THE COMPARISON OF BEARING CAPACITY CALCULATION OBTAINED BASED ON SPT-N FROM SEISMIC SURVEY AGAINST STRENGTH PARAMETERS (C' AND ϕ) VALUE FROM SOIL DRILLING

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by

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17558

Dissertation submitted in partial fulfilment of the requirements for the

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Universiti Teknologi PETRONAS,

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CERTIFICATION OF APPROVAL

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in partial fulfilment of the requirement for the
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(CIVIL ENGINEERING)

Approved by,

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September 2016

CERTIFICATION OF ORIGINALITY

This is to certify that I am responsible for the work submitted in this project, that the original work is my own except as specified in the references and acknowledgements, and that the original work contained herein have not been undertaken or done by unspecified sources or persons.

MOHAMAD AIMAN SYAFIQ BIN KAMARUDIN

CERTIFICATION OF LAB WORKS APPROVAL

This is to certify that the student had conducted her research on this project adhering the rules and regulations in the lab that are in line with the health, safety and environment policy of the lab.

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IZHATUL IMMA YUSRI
Lab Technologist

ABSTRACT

Soil investigation is a method of obtaining data for a particular area regarding the properties of subsoil which includes fieldworks and laboratory tests. Nowadays, conventional techniques are mostly used for soil investigation but there are a few disadvantages of using it have been discovered. The disadvantages of using conventional techniques are costly, time consuming, destructive and limited of equipment mobilization. The scope of study for this research is to determine the bearing capacity calculation obtained based on two different methods which are SPT-N from seismic survey and strength parameters (c' and ϕ) from soil drilling. The objectives of this research are to determine the correlation of bearing capacity calculation between SPT-N from seismic survey and (c' and ϕ) and to verify the accuracy of SPT-N (seismic) from SPT-N (boreholes). There are two scope of works involved in this research which are fieldwork and laboratory work. The seismic survey test has been conducted at a few fieldwork locations in Malaysia to obtain the SPT-N value. The samples for laboratory works were taken at fieldwork location by conducting soil boring test. Then, direct shear test was conducted to determine the shear strength parameters (c' and ϕ) value. The experimentation results from both methods are studied and analyzed. The correlation of bearing capacity value obtained based on SPT-N value from seismic survey and (c' and ϕ) is established but the regression value is relatively low (R^2=0.3205). However, the correlation of SPT-N from seismic survey and SPT-N from boreholes shows high regression value which has verified the accuracy of SPT-N from seismic survey (R^2=0.8061).

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CHAPTER 1

INTRODUCTION

1.1 Background of Study

Generally, soil investigation is a method obtaining data for a particular area regarding the properties of subsoil includes reconnaissance, field works and providing sample for laboratory tests. It involves the investigation at the surface and subsurface that acquiring all type of ground information or data effecting design and construction of the project. Besides, it provides acquire data for geotechnical model of the ground that will be encountered and affected by the construction and it also help to predict the reaction of the ground to the construction of the project.

In this research, the scope of study is focused on obtaining the bearing capacity calculation value of the soil. Bearing capacity is one of the criterion of structural stability. The failure criterion of foundation soil is known as the ultimate bearing capacity (Qu). There are various bearing capacity formulas that have been formulated by the scientists and can be used easily in geotechnical design. However, there are two types of bearing capacity calculation that have been chose and applied in this research which uses SPT-N value from seismic survey and strength parameters (c' and ϕ) from soil drilling.

The samples of soil for the research were taken at few random locations in Malaysia by using conventional technique which is soil boring test. Then, those samples from soil drilling were brought to the laboratory to conduct a few laboratory tests such as moisture content, direct shear and Atterberg limit test. The purpose of conducting direct shear test is to determine the shear strength parameters which are cohesion, (c’) and angle of internal friction (ϕ) value.

Besides, the fieldwork test such as seismic survey using surface wave method has been conducted at all fieldwork locations. The purpose of conducting seismic survey test is to obtain the SPT-N value. The SPT-N values from seismic survey were obtained by the correlation of shear wave velocity with SPT-N. The coefficient of correlation between shear wave velocity and SPT-N value is 0.868 (IMAI et al, 1975). Then, the accuracy of SPT from seismic can be verified by SPT borehole. Therefore, the graph of bearing capacity from SPT-N (seismic) against bearing capacity from strength parameters (c’ and ϕ) can be plotted at the end of research.

1.2 Problem Statement

Nowadays, the conventional techniques such as soil boring and standard penetration test (SPT) are widely used in industry to determine the various strength parameters for various geotechnical design including the bearing capacity calculation. Unfortunately, there are a few disadvantages of using conventional techniques that have been discovered.

The disadvantages of using conventional techniques are as follows:

- The material and equipment are so expensive.
- The installation of the equipment for conducting the test were taking longer time due the heavy weight and big size material.
- The test conducting at fieldwork location might destruct or disturb the originality of soil structure.
- It is difficult to bring the equipment at the narrow space or hilly area at site.

However, conventional techniques are still the most used method in industry since it still gives the most accurate strength parameters to be used in geotechnical design.

