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Excess Pore Pressure Generated by Pile Driving using Numerical Method and Soil Setup Prediction In Clay

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Abstract— Excess pore pressure arises as a result of pile driving in case of soft soil. Considerable excess pore pressure causes low pile capacity. As the soil surrounding the pile recovers from the installation disturbance, pile capacity increases as a function of time. This phenomenon is referred to as set-up. This research aims to determine the excess pore pressure behavior and predict the soil setup with modelling pile driving using cavity expansion theory. This research is carried out with soil data from Cirebon soil, which is dominated with clayey soil and a thin layer of sand. The spun pile with closed ended pipe was driven into the depth of 35 m. Dynamic test monitoring method was performed during pile driving and restrike was conducted with a time variation from 30 min to 7 days to determine the soil setup phenomena. This research performed using finite different analysis with pile modeled as axisymetric. A horizontal prescribed displacement of half day was applied to the pile to generate excess pore pressure and a vertical prescribed displacement of 25 mm was applied at the top of the pile to determine the capacity of the pile. The variation in time from end of driving to 365 days was given to determine the soil setup value. Result show that the value of excess pore pressure decreases as time shortens, whereas it increases as pile resistance rises. Skov and Denver method was applied to predict the soil setup value. The soil setup value is 0.13 when in situ test using PDA and axial load test is applied, whereas the value ranges from 0.14 to 0.19 when finite element method is used. Thus, this research contributes to the prediction of soil setup based on numerical result and verification by in situ test.

Index Terms— Pile Driving, excess pore pressure, soil setup, cavity expansion, PDA testend.

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1 Introduction

THE geotechnical design of driven pile capacity is mostly estimated by static formulas using post soil investigation data before pile driving operations are carried out. However, pile driving process may impact the soil condition on surrounding pile shaft and tip become highly disturbed. This impact to the different pile capacity values, before and after pile driving process. De Mello (1969) have been classified the effect of pile driving in clays into four major categories: (1) remolding of the soil surrounding the pile, (2) alteration of the stress state in soil in the pile, (3) dissipation of the excess pore pressures, (4) long term phemomena of strength-regain in soil. In a certain period of time, the soil condition surrounding the pile will recover and excess pore pressure will gradually dissipate. The phenomena of increasing pile capacity against time is known as soil setup. On this paper, case study taken from a project located in Cirebon, Central Java, Indonesia. This project uses a type of pile foundation with a 600mm diameter of spun pile. PILECO D46-32 of diesel hammer type is used for driving the pile. During driving, the pile behavior and hammer performance was controlled by PDA continues monitoring. To predict the value of soil setup, PDA is carried out at different time variations. This paper is tried to analysis the changing of excess pore pressure and to predict the soil setup using numerical analysis.

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2 Project Data

2.1 Soil Investigation Data

Geotechnical investigation are consist of drilling log and SPT (Standard Penetration Test). The location of the soil investigation test is shown in Figure 1.

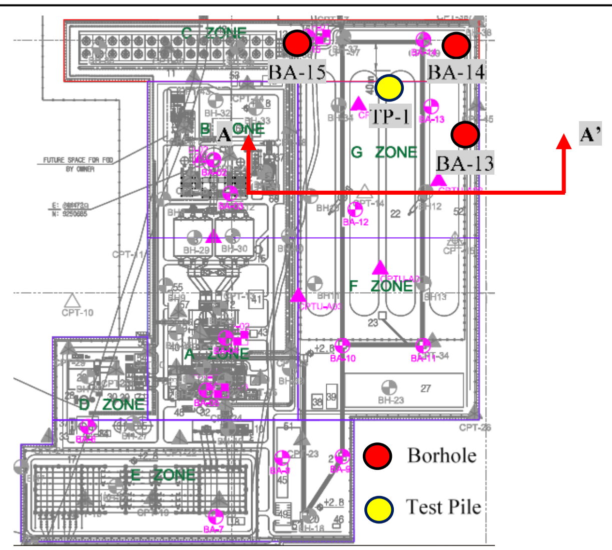

Fig. 1. Test pile and geotechnical investigation layout.

Soil stratification in this project is represented by Section A-A'.

