Probabilistic Distribution of Ultimate Pile Capacity Based on Pile Load Test Data.

By

Anas Khaled Ali Bin Al-Sheikh Ali

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Universiti Teknology PETRONAS

Bandar Seri Iskandar

31750 Tronoh

Perak Darul Ridzuan

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ABSTRACT

Soil bearing capacity is crucial to any foundation design process. It defines the max stress capacity a ground can sustain prior failing. Field pile tests were developed to verify the design process. From maintained load tests to Pile Dynamic Analysis, which essentially impose a specific load & record the further movement of the pile. Using appropriate interpretation methods, the designer can determine the true/actual bearing capacity based on a more realistic data such as pile-soil settlement. Afterwards, using the probabilistic inverse method, a probability density of frictional & point bearing will be constructed to be used in Bayesian's statistical interpretation in order obtain the variation of Ultimate pile capacity along the boundaries of the tested piles. The probability distribution of ultimate pile capacity based on pile load test data will prove valuable in estimating design value of Ultimate bearing Capacity.

Chapter 1:

Introduction

1. Background:

The soil has been the point of interest by many civil engineers for it holds the safety of any structure around the globe. The necessity of determining the soil bearing capacity is extreme due to its vital role in resisting loads transferred from a superstructure like commercial buildings, bridges, skyscrapers and so on. Studies produced by geotechnical engineers along the history to estimate & predict the ground behavior to various stresses. Engineers like Terzaghi & Meyerhoff formulated a set of equations using soil parameters such as friction angle and cohesion in which were widely used among civil engineering platform. The paper will review different results data of soil report, pile lengths, and pile load test conducted & collected from an on-going-project as experimental data. By using results obtained from pile tests like Maintained load test, true bearing capacity will be estimated. Based on the finding, back calculation using probabilistic methods will evaluate point & frictional bearing to obtain the probability distribution of Ultimate bearing capacity of the area aiming to determine the variations of soil capacity. This will prove valuable in the design process.

2. Problem Statement

As in issue in which it concerns most of the civil society, a decision, which involves a foundation on the ground, must be chosen carefully because soil may deform at any moment due to change in lateral earth pressure developing excessive settlement may by many reasons failing. Engineers & designers sometimes tend to either over or underestimate soil bearing. This causes either economic disadvantages such as wasted pile length or damages to the structure in the future like excessive cracks developed by uneven settlements.

3. Objectives

i. Analyse and evaluate Experimental/True bearing capacity using Davisson's offset line method

ii. Using Probabilistic inverse & Bayesian's method, through Mathematica, to obtain the variation of frictional and point bearing capacities based on the true bearing capacities.

iii. Using Markov Chain Monte Carlo: Normal Distribution, variations of Ultimate bearing capacity are to be generated.

4. Scope of Study

Theoretical & Experimental Bearing Capacities using Meyerhof's & Davisson's respectively are to be evaluated. Then through probabilistic methods, probability distribution of Fs & Qb will be estimated using Probabilistic Inverse method: Gaussian's Model. From the results obtained, a Joint Probability Distribution can be plotted in a 3D graph. Lastly, using the space model formulated in Markov Chain Monte Carlo simulation: Normal Distribution, a histogram will be generated illustrating the probabilistic distribution of ultimate bearing capacities to be found on site. From the histogram, the design value of ultimate bearing capacity will be evaluated based on mean & standard deviation of the graph.

CHAPTER 2

Literature Review

1. Standard Penetration Test

In most cases of any subsoil investigation, the method of obtaining SPT numbers is widely used due to its effectiveness & its economic cost. Dropping a dead weight of a slide hammer from a specified height on a thick-walled sample collector tube in which has a specified weight and dimensions, the tube is injected then injected into the ground caused by the pressure of the hammer drop. This is called a blow. The number of blows is recorded for every 150mm, N. afterward, the recovery ratio is then measured, which is defined as the length of the sample obtained from the sampler penetration depth, to give a representation of the soil obtained. Sandy soils usually have low recovery ratio (<200/450) whereas cohesive soils have much higher recovery ratios (>200/450). Although the relative density of any soil can be obtained by analyzing an undisturbed soil sample, it is practically impossible to obtain an undisturbed soil sample. With the usage of SPT, relative density can be obtained with an undisturbed soil sample. For a more detailed report, N was recorded at each 75mm depth for both seating and test stages. In the case of a test termination, the depth achieved with the corresponding number of blows was recorded. Also, the sample collector now contains a soil sample that was preserved as a disturbed soil sample.

E.g.: $\frac{4lows}{75mm}\ \frac{7}{75}\ \frac{8}{75}\ \frac{11}{75}\ \frac{9}{75}\ \frac{12}{75}$

$$\because N = \frac{8+11+9+12}{75+75+75+75} = \frac{40blows}{300mm} = 40$$

The test is made into two stages. However, the initial penetration of the drive rod under its dead load is required to reach 450mm. Upon reaching it, the number of blows N is ignored and zeroed.

I) Seating drive stage:

In this stage, the drive assembly is penetrated under standard blows. Then it was driven until 150mm or 25 blows are reached, finishing this stage.

II) Test drive stage:

A further penetration of 300mm is needed, and the number of blows to reach such depth is recorded as Penetration Resistance, N. However, if the number of blows was maximized to 50 blows without reach the desired depth, the stage was omitted.

This means that for the first 150mm or 25 blows, the number of blows is not computed in penetration resistance. Afterwards, the number of blows to reach 300mm is recorded. If it reaches 50 blows, the depth is recorded.

E.g.: For 10.5m on BH2, N obtained as follows:

$$\frac{6blows}{75mm}\ \frac{9}{75}\ \frac{12}{75}\ \frac{15}{75}\ \frac{18}{75}\ \frac{5}{10}$$

Notice after the first two reading, the depth has reached 150mm, hence omitting N. afterward, Test drive starts, and stop in this case due to N=12+15+18=45-50=5Blows. Finally, the depth reached with five blows is recorded, which in this case is 10mm.

$$\because N = \frac{(12+15+18+5)blows}{(75+75+75+10)} = \frac{50}{235}$$

$$\text{and recovery ratio measured, } R/r = \frac{210}{450} \because \text{ Silty or clayey Soil}$$

III) SPT-N Correction

Although the raw value of N can be obtained, it is required to be corrected from any deficiencies associated with the type of equipment used. In this paper, an automate hammer was used thus +60% of N is N_{60}.

$$N_{60} = N * (1 + 60\%)$$

1.1 Correlation of SPT numbers:

In an effort of estimating some of the soil properties from such in-situ test, engineers and researchers have come up with equations correlating the standard penetration numbers obtained. From estimating friction angle to Ultimate bearing capacities, many equations were found which make use of SPN.

1.1.1 Drained Fiction angle, ϕ'

Das & Sobhi, 2014, stated a method of correlating the standard penetration numbers to friction angle:

- Introduced first by N. Schmertmann in 1975, the following correlation is improved by Kulhawy & Mayne in 1990:

$$\phi' = \tan^{-1}\left[\frac{N_{60}}{12.2 + 20.3\left(\frac{\sigma'_o}{p_a}\right)}\right]^{0.34}$$

Equation 1: Drained Angle of friction correlated with SPT N.

Where:

$$\sigma'_o = \text{Average pressure.} = \frac{\Sigma(\text{depth} * \text{unit wieght})}{\mathit{Final\ Depth}}$$

$$\boldsymbol{p_a} = \textbf{atomspheric Pressure}. (= \textbf{100 kPa}).$$

1.1.2 Undrained Shear Strength

While you always can conduct triaxial shear tests to obtain such results, an easier way would be using standard penetration test. Hettiarachchi & Brown , 2015, have written about obtaining Undrained shear strength through the use of SPT-N:

- Introduced by Hara et al. 1974:

$$\frac{\boldsymbol{c_u}}{\boldsymbol{p_a}} = 0.29(N_{60})^{0.72}$$

Equation 2: Undrained Shear Strength using SPT Numbers.

2. Ultimate Bearing Capacity

Any deep foundation should resist & transmit loads to soil safely. In some cases, the soil may not be able to sustain the load which creates a deformation leading to foundation failures. The ultimate bearing capacity is expressed as the sum of point-bearing & frictional bearing.

$$Q_u = Q_s + Q_b$$

Equation 3: Ultimate Bearing Capacity

Point-Bearing Capacity, Q_b

According to Das & Sobhan, Qp is defined as the capacity of a pile to resist the load at the pile point. The general expression for point bearing:

$$Qp = A_P q_P \text{ (kN)}$$

Equation 4: Point Bearing Capacity, $\boldsymbol{Qp}$.

Where:

A_P = area of the pile.

Over the years, theories have been developed for estimating q_P. To generalize the design of piling, methods tend to have general conditions. Meyerhof's equations and methods are going to be used to estimate the ultimate resistance:

a. Qp in Sand ($c' = 0$)

$$q_P \leq q_l$$

Where:

$$q_P = q' N_q^*$$

Equation 5: Unit Point Resistance, $\boldsymbol{q_P}$

$$q_l = 0.5 \left(100 \frac{kN}{m^2}\right) N_q^* \tan \phi'$$

Equation 6: Limiting Point Resistance, $\boldsymbol{q_l}$.

To get Effective stress, $q' = L * \gamma$ & $N_q^* = f(\phi')$

b. Qp in Clay ($\phi' = 0$)

$$q_P = N_c^* c_u$$

Equation 7: Unit point Resistance in Clay.

Where:

c_u = Undrained cohesion of Soil below tip of the pile.

$N_c^* = 9$.

c. Qp Using SPT Numbers

However, using SPT numbers are suitable to any project to demonstrate the estimating of Pile capacity through the use of Meyerhof's equation in a granular soil:

$$q_P = 0.4 \mathrm{p_a N_{60}} \frac{\mathrm{L}}{\mathrm{D}}$$

Equation 8: Point Bearing Stress using SPT N.

Where:

$\mathrm{p_a}$ = Atomspheric pressure. $\left(= 100 \frac{\mathrm{kn}}{\mathrm{m^2}}\right)$

$\overline{\mathrm{N}}_{60}$ = Average number of standard penetration test above 5D and below 10D.

L = Length of the pile.

D = Diameter or Width of the pile.

Skin Frictional Bearing Capacity, Q_s

For piles, stress will be developed at the skin of the pile due to surrounding soil at that specific depth. This stress had to be accounted for when estimating the length of the piles. The general expression for Qs:

$$Q_s = \mathrm{pL} f_{av}$$

Equation 9: Frictional Bearing Capacity, $\boldsymbol{Q_s}$.

Where:

p = Perimeter of the pile = 4B.

L = Length of the pile.

f_{av} = Average unit frictional resistance.

a. $\mathrm{Q_s}$ in Sand $(c' = 0)$

Meyerhof's stated an expression for f_{av} in sand

$$f_{av} = K \bar{\sigma}_o' \tan \delta'$$

Equation 10: Average Frictional Resistance in Sand.

Where:

$K = (1.5)$[1] For Precast Concrete.

$\sigma'_o = \frac{\sum c_{u(i)} L_{(i)}}{L_T}$. Mean Effective stress.

$\delta' = 0.8(\phi')$. Soil − Pile friction.

Q_s in Clay ($\phi' = 0$)

In this situation, three methods have been introduced to estimate f_{av}. However, only λ method will be revieed:

b. λ Method: introduced by Vijayvergiya & Focht (1972)

$$f_{av} = \lambda(\bar{\sigma}'_o + 2c_u)$$

Table 1: Variations of Lambda concerning length.

Embedment Length, L (m)	λ
0	0.5
5	0.336
10	0.245
15	0.200
20	0.173
25	0.150
30	0.136
35	0.132
40	0.127
50	0.118
60	0.113
70	0.110
80	0.110
90	0.110

Equation 11: Average Frictional Resistance, Lambda.

