vegetative measures for erosion prevention and other environmental conservation. It can be understood that for some cases the application of vegetative measures for erosion control are more effective because they are ecologically friendly, lower in cost and easy in techniques compare to engineering control. The objective of this study is to find out the pull out tensile resistance force of understory vegetation species suitable for erosion control in forest slopes. The study is conducted in Kelara watershed, South Sulawesi, Indonesia. In order to investigate the pull out resistance force, thirty plant species with different sizes were randomly chosen. Before carrying pull out tests, soil surface around the plant was carefully cleared from litter exposing stem base. One end of a strong string was tied around the stem base of the plant and another end was connected to a hand-held portable Push Pull Digital Gauge AIKOH Model RX-100. The results showed that the species deep rooted with lateral spreading have greater pull out resistance force and are considered to be suitable species for erosion control and to decrease the negative impacts on forest slopes, such as surface erosion or shallow landslides in the study area.

Keywords: *Soil erosion, forest slope, understory vegetation, erosion control, root strength*

INTRODUCTION

The role of vegetation for site restoration, slope stabilization and control erosion is widely investigated and discussed. Studies have documented the effect of root density in preventing landslides (Abe, 1997; Genet et al., 2008; Gray and Sotir, 1996), the relationships between soil water and root profile (Normaniza and Barakban, 2006) as well as the root system development on different sloping conditions (Di Iorio et al., 2005; Chiatante et al., 2003,). Roots increase soil shear strength by anchoring a soil layer and by modifying soil structure (Greenway, 1987; Schmidt et al., 2001; Waldron, 1977; Waldron and Dakessian, 1981). The soil influences the micromechanical properties of the roots at tissue and fiber level (Burgeret et al., 2004; Krämer et al., 1996; Pan Chun and Nii, 2005) and at system level (De Baets et al., 2008) modifying the root architecture and the root anatomical development.

The effective roots contribution can be measured in two ways: through the soil blocks shear strength or through the root tensile strength and the root architecture. Shear test of soil root has been performed to study the mechanical reinforcing effect of plant roots on the stability of slopes (Normaniza et al., 2008; Wu and Watson, 1998; Ziemer, 1981) and stream banks (Simon and Collison, 2002; Pollen, 2007). Root tensile strength can be measured by in situ root pull-out tests (Burroughs and Thomas, 1977; Docker and Hubble, 2008; Norris, 2005; Norris et al., 2006, 2008; Riestenberg and Sovonick-Dunford, 1983;) and by laboratory root tensile tests (Abe and Ziemer, 1991; Bischetti et al., 2009; De Baets et al., 2008; Genet et al., 2008; Mattia et al., 2005; Tosi, 2007; Riestenberg, 1994;). Roots can respond to shearing force in three different ways: stretching, slipping and breaking (Abe and Ziemer, 1991; Waldron and Dakessian, 1981). Since such response depends on species and on growing condition, there is a high degree of variability and, as expressed by Mattia et al. (2005), a great deal of data collection is still needed. Biotechnical characteristics (correlation between mean values of root strength and root diameter) of Lygeum spartium, Atriplex halinus and P. lentiscus have been tested (Mattia et al., 2005). Bischetti et al. (2005) tested eight typical Northern Italy species: Alnus viridis Fagus sylvatica, Salix purpurea, Salix caprea, Corylus avellana, Fraxinus excelsior, Picea abies and Larix decidua. Operstein and Frydman (2000) conducted root tension test on four different Mediterranean plants: Medicago sativa, Rosmarinus officinalis, P. lentiscus and Cistus sp.; however, there are several other investigations that could be conducted to improve the understanding of soil reinforcement generated by roots. In this study, an attempt is done to know the characteristics of the root strength of understory vegetation for erosion control on forest slopes of Kelara watershed, Indonesia.

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STUDY AREA

The study area is located in Kelara watershed in South Sulawesi, Indonesia (Fig. 1). The watershed area is between 05°32'71" and 05°70'06" South latitude and between 119°72'96" and 119°89'07" East longitude. In general, the configurations of the watershed areas are ranging from flat to steeply sloped areas. There are two distinct seasons i.e. dry and rainy. The dry season is observed from April through October with about 25% of annual rainfall. On the other hand, the remaining 75% of annual rainfall occurs in November through March, and attains the maximum due to the Monsoon. The annual rainfall is about 1,056 mm/year in the downstream of watershed and 2,952 mm/year in the middle to upstream of watershed. The geology of sites in Kelara watershed was composed of conglomerate, lava, breccia, mudflow deposits and tuff with soil type is andosol.

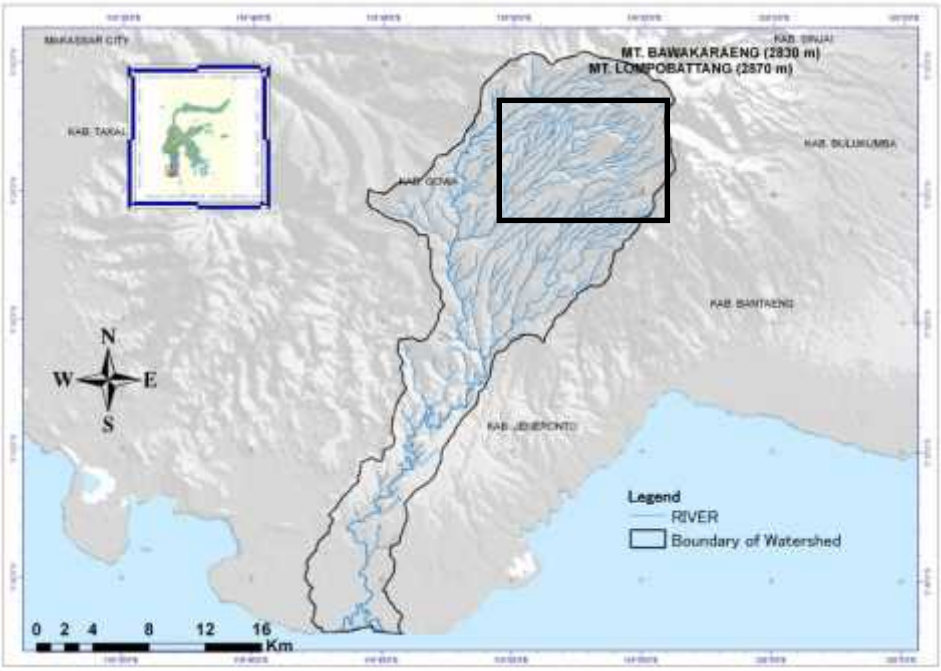

Fig. 1 Study area in Kelara watershed of South Sulawesi, Indonesia

METHODOLOGY

Vegetation Investigation

The condition of naturally colonized vegetation on forest slopes was examined in July 2011, by field investigation. Six quadrats of 5 x 5 m^2 size are established (Fig. 2). In all quadrat plots, thirty plant species with different sizes were randomly identified, counted and measured to investigate the quantitative vegetation parameters. The above and belowground morphological characteristics of plants like stem height, root numbers, rooting depth were identified from extracted plants. All the root tendrils of each species was counted manually, maximum diameter and lengths were measured by caliper and measuring tape, respectively. The roots were separated from the stem by using sharp blade. The soil on the root was separated carefully by hand wash and then was placed inside the room ten days.

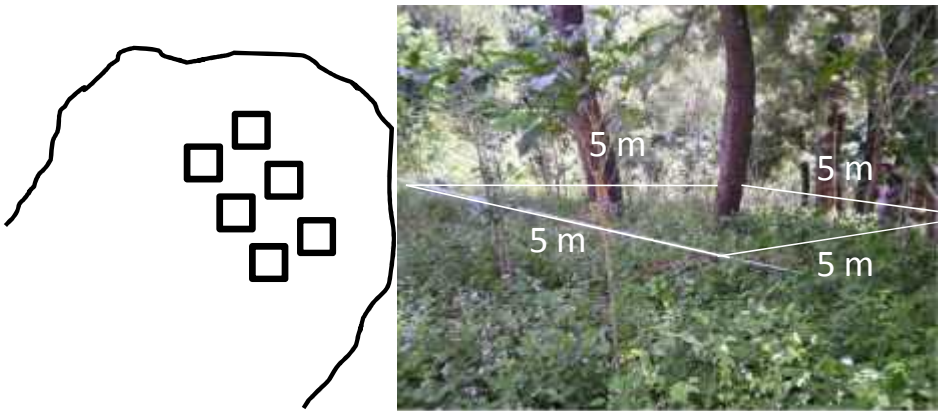

Fig. 2 Lay out position of sampling quadrats and view of a quadrat plot

Pull Out Resistance of Understory Vegetation

The uprooting resistance force provides significant information on role of root hairs played in anchorage (Bailey et al., 2002). When a plant is pulled vertically from soil, force is transmitted to the root system, the root-soil bond and the strengths of the roots themselves in tension. A single vertical force applied to a stem will be transmitted to numerous roots, either because of lateral branching or because of adventitious roots from the stem base (Enno, 1993). In order to investigate the pull out resistance, thirty plant species with different sizes were randomly chosen. Before each pull out test soil surface around the plant was carefully cleared from litter exposing stem base. One end of a strong rope was tied around the stem base of the plant and another end of the rope was connected to a hand-held portable Push Pull Digital Gauge AIKOH Model RX-100 as shown in Figure 3.

The vertical force was applied manually till plant was uprooted successfully (without mechanical damage to the root systems). In some tests, where roots were broken during extraction, tests were repeated. The peak force, at which were just started to break from the soil were recorded from the digital pulling gauge.

During uprooting, it was revealed that the resistive force was influenced by morphological characteristics such as shoot height, total numbers of roots present and root size of an individual species. The co-relationship between the pull out resistance with morphological characteristics (shoot height, length of root, number of roots, and diameter of root) were investigated for four dominant understory vegetation.

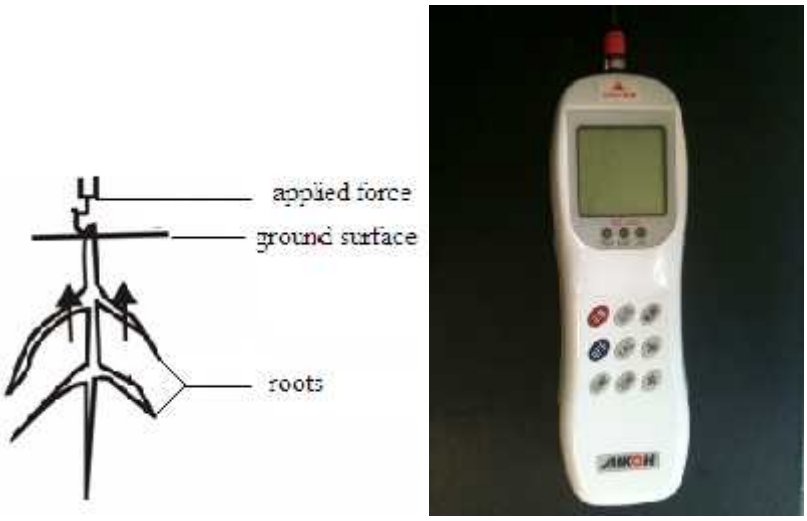

Fig. 3 Illustration of pull out test and instrument used for uprooting a plant (Digital Gauge AIKOH Model RX-100)

Measure Permeability of Soil

Permeability, *k* (cm/sec.) of the collected soil was determined using the following basic equation 1 and simplified equation 2. For experiment, two sample cans having same size were taken and tested as shown in Figure 4. Two cans were tied over each other containing soil and water and permeability was determined by taking change in time to drain all water from the upper can (Das, 1997):

$$k = 2.3 \ \frac{aL}{A(t_2 - t_1)} \ \log_{10}(\frac{h_1}{h_2}) \tag{1}$$

where, $\frac{h_1}{h_2} = 2$ and $a = A$, $L = 5\,\text{cm}$, $t_2 - t_1 = \Delta t$, *a* and *A* are the cross sectional area of the upper and lower cans, t_2: ending time (sec) and t_1: starting time (sec). The Equation is modified by substituting values in equation 1, permeability can be estimated as,

$$k = \frac{3.46}{\Delta t}. \ . \tag{2}$$

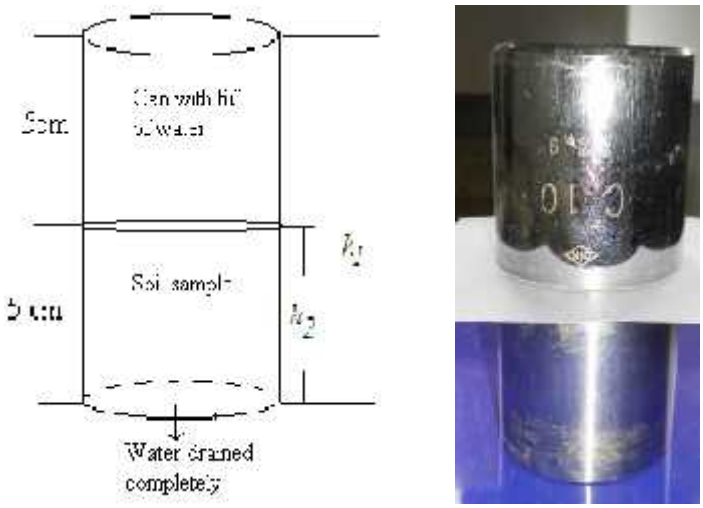

Fig. 4 Methods applied to measure permeability of soil

RESULTS

Soil Permeability

The soil in the eroded sites might have formed through the interaction of many factors such as parent material, climate, time, topography or relief, and activities of microorganisms. The relative influence of each factor depends on geographical conditions and locations, but the combination of all factors normally determines the kind of soil and thickness developing in any location. The soil permeability were examined with the results are shown in Table 1. The soil permeability has ranged from 0.0090 cm/sec to 0.0098 cm/sec.

Table 1. Soil permeability with 6 sites in Kelara watershed, Indonesia

Site Number	Permeability (cm/sec)
1	0.0096
2	0.0095
3	0.0090
4	0.0098
5	0.0094
6	0.0096

Morphology Characteristics

The morphological characteristics such as shoot length, length of root, number of roots of an individual plant, and uprooting peak resistive force are summarized in Table 2. On average, the pullout strength of plants observed in the study sites were ranged from 38.0 N to 75.9 N plants like length of root, number of roots, shoot height, and diameter of roots.

Table 2. Morphological characteristic of understory vegetation

No	Species	Length of root (cm)	Number of roots	Shoot Hight (cm)	Diameter	Pull out strength (N)	Types of root
1	*Melastoma malabatricum*	6.5	6	35.5	3.3	38.0	Taproot
2	*Ageratum conyzoides*	7.2	5	40	3.3	58.0	Taproot
3	*Dicranopteris linearis*	16.5	9	35.6	3.6	51.9	Fibrous roots
4	*Eupatorium odoratum*	7.2	7	44.6	3.4	75.9	Taproot

The correlations analysis between the pullout strength with morphological characteristics was investigated for four dominants species in site and the results are shown in Figure 5. For all the species, positive co-relationships were found between the pullout strength and all measured morphological parameters. In general taller plants with greater rooting depth have shown higher pull out strength. The plants with spread root system were able to resist uprooting better than

the plants with root system that do not reach far from the stem base. The entire morphological views of some extracted plants are shown below in Figure 6.

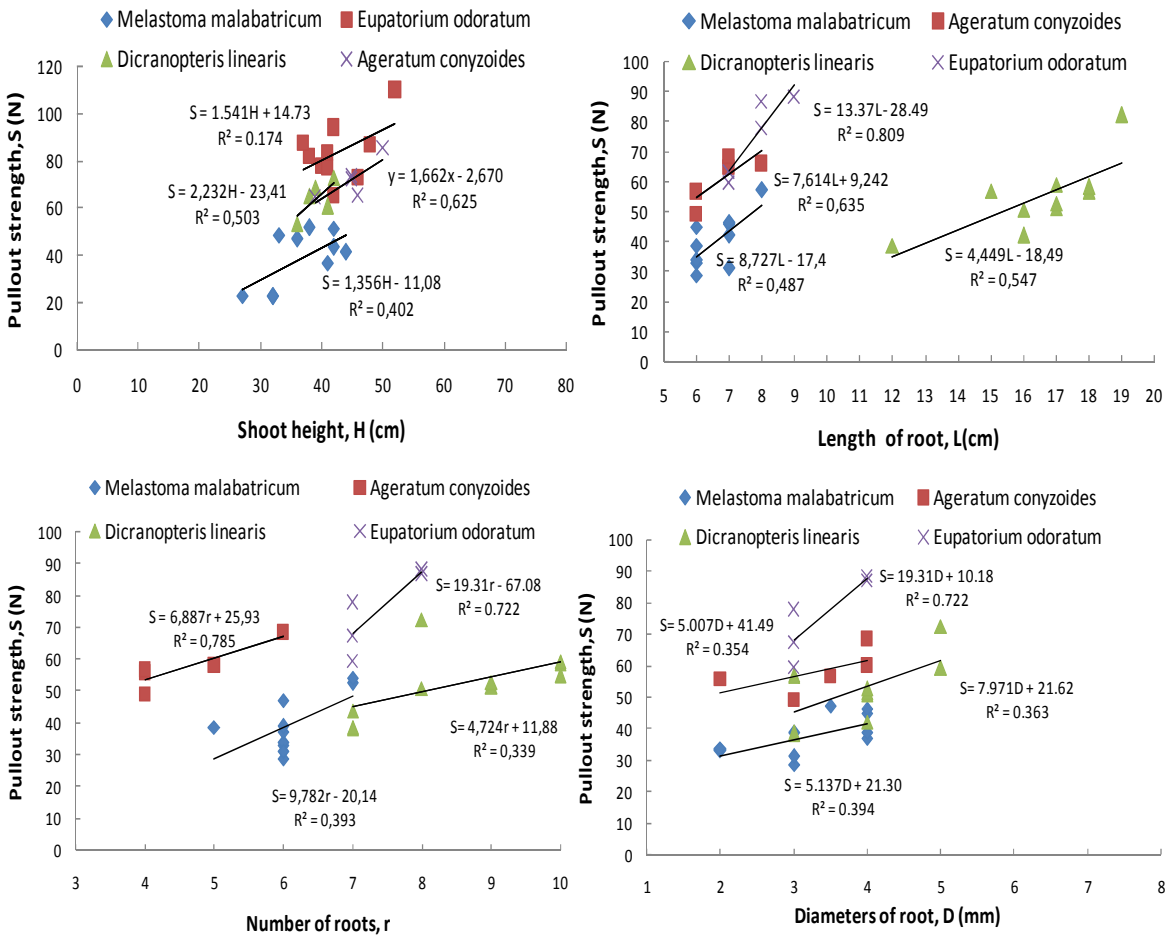

Fig. 5 Co-relationships between plant morphological features and pullout strengths

Fig. 6 Morphological view of some extracted plants: 1. *Ageratum conyzoides*, 2. *Dicranopteris linearis*, 3. *Eupatorium odoratum* , 4. *Melastomata malabatricum*

CONCLUSION

The pullout resistive strength of the plants is next important steps in examining strength of an individual species in erosion processes. In this research, we examined pull out resistive force for 4 plant species. From data, it was revealed that pull out resistive force was jointly determined by the combined morphological features of the plants like shoot height, roots morphology. The species with deep rooted with lateral spreading have shown greater pull out resistance

and are considered to be suitable species for erosion control and to decrease the negative impacts on forest slopes, such as surface erosion or shallow landslides in the study area. The selection of plant for erosion control measures might be effective if selection is based on considering criteria such as; 1) locally available plants that can grow successfully on eroded sites, 2) dense and deep rooting system which can add strength to the surface soil layers, 3) fast and simple propagation.

ACKNOWLEDGEMENTS

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HYDRODYNAMIC MODELING OF JENEBERANG ESTUARY

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ABSTRACT: Jeneberang Estuary, located south of Makassar, Indonesia, is one of the largest and most important river in Sulawesi, with anthropogenic changes, mainly related to flood control and water supply. A hydrodynamic model of the Jeneberang estuary was established by applying the ECOMSED modeling. The hydrodynamic model is derived from the hydrostatic assumption and Boussinesq approximation. A high-resolution computational grid was generated covering the Jeneberang estuary. The model was run for 30 days driven by tidal forcing at the ocean boundary and river hydrograph at the upstream. The observed tidal data and hydrography were accessible for the set up of the model. Hydrodynamic simulations have been performed and computed water levels were compared to observations of existing water level gauges along the estuary. An extensive model calibration was needed to adjust model configurations and uncertain parameters to optimize the matching of the model with observations. For the period of a neap-spring-neap cycle, the model settings determined in the calibration process are verified satisfactions with respect to water level measurements. Good agreement was found between model results and observed temporal and spatial variations in water elevation and currents, in the Jeneberang Estuary. The configuration of the hydrodynamic model of the Jeneberang estuary will be applied as a basis for further hydrodynamical investigations of historical topographic states and for morphodynamical computations.

Keywords: *Hydrodynamic model, ECOMSED, grid, estuary*

INTRODUCTION

Jeneberang Estuary is located in the southern of Makassar, Indonesia. The Jeneberang River flows from east to west across the province, is one of the largest and most important river in Sulawesi (Fig.1). Between these river mouths, the coastal of Jeneberang estuary had developed. The coastal area included Barombong Beach in the south to the Tanjung Bunga at north. The Estuary Jeneberang coast is very dynamic, natural, accumulation sediment from fluvial, coastal, marine processes and human activities. An active delta system which has been formed in humid tropical environments under condition of relatively tidal amplitude, low wave-energy, and fluvial input.

Originating from Mt. Bawakaraeng (2,833 m), it flows to the Makassar Strait. The river is 90 km long with a catchment area of 727 km2. There are two reservoirs, which have constructed in the catchment, the Bili-bili reservoir located on the Jeneberang River, holds 375,000,000 m3 water flow and the Jenelata Reservoir located on the tributary Jenelata River. In the rainy season (October-April), floods are normally caused by rainstorms, and often short floods are seasoned. There is a productive land for paddy rice and the area under irrigation area is 17,400 ha. The Jeneberang river mouth had formed a delta, which divided the river into two river mouths, the north river mouth and the south river mouth (Fig. 2).

Improvements Lower Jeneberang River work covered dredging, excavation, embankment and revetment constructions for a 9.6 km section between the mouth of the Jeneberang River and the Sungguminasa Bridge. The project was to conduct river improvements for the Jeneberang River, to provide flood protection of Makassar. [2]. Development activities that have been conducted in the Jeneberang to support the growth of Makassar include: River Improvement (1993), Long Storage / Closure North Estuary (1994), Jeneberang Rubber Dam (1999), and Bili-Bili Multi Purpose Dam (1999). Beside to reduce flood damage, The Bili-billy Dam also stabilize supplies of raw water by installing a raw water transmission main to the Somba Opu water treatment plant, leading to the response to water demand from Makassar and the surrounding area (JICA, 2001). Jeneberang Estuary is in the western part of the southern coast of the island of South Sulawesi, which consists of tidal areas, fluvial and river delta areas that are low slope (<2%) and is covered by alluvium and coastal sediments.

The scope of this study is to implement and improve the hydrodynamic numerical modeling system for estuarine zone. The model used is fully integrated three dimensional (3-D), time dependent, hydrodynamic, and sediment transport. This module of ECOMSED (Estuarine Coastal and Ocean Modeling System with Sediment), developed by HydroQual, Inc., (2002), has been successfully applied to the coastal and estuarine waters. The development of

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ECOMSED was originated in the mid 1980's with the creation of the Princeton Ocean Model (Blumberg and Mellor, 1987). Some recent applications of the module include Chesapeake Bay (Blumberg and Goodrich, 1990), Delaware Bay and Delaware River (Galperin and Mellor, 1990), the Gulf Stream Region (Ezer and Mellor, 1992). Mandang and Yanagi (2008) well reproduced the tides and tidal currents with the use of a horizontally two-dimensional barotropic hydrodynamical model ECOMSED which was applied to the Estuary of Mahakam, East Kalimantan, Indonesia.

The objective of this study is to identify the circulation pattern of water flow in the Jeneberang Estuary.

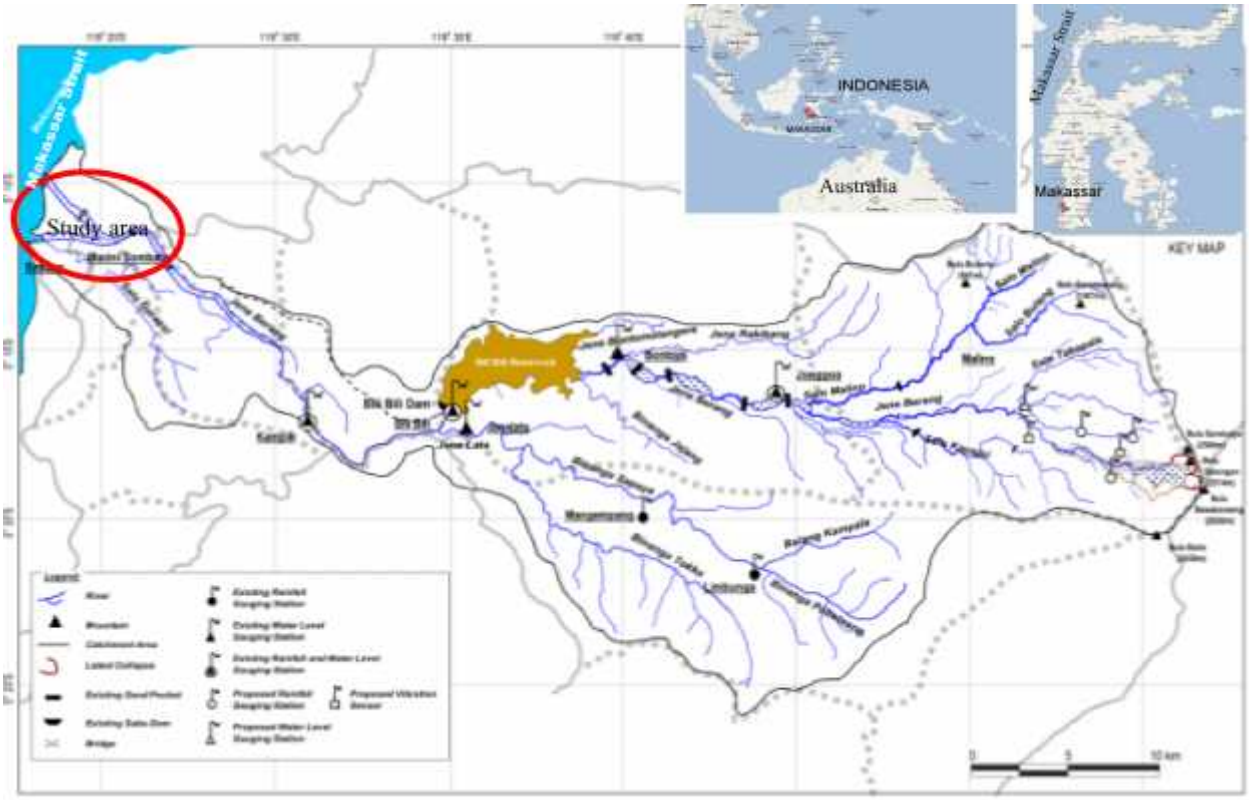

Fig. 1 Jeneberang River watershed

Fig. 2 Study area (Google earth image 2015)

MODEL DESCRIPTION

The hydrodynamic module, ECOM described here is a three-dimensional, time dependent model developed by Blumberg and Mellor (1987). This module of ECOMSED, has a long history of successful applications of oceanic, coastal and estuarine waters. The equations which form the basis of the circulation model describe the velocity and surface elevation fields. Two simplifying approximations are used (Blumberg and Mellor (1987).; first, it is assumed that the weight of the fluid identically balances the pressure (hydrostatic assumption), and second, density differences are neglected unless the differences are multiplied by gravity (Boussinesq approximation). The governing equations of the hydrodynamic component in the model are the continuity and momentum equations. The hydrodynamic part of the model uses the following basic equations. The basic equations for the three-dimensional mode are:
The continuity equations:

$$\frac{\partial U}{\partial x}+\frac{\partial V}{\partial y}+\frac{\partial W}{\partial z}=0 \tag{1}$$

The momentum equations:

$$\frac{\partial U}{\partial t}+U\frac{\partial U}{\partial x}+V\frac{\partial U}{\partial y}-fV=-\frac{1}{\rho_0}\frac{\partial P}{\partial x}+\frac{\partial}{\partial z}\left(K_m\frac{\partial U}{\partial z}\right)+F_x \tag{2}$$

$$\frac{\partial V}{\partial t}+U\frac{\partial V}{\partial x}+V\frac{\partial V}{\partial y}+W\frac{\partial V}{\partial z}+fU=-\frac{1}{\rho_0}\frac{\partial P}{\partial y}+\frac{\partial}{\partial z}\left(K_M\frac{\partial V}{\partial z}\right)+F_y \tag{3}$$

$$\rho g=-\frac{\partial P}{\partial z} \tag{4}$$

where (U,V,W) are the eastward (z), northward (y), and upward (z) components of the current with the ρ_0 reference density, the in situ density, g the gravitational acceleration, P the pressure, K_M the vertical eddy diffusivity of turbulent momentum mixing. A dynamic boundary condition evaluated at the sea surface $z=\eta$ will indicate the relation between sea surface elevation h and vertical velocity at the sea surface W_η as,

$$\frac{\partial \eta}{\partial t}+\frac{\partial \eta}{\partial x}U_\eta+\frac{\partial \eta}{\partial y}V_\eta=W_\eta \tag{5}$$

DESIGN AND RESULT

Model Grid

Raw bathymetry data contain in XYZ format, obtained in two different fields, topographic surveys (December-2015) and Bathymetry data of the ocean domain were obtained from DISHIDROS (Indonesian Navy Hydrographic Department) of Indonesian Navy. The model domain in this study covers the area of Jeneberang, (Fig. 3).

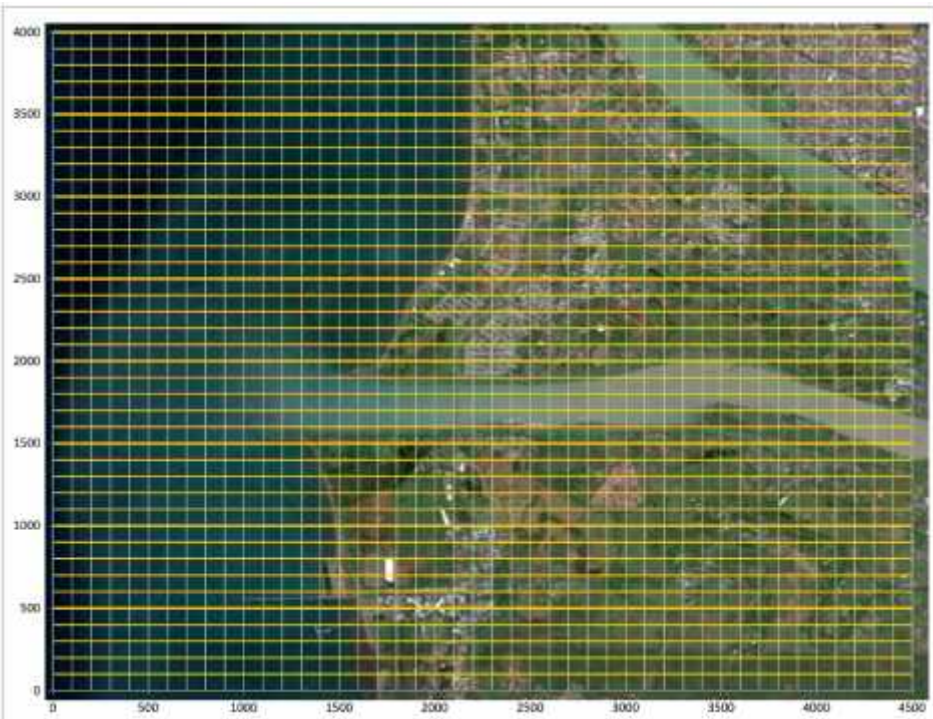

Fig. 3 Grid model of Jeneberang Estuary

The spatial extent of data is approximately 4000m × 4500 m. The data points contain elevation of the sea, stretching 4500 m seaward and river floodplain. The elevation generally ranges from 0 m up to -10 m. Based on the elevation data collected, a cartesian grid of the model was set up with with 45 × 40 grid cells, sized 100 m × 100 m. The x axis is pointing to the easts (ocean to landwards). A cell size of 100 m is coarse compared two sizes of the geomorphological features of the estuary, which are are in the order of 100 m. However, for preliminary calculation, it would suffice. First the XYZ file is transformed into a point-figure file with elevation as the property of each point. The result grid is saved as an ASCII file whose file name to be declared in the variable model.

Hydrodynamic boundaries

The data needed to model the Jeneberang Estuary are the bathymetry, the tidal forcing at downstream open boundaries, the river discharges at the upstream boundaries and the wind forcing. The boundary condition along the open boundary, was given as the tidal elevation.

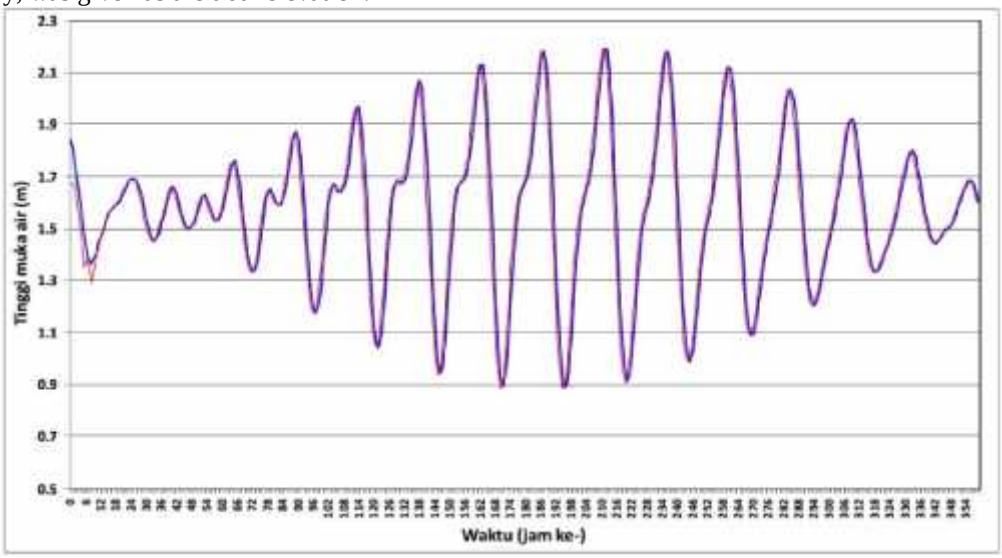

Fig. 4 Water elevation data at the Jeneberang mouth

The river discharge at the upstream boundary in December 2013 was given at the Jeneberang Rubber Dam. The averaged river discharge data of the Jeneberang Rubber Dam (2003–2013) obtained from the Balai Sungai Jeneberang Pompengan, are shown in Fig. 5. As shown in the figure 5, the maximum and averaged daily discharge of Jeneberang River in the period of 2003 to 2013 was relatively constant in between 99.39 - 100.01 m^3/s; while the the minimum daily discharge was decreasing with time and attained the lowest level in 2011, which was 75.22 m^3/s.

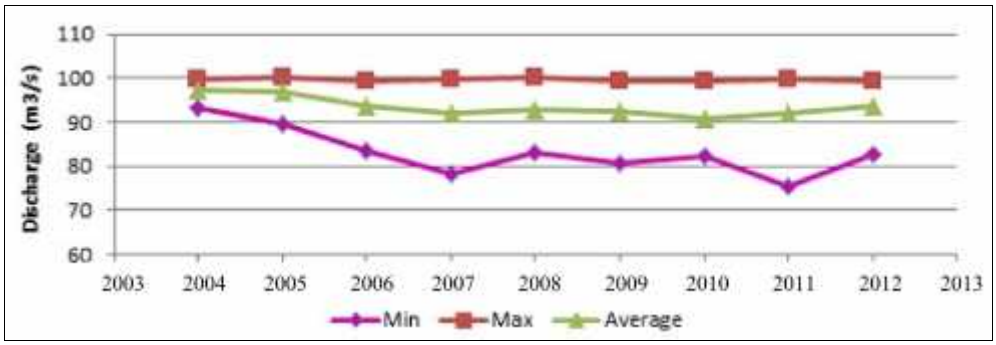

Fig. 5 River discharge data of the Jeneberang River

By introducing a river discharge, we have to set up the upstream flow boundary and accounting for the influence of the inflow water to the system.

Calibration

The first part of the calibration is to demonstrate the model's capability for reproducing the time variable water elevations in the ocean and rivers. Water level calibration is fundamentally important because it demonstrates that the model bathymetry, geometry and hydraulics are configured correctly. The hydrodynamic was calibrated with available

data from 1 December to 31 December 2013. The hydrodynamic model was calibrated with water level data in the Jeneberang mouth. The calibration of the model is based on the comparison of modelled and observed temporal series of free surface elevation. The calibration consists in adjusting the Manning coefficient in order to represent the tidal signal correctly. The comparison of the tidal part of the observed and modelled elevation is rather easy if it's based on harmonic decomposition.

Figure 4 shows the time series of the observed and modelled elevation. The spring-neap tidal cycle is clearly visible and well represented by the model. A constant value of 0.022 was used in the ocean domain. Then, the applied Manning coefficient decreases linearly to a value of 0.017 at the upstream part of the estuary. This variation takes into account in a simple way the gradual transition between marine and riverine environments. The value of 0.017 was also chosen for the Manning coefficient in the riverine part. The ability of the hydrodynamic model to simulate hydrodynamic processes in the Jeneberang Estuary was assessed by comparing model output with observed data.

The vertical eddy diffusivity coefficients of momentum are obtained through the 2.5 level turbulence closure scheme developed by Mellor and Yamada (1982). The horizontal mixing coefficients used in the calibration were adjusted using a non-dimensional parameter, which ranged from 0.1 - 0.2 in the rivers and ocean.

A minimum bottom friction coefficient of 0.0024 (nondimensional) and bottom roughness coefficient of 0.003 meters was used throughout the model domain for the calibration.

Hydrodynamic Result

The hydrodynamic model realistically reproduces observed water levels with measurements in the Jeneberang Rivers mouth. Based on the good model calibration to observed water level and velocity data, the overall water circulation in the model domain is well characterized. Figure 6 shows the time series of the observed and modelled elevation. The spring-neap tidal cycle is clearly visible and well represented by the model.