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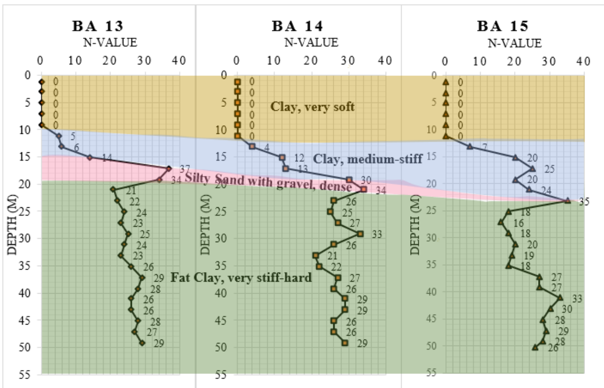

Fig. 2. Soil stratification section A-A'.

Section A-A' shows this area dominated with clay soil, which very soft clay layer found at 0 to 12m depth, followed with medium to stiff clay up to ±12-18m depth. A thin trace of dense sand found below. Finally, from 20 to 50m found a very stiff clay layer.

2.2 Pile Test Data

The foundation system is spun pile with 600 mm of diameter. The hammer specified to drive the pile is use PILECO D46-32 of diesel hammer type, the weight of this hammer is 46 kN with 2 m of ram stroke. The pile was penetrated up to 35 m depth. PDA continues monitoring was conduct during driving, thus the pile behavior and hammer performance could control well every depth of penetration. Figure 3 shows the result of total capacity (RMX) and the energy measurement of this hammer (EMX) at each depth of penetration.

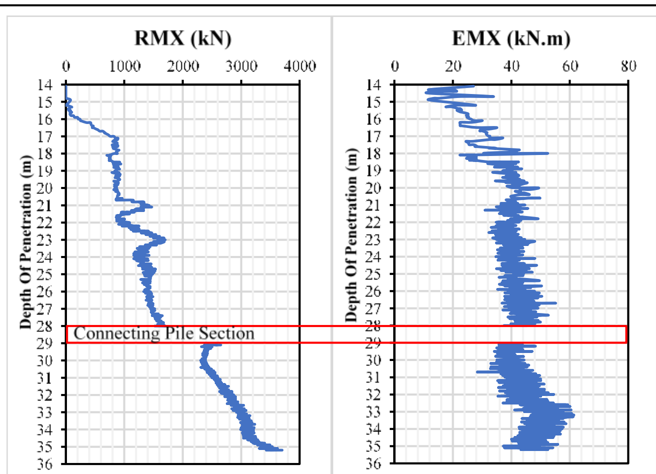

Fig. 3. PDA continues monitoring result.

The PDA monitoring was started from middle section of pile, since pile bottom was penetrated into soil without any resistance. Total blow and final measurement of this pile driving were 1905 blow and 23 mm/10 blow. From PDA continues monitoring can determine the consistency of the hammer, EMX value shows consistent in 40 to 50 kN-m, thus indicated that the hammer is in good performance. The pile was stopped at very stiff clay layer with capacity measurement from RMX value shows in range 3000 – 3500 kN. After end of driving (EOD), PDA restrike conduct in 3 different time: 30 minutes, 3 days, and 7 days. This test aim to figure out the soil setup value. PDA result shows time variations in Table 1.

TABLE 1
RESULT OF PILE CAPACITY USING PDA TEST

Time	RMX (kN)	*CAPWAP Analysis*		
		Friction capacity (kN)	End Bearing capacity (kN)	Ultimate capacity (kN)
EOD	3220	2310.5	554.1	2864.8
30min	3570	2390.7	488.9	2879.6
3days	4230	3923.2	391.4	4314.6
7days	4290	3997.6	398.7	4396.6

The result of PDA test shows there is a significant increase of pile capacity at time 30min to 3 days, while small increase of capacity shows at time 3 days to 7 days. CAPWAP analysis is a software which aim to match the wave curve between actual data and computed data, CAPWAP also give the information of pile capacity distribution in friction and end bearing.
Table 1 shows the friction capacity is bigger than end bearing capacity. Increasing of capacity shows at friction capacity, while end bearing capacity does not show an increase because of the energy of hammer may not mobilize to the whole pile capacity.

3 Excess Pore Pressure Behavior due to Pile Driving

In saturated clay, pile driving may induce the excess pore pressure as a transient flow. The ratio of Δ_u compared to effective vertical stress (σ_v') approaching to 1.5 – 2.0 times, and the Δ_u will slowly decreases towards zero when reaching the hydrostatic conditions at a radius 30 – 40 pile diameter. Several methods have been developed to predict the distribution of excess pore pressure surrounding the pile shaft. Lo and Stermac (1965) derived an expression to estimate the maximum pore pressure developed near pile surface, as in:

$$\frac{\Delta u_m}{\sigma_{vo'}} = \left[(1 - K_o) + \frac{2S_u}{\sigma_{vo'}}\right] A_f \qquad (1)$$

where Δu_m is the excess pore pressure maximum, $\sigma_{vo'}$ is the effective vertical stress, K_o is the is-situ coefficient of earth pressure at rest, A_f is the pore pressure coefficient at failure, and S_u is the undrained shear strength.