[1] (Braja, 2013 p.570)

Where:

$\boldsymbol{\lambda = f(L)}$ Can be obtained **Error! Reference source not found.**

3) Q_s using SPT Numbers

Most cases, Soil investigation, is executed prior designing the foundation. Standard Penetration Number is the number of blows per 300mm. Then, compute it in the following expression to obtain the average friction.

$$f_{av} = 0.01 p_a (\overline{N}_{60})$$

Equation 12: Average Friction of Pile skin using SPT-N, $\boldsymbol{f_{av}}$.

3. Maintained Load Test

Piles are tested for their ultimate load behavior. In every country, standards are there to guide engineers in achieving the allowable failures such as settlement…etc. Almost after any piling installing, we are required by JKR Standard to ensure a maintained load test is to be executed on a random pile by the contractor.

JKR has this procedure, in details, standardized to ensure the maximum potential from MLT:

After installation & setting the equipment, the test load shall be at a rate of either 12.5% or 25% load increment per hour until it reaches twice its working load. Afterwards, Full Load Test (FLT)[2] is applied. The test shall be carried out for 24hrs maintaining a full load representing the continuous load on the pile in the future.

When 24 hours FLT has passed on the pile, unloading shall be in 4 hours for the first cycle & 8hrs for the second cycle, Again, according to JKR Specification, 2014 "A time-settlement graph shall be plotted to indicate when the rate of settlement of 0.05 mm in 15 minutes is reached" Hence, readings is taken after 15 minutes from the start

[2] "The Full Test Load (FTL) on a pile shall be twice the Working Load (WL) noted on the Drawings" JKR Standard (2014, p.89)

of the test. However, after a couple of hours, they can be taken after 30 minutes instead of 15.JKR Specs, 2014 p.84, states "working Drawings showing the method and equipment he proposes to use in the performance of the load test and the measurement of settlements" which requires the contractor to present a clear, detailed method statement for the load test.

Schedule

a. Quarter's Site

i. 25% Load is imposed in the first hour
ii. 50% Load in the 2nd hour. And so on until 200%.
iii. With a rate of 25%/hr to reach 200% of the working load (1000kN), thus the first 8 hours, loading increment is at a rate of = 125kN/Hr. Readings are taken every 15 minutes from dial gauges to compute the average settlement. Thus, the first 8 hours, loading increment at a rate of= 125kN/hr
iv. After reaching the maximum ultimate load, the load is maintained for 24, Full Load Test. Readings are taken after 30 minutes instead of 15mins.
v. Unloading after 24hrs passed with a rate of 50% of ultimate load per hour. Thus the rate of unloading is 250kN per hour, hence after 4 hours unloading stops & the test as well.
vi. Total hours = 36 hours.

b. Clinic Foundation:

i. 25% Load is imposed in the first 2 hours
ii. 50% Load in the 3rd-4th hour. And so on until 200%.
iii. With a rate of 25%/2hrs to reach 200% of the working load (1000kN), thus the first 16 hours, loading increment is at a rate of = 62.5kN/Hr. Readings are taken every 15 minutes from dial gauges to compute the average settlement. Thus, the first 8 hours, loading increment at a rate of= 62.5kN/Hr /hr
iv. After reaching the maximum ultimate load, the load is maintained for 24, Full Load Test. Readings are taken after 30 minutes instead of 15mins.
v. Unloading after 24hrs passed with a rate of 50% of ultimate load per hour, thus the rate of unloading is 125kN/hr, hence after 8 hours unloading stops & the

test as well.

vi. Total hours = 48 hours.

Acceptance Criteria

The piles tested may be categorized as failed piles if one the following requirements have not been satisfied:

I) At the working load (100%), the settlement shall not exceed either 19mm or 5% of pile Width.

II) The rate of the settlement shall not exceed 0.5%/hr.

III) The settlement at the ultimate load shall not exceed either 38mm or 10% of pile width.

IV) The setback, residual settlement, resulted after unloading shall not exceed 6.5mm.

V) Davisson's Pile Load Test Interpretation

Selected piles in a site should be subjected to testing to ensure the safety of foundation design predictions. Pile tests can be static load tests or dynamic load tests dependant on the requirement. After completion of pile test, a "Load vs. Settlement" graph may be plotted for further analysis. Various methods for analyzing static pile load test results have been developed such as De Beer's or Chin's methods. However, in the paper, Davisson's (1973) method will be used. Based on

$$s_u = \frac{Q_u L}{A_P E_P} + 0.012B_r + \frac{0.1B}{B_r}$$

Equation 13: Davisson's Offset Limit Line.

Where:

- $Q_u = Load\ Applied. (kN)$
- $L = Length\ of\ Pile. (m)$
- $A_P = Area\ of\ Pile. (m^2)$
- $E_P = 20\ MPa$
- $B_r = Referenc\ width = 0.3m = 300mm.$
- $B = Width\ of\ the\ Pile.$

Davisson stated that the intersection between **Error! Reference source not found.** and load-settlement curve is ultimate load failure, Q_u.

VI) Improved Soil Parameters

Due to the percentage of uncertainty resulted from the known soil parameters, and other factors such as variability of construction, the soil parameters may be inferred as probability distributions. Different Methods have been developed for back-analysis of soil parameters, For instance, Least Square Method, LSM (Xu and Zheng 2001), Max. Likelihood Method, MLM (Ledesma et al. 1996), Bayesian's method (Zhang et al. 2010a), and many more. Probabilistic methods can determine diverse sets of stability parameters with uncertainty.

Probabilistic Inverse Method

Assume a function of f which maps parameters into theoretical quantity such that $d = f(m)$ where $d = \{d^i, \ldots, d^{ND}\}$ and $m = \{m^i, \ldots, m^{NM}\}$, the goal is to determine m given m. In the context of pile-load-test, to determine f_S, q_b where Q_u can be obtained from interpretation of the results.

Data Space

From the results, d_{obs}, the probability density model in which describes experimental uncertainty can be written as follows:

$$\rho_D(d) = Exp\left(-\frac{1}{2}\sum\left(\frac{d^i - d_{obs}}{\sigma^i}\right)^2\right)$$

Equation 14: Experimental Uncertainty.

Model Space

In a typical problem, model parameters which have a complex probability distribution over the model space. The probability density is $\rho_M(m)$. Suppose that the joint probability density function $\rho(m, d)$. And $d = f(m)$, then the conditional probability density function,

$$\sigma_M(m) = \rho_{M|d(m)}\big(m|d = f(m)\big)$$

Can be re-written as follows,

$$\sigma_M(m) = k\rho_M(m)\frac{\rho_D(d)}{D(d)}|_{d=f(m)}$$

Where k is a normalizing factor.

Bayesian's interpretation

The Ultimate bearing capacity is given in Equation 3: Ultimate Bearing CapacityEquation 3. Since the pile dimensions are known, the model space $m = (f_S, q_b)$. The probability density model to describe experimental uncertainty, Equation 14, is formed using theoretical model $d = f(m)$. The joint probability density after $d_{obs}, \sigma_M(m) = \sigma_M(f_S, q_b)$. Prior knowledge can be incorporated in $\rho_M(m) = \rho_M(f_S, q_b)$. The effect of prior knowledge on ultimate pile capacity can be investigated using various forms of density distribution.

$$\sigma_M(f_S, q_b) = \int_{-\infty}^{\infty} \sigma_M(f_S, q_b)\, df_S$$

Equation 15: Joint Probability Distribution

These methods are to be used in a Mathematica alpha <u>to</u> get accurate results with plotted graphs indicating variations of bearing capacity. The program will be in the appendix for further illustrations.

Markov Chain Monte Carlo Normal Distribution Simulation Sampling

For Ultimate bearing variations, MCMC Normal distribution is to be used. The concept is to generate sampling points over the space model by controlled random walk. In Markov Chain Approach, the sequence of random variables X_n at each time t . The next state X_t is sampled from a distribution $P(X_{t+1}|X_t)$ that depends on the state at time t. thus the expectation is approximated as:

$$\hat{\mu} = \frac{1}{n}\sum_{t=1}^{n} g(X_i)$$

Similar to Monte Carlo Simulation, if sufficient numbers of sampling points are obtained, then approximation to the expected value is evaluated.

MCMC has general rules to be followed:

I) A proposal Markov Chain Rule Expressed by a transition Kernel $q(x, y)$

II) An accept/reject rule which accepts/rejects a newly proposed $Y_K = q(X_K)$ where X_K is recently accepted a random variable, and

III) Stopping rule.

The following is a basic MCMC Algorithm

I) Draw X_0 as the initial state.

II) Do i = 0 to number of sampling points.

III) Obtain proposed sample $Y_{t+1} = q(X_t, Y)$

IV) $X_{t+1} = \begin{cases} Y_{t+1} \; with \; probability \; A(X_t, Y) \\ \quad Otherwise \; X_t \end{cases}$

Chapter 3:

Methodology/Project Work

The research method is to collect an existing data related to this paper throughout results obtained from a structure built in Seri Iskandar, Malaysia. Afterward, from the tests done to the pile, we can estimate the experimental soil properties concerning friction angle and cohesion. For piles, the method of installing was Jack-in due to noise pollution. The information used regarding experimental data is taken from the project I was working on during my internship, Klinik Keshihataan 3 located in 32600 Bota, Perak. Malaysia. (4.369444, 100.953063)

1. Data Capture

The paper is an analysis based paper. Hence the information gathered is crucial. The project KK3 is located in Seri Iskandar near JKR. As a former employee of JKR, the data gathered was directly from JKR. The data gathered are as following:

i. **Soil tests results:**

As the company assigned a Geotechnical expertise to perform field & Laboratory tests on the soil found in the project's site. SPT-N was also determined in this phase.

ii. **Piles locations & Length:**

It is crucial to obtain the lengths & the locations as well of the selected piles to analyze accurately.

iii. **Maintained Load Test results**:

The author personally monitored the pile tests and recorded the settlements over time.

2. Piling

The piles were injected using Jack-in piling machine with a certain pressure dependant on the load transmitted from the columns. The method of determining the injecting pressure & as well as the machine information will be in APPENDIX

3. Ultimate Bearing Capacity Estimation

Theoretical

The paper will attempt to estimate the theoretical bearing capacity based on the soil report obtained from JKR. The equations used in determining Qu are shown in page 14.

Experimental

The values of Qu experimental will be determined by using Davisson's Offset Method as shown on Page.19

4. Tools/Software

In the process of this research, the data captured were saved and analyzed using Wolfram Alpha Mathematica. For documentation purposes, Microsoft Office was regularly used due to Word & Excel in both writing and creating graphs and functions.