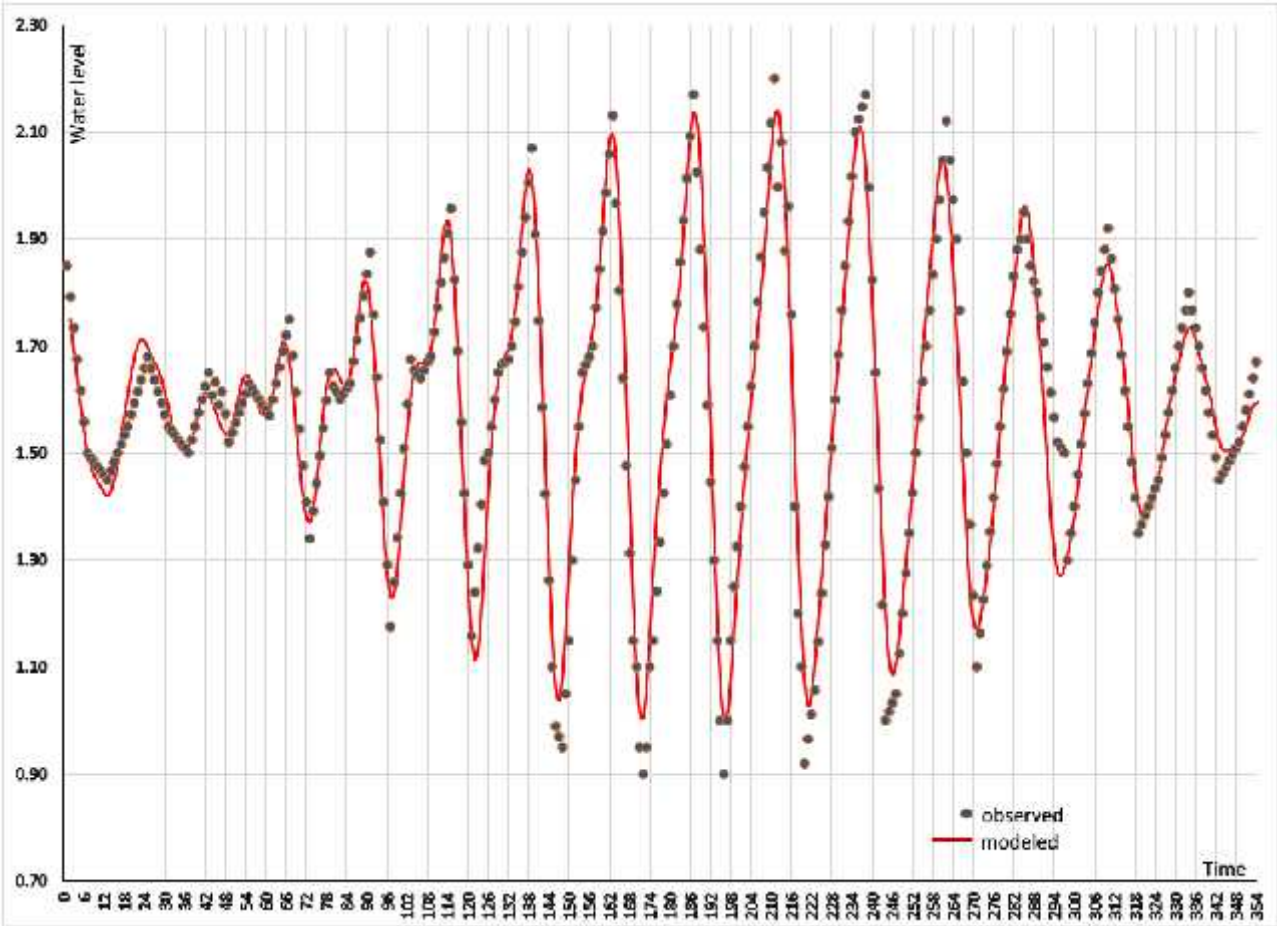

Fig. 6 Water level observed and modelled

Fig. 7. Spring tide flow condition

Fig. 8. Neap tide flow condition

Fig. 9 .Low tide flow

The fluctuations in sea level simulation results will be followed by the periodic movement of water masses (Hatayama et al., 1996) as shown in Figure 6. Water level verification is done by comparing simulation results with the results of measurements of field data. In general, the results of simulation models can represent quite well the pattern of water level in the waters of Makassar Coast (Figure 6).

The three plots show the pattern of current and direction, when the spring tide, neap and at low tide. Mass flow pattern of water in spring tide conditions against the tide dominated by the flow moving toward the north turn and then flows east toward the shoreline (Figure 7). The model also shows the tide into the Jeneberang River during peak tide. At the neap tide, the flow pattern determine move from the south to the north. At the time of low tide, the current pattern of moving from the river into the sea (Figure 9). In addition, the flow around the river mouth is also appear to have currents moving north, with a pattern away from the shoreline. Tidal flow patterns on the neap tides and low tide condition showed the flow is dominated by a movement towards the north (Figure 8).

CONCLUSION

ECOMSED models have described the hydrodynamics of the Jeneberang Estuary. The hydrodynamic model realistically reproduces observed water levels with measurements in the Jeneberang Rivers mouth. The spring-neap tidal cycle is clearly visible and well represented by the model. Good agreement was found between model results and observed temporal. There is a difference between spring and neap tide conditions. The velocity magnitude in spring tide is larger and shows the tide into the Jeneberang River during peak tide.

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MEASURING OCEAN WAVE POTENTIAL ENERGY USING DIGITAL VIDEO PROCESSING SOFTWARE

(Case Study West Coast of Kab. Pinrang, South Sulawesi Indonesia)

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ABSTRACT: Wave energy potential in coastal area may be varies based on environment characteristics and ocean wave's parameters. To date there is limited number of technology that is easy to use to quantify the wave parameters such as wave height and period. In this work an analytical method was introduced by means of digital video processing software that describes a simple and accurate way of on site wave parameters measurements. This method has been implemented using Adobe Premier Pro digital video processing software at Kajuangin Port, west coast of Pinrang Regency, South Sulawesi Indonesia. The measurement result showed that wave height variation between 0.56 – 0.85 meter and wave period 4 – 6 seconds. Simulation results using MATLAB based on linear wave theory shows that the potential waves energy in this area are 0.3 – 1.6 kW.

Keywords : *wave energy, ocean waves, digital video processing*

BACKGROUND

The Indonesian archipelago consists of 18,000 islands. Among 3000 islands are inhabited islands (Wikipedia, 2015). Thousands of islands that are mostly small islands surrounded by oceans. The people who inhabit these small islands facing difficulties to access the national electricity grid. On the other hand, as an archipelagic country Indonesia posses the second longest coastal area in the world about 54 million km (Briney, 2015). These coastal areas possess a huge potential renewable energy sources such as tidal, waves, salinity gradient and ocean current energies (Carbon Trust, 2015). However, there is no adequate information about the characteristics of green coastal energy sources such as ocean wave's data in certain areas along the coast line. Therefore, on site low cost measurement is needed to asses the characteristic of waves before deciding to install suitable wave energy converter in those areas.

Remote sensing image processing technology for the measurement of oceanic sea states has attracted renovated attention recently.Currently, there are several ways to measure the wave parameters such as Sea State Estimated by Observers at Sea, Satellite Altimeter, Synthetic Aperture Radars on Satellites, Accelerometer Mounted on Meteorological or Other Buoy and Wave Gages (Stewart, 2005) (Legresy, 2010) (Cervenka & Moustier, 1993). Though, these techniques need specialized and expensive equipment to acquire the ocean waves parameters. These method can provide ocean wave characteristics, however, the main constraint is cost and availability of such equipment if needed in remote areas for instance in small islands.

To cope the flaws of existing measuring techniques of ocean wave parameters, this work proposed a new low cost wave parameters measurement techniques using digital video processing technology. This method has been applied to measure the wave height and period using portable digital pocket camera and Adobe Premiere Pro 2.0 as a digital video processing software (Adobe, 2013).

METHODOLOGY

Experimental Steps

The implementation of proposed measurement technique was conducted by means of several steps. The first step was setting up the equipment and tool required to measure the significant wave parameters (wave height and period). Second step was recording the wave for a period of time using digital video camera. Third step was analyzing the digital recorded video using digital video processing software package to calculate the wave height and period based on time line recording. Finally, the extracted wave parameters were use to calculate the predicted wave energy potential by means of wave energy formula Equation (1).

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Wave parameter that determine the energy flux of the ocean waves need to be identified before deciding to measure these parameters based on ocean wave linear theory (Thomas, 2015).

$$P = \frac{\rho g^2 T H^2}{3\ \pi} \qquad W \quad /m \tag{1}$$

Where:

ρ = Sea water density 1.025 kg/m3,
g = Gravity acceleration 9,8 m / s,
T = Wave period (s)
H = Wave height (m)

Based on ocean wave's linear theory, equation (1), the amount of wave power depends of wave height and wave period. However, wave height is the most influenced factors that define its energy density. Therefore, this work main concerns are measuring the wave height and wave period.

Wave Parameter Measurements

1. Tools and equipment

Implementation the proposed measurement method of ocean wave's parameters is realized by means of several tools as follows;

a) Digital Video Camera
b) Digital Computer
c) Digital Video Processing software
d) Computing software(Matlab)
e) Roll Meter
f) GPS

2. Setting Up Wave Height and Length Measurement

Before conducting wave height measurement the required tools need to be set up properly. In this experiment roll meter was set up on a vertical direction at a fixed point pole Figure (2.1). This roll meter was use to indicate the waves height. The next steps in wave height measurement was recording the waves that pass over the vertical roll meter using digital video recorder. Measurement set up of wave period was shown in Figure 2.2. Where the roll meter was set up horizontally perpendicular to the incoming wave direction.

Figure 2.1. Wave height measurement set up

Figure 2.2. Wave period measurement set up

VIDEO ANALYSIS AND DATA ACQUISITION

The recording was conducted at Kajuangin Port Desa Kajuangin Kec. Lembang Kab. Pinrang, South Sulawesi Indonesia on June 1st 2015 at 11.00 + 8 GMT, using Digital Pocket Camera 30 frame per second (fps) with resolution of 640 x 640 pixel. The recorded video was analyzed using Adobe Premiere Pro Software package. By moving the time line tools in the package the parameter or of ocean waves such as height and period is extracted using the roll meter length as a background reference. The analysis of wave length and height using Adobe Premiere Pro 2.0 interface is shown in Fig. 2.3 and the result was presented in Table 3.1.

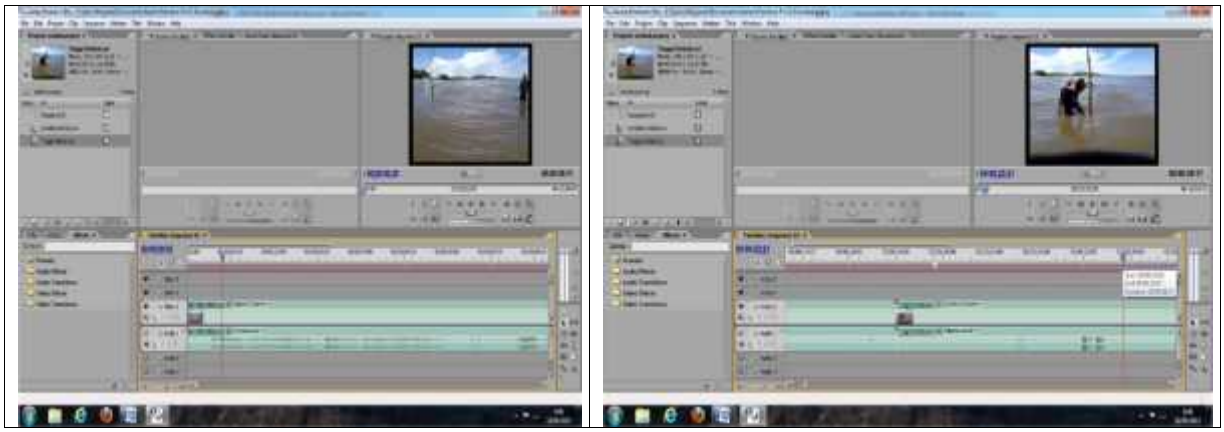

Figure 2.3. Wave length and height video analysis using Adobe Premiere Pro 2.0

Table 1: Measurement result based on recorded video

No Video	Wave Period (Sec)	Wave Height (m)
1.	4	0.8
2.	4.1	0.7
3.	5	0.8
4.	6	0.75
5.	4	0.56
6.	4.5	0.8
7.	5	0.7
8.	5.6	0.8
9.	4.4	0.75
10.	4.1	0.85

Wave energy potential of the observed data on Table (1) can be predicted using shallow water wave energy formula (Wikipedia, 2015). The calculation result using MATLAB was presented in fig 2.4.

$$P = \frac{\rho g^2}{64\pi} H_{m0}^2 T_e \approx \left(0.5\ \frac{k}{m^3.s}\right) H_{m0}^2 T_e, \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (2)$$

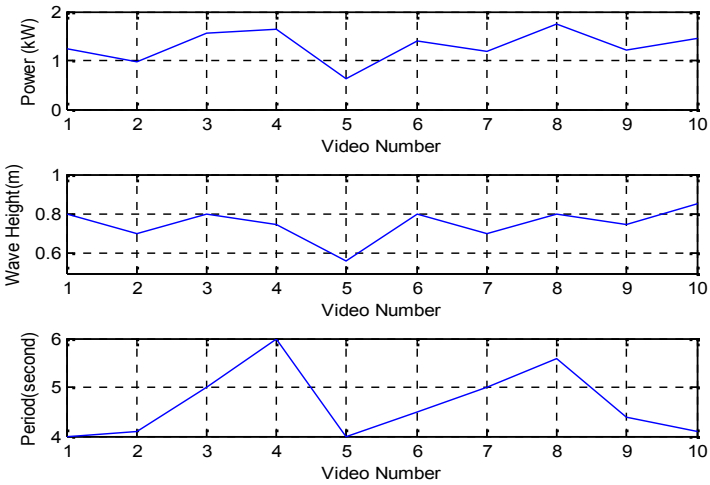

Figure. 2.4. Predicted potential power of recorded ocean waves using MATLAB

DISCUSSION

In this study, the goal is to measure the characteristics of the ocean waves using digital video image processing technology. The results shows better performance compared with currently available methods such as satellite altimeter technology, radar, stereoscopic camera, power buoy and etc. The advantages of this works lies on its simplicity but provides accurate results (Sandwell & Smith, 2008) . This method is not affected by the intensity variation of light as seen in the image stereoscopic method using 3D cameras. Another benefit gained by using digital video image processing method is the utilization of Time-Line interface of digital video image processing software.

Time-Line interface enable us to observe the properties of recorded ocean waves. Therefore, the properties of the ocean waves (period and wave height) can be determined accurately. These results can be further used to calculate the potential energy of sea waves in certain areas by linear ocean wave formula equation (2) and the result is presented in Figure 2.4 (Wikipedia, 2015). The measurement results can also be used to calculate the amount of electrical energy that can be generated by previous research (Masjono, et al., 2014).

This method can be used to measure the height and period of the ocean waves easily, before, constructing on shore or offshore infrastructure such as breakwaters, Sea Port and ocean wave energy converter. Unfortunately, data acquisition need more time due to manual observation of digital video processing interface. Therefore, algorithm that automatically detect and display the ocean wave characteristics based on recorded or live video need to be developed further.

CONCLUSION

The implementation of digital video processing on ocean wave characteristics measurements gave a new and simple method in acquiring accurate ocean wave height (0.58 – 0.85) m, wave period (4 - 6) second and estimated power (0.3 – 1.6) kW. This method could reduce the use of complicated and expensive tools with the same result. The time-line interface of the digital video processing software provide useful tools to visually observe and calculate the wave period and height. This method can be implemented in observing the ocean wave characteristics before constructing ocean infrastructure such as wave break, sea ports and wave energy converter.

Although some progress has been made using this measurement method, however, the data acquisition were collected manually through visual observation on the digital video processing software interface. Therefore, future works are required to automate the data acquisition.

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THE APPLICATION OF SYSTEM RICE INTENSIFICATION (SRI) TO INCREASE RICE PRODUCTION IN LAMPUNG PROVINCE, INDONESIA

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ABSTRACT: Change in land use and climate is the dominant factor affecting food security in Indonesia. Conversion of paddy fields into non-agricultural land between the years 1999 – 2002, which reached 563,159 ha or 187,719.7 ha/year, is one of the problems that threaten the food security program implemented by our government. On the other hand, a natural phenomenon caused by climate change, known as the La Nina and El Nino, also affect the availability of water in agriculture. Therefore we need proper farming methods in order to anticipate problems that hinder the achievement of food security in Indonesia. One of the government's efforts to increase rice productivity is to encourage the planting of rice using the SRI (System of Rice Intensification). SRI method is a method of saving water (intermitten flow irrigation) with good crop management method that can improve the productivity of rice plants up to 30-100% when compared to using a conventional irrigation methods (continuous flow irrigation). Experiments with SRI methods have been developed in Lampung province, Indonesia, to introduce the advantages of this method to farmers. The results of SRI demonstration plots conducted by BBWS Mesuji Sekampung for the period 2010 to 2013 in six irrigation areas showed an increase in rice production significantly in the four areas of irrigation. Two irrigation areas that do not show an increase in production are new irrigation areas or areas previously irrigated applying too much chemical fertilizer.

Keywords: *System of Rice Intensification (SRI), Rice Production, Lampung Province Indonesia.*

INTRODUCTION

Indonesia consists of 34 provinces where the people in general eat rice as primary food. Every Indonesian averagely consumes rice about 139.5 kg/year (Agriculture Ministry, 2011). Similar to other developing countries, the increase of population in Indonesia has negatif correlation with the reduction of agriculture areas especially paddy area. Therefore, food crisis will be a big problem in the future. Food crisis in the world has taken place about 5 times between year 2000 and year 2007. Food crisis in this period was recorded as one of the biggest global crisis in the world. The impact of the crisis affected about 36 countries in the world, including Indonesia. At that period, the stock of food commodities such as rice, weath, soy bean, and corn decreased significantly. This crisis was followed by the drop of plantation production such as palm oil and sugar cane (Tambunan, 2008). Change in land use and climate is the dominant factor affecting food security in Indonesia. Conversion of paddy fields into non-agricultural land between the years 1999 – 2002, which reached 563,159 ha or 187,719.7 ha/year, is one of the problems that threaten the food security program implemented by our government. On the other hand, a natural phenomenon caused by climate change, known as the La Nina and El Nino, also affect the availability of water in agriculture.

Lampung Province is one province in Indonesia which is one of the strong supporter for National Food Security program. This province is the seventh of Indonesia National Granary with the area of 35,376.50 km^2 (Nurhayati, 2010). Agriculture areas in this provinces is spread in almost all districts and cities which are 15 in numbers. Total area of paddy field in the province in year 2013 was about 584,479 ha with crops of about 3.04 tons (BPS Provinsi Lampung, 2014). The areas of technical irrigation in Lampung Province was about 219,000 ha which spread in 3 river basin: Seputih-Sekampung, Mesuji Tulang Bawang, and Semangka (Public Work Department, 2005).

One of government effort to increase the agriculture production, mainly rice, is by promoting method called System of Rice Intensification (SRI). SRI method is kind of water-saving irrigattion with good farming management. In theory, SRI method can increase rice production into 30 – 100% if compared to conservative irrigation method (continuous flow irrigation) (Huda et al., 2012). SRI is not a technology but a methodology of farming which is based on idea to create better environment of paddy areas (Richardson, 2010).

This paper aims to demonstrate the application of SRI method in the rice field in Lampung Province through demonstration plots. Location of the study is six irrigation areas in several districts and cities in Lampung Province which apply groundwater irrigation. The demonstration plots of the SRI paddy area took place between year 2010 and year 2013.

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SYSTEM OF RICE INTENSIFICATION (SRI)

SRI, stands for System of Rice Intensification, is one of innovation in rice farming method which is developed by Fr. Henri de Laulanie in 1980's in Madagascar. Since the end of the year 1990's SRI has been introduced to the world by Prof. Norman Uphoff, former Director of Cornell International Institute for Food, Agriculture and Development (CIIFAD). In 1999 SRI was, for the first time, applied in outside Madagascar. It is applied in China and Indonesia. After the succes of the application, SRI has been applied in more than 25 countries with crop production of 7-10 tons/ha (PPK Sampoerna, 2009). Three pricipal of SRI method are:

1. Seedling management
   Seedling management consists of the selection of seeds, planting seeds in young age (less than 10 days after seedbed), spacing management, and depth of planting management. In the SRI method, one seed is planted in one hole and the root is planted horizontally.
2. Water management
   SRI does not need continuous water flow. Water irrigation is supplied to the paddy field only when the soil is flooded with 0.5 cm of water.
3. The use of organic fertilizer
   SRI also does not use chemical fertilizer in the farming. Organic fertilizer such as compost is the one usually used in SRI. Compost is nutrient supplier and bioreactor component that lead the optimum process of paddy growth.

The excellence of SRI if compared to the convensional method of farming are:

1. Saving the money by using less seeds
2. Saving water by using intermitten flow
3. Saving time because harvest time is coming faster
4. Increasing crop production
5. Environmental friendly because the use of organic fertilizer is promoted

The SRI method is not always suitable for all condition of agricultural land. The most appropriate agriculture land for the SRI method is the one with conditions as follows:

1. Experiences the shortage of water
2. Having excessive use of chemical fertilizer
3. Having low production due to the soil condition

In Indonesia,SRI method has been applied in some irrigation areas in Java, Bali,West Nusa Tenggara, Sumatera, Kalimantan, Sulawesi, and Papua. However, the spread of application of SRI method is generally limited only for demonstration plot. SRI application in Ciamis and Garut of West Java has showed that the yield of rice increased if compared to one produced by traditional way (Anugrah et al., 2008). Research conducted in Kabupaten Malang, East Java, showed that the application of SRI method has increase farmer's income into 42% if compare to the income gained by traditional method. In Madagascar, farming using SRI method could yield about 10 to 15 tons/ha.

However, not all irrigation areas are suitable for SRI. Research in China and Phillipine about SRI indicated there is no significant impact of rice yield by using SRI method. Some problems also found in SRI application in Indonesia such as:

1. Irrigation management is not easy to apply by farmers
2. SRI needs more workers if compared to traditional way
3. It is difficult to apply 'one seed for one hole' plantation
4. Some plant diseases attack young seed after plantation

DEMONSTRATION PLOT OF SRI IN LAMPUNG PROVINCE

SRI method has been developed widely in Lampung Province through demonstration plots with purpose to introduce the benefit of this method to the farmers. Balai Besar Wilayah Sungai Seputih Sekampung in collaboration with several water user farmer groups has carried out several demonstration plots in six irrigation areas in several districts and cities in Lampung Province. They are East Lampung District, Pringsewu City, Metro City, Central Lampung District, and South Lampung District, during 2010 – 2014. The scope of demonstration plot program captured:

1. Preliminary survey of demonstration plot locations
2. Workshop about SRI for the farmers
3. Demonstration plot action
4. Training of Trainer (TOT) for government officer about SRI

The location of demonstration plots are given in the table below:

Table 1 Locations of demonstration plot of SRI in Lampung Province

No.	Irrigation area	District/City	Area (ha)			
			2010	2011	2012	2013
1	Sekampung Batanghari	East Lampung	7	10	33	33
2	Metro Kota	Metro	5	7	10	20
3	Purbolinggo	East Lampung	3	16	18	33
4	Trimurejo	Central Lampung	-	-	5	12
5	Seputih Raman	East Lampung	-	-	-	10
6	Pringsewu	Pringsewu	-	-	-	5

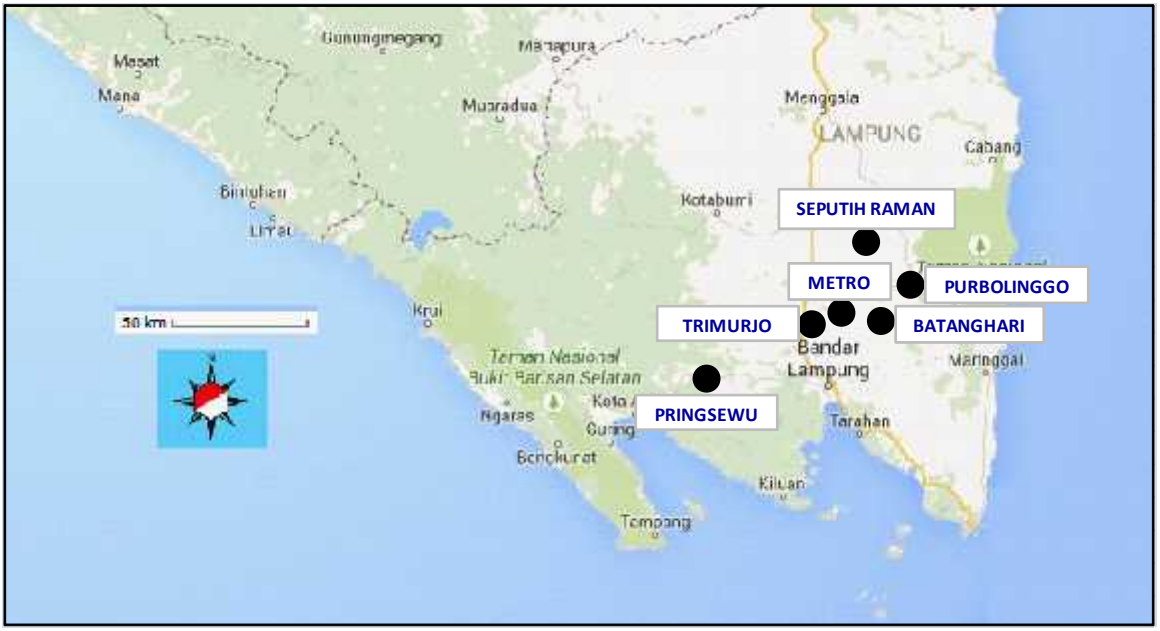

Fig. 1 Locations of SRI demonstration plot in Lampung Province

Rice production for each irrigation area is given in the table below:

Table 2 Production of rice in the locations of demonstration plot

No.	Irrigation area	District/City	Area (ha)/Production (tons/ha)			
			2010	2011	2012	2013
1	Sekampung Batanghari	East Lampung	7/5.00	10/6.76	33/5.40	33/8.95
2	Metro Kota	Metro	5/4.80	7/7.14	10/10.40	20/6.20
3	Purbolinggo	East Lampung	3/4.60	16/5.40	18/9.17	33/9.97
4	Trimurjo	Central Lampung	-	-	5/11.52	12/7.50
5	Seputih Raman	East Lampung	-	-	-	10/17.00
6	Pringsewu	Pringsewu	-	-	-	5/11.20

In other way, the increase of paddy production can be given as follows:

Table 2 Production of rice in the locations of demonstration plot

No.	Irrigation area	Area (ha)/Production (tons/ha)				Production increase (%)		
		2010	2011	2012	2013	2010/2011	2010/2011	2011/2012
1	Sekampung Batanghari	7/5.00	10/6.76	33/5.40	33/8.95	35.20	-20.12	65.66
2	Metro Kota	5/4.80	7/7.14	10/10.40	20/6.20	48.81	45.60	-40.38
3	Purbolinggo	3/4.60	16/5.40	18/9.17	33/9.97	17.39	69.75	8.76
4	Trimurjo	-	-	5/11.52	12/7.50			-34.90
5	Seputih Raman	-	-	-	10/17.00			
6	Pringsewu	-	-	-	5/11.20			

Fig. 2 Demonstration plots of SRI method in Lampung Province

RESULTS

The breakdown explanation for every demonstration plots area is given below.

1. Sekampung Batanghari Irrigation Area

Graphically, the rice production based on the planted area in the area is given below:

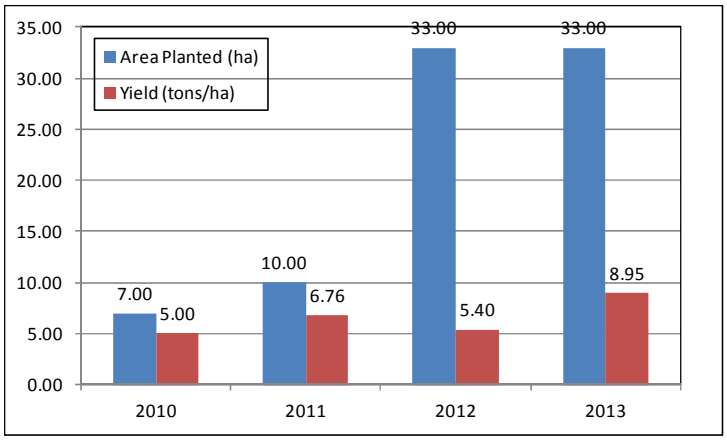

Fig. 3 Rice production based on the planted area in Sekampung Batanghari Irrigation Area

In Sekampung Batanghari Irrigation Area, it is obvious that the extreme additional area planted will decrease the productivity of farming. The extreme change of area planted between 2011 and 2012, where area planted changed from 10 ha into 33 ha, causes the decrease of farming productivity, which is from 6.76 tons/ha into 5.40 tons/ha.

2. Metro Kota Irrigation Area

Graphically, the rice production based on the planted area in the area is given below:

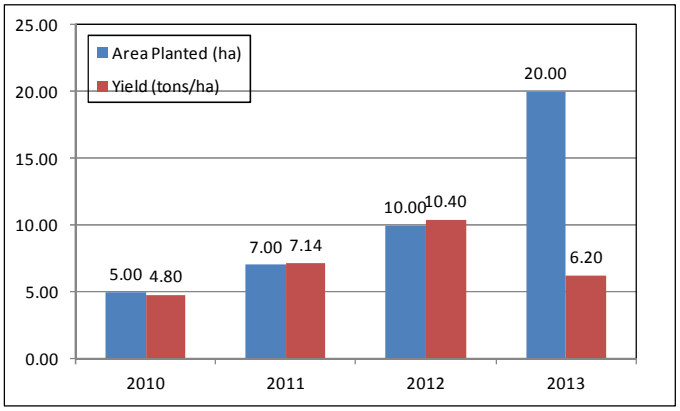

Fig. 4 Rice production based on the planted area in Metro Kota Irrigation Area

In Metro Kota Irrigation Area, it is obvious that the extreme additional area planted will decrease the productivity of farming. However, the graph also shows that effective additional area planted will not give effect to the decrease of the productivity of farming. The fact indicates there is specific ratio between existing area planted and additional area planted that can be used to defend against the decrease of the productivity of farming.

3. Purbolinggo Irrigation Area

Graphically, the rice production based on the planted area in the area is given below:

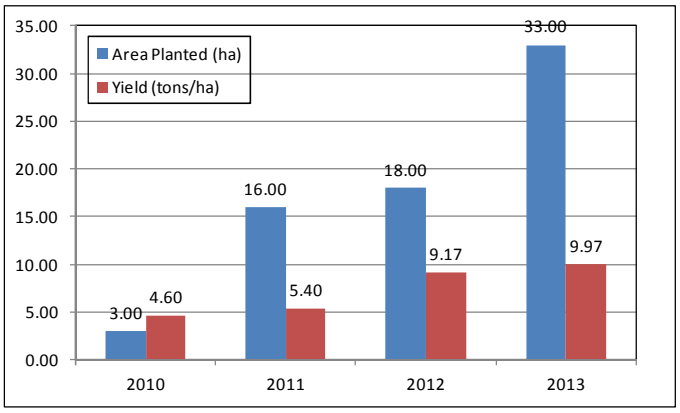

Fig. 5 Rice production based on the planted area in Purbolinggo Irrigation Area

In Purbolinggo Irrigation Area, the decrease the productivity of farming due to additional planted area is not found at all. During 2010 to 2013 there is no decrease of the productivity of farming even there is extreme additional area planted in the year 2013.

4. Trimurjo Irrigation Area

Graphically, the rice production based on the planted area in the area is given below:

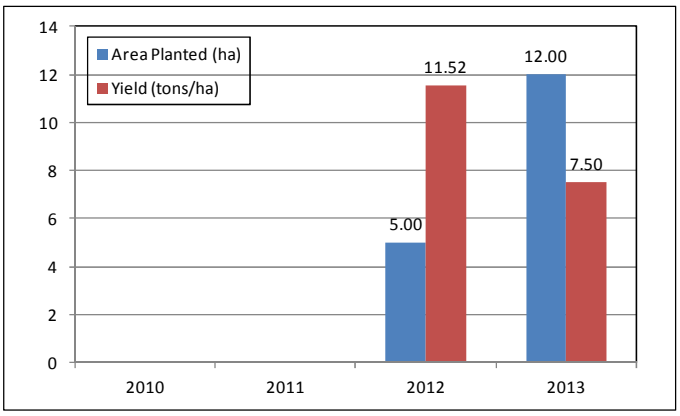

Fig. 6 Rice production based on the planted area in Trimurjo Irrigation Area

In Trimurjo Irrigation Area, demonstration plot was started in 2012. The characteritic of this irrigation area is not so different from Sekampung Batanghari and Metro Kota Irrigation areas. Farming productivity will decrease if there is significant addition of area planted.

5. Seputih Raman Irrigation Area (a) and Pringsewu Irrigation Area (b)
Graphically, the rice production based on the planted area in the area is given below:

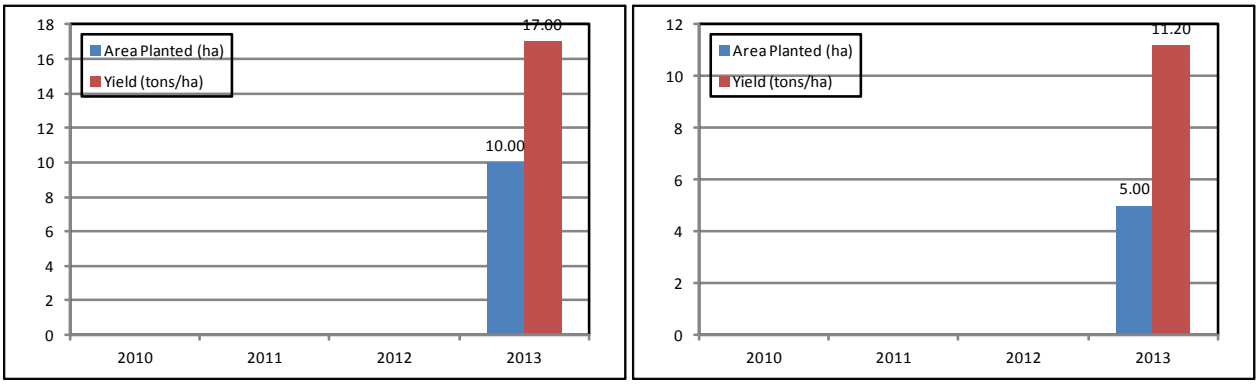

Fig. 7 Rice production based on the planted area in Seputih Raman and Pringsewu Irrigation Area

Demonstration plot in Seputih Raman and Pringsewu Irrigation area was started in 2013. Both areas indicate good performance of irrigation using SRI method. Both areas yield rice of more than 10 tons/ha. The effect of additional area planted to farming productivity has not been identified yet in both areas.

DISCUSSIONS AND CONCLUSSIONS

In general, the application of SRI in the demonstration plot areas give positive effect to the farmer. Almost all areas have good productivity. Decrease in rice production is generally happened when the area planted are extremely added. The additional area planted are usually the areas that have long term experienced of chemical fertilizer use. Long term application of chemical fertilizer leads to the critical condition of soil and makes the soil addicted to the chemical fertilizer. Nowadays, about 66% of 7 millions of Indonesian agriculture lands are in critical condition due to the addiction of chemical fertilizer. Excessive application of chemical fertilizer can cause soil polluttion and soil degradation. Soil degradation happened due to the damage of humus regeneration in the soil (Romli, 2012). Thus, soil fertility is decreased and farming productivity will drop significantly. In the SRI method, the effect of chemical fertilizer can be reduced by introducing organic fertilizer. The fertilizer is environmental friendly fertilizer and able to conduct humus regeneration in the soil. To change soil structure which previously used chemical fertilizer into the normal one, needs time. Therefore, if SRI method is applied to this kind of soil, the yield of rice will not directly increase but need some times called remediation time.

The research has indicated that SRI is a method which gives more benefit to the rice farmers due to its high rice yielded. This method also promotes the efficiency of water use and the use of organic fertilizer in order to rehabilitate soil condition. The reduction of using chemical fertilizer, on the other hand, will reduce soil and water pollutant in surrounding irrigation areas. Balai Besar Wilayah Sungai Mesuji – Sekampung in Lampung Province is still countinuing SRI method in more irrigation areas in Lampung Province. In the future, SRI method will be a media for increasing the farming productivity in Lampung Province. It is necessary to note that SRI has to be informed to all farmers and NGO which concern to the rice productivity in Lampung Province. Workshop about SRI is very important to carryout. The workshop can be a media of information distribution among farmers, government , and NGO. The workshop can also be a place to share some experiences in using SRI and evaluate the experiences for better results in the future.

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THE EFFECT OF POROSITY OF REVETMENT AGAINST WAVE REFLECTION

Aswar[1], Arsyad Thaha [2] and Rita Lopa[3]

Abstract: Porous revetment is a new model in the coastal protection system and is expected to reduce the impact of wave reflection when it comes hit the revetment. The aim of this study is to obtain the amount of reduction in wave reflection due to the porosity of the revetment. Two porous revetment models tested in this study in which the value of the S/D represents the pore structure (S is the distance between the holes and D is the diameter of the hole). The results show that the porosity of the revetment is able to reduce wave reflection. Reduced wave reflection was 5% - 20% in the presence of this porosity.

Keywords: *Perforated revetment, wave reflection.*

INTRODUCTION

Reflected waves may lead to danger to vessels navigating close to structure, and may reduce the availability of berths within the harbor. Wave reflection may also increase local scour or general reduction in sea bed levels.In the design of breakwaters, seawalls, and coastal revetments, it is therefore important to estimate and compare the reflection performance of alternative structure types. Researchto reduce reflection have been carried out by physical model, including increase crest of revetment, make a rough wallor porous wallon revetment and reducing wave energy to the revetment wall by building obstacles.

Allsop and Hettiarachchi (1988), indicated rubble breakwaters, seawalls, and revetments will dissipate significantly more wave energy than the equivalent non-porous slope. Recent investigations (Oumeraci et al. 2010 & Oumeraci et al. 2012) indicated that the preservation of the pore structure is of fundamental importance due to its effect on the hydraulic performance and on the wave loading of the revetment and its foundation. Especially aging processes, marine growth (e.g. Barnacles or shells, growing on and in the revetment) and sediment deposition in the pores may affect the porosity of the structure over time. In this context, it is crucial to know the porosity reduction, which may substantially affect the wave loads and the hydraulic performance, and which may thus result in structural or functional failures.

LITERATURE

The distribution of the amount of wave energy that's reflected, dissipated, and transmitted depends on the characteristic of the arriving waves (wave height and period), coastal protection type (rough or smooth surface, permeable or impermeable) and the dimension also protection geometry (slope, elevation, and the barrier width) also the surrounding environmental condition (water depth and coastal base contour). The amount of waves that's dissipated (Hd) is the arriving waves high (Hi) reduced by the reflected and transmitted high (Ht). The wave parameter according to Airy theory is the parameter based on the assumption of a harmonic sinusoid (Triatmodjo, 1999), with few wave characteristics, those are the wavelength (L), the complete description will be shown below. Several used notations are:

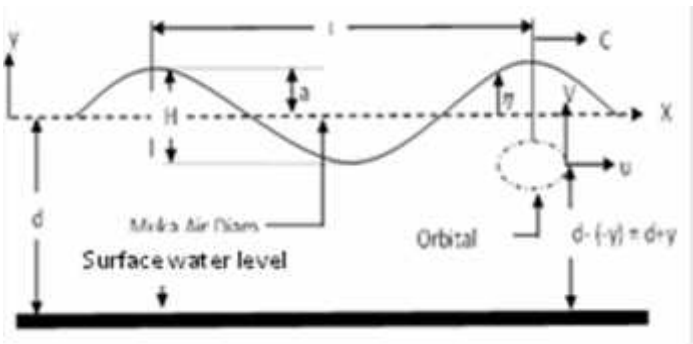

Figure 1. Wave Airy Theory, (Triatmodjo, 1999)

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d : distance between still water level and sea floor
η (x,t) : fluctuation toward mean water level
a : wave amplitude
H : wave height = $2a$
L : wave length
T : wave period, time interval that's needed by water particle to return to the same point similar to prior point
C : wave velocity = L/T
k : wave index $2\pi/L$
σ : wave frequency $2\pi/T$

The Figure 1 shows that the wave moves with the velocity of C in the water with depth of d. in this case, the moving element is just the water level form. Unlike the river basin in which the water (mass) particle moves in one closed orbit so that it doesn't move forward. A buoy in the sea will only move up and down following the wave and does not move (in the wave direction) from its origin point. Position of particles every moment of the orbit moves is given by horizontal coordinate (ξ) and vertical (ε) towards central orbit. The vertical velocity is u and v, and water level elevation towards the still water (x axis) in every point is η.

