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ADDITIONAL BEARING CAPACITY OF PILES DUE TO TIME DELAY OF INJECTION

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ABSTRACT: The use of long piles to support construction loads has been recognized for thousands of years. In the last century, it has been introduced the use of pressure for embedding pile foundations that is named injected piles. In field implementations, if the design depth of the piles is long enough then additional piles are required to be connected. It needs time delay to connect between the earlier embedded pile to the additional one. The time delay can be also caused by other things such as equipment problems, unwilling bad weather etc. Based on the experience of pile injection records, due to the time delay, there is a temporary increase in pile bearing capacity, so it takes more effort to continue pile penetrations until the specified depth. In this study, the effects of erection time delays on piles are investigated. This study is conducted using experimental injected-pile models at the laboratory. The initial depth of earlier piles prior to further injection is set the relatively same, while the time delays are varied. The injection loads for penetration are recorded as well as the depth of penetration. The results of the experiments in terms of variation of time delays and additional penetration loads show that there are 'jump loads' at the initial last depth due to time delay until a certain period. It can be concluded that the time delay on the pile can increase the bearing capacity and must be limited to a certain time in case of pile connection work on site. The variation additional loads due to the time delays in a soil mass may be affected by soil parameters such as density and particle size of the soil.

Keywords: Bearing capacity, Injection pile, Time delay, Pile foundation

1. INTRODUCTION

Pile foundations have been used to transfer the building load to the underlying soil layer since the earlier the first century [1]. Based on their load transfer mechanism, piles may be classified by end-bearing and friction piles or combination one. End-bearing piles develop most of their bearing capacity mainly at the tip of the pile. Friction piles transfer the load to the surrounding soil along the pile length by friction between the pile surface and the soil. Piles also can be differed by the method of construction that are driven piles (or displacement) and bore piles (or replacement). The driven pile may be placed into the ground by displaced surrounding soil by a hammer, injected, vibrated or screwed. Bored piles are placed into the ground by replacing soil in boreholes with pile materials. In this paper, the piles may classify as injected end-bearing piles.

The pile tip resistance can be calculated based on a traditional formula using the cohesion and internal friction angle parameters of the soil. If the soil at the end of the pile consists of sands only, then the cohesion parameter can be avoided in the calculation, so that the end bearing capacity formula becomes:

$$Q_u = q_u A_p \quad (1)$$

With
$q_u = q' N_q + {}^1/_2\, \gamma B N_\gamma$

Where:
Q_u = ultimate axial load
q' = effective overburden
B = pile tip diameter
γ = unit volume of underneath soil
N_q and N_γ = bearing capacity factors

The effective overburden stress is the difference between total overburden stress with the pore water pressure. Under static conditions, the effective stress may be the result of reduction by hydrostatic pressure. The pore water pressure a dynamic condition may exceed the hydrostatic pressure. The excess pore water pressure in dynamic condition may generate during the pile placement into the ground using dynamic ways such as jacking and driving. The effective overburden then can be written as:

$$q' = to - u \quad (2)$$

Where to = total overburden stress
u = excess pore water pressure

Besides static bearing capacity of pile foundation, there are many problems must be considered during the design. Some problems that might be taken into account in the used of piles are potential consequences of liquefaction, differential settlement, lateral loads and movement and possibility on soil bearing capacity reduction [2]. Pile foundations may not only subjected to static loads but also dynamic loads simultaneously. The study on the dynamic response of a single concrete pile in soft clay subjected to static and dynamic simultaneously has been presented [3]. The connection between pile to the cap also may be investigated to understand its behavior respected to the applied loads. The investigation of the moment capacity and load-displacement relationship of laterally loaded the pile to cap connection with different connection details has been reported [4].

It is has been expected that the piles capacity will be increased with the time as many researchers reported. The benefit of this bearing capacity growth usually performed within a few days after installation. The study on steel piles tension tests in dense sand reported more marked increases time-skin friction relationship than expected [5]. This report, it was not certainly stated the additional condition such as scale, or sand type, which affected the longer-term skin capacity growth.

Recently, the study on the ultimate load capacity of pipe piles affected by grain size distribution of sand under dry and fully saturated state has been conducted through experimental models [6]. The sand was sieved to obtain poorly graded sand with fine, medium and coarse grain size. The experimental results showed that the bearing capacity of small open-ended piles in medium grain sand was higher than the other size sands.