Airhart *et al.* (1969) states that near pile tip, even greater excess pore pressure may be developed 3 to 4 times the effective vertical stress. Beyond r/a of about 4 in normal clays, and about 8 in sensitive clays, a rapid decrease in excess pore pressure with distance occurs. Whereas, if the r/a greater than 30, the excess pore pressure may virtually negligible.

4 Study on Soil Setup by Dynamic Test

Measurement the bearing capacity due to setup, requires a minimum of two capacity measurement at different time. The first measurement is carried out at the end of driving, then second

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measurement is carried out in relatively longer period (Komurka, 2004). Tan et al (2004) proposed that second measurement may carried out as long as 24 hours for the sand soil, it is considered caused by the rapid dissipation of excess pore pressure in sand.

There are several empirical equations can predict the changing of pile capacity against time period. Skov and Denver (1988), one of most common method gives an empirical equation, namely:

$$\frac{Q_{st}}{Q_{so}} = A.\log\left(\frac{t}{t_o}\right) + 1 = \left(\frac{m_s}{Q_{so}}\right)\log\left(\frac{t}{t_o}\right) + 1 \qquad (2)$$

where A is the dimensionless setup factor, Q_{st} is the shaft friction pile capacity at time t, Q_{so} is the shaft pile capacity at initial time t_o, t is the time elapsed since EOD, t_o is the reference time at start of log linear capacity, and m_s is the semilog linear slope of Q_s versus log t.

Bullock et al. (2005), Axelsson (1998a), and Chow et al. (1998) concluded that soil setup occurs as a result shaft friction increase not end bearing. Penetration of the pile pushes soil away from the pile, destructuring and shearing it to a greater extent adjacent to the side of the pile than at pile tip. Thus, reducing reducing the side resistance during installation (and increased aging effects).Capwap analysis confirms that almost no increases in end bearing capacity with different times. Increases of friction against time more dominant in increasing of total pile capacity. Figure 5 shows the increasing of shaft friction capacity vs time in logarithmic scale from PDA test at this project.

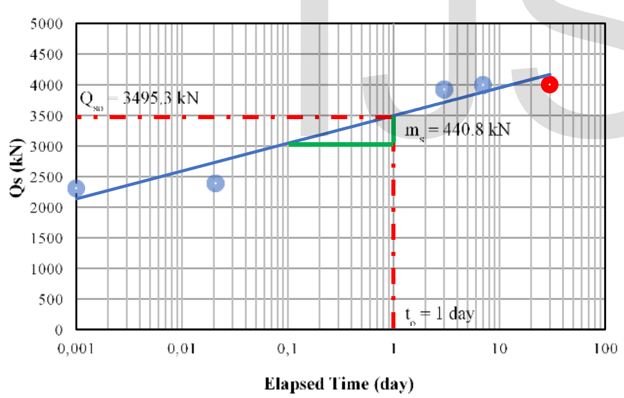

Fig. 4. Determination of m_s and Q_{SO}.

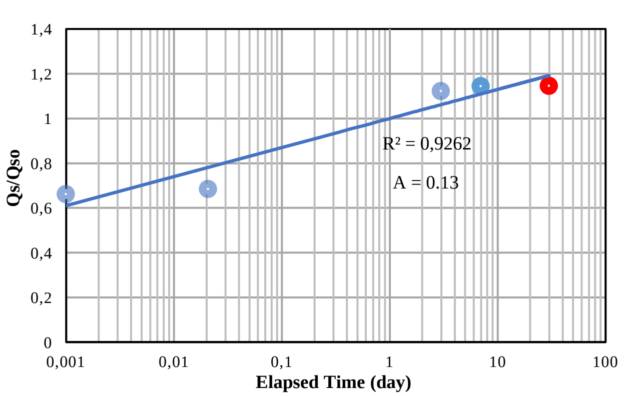

Fig. 5. Determination of soil setup value (A).