METHODOLOGY

The study was conducted with a 2D physical model in the Laboratory of Hydraulics and Coastal Engineering, Department of Civil Engineering Faculty of Hasanuddin University Gowa. Model revetment with porous surfaces such as concrete blocks with a slope of30°. To be able to determine the effect of several variables on the reflection of the wave, then the simulation. Two types of models are distinguished on the value of perforations (S/D) with S is space between perforation and D is the diameter of perforation hole.Value S/D for model-1 is 2,333 and the value of S / D for model-2 is 1,75.Models that were tested in the present study can be seen in Figure 2. The model simulated by each of three kinds of wave high (H) and wave period (T) and 3 kinds of water depth (d). To meet the scaling requirements of all variables, then used a geometric scale 1:20.

Figure 2. Model-1 and Model-2

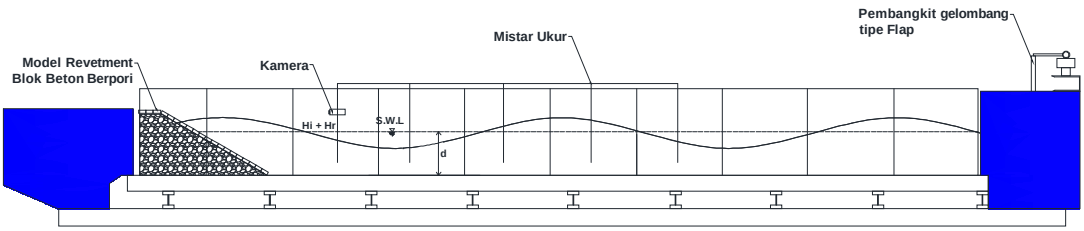
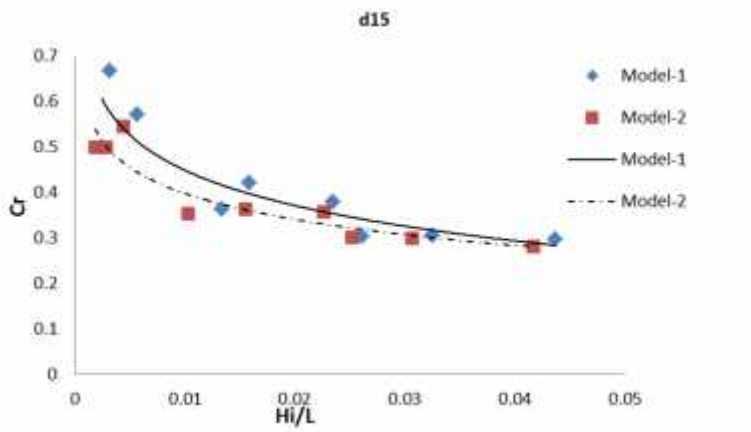


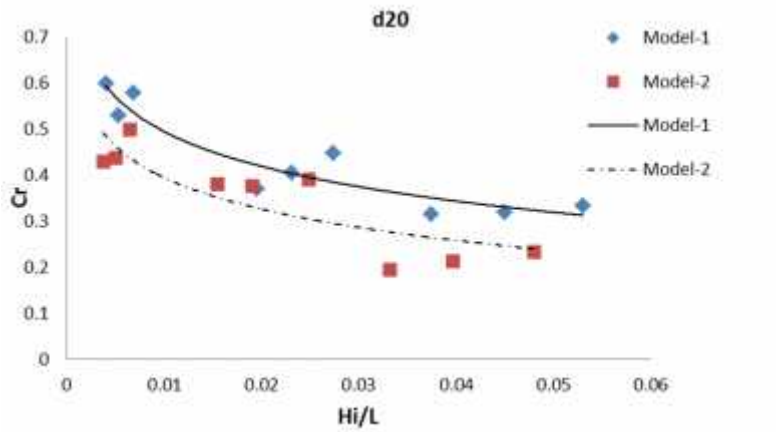
Figure 3. Experimental setup on laboratory

RESULT AND DISCUSSION

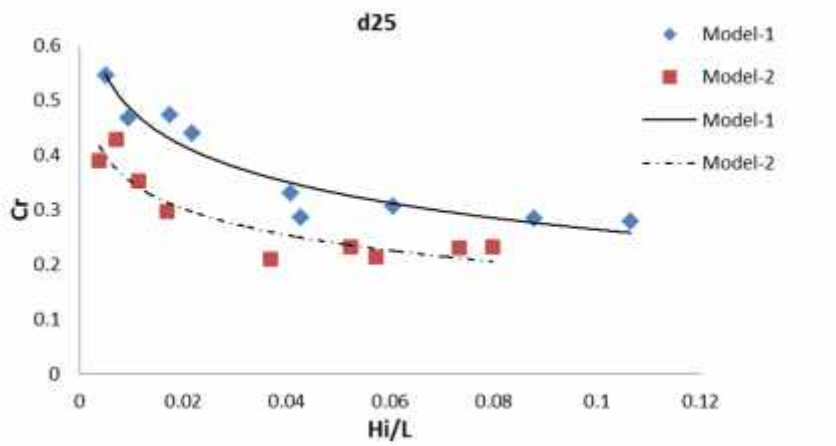
Based on research obtained wave reflection is reduced in the presence of porosity in the revetment. Reflected waves is reduced by 5% - 20% with the porosity of the revetment.The effect of porosity on wave reflection can be seen in the Figure 4.While the relation between HiS/DL parameter and reflection coefficient (Cr) can be seen in the Figure 4.



(a)



(b)



(c)

Figure 4. Relationship between wave steepness (Hi/L) and reflection coefficient (Cr), with depth (a) 15 cm, (b) 20 cm, (c) 25 cm

Figure 4.Shows that the variation of the reflection coefficient(Cr) depend on wave steepness.Higher wave steepness (Hi/L) causes large wave dissipationand less reflection. While the effect of porosity on revetment seen on the graph where the reflection wave in model-2 is smaller than the wave reflection on the model-1.

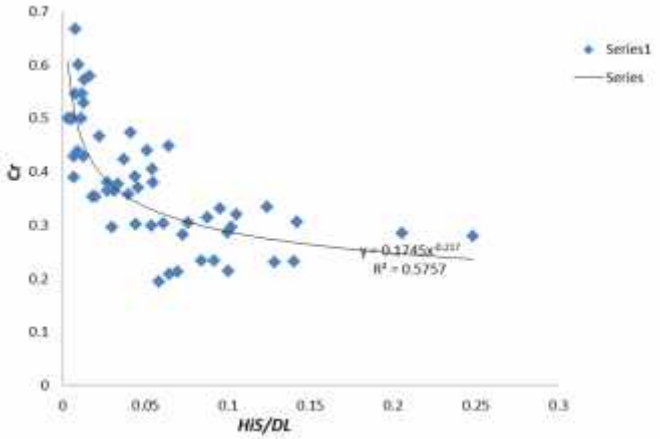


Figure 5.Relationship between HiS/DL parameter and reflection coefficient (Cr)

Figure 5. shows the relationship between parameter HiS/DL(relative wave steepness) and Cr (reflection coefficient) wherehigher relative wave steepnes (HiS/DL) causes large wave dissipation and less reflectionin agreement with Suh et al (2006) that found that increasing wave steepness leads to less reflection and Faraci et al (2012),Thaha A et al (2013) Altomare et al (2013), who confirmed those results. The result from Altomare et al (2013) research can be seen in Figure 6. From this research obtained the empirical equation for Cr is :

$$C = 0{,}1745\left[\frac{H}{D}\right]^{-0.2} \tag{1}$$

With Cr = reflection coefficient; Hi = Incident wave high; S = Space between perforation; D = Diameter perforation; L = Wave length (calculated using wave period).

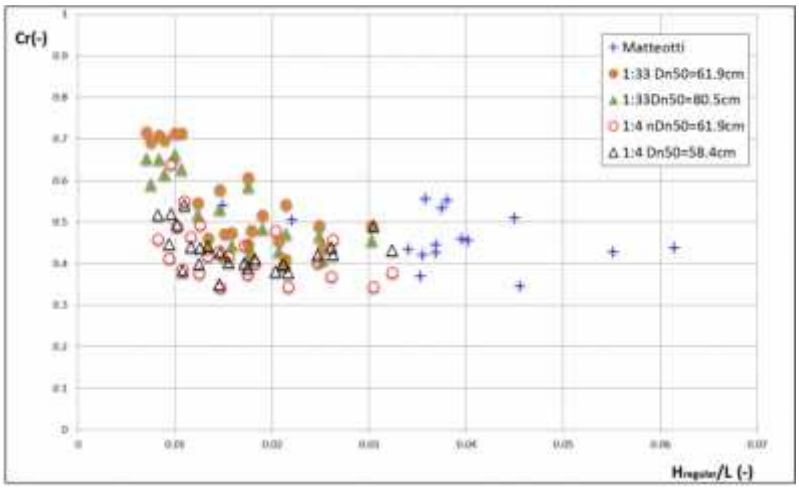


Figure 6. Reflection coefficient v's Wave steepness, (Altomare et al, 2013)

CONCLUSION

1. It was found that the reflection coefficient (Cr) decreases as the wave steepness (Hi/L) increase.
2. Perforated revetment can decrease the reflection coefficient (Cr) until 20%.
3. Equations obtained in this research only applies to regular wave and slope structure 30°.

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THE EFFECT OF POROSITY OF VERTICAL CYLINDER SUBMERGED BREAKWATER AGAINTSTO THE WAVE TRANSMISSION AND REFLECTION

Indah B [1], Arsyad Thaha[2], Mukhsan P. Hatta[3]

ABSTRACT: This study aims to determine the effect of porosity (Ɛ) and the width of structure (B) of a vertical cylinder typeof submerged breakwater againts transmission and reflection of wave propagations. Inter-parameter relationship will be formulated in the Non-dimentional Parameters (NDP). This study is experimental with physical tests performed in the wave channel at the Hydraulics Laboratory of Civil Engineering, Faculty of Engineering, Universitas Hasanuddin. Models created with multiple configurations in the various porosity and structure width and simulated with various wave height and period. The Simulationswere performed in two conditions, there arecylindrical structure in filled and unfilled byrock stones underconstant water depth. The results showed that the transmission coefficient (Kt) is getting smaller with the increasing the structurewidth and the decreasing the porosity. The reflection coefficient (Kr) is getting greater with the increasing the structure width and the decreasing the porosity. The equation results presentedin the relationship between $HiB/ƐL^2$ with Kt and Kr. The Equation can be written as $Kt = 0.1677\ (HiB/ƐL^2) - 0.302$and $Kr = 0.1338\ \ln\ (HiB/ƐL^2) + 0.9595$for the unfilledcondition, while $Kt = 0.1482\ (HiB/ƐL^2) - 0.279$and $Kr = 0.1063\ \ln\ (HiB/ƐL^2) + 0.9376$for the rock filled condition. Transmission coefficient (Kt) and reflection (Kr) in the unfilled conditions were found are greater than Kt and Kr in the stone filled condition.

Keywords : *porosity, transmission coefficient, reflection coefficient.*

INTRODUCTION

Indonesia is an archipelago country that has a lot of beaches with diverse characteristics. Various problems occur on a beach in Indonesia, one of which erosion or abrasion. Coastal protection in Indonesia needs to be done to reduce abrasion or erosion on the beach. Several methods are used to protect beaches such as by installing a wave absorbers made from rock like tetrapod, gamapod, A-jack and others.

Many researchers are making various types of break waters which aims to protect the eroded beaches, shore line stability, protection of tourist are a sand natural coral. Paotonan (2010), conduct the oretical and experimental studies of wave propagation through the breakwater under the focus of the studyis the transmission and reflection waves. Leonanda (2006), examined the behavior ofthe cylinderto fluid flowand stated that the ability ofthe cylinderto regulate fluid flow patternis very good considering the vortex shedding, the drag force and the drag which causes an increase instability. Sudoto (2008), examined the reduction of the transmission waves which pass through hollow cube formartificial reefs as submerged breakwater and stated that the transmission coefficientin the form of a cube perforated breakwaters affected by the wave period, the depth of the peak and peak width of the structure. This study aims to determine the effectof transmission and reflection in a vertical cylindrical porous by assessing the effect of wave steepness, porosity and composition breakwater structure and get relationship dimensionless parameter (Non-dimentional Parameters, NDP).

LITERATURE

A wave of rising and falling water movement in the direction perpendicular to the surface of the sea water that forms a sinusoidal curveorgraph. Waves occur daily and taken into account in the field of coastal engineering is wind waves and tidal (tidal). Waves can formand damaging effect on the coast and coastal buildings.

Transmissionis a continuation of the wave through a building that parameteris expressed as the ratio between the transmitted wave height (Ht) with the coming wave height (Hi) or the root of the wave energy trasnmisi (Et) and the coming wave energy (Ei). Mathematically it can be expressed in the equation:

$$Kt = \frac{Ht}{Hi} \tag{1}$$

$$H_t = \frac{2a_{max} + 2a_{min}}{2} = \frac{H_{tmax} + H_{tmin}}{2} \tag{2}$$

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Wave reflection is an event where the wave or hit a building will be reflected partially or completely. Overview of wave reflection is important in the planning of coastal structures, especially in building the port. Great ability of an object to reflect wave reflection coefficient is given by the ratio between the height of the wave reflection and wave heights come. Mathematically it can be expressed in the equation:

$$Kr = \frac{Hr}{Hi} \tag{3}$$

Where :

Hi = high wave before the structure (m)
Hr = high wave reflection (m)
High wave come is:

$$H_i = \frac{2a_{max} + 2a_{min}}{2} = \frac{H_{imax} + H_{imin}}{2} \tag{4}$$

High wave reflection is:

$$H_r = \frac{2a_{max} - 2a_{min}}{2} = \frac{H_{rmax} - H_{rmin}}{2} \tag{5}$$

METHODOLOGY

The study was conducted with experimental methods with physical testing in the wave channel Hydraulics Laboratory of Civil Engineering Faculty of Engineering, University of Hasanuddin. Materials and equipment used is the channel wave (wave flume) the length of 15 m and a width of 30cm. Effective depth of the channel is 45 cm. Breakwaters made of a base material of the pipe with a diameter of 140 mm and a thickness of 4.2 mm pipe. At the breakwater created a hole with a diameter (D) 14 mm and distance variation of pore are 14 mm, 21 mm and 28 mm. Thus obtained three variations of porosity are 0.1792, 0.1092 and 0.077. Wave length use disregular wave.

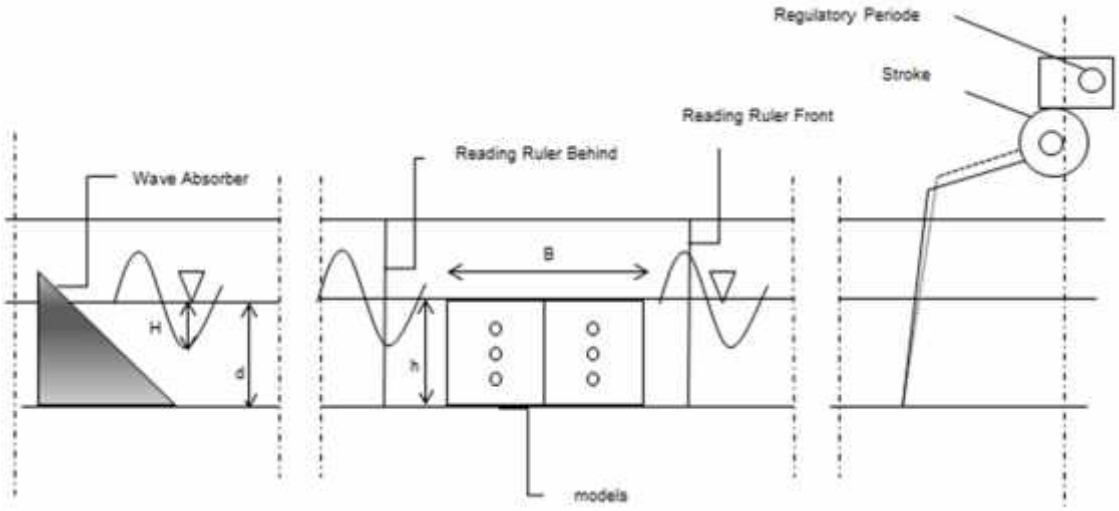

Fig. 1 Model Placement Sketch on*Wave Flume*Side View

Fig.2 Models Breakwater M1,M2, dan M3

Simulations were performed in this study is to put the model breakwateri n the middle flume equipped with measuring devices at the front and back of the model. After that, the wave flume raised. The parameters measured were the reflection wave (Kr) and wave transmission (Kt) which is influenced by the high waves coming (Hi), wave period (T), wave length (L), water depth (d), the high structure (h), the composition structure (SU) and porosity(Ɛ). Simulations performed under two conditions, there are empty conditions and the condition of filled with stones.

The data obtained in the laboratory processed using dimensional analysis Langhaar method for obtaining the relationship between the parameters that will produce a dimensionless number. Thus these results can be generalized (Yuwono, 1996).

RESULTS AND DISCUSSION

The results were obtained relationship between the research data with the transmission coefficient(Kt) and the reflection coefficient(Kr) in the form ofa dimensionless relationship.

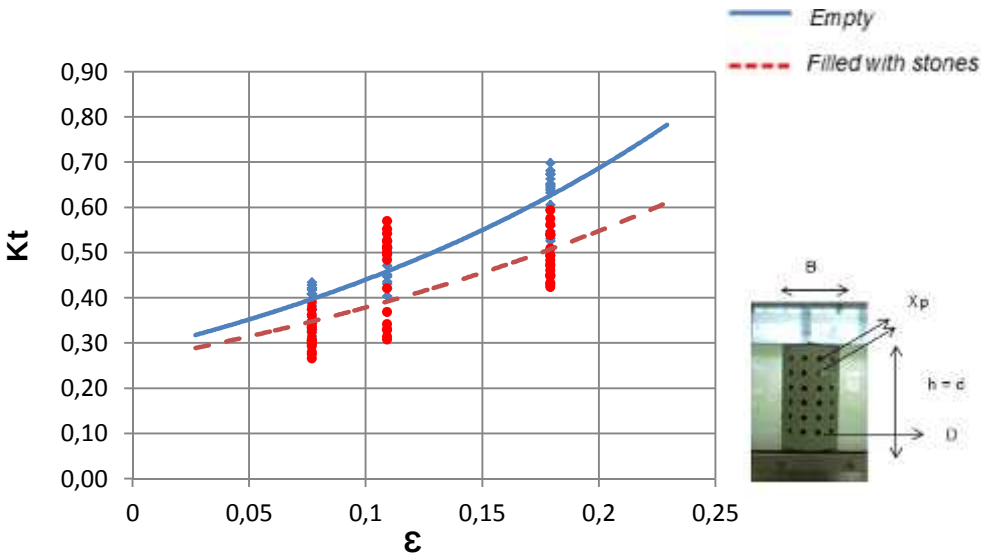

Fig. 4 Porosity Relations(Ɛ) with Transmission Coefficient (Kt)

on the condition of Empty and Filled with Stones.

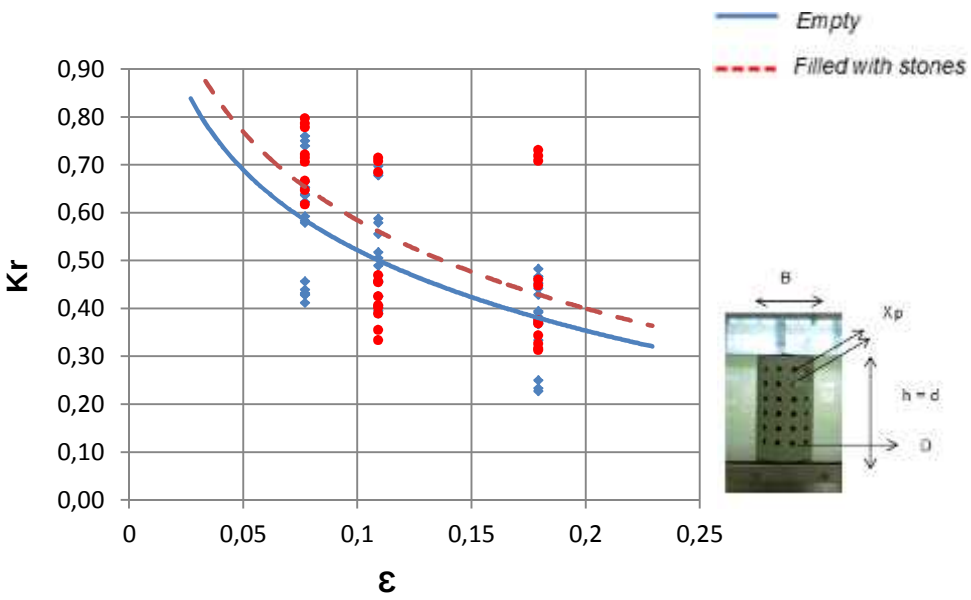

Fig.5 Porosity Relations (Ɛ) with Reflection Coefficient (Kr)
on the condition of Empty and Filled with Stones.

From the analysis obtained correlation dimension (Hi B/ƐL²) against the transmission coefficient(Kt) and the reflection coefficient (Kr). When plotted on a graph where dimensional analysis as Kt X-axisand Y-axis or Krasit will obtain the curve. From the graph will be obtained equation is Kt = 0.1677(HiB /ƐL²) -0.302 and Kr = 0.1338 ln (HiB /ƐL²) +0.9595for the empty condition. And the equation Kt = 0.1482 (HiB /ƐL²) -0.279 and Kr=0.1063ln (HiB /ƐL²) +0.9376for the condition of filled with stones. Transmission and reflection equation is calculated based on the chart combined with numerical methods (Setiawan, 2006).

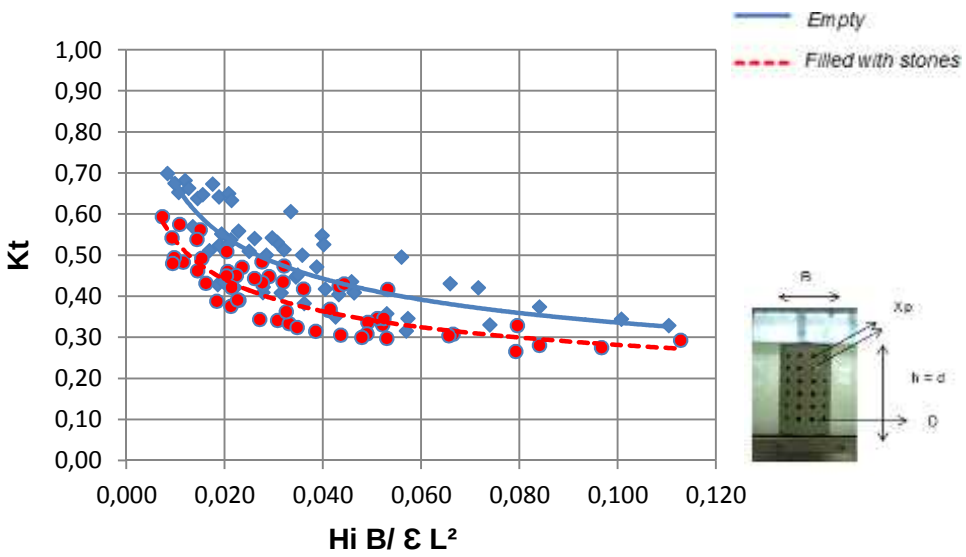

Fig.6 relationship *Hi B /Ɛ L²* with *Kt* for all data dan conditions

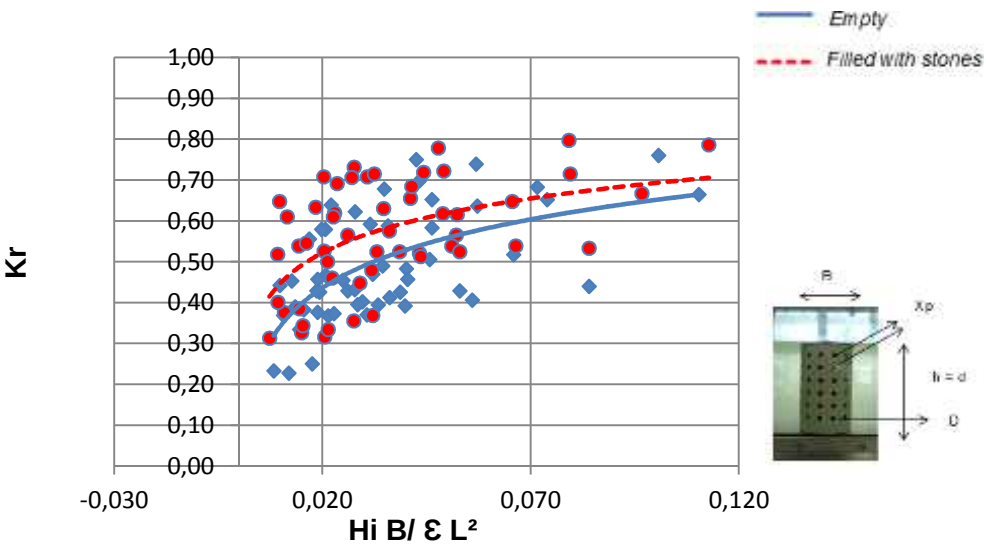

Fig.7 Realtionship *Hi B /Ɛ L²* with*Kr* for all data and condition

The relationship between the dimensionless parameter (Hi B / Ɛ L²) against the transmission coefficient (Kt) and the reflection coefficient (Kr), where the incresase thevalue of (Hi B / Ɛ L²), the value of Kt is decrease and the value of Kr is increase. In detail, can be described that the increase the wave steepness (Hi / L) as well as the composition of the model (B / L), the value of Kt is getting smaller and the value Kr is getting greater. The increase the porosity (Ɛ) or a small pore distance (Xp), makes the value of Kt is getting greater and the value Kr is getting smaller. Conditions of empty and the conditions filled with stones also affects the transmission and reflection. Kt values in empty condition is larger than the conditions filled with stones. Kr values in empty condition is smaller than the condition of filled with stones. The influence of the parameters of the Kr in accordance with the theory that high waves that pass through an obstacle will be reduced non-linear (exponential) with high reduction value of the smaller waves. The resulting graph is exponential graph is based on the theory if progressive and standing waves propagating through a medium that axis, then the amplitude of the wave decreases exponentially (Dean and Dalrymple 1993).

Based on the analysis of the research results, the damping of waves that occur in each model is based on the effect of porosity (Ɛ). The increase the porosity (distance between the pores are smaller) give little reflection and transmission are large compared to the decrease the porosity (distance between the pores are larger). In addition, the composition of the model (SU) and the condition of the model in a empty condition and the condition of filled with stones also provide wave attenuation effect, in which increasing the compotition of models (SU) or peak width (B) so the wave transmission is decrease and the wave reflection is increase for the condition empty or filled with stones. This fits the theory that the structure of the breakwater which has a greater width will give a smaller transmission coefficient due to the large width of the peak will have a broad field of friction greater waves, so the ability to reduce the waves are big (Imawan et al., 2009).

CONCLUSION

1. The greater of the vertical cylindrical porosity(Ɛ), the increasing the transmission coefficient (Kt) and the decreasing the reflection coefficient (Kr).
2. The increasing of the wave steepness(Hi/L), the decreasing of Kt and the increasing of Kr.
3. The larger therelative widthof structure (B/L), the smaller of Kt while Kr become greater.
4. The finding equation can be written as Kt = 0.1677 (HiB/ƐL²) -0.302 and Kr = 0.1338 ln (HiB/ƐL²) + 0.9595for the unfilled condition, while the equation for the rock fillet conditions can be written as Kt = 0.1482 (HiB/ƐL²) - 0.279 and Kr = 0.1063 ln (HiB / ƐL²) + 0.9376.
5. Transmission coefficient (Kt) and reflection (Kr) in the unfilled conditions were found are greater than Ktand Kr in the stone filled condition.

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THE CHALLENGES AHEAD FOR THE INSTALLATION OF A SANITATION SYSTEM IN DILI

Piedade, J.[1] ; Cruz, G.[2] ; Almeida, O.[3]

Abstract : The city of Dili, East Timor's capital, is a rather flat city, but with a very small area, lying nestled between the mountains and the sea. According to the latest estimates, the city has a number of inhabitants which exceeds 200,000 and a strong population growth is expected to take hold in the coming years. Under a Cooperation Agreement between the Government of East Timor and the Government of Victoria (Australia) was developed between 2010 and 2012 one Master Plan for Sanitation and Drainage in Dili, whose main conclusions and recommendations were approved by the Government of East Timor. Also during the preparation of this first study were built, as a pilot project, two Community Sanitation facilities in different locations about which we can already make an assessment of its operation. For the course of Phase 2 (the current phase) MOPTC of the GDTL decided to hire AdP TL, a subsidiary of Portugal Water company for the development of the priority projects of drainage but also for priority Sanitation projects. The following works were referred were AdP TL responsibility. It was planned in Phase 1, that Phase 2 will start with the installation of 4 Community Sewage Treatment Systems, one to be installed in a school, another in a health clinic and two in neighborhoods, one of a small size, other of medium/large size. These engineering projects are already established but the works have not been completed yet. Also in Phase 2 it is planned the building of more 50 CSTS and 1 DEWATS, and in Phase 3 the construction of 7 more DEWATS, covering all Dili city. The current situation is based on individual septic tanks with sewage collection by tank truck and transportation to an existing sewage treatment plant in Tibar, about 20 kms from Dili, therefore any Sanitation network is absent. The discussion will now be centered on two different visions for the future of sanitation in the city: either the phased construction of multiple decentralized systems (CSTS or DEWATS, depending on their size) or the construction of a main collector through the entire city (about 11 kms, with several pumping stations), a network of secondary collectors and later a network of local home collectors without the use of local transportation and treatment systems, it will be done entirely by the city network and a new WWTP. In both solutions, investments are quite different, as well as the future activities of operation and maintenance, as well as the phasing of investments, but it is a technical, social and environmental discussion that is ongoing and will need to be deepened, until the Government make a final decision and decide to implement. The purpose of the presentation is to inform the 2nd MICCE, of the 4 detailed engineering designs already prepared to act as pilot projects and continue to collect information that leads to future solutions to be adopted in the city Sanitation area. There will also be presented, in detail, the two solutions now under study: multiple decentralized systems, or a general collector and progressive routing of all sewage to a new WWTP[4].

Keywords : *Community Systems (CSTS), DEWATS, Dili, Sanitation, Wastewater System, WWTP*

List of Acronyms:
CBO/BOD – Biochemical Oxygen Demand)
CQO/COD – Chemical Oxygen Demand)
CSTS – Community Sewage Treatment System
DEWATS - Decentralized Wastewater Treatment Systems
DNSA –National Directorate of Water (Direção Nacional dos Serviços de Água)
DNSB –National Directorate of Basic Sanitation (Direção Nacional de Saneamento Básico)
DNTPSC –National Directorate of Land, Property and Cadastral Services (Direção Nacional de Terras, Propriedades e Serviços Cadastrais)
ETAR - Wastewater Treatment Plant (Estação de Tratamento de Águas Residuais)
GRDTL –Government of the Democratic Republic of East Timor (Governo da República Democrática de Timor Leste)
MCK – Mandi Cuci Kakus (Sanitary Community Facilities)

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[4] This presentation is based on a technical study carried out in 2014/2015 to the government of East Timor by AdP - Águas de Portugal (www.adp.pt)

MOPTC –Ministry of Public Works, Transport and Communication (Ministério das Obras Públicas, Transportes e Comunicações)
PMSDD/DSDMP –Dili Sanitation and Drainage Masterplan (Plano Mestre de Saneamento e Drenagem de Dili)
RAC - Anaerobic Reactor Compartmented (Reactor Anaeróbico Compartimentado)
SNC –National Bureau of Land Registration (Serviço Nacional de Cadastro)

INTRODUCTION

Dili Sanitation Masterplan

In October of 2010 the Government of Victoria - Australia made a service offering to GRDTL for the preparation of a Master Plan of Sanitation and Drainage to Dili. In charge of this study were the companies - Melbourne Water and Yarrey Valley.

Between October of 2011 and September of 2012, the consultant in engineer Sinclair Knight Merz (SKM) outsourced by Melbourne Water and Yarrey Valley, produced a series of technical reports under the direction of two Australian water authorities and in cooperation with the GRDTL. During this period, two sanitation pilot projects were developed and implemented.

Phase 1 2011-2012	Phase 2 2013-2017	Phase 3 2018-2025
Development of the Master Plan and two Pilot Projects	(CSTS – Community Sewage Treatment System) Implementation of community sanitation facilities and small-scale sewage treatment plants (CSTS – Community Sewage Treatment System)	
	Decentralized wastewater treatment Station (average - large scale) in Comoro	New Seven decentralized stations for wastewater treatment (medium scale)

Table 1 - Phasing the expected improvements

The recommended infrastructure improvements to Dili, include:

- Community Sanitation Facilities (MCK)
- Community Sewage Treatment Systems (CSTS)
- Sewage Treatment Plants

The goal of short-medium term for the Sanitation Master Plan is the implementation of community facilities of sewage treatment, since it ensure the sewage treatment locally, being the most effective means of ensuring sanitation improvements in Dili, under the current conditions, at little cost and with almost immediate effect, after its construction phase.

Sanitation Pilot Projects in Phase 1

These infrastructures consist of bathrooms, showers, laundries and a sewage treatment plant, shared within small communities. Both Community sanitation pilot projects were implemented as part of Phase 1 of PMSDD in Sucos Bidau Santana and Mascarenhas.

Suco Bidau Santana,Village Mota Klaran

The project covers about 20 dwellings. It was built a Community health facility (MCK - Mandi Cuci Kakus) with 3 toilets, 2 showers and a laundry area (see Figure 1); was made a new water hole and built a RAC (compartmented anaerobic reactor).

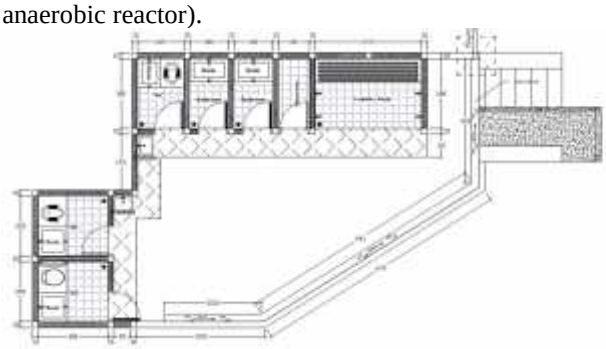

Figure 1 - Drawing of MCK on Bidau Santana showing toilets, showers and

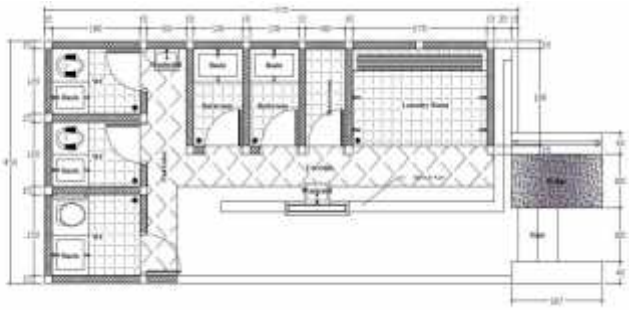

Figure 2 - Drawing of MCK on Mascarenhas showing toilets, showers and laundry

Suco Mascarenhas, Village 03

The project covers about 7 dwellings. A community health facility was built (MCK - Mandi Cuci Kakus) with 3 toilets, 2 showers and a laundry area (see Figure 2); was made a new water hole and built a RAC (compartmented anaerobic reactor).

The sewers of MCK are treated at a wastewater treatment plant (under the MCK), and the treated effluent is discharged to a drainage channel or the nearest river. The type of treatment recommended for these facilities is the anaerobic reactor compartmentalized - RAC (see Figure 3). The sludge must be removed every 2-3 years in tank trucks and taken to the WWTP - Wastewater Treatment Plant in Tibar.

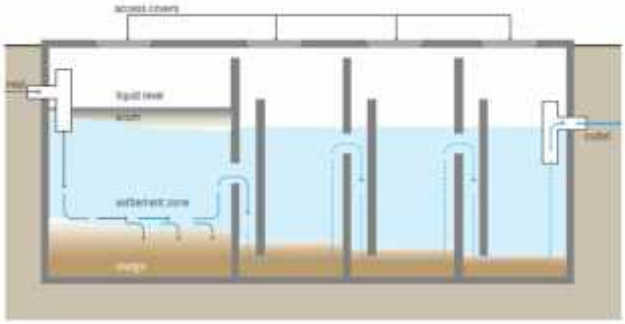

Figure 3 - Anaerobic Compartmentalized Reactor (RAC), usually built in underground level

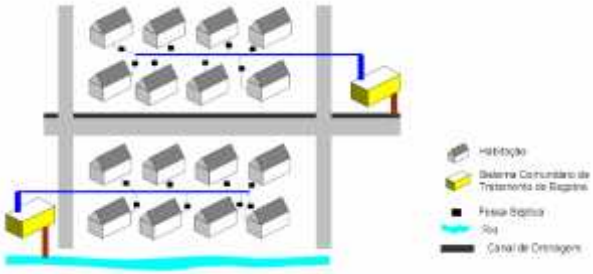

Figure 4 – Typical Community Sewage Treatment System for a residential area

Recommended Improvements to Dili - Phases 2 and 3

Community Sanitary Facilities

Bathrooms, showers, laundries and sewage treatment facilities, shared in small communities (see Figure 1 and 2). This infrastructure will be similar to the infrastructure of the two sanitation pilot projects implemented as part of Phase 1 of PMSDD.

Community Sewage Treatment System

Small and medium-sized sewage treatment plants, used to treat sewage in residential areas of small and medium size.

The pipes of waste water from toilets, bathrooms, kitchens and / or laundry rooms in each house are connected (via individual household septic tanks) to a shared sewage treatment plant for approximately 10 to 500 dwellings (see Figure 4).

A Typical Community Sewage Treatment System for a residential area includes:

1. Collection of sewage in a solid intersection tank or septic tank
2. Wastewater Transfer in small diameter pipes for a community station of wastewater treatment.
3. Treatment of wastewater and disinfection of the effluent treated to reduce pathogens.
4. Discharge of the treated effluent to the drainage channels and / or rivers.
5. Sludge collection in tank trucks and discharge at the Tibar Treatment Station.

Wastewater Treatment Plants

Large wastewater treatment plants, used to treat the wastewater of large residential and commercial areas. The channeling of sewage from households is connected to a wastewater treatment plant, shared by approximately 10,000 households (see Figure 5).