An experimental study with back analysis of centrifuge tool models was conducted to investigate the effect of soil permeability due to dynamic loading. The study focus on the responses of end-bearing single piles and pile groups subjected to lateral spreading in a box [7]. The results of this tests show the evidence of the importance of soil permeability on piles response against lateral spreading especially when liquefaction reached the surface. Further, it was found that the pile's response in terms of bending was greater in silty sands compares to clean sands. This study may be enhanced by evaluating practical cases due to lateral spreading pressure.

On-site pile work in order to achieve sufficient bearing capacity at the certain depth, often piles connection work is needed. The pile to pile connection thus become a special consideration in practical works. The required pile to pile connection in some cases are caused by the manufacture limitations at the factory and/or transportation problem of piles from the factory to the site. Extremely long piles will be difficult to be distributed from factory to the site. Very long piles also create difficulties on their mobilization on site.

The pile connection process may need the time and the driving process must be stopped temporarily. The most popular pile connection in Indonesia is mainly done by welding to joint the pile ends. Each pile then has the end that made of steel which may function as a pad/base for pre-stressed wire of pile reinforcement. The steel pads are then to connect the embedded and the subsequent piles on the site.

The earlier study on time-dependent pile capacity, especially during installation, has suggested three phases to figure time-capacity relationship [8]. The first phase of the logarithmically nonlinear rate of excess pore pressure dissipation. The second logarithmically linear rate of excess pore pressure dissipation. And the last is an aging phase, where the pile capacity is independent of effective stress. The first phase is related to the pile installation work, where the capacity ratio is less than one compared to the pile capacity at the time that just at the end of driving. This short-term time-dependent capacity becomes interested in this study which is related to the pile installation efforts.

The time-dependent bearing capacity at time t for long-term phase was suggested to be estimated using the following formula [9] [10]:

$$Q_{u,t} = Q_{to} + A\, Q_{to} \log (t/to) \tag{3}$$

Where: $Q_{u,t}$= ultimate axial load at time t
Q_{to} = initial ultimate axial load at time to
A = set-up factor, ranges from 0.2 to 0.8

The set-up factor A represents the effect of soil type, pile material, type, size, and capacity, but is independent of depth as well as water pressure dissipation. The initial reference time to equal to 1 day is reasonably suggested

2. ON FIELD DATA

The soil sediment in the field consists of a layer of sand and silty sand until 20m of depth and gravelly sand for deeper. Groundwater in the field is at depth 2m from the ground surface. This indicates that the soil beneath the groundwater table is under saturated condition. The mechanical properties of the soil layers are shown in Table 1. Based on standard penetration test data on site (Fig.1), the design depth of the 60cm of diameter piles were about 26 to 30 m.

The spun-piles had been used to support the Public Work Building in Padang. The piles came from the factory have the length of 10m and 12m. Consequently, they were needed pile connection work to complete penetration every point on site.

During the penetration of piles on site, the applied loads on every single pile were monitored and recorded (Fig.2).

Table 1 Properties of the soils

Depth (m)	Type (Code)	Mechanical Properties		
		ϕ	c (t/m^2)	γ (t/m^3)
4.0-4.4	Sand (S)	33.6		1.5
14.-14.5	Silty sand (SM)	29	4.5	1.4
24-24.5	Gravelly sand (SG)	35.4		1.9

It can be seen that the first joining work at the depth of 11m was finished within 16 minutes. The increment of penetration load after 16 minutes increased until 339t then came down to 226t for the next record. It means for 16 minutes delay time, the load of penetration increased until 113t. The same thing happened after pile connection work at the depth of 21m, the penetration load increase by 85t for 14 minutes delay.