Bullock et al. (2005) recommend to use t_o = 1 day as this removes the difficulty of finding the actual start of the semilog-linear setup and also use A to describe the friction capacity only, not the whole capacity. As an approximation, the EOD capacity plot at 1 min elapsed time. Based on Figure 5 and using Equation 2, the pile capacity at this project will increase as 1.13 times capacity at EOD time.

5 Cylindrical Cavity Expansion

The cavity expansion theory can be divided into elastic and plastic zone. Cylindrical cavity expansion give the basic assumptions, as follows:

- Supposing axial symmetric stress condition in normally consolidated soil.
- Assumption that the soil is incompressible solid particles, that means deformation of the soil is created by the gap between particles from the water/or water is driven out.
- The soil has low cohesion.
- The cylinder is considered infinite in scope.
- The principle stresses around the embedded cylinder is : $\sigma_r = \sigma_1$ and $\sigma_t = \sigma_3$.

Figure 6 illustrated the cavity expansion process by using Mohr-Coulomb failure criterion and radial stress vs modulus of deformation.

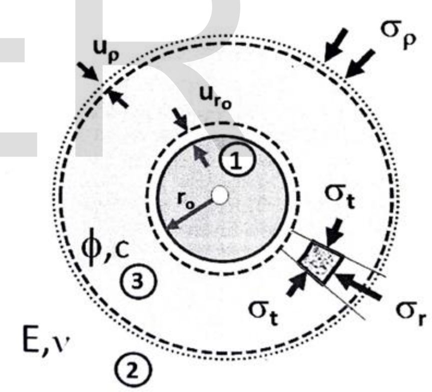

Fig. 6. Illustration of cylindrical cavity expansion (Mecsi, 2013).

Mecsi give the understanding of cavity expansion that there is three zone when the soil start to expand.

- Zone 1 called as the soil start to expand, with the equation $(r_u^2 - r_o^2).\pi$ (3)
- Zone 2 called as the soil displace into elastic zone $[\rho^2 - (\rho - u_p)^2].\pi$ (4)
- Zone 3 called as the volume change of soil into plastic zone $\Delta.(\rho^2 - r_u^2).\pi$ (5)

Thus, the expanded body of the soil can be calculated as total of zone 2 plus zone 3.

6 Analysis Model and Result

The idea of the analysis is tries to simulate the changing of excess pore pressure behavior and soil setup prediction using Plaxis 2D software. Plaxis software use is 2017 version, this

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version is kindly helpful for this research. Pile was modelled as an axisymmetry with a linear-elastic and non-porous material, modulus (E) given as 35400000 kN/m^2. The pile driving model is carried out by theoretical cavity expansion approach, where the pile modelled with an initial diameter of 0.05 m, then the pile as if developed 0.25 m. To determine the behavior of changing in excess pore pressure, consolidation analysis was done with time different, namely EOD, 30 min, 1 day, 3 days, 7 days, 14 days, 30 days, 60 days, and 365 days. Figure 7 shows the model of analysis. Estimating pile axial bearing capacity also done by giving a vertical displacement as value 25mm, this value is considered as a reference movement limit in mobilizing the whole capacity of the pile.
Pile properties: pile diameter is 0.6 m, pile length is 35 m, and ground water level is 2 m. The analysis model shown in Figure 7.

