UNIVERSITI SAINS MALAYSIA

Laporan Akhir Projek Penyelidikan Jangka Pendek

A Study on the Increase in Load Capacity of Driven Pile Over Time or 'Set-Up'

by

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Estimation of Side Shear Friction of Driven Pile based on CPT

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ABSTRACT: Cone Penetration Test (CPT) is commonly used in characterising soil parameters and geotechnical engineering application. Due to its similarity of insertion to pile installation, the cone data has been used to predict pile bearing capacity widely. This study focuses on CPT data used to predict pile side shear friction. Furthermore, it is also proposed to estimate the soil/pile set-up parameter such as the increased shaft capacity with time. The study consist a set of model piles which designated to simulate the actual pile driving mechanisms. The model pile has size similar to CPT in order to eliminate the size factor. CPT data such as tip resistance, q_c and sleeve friction (f_s) are used to predict the side shear friction of model pile and further compared to the actual tested capacity. Besides that, an indirect approach is proposed to find the soil/pile set-up parameters by sensitivity parameter surveyed by CPT. Findings of the study shown some new factors or friction coefficient of CPT method in estimation pile capacity is proposed for this particular side. that the factors such as empirical or semi-empirical will justify the suitability of CPT formula to predict pile capacity and soil/pile set-up parameter.

Key words: cone penetration test, soil/pile set-up, tip resistance, sleeve friction, side shear friction.

1.0 INTRODUCTION

1.1 Development of Cone Penetration Test

Cone penetration has been used in soil characterisation since 1934 in Holland. This mechanical cone was used to locate and evaluate sand density originally. The system comprised of inner tube and outer tube and was pushed into the ground to obtain the total soil resistance. The total soil resistance comprised soil shear friction and tip resistance in one. In order to further differentiate the tip resistance from total resistance, the inner tube with cone point was introduced a small distance into the ground while outer tube (sleeve) was held in its position. Later, sensors like strain gauges were developed to measure the tip and side shear friction directly and used widely in the late 1960s.(Begemann, 1963). This cone is known as the electrical cone penetrometer as shown in Figure 1.

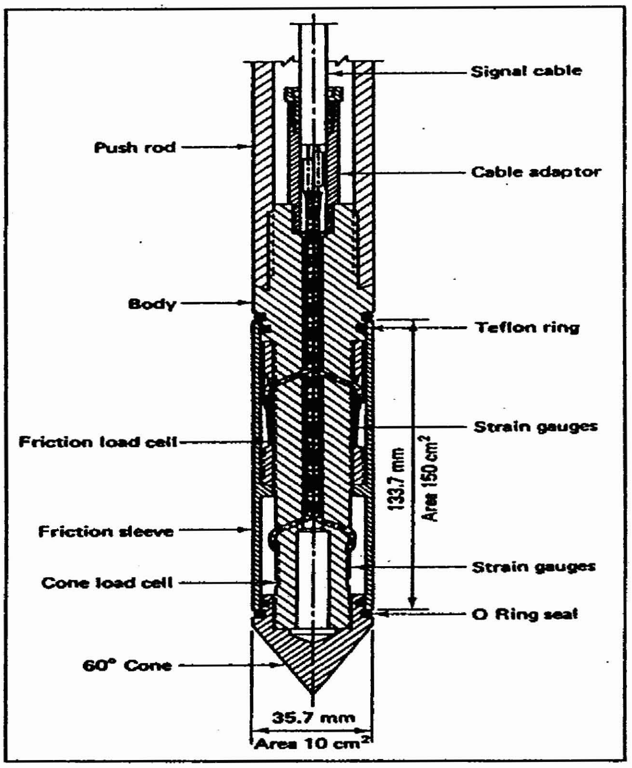

Figure 1: Schematic diagram of electric cone penetrometer (Jaksa, et. al., 2000)

Piezocone or CPTu was introduced in the early 1980's. This device able to measure tip resistance, side shear friction and pore water pressure at close distances. Measurement of CPT is not static and is standardized to 20mm/s. The advancing of cone in ground induced excess pore water pressures and made the measurements required necessity correction before adopted in routine design. The induced pore water pressure largely depends on soil type. Measurement of pore water pressure could be negative in dense fine sands due to dilation effects, small values in pervious soil and significant large in silts and clays.(Fellenius et. al., 2000). Schematic diagram of piezocone is shown on Figure 2.

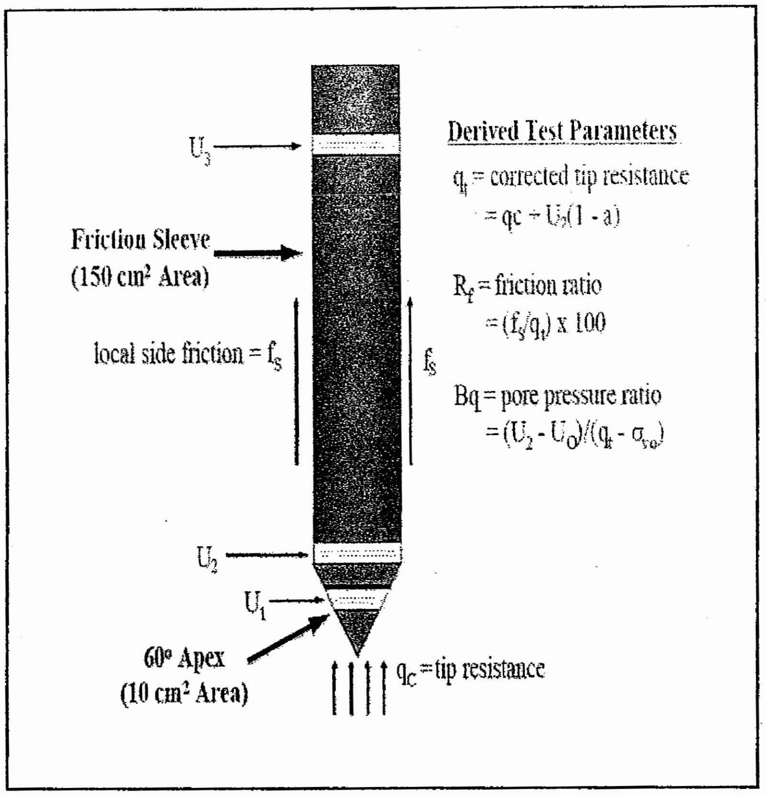

Figure 2: Schematic diagram of piezocone (Bennert et. al., 2002)

2.0 INTERPRETING CONE PENETRATION TEST

2.1 Application of CPT

CPT is a very useful tool for in-situ soil characterisation. Titi & Abu Farsakh (1999) explained CPT data can be used to identify soil strata and classification. Normally, low cone tip resistance with high sleeve friction or high friction ratio indicates the existence of clayey soils. Else is about sandy soils. Several methods were proposed for soil profiling such as Begemann (1965), Schemertmann (1978), Dougles & Olsen (1981), etc. their methods are detail explained in their related papers which are excluded here. Besides that, CPT can be used to find out the side shear friction of pile, as well as end bearing capacity of pile. Methods of Schmertmann, De Ruiter & Beringen, Tumay & Fakhroo, Price & Wardle, and Philipponnat are described in details in section 2.3.

2.2 Effects of pile driving in soils

Driven pile is driven into ground by designated hammer weight and drop high. As the pile is driven, the soil at its tip is pushed laterally to location at or beyond the pile radius and this will cause shear failure of the soil. The soil in the immediate vicinity of the pile is significantly remoulded by the driving process and vertical cracks might form in this region. This region is called plastic zone. After the plastic zone, the soils at greater distance are deformed outward, but do not undergo extensive shearing. This is because the shear stresses are not sufficient to cause failure of soil. As a result, the soil is stressed elastically, and no loss of their interparticle bonds, fabric arrangement and or cementation. The stored elastic strain in soil tries to force the soil back against the pile.