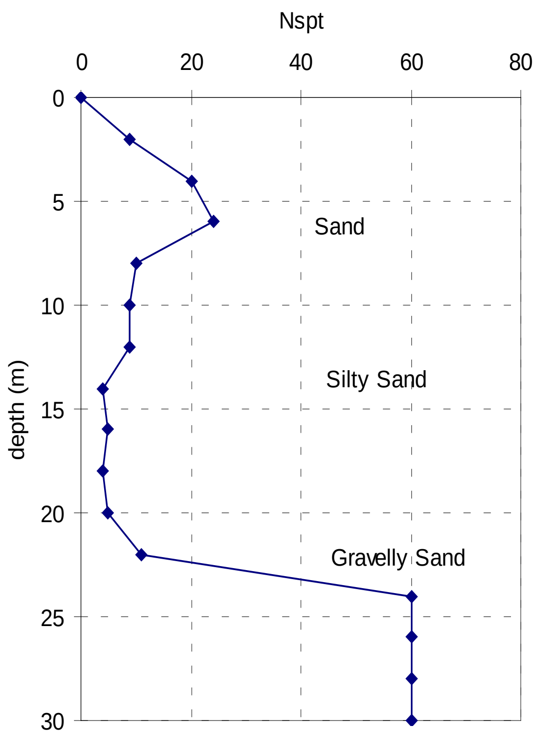

Fig.1 Nspt values on the site

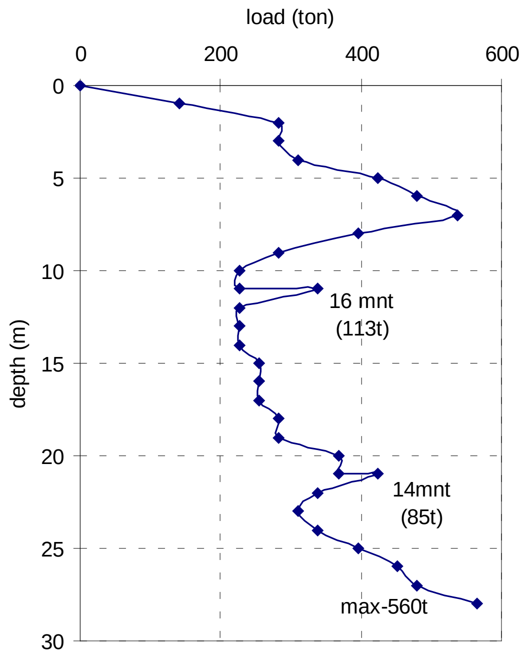

Fig.2 Penetration load record on site

There was also an unwilling case due to damage penetration tools that cause penetration process was stopped. The repair work by replacement of broken spare part took a few days. When the penetration was continued, the loads have shown the value that exceeded the maximum capacity of the pile injection tool. The depth of the pile at that point has not reached the specified depth. In order to continue penetration, the pile the was pulled first, it pushed again into the ground to continue penetrating to the specified depth. This case indicates that the dissipated pore water pressure within a few days may result in an increase the effective stress in soil mass so consequently increases the pile bearing capacity.

For longer time-dependent increment in load capacity has been reported based on field load tests on driven tapered and straight-sided concrete piles [11]. The load tests were conducted after about a month and 9 months after the installations. Those 12 m piles were driven close together in the same saturated cohesive soil. Event the study was proposed for comparison between tapered and uniform piles, the results also figured the time effect to the load capacity. The test results showed the load capacity of the piles increased significantly after months resting time. It was 30 tons in the first month and increased to 90 and 160 tons for tapered and uniform pile respectively, after 9 months [Fig. 3]. In the figure also plotted the trend lines to connections between the loads in the same piles for a different time. The results indicated the driving induced pore pressure had been dissipated to generate effective stress in the soil mass along the pile. The question arises whether there is a significant time interval that importantly effects the bearing capacity associated with the dissipated pore water pressure. The study then suggested for further field and laboratory observations.

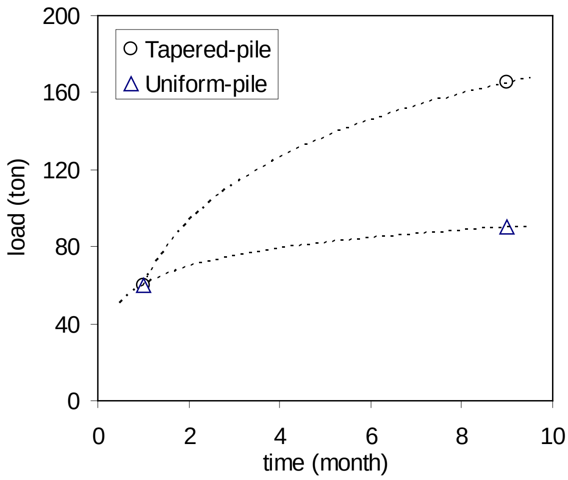

Fig.3 Pile load capacity versus time from [11]