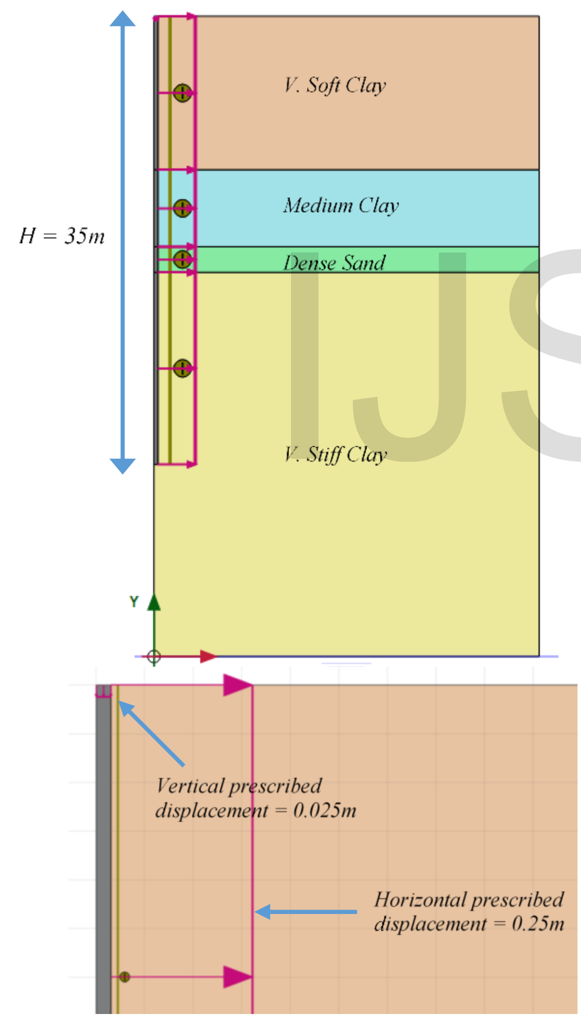

Fig. 5. Determination of soil setup value (A).

Soft clay layer is modelled as soft soil model (SS), while medium-stiff layer and dense sand are modeled with two different models, as Mohr Coulomb (MC) and hardening soil model (HS). The main soil parameters of this analysis are shown in Table 2 and Table 3.

TABLE 2
SOIL PARAMETERS WITH SS-MC MODEL

	Soft Clay	Medium Clay	Dense Sand	Stiff Clay
Model	Soft Soil	Mohr Coulomb		
Type	Undrained A	Undrained A	Drained	Undrained A
γ unsat	15.5	17.6	20	20
γ sat	14.5	16.6	19	19
λ^*	0.098	-	-	-
κ^*	9.84×10^{-3}	-	-	-
c	0	0	0	0
ø'	20	24	36	28
k	2×10^{-5}	7.4×10^{-5}	4.32	4.4×10^{-4}
$R_{inter\text{-}face}$	0.75	0.75	1.0	0.75

TABLE 3
SOIL PARAMETERS WITH SS-HS MODEL

	Soft Clay	Medium Clay	Dense Sand	Stiff Clay
Model	Soft Soil	Hardening Soil		
Type	Undrained A	Undrained A	Drained	Undrained A
γ unsat	15.5	17.6	20	20
γ sat	14.5	16.6	19	19
λ^*	0.098	-	-	-
κ^*	9.84×10^{-3}	-	-	-
E_{50}^{ref}	-	5100	26300	6400
E_{EOD}^{ref}	-	5100	26300	6400
E_{ur}^{ref}	-	10200	52600	12800
c	0	0	0	0
ø'	20	24	36	28
k	2×10^{-5}	7.4×10^{-5}	4.32	4.4×10^{-4}
$R_{inter\text{-}face}$	0.75	0.75	1.0	0.75

6.1 Changing of Excess Pore Pressure Behavior

The result of the analysis using both models shows similarities of changing excess pore pressure behavior. The behavior configurate the curve as known as isochrone curve at the position of clay layer, while there is no excess pore pressure developed at sand layer since sand is a drain material. The excess pore pressure become gradually dissipated with time increase. Higher significant dissipated showed at time between 30 minutes to 1 day. Figure 8 shows the result of analysis of excess pore pressure with different parameter model. The maximum pore pressure value at each soil layer with time variation will shown in Table 4 and Table 5.

The finite different analysis was continued to determine the distribution of excess pore pressure with distance from pile driving installation at time during installation. This analysis result was focus on soft clay layer, the value is taken from the maximum of Δ_u. The analysis was carried out at distance varying from the point of pile driving (0.1 m; 0.5 m; 1 m; and 1.5 m). Based on SS-MC model, the value of Δ_u increased by 1.5 times σ_v', whereas with SS-HS model resulted the Δ_u increased by 1.7 times σ_v'. Then, the value of Δ_u/σ_v' at each model plotted against r/a to be compared with a Poulos & Davis, 1980

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research publication.

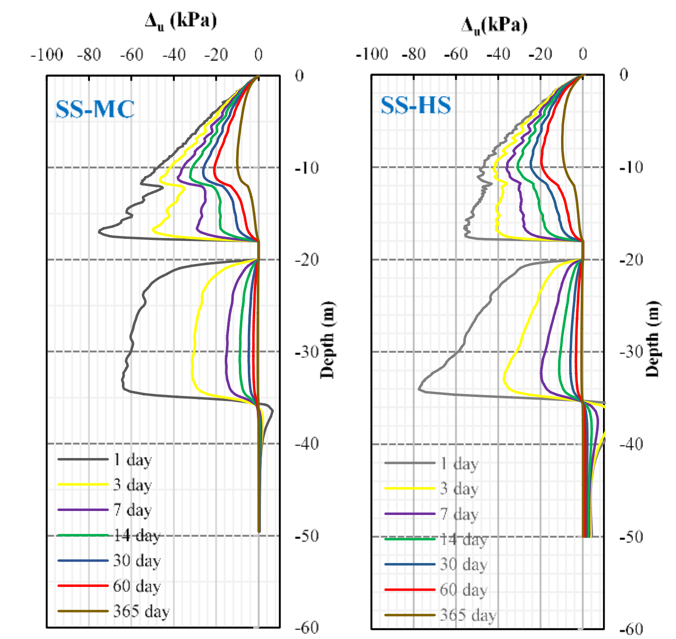

Fig. 8. Result of excess pore pressure behavior with time function.