Fig. 3 shows the various zones of region driven by a pile. In the centre is the displaced zone, surrounded by the plastic zone, and on the outside, the elastic zone. The tangential horizontal stress (σ_t) decreases, while the pore water pressure increases as the soil pushed outwards. As a result, vertical cracks forms in the soil which controlled by maximum possible radial pressure (σ_r). (Massarch, 1978).

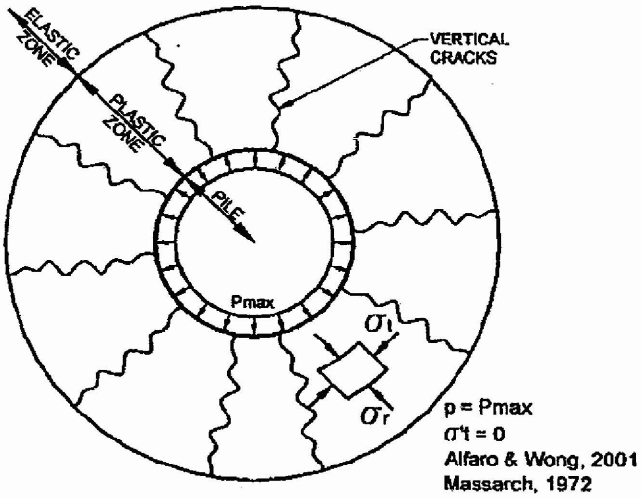

Figure 3: stress zone around pile (after Alfaro & Wong, 2001)

2.3 Application of CPT parameters in pile design

Some researchers found that CPT methods in pile capacity give better estimation compared to most conventional methods. (Roberston et. al., 1988; Briaud & Tucker, 1988; Tand & Funegard, 1989; Sharp, et. al., 1988). This is due to the continuous soil profiling provides the entire soil response along the pile shaft. Reduction factor should be considered during measurement of CPT values. Reasons are difference in insertion, scale effects, differences of soil displacement between actual pile and cone, etc. Generally, two parameters in CPT are referred during pile design. Measured cone tip resistance (q_c), is used to find out the unit tip bearing capacity of the pile (q_t). While side shear resistance of the pile (f) is either evaluated from sleeve (f_s) or cone tip resistance (q_c). Main focus of the study is about side shear resistance of driven pile in clay, so a total nine (9) relevant methods were compiled and summarized in table 1.0.

Table 1.0: CPT method to predict unit shaft capacity of driven pile

Methods	Unit shaft capacity (f) of pile in clay
Schmertmann (1978)	$f = \alpha' f_s$ α': ratio of pile to cone sleeve friction (Fig. 4) f_s = average sleeve friction
De Ruiter & Beringen (1979)	$f = \beta \frac{q_c(side)}{N_k}$ β: adhesion factor, normally consolidated clay = 1, and 0.5 for overconsolidated clay, N_k: cone factor ranges from 15 to 20 depends on local experience. q_c(side)= the average cone tip resistance along the soil layer
Tumay & Fakhroo (cone-m method) (1982)	$f = mf_{ca}; f_{ca} = \frac{F_t}{L} \leq 72kPa$ $m = 0.5 + 9.5e^{-0.09fca}$ f_{ca}: average local friction in kPa, F_t: total cone penetration friction determined for pile penetration length (L)
Price & Wardle (1982)	$f = k_{s1} f_s \leq 0.12TSF(120kPa)$ k_{s1} : pile type factor (0.53 for driven pile)
Philipponnat (1980)	$f = \frac{\alpha_{s1}}{F_{s2}} q_c(side)$ α_{s1} : depends on the pile type = 1.25 for precast driven pile F_{s2} : factor depends on the soil type (Table 2.0)
Penpile (Clisby et. al., 1978)	$f = \frac{f_s}{1.5 + 14.47 f_s}$ f and f_s (MPa)
LCPC (Bustamante & Gianeselli, 1982)	$f = \frac{q_c(side)}{k_{s2}} \leq f_{max}$ k_{s2} & f_{max} refer to table 3
Eurocode-7-3	$f = \alpha_{s2} q_{c,z}$ q_c : cone tip resistance at depth z; α_{s2} : factor depending on the pile class and soil conditions (Table 4)
ERTC3	$f = \alpha_{s3} q_{c,z}$ Similar to Eurocode-7-3, but α_{s3} values are different (Table 4)

Table 2.0: Empirical factor Fs. (Titi & Abu Farsakh, 1999)

Soil type	F_{s2}
Clay, and calcareous	50
Silt, sandy clay and clayey sand	60
Loose sand	100
Medium dense sand	150
Dense sand and gravel	200

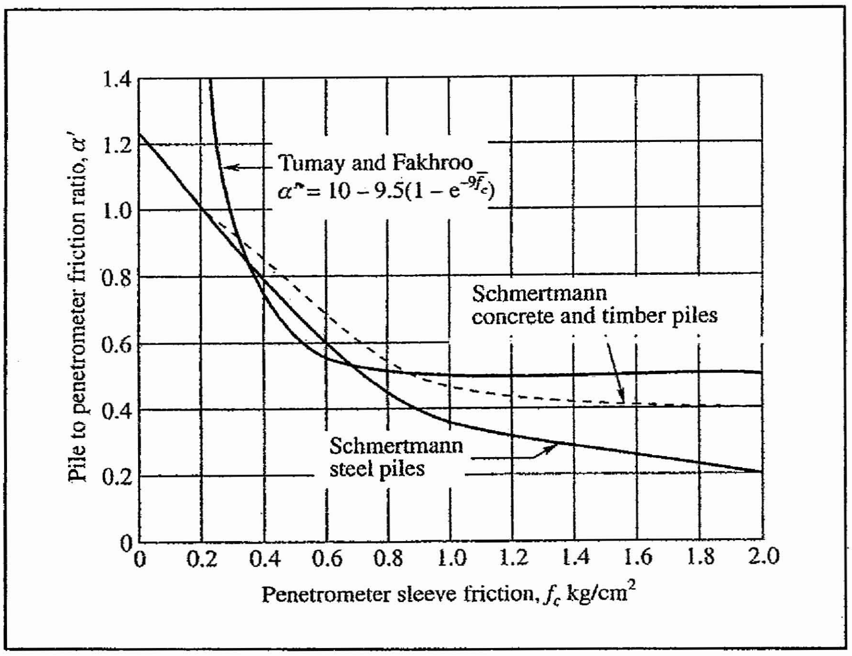

Figure 2.1: Ration of pile to sleeve friction (α') (Murthy, 2003)

Table 2.1: Friction coefficient, (k_{s2}) and maximum limit of unit shaft friction (f_{max})

Natural of soil	Friction coefficient, (k_{s2})	Maximum unit shaft friction (f_{max}) (KPa)
Soft clay and mud	90	15
Moderately compact clay	40	35 (80)*
Silt and loose sand	60	35
Compact to stiff clay & compact silk	60	35 (80)
Soft chalk	100	35
Moderately compact sand and gravel	100	80 (120)
Weathered to fragmented chalk	60	120 (150)
Compact to very compact sand and gravel	150	120 (150)

* f_{max} values in bracket apply to careful execution and minimum disturbance of soil due to construction

Table 2.2 : Friction coefficient, α_{s2} (EUROCODE-7-3)

Soil type	Relative depth z/D	αs
Fine to coarse sand		0.006
Very coarse sand		0.0045
Gravel		0.003
Clay/silt ($qc \leq 1$ MPa)	$5 < z/D < 20$	0.025
Clay/silt ($qc \geq 1$ MPa)	$z/D \geq 20$	0.055
Clay/silt ($qc > 1$ MPa)	Not applicable	0.035
Peat	Not applicable	0

Table 2.3 : Friction coefficient, α_{s3} (ERTC3)

Soil group	Soil type	α_s
Cohesionless	Gravel	0.003
	Sandy gravel	0.0045
	Fine sand	0.006
	Sandy silt	0.008
	Silt	0.01
Cohesive	$q_c > 2500$ kPa	0.015
	1500 kPa $< q_c <$ 2500 kPa	0.025
	1000 kPa $< q_c <$ 1500 kPa	0.035
	500 kPa $< q_c <$ 1000 kPa	0.045
	$q_c <$ 500 kPa	0.055

3.0 Research Methodology

3.1 Site description

This study was conducted at Engineering Campus, University Science Malaysia, Nibong Tebal, Pulau Pinang. A total 3 testing pits were identified for model pile installation. Previous soil investigation shown that the site mainly consists of clayey soil on first 9 m b.g.l, SPT N values varies from 0 blows to 10 blows. Locations of the testing pits were shown on Figure 3.1.