3. LABORATORY TESTS

In order to understand the increment of load due to the excess pore water pressure during the pile penetration, a series of a laboratory test is conducted. This test is carried out using a small scale pile model. A pile model has a dimension of length 30 cm and the cone tip with a diameter of 1 cm (Fig. 4).

In this study, the pile model is tested into two different soil media. First, the soil is taken from Siteba and the second is from Lubuk Alung. The soils are made up of fine sands that sieved pass through sieve no 40. Both soils are made in a saturated condition. With the purpose to avoid result differences due to soil variations, the tests were conducted using the same soil for every desired time delay. Prior to the load tests on the pile model, the soils were tested to obtain the engineering parameters. The parameters are shown in Table 2.

The pile model is then placed into two soil media into the depth of 25cm for the first one and will be injected for a certain time to obtain the load capacity. The load is applied to push the pile model into the soils to have tip resistant. The sleeve of the pile model has a casing that keeps staying still during the tests. Consequently, the skin friction on the pile model can be ignored in these tests. So, the applied load to injecting pile is considered equal to the tip resistant of the pile only.

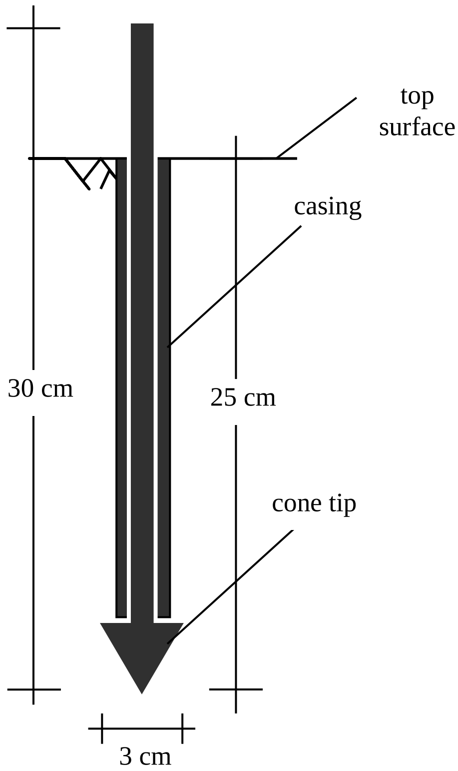

Fig.4 Dimension of the pile model

Table 2 The soils' properties

Properties of the soils			units	Siteba	Lubuk Alung
1	Dry Density	γ_{max}	(t/m^3)	1.52	1.72
		γ_{min}	(t/m^3)	1.37	1.60
2	Relative density at test	Dr	%	47	33.5
3	Int. Friction angle	ϕ	degree	28	37
4	Mean grain size	D_{50}	mm	0.22	0.15
5	Coef. Uniformity	Cu	-	1.7	3.2

Table 3 Test results for Siteba soil

Depth (cm)	25.10	25.26	25.44	25.54	25.62
Time interval (minute)	1	10	20	30	60
Increment (kg)	1.02	4.10	6.15	7.17	7.17

The procedure of pile tests is started injecting the pile into the soil to the depth of 25cm (Fig. 4). During the initial injection, the given loads for every unit length are recorded. The next procedures are purposed to record the additional load due to the time interval between the first injection to the next one. The time interval is set for the first minutes until the first hour. So that will be obtained the influence of the time delay within the first hour to increase the bearing capacity. The limited delay time interval is only for 60 minutes in this study with the reason that the delay time experiences during the pile injection in the field due to the pile connection and equipment setting time usually less than an hour. These test procedures are then applied to both test media of Siteba and Lubuk Alung soils. The first results of the test procedure for Siteba soil are shown in Table 3 and plotted in Fig.5 as well.

The second test with the same procedure as before was done with the sandy soil from Lubuk Alung. The series test results are shown in Table 4. The same values for this test are also plotted in Fig.6.