Chapter 4:

Results & Discussion

1. Soil Condition of Sites

For simplicity, the paper will consider only the ground conditions underneath the selected piles. For the selection of pile, it was ensured to select a pile with a soft ground to stimulate the worst settlement in which might occur. The following is a summary of both foundations selected piles. All piles are 250mmx250mm

a. Quarter's Pile:

i. Laboratory Values:

Borehole No.	Final Depth (m)	ω_{avg} (%)	Density $\left({}^{g}/_{cm^3}\right)$			Unit $\left(\frac{kN}{m^3}\right)$		Atterberg Limits				Aggregate			Triaxial Test	
			ρ	ρ_d	S_G	γ	γ_d	LL	PL	PI	LS	SA	SI	CL	ϕ'	$c'\left(\frac{kN}{m^2}\right)$
BH3	18	36	1.98	1.61	2.76	19.42	15.7	73	36	37	16	5	62	33	0	64

Table 2: Soil properties of Quarter's Pile.

ii. Field Values:

Scale (m)	Description	N	R/r (%)
0.00	Medium Brown Silty Sand		270/450
1.50	Very stiff light brown silty fine Sand	17	270/450
3.00	Very Dense light grey silty fine Sand.	44	330/450
4.50	Very Dense light grey silty fine Sand.	19	300/450
6.00	Stiff light grey clayey Sand	13	340/450
7.50	Soft medium red to light grey silty Clay	4	370/450
9.00	Stiff light brown to brown clayey Silt		700/700
9.70	Hard medium brown clayey Silt	39	300/450

10.5	Hard medium brown streaked purple clayey Silt	31	330/450
12	Hard medium molted light grey clayey Silt	50	290/450
13.5	Hard medium molted light grey clayey Silt	50	330/450
15	Hard medium brown clayey Silt	50	250/450
16.5	Hard medium brown clayey Silt	50	300/450
18	Hard medium brown clayey Silt	50	290/450

Table 3: Soil layers in Quarter's.

b. **<u>Clinic Foundation:</u>**

i. Laboratory Values:

Borehole No.	Final Depth (m)	ω_{avg} (%)	Density $\left({}^{g}/{}_{cm^3}\right)$			Unit $\left(\frac{kN}{m^3}\right)$		Atterberg Limits				Aggregate			Triaxial Test	
			ρ	ρ_d	S_G	γ	γ_d	LL	PL	PI	LS	SA	SI	CL	ϕ	$c\left(\frac{kN}{m^2}\right)$
BH6	29	27	1.71	1.1	2.54	16.73	10.265	51	24	28	15	70	25	4	28	0

Table 4: Soil properties of Clinic Pile.

ii. Field Values:

Scale (m)	**Description**	**N**	**R/r (%)**
0.00	Medium Brown Clayey Silt		270/450
1.50	stiff medium brown silty sandy Clay	10	270/450
3.00	Medium brown silty sandy Clay	5	130/450
4.50	Medium Stiff Medium brown silty clayey Silt.	5	300/450
6.00	Medium Stiff medium brown to medium grey silty clayey Sand	4	330/450
7.50	Medium stiff medium brown silty clayey Sand	6	300/450
9.00	Medium Stiff medium brown silty clayey Sand	5	700/700
10.50	Medium stiff medium red yellow dappled silty clay	5	300/450
12.00	Very stiff medium brown silty clay	25	350/450
13.50	Stiff medium brown clayey silt	11	330/450

15.00	Very stiff medium brown clayey silt	20	430/450
16.50	Stiff medium brown to dark brown clayey silt	12	430/450
18.00	Hard dark brown clayey silt	50	300/450
19.50	Hard medium brown streaked purple clayey Silt	50	330/450
21.00	Hard medium brown streaked white clayey silt	42	360/450
22.50	Hard medium yellow clayey silt	50	190/450
24.00	Hard medium yellow clayey silt	50	
25.5	Hard dark brown clayey Silt with traces of fine sand	50	150/450
27.00	Hard medium brown clayey silt with traces of sand	50	160/450
28.5	Hard medium brown clayey silt with traces of sand	50	150/450

Table 5: Soil Layers in Clinic.

2. Theoretical Ultimate Capacity

The results of calculations shown here are to be put in APPENDIX (C)

Quarter's Pile

The pile length, $L = 15$m. The theoretical value estimated for Ultimate bearing capacity is, $Q_{u_{C1/8}{}_{(2)}} = 612\ kN$.

Clinic's Pile

The pile length, $L = 17.7$m. The theoretical value estimated for Ultimate bearing capacity is, $Q_{F/1'}{}_{(1)avg} = 766\ kN$.

3. Maintained Load Test Results:

Quarter's Pile

Settlement vs. Load

Load	Settlement	Load	Settlement	Load	Settlement
12.5	0.775	100	9.99	100	10.34
12.5	0.765	100	10.0025	100	10.345
25	1.4825	100	10.0205	100	10.348

25	1.52	100	10.05	100	10.35
37.5	2.2575	100	10.05	100	10.3525
37.5	2.3175	100	10.095	100	10.3525
37.5	2.33	100	10.115	75	9.9505
50	3.52	100	10.1275	75	9.7875
50	3.6375	100	10.1525	50	8.5275
50	3.6725	100	10.1525	50	8.3475
62.5	4.6925	100	10.1575	25	6.205
62.5	4.8175	100	10.16	25	6.095
62.5	4.8775	100	10.17	0	2.5225
75	5.4425	100	10.175	0	2.275
75	6.05	100	10.205	0	2.1356
87.5	6.1225	100	10.205		
87.5	6.87	100	10.2075		
87.5	7.0325	100	10.2075		
100	7.0895	100	10.21		
100	9.0425	100	10.21		
100	9.36	100	10.24		
100	9.5125	100	10.285		
100	9.5125	100	10.2875		
100	9.64	100	10.243		
100	9.6975	100	10.25		
100	9.7675	100	10.2575		
100	9.845	100	10.2775		
100	9.905	100	10.28		
100	9.9325	100	10.288		
100	9.945	100	10.303		
100	9.957	100	10.3125		
100	9.9725	100	10.3075		

Table 6: Settlement vs. Load for Quarter Site.

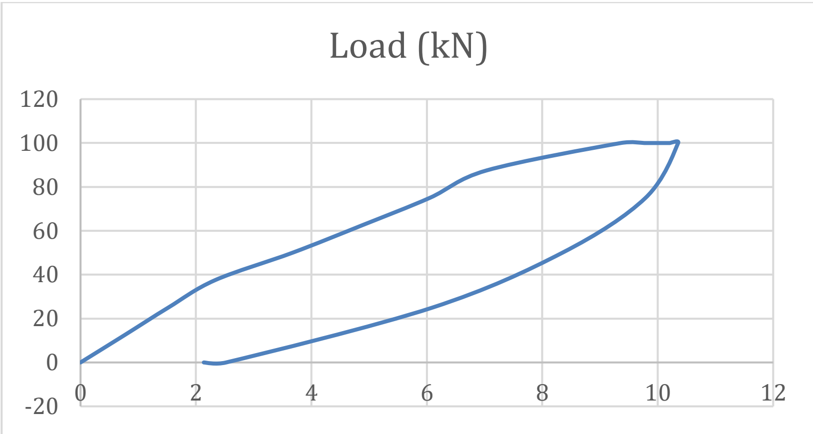

Figure 1: Settlement vs. Load plotted graph.

Clinic's Pile:

Settlement vs. Load

Load	Settlement	Load	Settlement	Load	Settlement
0	0	87.5	3.685	100	5.425
12.5	0.0275	100	4.8725	100	5.455
12.5	0.0275	100	4.9124	100	5.485
12.5	0.035	100	4.9225	100	5.4975
25	0.1225	100	4.9725	100	5.51
37.5	0.138	100	4.9825	100	5.5275
37.5	0.138	100	5.03	100	5.535
50	0.4	100	5.08	100	5.54
50	0.4125	100	5.11	100	5.545
50	0.465	100	5.12	100	5.545
50	0.475	100	5.14	100	5.5475
62.5	1.2	100	5.17	100	5.5475
62.5	1.3375	100	5.1875	75	5.155
62.5	1.375	100	5.205	75	5.13
75	2.33	100	5.2475	75	5.1225
75	2.3725	100	5.2775	75	5.1125
75	2.3975	100	5.2975	50	3.8275
75	2.4475	100	5.3475	50	3.78
87.5	3.535	100	5.385	50	3.74
87.5	3.595	100	5.4	25	1.84

Table 7: Settlement vs. Load. Clinic Site.

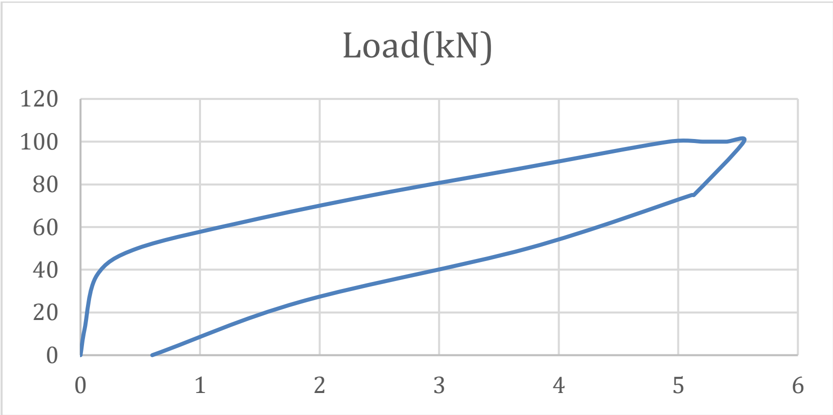

Figure 2: Settlement vs. Load Graph. Clinic site.

4. Interpretation of ML Results:

As mentioned in Davisson's Pile Load Test Interpretation section, the analysis revealed the following:

Quarter's Pile

It appears that the Quarter pile has experienced much higher settlement than the clinic's pile which indicates lower capacity. The Davisson's Ultimate Bearing Capacity, ${Q_{ult}}_{Quarter} = 610kN$

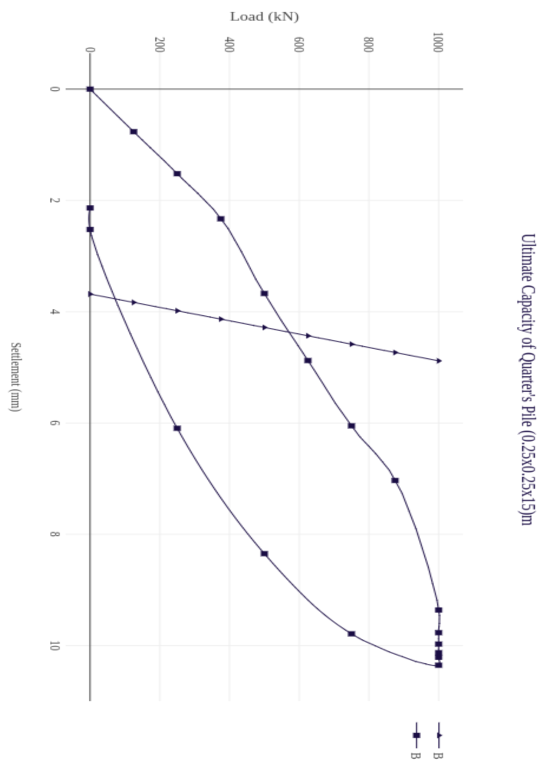

Figure 3: Pile Load test results, Quarters.

$$s_u = \frac{Q_u L}{A_P E_P} + 0.012B_r + \frac{0.1B}{B_r}$$

$Q_u = Ultimate\ Load.(N)$

$L = 15\ (m)$

$A = 250 \times 250.(mm^2)$

$E = 20\ MPa$

$B_r = Referenc\ width = 0.3m = 300mm.$

$B = 250mm.$

$$\Delta = \frac{Q_u * 1000 * 15 * 1000mm}{0.25 * 0.25 * 1000mm * 200 * 10^5} + 0.012(300mm) + \frac{0.1(250mm)}{300mm}$$

$$s_u = 3.683 + 0.012Q_u$$

Substituting Q_u will generate a line graph with the following points

$Q_u(N)$	0	12.5	25	37.5	50	62.5	75	87.5	100
$s_u(mm)$	0	3.833	3.983	4.133	4.283	4.433	4.583	4.733	4.883

As noticed after plotting Davisson's offset limit, the ultimate bearing capacity,

- $Q_u = 59Tons = 600$

Clinic's Pile

While Quarter pile max. The settlement is 10.35mm, the clinic pile, on the other hand, has only 5mm as max. Settlement.

The Davisson's Ultimate Bearing Capacity, $Q_{ult_{Quarter}} = 1015kN$.