1.3 Objectives

The objectives of this study are:

- To determine the correlation of bearing capacity calculation between SPT-N from seismic survey and strength parameters (c and ϕ) from soil drilling.
- To verify the accuracy of SPT-N (seismic survey) from SPT-N borehole.

1.4 Scope of Study

The scope of study for this research are:

- Literature review on the research by previous researchers.
- Soil boring test at a few locations in Malaysia and preparation of the sample for laboratory tests.
- Determination of SPT-N value from seismic survey test (surface wave method).
- Determination of shear strength parameters (c' and ϕ) from laboratory test which is direct shear test.
- Determination of additional data for the research such as moisture and plasticity index of the soil by conducting laboratory works such as moisture content and Atterberg limit test.
- Analysis of the data and presentation of the findings.

CHAPTER 2

LITERATURE REVIEW

2.1 Bearing Capacity Calculation of Soil

The ultimate bearing capacity value of soil under shallow footing was investigated theoretically by Prandtl (1992) and Reissner (1924) by using the concept of plastic of equilibrium in 1924. Then, the formulation was slightly modified, generalized and updated by Terzaghi (1925), Meyerhof (1956), Hansen (1968), De Beer (1970) and Sieffert and Bay-Gress (2000). There are various uncertainties in representing the real in-situ soil conditions by means of a few laboratory tests to obtain the shear strength parameters. The basic soil parameters are cohesion (c’), undrained shear strength and angle of internal friction (ϕ), which can only be determined by laboratory testing of undisturbed soil samples such as direct shear test and undrained unconsolidated triaxial test.

2.1.1 Bearing Capacity Calculation from (c’ and ϕ)

In this research, Meyerhof formula has been used to calculate the bearing capacity from (c’ and ϕ). Meyerhof (1963) has suggested the following form of the general bearing capacity equation:

$$qu = c'NcFcsFcdFci + qNqFqsFqdFqi + 0.5\gamma BN\gamma F\gamma sF\gamma dF\gamma i$$

In this equation:

c’ = cohesion

q = effective stress at the level of the bottom of the foundation

γ	= unit weight of soil
B	= width of foundation
Fcs, Fqs, Fγs	= shape factors
Fcd, Fqd, Fγd	= depth factors
Fci, Fqi, Fγi	= load inclination factors
Nc, Nq, Nγ	= bearing capacity factors

The original equation for ultimate bearing capacity is derived only for the plane stress such as for continuous foundations. The shape, depth and load inclination factors are empirical factors based on experimental data. The equations of bearing capacity factors are as follow:

$$Nq = \tan^2(45 + \frac{\phi'}{2})e^{\pi \tan \phi'}$$

$$Nc = (Nq - 1)\cot \phi'$$

$$N\gamma = 2(Nq + 1)\tan \phi'$$

Equation for *Nc* was originally derived by Prandtl (1921) and equation for *Nq* was presented by Reissner (1924). Besides, Caquot and Kerisel (1953) and Vesic (1973) gave the relation for *Nγ*.

Table 2.1: Shape, Depth and Inclination Factors (DeBeer (1970); Hansen (1970); Meyerhof (1963); Meyerhof and Hanna (1981))

Factor	Relationship	Reference
Shape	$Fcs = 1 + (\frac{B}{L})(\frac{Nq}{Nc})$ $Fqs = 1 + (\frac{B}{L}) \tan \phi'$ $F\gamma s = 1 - 0.4(\frac{B}{L})$	DeBeer (1970)
Depth	$Df/B > 1$ For ϕ=0: $Fcd = 1 + 0.4 \tan^{-1}(\frac{Df}{B})$ $Fqd = F\gamma d = 1$ For $\phi > 0$: $Fcd = Fqd - \frac{(1 - Fqd)}{(Nc \tan \phi')}$ $Fqd = 1 + 2 \tan \phi'(1 - \sin\phi')^2 \ \tan^{-1}(\frac{Df}{B})$ $F\gamma d = 1$	Hansen (1970)
Inclination	$Fci = Fqi = (1 - \frac{\beta^{o}}{90^{o}})^2$ $F\gamma i = (1 - \frac{\beta}{\phi'})$ Β = inclination of the load	Meyerhof (1963); Hana and Meyerhof (1981)

2.1.2 Bearing Capacity Calculation from SPT-N Value

It is difficult to obtain undisturbed samples of coarse-grained soils for testing in the laboratory. The allowable bearing capacity and settlement of footings on coarse-grained soils are often based on empirical methods using test data from field tests. One of the popular method utilizes results from the standard penetration test (SPT). It is customary to correct the N values for overburden pressure. Various correction factors have been suggested by many investigators. Two suggestions for correcting N values for overburden pressure are as follow:

$$CN = (\frac{95.8}{\sigma'zo})^{1/2} \text{ ; } CN \le 2 \text{ (Liao and Whitman, 1985)}$$

$$CN = 0.77 \log_{10}\left(\frac{1916}{\sigma'zo}\right); \; CN \le 2, \; \sigma'zo > 24kPa \text{ (Peck et al., 1974)}$$

Where CN is a correction factor for overburden pressures, and δ'zo is the effective overburden pressure in kPa. A further correction factor is imposed on N values if the groundwater level is within a depth B below the base of the footing. The groundwater correction factor is as follow:

$$cw = 1/2 + z/2(Df + B)$$

Where z is the depth to the groundwater table, Df is the footing depth, and B is the footing width. If the depth of the groundwater level is beyond B from the footing base, cw=1.