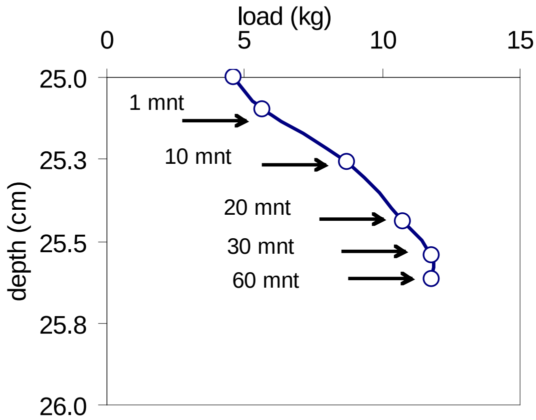

Fig.5 Maximum load .vs. time for Siteba sand

Table 4 Test results for Lubuk Alung soil

Depth (cm)	25.10	25.26	25.44	25.54	25.62
Time interval (minute)	1	10	20	30	60
Increment (kg)	3.33	4.66	9.22	12.30	14.35

From the time-load graph, it can be seen that the increment load increased with the time delay and reached the maximum after reach time delay about 30 minutes (Fig.7). This graph indicated that the excess pore pressure due to the pile injection was fully dissipated after 30 minutes in this test. In the field, if this delay time may be happened due to the joining work of piles or other injection disturbance. In the real work on site, the delay time may cause the more efforts to continue penetrating the pile are needed.

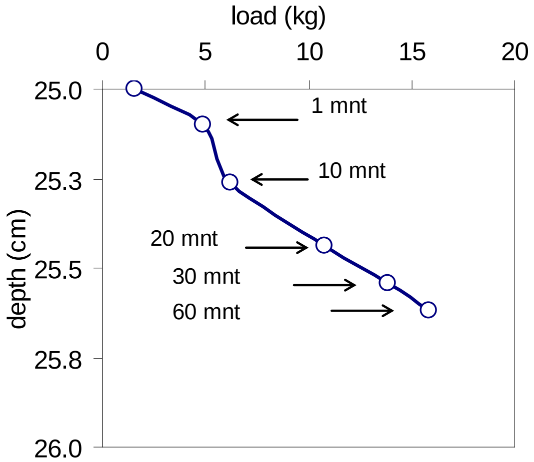

Fig.6 Maximum load .vs. time for Lubuk ALung

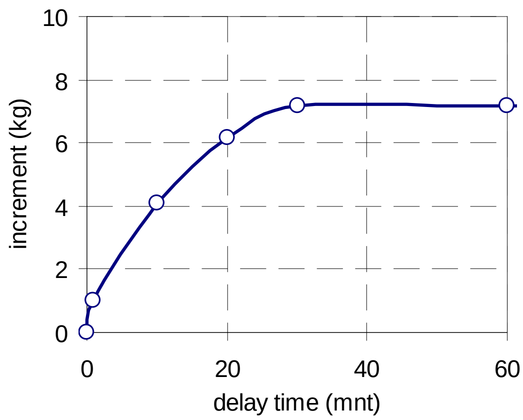

Fig.7 Delay time - load increment for Siteba sand

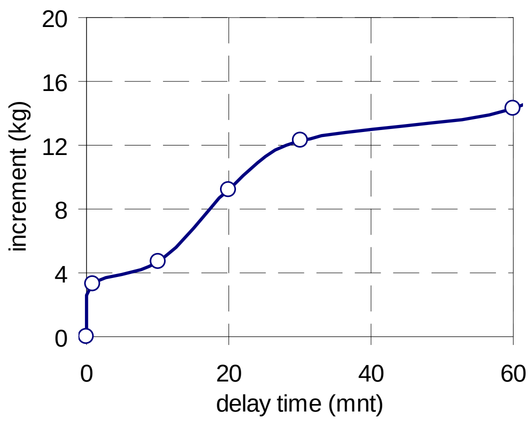

Fig.8 Delay time vs load increment for Lubuk Alung sand

From the time-load graph, it can be seen that the increment load increased with the time delay. In this test, the load increment increases sharply during half an hour and then gradually reduce with the time delay (Fig.8). This graph indicated that the excess pore pressure due to the pile injection still exists in the soil mass during this test. However, the load increment due to the time delay gradually decreases with the time.