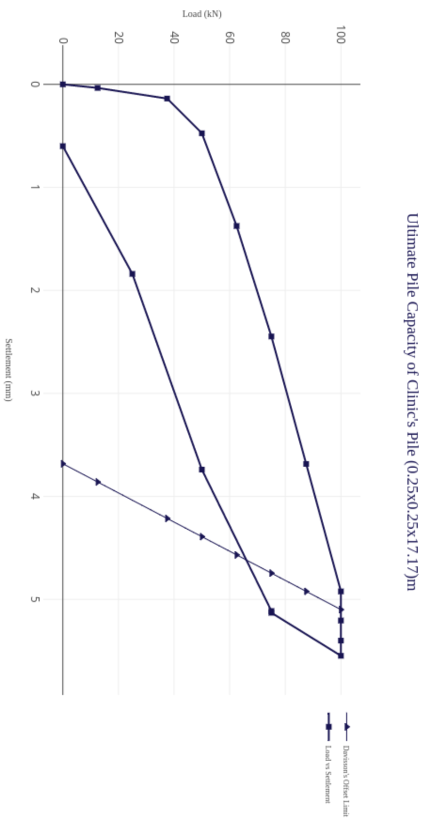

Figure 4: Pile load test results, Clinic.

$$s_u = \frac{Q_u L}{A_P E_P} + 0.012B_r + \frac{0.1B}{B_r}$$

$$\Delta = \frac{Q_u * 1000 * 17.7 * 1000mm}{0.25 * 0.25 * 1000mm * 200 * 10^5} + 0.012(300mm) + \frac{0.1(250mm)}{300mm}$$

$$s_u = 3.683 + 0.01416Q_u$$

Q_u	0	12.5	37.5	50	62.5	75	87.5	100
S_u	0	0.177	0.531	0.708	0.885	1.062	1.239	1.416

$\because$ From the graph, the intersection point yields an ultimate Capacity, $Q_u = 1015\ kN$.

5. Ultimate Capacity Variations

As one of the objective, the probabilistic methods used in Wolfram alpha determined the distribution of frictional bearing along the site as well as the point bearing.

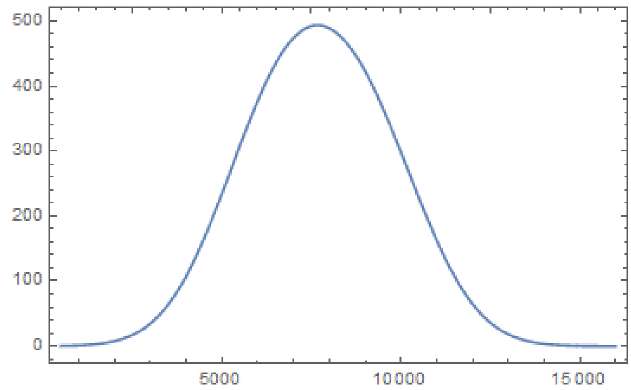

Figure 5: Probability Distribution of End Bearing stress.

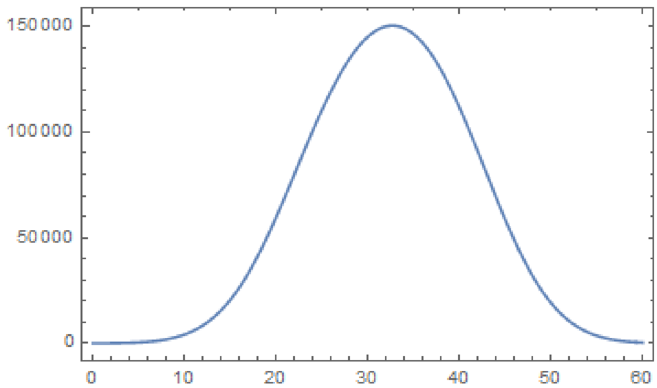

Figure 6: Probability Distribution of Frictional Bearing stress.

From the above graphs, we can notice the variations in frictional & point bearing. Using both results in to generate a 3D plot of Joint Probability Distribution.

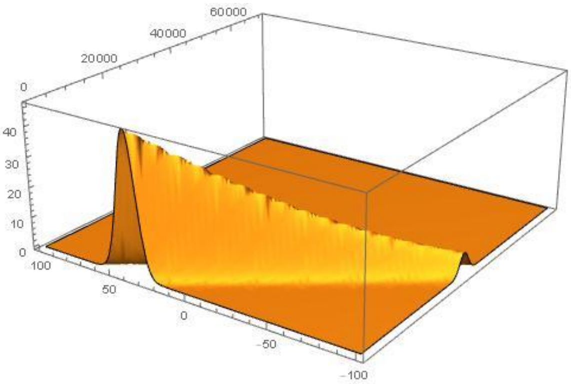

Figure 7:Joint Probability Distribution of Fs & Qb .

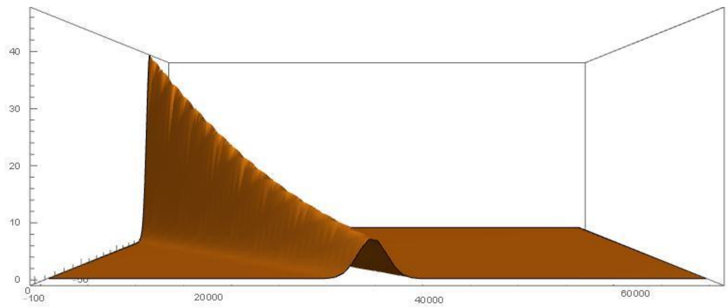

Figure 8: Joint Probability Distribution of Fs & Qb. Front View. Y: Frictional Bearing; X: End Bearing.

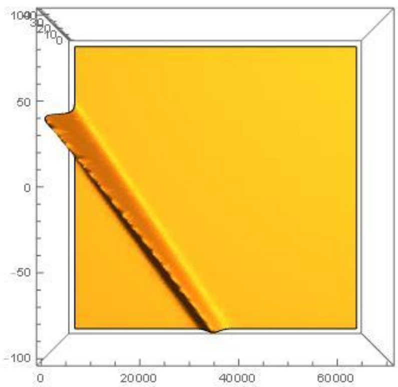

Figure 9: Joint Probability Distribution, Top view.

From the 3D plot, we can estimate the variations of Ultimate bearing capacity by Plotting Fs and Qs, any point's bearing capacity could be determined.

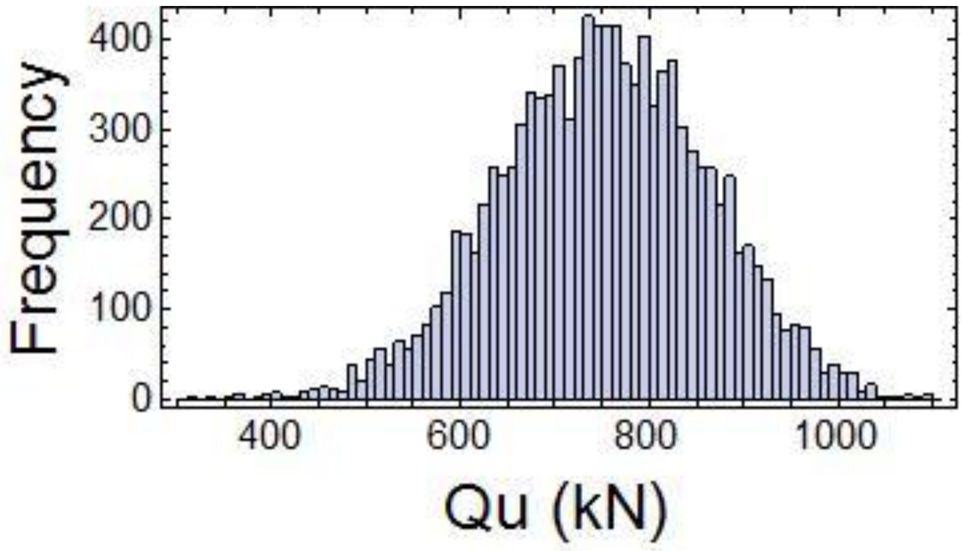

Figure 10: MCMC Histogram illustrating Variations of ultimate bearing capacity in site.

The histogram above illustrates the minimum & the maximum bearing capacity and its probability of occurring. The theoretical values obtained from the equations in page.14 & page.15 were 612 kN & 700.357 kN for the quarter's pile & the clinic's foundation which fall nearly at the median location. However, the experimental/True values of bearing capacity fall nearly in the range of maximum & minimum bearing capacities that would be encountered at the site. The mean, median and standard deviation of the histogram are calculated. 752.0137 kN. 753.24 kN. 110.294. Respectively. As noticed, the distance from the minimum capacity(=600kN) to the median is nearly the amount of Median – Standard deviation. Whereas Maximum True capacity (=1015kN) is relatively higher then median & standard deviation summed.

Chapter 5:

Conclusion

The experimental data obtained from maintained load test proved useful in the probabilistic methods. The variation of bearing capacities in the site is a more accurate illustration of the possibility of encountering such values. We conclude that if this method is to be used in the designing phase rather than the construction phase, it will reduce uncertainties which directly cut cost of wasted pile length as reduce the factor of safety. Using the histogram generated, designer will have a clearer & more accurate representation of soil capacity rather than assumptions which almost always generate wastage in the name of safety. Since the uncertainties are reduced upon obtaining variations of Bearing capacity along the site, the design value of ultimate bearing capacity can be take as follows:

$$Q_{u,Deisng} = Mean \pm Standard\ Deviation$$

Using the above formula, designers can optimize their design process regarding safety & cost. The probabilistic methods have generated two graphs illustrating values of frictional & point bearing & their possibilities. The maintained load test interpretation indicated that the piles used for testing had nearly the max & min values of bearing capacities.

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JACK-IN PILE (M) SDN BHD. *M* .

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Appendix

- Drained Angle of Friction correlated with SPT N.

$$\phi' = \tan^{-1}\left[\frac{N_{60}}{12.2 + 20.3\left(\frac{\sigma'_o}{p_a}\right)}\right]^{0.34}$$

Where:

$p_a = \text{atomspheric Pressure.} (= 100 \text{ kPa}).$

$\sigma'_o = \text{Average pressure.} = \text{depth} * \text{unit wieght.}$

- Equation 2: Undrained Shear Strength using SPT Numbers.

$$\frac{\boldsymbol{c_u}}{\boldsymbol{p_a}} = 0.29(N_{60})^{0.72}$$

- Ultimate Bearing Capacity

$$Q_u = \mathrm{Q_s} + \mathrm{Q_p}$$

Equation 4: Point Bearing Capacity, $\boldsymbol{Qp}$.

$$Qp = \mathrm{A_P q_P}\ (\mathrm{kN})$$

where:

A_P = area of the pile.

- Equation 5: Unit Point Resistance, $\boldsymbol{q_P}$

$$q_P = \mathrm{q' N_q^*}$$

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$$q_l = 0.5\left(100\,\frac{\mathrm{kN}}{\mathrm{m^2}}\right)\mathrm{N_q^*}\tan\phi'$$

- Equation 7: Unit point Resistance in Clay.

$$q_P = \mathrm{N_c^* c_u}$$

Where:

c_u = Undrained cohesion of Soil below tip of the pile.

$N_c^* = 9$.

- Equation 8: Point Bearing Stress using SPT N.

$$q_P = 0.4\mathrm{p_a N_{60}}\frac{\mathrm{L}}{\mathrm{D}}$$

Where:

$\mathrm{p_a}$ = Atomspheric pressure. $\left(= 100\,\frac{\mathrm{kn}}{\mathrm{m^2}}\right)$

$\overline{\mathrm{N}}_{60}$ = Average number of standard penetration test above 5D and below 10D.

L = Length of the pile.

D = Diameter or Width of the pile.

- Frictional Bearing Capacity, $\boldsymbol{Q_s}$.

$$Q_s = \mathrm{pL}f_{av}$$

Where:

p = Perimeter of the pile = 4B.

L = Length of the pile.

f_{av} = Average unit frictional resistance.

- Average Frictional Resistance in Sand.

$$f_{av} = K\bar{\sigma}_o' \tan\delta'$$

Where:

$K = (1.5)$ For Precast Concrete.

$\sigma_o' = \frac{\sum c_{u(i)} L_{(i)}}{L_T}$. Mean Effective stress.