The maximum pore pressure value at each soil layer with time variation will shown in Table 4 and Table 5.

TABLE 4
CHANGING OF Δ_U WITH SS-MC MODEL

Soil	Δu (kPa)								
	EOD	30 min	1 days	3 days	7 days	14 days	30 days	60 days	365 days
Soft Clay	107.8	76.7	55.4	46.6	38.0	32.3	26.0	20.9	10.1
Medium Clay	146.5	113.7	75.2	49.8	29.1	21.3	16.5	12.8	4.9
Dense Sand	82.0	7.8	0.2	0.1	0.0	0.0	0.0	0.0	0.0
Stiff Clay	318.5	284.8	64.4	31.4	15.3	8.9	4.7	2.5	0.5

TABLE 5
CHANGING OF Δ_U WITH SS-HS MODEL

Soil	Δu (kPa)								
	EOD	30 min	1 days	3 days	7 days	14 days	30 days	60 days	365 days
Soft Clay	109.6	58.2	49.8	42.3	36.2	30.8	24.7	19.7	9.7
Medium Clay	165.8	78.5	56.1	41.1	30.4	23.4	16.7	11.8	4.2
Dense Sand	157.3	15.5	0.3	0.1	0.1	0.0	0.0	0.0	0.0
Stiff Clay	300.3	292.6	77.6	37.4	19.8	11.2	5.9	3.4	0.7

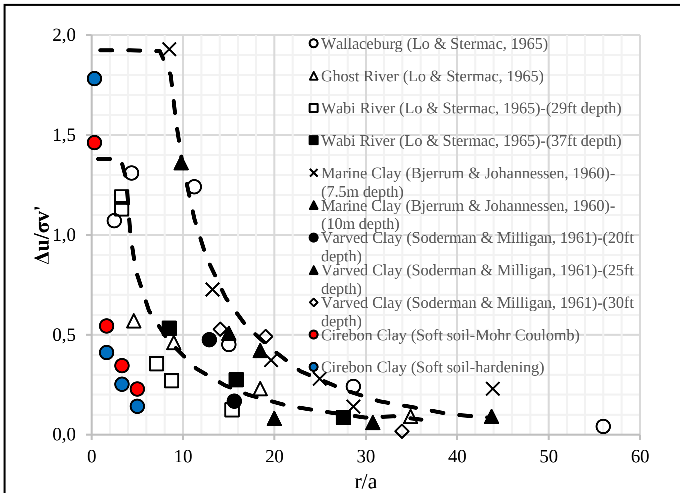

Fig. 9. Comparison between finite different analysis and pore pressure measurement (Poulos & Davis, 1980).

Analysis result shows in Figure 9 is little bit underestimate from the expected line. However, at very near position with the pile installation, the increases of excess pore pressure from vertical effective stress still within in range 1.5-2.0 from Lo Stermac expectations.

6.2 Pile Capacity Estimation

Estimating pile capacity was done by giving a 25 mm of vertical prescribed displacement. From Plaxis software can described the load transfer along the pile.

The result of the load transfer shows increases with time different, thus the relation between excess pore pressure behavior with load transfer development is proportional. When the excess pore pressure starts to dissipate, then the shear strength of the soil will start to increase. From Figure 10 shows the significant increase of pile capacity indicated at time 30 minutes to 1 day.

The capacity was not significant increase in very stiff clay layer from time 1 day to 365 days, while the capacity in very soft clay still show increase from time 1 day to 365 days. The increasing of capacity can be seen in the gap between capacity in each time. Loading test with instrumentation Vibrating Wire Strain Gauge (VWSG) was done on this project. The objective of this test is to determine the load transfer using strain change measurement. Loading test was conduct after 30 days from pile installation, the load was performed with 200% x 2000kN.