Figure 5 - A Decentralized Wastewater Treatment Station (large)

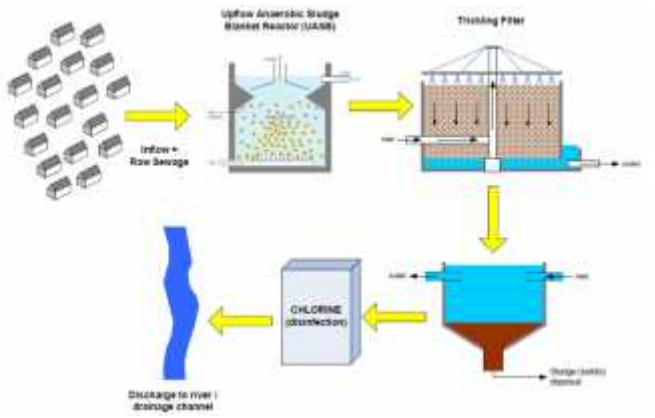

Figure 6 - Provisions of Decentralized Wastewater Treatment Station (large)

A typical wastewater treatment plant for a large residential area (up to approximately 10,000 dwellings) includes:

1. Collection of sewage in a solid intersection tank or septic tank
2. Wastewater Transfer in small diameter pipes for a community station of wastewater treatment.
3. Pumping of wastewater through the sewage network to the wastewater treatment plant (STP)
4. Treatment of wastewater and disinfection of the treated effluents
5. Discharge of the treated effluent to the drainage channels and/or rivers.

Some of the community wastewater treatment systems can be connected to decentralized wastewater treatment plants allowing them, in the future, to be dismantled.
Due to the large volumes of waste water to be treated on a decentralized unit of sewage treatment, a different technology is needed.
The recommended technology for eight decentralized wastewater treatment plants in Dili is a treatment process with three stages, involving:

a) A Anaerobic Reactor of upflow sludge blanket,
b) A biological filter,
c) A Tank of Secondary Sedimentation.

Investment Plan

In the investment plan, presented to the Council of Ministers on the 18th of October 2012, are listed the following investments relating to sanitation:

Sanitation Infrastructures in Phase 2 (2013-2017)

Most of the infrastructures to be implemented in phase 2 will consist of a large number of Community wastewater Treatment Systems (approximately 50 in phase 2).

Are also recommended a small number of community facilities, and a decentralized wastewater treatment station (serving approximately 3,000 dwellings).

The total estimated cost for improvements in sanitation facilities in Phase 2 is approximately of $ 75 million.
At the end of Phase 2, about 9,500 families will be covered.
The Master Plan for Sanitation should be reviewed and updated during Phase 2.

Sanitation Infrastructures in Phase 3 (2018-2025)

The implementation of community sewage treatment systems and community health facilities will continue in Phase 3.

For Phase 3 there are additional significant infrastructures proposals: seven Sewage Treatment Decentralized Stations in Dili, serving residential, commercial and industrial customers.

The total estimated cost for improvements in sanitation infrastructures in Phase 3 is approximately of $ 400 million.
By the end of Phase 3, about 43,000 households will be covered.

The Marterplan concerning Sanitation should be reviewed and updated during Phase 3.

Sanitation Infrastructures in Phase 4 (from 2025)

The Sanitation Master Plan should be reviewed and updated at the end of each phase, if it proves necessary.
In this phase, the GRDTL will decide, whether or not, if wants to rationalize its sewage treatment systems, changing from 8 decentralized wastewater treatment stations to a centralized treatment system with 2 or 3 large centralized sewage treatment plants, with an cost approximately of $ 80 million., or build a separative network and bring all sewage to Tibar (then several pumping stations will be necessary), having to remodel and increase the treatment capacity of the WWTP in Tibar.

BASIC DATA AND FORESEEN SOLUTIONS FOR THE IMPLEMENTATION OF THE DILI MASTERPLAN OF SANITATION AND DRAINAGE - PHASE 2

In the Dili Masterplan for Sanitation and Drainage the following data are considered:

Characteristics of Wastewater

Wastewater from households

Pollution loads

In the PMSDD are presented typical capita values for the pollution loads from domestic sources.

Parameters	Median			Ranges	
	25%ile	50%ile	75%ile	Low	High
BOD	38.5	41.5	49	25.5	56
COD	78.8	110	129.1	47.5	180
COD/BOD	2.0	2.7	2.6	1.9	3.2
NITROGEN	10	12.7	14.2	4.4	20.5
PHOSPHORUS	1.6	1.9	2.5	0.7	3.4

Table 2 - Summary of pollutants loads of a typical domestic wastewater

From the analysis performed to capitations used in several Asian countries, and overall design study to Dili drafted in 2003, it is proposed a capitation 35 g / inhab.day for BOD5, and AdP estimated and considered in its Calculations pollutants charges associated with contributions of COD, TSS, total N and Ptotal, according to international standards or guidelines.

Projection of Wastewater Volumes and their Characteristics

Contributions from households

The PMSDD considered a consumption of drinking water of 125 L / inhabitants.day and a residual water coefficient of 80% of water inflow, whereby the wastewater capitation will be of 100 L / inhabitants.day, but by option of DNSB was used an expected water consumption of 150L / inhabitants.day, and is this last value of water consumption which is considered in all calculations.

Infiltration Contributions

The PMSDD proposes a value for the infiltration of 9 m3 / ha / day, due to the existing high phreatic level in Dili.
The wastewater flows considered to Dili in the PMSDD are as follows:

Year	Area	Population	Potential BOD Load from Priority Areas (Tonnes/day)			
			Domestic[1] Cubic m/d	Commercial Cubic m/d	Septage[2] Cubic m/d	Total [3] Cubic m/d
2010	Priority Area 1	51,362	5,136	3,424	4,580	13,151
	Priority Area 2	70,900	7,090	4,727	9,424	21,257
	Priority Area 3	26,700	2,670	1,780	4,972	9,428
	Total	148,962	14,896	9,931	18,976	43,836
2017	Priority Area 1	59,826	5,983	3,988	4,580	14,565
	Priority Area 2	90,972	9,097	6,065	9,424	24,608
	Priority Area 3	34,603	3,460	2,307	4,972	10,747
	Total	185,401	18,540	12,360	18,976	49,920
2025	Priority Area 1	67,674	6,767	4,512	4,580	15,875
	Priority Area 2	120,956	12,096	8,064	9,424	29,614
	Priority Area 3	46,507	4,651	3,100	4,972	12,735
	Total	235,137	23,514	15,676	18,976	58,224

Table 3 - Summary of expected flows to Dili

Projection of the affluent organic loads

The PMSDD presents the estimate of the affluent organic loads, from households, commercial and septic tanks (resulting from sludge removed).

Year	Area	Population	Potential BOD Load from Priority Areas (Tonnes/day)			
			Domestic[1]	Commercial[2]	Septage	Total [3]
2010	Priority Area 1	51,362	1.8	1.7	0.1	3.6
	Priority Area 2	70,900	2.5	2.4	0.1	4.9
	Priority Area 3	26,700	0.9	0.9	0.0	1.9
	Total	148,962	5.2	5.0	0.2	10.4
2017	Priority Area 1	59,826	2.1	2.0	0.1	4.2
	Priority Area 2	90,972	3.2	3.0	0.1	6.3
	Priority Area 3	34,603	1.2	1.2	0.0	2.4
	Total	185,401	6.5	6.2	0.3	12.9
2025	Priority Area 1	67,674	2.4	2.3	0.1	4.7
	Priority Area 2	120,956	4.2	4.0	0.2	8.4
	Priority Area 3	46,507	1.6	1.6	0.1	3.2
	Total	235,137	8.2	7.8	0.3	16.4

1. Domestic is based on 35 g/p/d
2. Commercial is based on 500 mg/L
3. Total excludes contribution from infiltration and industrial

Table 4 - Summary of the proposed organic loads to Dili

General Criteria Design

In the PMSDD are presented some of the general criteria for the design considered in the preparation of the plan.

Stormwater - The sewerage system will be separated from the stormwater system. However, the infiltration is considered to be a contribution to the design of the system components.

Grey Waters - The system includes gray water from the baths, laundry, cleaning and cooking. Gray water must be drained to the septic tank or the solids interceptor tank and then into the sewage system.

Septic tanks - For the properties that will be linked to a community treatment plant it will be built an interceptor tank of solids as pre-settlement tank.

Project - When Manning formula is used for the design of sewer is suggested the use of a pipe roughness coefficient (n) of 0.013.

Tip factor – it is suggested the value of 2 to the sewage flow passing through the solids interceptor tank;

Main collector - It is suggested a minimum speed of 0.7 m / s at the end flow conditions and 0.6 m / s in initial flow conditions to ensure self-cleaning. The main collector to transport the wastewater to decentralized treatment plants is designed to meet the total population of the drainage basin.

Maximum speed - A maximum of 3 m / s to avoid damaging the tubes due to washing;

Pumping - The use of pumping networks should be avoided.

Rules on the disposal of wastewater - Timor-Leste currently has no standards for the discharge of treated wastewater. In this case we used the WHO standards shown in Table 5.

Parameter	Units	WHO Guideline Low	WHO Guideline High
pH	mg/L	7	10
BOD5 (unfiltered)	mg/L		50
COD	mg/L		100
TSS	mg/L		
NH3-N	mg/L		10
O-PO4	mg/L		30
TP	mg/L		30
S(2-)	mg/L		0
SO4(2-)	mg/L		50
Conductivity	uS/cm	100	
Faecal Coliforms	CFU/100 mL		<1000

Table 5 - WHO Guidelines for the use of treated wastewater in agriculture

Treatment systems to be adopted in Dili

In the PMSDD is performed a technical-economic comparison of several possible treatment alternatives, according to a set selected matrix.

For the Community sewage treatment systems (CSTS), to be implemented at the beginning of Phase 2, the proposed solutions are as follows:

Clinic of Bairro Pite – Suco of Bairro Pite

Figure 7 – Plant of Bairo Pite Clinic

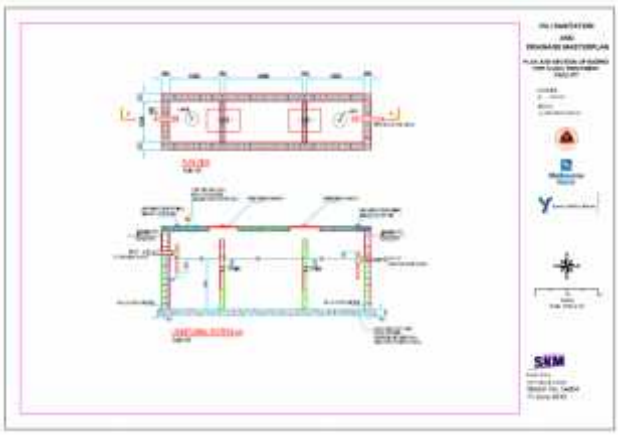

Figure 8 - Plant and profile of the treatment facility of Bairro Pite

Basic School Fatumeta – Suco Bairro Pite

Figure 9 – Plant of the Basic School Fatumeta

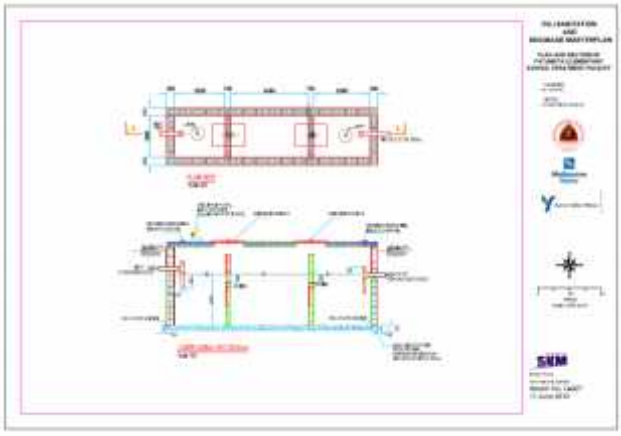

Figure 10 - Plant and profile of the Basic School treatment

Village Moris Ba Dame - Suco Bairro Pite

Figure 11 - Plant Moris Ba Dame

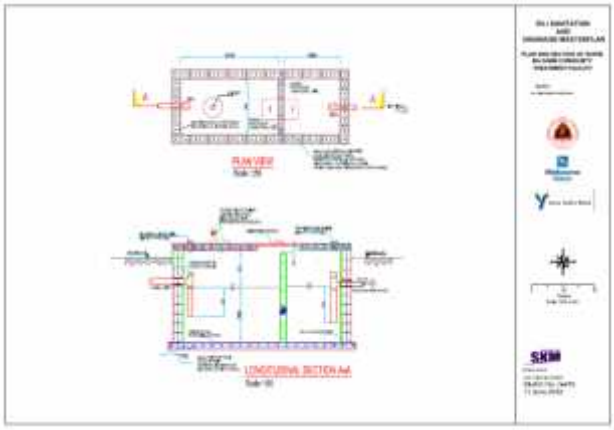

Figure 12 - Plant and profile of Moris Ba Dame treatment facility

Village 01 Matan Ruak - Suco Fatuhada

Figure 13 – Fatuhada Plant

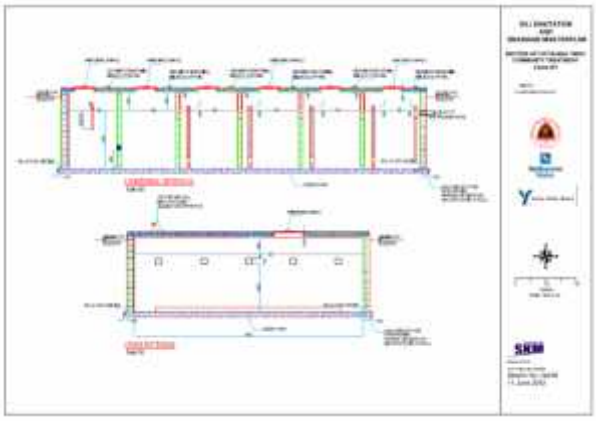

Figure 14 - Plant and profile of Fatuhada treatment facility

For the sewage treatment plant was proposed as an eligible solution an anaerobic reactor system of up flow sludge blanket (UASB) + percolator bed, serving various factors such as smaller footprint, low cost, easer operation and robustness in the treatment of wastewater.

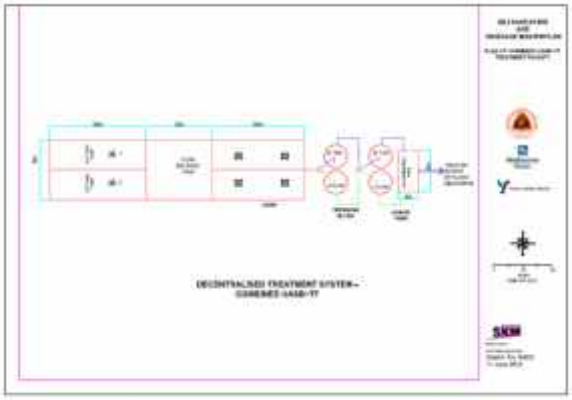

Figure 15 - Scheme of the Treatment Plant

Final Destination for Produced Sludge

In the PMSDD the final destination for the sludge produced is a landfill.

PROJECT DEVELOPMENT

Evaluation of the places defined for the implementation of the Community Sewage Treatment Systems (CSTS)

Several visits were made to the proposed sites for the purpose of reevaluate local conditions and verify the applicability of the proposed methodology in the study.

Clinic of Bairro Pite– Suco of Bairro Pite

Despite having connection to the public water supply, the clinic still needs to buy the necessary water, since the public service does not guarantee a minimum supply to ensure the filling of existing reservoirs.

The wastewater generated by the toilet blocks are forwarded to sinks. One of the sinks overflows always whenever more intense precipitation events occur, flooding the maternity ward. This is due to the fact that several buildings are linked to the sink, when it should only be connected to the toilet blocks.

In the preliminary phase of the study, ADP-TL agreed with the treatment solution proposed in the PMSDD and with the intended location, proposing an adjusting treatment for the effluent that comes from the anaerobic treatment using a biological filter (or percolator-bed) of low load and a final disinfection with chlorine (sodium hypochlorite) before the discharge into the receiving environment.

After new visits to the locations and contacts established with various entities, it was concluded that the major constraint to the implementation of the treatment system is the lack of land, as there is no adjoining land available and the clinical campus is practically all built . Thus, it can be concluded that the type of treatment for this location should be a septic tank, after a following disinfection and finally the treated effluent discharge to Maloa River.

Basic School Fatumeta – Suco Bairro Pite

The school is supplied by water services, but also has a hole that secures the supply in case of failure on the public network.

The school is served by two toilet blocks.

The wastewater generated by the two toilet blocks are forwarded to the respective absorption wells.

In the preliminary phase of the study, ADP-TL agreed with the treatment solution proposed in the PMSDD and with the intended location, proposing an adjusting treatment for the effluent that comes from the anaerobic treatment using a biological filter (or percolator-bed) of low load and a final disinfection with chlorine (sodium hypochlorite) before the discharge into the receiving environment.

Due to new site visits and contacts with the various entities, a more complete treatment solution was chosen to be implemented, composed of septic tanks, and followed by a percolator filter and disinfection. For the discharge of treated effluent, and because a gravity discharge into the nearest channel is not possible, it was decided also to build infiltration wells as a resource in case of a power failure.

Village Moris Ba Dame - Suco Bairro Pite

The neighborhood is served by public water supply, but the service is poor not ensuring adequate supply, so the inhabitants have the need to buy water.

The houses have individual bathrooms, and there is a block of bathrooms serving the community center.
The waste water is forwarded to the respective absorption wells.

In the preliminary phase of study AdP-TL agreed to the treatment solution proposed in the PMSDD but not with the expected location, as this would prejudge the future connection to the decentralized treatment system that will serve a larger area due to the depth of excavation required to achieve dimensions compatible with a gravity drainage and on the other hand, this location would make it more difficult for sludge extraction operation, due to the narrow access not allowing the passage of truck cleaner tanks. For the discharge of effluent from the anaerobic treatment has been proposed an adjustment treatment using a biological filter (or bed percolator), or a low load final disinfection with chlorine (sodium hypochlorite) before discharge into the receiving environment.

Due to new visits to the location and contacts with the various entities, and given that it is a small community, a simplified treatment regimen was chosen for this location , consisting of a septic tank followed by disinfection and effluent discharge treated into the nearest channel.

Village 01 Matan Ruak - Suco Fatuhada

The neighborhood is served by a public network water supply, but the service is poor and it does not guarantee an adequate supply, therefore the inhabitants have the need to buy water and most homes have their own holes to ensure the supply.

For the domestic wastewater generated, it is mostly forwarded to absorption wells and only a few households have septic tank.

Some additional dwellings were identified.

In the preliminary phase of the study, ADP-TL agreed with the treatment solution proposed in the PMSDD and with the intended location, proposing an adjusting treatment for the effluent that comes from the anaerobic treatment using a biological filter (or percolator-bed) of low load and a final disinfection with chlorine (sodium hypochlorite) before the discharge into the receiving environment.

After new visits to the locations and contacts established with various entities, it was concluded that it was not possible the implementation of the treatment system in the location originally projected, because of a lack of land, so it was proposed a new location in the vicinity, on land apparently available. Given the significant size of the community, the implementation of a more complete treatment solution was decided, made up of septic tanks, followed by sand filtration and disinfection, and the treated effluent discharged by gravity to an existing channel nearby.

Characteristics of the wastewater

Wastewater from households

Pollution loads

AdP-TL proposed in its Preliminary Report, the following values:

- CBO – 35 g/inhabitants.day; CQO – 74 g/inhabitants.day; SST – 45 g/inhabitants.day
- Ntotal – 8 g/inhabitants.day; Ptotal – 1 g/inhabitants.day

After several discussions, the values accepted for pollutant loads were as follows:

- CBO – 40 g/inhabitants.day; CQO – 85 g/inhabitants.day; SST – 45 g/inhabitants.day
- Ntotal – 8 g/inhabitants.day; Ptotal – 1 g/inhabitants.day

Design of wastewater volumes and their characteristics

The Consultant (AdP-TL) proposed in its preliminary report the following projections:

Contributions from households

With regard to the per capita amount of wastewater influent to the network, it is proposed a reduction in values, but was used capita water consumption has already been indicated previously in this Report, by decision of DNSB, which was 150 l /inhab.day

Infiltration Contributions

It was proposed to keep a value for the infiltration of 9 m3/inhab.day, due to the high phreatic level existing in Dili as in PMSDD.

The following table summarizes the proposed revision to the wastewater flow in Dili presented by the Consultant.

			Projeções de caudal para Dili (M³/day)				
Ano	Área	População	Doméstico	Comercial	Infiltração	Fossas	Total
2010	Área prioritária 1	51362	4.109	1.644	4.580	11	10.344
	Área prioritária 2	70900	5.672	2.269	9.424	16	17.381
	Área prioritária 3	26700	2.136	854	4.972	6	7.968
	Total	**148962**	**11.917**	**4.766,8**	**18.976**	**33**	**35.693**
2017	Área prioritária 1	59826	4.786	1.914	4.580	14	11.295
	Área prioritária 2	90972	7.278	2.911	9.424	22	19.635
	Área prioritária 3	34603	2.768	1.107	4.972	8,0	8.856
	Total	**185401**	**14.832**	**5.933**	**18.976**	**44**	**39.785**
2025	Área prioritária 1	67674	5.414	2.166	4.580	16	12.175
	Área prioritária 2	120956	9.676	3.871	9.424	30	23.001
	Área prioritária 3	46507	3.721	1.488	4.972	12	10.193
	Total	**235137**	**18.811**	**7.524,4**	**18.976**	**58**	**45.369**

Table 6 - Projected wastewater flows to Dili

After several discussions, the values accepted for the calculation of the wastewater flow for dimensioning purposes, were the following:

Clinic of Pite Neighborhood

Potable water consumption (l/inhab.day): User Inpatient 20; User Ambulatory 10; Escorts 5; Staff 10
Influx coefficient 80%; infiltration 20%
Population to be served:
Users Inpatient 55; User Ambulatory 300; Escorts 438 (1 for each user in the clinic, 2.5 per hospitalized user); Staff 89

Basic School Fatumeta – Suco Bairro Pite

Potable water consumption 20l/inh.day; Influx coefficient 80%; Infiltration 20%
Population to be served: Registered students 1228; Staff 28

Moris Ba Dame e Fatuhada

Potable water consumption 150 L/inh.day; Influx coefficient 80%; Infiltration 50%;
Population to be served: household 7

General Criteria used for Dimensioning

The general criteria assumed for the design of treatment systems, are:

Septic tanks:

Hydraulic Retention Time: 1 to 2 days; Time between cleanings: 365 days; Digestion time: 60 days
Removal of CBO5 : 25%; Removal of SST: 50%

Sand Filter

Removal of CBO5: 80%; Removal of SST: 75%

Percolator filter

Removal of CBO5: 35%; Removal of SST: 35%

Disinfection

Content Fecal Coliforms entry 10 /100 ml; Content Fecal Coliforms exit 2000/100ml; Minimum Contact time 60 min

IMPLEMENTATION PROJECTS OF THE COMMUNITY SYSTEMS OF SEWAGE TREATMENT

For the 4 Community Sewage Treatment Systems (CSTS) their Implementation Projects have been prepared.
The Implementation Projects have the following content:

- Main Report
- Designed Parts
- Bill of Quantities

- Estimated budget
- Technical Specifications

The treatment solutions for the 4 community sewage treatment systems (CSTS) are as follows:

Clinic of Bairro Pite– Suco of Bairro Pite

For the treatment plant that will serve the Clinic of Bairro Pite was considered a treatment facility with the following sequence of unit operations:

Figure 16 - Diagram of the treatment solution for the WWTP of Bairro Pite Clinic

The Wastewater suffers a primary treatment, in the septic tank, where there is a removal of the suspended solids by flotation and sedimentation and its anaerobic digestion.

The effluent out of the septic tank shall be subject to disinfection, whose purpose is to reduce the pathogens before the effluent is released in Rio Maloa.

Figure 17 - Drainage Network of the Bairro Pite Clinic

Figure 18 - Plant of the WWTP of Bairro Pite Clinic

Basic School Fatumeta – Suco Bairro Pite

For the treatment plant that will serve the Basic School Fatumeta it was considered a treatment facility with the following sequence of unit operations:

Figure 19 - Diagram of the treatment solution for the WWTP of the Basic School Fatumeta

The Wastewater suffers a primary treatment, in the septic tank, where there is a removal of the suspended solids by flotation and sedimentation and its anaerobic digestion.

The effluent at the exit of the septic tank is subjected to a new treatment, in the percolator filter, which processes the aerobic digestion of organic matter and sewage removal of the suspended solids by filtration.

The effluent at the exit of the percolator filter is subject to disinfection, whose purpose is to reduce the pathogens before the effluent is released into the drainage channel, or in the infiltration wells.

Figure 20 - Drainage Networkof the Basic School Fatumeta

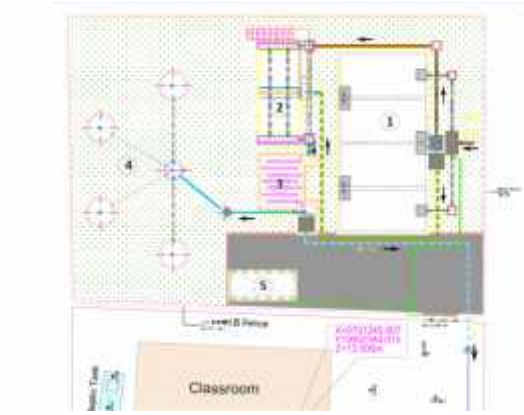

Figure 21 – Plant of the Wastewater Treatment Plant Basic School Fatumeta

Village Moris Ba Dame - Suco Bairro Pite

For the treatment plant that will serve Moris Ba Dame Village was considered a treatment facility using the following sequence of unit operations:

Figure 22 - Diagram of the treatment solution for the WWTP Moris Ba Dame

The Wastewater suffers a primary treatment, in the septic tank, where there is a removal of the suspended solids by flotation and sedimentation and its anaerobic digestion.

The effluent out of the septic tank shall be subject to disinfection, whose purpose is to reduce the pathogens before the effluent is released in the drainage channel.

Figure 23 - Drainage Network Moris Ba Dame

Figure 24 - WWTP plant of Moris Ba Dame

Village 01 Matan Ruak - Suco Fatuhada

For the treatment plant that will serve the Village 01 Matan Ruak was considered a treatment facility with the following sequence of unit operations:

Figure 25 - Diagram of the treatment solution for the WWTP Fatuhada

The Wastewater suffers a primary treatment, in the septic tank, where there is a removal of the suspended solids by flotation and sedimentation and its anaerobic digestion.

The effluent at the exit of the septic tank will be subjected to a new treatment, in the sand filter, which processes the aerobic digestion of organic matter and sewage removal of the suspended solids by filtration.
The effluent out of the sand filter shall be subject to disinfection, whose purpose is to reduce the pathogens before the effluent is released in the drainage channel.

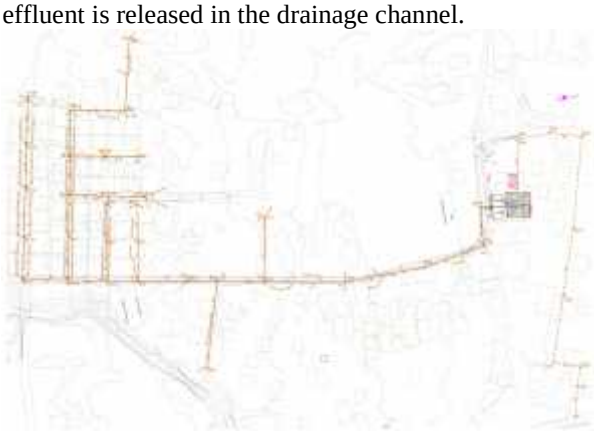

Figure 26 - Drainage Network Fatuhada

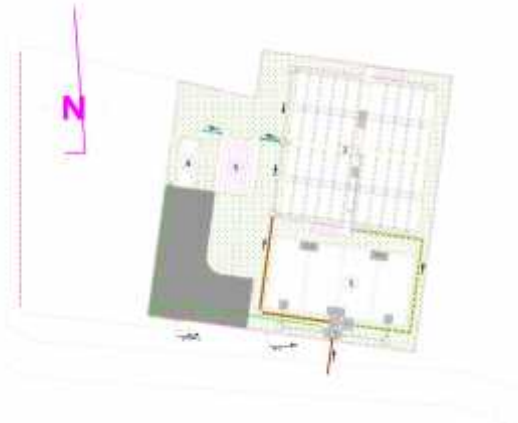

Figure 27 - WWTP Fatuhada

Estimated Cost in the Presented Solutions

IMPLEMENTAÇÃO DO PLANO DIRECTOR DE SANEAMENTO E DRENAGEM DE DÍLI - FASE 2
IMPLEMENTATION OF THE DÍLI SANITATION AND DRAINAGE MASTERPLAN - PHASE 2
CSTS | CSTS

ESTIMATIVA DE CUSTO | COST ESTIMATE
RESUMO | SUMMARY

		Bairro Pite Clinic	Fatumeta School	Moris Ba Dame	Fatuhada
1	ESTALEIRO E TRABALHOS PRELIMINARES CONSTRUCTION SITE AND PRELIMINARY WORKS	15.924	23.880	15.952	43.330
2	REDE DE COLECTORES DE DRENAGEM DE ÁGUAS RESIDUAIS SEWERAGE NETWORK	20.306	15.810	29.804	299.085
3	ESTAÇÃO DE TRATAMENTO DE ÁGUAS RESIDUAIS WASTEWATER TREATMENT PLANT	104.660	271.747	95.025	371.196
4	TRABALHOS FINAIS FINAL WORKS	13.881	14.544	14.586	16.699
	TOTAIS | TOTALS	154.770	325.982	155.366	730.310
	TOTAL | TOTAL		1.366.428		

Table 7 – Estimated Cost for Community Sewage Treatment System (CSTS)

LANDS

The existence of land available for building the infrastructures of sewage treatment plants, is a conditioning factor in the definition of treatment solutions to the four community sewage treatment systems (CSTS) defined and the viability of its construction.

For this reason, the Consultant initiated contacts with the National Directorate of Land, Property and Cadastral Services (DNTPSC) and with the National Registration Service (CNS), in order to verify the existence of land owned by Government, available for the construction of sewage treatment plants.

After several contacts, which began on 28th August, it was possible to schedule for 15th October, a joint visit with the National Directorate of Basic Sanitation (DNSB) and the National Directorate of Land, Property and Cadastral Services (DNTPSC), to the locations intended for community sewage treatment systems (CSTS).

The SNC had previously provided the plants and the list of parcels and their owners, identified by the Ita Nia Rai .

Area	Party_Id	Claim_Id	Name	Gender	Birth Place	Birth Date
MOP01	3605401	51-5.072.28780A	Venancio Alves Maria	M	Baucau/Baguia	26-May-71
	3606070	51-5.072.31001A	Adozindo Hermogenes Pedro Sequeira	M	Oecusse	01-May-61
MOP02	N/A	51-5.073.60630	N/A			
	N/A	51-5.073.60651	N/A			
	3604287	51-5.073.60637A	Lima Dinas Aeroporto (Estado)			
	3604285	51-5.073.60638A	Lima Dinas Aeroporto (Estado)			
	3604244	51-5.073.60639A	Lima Dinas Aeroporto (Estado)			
	3604306	51-5.073.60642A	Luis Ricardo	M	Maubara	06-Mar-46
MOP03	3801113	51-5.072.25920A	Lourenco Morais Fausto de Oliveira	M	Dili	10-Aug-37
MOP04	3603536	51-5.073.60305A	Angelo Henriques	M	Lautem	05-Nov-53

PARAMETER / SAMPLING LOCATION	ENTRY OF THE SEPTIC TANK	EXIT OF THE SEPTIC TANK	FINAL WASTEWATER
pH	weekly	weekly	weekly
Conductivity	weekly	weekly	weekly
Temperature	weekly	weekly	weekly
Dissolved Oxygen	weekly	weekly	weekly
CBO_5 (20°C)	Monthly	Monthly	Monthly
CQO	Monthly	Monthly	Monthly
Total Nitrogen	Monthly	Monthly	Monthly
Nitrates	Monthly	Monthly	Monthly
Total phosphorus	Monthly	Monthly	Monthly
SST	Monthly	Monthly	Monthly
Total coliforms	Fortnightly	Fortnightly	Fortnightly
Fecal coliforms	Fortnightly	Fortnightly	Fortnightly
E. Coli	Fortnightly	Fortnightly	Fortnightly

Table 8 - Owners of land plots identified by Ita Nia Rai

MONITORING

To control the treatment process in order to obtain the desired goals, it is necessary to have a permanently knowledge of the quality of the effluent during the various processing steps.

From the prior the theoretical knowledge of the various parts, it can be determined the expected quality of the effluent at the exit of body process.

The determination of quality through analytical characterization of samples, allows comparison of the real quality with the expected, and determine and eliminate the causes of any deviations detected.

It is therefore necessary to define a monitoring plan, which will demonstrate the degree of treatment achieved by the system.

The monitoring plan consists essentially of an analytical control plan, defining a set of analytical determinations to be made to the wastewater collected by the system at the intervals and on parameters set prior to discharge into the receiving environment.

Bairro Pite Clinic and Moris Ba Dame

For CSTS operational control should be taken samples at the locations and with the periodicities defined in the following table:

- Collection of composite samples (with autosampler and 24 hourly collection) of the CSTS effluent at the exit of the treatment, immediately upstream of the discharge in the receiving environment.
- Collection of instantaneous samples in all other locations;

Table 9 - set of parameters and frequency for control of Bairro Pite Clinic CSTS and Moris Ba Dame

CSTS of the Basic School Fatumeta and Fatuhada

For operational control of CSTS it should be taken samples in locations and with the periodicities defined in the following table:

- Collection of composite samples (with autosampler and 24 hourly collection) of the CSTS effluent at the exit of the treatment, immediately upstream of the discharge in the receiving environment.
- Collection of instantaneous samples in all other locations;

PARAMETER / SAMPLING LOCATION	ENTRY OF THE SEPTIC TANK	EXIT OF THE SEPTIC TANK	FILTER EXIT	FINAL WASTEWATER
pH	weekly	weekly	weekly	weekly
Conductivity	weekly	weekly	weekly	weekly
Temperature	weekly	weekly	weekly	weekly
Dissolved Oxygen	weekly	weekly	weekly	weekly
CBO_5 (20°C)	Monthly	Monthly	Monthly	Monthly
CQO	Monthly	Monthly	Monthly	Monthly
Total Nitrogen	Monthly	Monthly	Monthly	Monthly
Nitrates	Monthly	Monthly	Monthly	Monthly
Total phosphorus	Monthly	Monthly	Monthly	Monthly
SST	Monthly	Monthly	Monthly	Monthly
Total coliforms	Fortnightly	Fortnightly	Fortnightly	Fortnightly
Fecal coliforms	Fortnightly	Fortnightly	Fortnightly	Fortnightly
E. Coli	Fortnightly	Fortnightly	Fortnightly	Fortnightly

Table 10 - set of parameters and frequency for control of CSTS of the Basic School Fatumeta and Fatuhada

To further check the operability of 4 CSTS should also be made:

- Measurement of the sludge blanket height inside the Septic Tank, every three months (expedite method - for example, by dipping a wand with a degree or not);

Considering the set of parameters and the proposed schedule, it was established the analytical control plan, the main tool of the monitoring plan for each of the systems, according to the following tables, which will demonstrate their level of treatment.

Dia Amostragem	Semana 5			Semana 6			Semana 7			Semana 8		
Local Amostragem	Afluente a)	Saída FS	Efluente b)	Afluente a)	Saída FS	Efluente b)	Afluente a)	Saída FS	Efluente b)	Afluente a)	Saída FS	Efluente b)
Parâmetro												
pH	x	x	x	x	x	x	x	x	x	x	x	x
Condutividade	x	x	x	x	x	x	x	x	x	x	x	x
Temperatura	x	x	x	x	x	x	x	x	x	x	x	x
Oxigénio Dissolvido	x	x	x	x	x	x	x	x	x	x	x	x
CBO_5	x	x	x									
CQO	x	x	x									
Azoto Total	x	x	x									
Nitratos	x	x	x									
Fósforo Total	x	x	x									
SST	x	x	x									
Coliformes Totais	x	x	x				x	x	x			
Coliformes Fecais	x	x	x				x	x	x			
E. Coli	x	x	x				x	x	x			
Dia Amostragem	Semana 9			Semana 10			Semana 11			Semana 12		
Local Amostragem	Afluente a)	Saída FS	Efluente b)	Afluente a)	Saída FS	Efluente b)	Afluente a)	Saída FS	Efluente b)	Afluente a)	Saída FS	Efluente b)
Parâmetro												
pH	x	x	x	x	x	x	x	x	x	x	x	x
Condutividade	x	x	x	x	x	x	x	x	x	x	x	x
Temperatura	x	x	x	x	x	x	x	x	x	x	x	x
Oxigénio Dissolvido	x	x	x	x	x	x	x	x	x	x	x	x
CBO_5	x	x	x									
CQO	x	x	x									
Azoto Total	x	x	x									
Nitratos	x	x	x									
Fósforo Total	x	x	x									
SST	x	x	x									
Coliformes Totais	x	x	x				x	x	x			
Coliformes Fecais	x	x	x				x	x	x			
E. Coli	x	x	x				x	x	x			

Table 11 - Analytical Control Plan for CSTS Clinic of Bairro Pite

The plan is repeated in every 4 weeks cycle for the 52 weeks of the year.

MORIS BA DAME

PLANO DE CONTROLO ANALÍTICO

Dia Amostragem	Semana 1			Semana 2			Semana 3			Semana 4		
Local Amostragem	Afluente [a)]	Saída FS	Efluente [b)]	Afluente [a)]	Saída FS	Efluente [b)]	Afluente [a)]	Saída FS	Efluente [b)]	Afluente [a)]	Saída FS	Efluente [b)]
Parâmetro												
pH	x	x	x	x	x	x	x	x	x	x	x	x
Condutividade	x	x	x	x	x	x	x	x	x	x	x	x
Temperatura	x	x	x	x	x	x	x	x	x	x	x	x
Oxigénio Dissolvido	x	x	x	x	x	x	x	x	x	x	x	x
CBO_5							x	x	x			
CQO							x	x	x			
Azoto Total							x	x	x			
Nitratos							x	x	x			
Fósforo Total							x	x	x			
SST							x	x	x			
Coliformes Totais	x	x	x				x	x	x			
Coliformes Fecais	x	x	x				x	x	x			
E. Coli	x	x	x				x	x	x			

Notas:

a) À entrada da Fossa Séptica

b) À saída do tratamento

Dia Amostragem	Semana 5			Semana 6			Semana 7			Semana 8		
Local Amostragem	Afluente [a)]	Saída FS	Efluente [b)]	Afluente [a)]	Saída FS	Efluente [b)]	Afluente [a)]	Saída FS	Efluente [b)]	Afluente [a)]	Saída FS	Efluente [b)]
Parâmetro												
pH	x	x	x	x	x	x	x	x	x	x	x	x
Condutividade	x	x	x	x	x	x	x	x	x	x	x	x
Temperatura	x	x	x	x	x	x	x	x	x	x	x	x
Oxigénio Dissolvido	x	x	x	x	x	x	x	x	x	x	x	x
CBO_5							x	x	x			
CQO							x	x	x			
Azoto Total							x	x	x			
Nitratos							x	x	x			
Fósforo Total							x	x	x			
SST							x	x	x			
Coliformes Totais	x	x	x				x	x	x			
Coliformes Fecais	x	x	x				x	x	x			
E. Coli	x	x	x				x	x	x			
Dia Amostragem	Semana 9			Semana 10			Semana 11			Semana 12		
Local Amostragem	Afluente [a)]	Saída FS	Efluente [b)]	Afluente [a)]	Saída FS	Efluente [b)]	Afluente [a)]	Saída FS	Efluente [b)]	Afluente [a)]	Saída FS	Efluente [b)]
Parâmetro												
pH	x	x	x	x	x	x	x	x	x	x	x	x
Condutividade	x	x	x	x	x	x	x	x	x	x	x	x
Temperatura	x	x	x	x	x	x	x	x	x	x	x	x
Oxigénio Dissolvido	x	x	x	x	x	x	x	x	x	x	x	x
CBO_5							x	x	x			
CQO							x	x	x			
Azoto Total							x	x	x			
Nitratos							x	x	x			
Fósforo Total							x	x	x			
SST							x	x	x			
Coliformes Totais	x	x	x				x	x	x			
Coliformes Fecais	x	x	x				x	x	x			
E. Coli	x	x	x				x	x	x			

Table 12 - Analytical Control Plan for CSTS of Moris Ba Dame

The plan is repeated in every 4 weeks cycle for the 52 weeks of the year.