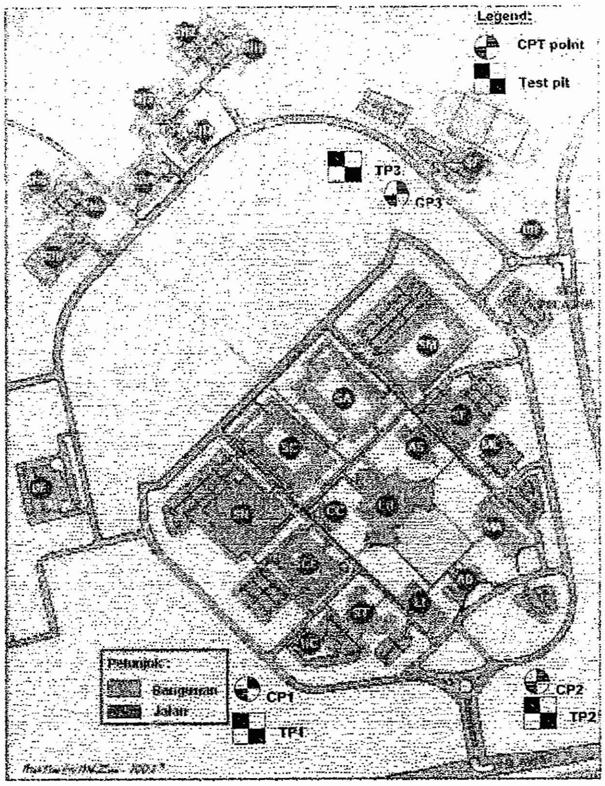

Figure 3.1: Location of testing pit and CPT point in Engineering campus, USM.

3.2 Model pile installation

Total 4 numbers of steel rods were driven to 2.4m deep from original ground level. Two (2) different driving energies were used to drive the rods. At TP1 and TP2, the lifting height was 300mm with driving energy 29.4J. While TP3 uses 20kg hammer and driving energy was 39.2J. Blow numbers were recorded at every 100mm penetration. Pile verticality was monitored during driving process.

3.3 Tension test

Tension test was conducted when the waiting time of model pile matured. Each rod was left in place for certain time frame after installation. For example, TP1, rod A was tested at Day 1 (D1) after installation, while other rods were tested at D3, D5 and D7 respectively. Waiting time of each rod tabulated in table 1.0. Rod was lifted by using a pulling frame as on Figure 5.0. Load was applied on weight plate holder. Each load increment was 196.2 N (20kg) until the rod failed. Holding time about 2 minutes was required after each load increment in order to allow soil has sufficient time to respond to the load. After the rod failed, load was reduced to check the residual load. Rod displacement was measured by dial gauge with accuracy of 0.02mm. The testing process is shown on figure 6.0.

Table 1.0: Elapsed time of installed rod before pulling test

Test Pit	Rod number (rod label)	Elapsed Time (Days after installation)
TP1	1 (TP1-1)	1
	2 (TP1-2)	3
	3 (TP1-3)	5
	4 (TP1-4)	7
TP2	1 (TP2-1)	1
	2 (TP2-2)	7
	3 (TP2-3)	14
	4 (TP2-4)	28
TP3	1 (TP3-1)	1
	2 (TP3-2)	14
	3 (TP3-3)	28
	4 (TP3-4)	49

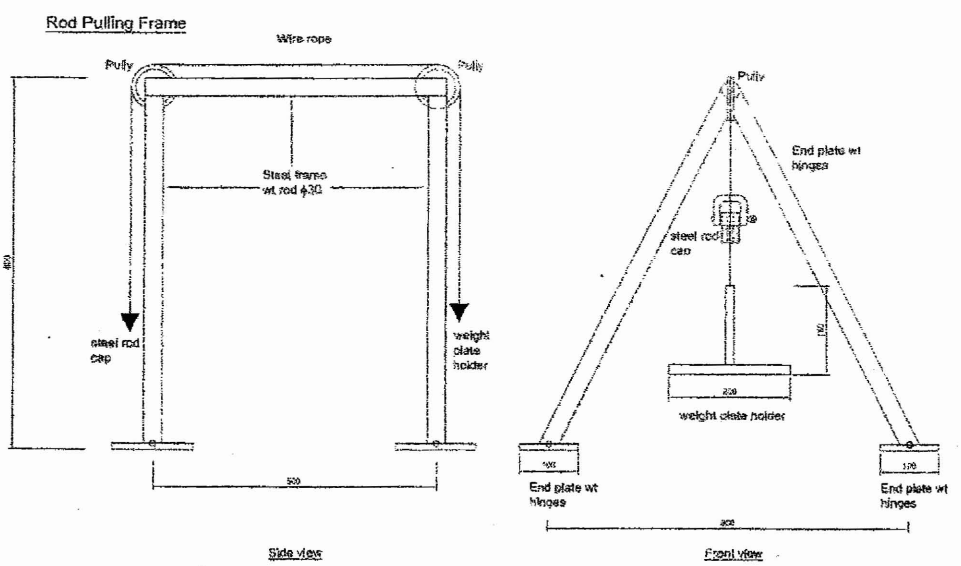

Figure 3.2: Schematic diagram of pulling frame

4.0 Comparison of test results and CPT' predicted values

A total 12 testing results of model pile were available for comparison with side shear resistance predicted by CPT methods. Focus of this study is to investigate the most reliable method to predict pile shaft capacity and the relevant CPT parameters which lead the changes of shaft capacity with time.