$\delta' = 0.8(\phi')$. Soil − Pile friction.

- Average Friction of Pile skin using SPT-N, $\boldsymbol{f_{av}}$.

$$f_{av} = \lambda(\bar{\sigma}_o' + 2c_u)$$

Where:

$\lambda = f(L)$

- Average Frictional Resistance, Lambda.

$$f_{av} = 0.01p_a(\bar{N}_{60})$$

- Load in Tons given by Piling Company.

$$Q_{Tns} = \frac{A * n * P}{1000}$$

Where:

Q_{Tns} = Load in Tons.

A = Area of a Cylinder = $\frac{\pi D^2}{4}$ (cm^2).

P = Coversion Factor = Pressure in MPa $(\frac{kN}{m^2}) \times 10.2 \frac{kg}{cm^2}$.

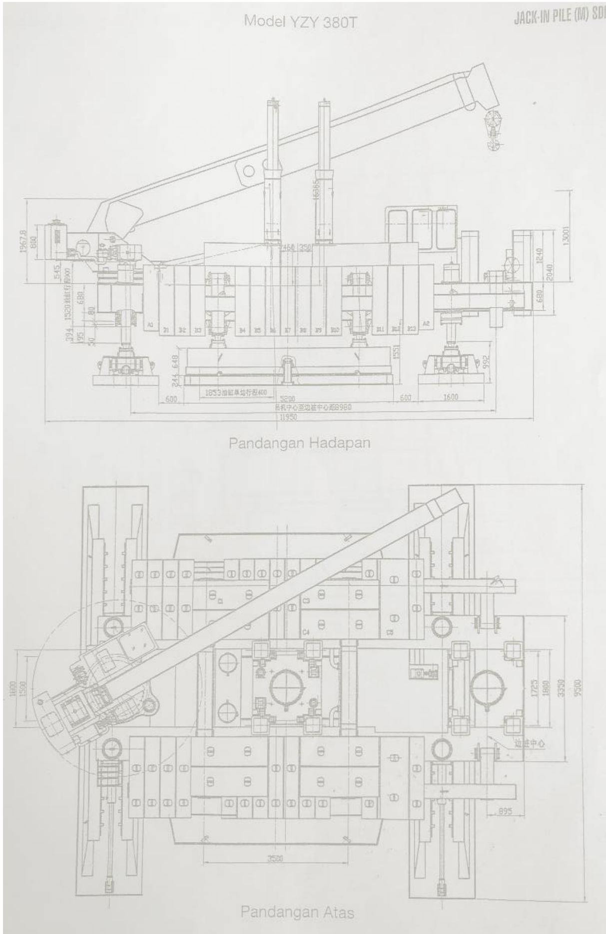
n = Number of Cylinders used.

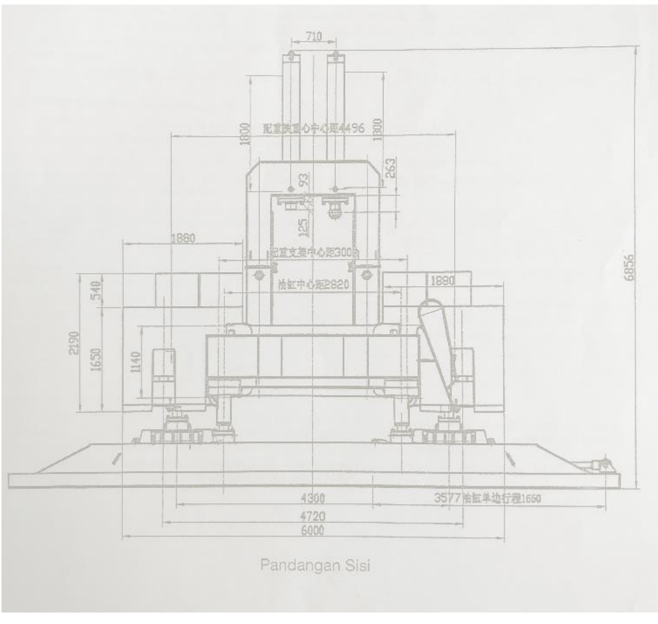
APPENDIXES

APPENDIX A:

PILING

Machine


Model YZY 380T
Pandangan Hadapan
Pandangan Atas


710
1880
1890
2190
1650
1140
540
6856
4300
4720
6000
Pandangan Sisi

Jack in Force

In this project, piles are injected with a Jack-In Machine. Dependent on the column load of each piling point, the pressure value to be exerted on a pile is then calculated respectively. The load in tons can be estimated using the following expression:

$$Q_{Tns} = \frac{A * n * P}{1000}$$

Equation 16: Load in Tons given by Piling Company.

Where:

Q_{Tns} = Load in Tons.

A = Area of a Cylinder $= \frac{\pi D^2}{4}$ (cm^2).

P = Coversion Factor = Pressure in MPa $\left(\frac{kN}{m^2}\right) \times 10.2\,\frac{kg}{cm^2}$.

n = Number of Cylinders used.

This is the equation used by the piling company. Area of a cylinder is known since $D = 180mm = 18cm$. n Varies dependent on the soil. When the soil is soft, two cylinders are sufficient enough to jack-in. on the other hand, when soil is hard to penetrate, additional two cylinders are introduced to the process. n total of 4.

An example: Pressure in MPa = 1 MPa

As mentioned earlier, $P =$ Pressure in MPa $* 10.2\,\frac{kg}{cm^2}$ $\because P = 1\text{ MPa} * 10.2\,\frac{kg}{cm^2}$

Using:

$$Q_{Tns} = \frac{A * n * P}{1000}$$

Where:

$$A = \frac{\pi D^2}{4} = \frac{\pi 18^2}{4} = 254.469\text{ cm}^2$$

If $n = 2$, $\because Q_{Tns} = \frac{A*n*P}{1000} = \frac{254.469\text{ cm}^2 * 2 * 1\text{ MPa} * 10.2\frac{kg}{cm^2}}{1000} = 5.19\text{ Tons}$.

If $n = 4, \because Q_{Tns} = \frac{A * n * P}{1000} = \frac{254.469\ \text{cm}^2 * 4 * 1\ \text{MPa} * 10.2 \frac{\text{kg}}{cm^2}}{1000} = 10.38\ \text{Tons}$.

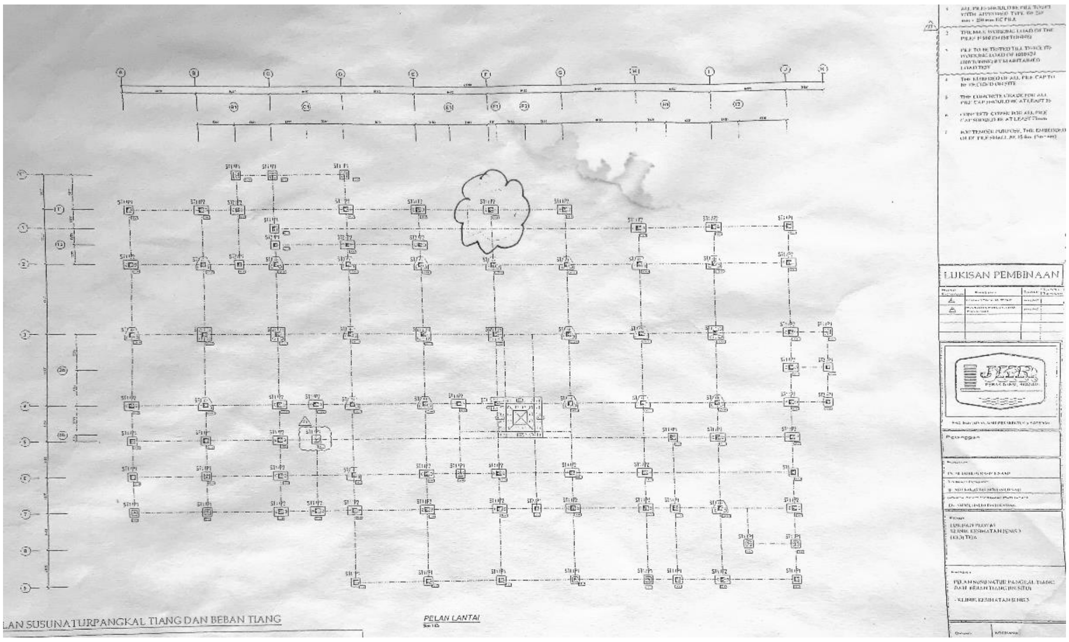
To simplify the procedure, the piling company has a prepared table:

Tekanan Hidraulik (Mpa)	Daya Tekan (Tan)			Tekanan Hidraulik (Mpa)	Daya Tekan (Tan)		
	2 nos x 180mm Dia. Silinder	2 nos x 180mm Dia. Silinder	Gabungan 4 nos x 180mm Dia. Silinder		2 nos x 180mm Dia. Silinder	2 nos x 180mm Dia. Silinder	Gabungan 4 nos x 180mm Dia. Silinder
1.0	5.19	5.19	10.38	13.0	67.49	67.49	134.99
1.5	7.79	7.79	15.58	13.5	70.09	70.09	140.18
2.0	10.38	10.38	20.77	14.0	72.69	72.69	145.37
2.5	12.98	12.98	25.96	14.5	75.28	75.28	150.56
3.0	15.58	15.58	31.15	15.0	77.88	77.88	155.76
3.5	18.17	18.17	36.34	15.5	80.47	80.47	160.95
4.0	20.77	20.77	41.53	16.0	83.07	83.07	166.14
4.5	23.36	23.36	46.73	16.5	85.67	85.67	171.33
5.0	25.96	25.96	51.92	17.0	88.26	88.26	176.52
5.5	28.56	28.56	57.11	17.5	90.86	90.86	181.71
6.0	31.15	31.15	62.30	18.0	93.45	93.45	186.91
6.5	33.75	33.75	67.49	18.5	96.05	96.05	192.10
7.0	36.34	36.34	72.69	19.0	98.64	98.64	197.29
7.5	38.94	38.94	77.88	19.5	101.24	101.24	202.48
8.0	41.53	41.53	83.07	20.0	103.84	103.84	207.67
8.5	44.13	44.13	88.26	20.5	106.43	106.43	212.87
9.0	46.73	46.73	93.45	21.0	109.03	109.03	218.06
9.5	49.32	49.32	98.64	21.5	111.62	111.62	223.25
10.0	51.92	51.92	103.84	22.0	114.22	114.22	228.44
10.5	54.51	54.51	109.03	22.5	116.82	116.82	233.63
11.0	57.11	57.11	114.22	23.0	119.41	119.41	238.82
11.5	59.71	59.71	119.41	23.5	122.01	122.01	244.02
12.0	62.30	62.30	124.60	24.0	124.60	124.60	249.21
12.5	64.90	64.90	129.80	24.5	127.20	127.20	254.40