CSTS FATUMETA SCHOOL

PLANO DE CONTROLO ANALITICO

Dia Amostragem	Semana 1				Semana 2				Semana 3				Semana 4			
Local Amostragem	Afluente [a)]	Saída FS	Saída Filtro	Efluente [b)]	Afluente [a)]	Saída FS	Saída Filtro	Efluente [b)]	Afluente [a)]	Saída FS	Saída Filtro	Efluente [b)]	Afluente [a)]	Saída FS	Saída Filtro	Efluente [b)]
Parâmetro																
pH	x	x	x	x	x	x	x	x	x	x	x	x	x	x	x	x
Condutividade	x	x	x	x	x	x	x	x	x	x	x	x	x	x	x	x
Temperatura	x	x	x	x	x	x	x	x	x	x	x	x	x	x	x	x
Oxigénio Dissolvido	x	x	x	x	x	x	x	x	x	x	x	x	x	x	x	x
CBO_5					x	x	x	x								
CQO					x	x	x	x								
Azoto Total					x	x	x	x								
Nitratos					x	x	x	x								
Fósforo Total					x	x	x	x								
SST					x	x	x	x								
Coliformes Totais					x	x	x	x					x	x	x	x
Coliformes Fecais					x	x	x	x					x	x	x	x
E. Coli					x	x	x	x					x	x	x	x

Notas:

a) A entrada da Fossa Séptica

b) A saída do tratamento

Dia Amostragem	Semana 5				Semana 6				Semana 7				Semana 8			
Local Amostragem	Afluente [a)]	Saída FS	Saída Filtro	Efluente [b)]	Afluente [a)]	Saída FS	Saída Filtro	Efluente [b)]	Afluente [a)]	Saída FS	Saída Filtro	Efluente [b)]	Afluente [a)]	Saída FS	Saída Filtro	Efluente [b)]
Parâmetro																
pH	x	x	x	x	x	x	x	x	x	x	x	x	x	x	x	x
Condutividade	x	x	x	x	x	x	x	x	x	x	x	x	x	x	x	x
Temperatura	x	x	x	x	x	x	x	x	x	x	x	x	x	x	x	x
Oxigénio Dissolvido	x	x	x	x	x	x	x	x	x	x	x	x	x	x	x	x
CBO_5					x	x	x	x								
CQO					x	x	x	x								
Azoto Total					x	x	x	x								
Nitratos					x	x	x	x								
Fósforo Total					x	x	x	x								
SST					x	x	x	x								
Coliformes Totais					x	x	x	x					x	x	x	x
Coliformes Fecais					x	x	x	x					x	x	x	x
E. Coli					x	x	x	x					x	x	x	x

Dia Amostragem	Semana 9				Semana 10				Semana 11				Semana 12			
Local Amostragem	Afluente [a)]	Saída FS	Saída Filtro	Efluente [b)]	Afluente [a)]	Saída FS	Saída Filtro	Efluente [b)]	Afluente [a)]	Saída FS	Saída Filtro	Efluente [b)]	Afluente [a)]	Saída FS	Saída Filtro	Efluente [b)]
Parâmetro																
pH	x	x	x	x	x	x	x	x	x	x	x	x	x	x	x	x
Condutividade	x	x	x	x	x	x	x	x	x	x	x	x	x	x	x	x
Temperatura	x	x	x	x	x	x	x	x	x	x	x	x	x	x	x	x
Oxigénio Dissolvido	x	x	x	x	x	x	x	x	x	x	x	x	x	x	x	x
CBO_5					x	x	x	x								
CQO					x	x	x	x								
Azoto Total					x	x	x	x								
Nitratos					x	x	x	x								
Fósforo Total					x	x	x	x								
SST					x	x	x	x								
Coliformes Totais					x	x	x	x					x	x	x	x
Coliformes Fecais					x	x	x	x					x	x	x	x
E. Coli					x	x	x	x					x	x	x	x

Table 13 - Analytical Control Plan for CSTS of Fatumeta School

The plan is repeated in every 4 weeks cycle for the 52 weeks of the year.

FATUHADA
PLANO DE CONTROLO ANALITICO

Dia Amostragem	Semana 1				Semana 2				Semana 3				Semana 4			
Local Amostragem	Afluente [a)]	Saída FS	Saída Filtro	Efluente [b)]	Afluente [a)]	Saída FS	Saída Filtro	Efluente [b)]	Afluente [a)]	Saída FS	Saída Filtro	Efluente [b)]	Afluente [a)]	Saída FS	Saída Filtro	Efluente [b)]
Parâmetro																
pH	x	x	x	x	x	x	x	x	x	x	x	x	x	x	x	x
Condutividade	x	x	x	x	x	x	x	x	x	x	x	x	x	x	x	x
Temperatura	x	x	x	x	x	x	x	x	x	x	x	x	x	x	x	x
Oxigénio Dissolvido	x	x	x	x	x	x	x	x	x	x	x	x	x	x	x	x
CBO_5													x	x	x	x
CQO													x	x	x	x
Azoto Total													x	x	x	x
Nitratos													x	x	x	x
Fósforo Total													x	x	x	x
SST													x	x	x	x
Coliformes Totais					x	x	x	x					x	x	x	x
Coliformes Fecais					x	x	x	x					x	x	x	x
E. Coli					x	x	x	x					x	x	x	x

Notas:
a) À entrada da Fossa Séptica
b) À saída do tratamento

Dia Amostragem	Semana 5				Semana 6				Semana 7				Semana 8			
Local Amostragem	Afluente [a)]	Saída FS	Saída Filtro	Efluente [b)]	Afluente [a)]	Saída FS	Saída Filtro	Efluente [b)]	Afluente [a)]	Saída FS	Saída Filtro	Efluente [b)]	Afluente [a)]	Saída FS	Saída Filtro	Efluente [b)]
Parâmetro																
pH	x	x	x	x	x	x	x	x	x	x	x	x	x	x	x	x
Condutividade	x	x	x	x	x	x	x	x	x	x	x	x	x	x	x	x
Temperatura	x	x	x	x	x	x	x	x	x	x	x	x	x	x	x	x
Oxigénio Dissolvido	x	x	x	x	x	x	x	x	x	x	x	x	x	x	x	x
CBO_5													x	x	x	x
CQO													x	x	x	x
Azoto Total													x	x	x	x
Nitratos													x	x	x	x
Fósforo Total													x	x	x	x
SST													x	x	x	x
Coliformes Totais					x	x	x	x					x	x	x	x
Coliformes Fecais					x	x	x	x					x	x	x	x
E. Coli					x	x	x	x					x	x	x	x

Dia Amostragem	Semana 9				Semana 10				Semana 11				Semana 12			
Local Amostragem	Afluente [a)]	Saída FS	Saída Filtro	Efluente [b)]	Afluente [a)]	Saída FS	Saída Filtro	Efluente [b)]	Afluente [a)]	Saída FS	Saída Filtro	Efluente [b)]	Afluente [a)]	Saída FS	Saída Filtro	Efluente [b)]
Parâmetro																
pH	x	x	x	x	x	x	x	x	x	x	x	x	x	x	x	x
Condutividade	x	x	x	x	x	x	x	x	x	x	x	x	x	x	x	x
Temperatura	x	x	x	x	x	x	x	x	x	x	x	x	x	x	x	x
Oxigénio Dissolvido	x	x	x	x	x	x	x	x	x	x	x	x	x	x	x	x
CBO_5													x	x	x	x
CQO													x	x	x	x
Azoto Total													x	x	x	x
Nitratos													x	x	x	x
Fósforo Total													x	x	x	x
SST													x	x	x	x
Coliformes Totais					x	x	x	x					x	x	x	x
Coliformes Fecais					x	x	x	x					x	x	x	x
E. Coli					x	x	x	x					x	x	x	x

Table 14 - Analytical Control Plan for CSTS of Fatuhada

The plan is repeated in every 4 weeks cycle for the 52 weeks of the year.

The global analytical control plan, established as the main instrument of the monitoring plan, is attached to this report and is in digital format.
All results, the analysis performed as the analytical plan established, shall be duly recorded in the tables that follow, so that you can obtain the efficiencies of the treatment of each Community sewage treatment system.
The calculation of average removal efficiencies will be automatically obtained by the introduction of the values obtained in each analysis and for each of the parameters.

PILOT PROJECTS - MCK (MANDI-CUCI-KAKUS) - CURRENT CONDITIONS

Several visits were made to the locations where the pilot projects of community toilets were implemented.

Location [1]: Village "Motaklaran" Suco "Bidau Santana," Sub-District "Cristo Rei".

The facility is in poor condition and shows some signs of vandalism, so that its use is limited and less suitable.
The following issues were identified:

The discharge pipe from the septic tank is damaged, and sewage is discharged in the surrounding area (high risk of contamination of drinking water), instead of being forwarded to the river;

Figure 32 - Discharge pipe of septic tank is damaged

Figure 33 - surrounding area flooded with sewage

Several water taps of the toilet were not functional;

Figure 34 – Damaged tap

Figure 35 – Visible lack of maintenance

Lack of regular maintenance of the structure; The water pump was not connected.

Figure 36 - Water Pump

Figure 37 - Direct Discharge For The Ditch

It appears that there is no involvement of the community and as a result there is no responsibility for its maintenance.

There are no economic resources and technical expertise in the community to ensure the regular operation of the facility.

It does not exist involvement by government institutions in the maintenance of this infrastructure.

Location [2]: Village "Zero Tres (03)" Suco"Mascarenhas," Sub-District "Vera Cruz"

The facility is in service and maintenance is in a satisfactory condition. Despite the community's economic resources being very scarce, some interventions have been made in the infrastructure ensuring its proper functioning.

The main interventions stood at the level of replacing some damaged taps and repair the water pump.

The following issues were identified:

The water supply is not regular;

The available water does not undergo any disinfection treatment nor is the population sensitive to the beneficial effects of the use of disinfectant in water that is used;

The access to infrastructure is very narrow, preventing car access to extract sludge from the anaerobic reactor;

The discharge of wastewater is not adequately protected, allowing easy access, particularly for children, which constitutes a high health risk.

Recommendations for future projects

It is foreseen in the PMSDD the future implementation of similar facilities, so it should be taken into account the following aspects, so that we can improve the operating conditions of this type of infrastructure:

The management of infrastructure should be left to the National Directorate of Basic Sanitation (DNSB), appointing a representative from the local community as a link, to ensure the integrity of the installation and report to DNSB the intervention needs as necessary.

The community can be engaged the to carry out small jobs of simple maintenance, but which do not require skilled labor.

It will be essential to define a model for the management of these infrastructures, identifying stakeholders and their respective responsibilities.

The location of the infrastructure is of utmost importance in the sense that access should be guaranteed, not only for the maintenance work that need to be performed, but also for cleaning the sludge by drain cleaning vehicles.

The design features that prevent degradation and easy damage to the infrastructure, such as pipelines in sight and not protected must be observed, as in the case of Bidau Santana (damaged discharge pipe - see Figure 32) and dangerous situations to public health , as in the case of Mascarenhas (open discharge - see Figure 37).

Must be ensured a disinfecting system of the water used.

PROPOSAL FOR NEW COMMUNITY FACILITIES - PHASE 2

Framework

Following the contract established for the preparation of 4 pilot projects for CSTS, the DNSB decided to extend this contract and request the AdP TL to assess the possibility of implementing new Community Systems of Sewage Treatment in other communities.

Selection Criteria

The selection of the location for the implementation of CSTS (Step 1) was based on the following criteria:

- ☹ water supply available
- ☹ availability of land
- ☹ community willingness to pay
- ☹ community participation in the operation and maintenance of the treatment system

In practice, these assumptions were not met for 4 CSTS defined in Phase 1, so it will be necessary to pay more attention to these criteria for the choice of new CSTS in Phase 2.

Thus, it was decided by DNSB as the main condition, that the implementation of the new CSTS would be carried out in areas where they are to achieve improvements in water supply.

Similarly, and due to difficulties in the future implementation of the 4 CSTS already defined, it was considered that the availability of land would be a prerequisite for the choice of new locations.

For this phase of work, and in order to make a decision, it will be considered as leading the following weighting criteria:

- ❖ Availability of water supply
- ❖ Availability of land
- ❖ Accessibility for O & M

- ❖ The existence of inadequate sanitation
- ❖ Population density

There are other weighting criteria, of social relevance, that are also important to be considered, such as:

- – The community acceptance
- – The community's willingness to pay
- – The community's willingness to participate in the operation and maintenance of the infrastructure

It is considered however, that these criteria are subjective and subject to change, which will allow, with a serious work and programmed within the community development, we can ensure acceptance and community availability, since previously we could ensure acceptance and availability from the local leaders.

Intervention Area

As a starting point, it was necessary to define the area of intervention.

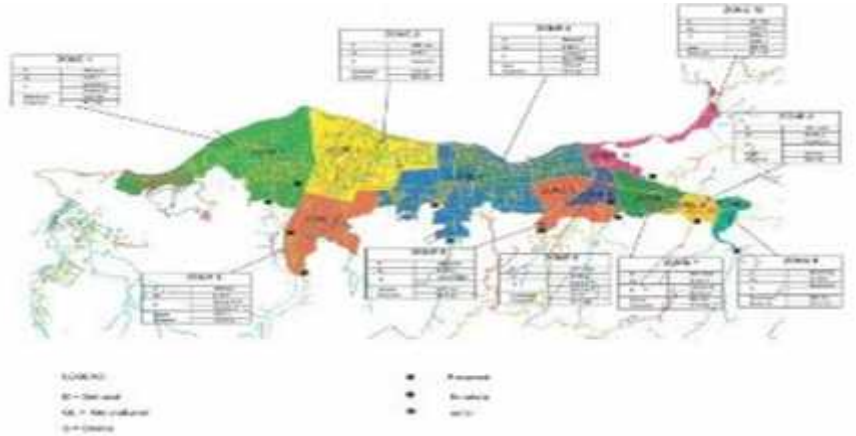

Figure 38 - National Directorate of Water Services (DNSA) - Water Supply Zones

With the National Directorate of Water Services (DNSA) was possible to identify two areas where improvements have already been introduced in the water supply. These zones are Zone 1 - Airport and Zone 10 - Bidau Santana and Metiaut. According to information given by the responsible of DNSA, Zone 10 is more controlled and more stable in terms of supply and also in community involvement issues.

According to the DNSA information and taking into account the dimensions of the two zones, DNSB chose Zone 10, because it has better water supply conditions and is of smaller dimensions, given the time limit for completion of the work.

Project Development

Zone 10 covers two Sucos: Bidau Santana and Metiaut.

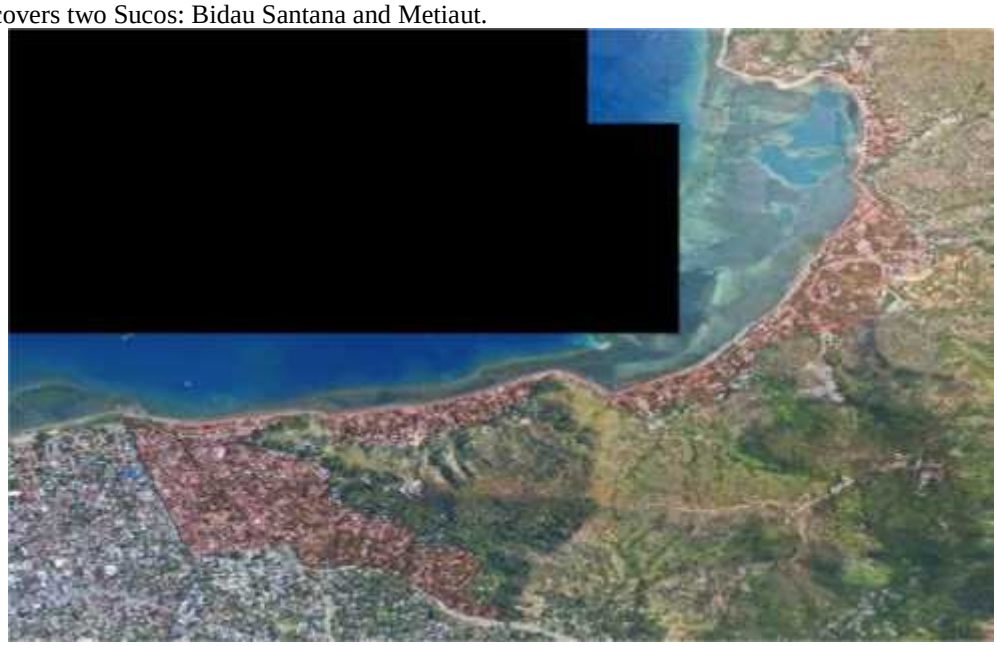

Figure 39 – Area of Zone 10: Sucos Bidau Santana and Metiaut

Bidau Santana Suco consists of the Sagrada Familia Village, Manumata, Mota klaran and Toko Baru 1. Metiaut Suco has 3 Villages: 17 de Abril, Carungulau and Fatucama.
The area served by the water supply network is identified below.

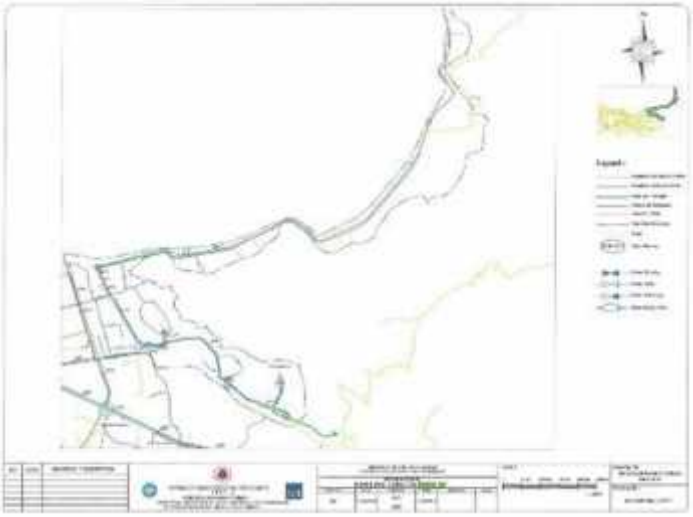

Figure 40 - Zone 10 - Water Supply Network

For these two Sucos, it was obtained from DNSA the number of consumers and consumption of water.

REKAPITULASAUN DADUS KONSUMIDOR BEE MOOS
SUCO BIDAU SANTANA, SUB-DISTRITO CRISTO REI

UPDATE 2014

NO.	ALDEIA	No. KONSUMIDOR	Dadus Populasaun*	
			No. Familia	No.Membro
1	Sagrada Familia	183	422	2.322
2	Manumata	190	130	769
3	Bidau Mota Klaran	42	120	775
4	Toko Baru 1	117	147	864
Total:		**532**	**819**	**4.730**

*Note: *Dadus populasaun, husi Suco Bidau Santana 5*

Dili, 24 Setembro 2014.
DAC - DNSA

Table 21 - Number of Bidau Santana consumers

REKAPITULASAUN DADUS KONSUMIDOR BEE MOOS
SUCO METIAUT, SUB-DISTRITO CRISTO REI

UPDATE 2014

NO.	ALDEIA	No. KONSUMIDOR	Dadus Populasaun*	
			No. Familia	No.Membro
	17 de Abril	97	108	912
	Carungulau	68	64	368
	Fatucama	20	34	221
Total:		**185**	**206**	**1.501**

*Note: *Dadus populasaun, husi Suco Metiaut 29 Agosto*

Dili, 24 Setembro 2014.
DAC - DNSA

Table 22 - Number of Metiaut **consumers**

DADUS UZA BEE ZONA 10
Fulan: Janeiro - Novembro 2014

Suco	Aldeia	Volume Bee m3												Total
		Jan	Fev	Mar	Abl	Mai	Jun	Jul	Ags	Set	Out	Nov	Dez	
Bidau Santana	Sagrada Familia	6.402,00	4.272,00	5.333,00	4.802,00	4.305,00	4.339,00	4.712,00	5.426,00	4.846,00	7.804,00	6.623,00	-	58.864,00
	Manumata	9.707,00	7.851,00	7.851,00	6.998,00	9.318,00	5.313,00	7.772,00	8.613,00	7.008,00	7.573,00	12.763,00	-	90.767,00
	Bidau Mota Klaran	1.910,00	1.969,00	1.076,00	1.935,00	1.527,00	1.049,00	1.826,00	1.440,00	1.581,00	1.703,00	1.559,00	-	17.575,00
	Bidau Toko Baru 1	6.474,00	5.349,00	5.692,00	5.439,00	5.955,00	1.227,00	5.113,00	6.012,00	7.270,00	6.560,00	6.934,00	-	62.025,00
Sub Total		24.493,00	19.441,00	19.952,00	19.174,00	21.105,00	11.928,00	19.423,00	21.491,00	20.705,00	23.640,00	27.879,00	-	229.331,00
Metuaut	17 de Abril	5.392,00	3.579,00	3.912,00	4.099,00	5.777,00	2.974,00	3.164,00	2.028,00	1.632,00	2.339,00	2.708,00	-	37.604,00
	Carungulau	254,00	250,00	332,00	252,00	335,00	287,00	873,00	856,00	1.059,00	1.689,00	1.713,00	-	7.900,00
	Fatucama	-	-	-	-	-	-	56,00	76,00	38,00	116,00	103,00	-	389,00
Sub Total		5.646,00	3.829,00	4.244,00	4.351,00	6.112,00	3.261,00	4.093,00	2.960,00	2.729,00	4.144,00	4.524,00	-	45.893,00

Dili, 3 Dezembro 2014.
DAC - DNSA

Table 23 - Water consumption in Zone 10

Comparing the areas of the two Sucos and their populations, it appears that the Suco of Bidau Santana has a higher population density.

The Metiaut Suco is much dispersed, extending along the coast and many of its water consumers are restaurants, hotels and bars existing along the beach.

The construction of any Community system of sewage treatment in Bidau Santana Suco may serve a greater number of inhabitants, with a lower cost of investment, since the concentration of housing is much greater than in Metiaut Suco.

On the other hand the construction of any infrastructure in Bidau Santana serve mainly the inhabitants of the Suco, while the service provided in Metiaut will mainly reach the existing trade in the area, which initially presents better economic conditions to ensure by itself the sewers treatment.

Accordingly, it is considered that the Bidau Santana Suco gathers more conditions for the implementation of sanitary facilities.

Of the contacts with local leaders, it was revealed that, of the four villages of Bidau Santana, Suco, the Toko Baru village gathers more conditions and clarity regarding the criteria previously established:

- The Village of Toko Baru consists mainly of an old neighborhood of Indonesian government officials.
- The neighborhood is well organized, with well-defined streets and all lands are occupied with housing.
- The dwellings are connected to the public water supply and many have also holes, built to address the flaws in the public supply.
- The houses have toilets and absorbing wells / septic tanks for black water.
- The gray waters are drained into the drainage channels and watercourses, for the most part.

Of multiple contacts with the Chief of Suco was demonstrated the interest of the Suco in the implementation of new Community Systems:

– The construction of a community sewage treatment system (CSTS) in Toko Baru, the old quarter of Indonesian government officials could solve an old problem, and serious in your opinion, sewage release into the "playground" adjacent to neighborhood and that is causing a health hazard situation as for the surrounding housing, as for the field users.

Figure - Toko Baru: old quarter of Indonesian government officials and playground

Figure - sewage discharges to the playground

The construction of community toilets (MCK) in the villages of Sagrada Familia and Manumata , located in more remote and more mountainous areas , allow the poorest families to have access to toilets with sewage treatment and, as such, better living conditions and health.

Figure - Zone to be served with MCK in Sagrada Familia Village

Figure - Zone to be served with the MCK Manumata Village

Implementation of new systems

The construction of a new community wastewater treatment system (CSTS) in Toko Baru, the old quarter of Indonesian government officials, it seems feasible to fulfil all the essential requirements:

- ☺ Availability of water supply
- ☺ Availability of land
- ☺ Accessibility for O & M
- ☺ Existence of inadequate sanitation conditions
- ☺ Population density
- ☺ The community acceptance

The lifting of all dwellings that may be covered by the new system was done, and it was found that the new system can serve about 100.

Figure - Dwellings to be served by the CSTS Toko Baru

It will be needed to order the topographic survey of the area in order to know more accurately which units can be served, so that we can prepare the execution project.

It should be kept in mind that the implementation of this new CSTS will require first of all a serious understanding between the National Directorate of Basic Sanitation and the National Directorate of Youth and Sport on the use of the land, so as not to delay l the CSTS or future use of the land for the purpose it is intended.

It should not be overlook the need of community involvement, so that its acceptance is maintained and ensure a greater involvement to the success of the project.

For the new intended MCK, the situation has proved to be more complex.

Visits to these locations let foresee the greatest difficulties in the implementation of MCK in these areas:

- ☹ Difficult access as for the construction of the infrastructure, whether for its maintenance and also to ensure the car's clean-access tanks for cleaning the sewage treatment plant associated with the MCK,
- ☹ The difficulty of getting land already "taken" for the construction of the infrastructure.

Clearly, the Chief of Suco’s intentions have been disappointed for one reason or another:

In the case of the Sgrada Familia, the construction of the MCK will be conditioned by the construction of an access road planned for the area, according to the Chief of Suco. And to achieve the construction of the road, will still have to ensure that the land now available remain in the same situation,

In the Manumata case, the construction was soon compromised by the inability of the land transfer by the "occupant", who also will leave some of the houses without access, so the Chief of Suco should seek other alternatives to this area of the village.

The implementation of new MCK require more field surveys, with greater involvement of local leaders and where the biggest constraints lie in the existence of viable access and availability of land.

FUTURE WORKS

Regarding the investment plan submitted on 18 October 2012 to the Council of Ministers, there is a significant delay in the implementation of the PMSDD

Phase 1 2011-2012	Phase 2 2013-2017	Phase 3 2018-2025
	Plan	
PMSDD +2 MCK	MCK + 50 CSTS + 1 DEWATS	MCK + CSTS + 7 DEWATS
	Already done till current date	
PMSDD +2 MCK	DED 4 CSTS Proposed location for new CSTS (Bidau Toko Baru 1) and new MCK (Bidau Sagrada Familia)	
	To accomplish	
---	Implementation of 4 CSTS DED 1 CSTS + 1 MCK + DED 45 CSTS + N MCK + DED 1 DEWATS	

Table 24 - Table of planned investments to be made

Evolution of the Dili Sanitation System

The proposal of the authors of PMSDD studies, in its Phase 1, became the future of Sanitation solution in Dili, in small and medium decentralized systems without any connection between them, with a local treatment and subsequent removal of sludge to the WWTP Tibar for final treatment.

In the fundamental this proposal is based in the construction of about 50 CSTS, the suburb neighborhoods of Dili and in the construction of 8 small WWTP, or DEWATS, in areas where there may be space and conditions for a better treatment and for the existence of some small network locations.

There is always the possibility in the future to bind several CSTS and to bind several DEWATS to each other and start creating an integrated system, but in much of the city of Dili, it will be complicated the initial construction of CSTS and DEWATS and then its later connection between them, will present new works, with closing streets for the installation of collectors connecting two or more infrastructure.

We will not say that this situation is impossible, but will not be easy to solve when concerning to the land needed, and even for the agreement and support of the population.

However, the construction of multiple decentralized systems has obvious advantages that should be mentioned, for example:

- The investment is simpler to perform from a timing point of view;
- Will always be smaller infrastructures with a simpler operation and maintenance, taking each in isolation;
- It settles faster a case deemed urgent to solve;

This solution however raises a number of issues, weighted, can point to an alternative solution, and in this case are:

- Require the construction of a very large number of infrastructures (about 60) compared with an integrated (one WWTP already out of town, and 7 or 8 pumping stations - EE);
- Require a regular and we would say perpetual removal of sludge and transportation to Tibar across the entire city with multiple clean-tanks trucks, and in various places one clean-pits of normal dimensions, does not go into many streets of the several neighborhoods of Dili, forcing workarounds or the construction of a network to where the clean-tanks can move;
- Is much simpler the Operation and Maintenance of an ES than a CSTS or even more of a DEWATS.
- Is much simpler the construction of ES, which can be found in gardens (and the prescribed route to the main collector there is the existence of several garden areas, (Lecidere, May 5, Mandarin, etc.) where it can have a EE installation, without much problem as they take up little space and are located in small enclosed buildings.
- Dismiss or reduce a number without expression, the transportation in cleaning-tanks trucks to remove sludge of CSTS and DEWATS;

So our suggestion, subject always to further studies that lead to a better understanding of the situation, does not indicate the future solution for tens decentralized systems, leaving aside the possibility that, in a certain neighborhood or local, the connection difficulties of an integrated system do not apply in these individual cases, of systems and decentralized solutions.

But in our opinion, the sanitation solution in Dili should go through, in general, the construction of an integrated system.

As can be seen in the following figure, our proposal is based on building a general interceptor from one extreme to another of Dili, ending near Dili Rock, halfway between Dili and Tibar. On This site could be built a WWTP from where a submarine pipeline would part, out of the coral reef zone.

In terms of features, the main outfall will have an approximate length of 11 km and the submarine outfall about 1.8 km (to be confirmed with bathymetric studies and ocean tides in that area).

In terms of secondary outfalls (ie gravity and pumped links to the main outfall) can be estimated crudely a total of 4 km of aerial pipelines (4 emissaries) and about 21 km of discharges by gravity. The emissary go through the main road (connecting the WWTP Taci Tolu to Bidau Santana / Lecidere) and the secondary outfalls would always go along the water lines.

As for costs, the total investment can probe the 123 MUSD (One hundred and Twenty-Three Million Dollars) using an average value of 3.45 MUSD / km), which is based on the investment costs in Portugal, a work of SANEST, in the line of Estoril, near Lisbon.

For this system (SANEST), the updated total cost of both phases reach the amounts of 167 MUSD, comprising about 25 km of discharges (including outfall of 2,75 km).

To estimate the average value per km to Dili, we considered half the value obtained from SANEST, given that the population served is about half.

As for costs, and as discussed with ADN during the works of the Rivers and Channels, was considered the the work in Timor Leste will be cheaper about 15% of that in Portugal, and the costs of materials and machinery are slightly higher or equal, but lower the costs of hand labor.

The above figures already take into account the weighting of 15% reduction for the works in

As for the WWTP we can estimate a cost between 30-50 MUSD, depending on the complexity and requirement of the treatment, and support facilities that it may need or not to be built in the new plant. Is also be an improvement of the WWTP Tibar to be held in Phase 3, to receive and treat the sludge from the WWTP of Taci Tolu.

Figure - New proposal for a main Collector for Sanitation in Dili

If MOPTC considers to be better rethinking this possibility, a prior study must be performed examining any possible alternatives before proceeding with any project.

In the following table, it is indicated the timing of the works carried out in accordance with the proposal of Phase 1, as well as their estimated investment costs and also a possible phasing of our proposal, by comparing the estimated costs for this solution of an integrated sanitation system.

We only do this comparison to the investments, but we believe that in operation and maintenance, the costs of an integrated solution is much lower (and simpler) than the O & M of decentralized solutions.

		Unidade: Milhões de US Doláres			
		Projeto Fase I - DSDMP *Project Phase I Study*		Proposta AdP - TL *AdP - TL Proposal*	
		Fase 2 *Phase 2*	Fase 3 *Phase 3*	Fase 2 *Phase 2*	Fase 3 *Phase 3*
		M USD	M USD	M USD	M USD
CONTRUÇ CONTRUCTION	Construção 50 CSTS *(50 CSTS construction)* 1 DEWATS	75			
	Construção 7 DEWATS *(7 DEWATS contruction)* Const. X MCK Const. X Km Rede de interligação *(Const. X KM interconnection network)*		400		
	1 ETAR TASI TOLU *(1 WWTP Tasi Tolu)*			40	
	25 % do Coletor principal *(25 % Part of main collector)*			10	
	25 % da Rede coletora secundária *(25 % Network collector secondary)*			3	
	25% da Rede domiciliária *(25% Domicialiary network)*			3	
	3 Estações Elevatórias *(3 Pumping stations)*			6	
	Beneficiação da ETAR deTibar *(Improvement of WWTP Tibar)*				5
	75 % Coletor Principal *(75 % Main collector)*				28
	75% Rede coletor secundária *(75% Network collector secondary)*				9
	75 % Rede domiciliária *(75% Domiciliary network)*				9
	5 Estações Elevatórias *(5 Pumping stations)*				10
	Fase / Phase TOTAL	**75**	**400**	**62**	**61**
	TOTAL FINAL	**475**		**123**	

Table – Works and Cost Estimate

RECOMMENDATIONS FOR THE PROJECT SUSTAINABILITY AND CONCLUSIONS

The Sustainability of each of the projects in which AdP TL has been working based on various aspects (Environmental, Social, Financial and Organizational) and although there are contact points between the areas of sanitation and drainage, most or the most important issues to assure the sustainability of projects differ in each of the areas considered.

In the case of Sanitation and speaking specifically of the 4 CSTS building, its sustainability is based, in our opinion, in the following areas:

- ✓ Environmental: the treatment levels of each CSTS should be suitable into receiving environment and its resilience. This implies proper operation, with good levels of operation and maintenance of all organs of each infrastructures. On the other hand the removal of solids from the last compartment of each CSTS must be timely performed (our estimate is to be cleaned once a year, but must not exceed this period of time) so that there is no risk of overflow of the filling and concentrated sewage into the ground, contaminating local around the infrastructure and possible groundwater sheets where the population may be the supplied of water;
- ✓ Social: the community must be involved, feel that the infrastructure belong to them, support its management, and in our opinion this involvement should have a safety and proper use of CSTS than give responsibilities that the Community cannot ensure regularly, ie, the activities of Operation and Maintenance which should be at least during the start-up and testing (six months to a year), be under the responsibility of DNSB and then go progressively after being analyzed with the Community, which tasks can be decentralized, and not the other way;
- ✓ Financial: each CSTS is likely to have a different experience in the social area and in finance. If as with the existing MCK in Mascarenhas, there is the agreement of the population to make a financial contribution, even if symbolic (type 1 USD / month / family) this will lead to a progressive involvement of the population who will want to know where the money goes for and make them aware of the expenses of each CSTS. Practice shows that we value the goods and services that have a cost, however small, than the goods and services offered for free. And if there is a small payment by the Community it will give all the justification to DNSB explain where goes the money received and how much is costing the O & M of each CSTS. And maybe lead in a second year to a bigger contribution from the population, from a financial point of view, but DNSB should bear in mind that with the construction of each CSTS represents a regular cost of O & M that won't end. And its budget should be prepared for it, or it is better to phase further apart the investments. It is very bad for DNSB image to build equipment that then cannot afford to maintain;
- ✓ Organizational: the construction of the 4 CSTS and others that can follow shall include a new organizational capacity in DNSB. The Directorate should be prepared to directly monitor both, contacts with the population and assure all jobs of O & M, or have only a minimal internal monitoring framework and assure that has the financial capacity to delegate a significant part of the work, especially of O & M in contractors for this purpose and shall not delegate the regular contacts with the population, even if it is positive the involvement of local NGOs in this process.

Reflection on Sanitation

Management of the current MCK

The two current MCK, built during Phase 1 of the PMSDD, have different situations and it is important to consider why. In one of them the population is involved with the infrastructure, and in the other was always away from the infrastructure.

Thus, as proposed in the work of Community Development, some leaders and residents of Bidau Santana should go take an organized visit to Mascarenhas and examine together the reasons for the differences in the Operation and Maintenance of the two MCK.

Then, if the population of Bidau Santana decides to organize and commit to create a framework to manage the existing MCK, it should be recovered by DNSB and maintained in terms of division of labor and division of financial responsibilities, such as in Mascarenhas.

Management of the new CSTS - Division of labor between the Community and the Government

As already proposed in the previous paragraph, we propose that in a start-up phase, the population responsibility should be restricted to cleaning activities and the security of the facilities and all the functions of O & M are to be of DNSB. This initial phase 6-12 months should be with the full organizational and financial responsibility of DNSB and after this period then weigh the costs (which will have to be collected and recorded since the start of each CSTS) and see a possible division of the responsibilities.

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Tibar Waste Water Treatment Plant

Whatever the solution that will be adopted, the WWTP Tibar has an important role to play, to relieve the CSTS, DEWATS and even Tasi Tolu WWTP sludge treatment, which may be concentrated in Tibar.

This implies that within a few years the WWTP Tibar undergoes in renovation works and should then be adapted to its main function, which depend on the adopted solution for the Dili Sanitation.

In the previous Figure we point for Phase 3, which will begin in 2018, with an average size of intervention and a budget of about 5 Million Dollars.

Eventually its area should be increased to create conditions for drying sludge, which may subsequently have agricultural or power production use.

Use of drainage channels for Sanitation Network

One of the problems that we propose for consideration and which in our opinion should be the object of immediate measures and minimizing the current situation is the regular and widespread use of drainage channels as sewage drainage channels.

This use creates a very serious situation for public health and transforms a water line, which should be clean and running smoothly to the sea, in outbreaks of diseases, especially for children, who walk regularly playing within the channels.

The problem will be solved with the resolution of sanitation in Dili, but it may take more than 10 or 20 years.

A minimizing measure will be the construction of wells ruptured or if possible septic tanks, even without any treatment, at locations to be determined taking into account the gravity of the situation in each village, neighborhood or street, and the sewage accumulated there should be regularly removed to the WWTP Tibar for treatment. This investment in wells or tanks will be largely offset on the Health's side, because the current situation is very serious.

During the rainy season, although the effluent flowing in the channel has a preponderance of storm water, in the dry season it is almost exclusively sewage that runs in the channels what makes the situation very serious.

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GLOBAL WATER MARKET

B. Bakri [1], Y. Arai [2], A. Koizumi [2] T. Inakazu [2] and S. Pallu [1]

ABSTRACT: Water markets data is a useful tool for understanding the informal water sector. It provides information to assist decision-makers in establishing or improving water infrastructure.markets data is a useful tool for understanding the informal water sector. The water utilities market is divided to capital expenditure (CAPEX) and operating expenditure (OPEX). This study reviews and analyzes a report of global water market to describe municipal water market from all over the world. Global water markets expected to be increased over 2016. It is dominated by utilities category that consists of drinking water and wastewater sector. Investment of drinking water sector is expected to be dominated by developed countries such as: United States of America (USA), Japan, China, Germany and France etc., which are tend to rehabilitate their water distribution network. Drinking water management of countries in America and Caribbean region are mainly to develop their water distribution networks, it is contrary with countries in Europe that tend to rehabilitate existing their water distribution network. While in Asia, Pacific and Africa, there a lot of countries seem like putting effort to expansion their water network, and the others concentrated to rehabilitate distribution network.