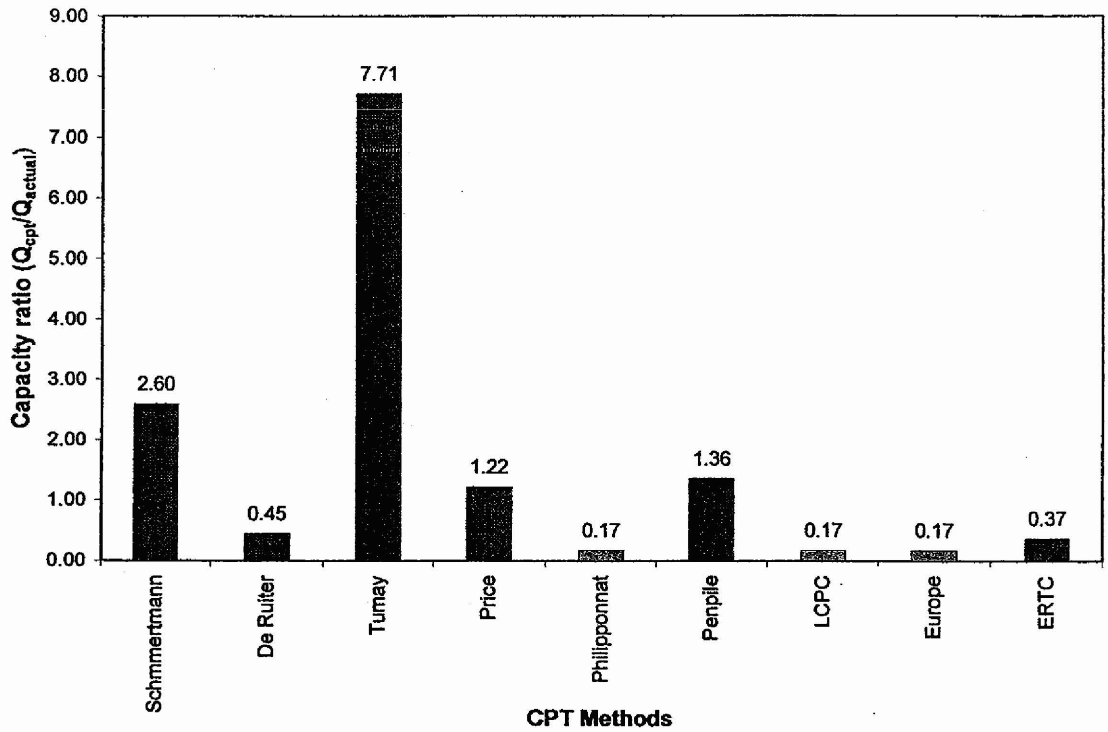

Figure 4.1: Capacity ratio (Q_{cpt}/Q_{actual}) of TP1

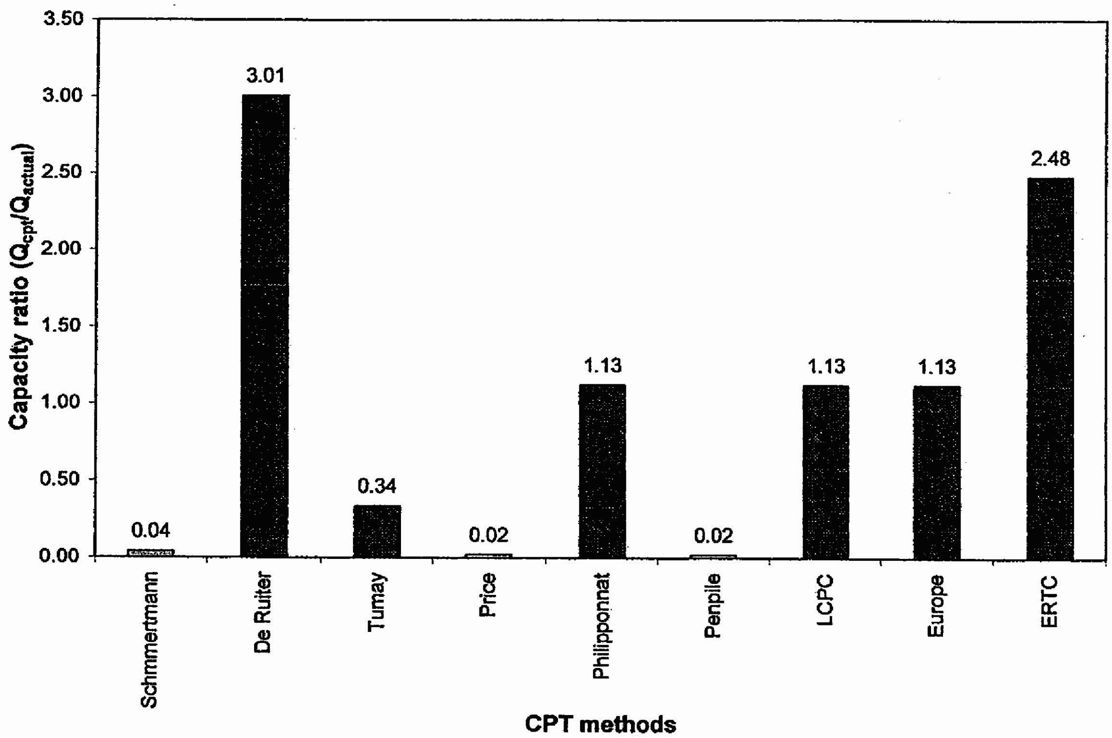

Figure 4.1: Capacity ratio (Q_{cpt}/Q_{actual}) of TP2

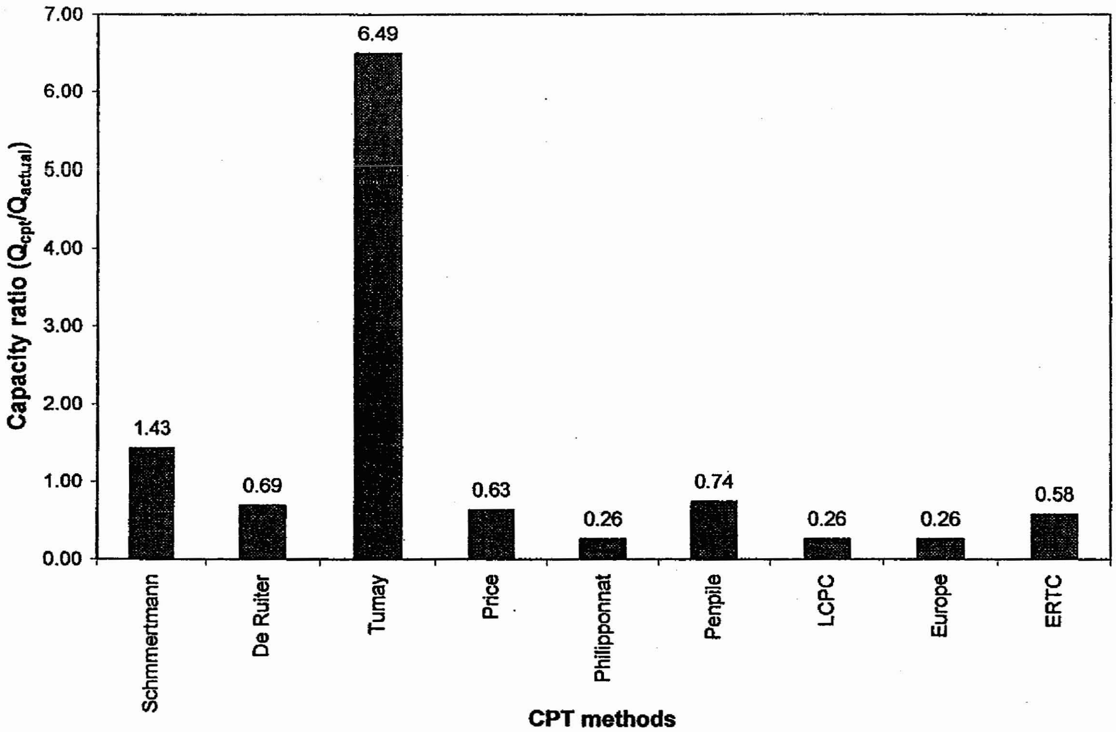

Figure 4.1: Capacity ratio (Q_{cpt}/Q_{actual}) of TP3

EVALUATION OF SOIL/PILE SET-UP FROM DRIVEN CONE PENETROMETER

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SUMMARY: Bearing capacity of driven pile often increases with time after installation. This phenomenon is known as soil/pile set-up. To predict the set-up effect and include it in the routine pile designing works, soil exploration tests are required to determine the soil parameters relevant for predicting soil/pile set-up. The driven cone penetrometer (DCP) is one method of investigation which has the similar installation to a driven pile. Basically, it consists of a small steel cone which is driven into the ground by hammer. A pull-up test is then conducted to check the unit shaft capacity at different elapsed times. In this paper, a method of DCP testing is proposed to identify the soil/pile set-up parameter during ground investigation. Findings of this study shows that the range of soil/pile set-up were 0.024 – 0.239 for a particular site.
Key words: driven cone penetrometer, driven pile, soil/pile setup.