By the finite different analysis shows the curve of load transfer is look quite similar tren with the result of VWSG. Also, from the load transfer analysis can give the information of friction distribution, it can be known from the slope of the load transfer line. The high friction can be seen at sand and very stiff clay layer, where the slope of the line was quite tilted than other soil layer, while the low friction showed at soft soil layer.

Related to the excess pore pressure behavior, the dissipation of excess pore pressure at very soft clay layer is longer than medium and very stiff clay. Thus, the increasing of shear strength at very soft clay will also longer than medium and very stiff clay.

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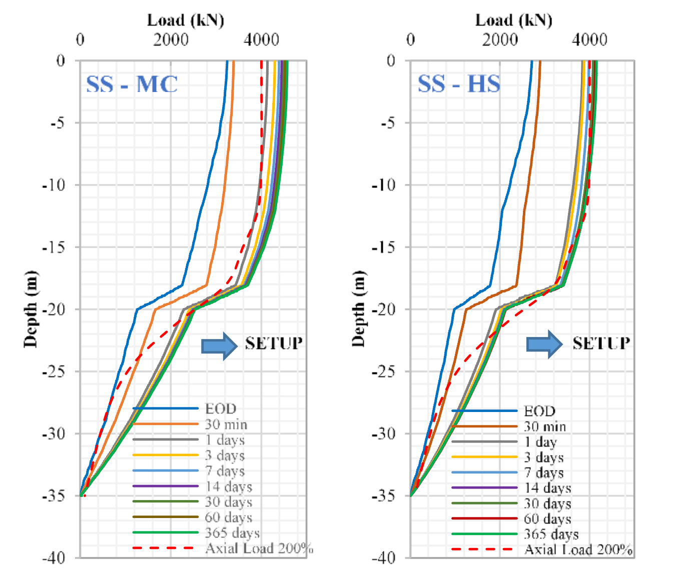

Fig. 10. Increasing pile capacity from Plaxis compared with instrumented load test measurement.

The friction capacity calculation at each soil layer from load transfer graph can be seen at Table 6 and Table 7.

TABLE 6
FRICTION CAPACITY WITH SS-MC MODEL

Soil	Friction Capacity (kN)						
	1 day	3 days	7 days	14 days	30 days	60 days	365 days
Soft Clay	163.4	170.4	174.3	175.7	181.6	188.6	198.0
Medium Clay	329.0	358.8	385.7	404.9	420.7	429.8	439.0
Dense Sand	848.6	852.7	856.5	859.0	861.4	863.0	866.9
Stiff Clay	1698.1	1792.5	1836.2	1857.4	1870.9	1877.8	1881.8

TABLE 7
FRICTION CAPACITY WITH SS-HS MODEL

Soil	Friction Capacity (kN)						
	1 day	3 days	7 days	14 days	30 days	60 days	365 days
Soft Clay	159.8	166.9	170.4	175.9	182.3	190.3	204.5
Medium Clay	225.0	253.9	277.9	295.9	307.9	316.5	323.4
Dense Sand	973.0	889.2	915.1	971.1	936.9	958.6	943.3
Stiff Clay	1365.1	1453.0	1490.1	1502.7	1516.6	1518.8	1512.4

6.3 Numerical Result vs Field Measurement

Following the numerical result, this research is tries to predict the soil setup value and compare with soil setup value resulted from field measurement (PDA test). Figure 11 shows the graph between increasing ultimate capacity (Q) vs logarithmic time, the trendline of numerical result with two model similar with field test result. The resulted curves are resemble with consolidation curve (curve S) in logarithmic time scale.