Keywords: Water market, capital expenditure, operating expenditure

INTRODUCTION

Water markets data is a useful tool for understanding the informal water sector. It provides information to assist decision-makers in establishing or improving water infrastructure (Grafton and Horne, 2014). In recent years, a number of water market studies have mainly focused to identify water market structure and it's affecting to the cost and economic developed. Howitt (1994) presented some empirical results to show direct benefits and costs of the water bank to the state and regional economic. At the same time, Crane (1994) carried out a study to inform the linkage water markets, market reforms and urban poverty in Jakarta-Indonesia. Ansink and Houba (2011) prepared a model market power in water markets to capture specific water market structure. Similarly Ansink and Houba, Wang (2012) presented a simulation of water markets model with transaction cost. It evaluated the potential of investment savings from introducing the trans-jurisdictional water market to achieve the water-saving targets of agricultural and industrial sectors. Contrary of the two previous studies, Carvalho et al. (2012) analyze scale and scope economies in the water sector using a meta-regression analysis.

This study reviews and analyzes a report of global water market to describe municipal water market from all over the world. The report is prepared by the research team at Global Water Intelligence (GWI). There are 50 countries was selected for detailed profiles generally represent the most significant water markets in the world. The report focuses on the municipal and industrial water market, and it does not cover agricultural water sector and the packaged or bottled (GWI, 2011).

WATER MARKET

Capital expenditure and operation expenditure

The water utilities market in the report is divided to capital expenditure (CAPEX) and operating expenditure (OPEX). CAPEX is anything invested to acquire or upgrade value of investment, while OPEX is cost that occurred on day today to operate the investment. Due to around 90% is provided by publicly owned water utilities, than CAPEX is the greater interest from a commercial point of water market view of the two (GWI, 2011). Drinking water CAPEX in 2010 will be applied to analyze the global water market strategy (Kaneda et al., 2013). CAPEX consists of water resources development, distribution network, network rehabilitation and water treatment plants. Reservoir, water towers, raw water mains, pumping station and whatever infrastructure is required to bring water from resources to plant are included on water resources development. Distribution network is maybe the most expensive, it mainly consists of civil engineering, pipes, valves, meter and the others pipe accessories. Pipe rehabilitation is required for repairing network due to deterioration or leakage. While water treatment plant (WTP) is related to the treatment processes from raw water to clean or drinking water. Population growth, income growth, regulation and scarcity are drivers of CAPEX in the water utility sector (GWI, 2011).

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Global water market

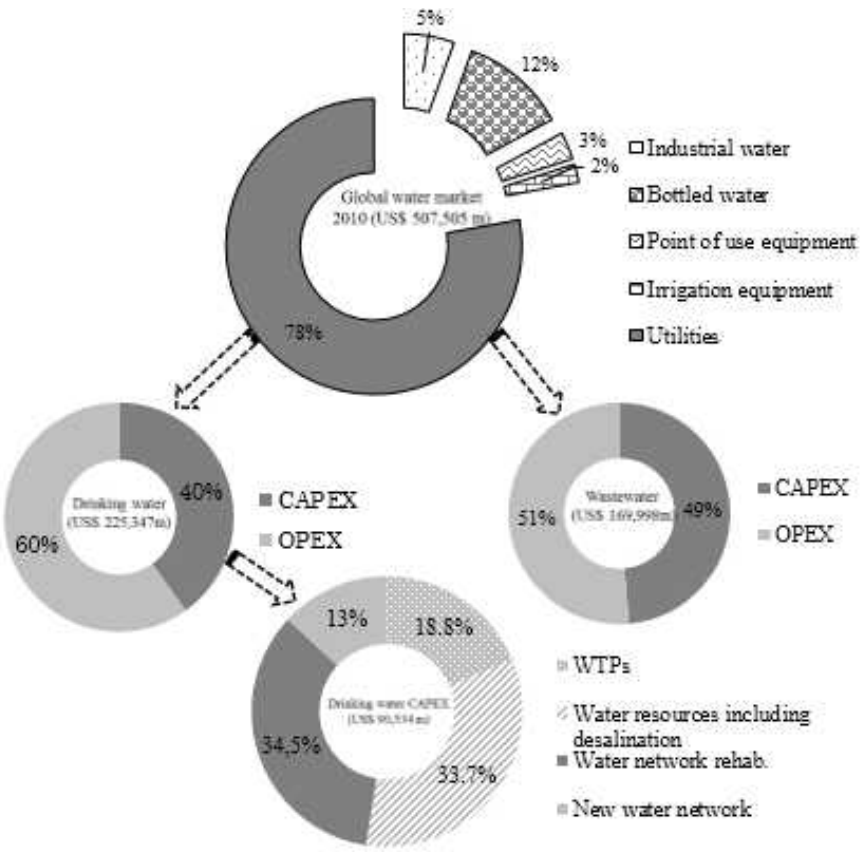

Figure 1. Global water markets in 2010

Figure 1 expresses global water market in 2010 (GWI, 2011). The figure shows that the global water market in 2010 was estimated around $507,505 million, it was dominated by utilities category as 77.9% of the total CAPEX or $395,566 million. This category consists of drinking water and wastewater. Market value for drinking water CAPEX was 40% of the total utility drinking market or approximately $90,534 million. Water network rehabilitation dominated investment in drinking water market CAPEX sector approximately 34.5% of the total CAPEX, it is slightly higher than market for water resources, accounting 33.7%. While total investment for WTP and new water network of drinking water sector reached 18.8% and 13% in respectively. To generate description of strategic drinking water market management in the world, then the next section selects 50 countries for detailed profile of their drinking water market *CAPEX*. These countries generally represent the most significant water markets in the world.

Top ten global water markets

Figure 2 shows the top ten countries at water market in 2010. The graph divides drinking water market CAPEX to the two categories, namely, water resources development and water distribution network. Water resources development consists of sea water and desalination, water resources and water treatment plant, while rehabilitation and new water network are included in water distribution network. China and United Stated of America with the most and third populated country were the first and second largest water market in the world. Japan with population number reached 126 million or the tenth of the world was the third largest country in water market investment. Algeria and South Africa represent the largest water market in developing countries. The total market of the tenth countries was $57,213 million or approximately 63% of the total drinking water CAPEX. The strategic drinking water market management of China, USA, Japan, United Kingdom (UK) and Germany seemed like putting effort to upgrade the their network by rehabilitation of distribution network. South Africa concentrated in investment of new water network. Australia and Saudi Arabia emphasized investment in water resources development, particularly in sea water and desalination sector. France and Algeria invested more much money in water resources sector.

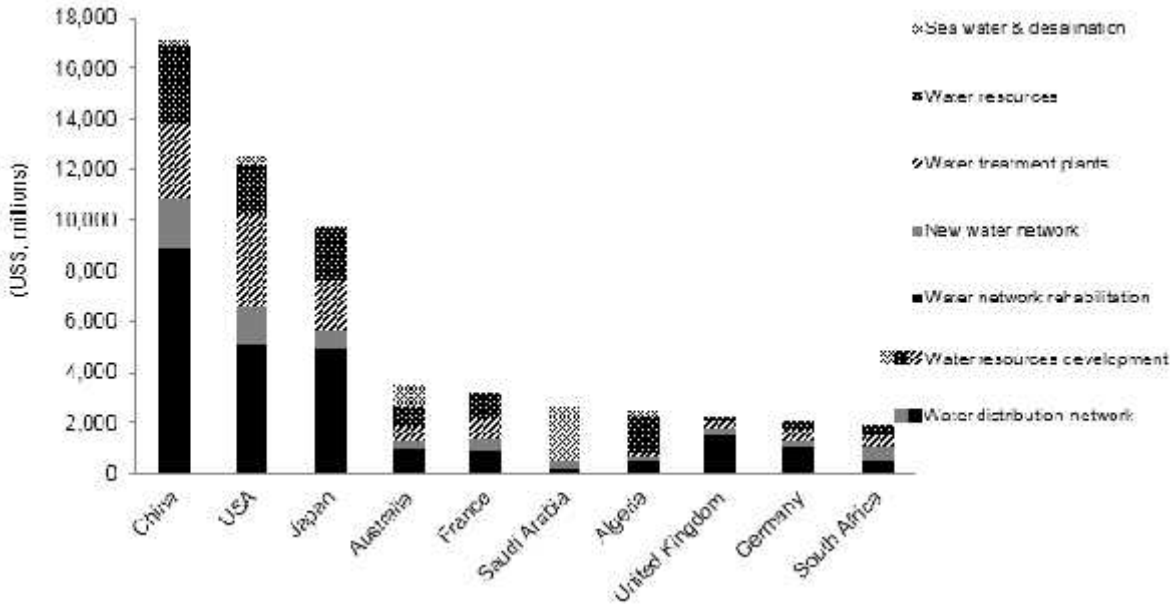

Figure 2. Top 10 countries at water market

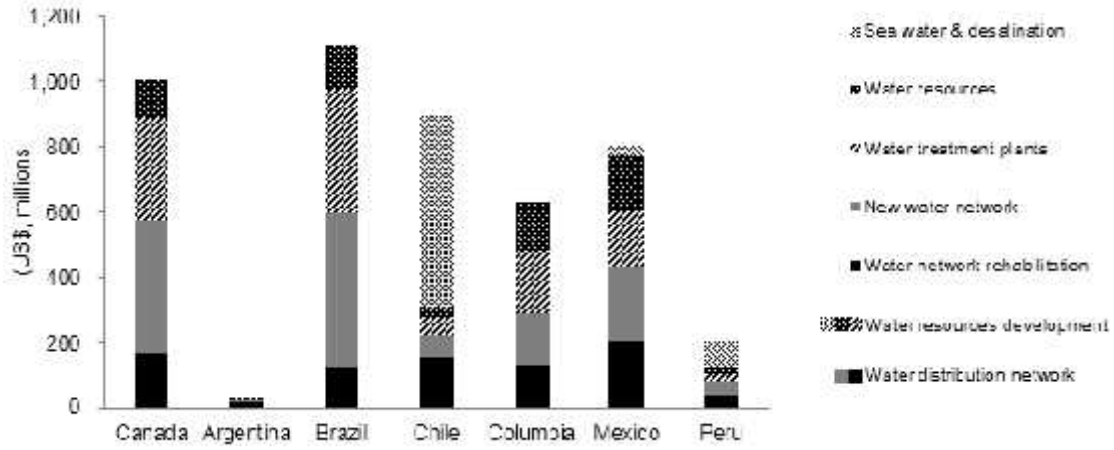

Figure 3. Water market in America and Caribbean region

American and Caribbean water market

Water market in America and Caribbean region without USA is shown on Figure 3. The figure clarifies Canada, Brazil and Mexico seek to expand their water distribution network, while Argentina focused on the rehabilitation of the distribution network. Strategic drinking water market management in Columbia, Chile and Peru is focusing on investment of water resources development. Water treatment plant sector was the largest investment in Columbia. In other hand, exploitation of sea water and desalination for meeting water demand was strategic drinking water management of Chile and Peru.

Europe drinking water market

Figure 4 shows investment in water drinking sector in Europe without France, Germany and UK. There are 15 countries presented to provide an overview in strategic drinking water management in this region. Drinking water investment in Austria, Belgium, Sweden and Czech Republic dominated by rehabilitation of the distribution network, while Netherlands, Switzerland and Belarus by expansion of new distribution system. The others countries put more attention in investment of water resources development, such as; Italy, Spain, Croatia, Romania, Kazakhstan and Russian federation, which are more likely to invest in water resources sector, and Poland and Hungary in water treatment plants sector.

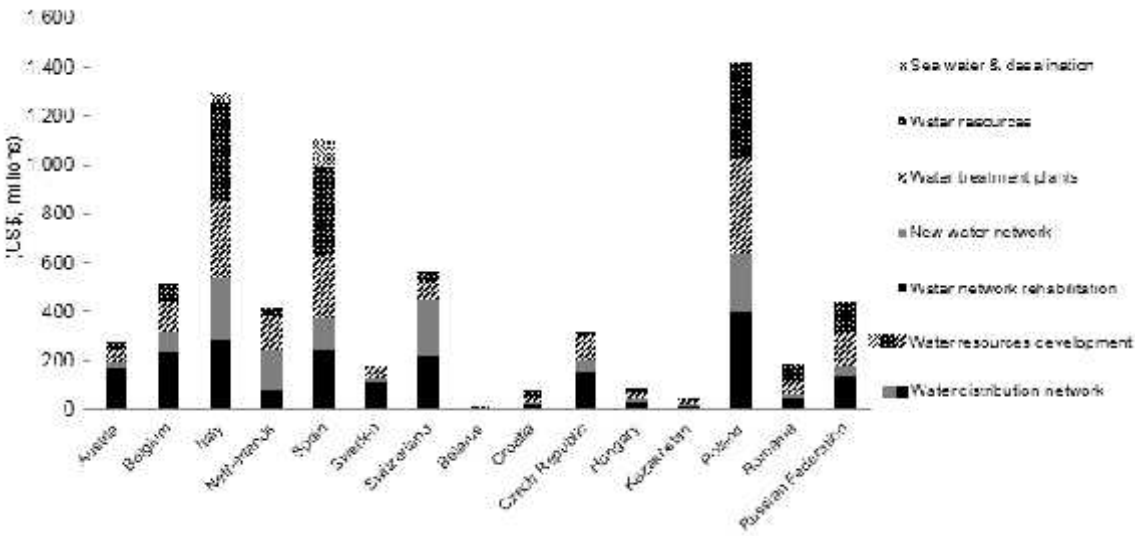

Figure 4. Water market in Europe

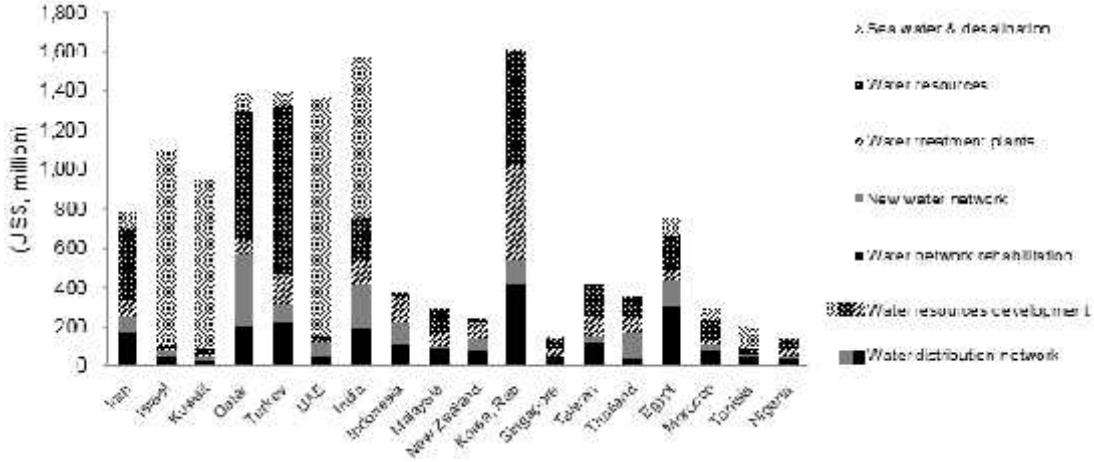

Figure 5. Water market in Asia, Pacific and Africa

Asia, Pacific and Africa drinking water market

Water market investment in Asia, Pacific and Africa without China, Japan, Australia, Algeria, and South Africa in 2010 is summarized in the Figure 5. The figure expresses that water market investment in the middle and south Asia that consists of Israel, Kuwait, United Arab Emirate (UAE) and India is dominated by water resources development, namely in sea water and desalination sector, while Iran, Qatar and Turkey were in water resources sector. Strategic drinking water management in EAP region, which are represented by Indonesia, Malaysia, New Zealand, Republic of Korea, Singapore, Taiwan and Thailand were put more attention investment in water resources development, with exception of Indonesia, New Zealand and Thailand. Furthermore, water market detailed of this region will be analyzed in the next section. Development of water distribution network was strategic water management in Egypt. The others African countries such as; Morocco and Nigeria concentrated in water resources sector. Huge investment in sea water and desalination sector was strategic water drinking management in Tunisia.

DRINKING WATER MARKET TRENDS

Forecasting global water market of each sector is shown on Figure 6 (GWI, 2011). The figure indicates that water network rehabilitation dominated market during 2007-2010, and it will grow constantly over the future six years, with growth rates over 3% per annum. The high market of network rehabilitation is caused by the global market dominated by developed countries such as: United States of America (USA), Japan, China, Germany and France which are tend to rehabilitate their distribution network. At the same time, water resources and treatment plants were around 30% and 20% of total water market in respectively and also will grow constantly in the future. Thus the total market is expected to be increased during 2010-2016, with growth rates around 7.4% per annum. While regional drinking water market will develop overs the years as shown on Figure 7 (GWI, 2011). Water infrastructure was estimated $90 billion in 2010, and expected rising to $131 billion by 2016.

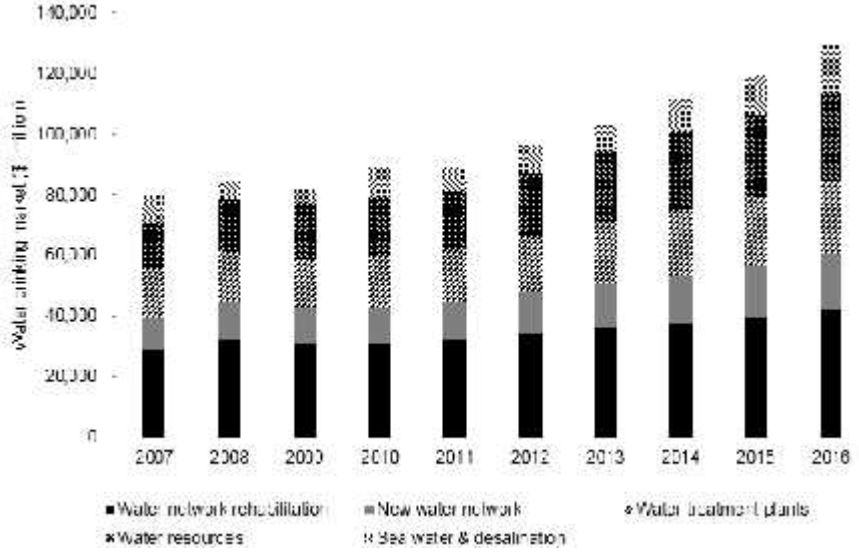

Figure 6. Breakdown of the global water infrastructure market

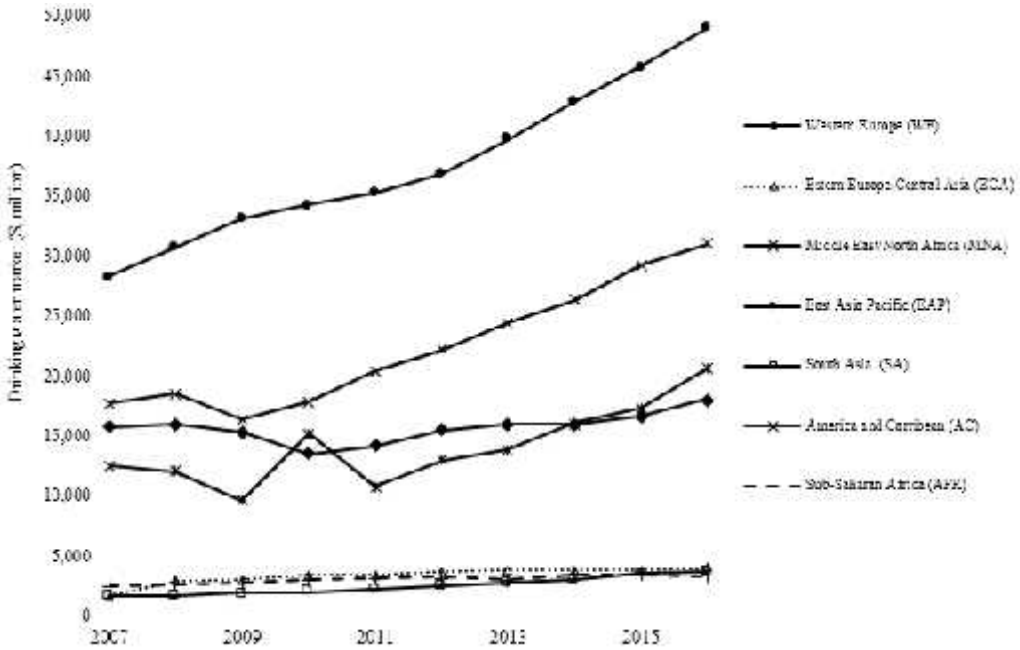

Figure 7. Forecast drinking water market

OVERVIEW OF DRINKING WATER MARKET IN EAST ASIA PACIFIC

Figure 7 indicates that the global water market is dominated by EAP region and is expected to increase over the years. Total population of this region was 2,1 billion or around 32% of the world population. The large population number of this region requires substantial cost in building and maintaining of water infrastructure. Water market will be dominated by Japan, China, South Korea and Australia over 2016, while developing countries in this region will give significant impact on the water market trends according to improvement of their economic and increasing population. Figure 8 summarized economic feature and water service in this region. Briefly, the figure explains that there is a significant difference of the countries in this region both of economic feature and water service coverage. Figure 9 was carried out to describe strategy pipe management in this region. Firstly, we make standardization of investment value of water resources development as “1.0”, and then calculate the network ratio (expansion : rehabilitation) to the water resources development (Bakri et al., 2014). The figure summarizes that strategic pipe management in this region appears to consist mainly of efforts to rehabilitate the water distribution network, with the exception of Indonesia, Thailand and New Zealand.

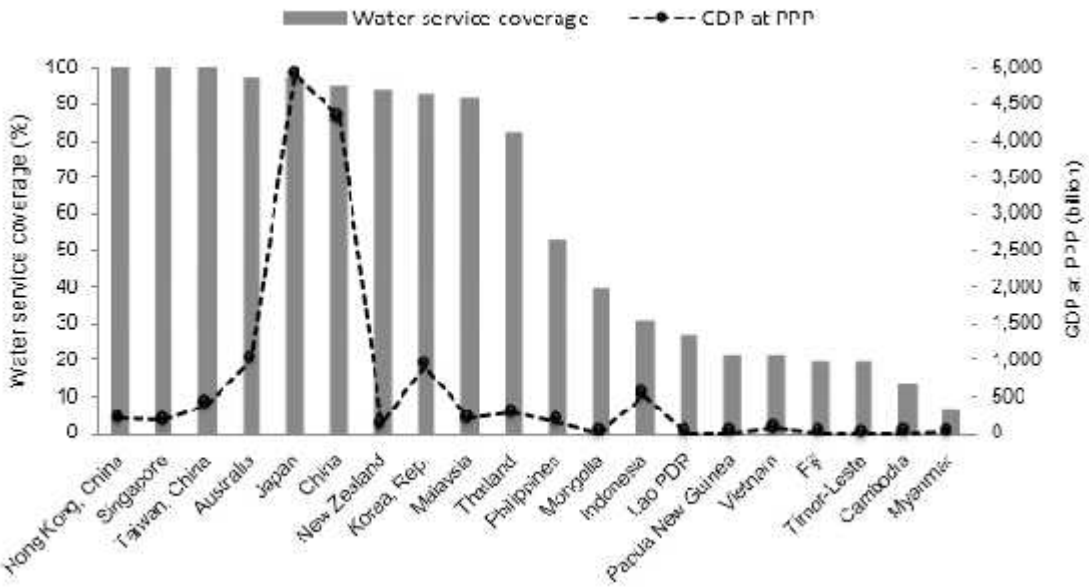

Figure 8. Water service coverage and GDP at PPP of EAP region

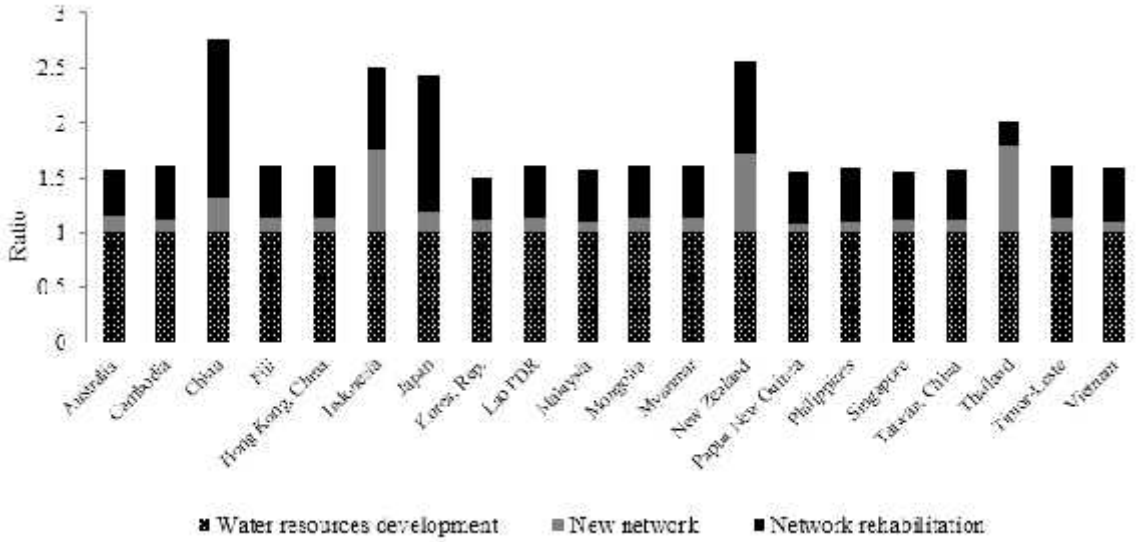

Figure 9. Pipe management strategies in EAP region

However, based on the Figure 8 and 9 then strategic pipe management of this region could be categorized to be three groups as following:

- Group 1. Countries which are tend to expand the new water distribution network. Thailand, New Zealand and Indonesia included in this group.
- Group 2. Countries which are put pay more attention to rehabilitate of their water distribution network due to water service has covered the entire population. This group is dominated by developed countries such as; Australia, Japan, China, Hong Kong, Republic of Korea, Singapore, Taiwan and Malaysia.
- Group 3. Cambodia, Fiji, Laos PDR, Mongolia, Myanmar, Papua New Guinea, Philippine, Timor-Leste and Vietnam included in this group. They are developing countries that seem like putting effort to rehabilitate their water distribution network even though most of their population have not been covered by water service. It is caused by limited fund of these countries for expansion of water distribution network.

CONCLUSIONS

Urbanizations, scarcity and income growth are driving the increased demands on the water sector. Water utilities need to invest more cost for infrastructure and technology to meet the demand. Water markets data is a useful tool for understanding the informal water sector. It provides information to assist decision-makers in establishing or improving

water infrastructure. Global water markets expected to be increased over 2016. It is dominated by utilities category that consists of drinking water and wastewater sector. Investment of drinking water sector is expected to be dominated by developed countries such as: United States of America (USA), Japan, China, Germany and France etc., which are tend to rehabilitate their water distribution network. Drinking water management of countries in America and Caribbean region are mainly to develop their water distribution networks, it is contrary with countries in Europe that tend to rehabilitate existing their water distribution network. While in Asia, Pacific and Africa, there a lot of countries seem like putting effort to expansion their water network, and the others concentrated to rehabilitate distribution network. The strategic drink water management of each country is based on the political and economic conditions they face. East Asia Pacific region with the largest population requires substantial cost in building and maintaining of their water infrastructure. Water market in this region will be dominated by Japan, China, South Korea and Australia over 2016. Their water market data indicates that developing countries investment in OPEX sector is higher than CAPEX. Contrary in developing country, investment in water market expected to be dominated by CAPEX.

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ROUTINE MAINTENANCE FUND ESTIMATION MODEL FOR URBAN ROAD (CASE STUDY: MAKASSAR, INDONESIA)

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ABSTRACT: Routine maintenance for road is carried out along the years in order to improve riding quality. Routine maintenance requires significant amount of fund; thus, it should be estimated optimally for the following year. This paper suggests a model of preliminary estimation for road routine maintenance. Study was conducted on twelve roads funded by the state well known as National Road in Makassar. The model is generated using multiple regression analysis with double log transformation and based on two independent variables: average daily traffic (ADT) and road area (AREA). The generated model is evaluated using determination coefficient and one-way analysis of variance (ANOVA) to determine the significance of variables. A preliminary estimation for financing routine maintenance of urban road can be calculated using the following equation: Ln(FUND) = 0.414 + 0.718 Ln(ADT) + 1.455 Ln(AREA)

Keywords: road, routine maintenance, estimation model

INTRODUCTION

Road maintenance program requires a significant amount of funding while cost estimation hold substantial role in construction process. Cost estimation is an important process in project phase in order to determine the amount of funding needed to develop an infrastructure. Any inaccuracy occurs in the cost estimation process brings the project into a inefficient situation. In addition, cost estimation is required as a tangible scale in order to determine the investment for operating and maintaining infrastructure.

For an infrastructure project there are several methods applied in cost estimation process depend on the objective and the scope of estimation. Throughout initial phase of road maintenance project such as during funding allocation phase, detail cost estimation is unable to be carried out since the bill of quantity and technical specification has not been performed yet. However, project owner should determine the approximate number of funding in order to manage the maintenance program. Therefore, project owner needs a preliminary cost estimation based on considerable variables such as the scale of the project.

Preliminary cost estimation mainly utilizes the dimension scale such as length, width or area. However, to obtain more accurate forecasting for required fund, it is necessary to consider another variable aside the scale variable. Since the deterioration of road structure mainly stimulated traffic load; therefore, traffic variable should be consider in preliminary cost estimation. Several considerations in developing model for preliminary cost estimation are simplicity and accuracy in order to obtain a reliable or accountable result.

METHODOLOGY

Study was conducted on 12 roads in Makassar that funded by the state which well known as National Road as shown in table 1 and figure 1. Traffic surveys were conducted in 6 days a week during peak hour, which are between 7:00 AM to 9:00 AM and between 04:00 PM to 06:00 PM. Secondary data were collected from Highway Division of the Department of Public Works.

The routine maintenance cost estimation model is generated using multiple regression analysis with double log transformation. Multiple regression is a flexible method of data analysis that may be appropriate whenever a quantitative variable (the dependent or criterion variable) is to be examined in relationship to any other factors (expressed as independent or predictor variables). Relationships may be nonlinear, independent variables may be quantitative or qualitative, and one can examine the effects of a single variable or multiple variables with or without the effects of other variables taken into account (Cohen et.al., 2003).

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Table 1. National Road in Makassar

Road Code	Road Name	Length (m)
010 12 K	Perintis Kemerdekaan	12,510
010 13 K	Jend Urip Sumiharjo	494
010 14 K	G. Bawakaraeng	1,110
010 15 K	Masjid Raya	1,224
010 16 K	G. Bulusaraung	675
010 17 K	Jend Ahmad Yani	700
010 18 K	Riburane	230
013 11 K	Nusantara	1,942
011 11 K	Veteran Utara	2,074
011 12 K	Veteran Selatan	2,158
011 13 K	Sultan Alauddin	3,702
011 14 K	A. P. Pettarani	4,370

Fig. 1 National road network in Makassar city (*colored in magenta*)

Many practical questions involve the relationship between a dependent or criterion variable of interest (Y) and a set of k independent variables or potential predictor variables (X_1, X_2, X_3,..., Xk), where the scores on all variables are measured for N cases. A multiple regression equation for predicting Y can be expressed in equation (1).

$$\hat{Y} = A + B1X1 + B2X2 + B3X3 \tag{1}$$

To apply the equation, each X_j score for an individual case is multiplied by the corresponding B_j value, the products are added together, and the constant A is added to the sum. The result is Y, the predicted Y value for the case.

For a given set of data, the values for A and the B_js are determined mathematically to minimize the sum of squared deviations between predicted $\hat{Y}$ and the actual Y scores. Calculations are quite complex, and best performed with the help of a computer, although simple cases with only one or two predictors can be solved by hand with special formulas.

The correlation between $\hat{Y}$ and the actual Y value is also called the multiple correlation coefficients, $R_{y.12...k}$, or simply R. Thus, R provides a measure of how well Y can be predicted from the set of X scores. The following formula can be used to test the null hypothesis that in the population there is no linear relationship between Y and prediction based on the set of k X variables from N cases:

$$F = \frac{R^2_{y.12...k} / k}{(1 - R^2_{y.12...k}) / (N - k - 1)}, df = k, N - k - 1 \quad (2)$$

One-way ANOVA is used to compare means from at least three groups from one variable. The null hypothesis is that all the population group means are equal versus the alternative that at least one of the population means differs from the others. Estimation for the one-way ANOVA can be performed one of two ways. First, calculate the total variation, within-level variation and across-level variation. These can be summarized in a table as shown below and tests can be made to determine if the factor levels are significant (Chatterjee, 2012). In general, the ANOVA for the one-way case is given in table 2.

Table 2. ANOVA one-way typical table

Source	Sum of Squares	DoF	Mean Square	F_0
Factor	$SS_F = J\Sigma(\bar{y}_{i.} - \bar{y}_{..})^2$	$I - J$	$MSF = SS_F / (I - 1)$	MSF / MSE
Residual	$SS_E = \Sigma\Sigma(y_{ij} - \bar{y}_{.i})^2$	$I(J - 1)$	$MSE = SS_E / (I(J - 1))$	
Corrected total	$SST = \Sigma\Sigma(y_{ij} - \bar{y}_{..})^2$	$IJ - 1$		

where $\bar{y}_{i.} = \frac{1}{J}\sum_{j=1}^{J} y_{ij}$ and $\bar{y}_{..} = \frac{1}{IJ}\sum_{i=1}^{I}\sum_{j=1}^{J} y_{ij}$ (3)

RESULT AND DISCUSSIONS

Data which was collected from Highway Division of the Department of Public Works shows that funding for routine maintenance program allocated for 12 roads in thousand rupiah. Figure 2 shows that the allocated fund is quite similar in the given year. In order to obtain an equal comparison, the fund in 2012 is compounded into 2013 value using inflation rate of 5.47% per annum.

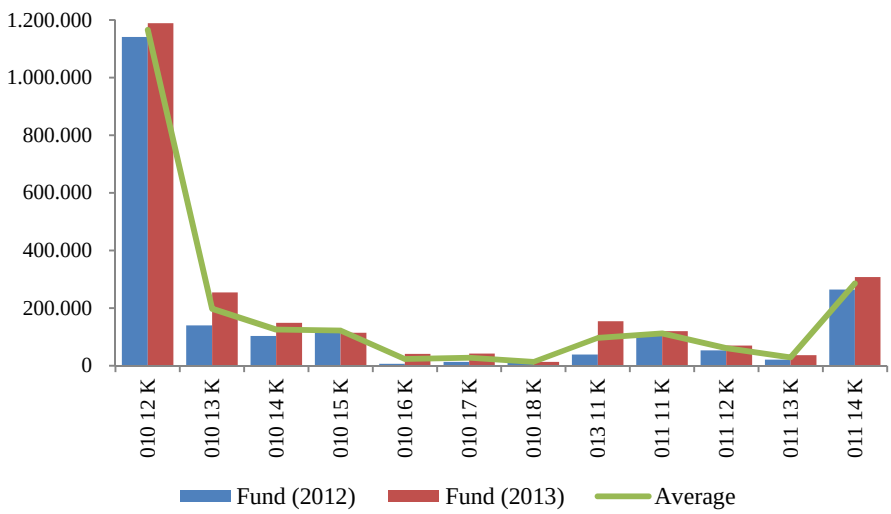

Fig. 2 Comparison of routine maintenance fund between 2012 and 2013

The average value of fund in 2012 and 2013 is utilized as dependent variable (FUND) which is measured rupiah. The independent variable are: average daily traffic (ADT) that is measured in passenger car unit per day and road area (AREA) which is measured in square meter. Table 3 shows the independent variable data. The econometric model that will be generated is a functional model in the form of double log (Hosmer Jr, 2013). Therefore, the estimating model is:

$$\text{Ln(FUND)} = \beta_0 + \beta_1 \text{Ln(ADT)} + \beta_2 \text{Ln(AREA)} + \varepsilon \quad (4)$$

Table 3. Independent variables data

Road Code	Area (m^2)	ADT (pcu/day)
010 12 K	300240	19.095
010 13 K	98860	6.379
010 14 K	19980	181.212
010 15 K	19584	122.533
010 16 K	9450	102.492
010 17 K	12600	143.458
010 18 K	5520	84.094
013 11 K	38840	45.543
011 11 K	49776	16.441
011 12 K	51792	11.031
011 13 K	74040	8.529
011 14 K	148580	9.702

In the regression analysis, the 95 percent confidence level is used, corresponding to 5 percent significance level. Figure 3 which shows the Ms Excel® output suggests that the estimated regression equation is:

$$\text{Ln(FUND)} = 0.414 + 0.718\,\text{Ln(ADT)} + 1.455\,\text{Ln(AREA)} \quad (5)$$

Regression Statistics					
Multiple R	0.929				
R Square	0.863				
Adjusted R Square	0.832				
Standard Error	0.510				
Observations	12				
ANOVA					
	df	*SS*	*MS*	*F*	*Significance F*
Regression	2	14.698	7.349	28.272612	0.0001
Residual	9	2.339	0.260		
Total	11	17.038			
	Coefficients	*Standard Error*	*t Stat*	*P-value*	
Intercept	0.414	2.817	0.147	0.887	
Ln(ADT)	0.718	0.200	3.582	0.006	
Ln(AREA)	1.455	0.211	6.895	7.109E-05	

Fig. 3 Regression analysis output

The regression analysis output shows that:

a. The R Square indicates that the model explains 86.3 percent of the variation in the logarithm of routine maintenance cost.
b. The model is significance overall which is indicated by the value of Significance F = 0.0001 is approaching zero.
c. The ADT and AREA variables are significant where P-value is far below the significance level (0.05).

Based on the model, it is indicated that any addition of 100 square meters in road area will increase the routine maintenance coat by approximately 500.000 rupiah. In other hand, every growth of 10 passenger car unit per day will raise the routine maintenance coat by approximately 400.000 rupiah. Figure 4 illustrates the comparison between real data and forecast generated by the model.

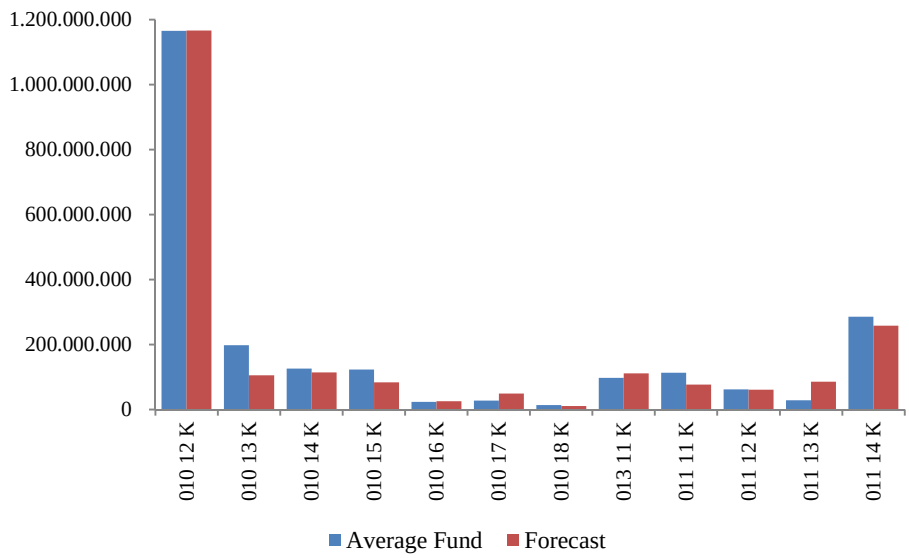

Fig. 4 Comparison between real data and forecasting

CONCLUSION

Regression analysis is utilized to develop a model to estimate the fund for road routine maintenance program. The model uses two independent variable which are: average daily traffic (ADT) and road area (AREA). The generated model is evaluated using determination coefficient and one-way analysis of variance to determine the significance of variables. A preliminary estimation for financing routine maintenance of urban road can be calculated using the following equation: Ln(FUND) = 0.414 + 0.718 Ln(ADT) + 1.455 Ln(AREA).