INTRODUCTION

Pile driving – effects on soil

A driven pile is installed by impact energy forcing the pile to penetrate the soil. Impact energy is created by air, diesel or steam impact hammers during pile driving. The impact energy of pile driving normally varies from 5 – 300 kJ/blow and its longitudinal oscillations are in the range of 7 – 50 Hz. As the pile is driven, the soil at its tip is pushed laterally to a location at or beyond the pile radius and this will cause shear failure of the soil. In figure 1, the soil in the immediate vicinity of the pile is significantly remoulded by the driving process and vertical cracks might form in this region. This region is called the plastic zone. After the plastic zone, the soils at greater distance are deformed outward, but do not undergo extensive shearing. This is because the shear stresses are not sufficient to cause failure of soil. As a result, the soil is stressed elastically with no loss of their inter-particle bonds, fabric arrangement and or cementation. The stored elastic strain in soil tries to force the soil back against the pile.

W K NG et. al., Evaluation of soil pile set-up from Driven Cone Penetrometer. *Proceedings of the BGA International Conference on Foundations, Dundee, Scotland,24 – 27 June 2008.* IHS BRE Press, 2008.

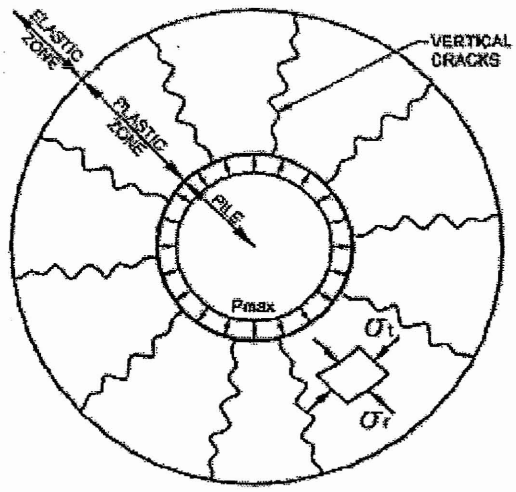


Fig. 1: *Stress zones around a driven pile (after Alfaro & Wong[2])*

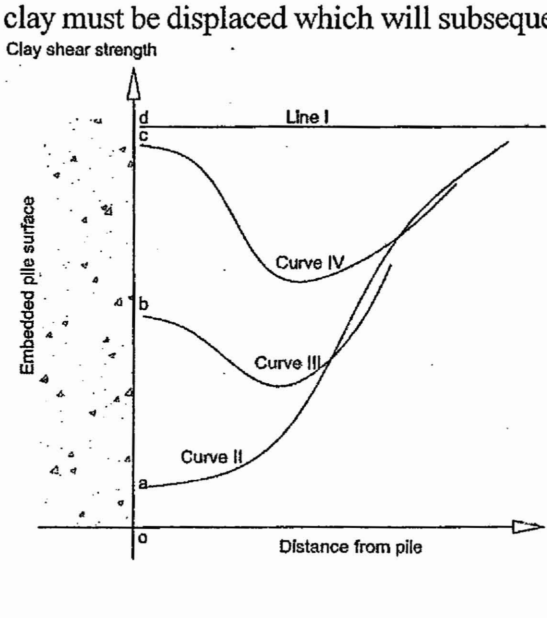
Figure 1 shows the zones of region around a driven pile. In the centre is the displaced zone which is surrounded by the plastic zone which in turn is surrounded by an elastic zone. The tangential horizontal stress (σ_t) decreases, while the pore water pressure increases as the soil is pushed outwards away from the pile shaft. As a result, vertical cracks forms in the soil which is controlled by maximum possible radial pressure (σ_r) (Massarch[1]).

Excess pore water pressure dissipation

Generally, pile driving induced pressure to its surrounding soil and part of it transferred to the soil in the form of excess pore water pressure. Dissipation of such excess pore water depends on soil type. Datta[3] stated that the excess pore pressures induced by pile driving in non-cohesive soil seldom exceed 20% of the effective overburden stress. Moreover, this takes place locally around the pile tip. Further up along the shaft the excess pore pressure becomes gradually smaller. Pile installation in clay is different from pile driving in sand. According to D'Appolonia[4], pile penetration into clay induced an excess pore pressure that can be much larger than the initial effective overburden stress. After the completion of pile driving and the dissipation of excess pore pressure, the soil reconsolidated. Roy et al.[5] showed that the increase in excess pore water pressure is negligible at distance beyond 8 pile diameters from the pile wall.

Changes of the state of stress in the vicinity of the driven pile

When a pile is driven into a deep deposit of impervious saturated clay, an equal volume of clay must be displaced which will subsequently affect the strength of the clay along the pile.



In Figure 2.0, a pile is shown to be embedded in a clay stratum. Before pile driving, undisturbed strength of the clay said is represented by ***od*** or ***Line I***. Immediately after pile installation, the shearing strength of clay is remoulded (***curve II***) and only a small fraction of original strength ***od*** remains and the remoulded strength of clay at this stage is ***oa***. The driving pressure is transferred to the pore water and a hydrostatic excess pressure is created. This hydrostatic excess pressure is high around the pile and becomes lesser at a distance from the pile.

Fig. 2: *Changes of shearing strength in saturated clay before and after installation of a driven pile*

Since flows always take place from points of high excess pressure to points of lower pressure, it results in the start of the consolidation process. Remoulded soil strength along the pile builds up with time (***ob***) and this phenomenon is called soil/pile set-up. The recovered strength of clay is now represented by ***curve III*** as a function of distance from the pile. If ***curve III*** represents the remoulded soil strength which is developed few days after driving, then ***Curve IV*** or remoulded strength ***oc*** represent the strength developed few weeks later after driving.

Soil/pile set-up factor, A

Many researchers have reported that the relationship of soil/pile set-up with the log of time is linear after the initial pile bearing capacity or the End of Driving (EOD). Skov and Denver[6] have proposed a dimensionless set-up factor, (A), to explain the effect of soil/pile set-up. The factor represents the relative increase in pile capacity per log cycle of elapsed time,

$$Q_t = Q_{initial}\,[A\log(\frac{t}{t_{initial}}) + 1] \qquad (1)$$

where A=dimensionless set-up factor; Q_t= total pile capacity at time t; $Q_{initial}$=total pile bearing capacity at initial reference time $t_{initial}$; t = time elapsed since initial driving.

A numbers of empirical formula to explain the relationship of set-up for driven piles have been proposed. For example, Guang-Yu[7] explained the relationship of set-up for driven pile in soft soils according to its sensitivity. Similarly, Huang[8] presented a formula to predict set-up rate in the soft soil in Shanghai while Svinkin[9] proposed an empirical relationship of set-up in sand according to load test data. Their formulas for soil/pile set-up are summarised in Table 1.

Table 1. *Examples of soil/pile set-up empirical formula*

Author	Empirical Formula	Eqn
Guang-Yu[7]	$Q_{14} = (0.375S_t + 1)Q_{EOD}$ where S_t = sensitivity of soil; Q_{14} = pile capacity at 14 days	(2)
Huang[8]	$Q_t = Q_{EOD} + 0.236[1 + \log(t)(Q_{max} - Q_{EOD})]$ where Q_t = capacity at time t; Q_{EOD} = capacity at EOD; Q_{max} = maximum capacity	(3)
Svinkin[9]	$Q_t = 1.4Q_{EOD}t^{0.1}$ upper bound $Q_t = 1.025Q_{EOD}t^{0.1}$ lower bound	(4) (5)