APPENDIX B

SITE LOCATION & PILE LENGTHS

- **Quarter Site**



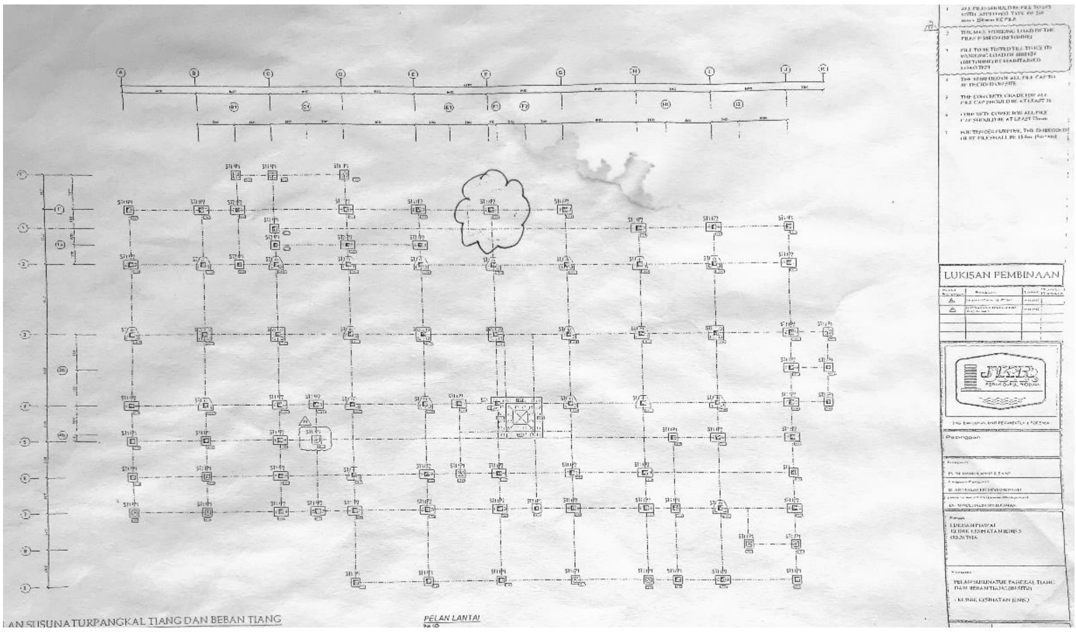
roject : Klinik Kesihatan (Jenis 3) Seri Iskandar, Perak Tengah, Perak darul Ridzuan

Description : Summary of pile

Location : Bangunan Kuarters

Item	Pile Location	Type of Pile Cap	Date Driven	No. of Pile			No. of Joint	Depth (m)	Pressure		Set (mm)
				9m	6m	6m			(mpa)	(tonne)	
1	A / 4a	P1	26.01.2017	1	1	1	2	16.80	11.00	114.22	2mm
2	C / 3 (3)	P3	26.01.2017	1	1	1	2	15.60	11.00	114.22	4mm
3	C1 / 8 (2)	P3	26.01.2017	1	1	-	1	15.00	11.00	114.22	4mm
4	E / 10 (1)	P2	26.01.2017	1	1	1	2	16.50	11.00	114.22	4mm
5	B / 13a	P1	26.01.2017	1	1	1	2	17.70	11.00	114.22	2mm

Clinic



Project : Klinik Kesihatan (Jenis 3) Seri Iskandar, Perak Tengah, Perak darul Ridzuan

Description : Summary of pile

Location : Bangunan Klinik Kesihatan

Item	Pile Location	Type of Pile Cap	Date Driven	No. of Pile			No. of Joint	Depth (m)	Pressure		Set (mm)
				9m	6m	6m			(mpa)	(tonne)	
1	C / 1a	P1	23.01.2017	1	1	1	2	17.70	13.00	134.99	4mm
2	F / 1' (2)	P2	23.01.2017	1	1	-	1	14.10	12.00	124.00	2mm
3	J / 2 (2)	P2	23.01.2017	1	1	1	2	18.00	11.00	114.22	3mm
4	I / 7 (3)	P3	24.01.2017	1	1	-	1	15.00	11.00	114.22	2mm
5	F2 / 4 (27)	Lift Pit	25.01.2017	1	1	1	2	16.80	11.00	114.22	2mm
6	B / 5	P1	25.01.2017	1	1	1	2	18.90	12.00	124.00	2mm

APPENDIX C:

THEORETICAL ULTIMATE BEARING CAPACITY CALCULATION

i. Quarter's Pile (Clay soil)

$\boldsymbol{Q_{C1/8_{(2)}}}$. When C1/8 (2) was jacked in with a max pressure of $\mathbf{7.5}$ **MPa**, the length needed to resist such pressure is $\boldsymbol{L = 15m}$.

1) Point Bearing Capacity in Quarter's:

$$Qp = A_P q_P \ (kN)$$

a. **Qp** in **Clay** ($\boldsymbol{c' = 64\frac{kN}{m^2}}$)

$$q_P = N_c^* c_u$$

$$q_P = 9c_u = 9\left(64\frac{kN}{m^2}\right) = 9180\frac{kN}{m^2}$$

$$\because \ \text{Qp} = \left((0.25 \times 0.25)m^2\right) * 576\frac{kN}{m^2} = 36\ kN.$$

2) Frictional Bearing Capacity in Quarters

$$Q_s = pLf_{av}$$

a. $\mathbf{Q_s}$ in **Clay** ($c_u = \boldsymbol{64\frac{kN}{m^2}}$)

$$f_{av} = \lambda(\bar{\sigma}_o' + 2c_u)$$

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$$\lambda = f(L) = f(15) = \mathbf{0.2}$$

$$\because \boldsymbol{f_{av}} = 0.2\left(\left(64 * 15m\frac{kN}{(15m)m^2}\right) + 2\left(64\frac{kN}{m^2}\right)\right) = \mathbf{38.4}\boldsymbol{\frac{kN}{m^2}}$$

$$Q_s = (4 * 0.25)m * (15m) * \left(38.4\frac{kN}{m^2}\right) = 576 \text{ kN}$$

After computing the average values, Ultimate bearing resistance is then can be calculated:

$Q_{u_{C1/8_{(2)}}} = Q_{s_{avg}} + \text{Q}_{p_{avg}} \because Q_u = 576\ kN + 36\ kN = \boldsymbol{612\ kN}$

ii. <u>Clinic Pile (Sandy soil)</u>

The length needed to resist such pressure is $\boldsymbol{L = 17.7m}$.

Assuming that it rests on **BH6 soil,**

1) Point Bearing Capacity in Quarter's:

$$Qp = A_P q_P \ (kN)$$

a. **Qp** in **Sand** ($\phi' = \mathbf{28}$)

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$$q_P \leq q_l$$

$$q_P = q' N_q^*$$

Since $\boldsymbol{L} = \mathbf{17.7}\boldsymbol{m}$ & $\boldsymbol{\gamma_{BH6}} = \mathbf{16.743}\,\frac{\boldsymbol{kN}}{\boldsymbol{m^3}}$. $\boldsymbol{N_q^*} = \boldsymbol{f}(\boldsymbol{\phi'}) = \boldsymbol{f}(\mathbf{28}) = (\mathbf{39.7})$[3]

$$\because q_P = \mathbf{11757.4}\,\frac{\boldsymbol{kN}}{\boldsymbol{m^2}}$$

Now, $q_l = 0.5\ \left(100\,\frac{kN}{m^2}\right)\left(N_q^*\right)(\tan\phi')$

$$\because q_l = 0.5\left(100\,\frac{kN}{m^2}\right)(39.7)(\tan 28) = \mathbf{1055.44}\,\frac{\boldsymbol{kN}}{\boldsymbol{m^2}}$$

Since $q_P > q_l \because\ Qp = A_P q_l$.[4]

$$\text{Qp} = A_P q_l = (0.25 \times 0.25)m^2\left(1055.44\,\frac{kN}{m^2}\right) = 65.965\ kN$$

2) Frictional Bearing Capacity in Quarters

$$Q_s = pLf_{av}$$

a. Q_s in **Sand** ($\phi' = \mathbf{28}$)

Using Equation 10

$$f_{av} = K\bar{\sigma}_o' \tan\delta'$$

Since $K = (1.5)$ For Precast Concrete. $\sigma_o' = \frac{\sum c_{u(i)} L_{(i)}}{L_T} = 64\,\frac{kN}{m^2}$. $\tan\delta' = \tan 0.8(\phi') = 0.4122$

$$f_{av} = 1.5\left(64\,\frac{kN}{m^2}\right)(0.4122) = \mathbf{39.568}\,\frac{\textbf{kN}}{\textbf{m}^\mathbf{2}}$$

$$\because \text{Q}_\text{s} = \big(4(0.25)\big)m\ (17.7)m\ \left(39.568\,\frac{kN}{m^2}\right) = \mathbf{700.359}\ \textbf{kN}$$

---

[3] **<u>(Braja, 2013, p. 558)</u>**

[4] This is to ensure that the element at its worst-case scenario is still safe to transfer column load.

After computing the average values, Ultimate bearing resistance is then can be calculated:

$Q_{F/1'_{(1)avg}} = Q_{s_{avg}} + \mathrm{Q}_{p_{avg}} \because Q_u = \mathbf{700.359}\ \mathrm{kN} + 65.965\ \mathrm{kN} = \mathbf{766.324\ kN}$

APPENDIX D:

CONCRETE & STEEL PROPERTIES

- ***Concrete***

General

▪ Specified Characteristic Strength @ 28 days	$= \mathbf{35} \frac{\boldsymbol{N}}{\boldsymbol{mm^2}}$
▪ Designed Standard Deviation	$= \mathbf{4.5} \frac{\boldsymbol{N}}{\boldsymbol{mm^2}}$
▪ Designed Margin (1.64 x S.D)	$= \mathbf{7.5} \frac{\boldsymbol{N}}{\boldsymbol{mm^2}}$
▪ Target Mean Strength	$= \mathbf{42.5} \frac{\boldsymbol{N}}{\boldsymbol{mm^2}}$
▪ Slump	$= \mathbf{7} \pm \mathbf{25}\boldsymbol{mm}\ \frac{\boldsymbol{N}}{\boldsymbol{mm^2}}$
▪ Free Water/Cement ratio	$= \mathbf{0.46}$
▪ Free Water Content	$= \mathbf{180} \frac{\boldsymbol{kg}}{\boldsymbol{m^3}}$
▪ Air Content	$= \mathbf{1.5}\%$

Table 8: General Concrete Properties.

Cement

▪ Cement Type/Brand	$= \mathbf{OPC}$
▪ Cement Content	$= 390 \frac{kg}{m^3}$
▪ Specific Gravity of Cement	$= 3.15$

Table 9: Cement Properties.

Aggregates

Coarse Aggregates

▪ Type of Crushed Aggregate	$= \mathbf{Granite}$
▪ Maximum Coarse Aggregate Size	$= 20\ mm$
▪ Coarse Aggregate Content	$= 1000 \frac{kg}{m^3}$
▪ Specific Gravity	$= 2.65$

Table 10: Coarse Aggregate Properties.

Fine Aggregates

▪ Type of Uncrushed Aggregates	= **River Sand**
▪ Grading of Fine Aggregates	= BS 882
▪ Proportion of Fine Aggregates	= 44.4%
▪ Fine Aggregate Content	$= 800\,\frac{kg}{m^3}$
▪ Specific Gravity	= 2.63
▪ Total Aggregate Content	$= 1800\,\frac{kg}{m^3}$

Table 11: Fine Aggregate Properties.

Admixtures/Additives

▪ Admixture 1 (Retarder)	= **Real Mix**
Dosage Rate	$= 0.40\,\frac{\text{Liters}}{\text{100kg Cement}}$
▪ Admixture 2 (Superplasticiser)	= Real PF 200
Dosage Rate	$= 0.40\,\frac{\text{Liters}}{\text{100kg Cement}}$

Table 12: Admixtures Properties.