Based on the model, it is indicated that any addition of 100 square meters in road area will increase the routine maintenance coat by approximately 500.000 rupiah. In other hand, every growth of 10 passenger car unit per day will raise the routine maintenance coat by approximately 400.000 rupiah. To increase the model liability, it is suggested that the model should consider more tangible variables.

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ARRANGEMENT MODEL ON BUILDING ESTUARY RIVER REGION (Case Study: Tallo River, Makassar Indonesia)

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ABSTRACT: The research objectives were to obtain the concept of river estuary river bank settlements in Makassar city is based on the local culture. The dynamics of the growth of Makassar city basically originated from the influence of the river where Jeneberang and Tallo River as a means of transportation and a source of life. The method used is a qualitative and geographic information systems (GIS) approach. Identification of the characteristics of the river mouth area Jeneberang settlements and residential areas and resulting spatial models of buildings and the river estuary area.

Keywords: Model, Arrangement, Estuary, River.

INTRODUCTION

a. Background

Indonesia is an archipelagic nation has five major islands and thousands of small islands that have a very long coastal region reached 80,000 km at the mouth of the river which is quite a lot. The region is an area very intensively used by the community as a residential area, government centers, ports, aquaculture, industry, fisheries, agriculture, tourism and so on. The more intensive the utilization of coastal areas and estuaries may cause an increase in needs, infrastructure and so on, which in turn will lead to the emergence of new problems; namely the withdrawal of the coastline, land arising from coastal sediment that can lead to the rapid advancement of shoreline, estuaries are silting deflection or Dapa tmenyumbat river flow, pollution and sea water intrusion.

Geographical conditions and history make the swampy estuary as a center of growth, movement paths, the source of life, life orientation and identity bagim asyarakat Makassar, which is called the culture of the river. Increasing population and urban growth, led to a culture of settlement banks of a river estuary shifted and began to lose its uniqueness as a decrease to the environmental damage due to the change in orientation of the river kedaratan society, but the river is still a big role in the lives of citizens. The research aims to obtain the concept of river estuary riverbank settlements in the city of Makassar is based on the local culture. The dynamics of the growth of the city of Makassar basically originated from the influence of the river where Jeneberang and Tallo River as a means of transportation and a source of life. In general, this search oriented location in an area that has the potential sources for the fulfillment of human needs. The geographic location of the Makassar city is located in a strategic location in the form of flashing Jeneberang river and cut off the city of Jeneberang River is a river that connects Tallo with areas in the city of Makassar.

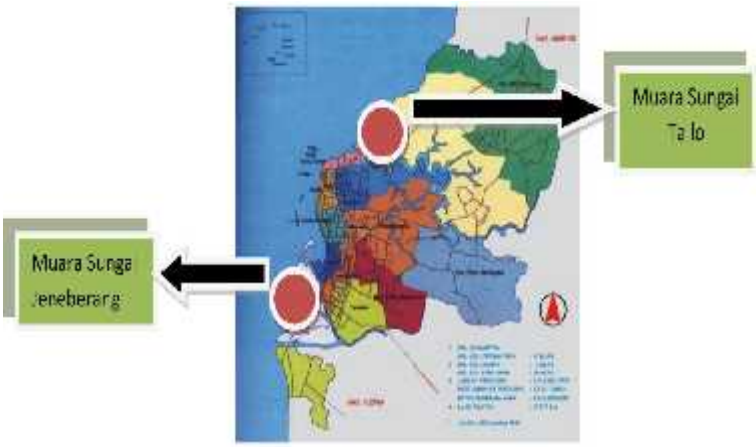

Fig. 1 Administration Map of Makassar City

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b. Research Purposes

1) Analyzing the characteristics of the riverbanks Jeneberang riverbanks and setlement areas.
2) Analyze the spatial model of the building and the river estuary area.

LITERATURE REVIEW

Mouth of the river is open water related denganlaut free, so that sea water with a high salinity can be mixed with fresh water. Combined effects of sea water and fresh water will produce a distinctive community, with varying environmental conditions, among others; 1). Tempatbertemunyaarussungaidengan tidal currents, the opposite cause a strong influence on the sedimentation, mixing water, and other physical characteristics, as well as bringing pengaruhbesarpadabiotanya; 2). mixing the two kinds of water produces a special physical properties are not the same environment with the nature of the river water and sea water properties; 3). changes that occur due to tidal require adjustment physiologically hold community with the surrounding environment. 4). the level of salinity in estuaries depending on the tide, the amount of water flow tawardanarus-aruslain, as well as the topography of the region Jeneberang tersebut.Estuaria estuary is a semi-enclosed waters associated freely to the sea, so the sea water with a high salinity can be mixed with water bargaining.

There was activity in order to exploit the potential inherent in coastal areas, often overlap, so it is not uncommon resource utilization is actually degrade or destroy the existing potential. It is because of these activities, either directly or indirectly, affects the lives of organisms in coastal areas, through environmental changes in the region. For example, the waste from both residential and industrial activity, although this waste may not affect the main constituent of plant or animal origin above the coastal ecosystem, but it is likely to affect biota other constituent. Heavy metals, for example, may have no effect on the lives of mangroves (mangroves), but extremely dangerous for the life of fish and shellfish in the forest live shrimp.

In the Government Regulation No. 38/2011, the river is a natural water flow or the container and / or made in the form of water drainage network together with water in it, from upstream to the estuary, with the right and left bounded by a border line. based on Law No. 7/2004 on Water Resources, watershed is an area of land which is a unity with the river and its tributaries, which serves to accommodate, store, and stream that comes from rainfall to the lake or to the sea in nature, which limits the land is topographical and boundary dividing the sea until the water area is still affected by land activities. World issues about water, river pollution and degradation of watershed (DAS) is a serious environmental problem and the real front of the eye.

A decrease in the quality and volume of available river water will greatly affect crop production as the main water source is the river irrigation system. Of total surface water (river, lake, swamp) which used in human activities reached 70% remaining in use for industry and meet the needs of the household.

The degree of criticality of a watershed shown by the decrease in permanent vegetation cover and widespread critical area so that the decrease in water storage DAS capability which increased flood frequency. Another factor that led to the watershed treatment has not worked well is the lack of coherence in the implementation and monitoring of pollution of watershed management, including in terms of financing. This is because the number of agencies involved in watershed management.

Watersheds (DAS) is identical to the (river basin; drainage basin, catchmentarea, watershed), depending on its review, because the watershed is considered as a system, then in pengembangannyapun, watershed should be treated as a system. By treating as a system and its development aims to meet the objectives of sustainable development, the target watershed development will create good characteristics as follows :

a. Able to provide high land productivity. Every area of land must provide productivity high enough so that it can support a decent life for farmers who cultivate it.
b. Able to realize, equalization productivity throughout the watershed.
c. Can ensure the sustainability of water resources

a. Housing and Settlement Development Approach Coastal Communities

Understanding the basic settlement in the Regulation No. 1 In 2011 is part of a residential neighborhood consisting of more than one housing unit that has the infrastructure, facilities, public utilities, as well as having other functions supporting the activities of the urban region or area of a settlement perdesaan.kegunaan is not only to provide shelter and protect the work place but also provide facilities for the care, communication, education and recreation

.

b. The cornerstone of Zoning Study

In achieving the ultimate goal of this study was to Assess the development of the spatial pattern of the estuary area of Makassar. We need a research methodology in accordance with the express and merumuskannya.Sehingga in this study, used the method of qualitative research based rationalistic philosophy which is based on rationalism. The

research method with a rationalistic approach requires the construction of meaning on empiri sensual, empirilogikal, and empiri ethics.

In this study need to be developed theoretical conceptual ability, with not just prepare objects. But look at all within the framework of the general holistik.Secara locations specified in the data collection site locations in the region that will be observed, as well as several locations in the vicinity of the site which is expected to be affected by the distribution of impact. In this way the environmental conditions or hue early in locations recipients can impact measurable / observable, so that later the magnitude of the impact in the study area can be predicted.

1. Assess and explore ecological knowledge of primary importance: the knowledge system and kebutuhanmasyarakat life and knowledge of local stakeholders, decision makers as users of the settlement area of the river estuary area Jeneberang and estuaries Tallo on water movement; the causes and consequences of the election / change of land use in a landscape (landscape) menurutpandangan various settlements users Jeneberang river mouth area and the mouth of the river Tallo.
2. Examining, observing, measuring environmental parameters expected to be affected by a large and important research activities.
3. Determine the environmental quality of the various parameters that are expected to hit large and significant impact on research activities.
4. Data collection and analysis of climate and hydrology settlements Jeneberang river mouth area and the mouth of the river Tallo, regional settlement data analysis Jeneberang estuaries and river mouths Tallo to obtain information about changes in land cover and watershed characteristics, analyzes the effect of land cover change on water balance including river flow settlements Jeneberang river mouth area and the mouth of the river Tallo using modeling and analysis of various scenarios of land cover changes that may occur to balance water use.
5. Study, observe, and measure the components of the action plan is expected to be affected by a large and important from the surrounding environment.
6. Estimates of changes in environmental quality initial research activities.

The goal of survey area settlement Jeneberang estuaries and river mouths Tallo is for: Digging and formulating ecological knowledge, experience, and perspective of local communities and stakeholders involved in the management of river mouths and estuaries Jeneberang overall Tallo. Digging and formulating ecological knowledge, experience, and perspectives of local communities and stakeholders involved in the management to achieve these goals, the methods of collecting ecological knowledge system approach based knowledge (knowledge based systems). Jeneberang river mouths and estuaries Tallo is done by modifying the data collection to save time and costs. A series of focus group discussions (focus group discussions) with local community groups and individual interviews with some stakeholders do with the snow ball technique to get an overview of ecological knowledge..

At this stage of the analysis, a statement from the local community and stakeholders also presented in the form of a diagram that illustrates the relationship between pernyataan.Sebelum conducted a series of interviews with some of the individual stakeholders, stakeholder identification and study of literature are steps being taken to get an overview of stakeholders that have a role important in the management of river mouths and estuaries Jeneberang Tallo in determining sebuahkeputusan.

Ecological knowledge study carried out together with the collection of spatial data and hidrologi.Langkah-step survey of ecological knowledge. To get an idea of ecological knowledge of local communities living in the region Jeneberang river mouths and estuaries Tallo, ecological knowledge survey conducted in two locations that represent: (1) the upstream and midstream Jeneberang and there in the area of the estuary; (2) the upstream and midstream Tallo and no area of the estuary. A series of focus group discussions (focus group discussion) at community level is done in coordination with relevant parties concerned with environmental issues in watershed management issues.

Each housing has its own system of values and practices that apply to warganya.Sistem the values differ between one housing with other housing, depending on the region or the state of society setempat.Secara bio-socio-economic and cultural geosfisik, coastal areas have special characteristics as a result of interaction among the processes that occur on land and in coastal lautan.Kawasan has many advantages including the potential to become a central location ekonomi.Penduduknya growth has social and economic activities are oriented to water and land, the average is economically weak educational backgrounds relatively limited. The majority of the population economically weak groups with limited educational background relatiff with knowledge of the environment healthy, harmonious, orderly and sustainable tends to be less, and occurs less environmentally conscious habits and tend to pay less attention to the hazards and risks..

In general, the estuary has an important ecological role, among others: as a source of nutrients and organic matter are transported through the circulation of tidal (tidal circulation), a provider of habitat for some animal species that depend on estuaries as a shelter and a place to find food (feeding ground) dansebagaitempatuntukberproduksidan / atautempattumbuh large (nursery grounds), especially for a number of species of fish and shrimp. Estuarine waters are generally exploited by human for the settlements, where the capture and aquaculture fish resources, means of transport, port and industrial area.

No activity in order to exploit the potential inherent in coastal areas, often overlap, so it is not uncommon resource utilization is actually degrade or destroy this ada.Hal potential for these activities, either directly or indirectly, affect the life of the organism in coastal areas, through environmental changes in the region. For example, the waste from both residential and industrial activity, although this waste may not affect the main constituent of plant or animal origin above the coastal ecosystem, but it is likely to affect biota other constituent. Heavy metals, for example, may have no effect on the lives of mangroves (mangroves), but extremely dangerous for the life of fish and shrimp-shrimp (crustaceans) that live in the jungle estuary is a merger of several rivers that converge and form a larger area, which influenced by several factors.

c. Characteristics Estuary

The type and pattern of distribution of seabed sediments which depends on the source and environmental conditions can be seen in shallow ocean sediment characteristics that are different from dalam.Sedimentasi sea and shallow sea the mechanism there is much influenced by waves, currents, detrital sources, rivers, interaction on the continent, and activities marine organisms, whereas in the sea, physics and chemistry factor as a determinant of the type and model of the sediment.

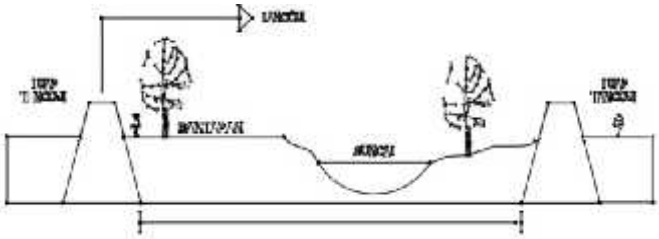

Fig. 2 Sketch of the River Plate

METHODS

1. Spatial Data

Spatial analysis in the RHA is done by using multiple spatial data types, namely: satellite imagery for mapping land cover, digital elevation model / DEM to map the characteristics of the watershed and thematic maps to view the configuration of the landscape. The data was obtained from government agencies or research institutes and other sources.

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2. Satellite Image

It takes satellite imagery from some point in time to make the land cover map of the river watershed and river Jeneberang Tallo. Land cover map over time (time series of land cover map) can depict changes in land cover at a certain time. In this study used Landsat satellite imagery with resolusis pasial 30 m were obtained from the Earth Resources Observation and Science (EROS).

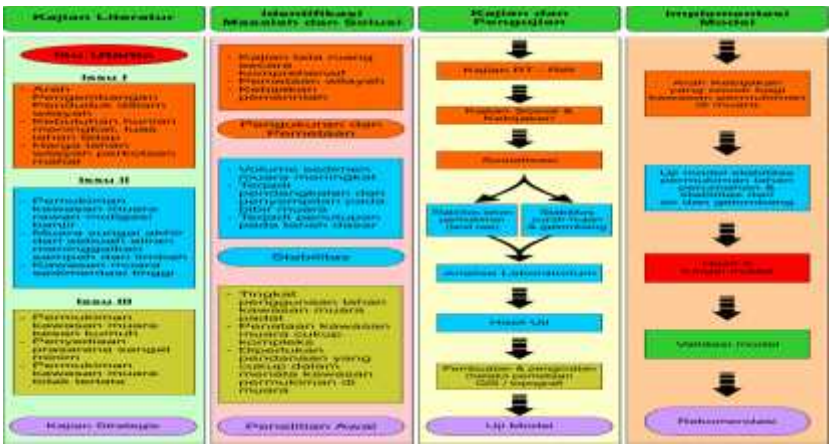

Fig. 3 Flowchart The Research

EXPECTED RESULTS

1. Identification of the characteristics of the river mouth area Jeneberang settlements and residential areas.
2. Produces spatial model of the building and the river estuary area.

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Undang-Undang Republik Indonesia Nomor 7 Tahun 2004 Tentang Sumber Daya Air (*Law 7/2004 on water resources*)

AN INVESTIGATION OF ON-ROAD MOTORCYCLE EMISSION UNDER HETEROGENEOUS TRAFFIC SITUATION ON AN ARTERIAL ROAD IN MAKASSAR

M. Arafah[1], M. Selintung[2], M. I. Ramli[3] and S. H. Aly[4]

ABSTRACT: This study attempts to grasp the characteristics of on-road motorcycle emission under heterogeneous traffic situation on an arterial road in Makassar City, Indonesia. The study conducted a field survey on an arterial road in the city to measure the exhaust emission of six motorcycles as the vehicle tests. The measurement used a portable emission measure which measured the motorcycle emission in second by second through the travel distance of the motorcycles. The equipment measured two types of motorcycle emission types, such CO, and HC. The survey results show that the exhaust emission of the motorcycle fluctuates according to the motorcycle motion. Furthermore, the average values of the CO emission type are vary from 177 ppm to 445 ppm. As well as, HC emission type has average values for the six motorcycles from 390 ppm to 654 ppm. In the other side, the total values of CO and HC emission types, vary from 17,700 ppm to 39,744 ppm, and from 39,000 ppm to 54,188 ppm, respectively. Briefly, index values of CO vary from 27 ppm/m to 45 ppm/m, and the index values of HC emission type vary from 56 ppm/m to 67 ppm/m.

Keywords: Motorcycle, emission, heterogeneous traffic, arterial road, Makassar City.

INTRODUCTION

In the recently years, the number of motor vehicle, in particular motorcycle is rapidly increasing in cities in developing countries such as in Makassar City, Indonesia (Arafah *et al.*, 2013a). The motorcycle population has been causing urban transportation problems in the cities such road traffic congestion (Hustim and Ramli, 2013), road traffic noise (Hustim *et al.*, 2011; Hustim and Fujimoto, 2012), and motor vehicle emission (Aly, *et al.*, 2013). For example in motor vehicle emission side, the motor vehicles have produced 14% of fossil-fuel-based carbon dioxide, 50% until 60% of carbon monoxide and hydrocarbons, and about 30% of nitrogen oxides emissions (Hwang *et al.*, 2007). In this context, the motor vehicle emission control is the important and urgent issue to be conducted on the global warming and climate change issue (Aly, *et al.*, 2011, 2013; Arafah *et al.*, 2013a). One effort in the emission control from the transportation policy is that the road administrators have to control the exhaust emission of motor vehicles on urban roads.

In order to control the motor vehicle emission, some efforts has been conducting in this field. For example, an inspection and maintenance (I/M) program to control air pollution from mobile source on road has been implemented (Simamora, 2006). The I/M program also has been conducting in some large cities in Indonesia such as in Jakarta, Bogor, Depok, Tangerang, and Bekasi area (Sudarmanto *et al.*, 2007), Bandung, and Surabaya (Sudarmanto *et al.*, 2010), and also Makassar (Aly, *et al.*, 2011; Arafah *et al.*, 2013b, 2013c). However, there are some second opinions addressed to the effectiveness of the I/M program in controlling the motor vehicle emissions on the roads (Hubbard, 1997; Wasburn *et al.*, 2001; and Bin 2003).

Regarding the evaluation of the achievements of the I/M program, some scholars have explored the emission level of on road vehicle and test failure model of cars on vehicle emission test related to factors that caused of the failure (Arafah *et al.*, 2013b, 2013c). In example, Pujadas *et al.* (2004) has used probit model in comparing vehicle emission factors in idle condition between experimental and calculated vehicle of fleet in Madrid City case, while Beydon *et al.* (2006) carried out a different approach model, such logit and regression analysis models. In Indonesia, Sudarmanto *et al.* (2007) has proposed bivariate probit model to grasp the failure of on-road passenger car emission in Jakarta City. As well as, Aly, *et al.* (2011) used binomial logit model approach for emission data of light vehicles in Makassar City. Furthermore, in particular the motorcycle emission in Indonesia, Sudarmanto *et al.* (2010) also has elaborated the similar approach in evaluating the motorcycle emission in Jakarta, Bandung, and Surbaya City. Regarding the motorcycle emission in Makassar, Arafah *et al.* (2013b, 2013c), has followed Aly, *et al.* (2011) methodology in order to describe the motorcycle emission phenomenon.

These previous studies have focused on the emission control of the motor vehicles under static condition. There was a little bit effort in the vehicle emission control under dynamic condition on urban roads in developing countries. In this

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regard Aly, *et al.* (2012), and Aly and Ramli (2013) have developed MARNI model, a modified IVEM model, in studying and predicting the behavior of the light vehicle emission under a heterogeneous traffic situation in Makassar, Indonesia. The authors of the present paper also have been focus to evaluate the relationship behavior between the motorcycle emission levels to the motorcycle motion under a heterogeneous traffic situation in the city (Arafah *et al.*, 2014). In order to extend the last previous research, the present paper attempts to investigate the behaviour of the on-road motorcycle emission under dynamic condition on heterogeneous traffic situation on an arterial road in Makassar City. In this regard, the present paper firstly describes the motorcycle emission phenomenon, and secondly we estimate the index values of CO (Carbon monoxide) and HC (Hydro carbon) emission.

The rest of this paper is organized as follows. Section 2 describes the study methods that involve the survey location, the survey equipments, the survey methods, and the analysis methods. Section 3 presents the results of the study including the data description and analysis. Finally, Section 4 presents conclusions of this study.

THE STUDY METHODS

The Survey Location

The location of the survey for the motorcycle emission measurement in this study was the Boulevard Street, an arterial road in the new CBD of Makassar City in Indonesia. The survey location as shown in Figure 1.

Fig. 1 The survey location

The Survey Equipments

The survey equipments consist of two main equipments, i.e., the test motorcycle and the exhaust emission measure. The test motorcycle involves six mototrcycle that selected in order to represent the various motorcycle types that available in Makassar City. The types of the six test motorcycles which used in this study are showed in Table 1. The

emission measurement of the motorcycles used a portable emission measure that designed for the study purpose as shown in Figure 2.

Table 1 The Types of the Test Motorcycle in this Study

The Test Motorcycles	Brand of Producers	Engine Capacity (cc)	Age (Years Old)
Motorcycle-1 (MC-1)	Honda Supra	125	> 7
Motorcycle-2 (MC-2)	Honda Tiger	200	4 – 7
Motorcycle-3 (MC-3)	Suzuki Satria	150	0 – 3
Motorcycle-4 (MC-4)	Suzuki Smash	110	4 – 7
Motorcycle-5 (MC-5)	Yamaha Mio	125	0 – 3
Motorcycle-6 (MC-6)	Yamaha Rx King	135	> 7

Fig. 2 The portable emission measure

The Survey Methods

The study carried out an exhaust emission measurement of the six motorcycles which passing on the arterial road, namely Boulevard Street, in second by second periods. The measurement used a portable emission measurement that designed for the study purpose. There are six motorcycles that used as the test vehicles, for representative of the motorcycle various types in Makassar City. The measurements of the exhaust emission was measured in second by second periodically. In order to represent the heterogeneous traffic situation in this study, the motorcycle emission measurement has followed the floating car method for the survey.

The Analysis Methods

The data survey were described using a statistical descriptive method in order to show the measurement data, and the statistical values, namely the average values and the total value of the emission, of the data. Finally, we estimated the index values of the CO and HC emission type the motorcycle emission.

THE RESULTS

The Measurement Results of the Motorcycle Emission

The results of the motorcycle emission measurement using the mobile equipment for the six motorcycles in the Boulevard street, are showed in Fig. 3 and Fig. 4 for CO and HC emission types, respectively.

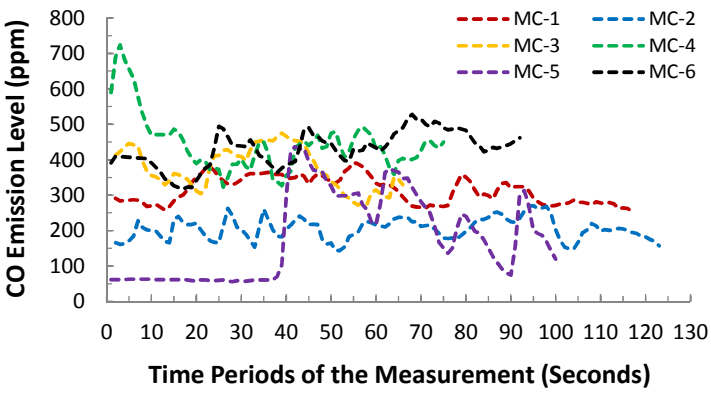

Fig. 3 The average level of CO emission type of the test motorcycles

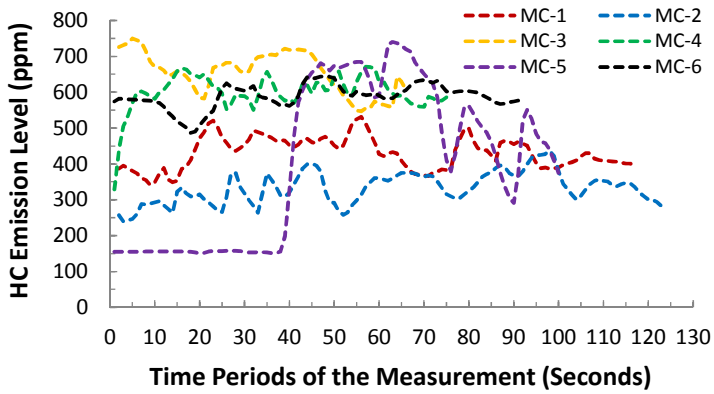

Fig. 4 The average level of HC emission type of the test motorcycles

Fig. 3 shows that the measured CO emission of the test motorcycles, are very fluctuative during the time period of the exhaust emission measurement, as well as for the measured HC emission as shown in Fig. 4. Furthermore, both figures, Fig. 3 and Fig. 4 describes the difference values of the starting values of both emission types, CO and HC, among the six test motorcycles.

The Statistic Value of the Motorcycle Emission

Regarding the results of the motorcycle emission measurement as shown in Fig. 3 and Fig. 4, we analyzed the emission data using the descriptive statistics approach. In this regard, we analyze the average values of both emission types, and the total values of CO and HC emissions for each test motorcycles. The average and total values of CO and HC emission types are showed in Fig. 5 and Fig. 6, respectively.

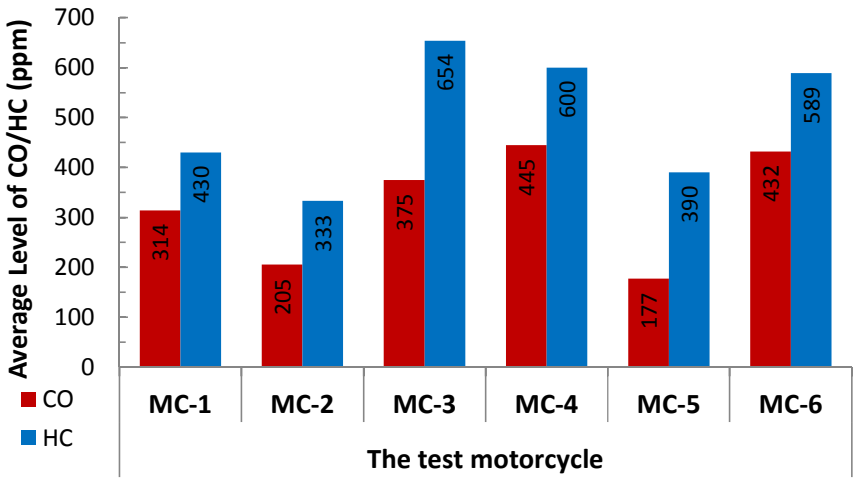

Fig. 5 The average level of CO and HC emission of the motorcycles

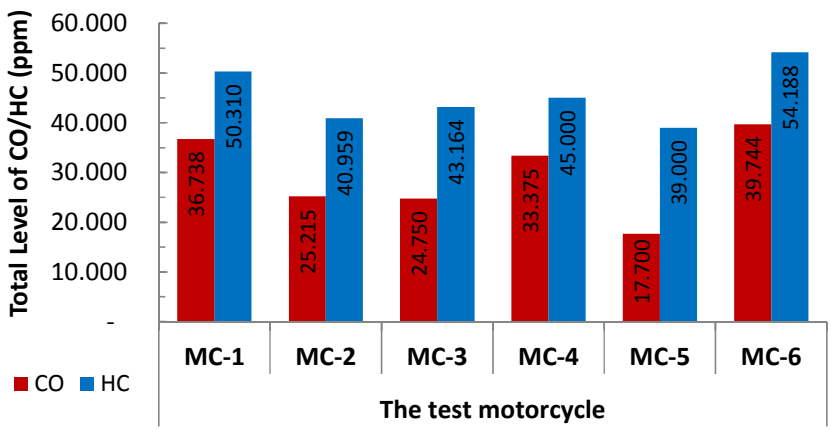

Fig. 6 The total level of CO and HC emission of the motorcycles

Figure 5 shows that the average values of the CO emission type are vary from 177 ppm to 445 ppm. In the other side, HC emission type has average values for the six motorcycles from 390 ppm to 654 ppm. In the total values of CO and HC emission types, Figure 6 shows that the total exhaust emission of the motorcycle vary from 17,700 ppm to 39,744 ppm for CO emission type, and from 39,000 ppm to 54,188 ppm for HC emission type. The values fenomena show that the exhaust emission are vary among the test motorcycles.

The Index Value of the Motorcycle Emission

According to the statistical values of CO and HC emission types, we propose to describe the phenomenon using an specific value that can represent the exhaust emission values objectively. In the present study, we estimated the index value of the motorcycle emission for the purpose. The estimation results of the index value is showed in Figure 7. Figure 7 shows that the index values of CO emission type vary from 27 ppm/m to 45 ppm/m, as well as, the index values of HC emission type vary from 56 ppm/m to 67 ppm/m.

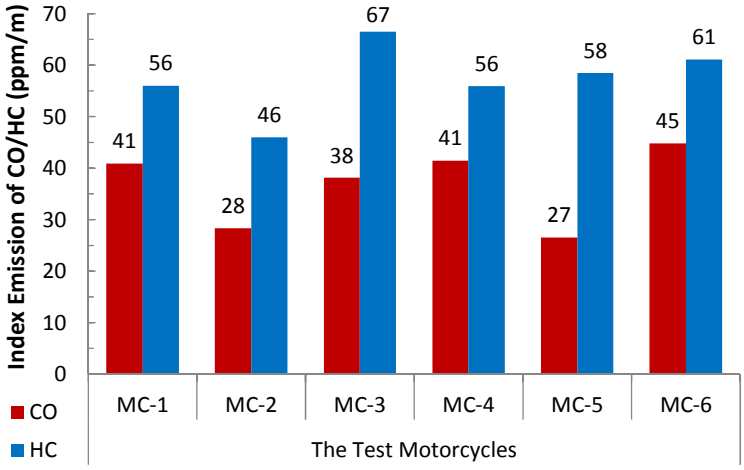

Fig. 7 The index value of CO and HC emission of the motorcycles

CONCLUSSION

The phenomenon of the motorcycle emission under heterogeneous traffic situation on arterial road in developing countries has been investigated in this study. Regarding a case study on an arterial road in Makassar City, one of the big cities in Indonesia, two type of the motorcycle emission, e.g., CO and HC emission types has been measured using an equipment for exhaust mobile emission test under on road condition of the six test motorcycles.

The CO and HC emission types has fluctuative values during the time period of the exhaust emission measurement, regarding the statistical values, and the index values of both emission types. The index values of CO and HC emission types vary from 27 ppm/m to 45 ppm/m, and from 56 ppm/m to 67 ppm/m, respectively.

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FACILITIES STUDY TERMINATION OF THE VALUE OF CITY TRANSPORT ERGONOMICS

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ABSTRACT: , Urban transportation bus stop systems are essential in supporting the city's ideal transportation facilities for urban communities . Technologies are developed over the years is to meet human needs . But if all that does not make people feel safe and comfortable , it means that in the process of designing and manufacturing , there has been a mistake orientation. Bus stops for public transport facilities in the city of Makassar to be made with regard to ergonomic aspects , but in fact that is not the case . For example, seating facilities at bus stops are not in accordance with the terms of ergonomics . the purpose of this study was to analyze the level of ergonomic facilities in relation to security and comfort on city transport stops facility . Data collection for this study is made by measuring the dimensions of the passenger's body and questionnaires were analyzed using house of quality and ergonomics approach. Safety level research results valuable facilities ergonomics dismissal of 95% percentile anthropometry results obtained sidewalk size 24.76 cm in hight, chair seat at stop halt with the size of 36.21 cm in width, 44.45 cm seats in height and backrest in height 56.92 cm and 411.4 cm in length

Keywords: safety, comfort, amenities termination, ergonomic

INTRODUCTION

Transportation means with public transports in the city of Makassar should be made with regard to ergonomic aspects, but not the case with public transportation. Respect for human values in general seems to still come second after economic objectives. So that in many cases there are still many human values are not used as an important reference in the engineering process.

The problems available in this research is about the public transport stops in the city of Makassar. The first issue, the safety of the sidewalks as used by pedestrians have varied measure in hight: there are very low, and some others are higher, and most of the roads in Makassar do not have sidewalks. Those of having no sidewalks cause the public transport users getting difficulties to get on vehicles. Secondly, lay off public transport passengers in vain place, the same thing ascending descending passenger public transportation, since the number of stops is very less, the existing facilities at the bus stop, for example: the seat is not in accordance with the dimensions of the human body, as well as the area is not adequate as a stopover vehicle.

In a research study to be achieved is a study of public transport stops to be safe and comfortable with using ergonomics. Based on the above description, the problem can be formulated in this study how to assess the level of safety and ergonomic comfort to public transport stops facility. while on the formulation of the problem, then the purpose of the study is to analyze the level of safety and ergonomic comfort to public transport stops facility

STRATEGIC ISSUES RESEARCH STUDY

Transportation is a business move, transport or transfer an object from one place to another more useful or may be useful for certain purposes. Transportation is defined as the transfer of goods and people from place of origin to destination. (Dedi S. S).

Bus stops in major cities in the world, the bus stop is one of the public spaces in the cities as the place where the bus transit, ascending and a descending passengers. Each stop has two lines, ie gate transit and for pedestrians. In the city of Denver, the bus stop mall shuttles have seats move, people can set their own seating relaxed while waiting for the bus. At stops and bus stop in the city of Morella Mexico, passengers can take a snack of fresh mango or pineapple. City of Portland, the bus stops in the city has a good information system on the transit system, ie each route given special logo color code for each direction, helping people navigate the bus network and can show the bus schedule. Bus stop in the city of Los Angeles, passengers must wait on benches ads. There is a sense of apprehension of self-protection which is

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too close to the traffic. While in the city of Barcelona, where the bus stops are made as thin as possible and transparently and in a beautiful shelter design and passengers can sit comfortably (Suisman, Doug).

Structuring the public transport system is not good to add to existing problems such as overlapping routes, the number of fleets that are too large, the low level of services, long journey times and others. This will lead to a decreased level of public transport services and can sometimes increase the level of congestion on the road.

The public transport services will be reviewed from several major dimensions such as infrastructure, facilities, and operating systems that have attributes such as Time, Cost, Safety, comfort, and performance expectations and perceptions so that users of public transportation services can be represented and accommodated (Dedi S.S).

Transportation systems

The transport system can be described in the form of macro transportation system which is divided into several subsystems micro interrelated and influence each other. The micro subsystem includes system activity (transport demand), the system of transport infrastructure networks (transport supply), system traffic movement (traffic) and the institutional system. Every system of land use activities or have certain types of activities that generate (generation) and pull (attraction) movement. The movement of people or goods that happen requires modes of transport (means) such as cars, buses, trains and transport infrastructure such as road networks, railways, terminals, airports, ports and stations. Activity and network systems will interact and generate the movement of people or goods. Changes to the system will affect the activities of the network system through a change in the service level movement system. Conversely, changes to the network system will also affect the system.

Dismissal facilities greatly affect the smooth movement of vehicles from one stopping point to the next stop. Where the bus system in the United States reduce some bus stops. Eliminating some bus stops can reduce about 40% of the time to stop. (Ranjay M. S dan Edmund J. Z).

BASIC CONCEPTS

The ultimate goal of man to always amend the design of the equipment used to facilitate the operation and impose its use. Scientific disciplines born and developed around the 20th century related to the design of equipment and working facilities considering the aspects of man as the wearer is known then under the name of ergonomics.

The application of ergonomics in general is the activity of design (design) or redesign (redesign). This may include hardware, such as: working tools (benches), platform, seat, handle work tools, control systems, props, street / alley, doors, windows, and others. Still in the discussion is about the design of the work environment, because if the system hardware in the form it will also change the work environment.

RESEARCH METHODS

Qualitative data collection was conducted to determine what desires the use of public transport in the city of Makassar in a way that made the spread of the questionnaire on the use of public transportation with a variety of questions / items.

Data collection is done directly on the comfort and safety on public transport stops, especially on the facilities and infrastructure such as roads kretria geometric design.

Once the data is obtained and then continued with statistical testing

- In a test of uniformity of data where the data is said to be uniform if located between Upper Control Limit and Lower Control Limits using a 95% confidence level and 5% level of accuracy.
- Adequacy test data to determine the data point fairly or not the condition N '<N, with a confidence level of 95% and 5% level of accuracy.
- Test percentile, percentile test using 5-th percentile, 50-th and 95-th.

ANALYSIS METHOD

In this study, a population which is the object of research, to determine the amount of sample used Bernoulli equation, namely:

$$N \geq \frac{\left(Z_{\alpha/2}\right)^2 p.q}{e^2} \quad (1)$$

Correlation calculation on each variable with a total score using the formula product moment, that is:

$$r = \frac{N(\sum xy) - (\sum x \sum y)}{\left\{\left[N \sum x^2 - (\sum x)^2\right]\left[N \sum y^2 - (\sum y)^2\right]\right\}^{1/2}} \quad (2)$$

The formula for the coefficient of variance (with cronbrach), as shown in the following equation:

$$\sim = \frac{k.\ r}{1 + (k - 1)\, r} \quad (3)$$

Testing uniformity observation data as a control tool calculated the average (mean), the upper control limit (BKA), lower control limits (BKB), using a 95% confidence level and 5% level of accuracy.

$$B = \bar{X} + 2\,\sigma_x \quad \text{and} \quad B = \bar{X} - 2\,\sigma_x \quad (4)$$

By using a 95% confidence level and 5% level of accuracy. Terms of body dimension data measured for each dimension will have a storage no more than 5%. Data said to be sufficient if N '<N, By using the formula, namely:

$$N' = \left[\frac{K / S\sqrt{N\sum X_i^{\,2} - \left(\sum X_i\right)^2}}{\sum X_i}\right]^2 \quad (5)$$

Test normality of the data was conducted to test whether the data were normally distributed or not with the formula:

$$i^2_{hit} = \sum \frac{(Q_i - e_i)^2}{e_i} \quad (6)$$

Terms normality of data: $i^2_{hit} \leq i^2_{tabel}$ (7)

Conducted if the data is not normally distributed. Layout percentile formula:

$$L_i = \frac{in}{100} \quad (8)$$

Percentile values

Percentile 5-th = $\bar{X} - 1.65\,\sigma_x$ (9)

Percentile 50-th = $\bar{X}$ (10)

Percentile 95-th = $\bar{X} + 1.65\,\sigma_x$ (11)

RESULTS AND DISCUSSION

Security level facilities dismissal based on the level of interest, the perceived level of satisfaction and contentment expected then analyzed by menggunan Quality Function Deployment (quality house) obtained the results of the first priorities and priorities fifth stop on the sidewalk, where the user needs to repair the vehicle and vehicle owners want their repair and incoming priorities. Pavement based on the results of anthropometric dimensions of the body by testing the 95-th pesentil influential knee high adalahsetengah obtained 24.76 cm. half the height of the knee is ideal in terms of the movement of users in stepping onto the road pavement.