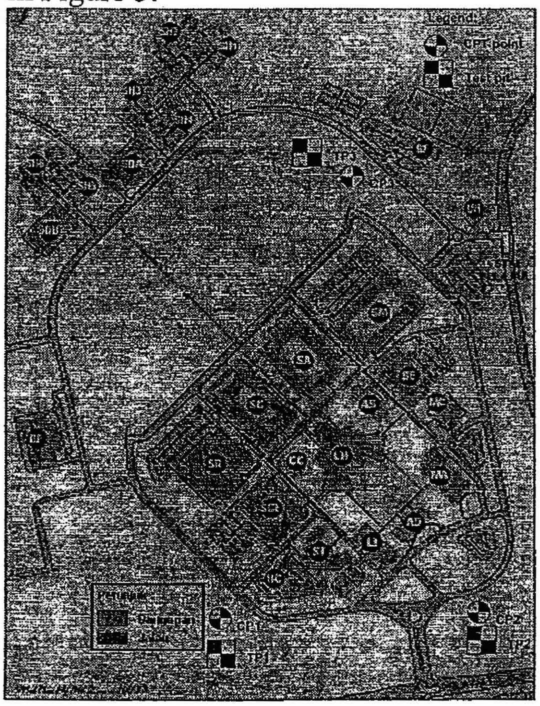
METHODOLOGY OF DRIVEN CONE PENETROMETER

Objective of the study

Main objective of the study is to develop a miniature tool to simulate the driven pile installation in order to study the development of shaft capacities with time. Generally, DCP is a common tool used to determine soil consistency during soil investigation. In this research, the DCP was further used to investigate the soil/pile set-up parameters due to its similarity of driven pile installation. The DCP is a straight sided smooth steel cylinder with diameter 34mm and 60^0 apex angles at its tip. The total length of DCP is 2.5m and the targeted embedded length is 2.4m. Each set of DCP consists of 4 rods and installed by driving method. Shaft capacity of each rod was tested by pull-up test in different time frame after installation. The increment of shaft capacities was analysed to determine the soil/pile set-up factor, A from using equation (1).

Site description

This study was conducted at the Engineering Campus, University Science Malaysia, Nibong Tebal, Penang, Malaysia. A total of 3 test pits (TP) were identified for DCP installation, which were labeled as TP1, TP2 and TP3. Locations of the test pits are shown in Figure 5.



Previous soil investigation has shown that the site mainly consists of clayey soil for the first 9 m b.g.l, with SPT N values varying from 0 to 10 blows. Cone penetration test (CPT) was conducted near to each TP to determine the soil consistency. A typical CPT result is shown in Figure 6. This device able to measure tip resistance (q_c), side shear friction (f_s) and pore water pressure (p_w) at close distances. Measurement of CPT was not static and was standardized to 20mm/s. From the CPT results, the existing soil profile of each TP mostly consisted of sensitive fine material or clay for the initial 2.9m b.g.l. Average q_c and f_s are tabulated in Table 2.

Fig. 5: *Location of test pit and CPT in Engineering Campus, University Science Malaysia*

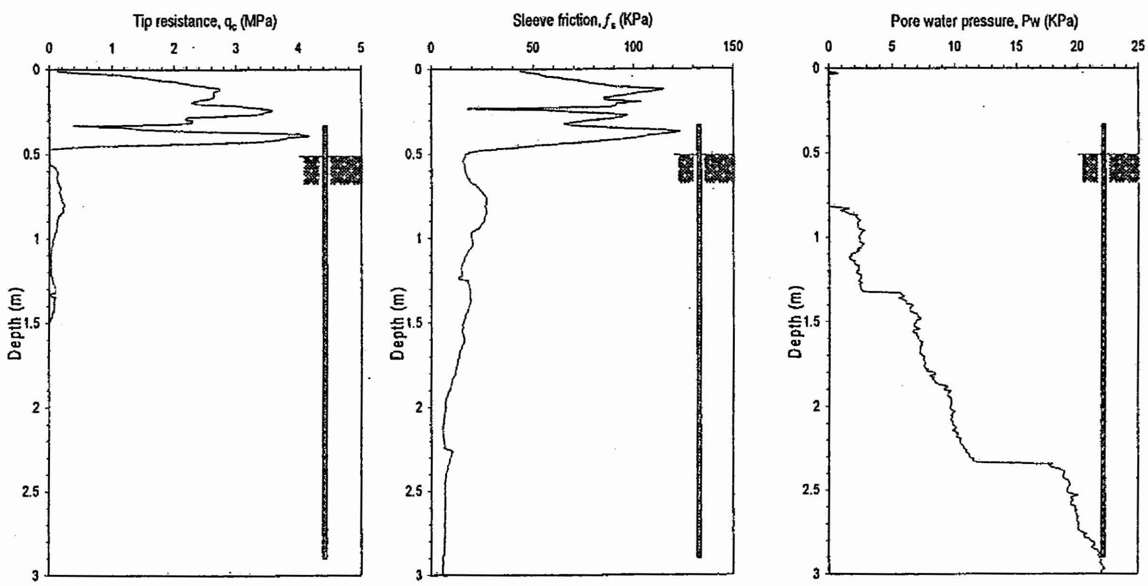


Fig. 6: *Cone penetration test result of TP1. (a) Tip pressure, q_c, (b) sleeve friction, f_s, (c) pore water pressure, p_w*

Table 2. *Average CPT parameters of test pit*

Test Pit	Tip pressure, q_c(MPa)	sleeve friction, f_s(kPa)
TP1	0.039	13.38
TP2	0.267	0.203
TP3	0.063	7.26

Installation of Driven Cone Penetrometer

The top soil about 0.5 m thick at each TP was removed before the installation of DCP. The cone was driven to 2.4 m embedded depth by hammer with a predetermined drop height.

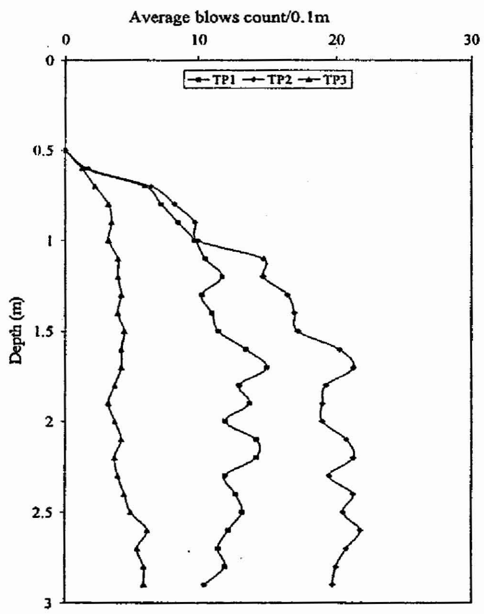


Fig. 7: *Average blow count per0.1 m penetration of test pit*

Two (2) different driving energies were used to drive the cones. At TP1 and TP2, the lifting height was 0.3 m with driving energy 29.4 J; while TP3 uses 20 kg hammer with driving energy 39.2 J. Blow counts were recorded at every 0.1 m penetration. Cone verticality was monitored during the driving process. Average blow count for each TP are summarised in Figure 7. From the figure, obviously TP3 has lower blow count compared to others due to the higher driving energy. At TP2, although its driving energy was the same to TP1, its blow counts were less compared to TP1. The soil at this particular test pit (TP2) is relatively softer compared to TP1.

Procedure of pull-up test

Each cone was left in place and be tested when the targeted elapsed time was due. For example, at TP1, cone 1 was tested at Day 1 (D1) which was 24 hours after installation; while other cones were tested at D3, D5 and D7. Elapsed time of each cone is tabulated in Table 3. During pull-up test, cone was lifted by using a pulling frame as shown in Figure 8. Load was applied on the weight plate holder and each increment of load was 196.2 N (20 kg). The corresponding displacement was recorded. Holding time of each load increment was initially 5 minutes which required by quick maintained load test method[10] but later reduced to 2 minutes. This was because author encountered the displacement was responded to the applied load during the first minute at this particular site. Ultimate load or failure was assumed when the displacement was continuous without any further load increase. After the cone failed at ultimate load, the subsequent loads were reduced to determine the constant displacement at residual load. Cone displacement was measured by dial gauge with an accuracy of 0.01 mm.