In order to compare the test result between the tests using different soils, both time-load graphs are plotted together in the same graph (Fig. 9). Generally, both tests give the similar result that the load increments are increased sharply in short time delay, and then decrease for a longer time. This graphs indicated that the excess pore pressure due to the pile injection exists during the injection and it needs time to be dissipated. This graphs also confirms the short-term phase of the time-dependent pile bearing capacity [11]. Since this graph figures the maximum load of piles for short-term time, the equation (3) is not fit to represent the results.

Since the soil parameters are widely different, then it is quite difficult to choose a parameter which more gives effect to the time delay than the other parameters. However as generally understood the excess pore water pressure may be dissipated longer in the finer soil particle that coarse one. It also can be seen from this tests that after an hour delay, the load increment in the Lubuk Alung sand still exist but not in the Siteba sand. The Lubuk Alung sand has mean grain size 0.15mm which less that Siteba sand with 0.22mm.

Moreover, the coefficient of uniformity of Lubuk Alung of 3.2 is greater than the Siteba sand of 1.7, it is understood that the Lubuk Alung considered being better graded than the Siteba. Then, the Lubuk Alung sand is also denser than the Siteba sand, even though its relative density during the test is less. The minimum density of the Lubuk Alung is 1.60 t/m^3 which is greater than 1.52 t/m^3 of the maximum density of the Siteba sand.

In this study, for considering the significance of soil permeability, the laboratory tests of the sands have been also conducted. The permeability coefficients for Siteba and Lubing Along sands are 0.025 cm/sec. and 0.018 cm/sec. respectively. It is found that the increment loads due to the time delay are greater in the Lubuk Alung sand which has the smaller permeability coefficient. It means that the pore pressure due to the injection force is greater in this finer and lower permeability soil. Clearly, the permeability of soil has the potential to be a very significant factor in the capacity load related to the time delay of a deep foundation. This finding confirms the higher lateral pressure on the piles due to the dynamic load in the silty sand than in the clean sand as previously has been observed [9]. In order to elaborate the quantitative effects of soil parameters related to the time delay on the pile bearing capacity, a sophisticated observation in the laboratory as well as in field may be needed.

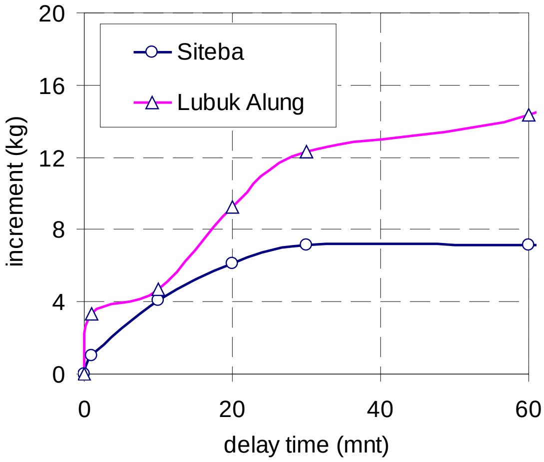

Fig.9 Delay time vs load relationship

4. CONCLUSION

The pile injection works on site showed that due to the time delay the pile experienced a temporary increase in the bearing capacity. The effects of time delay on pile bearing capacity in short-term that can be understood by a series of laboratory experiments. The experimental works using a pile model of saturated sands has succeeded demonstrating the load increment on penetration due to the time delay. In this study, two different soil media were used, the first soil is taken from Siteba and the second is from Lubuk Alung. The results of the experiments with of time delay variation show that the increment loads will decrease and may stop until a certain period. It seems that 30 minutes is an important interval for time delay.

In practical work, the time delay on the pile penetration must be limited to a certain time to avoid difficulty for continuing pile penetration to the deeper soil layers. The value of the time delay to dissipate the excess pore water pressure due to the injection in short-term may vary for different soils depend on the permeability of the soil as well as the other parameters such as particle size and density. Further study in the field and laboratory is suggested to be conducted to elaborate the qualitative effects of soil parameters on delay time and bearing capacity relationship.

5. ACKNOWLEDGEMENTS

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