Concrete

▪ Designed Density of Concrete	$\boldsymbol{= 2370\,\frac{kg}{m^3}}$

- Steel Properties

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Tel : 05-528 1577 Fax : 05-528 5750 Email : ipohlab2012@yahoo.com

TENSILE TEST

(Part 2 : Specimen's Strength & Elongation Info)

Project	KLINIK KESIHATAN (JENIS 3), SERI ISKANDAR, PERAK TENGAH, PERAK.	Test Method	MS 146 : 2006 & MS ISO 6892: 2002		
		Rpt No.	ILSB / 05 / 05 / TT / 302 / 1361		
		JS No.	20342		
Client	SURIA GEMILANG SDN BHD	Date Tested	14/02/2017	Temp.	28.0°C
Test No.	02 / 17 - 58	Date Received	14/02/2017	Humidity	64.0%

Type of sample	Y 25	Sample Marking	Y 25	
1) Effective cross-sectional area, S_o	470.54 mm²	2) Dia. of effective cross-sectional area	24.48	mm
Total length, L_t	604.00 mm	3) Original gauge length, L_o	125.00	mm
Mass of bar, M	2.231 kg	Final gauge length, L_u	154.90	mm

Yield point (N)	Max force (N)	4) Yield strength, R_e (N/mm²)	5) Tensile strength, R_m (N/mm²)	6) Elongation, A (%)
275,723.005	324,772.000	586	690	23.9

1) Effective cross-sectional area, $S_o = M / 0.00000785L_t$

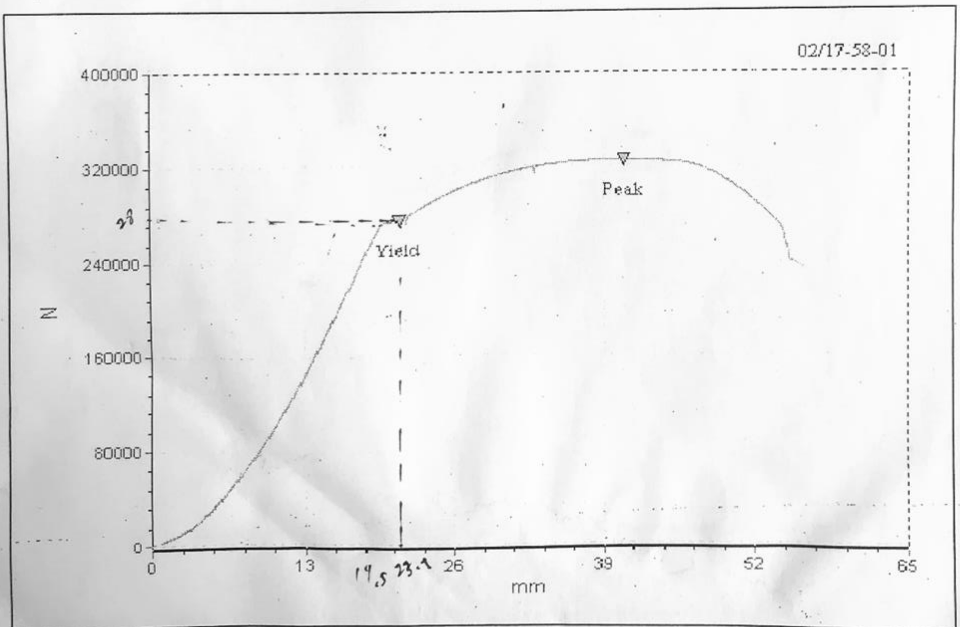
2) Dia. of effective cross-sectional area = $(4S_o/\pi)^{1/2}$

3) Original gauge length, $L_o = 5d$, where d is the nominal size of steel bar (high tensile) or $L_o = 5.65\sqrt{S_o}$ (steel welded).

4) Yield strength, R_e = Yield point / S_o

5) Tensile strength, R_m = Max force / S_o

6) Elongation, $A = [(L_u - L_o)/L_o] \times 100$



Remarks : Arm Steel Sdn Bhd

Note : Please note that these tests were carried out on the sample provided/supplied by client. We, (in any case), should not be made responsible on the origin of the sample.

Tested by :

	WITNESSED (1)	WITNESSED (2)
SIGN		
NAME	K. Nizam	NUR ATIQAH
DATE	14/02/2017	14/02/2017

Verified by :

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No. 135, Bulatan Cherry, Medan Perindustrian Cherry, Off Jalan Tun Abdul Razak,
30100 Ipoh, Perak Darul Ridzuan.
Tel : 05-528 1577 Fax : 05-528 5750 Email : ipohlab2012@yahoo.com

TENSILE TEST

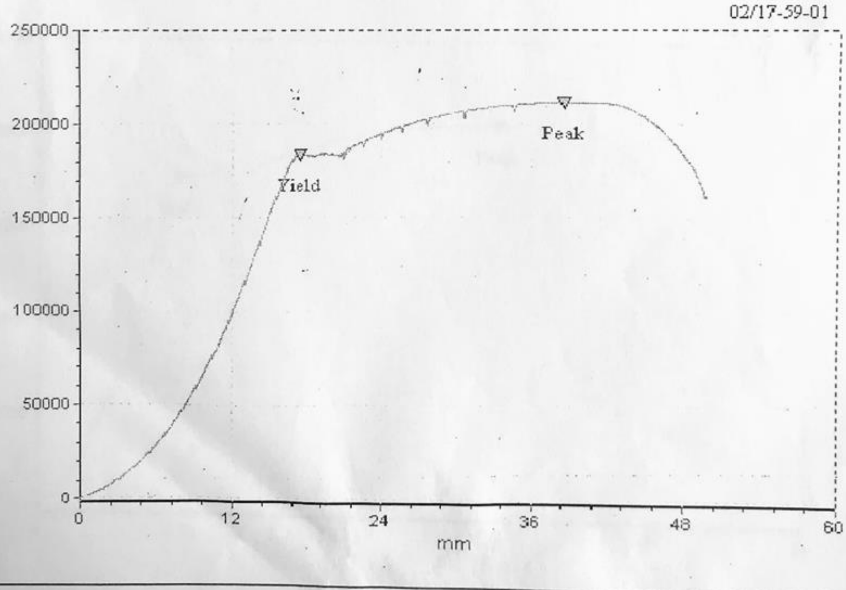
(Part 2 : Specimen's Strength & Elongation Info)

Project	KLINIK KESIHATAN (JENIS 3), SERI ISKANDAR, PERAK TENGAH, PERAK.	Test Method	MS 146 : 2006 & MS ISO 6892: 2002		
		Rpt No .	ILSB / 05 / 05 / TT / 302 / 1361		
		JS No.	20342		
Client	SURIA GEMILANG SDN BHD	Date Tested	14/02/2017	Temp.	28.0°C
Test No.	02 / 17 - 59	Date Received	14/02/2017	Humidity	64.0%

Type of sample	Y 20	Sample Marking	Y 20
1) Effective cross-sectional area, S_o	299.85 mm²	2) Dia. of effective cross-sectional area	19.54 mm
Total length, L_t	602.00 mm	3) Original gauge length, L_o	100.00 mm
Mass of bar, M	1.417 kg	Final gauge length, L_u	122.56 mm

Yield point (N)	Max force (N)	4) Yield strength, R_e (N/mm²)	5) Tensile strength, R_m (N/mm²)	6) Elongation, A (%)
183,848.004	211,248.804	613	705	22.6

1) Effective cross-sectional area, $S_o = M / 0.00000785L_t$
4) Yield strength, R_e = Yield point / S_o
5) Tensile strength, R_m = Max force / S_o
2) Dia. of effective cross-sectional area = $(4S_o/\pi)^{1/2}$
6) Elongation, $A = [(L_u - L_o)/L_o] \times 100$
3) Original gauge length, $L_o = 5d$, where d is the nominal size of steel bar (high tensile) or $L_o = 5.65\sqrt{S_o}$ (steel welded).



Remarks : Arm Steel Sdn Bhd

Note : Please note that these tests were carried out on the sample provided/supplied by client. We, (in any case), should not be made responsible on the origin of the sample.

Tested by :

	WITNESSED (1)	WITNESSED (2)
SIGN	[signature]	[signature]
NAME	K. Nizam	NUK ATIQAH
DATE	14/02/2017	14/02/17

Verified by :

IPOH LAB SDN. BHD. LAB COOR

IPOH LAB SDN. BHD. ENGINEER

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30100 Ipoh, Perak Darul Ridzuan.
Tel : 05-528 1577 Fax : 05-528 5750 Email : ipohlab2012@yahoo.com

TENSILE TEST

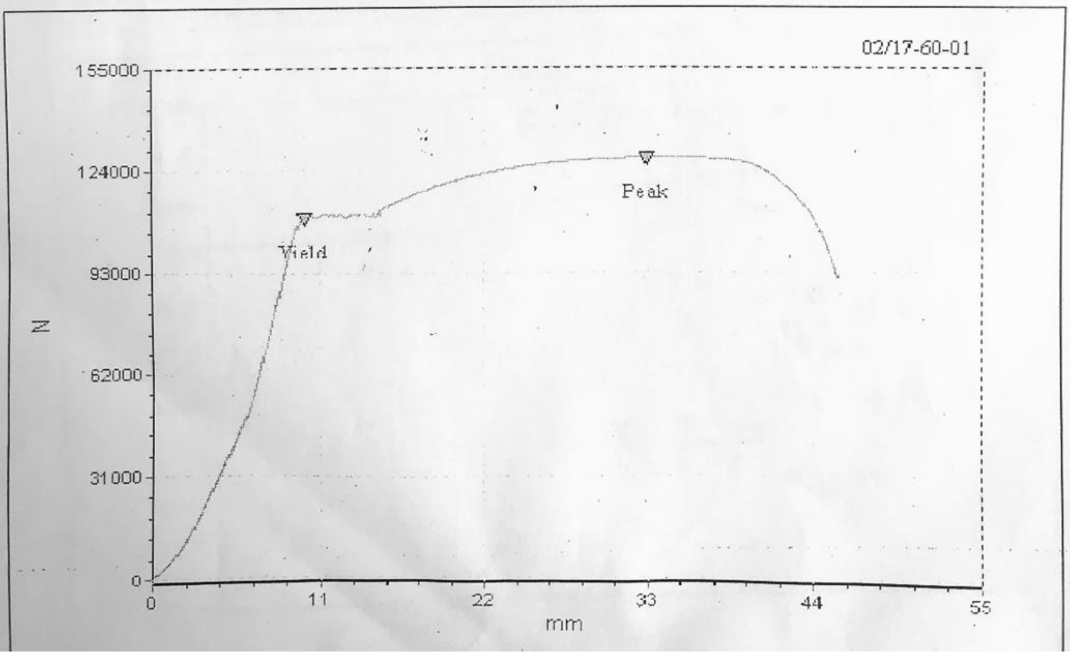
(Part 2 : Specimen's Strength & Elongation Info)

Project	KLINIK KESIHATAN (JENIS 3), SERI ISKANDAR, PERAK TENGAH, PERAK.	Test Method	MS 146 : 2006 & MS ISO 6892: 2002		
		Rpt No .	ILSB / 05 / 05 / TT / 302 / 1361		
		JS No.	20342		
Client	SURIA GEMILANG SDN BHD	Date Tested	14/02/2017	Temp.	28.0°C
Test No.	02 / 17 - 60	Date Received	14/02/2017	Humidity	64.0%

Type of sample	Y 16	Sample Marking	Y 16
1) Effective cross-sectional area, S_o	195.22 mm²	2) Dia. of effective cross-sectional area	15.76 mm
Total length, L_t	601.00 mm	3) Original gauge length, L_o	80.00 mm
Mass of bar, M	0.921 kg	Final gauge length, L_u	98.66 mm

Yield point (N)	Max force (N)	4) Yield strength, R_e (N/mm²)	5) Tensile strength, R_m (N/mm²)	6) Elongation, A (%)
109,936.402	127,556.803	563	653	23.3

1) Effective cross-sectional area, $S_o = M / 0.00000785L_t$
2) Dia. of effective cross-sectional area = $(4S_o/\pi)^{1/2}$
3) Original gauge length, $L_o = 5d$, where d is the nominal size of steel bar (high tensile) or $L_o = 5.65\sqrt{S_o}$ (steel welded).
4) Yield strength, R_e = Yield point / S_o
5) Tensile strength, R_m = Max force / S_o
6) Elongation, $A = [(L_u - L_o)/L_o] \times 100$



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30100 Ipoh, Perak Darul Ridzuan.
Tel : 05-528 1577 Fax : 05-528 5750 Email : ipohlab2012@yahoo.com