Table 3. *Elapsed time of pull-up test*

Test Pit	Cone number	Elapsed Time (Days after installation)
TP1	1 (TP1-1)	1
	2 (TP1-2)	3
	3 (TP1-3)	5
	4 (TP1-4)	7
TP2	1 (TP2-1)	1
	2 (TP2-2)	7
	3 (TP2-3)	14
	4 (TP2-4)	28
TP3	1 (TP3-1)	1
	2 (TP3-2)	14
	3 (TP3-3)	28
	4 (TP3-4)	49

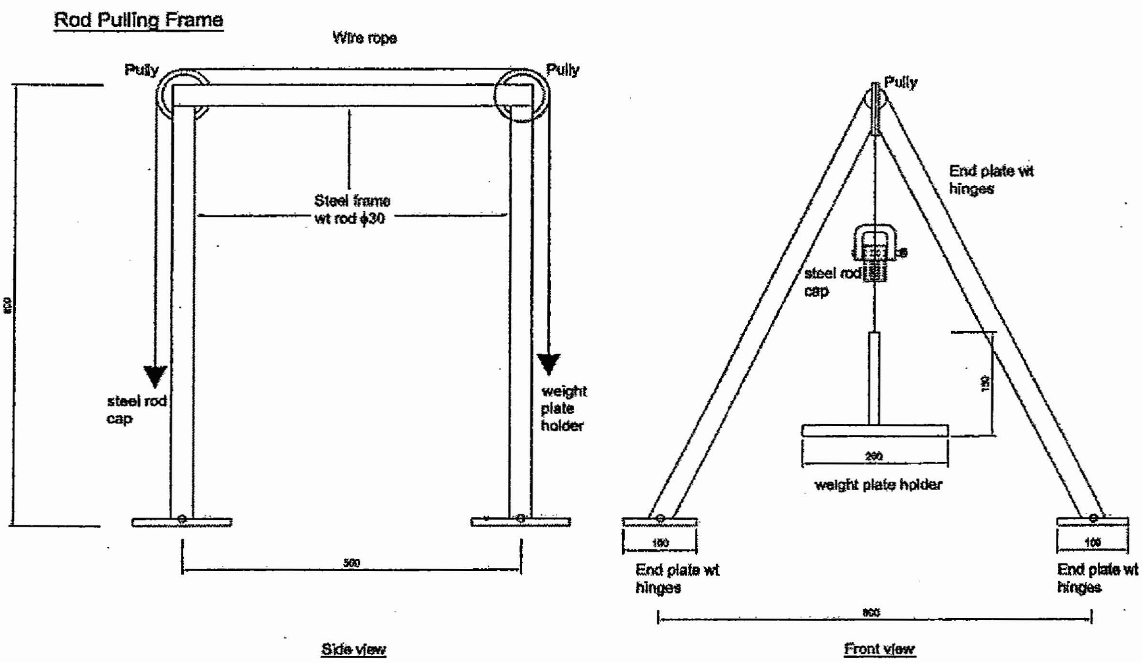


Fig. 8: *Schematic diagram of pulling frame*

DISCUSSION

Load ratio versus Displacement ratio

Pull-up test results at each TP were presented on Figure 10, 11 and 12. The ratio of applied load over failure load (load ratio) was compared to the ratio of displacement over failure displacement (displacement ratio). Generally, the corresponding load ratio of the similar displacement ratio increases with time. In Figure 10, the line's gradient on Day 3 was steeper than that on Day 1. It means that greater load is required to displace the cone a unit of displacement. This is because the soil particles move closer to each other after some time. It is assumed that the dissipation of excess pore water pressure and reconsolidation happened and subsequently increase the shear modulus of soil. This phenomenon increases its shear resistance along the cone shaft resulting in soil/pile set-up effects. However, not all the cones responded in a similar manner. For example TP2-1 had higher load ratio compared to TP2-3 and TP2-4; while TP3-4 showed the lowest load ratio compared to other cones in TP3. This may due to local variations in soil condition.