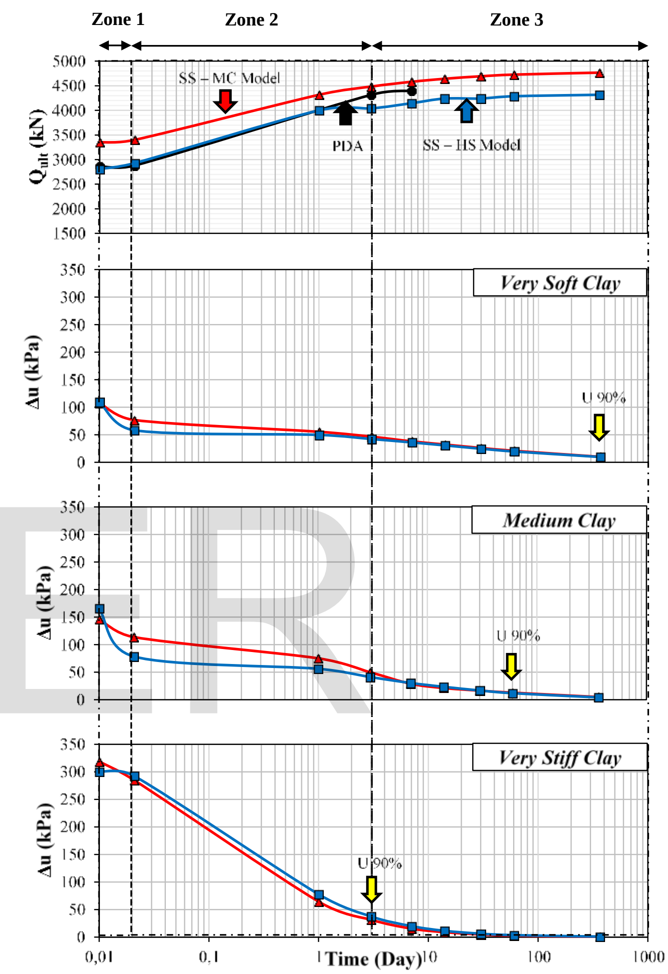

Fig. 11. Soil setup compare with excess pore pressure behavior.

By the numerical analysis result using SS-HS model, the increases of friction capacity at time EOD to 1 day is approach with PDA test result at the same time. While, analysis with SS-MC model resulted similar trendline with PDA test at time 3days to 7days. It can be concluded that the PDA result still within range from the result of numerical analysis.

Comparison between soil setup and excess pore pressure behavior which shown in Figure 11 can be divided into 3 zones, where zone 1: time EOD to 30 min; zone 2: time 30 min to 7 days; and zone 3: time 7 days to 365 days. Each zone is described the value of soil setup in this case. Using Skov & Denver method, A value at zone 1 obtained in range 0.01 – 0.09 with t_o = 0.015 days. Zone 2 obtained higher A value than zone 1, A value in range 0.14 – 0.19 with t_o = 0.3 days. And then, zone 3 resulted the A in range 0.03 – 0.05 with t_o = 5 days.

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Figure 11 also shows the changing of excess pore pressure at each soil type. From three types of clay layer shows the 90% degree of consolidation at stiff clay is approach in 3 days, medium clay approach in 60 days, and very soft layer approach in 365 days. Excess pore pressure at very stiff clay layer is dissipate faster than two other soil types. Compare to the result on Figure 10, the changing of excess pore pressure in each layer is related to soil setup behavior in each soil layer.

7 Conclusion

The case study was taken from Cirebon, Indonesia regarding the effect of pile driving on an excess pore pressure (Δ_u) behavior and soil setup (A) phenomena. Soil condition in this project is dominated with clay soil with a thin trace of dense sand.

The numerical analysis using Plaxis was performed to analyze the Δ_u and predict the soil setup. The idea of this analysis is tries to model the pile driving using cavity expansion theory. Where the initial pile was 0.05 m then 0.25 m of prescribed displacement perform to develop the Δ_u. Analysis continue to determine the pile capacity, after the pile develop, with an 25mm of vertical displacement was applicate to the top of the pile.

The PDA test was carried out with four (4) time different. Comparison from PDA re-strike and PDA at EOD time can predict the soil setup value (A). Using Skov & Denver method, resulted the A value is 0.13. Thus, the pile capacity at time t will increase 1.13 from pile capacity after installation.

The development of Δ_u is determine with consolidation analysis with time variation. Shows the result is the excess pore pressure become dissipate with increasing of time. This research tries to know the increase of Δ_u compared with the vertical effective stress (σ_v'), resulting the Δ_u/σ_v' = 1.5 and 1.7 with two different model.

The load transfer analysis by numerical method shows similar trendline compared with instrumented pile load test. The high friction capacity occurs at dense sand layer and very stiff clay layer.

Summary of this analysis resulted the curve of friction capacity (Q_s) against logarithmic time (t) in day. The curve shows quite similar with curve S (consolidation curve), as indicated low increase of Q_s at time EOD to 30 minutes; high increase of Q_s at time 30 minutes to 7 days; and flat increase of Q_s at time 7 days to 365 days.

Finally, this research proposes to modify the curve with dividing into three (3) zone, and also try to calculate the A value using Skov & Denver method. Zone 1 give the A value as range 0.01-0.09; Zone 2 give the A value as 0.14-0.19; and Zone 3 give the A value as 0.03-0.05.

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