Stop facility dismissal was based on the results of research anthropometric analysis seating is not much different from that on the vehicle seat only difference is the material while the result is wide seats with 95-th percentile is 36.21 cm, 44.45 cm high seat, high backrest 56.92 cm, and a length of 411.4 cm seat.

Discussion of the results of research that analyzes the 95-th percentile anthropometry obtained measure 24.76 cm high pavement. While the results of previous research community Anthropometrics premises obtained from the public interpolation British and Hong Kong (Pheasant, 1986) against to the people of Indonesia (suma'mur, 1989) as well as dimensional terms of (Nurminanto, 1991). knee height gained 54.4 cm or halved then obtained 27.4 cm. Whereas a seat on the seat width dieperoleh stop with the size of 36.21 cm, 44.45 cm seat height and backrest height 56.92 cm.

Previous research Nurmianto (1991) in his research Anthropometrics Indonesian society, high knees folded poplitian obtained 44.5 cm size. And a thick belly (abdominal) as a reference width of the seat with a size of 28.2 cm.

Table 1 Comparison Value Ergonomics at bus stops and sidewalk seating

Commentaries	Measurement stops in (cm)			
	Present condiition	Dep. Hub. DJP. Land	Previous research	Ergonomics value
Seats Height	40 – 67	50	44,5	44,45
Seats width	30 – 70	40	28,2	36,21
Backrest height	40 – 60	50	62,1	56,92
Seats length	2,5 – 6,5	400	408,1	411,4
Pavement height	17-32	10-30	10 - 20	24,76

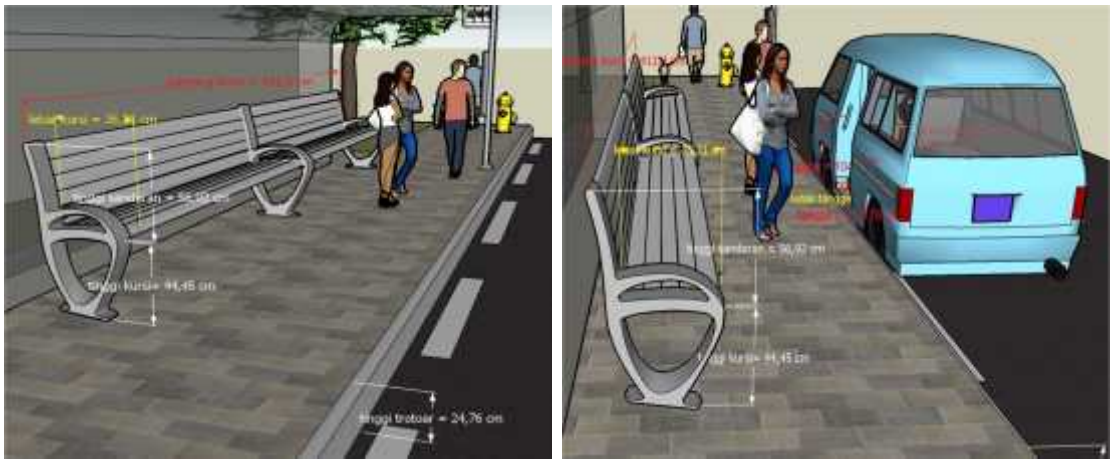

Fig 1. Facility stops

EMPIRICAL RESEARCH FINDINGS

Third is the empirical research findings obtained high chair 44.45 cm while the previous study Joan Duncan, Professor D. Ferguson, G. Andrews and P. Peach,) "Duncan Chair" was developed in Australia chair with seat height 46 cm. while the seat width refers to the buttocks to the abdomen 36.21 cm and 56.92 cm high backrest. While Nurmianto (1991) in his research Anthropometrics Indonesian society, high knees folded poplitian obtained 44.5 cm size. And a thick belly (abdominal) as a reference width of the seat with a size of 28.2 cm.

CONCLUSION AND RECCOMENDATION

The conclusion of the research study safety and comfort facilities public transport stops are worth ergonomics security level facility that stops the high-value ergonomics pavement 24.76 cm size. while the dismissal of the facility that is the seat stops seat width with the size of 36.21 cm, 44.45 cm seat height and backrest height 56.92 cm and a length of 411.4 cm. Level of comfort and security of public transport stops facility that is mutually teritegritasi between high pavement, facilities with public transport stops are worth ergonomic.

We recommend public transport in the city of Makassar need special attention, where many vehicles that do not fit ergonomic.

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INTERACTION EFFECT OF TRAFFIC CHARACTERISTICS, PHYSICAL OF ROAD, AND ROAD ENVIRONMENT ON NOISE

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ABSTRACT: All this time, efforts that were made to model the factors affecting the traffic noise generally only focus on the traffic aspect. Those efforts were made to build a mathematical/regression model. The purpose of this research is to develop a structural equation model to predict the interaction of traffic characteristics, physical aspects of the road, and the road environment, and its influence on the noise level. The analytical approach method is Partial Least Square (PLS) using software SmartPLS 2.0M3. The results showed that simultaneous variability of noise level can be explained by traffic characteristics, the physical aspects of the road, and the road environment by 62.80%. While 37.20% was explained by other factors that were not examined in this model. Structural equation models creates a concept that factors directly affecting the level of noise was only the traffic characteristics and environmental of the road. While the physical factors showed no direct influence on the noise, but it has a significant influence on traffic. In this case, physical factors of roads serves as an intervention variable that can trigger noise due to its influence on traffic characteristics. In addition, the environment factors of road have an influence of the interaction against physical aspects of road. Factors that significantly directly influence the level of noise namely characteristic of the traffic and road environment. Where the traffic characteristics are the most dominant factor influencing noise. While physical factors do not directly influence road noise but rather significantly to traffic. It can be interpreted that an increase or decrease of noise as a result of the traffic is strongly influenced by the intervention of road physical factor.[1]

Keywords: *effect, traffic, road physical, road environment, noise.*

INTRODUCTION

Several studies conducted in many countries showed that the highway is the main source of noise. It is caused by the immense amount of motor vehicle usage - including vehicles with two wheels, four wheels, and the wheels over four- compared to other types of vehicles. Each type of motor vehicle has a specific frequency. According to Mediastika (2005), each road has a different noise characteristics. This difference occurs because the traffic noise level is determined by many factors, such as motor vehicles passed, including: (1) the total number of vehicles per hour, the ratio of the types of motor vehicles passing through, and the average speed of vehicles, (2) the characteristics of the road, which is the road class (which generally includes the width and length of the road), the number of lanes (capacity), as well as the quality of the road surface, the slope of the road, road traffic flow arrangements (eg way road with no slow lane, two-way road with no slow lane, adjacent to the zebra-cross, near the traffic-light, and so forth), (3) other conditions in the road's surrounding, including the buildings on the side of the road and informal bustle along the edge of the road, such as street vendors, parking lots, and so on. Handayani (2007), states that the traffic noise is influenced by a number of parameters that do not depend on the vehicle itself, such as traffic parameters, environment parameters, weather parameters, as well as the parameters of residence.

Various attempts were made to identify the factors affecting the level of traffic noise in addition to the non-physical factors of vehicles, including; Murase et al (2000), attemped to construct a model of priority lane public nuisance in Sweden, Japan and Thailand to noise by non-acoustic factors; Rudianto (2003), used distance, vehicle speed, traffic density, and wind speed variables to generate multiple regression equation; Sooriyaarachchi (2008), has built a traffic noise prediction model based on the speed and distance of various types of vehicles; Golmohammadi (2009), has done a comparison of several models in order to predict traffic noise which can be used effectively under the conditions of cities in Iran with width and length of the road, gradient, high buildings, the volume of the vehicle, speed, and the percentage of truck as independent variables; Wedagama (2012), generates simple and multiple regression models the effect of local traffic on noise levels for arterial roads in Bali with locus By-Pass Ngurah Rai and Sunset Road, using the independent variable of traffic volume (MC, LV, HV), speed, and distance observations as independent variables.

In principle, the factors that trigger noise reviewed are still largely focused on the aspects of traffic by looking at the correlation or impact on the level of noise partially. Of some mathematical models generated showed that there are

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other factors that have not been studied. Based on that, it would require a more comprehensive study involving several suspected factors simultaneously affecting the level of traffic noise. Factors that are reviewed not only regarding traffic characteristics, but also road physical aspects and road environment.

MATERIAL AND METHODS

1. Study Location and Variable

This research was conducted in the city of Kendari, Southeast Sulawesi Province, Indonesia with a locus on arterial and collector roads. Selection of Kendari as the test site was because the city is one of the medium category with heterogeneous traffic characteristics that had a high growth rate of motor vehicle with average of 19% per year. In addition, the physical characteristics of the road and road environment is quite varied, so it supported the needs of the data to solve the substance research problem. The variables and indicators that are used is shown in Table 1.

Table 1. Resesarch Variables and Indicators

Variable/Construct	Indicator	Code
Traffic Characteristics (Tr)	▪ Traffic volume ▪ Composition of motorcycle ▪ Composition of light vehicles ▪ Composition of heavy vehicles ▪ Average speed ▪ Traffic density	▪ Vol ▪ KoMC ▪ KoLV ▪ KoHV ▪ V_{av} ▪ D_{tr}
Physical of Road (PR)	▪ Width of Road ▪ Superelevation ▪ Gradient ▪ Type of road ▪ Condition of road pavement	▪ L_{ja} ▪ Spel ▪ Grad ▪ Typ ▪ Cond
Environment of Road (ER)	▪ Density of buildings ▪ Density of trees ▪ Volume of leaf shades ▪ Type of ground surface road env. ▪ Temperature	▪ D_{bg} ▪ D_{ph} ▪ V_{da} ▪ P_{bu} ▪ Su
Noise Level (NL)	▪ Noise exceeded L_{10} ▪ Noise exceeded L_{50} ▪ Noise exceeded L_{90} ▪ Equivalent continous noise	▪ ExL_{10} ▪ ExL_{50} ▪ ExL_{90} ▪ L_{eq}

2. Partial Least Square Method (PLS)

It is known that regression analysis technique cannot accommodate the structure of the actual relationship between variables (theoretical). To be able to accommodate the interaction relationship between these variables, then the solution is the system of simultaneous equations using the method of partial least square (PLS). Partial Least Square is a powerful analytical method because it is negating OLS regression assumptions , such as: data may not necessarily distributed as normal multivariate , the indicators can be applied to categories, ordinal, interval, and ratio on the same model, the number of samples may not be necessarily big (recommended minimum of 30 samples) (Ghozali, 2014). PLS analysis consists of two sub-models: namely inner model, which is also, called the structural model that describes or explains the strength of the relationship between factors (latent variable). While the outer model or measurement model (measurement model) explains how manifest variable or indicator representing latent variable to be measured.

Stages that determine the structural modeling is model evaluation. Evaluation of models with PLS method consists of the evaluation of the measurement model, including the validity and reliability. Basic testing to meet the validity criteria and reliability of loading factor value must be> 0.5. While the structural model evaluation is done by looking at the value of R-square of each dependent construct (endogenous) as the predictive power of the structural model, Stone-Geisser Q-square to measure how well the observed values generated by the model and estimation of its parameters, as well as t-test to see significance of structural path coefficient. The equation to calculate the Q-square is (Jaya 2008):

$$Q^2 = 1 - (1 - R_1^2)(1 - R_2^2) \ldots (1 - R_p^2) \quad (1)$$

Where R_1^2, R_2^2, … R_p^2 is R-*square* endogenous variables in the model. The total of Q^2 has a value in the range of $0 < Q^2 < 1$, it means that the more it gets closer to 1 then it means the better predictive model relevance. Confidence level

used in this study was 95%, so the chance of error or significance level is (α) = 5% or 0.05. The stages of the PLS-SEM analysis is shown in Figure 1.

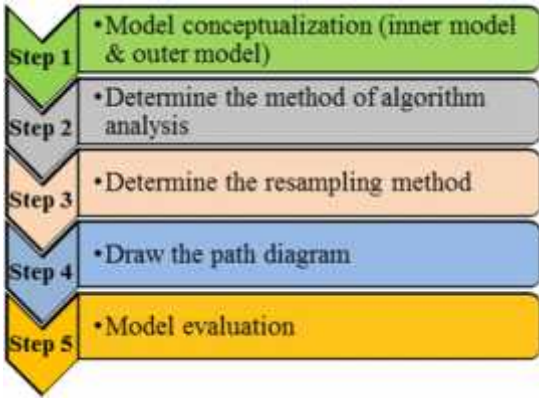

Fig. 1 Stages of PLS Analysis

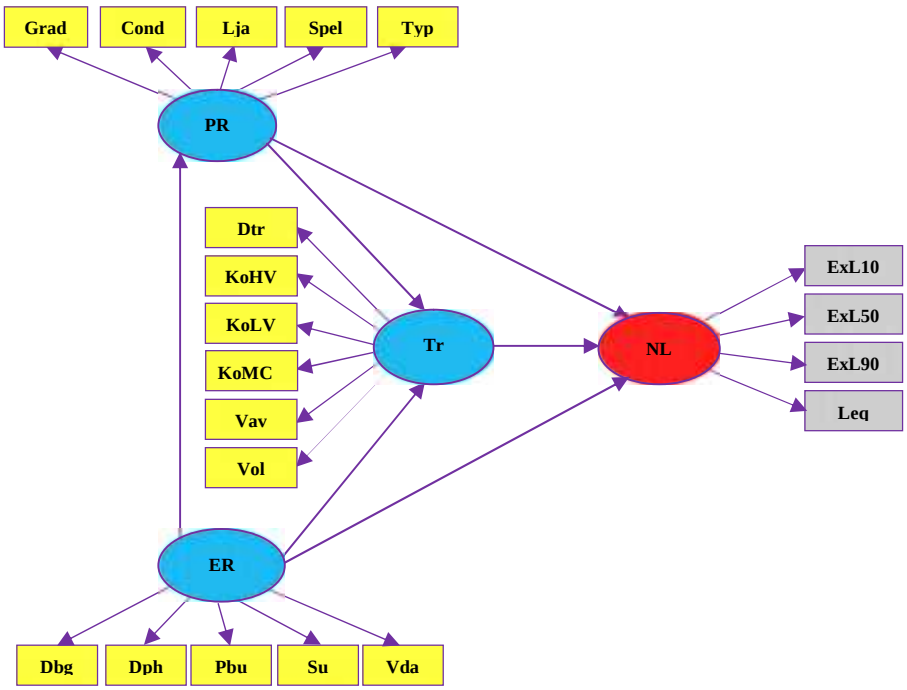

Fig. 2 Path Diagram concept of Interaction Model

RESULTS AND DISCUSSION

1. Evaluation of Measurements Model (Outer Model))

Validity Test

The design of the measurement model is intended to measure the dimensions that make up a factor based on empirical indicators, which was evaluated using confirmatory factor analysis technique. This model was used to determine the validity and reliability connecting indicators with latent variables. Each indicator should be able to form a construct that is intended to be valid. Indicator is considered as valid if it has the loading factor above 0.5 to construct the intended. After the initial run, then the output of SmartPLS 2.0M3 as shown in Figure 3.

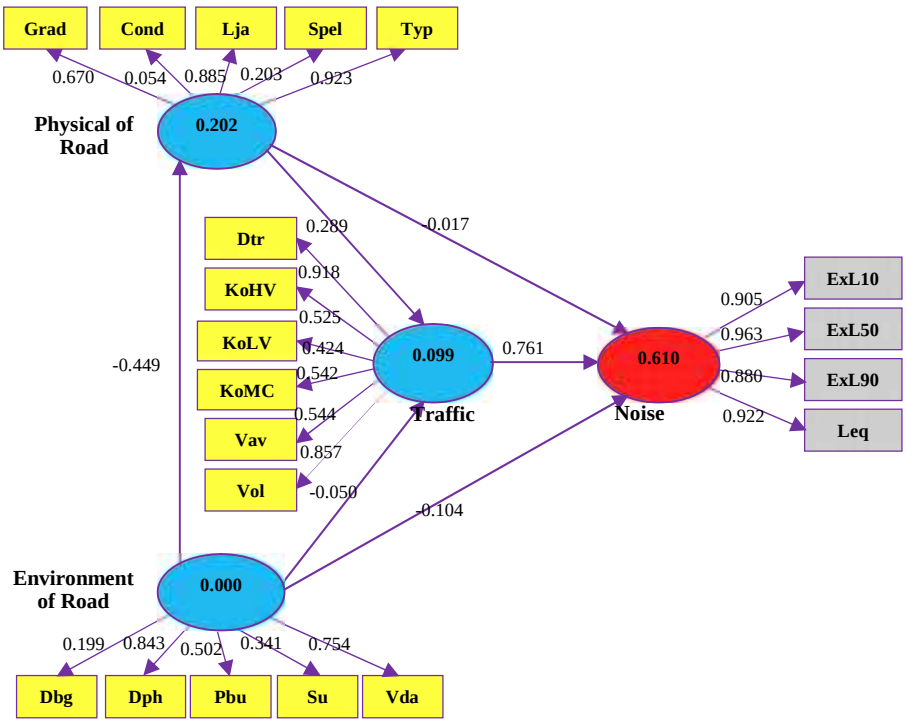

Fig. 3 Output Outer model

Figure 3 shows that there are some indicators that have a value below the loading factor of 0.5. For example for street physical factors namely pavement condition indicators (Cond = 0.054) and superelevation (Spel = 0.203). On the roads environmental factors namely indicator of the building density (Dbg = 0.199) and temperature (Su = 0.341). While the traffic factor namely the composition of the light vehicle (KoLV = 0.424). Therefore, these indicators should be excluded from the model. After the indicators issued then performed a run again, then the value of loading factor and cross loading of each indicator of model execution results as shown in Table 2 (Output SmartPLS 2.0 M3).

Table 2. Result for Cross Loading

	Noise	Physical of Road	Traffic	Env of Road
Dtr	0.686	0.219	**0.926**	-0.072
Dph	-0.145	-0.339	-0.105	**0.911**
ExL_{10}	**0.903**	0.228	0.660	-0.099
ExL_{50}	**0.963**	0.226	0.771	-0.172
ExL_{90}	**0.882**	0.224	0.787	-0.113
Grad	0.131	**0.648**	0.215	-0.228
KoHV	0.334	0.126	**0.564**	0.095
KoMC	0.275	0.089	**0.581**	0.221
Leq	**0.922**	0.190	0.624	-0.130
Lja	0.204	**0.899**	0.213	-0.297
Pbu	-0.170	-0.175	-0.024	**0.571**
Vav	0.319	-0.100	**0.552**	-0.014
Typ	0.242	**0.933**	0.307	-0.407
Vda	-0.053	-0.378	0.062	**0.881**
Vol	0.814	0.413	**0.861**	-0.116

In Table 2 it appears that all the indicators had met the criteria of convergent validity with loading factor values above 0.50 (see value in blue bold text). To see if an indicator has a higher loading factor against its construct intended than the other construct that is by looking the value of Cross loading. For example, an indicator of traffic density (Dtr) has the highest loading factor values against its construct namely traffic amounted to 0.926 compared to other constructs i.e. noisy (0.686), physical road (0.219) and the road environment (-0.072). This suggests that the density indicators (Dtr) is really an indicator of traffic. Similar results were also seen in other indicators that also has a correlation value against the higher its construct from the other constructs such as; Dph, ExL_{10}, ExL_{50}, ExL_{90}, Grad,

KoHV, KoMC, Leq, Lja, Pbu, Vav, Typ, Vda, and Vol. It can be concluded that these indicators can be used to measure each construct.

Another way to test the discriminant validity is to look at the value of average variance extracted (AVE), which the value suggested is above 0.5. In Table 3 shows all the variables in the model estimated have fulfilled the criteria of discriminant validity, namely the value of Average Variance Extracted (AVE) is above 0.5.

Table 3. Average Variance Extracted

Construct	AVE	√AVE
Noise	0.842	0.918
Physical of road	0.699	0.836
Traffic	0.590	0.768
Environment of road	0.644	0.802

Reliability Test

Reliability test is a test to see the accuracy, consistency and accuracy of an instrument to measure the construct. Reliability testing is done by looking at the value of composite reliability from the indicator block that measures the construct. Composite reliability results will show a good value if above 0.7. Reliability test can also be seen from the value of Cronbach's Alpha, which is above the recommended value of 0.6. In Table 4 it appears that composite reliability values for all constructs that is above 0.7 which shows that all constructs in the model estimated has met discriminant validity criteria. It is also reinforced by looking at the value of Cronbach's Alpha, where Cronbach's Alpha value is generated for all constructs that is above 0.6. Thus all the variables have good validity and reliability.

Table 4. Reliability Test Results

Construct	Composite Reliabilty	Cronbach Alpha	Note
Noise	0.955	0.938	Good (*fit*)
Physical of road	0.872	0.775	Good (*fit*)
Traffic	0.818	0.738	Good (*fit*)
Environment of road	0.839	0.709	Good (*fit*)

2. Evaluation of Structural Model *(Inner Model)*

Structural model was evaluated using the R-square for dependent constructs as the predictive power of the study model , Q-square test for predictive relevance, as well as t-test to see the value of their structural path coefficient. R-Square analysis results obtained coefficient of determination (R^2) = 0.628, which means that the model has a good goodness of fit. It is understood that simultaneously noise variability can be explained by three variables in the model, namely the physical road, traffic, and the road environment by 62.80%. While 37.20% is explained by other factors, which have not, been examined in this model. Partially, the interaction of exogenous construct effect towards endogenous in the structural model can also be identified by squaring the value of the latent variable Correlations. The magnitude of effect interaction is shown in Table 5.

Tabel 5. Effect of Partial Inter Construct

Construct	R-*square*
Physical of Road -> Noise	5,7
Traffic -> Noise	61,2
Environment of Road -> Noise	2
Physical of Road -> Traffic	8,9
Env. of Road -> Traffic	0,0
Env. of Road -> Physical of Road	14,8

In Table 5 it appears that traffic factor has the most dominant influence on the noise , namely 61.2% followed by 5.7% of the physical and road environment by 2%. Of the factors triggering noisy itself is also having an influence interactions, such physical factors of road towards traffic by 8.9%, the road environment towards physical road by 14.8%. For the predictive relevance model value in overall (Q^2) is:

$Q^2 = 1 - (1 - (R^2)$
$= 1 - (1 - (0{,}628)^2)$
$= 0{,}4$

The Q^2 value is in the range of 0 to 1.0. The closer to 1 means a better predictive relevance. From the calculation results the Q^2 value obtainedis 0.4. This indicates that the physical aspect of road, traffic, and road environment variables has a good predictor to the level of noise. Justification interaction among the factors triggering noise and the effect on noise were identified based on the value of t-statistics obtained through bootstrapping procedure. The inner weight coefficients in the model is considered significant if the value of the t-*stat* > t-*table*. The t-*table* value for a significance level of 95% is 1.96.

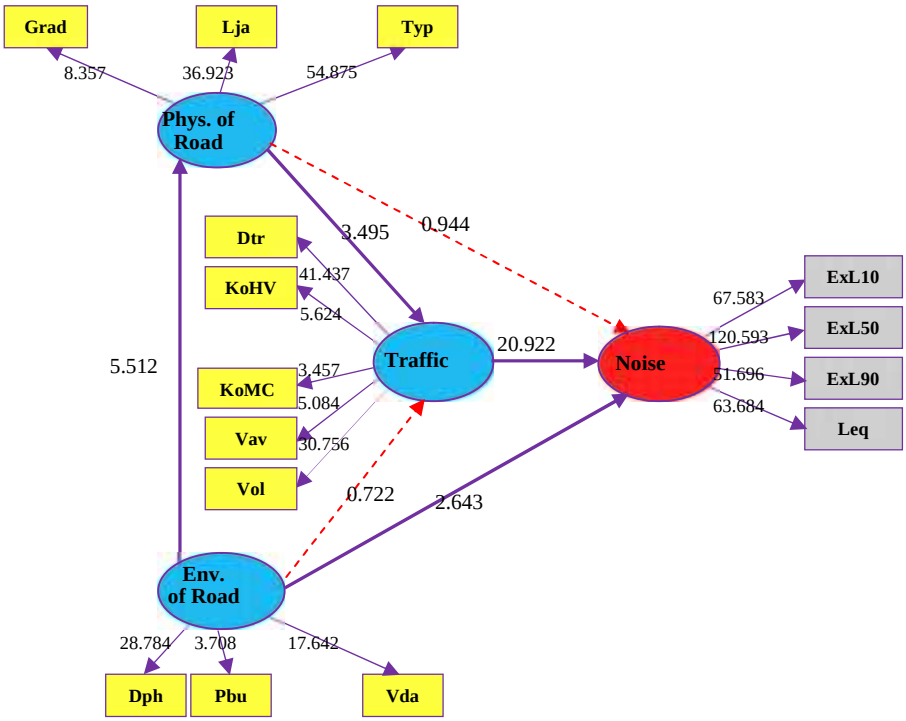

Fig. 4 Visualization of Structural Model

Figure 4 shows that the physical factor of road is manifested by the gradient (Grad), width of the road (Lja), and type of road (typ). Traffic factor is manifested by the volume of vehicles (Vol), composition of motorcycle (KoMC), composition of heavy vehicles (KoHV), speed (Vav), and traffic density (D_{tr}). Road environment is manifested by the density of trees (Dph), volume of leaf shade (Vda), and type of ground surface road environment (Pbu). While the noise level is manifested by noise exceeded (ExL_{10}, ExL_{50}, ExL_{90}), and L_{eq}. Those indicators were considered as variable measurer. Simultaneous structural model shows that among business trigger variable is actually a full interaction.

Based on the value of $t\text{-}_{stat}$, thus the influential factors for the noise which is significantly affecting directly against th elevel of noise is namely the factors of traffic and roads environment with t-count value of 20,922 and 2,643 bigger than t-table of 1,96. Whereas road physical aspect does not significantly influencing directly against noise. This could be interpreted that roads physical factor is only the intervention factors or cause of the traffic characteristics change, such as changes in speed due to the gradient or the type of road.

Table 6. Hypothesis Test

Construct	T-*tabel*	T-*Statistics* (|O/STERR|)	Note
Physical of Road -> Noise	1.96	0.944	H1 *rejected*
Physical of Road -> Traffic	1.96	3.495	H1 *accepted*
Traffic -> Noise	1.96	20.922	H1 *accepted*
Environment of Road -> Noise	1.96	2.643	H1 *accepted*
Env. of Road -> Physical of Road	1.96	5.512	H1 *accepted*
Env. of Road -> Traffic	1.96	0.722	H1 *rejected*

CONCLUSION

1. Simultanously, the variability of noise level can be explained by the traffic characteristics, physical aspect of road, and road environment by 62,80%. While 37.20% was explained by other factors that were not examined in this model.
2. Factors that significantly directly influence noise levels is only traffic characteristics and road environment. Where the traffic characteristics are the most dominant factor influencing the noise. While physical factors do not directly influence the road noise but rather significantly to traffic. It can be interpreted that an increase or decrease of noise as a result of the traffic is is strongly influenced by the intervention of road physical factors.

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THE PREFERENCE STUDY ON PRIVATE TRANSPORT MODE OF TRANSFORMATION BASED ON USER BEHAVIOR

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ABSTRACT: One of the causes in congestion that the number of vehicles are not proportional to the volume increase. Nowadays, common people prefer to use privite vehiches than public transport and various reasons such as convenience, faster travel time, the capacity of public transport is not operating as it should and so forth. This study is a study exploring perceptions of transformation preference for personal transport users (four-wheel and two-wheel) in terms of the behavior of the user journey mode. Draft analysis approach consisted of descriptive statistical methods to obtain the user travel behavior and perceptions of personal transport modes, as well as structural equation modeling (SEM) to obtain travel behavior user interaction modes of private transport towards the transformation of modes of perception. The Expected results are: (1) the characteristics of travel behavior and user perception of personal transport modes, (2) interaction model the characteristics of travel and travel behavior user and its relation to perception transformation of private transport modes

Keywords: : transformation, modes, travel behavior, private transport

INTRODUCTION

a.Background

Transportation problems in big cities such as Jakarta, Bandung, Surabaya, Medan and Makassar, is an issue that is quite alarming, especially in solving problems associated with public transport traffic jams. Compared with major cities in developed countries, becomes an irony that the public transport service system in our country is very underdeveloped and quite alarming. In developed countries people are more likely to use public transport than the people use of private vehicles. However, in major cities such as Makassar, in general, people prefer to use privite vehicles to public transport with a variety of comments and reasons such as discomfort, longer travel time, the capacity of public transport is not operated properly so that security cannot be obtained and there are many other reasons.

Aspects of the population, information technology, and values that are different in developed countries with cities in developing countries such as Indonesia. Therefore, this study specifically conducted on private transport when compared with previous studies, both in terms of types of modes, service level or even more priority to the provision of service (supply) rather than the demand side (demand).

Makassar city with an area of 175.77 km2 as the core city of Mamminasata Metropolitan Region and serves as the National Events Centre in eastern Indonesia with a population of approximately 1.5 million people that growth rate of 9% per year. Economic indicators town with PDRB dominated by trade and services sectors. Therefore, the Metropolitan Municipality Makassar very high tensile power, but faced various urban problems, namely urbanization and poverty, slums, transportation, flood, water, sanitation, and trash. An inter-city movement, moda choice factor plays is a fairly important, someone who would move from one city to another would have to consider a lot of things.

Transport problems encountered include: (1) the imbalance of development of road infrastructure dihandingkan with growth vehicle, (2) the growth of road infrastructure of approximately 4% per year, and (3) the growth of the vehicle approximately 14-15% per year. A survey conducted by ARSDS concluded that the ratio of the use of private vehicles and public transport is 611: 35. This clearly shows the inefficiency of use of road space by the pattern of use of the vehicle. The growth of the use of vehicles in Makassar showed the tendency of utilization of the motor vehicle and the disappearance of non-motorized vehicles. it will cause pressure on the environment both energy use and environmental pollution. Governments' policy, beside build the supporting infrastructure also to restrictions on private transport.

Thus, in practically is to encourage the private vehicle users become public transport users. However, Makassar economic growth and rapid population. The rapid growth impact the increasing public revenues, which encourage the people to have private vehicle. The Improved of transport infrastructure quality and the bad of public transport services, also increasingly encourage people to have a private vehicle. Those things, will become an obstacle

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to "socialize" the use of public transport. Beside the economic capability of a person and the availability of public transportation, the environmental factors affect a person's tendency to choose the mode of transport. A study in the United States shows that the issue of security and safety of users of public transportation services, such as crime becomes a major constraint to consider in the management of public transport.

Based on the foregoing, the author conducted research entitled " The Preference Study on Privite Transport Mode of Transformation Based on User Behavior".

b. The porpuse and Objective of the research

The objectives of the research is to determine the mode transformation preference in privite transport based The characteristics of travel and travel behavior.

While the goal are:

1. Analyzing the behavior of the user journey mode of privite transport
2. Analyzing the transformation perception of privite transport modes
3. Build the interaction model of the user travel behavior private transport in relation transformation of modes perception.

c. Limitation of the research

The limitation of the problems as follows:

1. The material of this research is limited in private transport modes (two-wheel, four wheels).
2. The research was conducted in privite transport modes users of two-wheeled and four-wheel travel as office workers.
3. The Location of research conducted at one of the government agencies in Makassar region.
4. Data collection is done by simple random sampling
5. The data were analyzed by using Structural Equation Model (SEM)
6. The behavior variables of the user journey mode of privite transportation.
7. Individual response sensitivity in transformation of the use of private transport modes (two-wheel, four wheels).

LITERATURE REVIEW

a. Modes of Transport

Basically, transport is an essential means to move people or goods from one place to another place. The aim is to help someone or groups of people to reach the desired place or to send goods from place its place to a destination. Vuchic (1981) divides the transport modes according its type and use as follows:

1. Modes of private transport
2. Modes of transport for public/public transport)
3. Modes of transport for-hire

According to Quality Insurance (LPM) of ITB (1997), mode of transport can be grouped into two kinds based on the use and manner of operation, namely:

1. Personal Transport (private transport): transport owned and operated by and for the benefit of the owner's personal use of private infrastructure and public infrastructures.
2. Public transport: namely transport owned by the operator that could be used for public purposes with certain prerequisites.

The public transport system in general can be categorized into passenger transport systems and freight transport system. However, the discussion in this paper is more emphasing on passenger transport system. Some procedures modal choice modeling the movement with only two modes of transport, namely public and private transport. Models of realistic modal choice is disaggregate, behavioral and probabilistic. The disaggregated model type is akind of individual travel behavior. It is considered as a behavioral model due to two considerations: the first is concerning with the economic and psychological behavior of consumers in determining the decision. Both models are based on hypotheses related to the identification of variables in determining the decision to take. It is then called probabilistic model since the model indicates a probability of the outcome of the decision-making of potential traveler.

Factors that influence a person to choose a mode of transportation, according Tamin (2000) can be grouped into:

1. Characteristics of traveling performers, such as:
   - Socio-economic conditions and the level of income.
   - The availability or ownership of the vehicle.
   - Possession of driver's license.
   - Household structure (young couples, families with children, pensioners and others).
   - Other factors such as the need to use the car to the workplace and the needs of the kids to school.
2. Characteristics of movement
   - The purpose of the movement.

     For example the movement into the workplace in developed countries is usually easier to use public transport because of the timeliness and level of service is excellent and the cost is relatively cheaper

compared to private transport (car). However, the opposite occurred in developing countries, people still use private car to work, although it is more expensive because of timeliness, convenience and other reasons that can not be met by public transport.

- The timing of the movement.
  If we want to move in the middle of the night, we definitely need a personal vehicle because at the time public transport is hard to find or even not operating.
- Distance traveled.
  Distance traveled has influence in selecting a mode of travel, the farther away, the more we tend to choose public transport compared to private transport.

3. Characteristics of the transport system
   Personally, the level of service offered by each means of transport is the determining factor in choosing the means of transportation. Service levels are grouped into two categories:
   a) Quantitative factors, such as
      - The length of travel time includes the time in the vehicle, the time of waiting and walking time.
      - Transportation costs (fares, fuel costs, etc.).
      - The availability of space and parking rates.
   b) Qualitative factors, such as:
      - Comfort and convenience.
      - Reliability and regularity.
      - Security.

b. User Behavior

The most important factor of the decision to travel is the process of choosing. Performer's of a journey is always faced with a condition to determine the choice. This condition can occur at any time and every situation. For example in determining the location of residence, determine the mode of transport that will be used in the work, to shop, to school, and others. There are several levels of individual behavior in selecting mode (Manheim, 1979: 6 1), which is based on the lifestyle aspirations, the pattern of the desired activity, site selection activities/locational choices, and travel travel choices/decision.

At the highest level, the pattern of the desired activity of individuals depends on the aspirations of his lifestyle. Then to support his activities, the individual must be at a certain location at a certain time; this is referred to as site selection activities. Finally, to support activities in locations that have been selected, the travel decisions will be taken by considering where, when, and how the journey will be done.

Selection of the mode of transport is influenced by the stage of the traveling performers in deciding the choice of a transportation mode (variable behavior). The traveling steps of traveler in deciding the final selection of the mode of transport to be used can be divided into several stages (Rosmiati, 1990; Koppelman and Pas, 1980).

c. Structural Equation Model

Structural Equation Model (SEM) is a set of statistical techniques that allow testing of a series of relative relationship "complicated" simultaneously. The complex relationships can be defined as a set of relationships built between one or more dependent variables with one or several independent variables where each variable shaped both dependent and independent factors built by several indicators that are observed or measured directly (Waluyo, 2011).

Structural equation modeling generally uses the Linear Structural Relationship, estimation method that is commonly used is the Maximum Likelihood method (ML). ML parameter estimation method in LISREL requires how critical assumptions such as sample size of at least 10 times the amount of indikaor or more than 100 units of observation, the data spread following the normal distribution. LISREL provides several approaches that can be used as parameter estimation models, including the Bootstrap or use Weighted Least Square method. However, the both approaches still require relatively large data so as to model a complex with a relatively small sample size and the data do not spread normally takes a new approach. One new approach introduced by Herman Wold is Partial Least Square (PLS) and is often called soft modeling. PLS is possible used structural equation modeling with relatively small sample size where it is not build multivariate normal assumption. (Jaya IGNM, Sumertajaya IM, 2008)

RESEARCH METHODS

a. Research Design

The method used is descriptive analysis, which depicts an event then conducted an analysis of the problems arisen. The study began by collecting literature and secondary data relating to the research conducted, then determines the survey technique used. In this study the events that will be observed is the selection of private transport modes. The variables that will be studied are the attributes of travel behavior. Data collection techniques are the interviews or questionnaires to users of private vehicles on the work (employees) with random system. Form question survey form

covers two things planned. First, the question is focused to determine the existing condition of the characteristics of current users of private vehicles.In this case we want to know the socio economic condition of the user and trip information that is done using private transport modes. Secondly, the question is directed to know the preferences of respondents who have offered several hypotheses conditions such as travel costs, travel time, trip distance, access to parking and so on. By using the respondents' perception of data is then analyzed to determine the characteristics of behavioral variables traveling on private transport modes in Makassar.

b. Location and Research Period

The research is conducted in the city of Makassar, especially to users of private vehicles on the work in one of the government agencies that performed directly personally to the respondents, especially during the questionnaire. This research will be carried out on every week day.

c. Instrument of Data Colection

1. Primary Data

Primary data in this study was obtained from the results of the survey interview, in this case is a questionnaire. Respondents selected criteria are as follows:

- Respondents are residents or non-residents of Makassar Makassar but do work activities as employees in a government institutions in one of the government agencies in the city of Makassar.
- Respondents using private transport as a means of transport to get to work.
- Fill out questionnaires to complete.
- Respondents on the consistency and influence will be tested by appropriate logic Structural Equation Model (SEM).

2. Secondary Data

Secondary data will be obtained from:

- One of the government offices in the city of Makassar, in the form of the number of employees in the private transport modes users to travel to the office.
- Makassar in figures 2014 at the BPS office of Makassar.

d. Variables and Indicators of the Research

Variables and indicators in this study cann be seen in Table 1

Variables	Indicator
Accessibility	- Passenger load factor - Access to parking
Mobility	- Frequency of use - Traveling time
Socio Economic Preference	- Revenue - Age - Education - Job - Gender - Ownership of vehicles - Frequently use transport modes

e. The Technique of Data Analysis

Data analysis method that will be used in this study are generally divided into three, namely:

- Methods Descriptive Statistics

  This method is used to determine the travel behavior of private transport users in the form of tables and diagrams, so it is easy to understand. The data used in this method originated in questionnaire distributed to a sample transport users private cars and motorcycles in the office employees of government agencies. The method of sampling will use the Simple Random Sampling.
- Descriptive Qualitative Method

  This method is used to determine the transformation of the perception of personal transport users based on user travel behavior mode of personal transportation in the city of Makassar.
- Method of Structural Equation Model (SEM).

  This method is used to analyze the interaction of the user travel behavior modes of private transport in relation to the perception of the transformation of the mode of personal transportation.

EXPECTED RESULTS

The expected results of this study are:

a. Characteristic behavior of the user journey mode of personal transport
b. Perceptions of personal transformation transport modes
c. User interaction models of travel behavior modes of private transport and its relation to the perception of personal transformation transport modes.

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