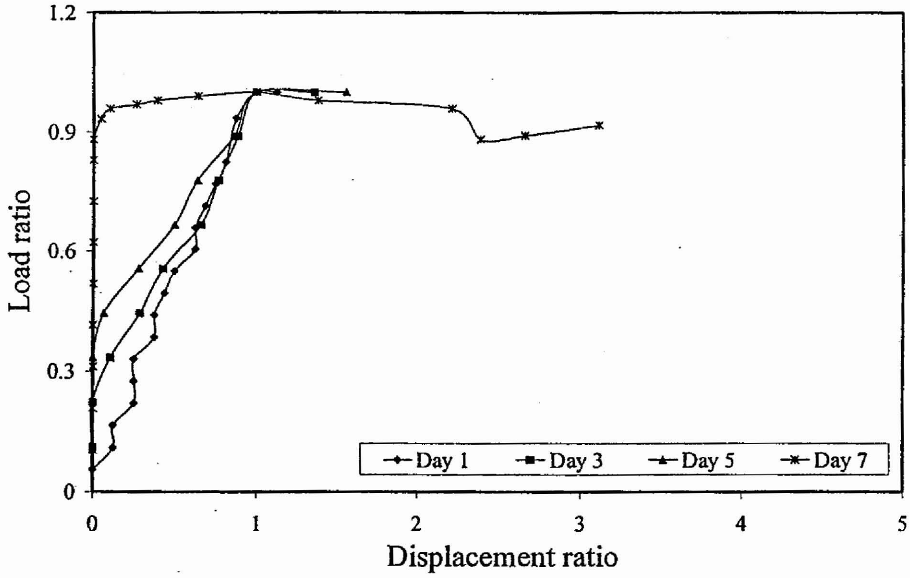


Fig. 10: *Load ratio versus displacement ratio of TP1*

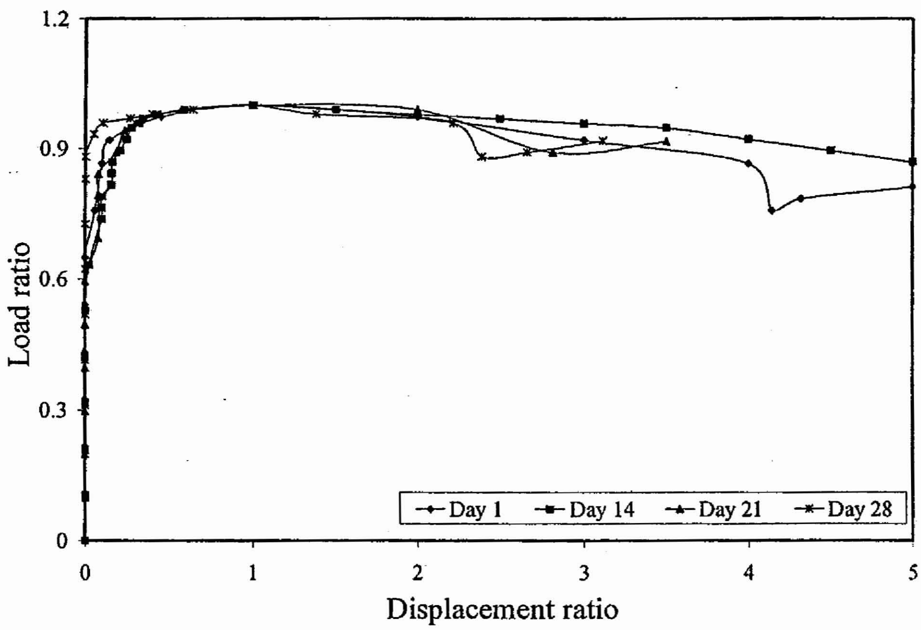


Fig. 11: *Load ratio versus displacement ratio of TP2*

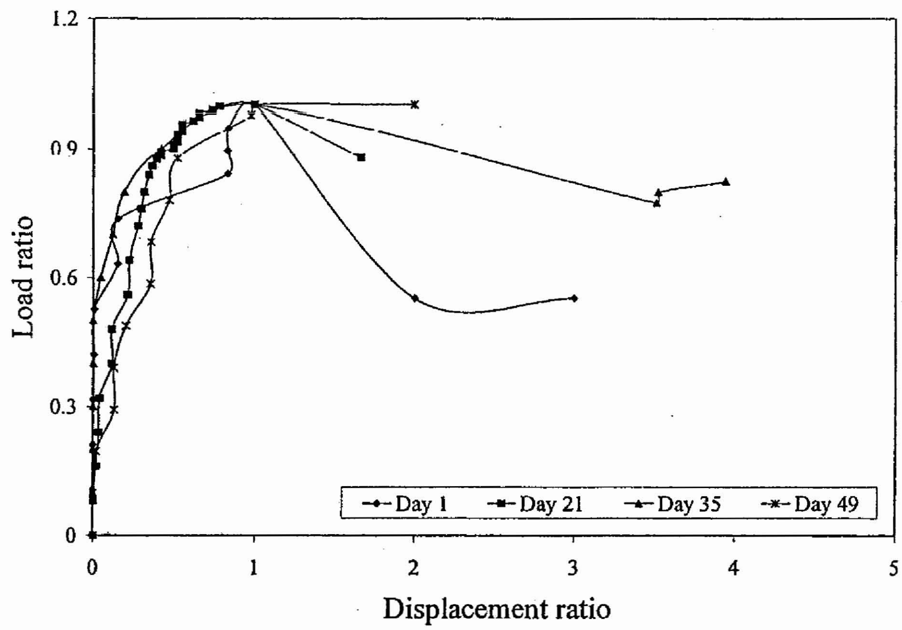


Fig. 12: *Load ratio versus displacement ratio of TP3*

Soil/pile Set-up Factor

The maximum unit shaft resistance of each cone was obtained from the pull-up test. The results were used to calculate the soil/pile set-up factor according to Equation (1) and the values are summarised in Table 3. Basically, the unit shaft resistance did not change much during the first 7 days especially for TP1. It might due to the excess pore water pressure remaining high during that period of time. The changes in shaft capacity become more significant as the elapsed time becomes longer (after D7). For example, the A factor of TP2 ranges between 0.024 – 0.07; while in TP3, the A factor ranges from 0.047 – 0.239. In figure 13, the soil/pile factor, A is plotted versus log of time. Although the values are drastic, the average value of soil/pile set-up at this particular site was about 0.082.

Table 3. *Soil/pile Set-up factor, A*

Test Pit	Rod number	Elapsed Time (Days after installation)	Unit shaft Resistance (kN/m^2)	Soil/pile Set-up factor, A
TP1	1 (TP1-1)	1	5.83	
	2 (TP1-2)	3	5.77	-0.022
	3 (TP1-3)	5	5.77	-0.015
	4 (TP1-4)	7	5.67	-0.032
TP2	1 (TP2-1)	1	5.93	
	2 (TP2-2)	7	6.09	0.024
	3 (TP2-3)	14	6.47	0.070
	4 (TP2-4)	28	6.18	0.030
TP3	1 (TP3-1)	1	6.09	
	2 (TP3-2)	14	8.01	0.239
	3 (TP3-3)	28	5.13	-0.102
	4 (TP3-4)	49	6.57	0.047
	Average soil/pile set-up factor*			0.082

* negative soil/pile set-up factor are ignored

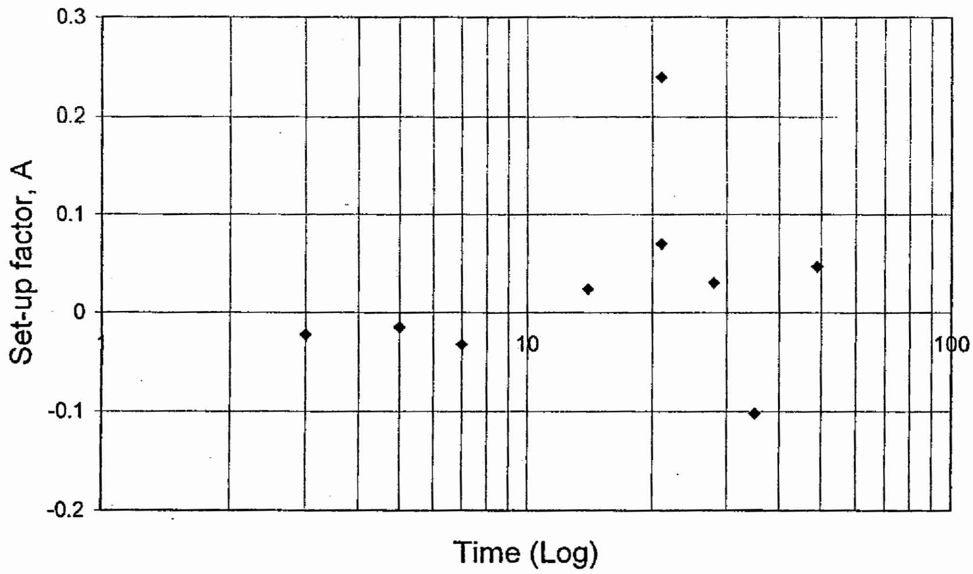


Fig. 13: *The variation of the soil/pile setup factor, A with time*

CONCLUSION

A soil exploration technique called the driven cone penetrometer (DCP) has proposed to determine the soil/pile set-up factor. This method simulates the driven pile mechanisms. It consists of a series of small scale steel cone (34mm in diameter), driven to a designated depth, which is then followed by pull-up test. The shaft capacities of steel cones are determined and compared to find out the soil/pile set-up factor.

From the study, the ultimate load of cone was increased with time generally. The gradient of load ratio versus displacement ratio becomes steeper with time and it maybe due to the soil surrounding the cone becomes stiffer as the elapsed time becomes greater. Study also shown that, soil/pile set-up effect was not significant during the initial seven (7) days after installation. It might be due to low dissipation rate of excess pore water pressure in cohesive soil like clay. Values of A ranged from 0.024 to 0.239 at this particular site which corresponded to the elapsed time of 7 days to 49 days. Findings from the literature papers showed that A ranges from 0.2 – 0.8 (Axelsson[11]; Chow et. al.[12]).

The empirical relationship representing the effect of set-up on pile capacity was established from the study at this particular site. Information on how to account for set-up in pile design can result in the use of smaller pile sections, shorter piles, higher pile capacities, and more economical installations of pile etc. which will benefit the industry associated with piling. This meaningful and cost-effective exploration-phase field test (DCP) can be added to the routine of sub-surface exploration programs to enable the development of a systematic approach which incorporate soil/pile set-up in the design optimization of piles.

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GT4

Incorporation of Pile-Soil Set-Up Effect for Cost Effective Pile Foundation Design – A Research Preamble

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Abstract

Preparations have been conducted as preamble to research at enhancing design principles in pile design. The study aims to obtain a reliable quantitative estimation on the time required after pile placement to achieve full capacity design for driven pile, to establish a time-strength relationship incorporating the effect of set-up on pile capacity, to determine a relatively low cost exploration-phase field testing at site in order to develop correlations between soil parameters and values associated with the set-up phenomenon, to reduce overall cost associated with pile foundation, and to optimize the existing pile foundation design procedure for Malaysian construction industry. Literature review conducted for this study reveals facts of implied over-design in driven pile foundations while most buildings in the country rest on piled footings. The study aims at verifying the possibility of reviewing design practice in order to minimize cost and to re-appoint appropriate capacity of driven piles as commonly found in construction industry.

Keywords: *pile-soil set-up, time-strength relationship, low-cost exploration-phase field test, set-up phenomenon*

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