TENSILE TEST

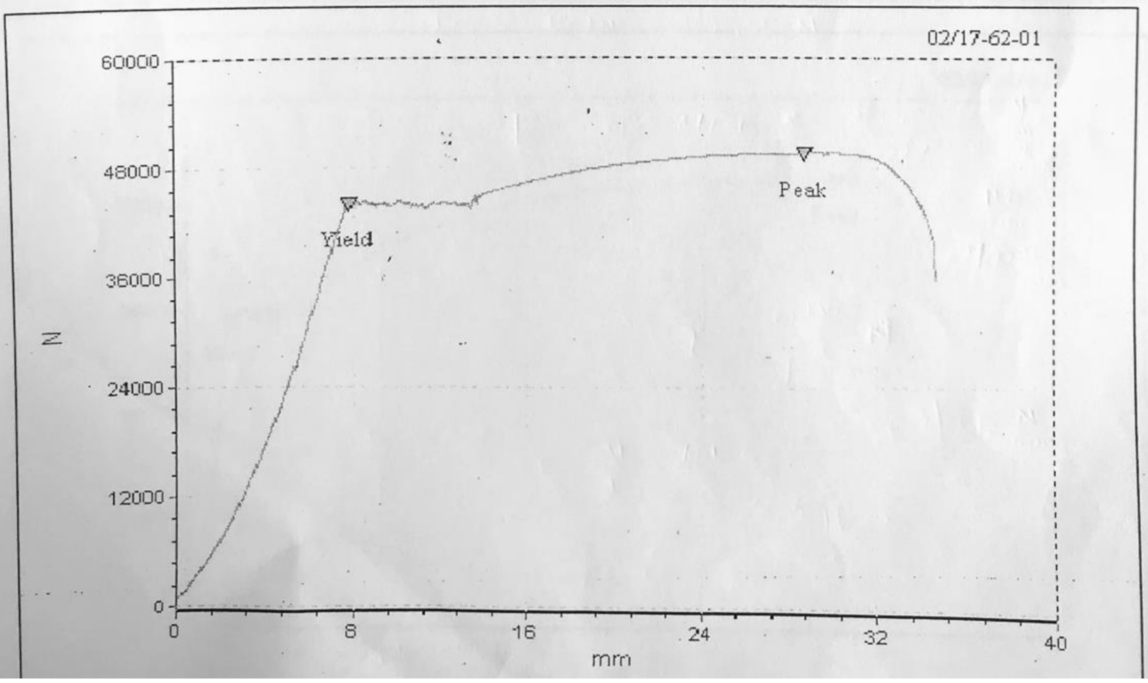
(Part 2 : Specimen's Strength & Elongation Info)

oject	KLINIK KESIHATAN (JENIS 3), SERI ISKANDAR, PERAK TENGAH, PERAK.	Test Method	MS 146 : 2006 & MS ISO 6892: 2002
		Rpt No .	**ILSB / 05 / 05 / TT / 302 / 1361**
		JS No.	20342
lient	SURIA GEMILANG SDN BHD	Date Tested	14/02/2017 Temp. 28.0°C
est No.	02 / 17 - 62	Date Received	14/02/2017 Humidity 64.0%

ype of sample	Y 10	Sample Marking	Y 10
Effective cross-sectional rea, S_o	75.21 mm²	[2] Dia. of effective cross-sectional area	9.78 mm
otal length, L_t	603.00 mm	[3] Original gauge length, L_o	50.00 mm
lass of bar, M	0.356 kg	Final gauge length, L_u	57.80 mm

Yield point (N)	Max force (N)	[4] Yield strength, R_e (N/mm²)	[5] Tensile strength, R_m (N/mm²)	[6] Elongation, A (%)
44,198.001	49,519.401	588	658	15.6

) Effective cross-sectional area, $S_o = M / 0.00000785L_t$
4) Yield strength, R_e = Yield point / S_o
5) Tensile strength, R_m = Max force / S_o
) Dia. of effective cross-sectional area = $(4S_o/\pi)^{1/2}$
6) Elongation, $A = [(L_u - L_o)/L_o] \times 100$
) Original gauge length, $L_o = 5d$, where d is the nominal size of steel bar (high tensile) or $L_o = 5.65\sqrt{S_o}$ (steel welded).



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No. 135, Bulatan Cherry, Medan Perindustrian Cherry, Off Jalan Tun Abdul Razak,
30100 Ipoh, Perak Darul Ridzuan.
Tel : 05-528 1577 Fax : 05-528 5750 Email : ipohlab2012@yahoo.com

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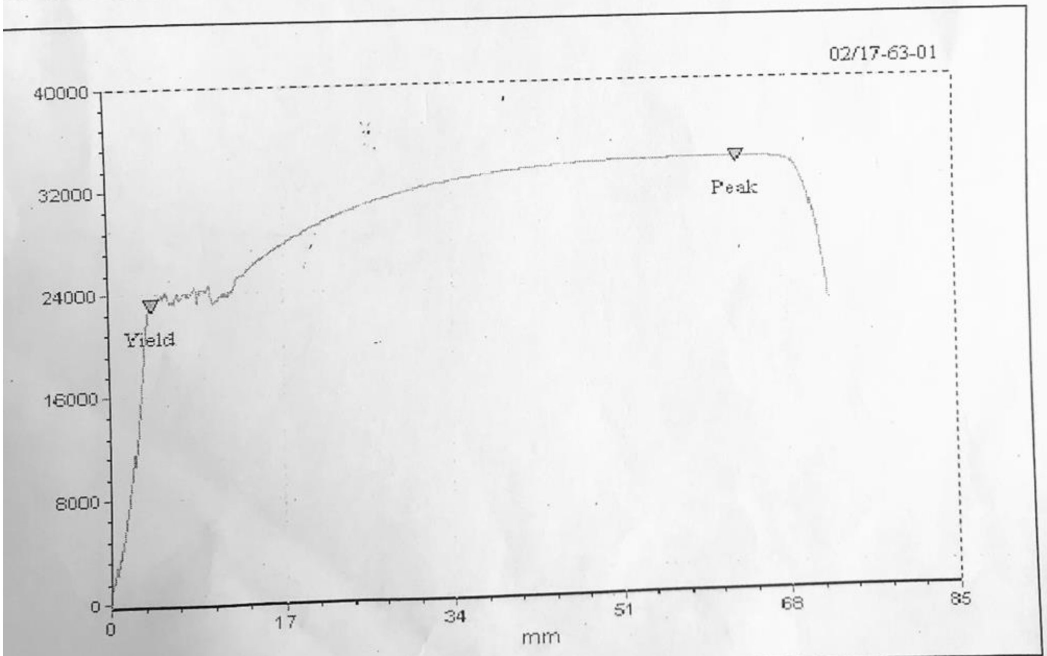
KLINIK KESIHATAN (JENIS 3), SERI ISKANDAR, PERAK TENGAH, PERAK.	Test Method	MS 146 : 2006 & MS ISO 6892: 2002			
	Rpt No .	ILSB / 05 / 05 / TT / 302 / 1361			
	JS No.	20342			
SURIA GEMILANG SDN BHD	Date Tested	14/02/2017	Temp.	$28.0^{O}C$	
02 / 17 - 63	Date Received	14/02/2017	Humidity	64.0%	

nple	R 10	Sample Marking	R 10	
cross-sectional	76.73 mm^2	2) Dia. of effective cross-sectional area	9.88	mm
n, L_t	601.00 mm	3) Original gauge length, L_o	50.00	mm
ar, M	0.362 kg	Final gauge length, L_u	65.32	mm

int	Max force (N)	4) Yield strength, R_e (N/mm^2)	5) Tensile strength, R_m (N/mm^2)	6) Elongation, A (%)
01	33,692.401	303	439	30.6

oss-sectional area, $S_o = M / 0.00000785L_t$
4) Yield strength, R_e = Yield point / S_o
5) Tensile strength, R_m = Max force / S_o
ctive cross-sectional area = $(4S_o/\pi)^{1/2}$
6) Elongation, $A = [(L_u - L_o) / L_o] \times 100$
ge length, $L_o = 5d$, where d is the nominal size of steel bar (high tensile) or $L_o = 5.65\sqrt{S_o}$ (steel welded).



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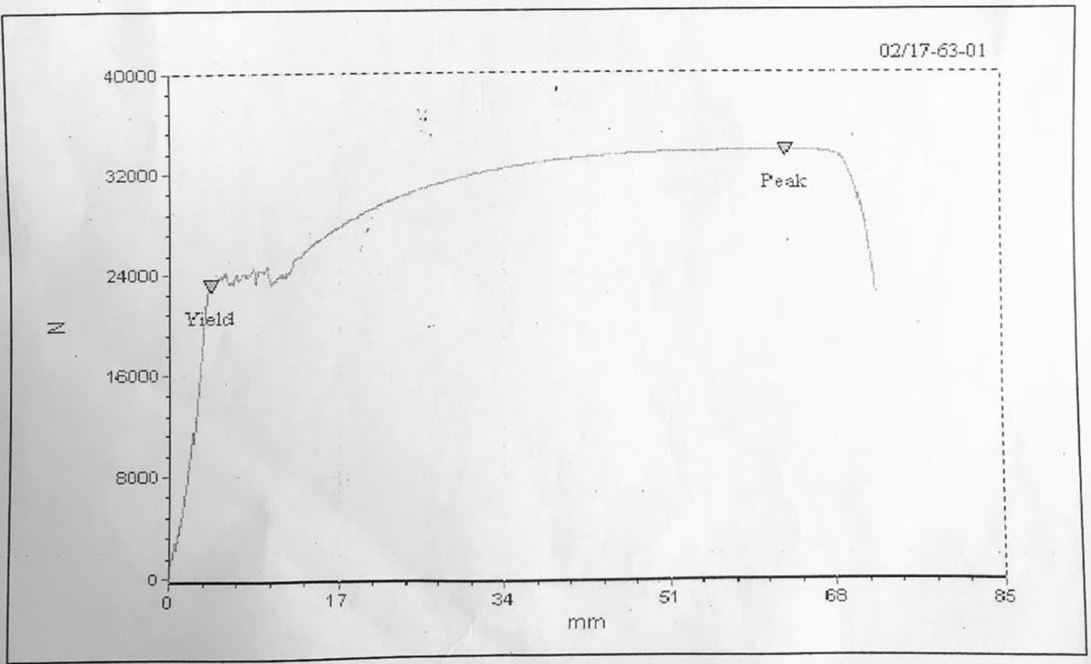
		Rpt No .	ILSB / 05 / 05 / TT / 302 / 1361		
		JS No.	20342		
Client	SURIA GEMILANG SDN BHD	Date Tested	14/02/2017	Temp.	28.0°C
Test No.	02 / 17 - 63	Date Received	14/02/2017	Humidity	64.0%

Type of sample	R 10		Sample Marking	R 10	
1) Effective cross-sectional area, S_o	76.73	mm^2	2) Dia. of effective cross-sectional area	9.88	mm
Total length, L_t	601.00	mm	3) Original gauge length, L_o	50.00	mm
Mass of bar, M	0.362	kg	Final gauge length, L_u	65.32	mm

Yield point (N)	Max force (N)	4) Yield strength, R_e (N/mm^2)	5) Tensile strength, R_m (N/mm^2)	6) Elongation, A (%)
23,275.001	33,692.401	303	439	30.6

1) Effective cross-sectional area, $S_o = M / 0.00000785L_t$
2) Dia. of effective cross-sectional area = $(4S_o / \pi)^{1/2}$
3) Original gauge length, $L_o = 5d$, where d is the nominal size of steel bar (high tensile) or $L_o = 5.65\sqrt{S_o}$ (steel welded).
4) Yield strength, R_e = Yield point / S_o
5) Tensile strength, R_m = Max force / S_o
6) Elongation, $A = [(L_u - L_o) / L_o] \times 100$



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APPENDIX E

MATHEMATICA PROGRAM