The corrected N value is as follow:

$$N1 = cN \times cw \times N$$

Thus, the ultimate bearing capacity for a shallow footing under vertical loads is as follow:

$$Qult = 32N1B(kPa)$$

Where B is the width in m. Each value of N in a soil layer up to a depth of 1.5B below the footing base is corrected and an average value of N1 is used in equation above.

	Soil type	N_{60}	c'_u	ϕ'_{av}	v_s	q_a (kPa)[b]
No.	Cohesive soils[a]	–	kPa	degree	m/s	100 200 300 400 500 600
1	Very soft clays and silts	0–2	0–20	20	0–100	50
2	Soft clays and silts	2–4	20–50	22	0–200	75
3	Medium stiff clays	4–8	50–100	24	200–350	75 150
4	Stiff clays	8–15	100–150	26	200–600	100 250
5	Very stiff clays, boulders	15–30	150–200	28	450–800	200 350
6	Hard clays, boulders	30–50	200–400	30	600–900	250 400
7	Very hard clays	50–R	400–600	30	800–1200	350 500
No.	Granular soils	N_{60}	D_r	ϕ'_{av}	v_s	
1	Very loose sand	0–4	0–20	28°	0–100	50
2	Loose sand and gravel	4–10	20–35	30°	100–350	50 150
3	Medium dense sand, gravel	10–30	35–65	32°	250–700	100 300
4	Dense sand and gravel	30–50	65–85	37°	600–1100	250 450
5	Very dense sand and gravel	50–R	85–99	40°	800–1500	350 600

c'_u = undrained effective shear strength (kPa), D_r = relative density *(percent)*, ϕ'_a = average effective internal angle of friction *(degrees)*, v_s = shear wave velocity (m/s), q_a = allowable soil bearing pressure (kPa).
[a]*Normally conslidated clays,*
[b]*If the foundation base is below the GWT, use the lower values of q_a within the range.*

Figure 2.1: Recommended ranges of allowable bearing capacities (kPa)

2.2 Geophysical method of Soil Investigation

In geotechnical field, researchers have been used geological physical methods to determine the Earth's subsurface structure and condition data. The seismic survey is one form of geophysical survey that aims at measuring the earth's properties by means of physical principles such as magnetic, electric, gravitational, thermal, and elastic theories. It is based on the theory of elasticity and therefore tries to deduce elastic properties of materials by measuring their response to elastic disturbances called seismic or elastic waves.

Besides, ground resistivity survey methods have been widely used to solve engineering, environmental and geological problems in the last decades. Subsurface resistivity distributions are measured by applying electrical current into the ground by using two current electrodes. The potential differences caused by the flow of current between any two points in linear line with the current electrodes are measured by a pair of potential electrodes. From the measured voltage (V) and current (I) values, the resistance at the specified point in the subsurface can be determined.

Electrical resistivity is known to be highly variable among other physical properties of rock. In some cases, different in extreme values of a single rock type can differ by a factor approaching several orders of magnitude. Wide range of rock's resistivity parameter has always been the reason that makes it difficult to distinguish subsurface rock type if no information on the geological surroundings of field survey is available.

However, most of field resistivity surveys conducted by geophysicist are not always validated by laboratory measurement. The difficulty in obtaining the core sample, where the drilling works should be preceded by resistivity survey has made it difficult for geophysicist to analyse the samples in laboratory.

2.3 General Theories of Seismic Wave Survey

Basically, there are many physical parameters can take into our consideration before making important decision for civil construction. These physical parameters play an essential role in indicating their behavior which is due to time and condition changes. For some researches, the data for the seismic refraction (tomography) method is correlated with the borehole data collected from the study site. The seismic refraction method is usually used to determine the lithology and stratigraphic geometry of geological sites.

The seismic refraction investigates the subsurface by generating arrival time and offset distance information to determine the path and velocity of the elastic disturbance in the ground. Usually the disturbance is created by hammer, weight drop or some other comparable method for putting impulsive energy into the ground. Detectors lie out at the regular interval to measure the first arrival of the energy and its time. The data are plotted in time and distance graphs from which the velocities of the different layers and their depth can be calculated.

In addition, a deeper understanding of the seismic process can contribute to improve the interpretations. A lot of information on seismic arrivals are currently attainable by use of the engineering seismographs which display the complete waveform. The interpreters should know about possible wave types, seismic wave types and expected travel-time patterns to get the understanding of seismic arrival. Besides, seismic images can be more accurate with the development of more sophisticated velocity models which contain information about the speed with which the seismic waves travel through the rock layers.

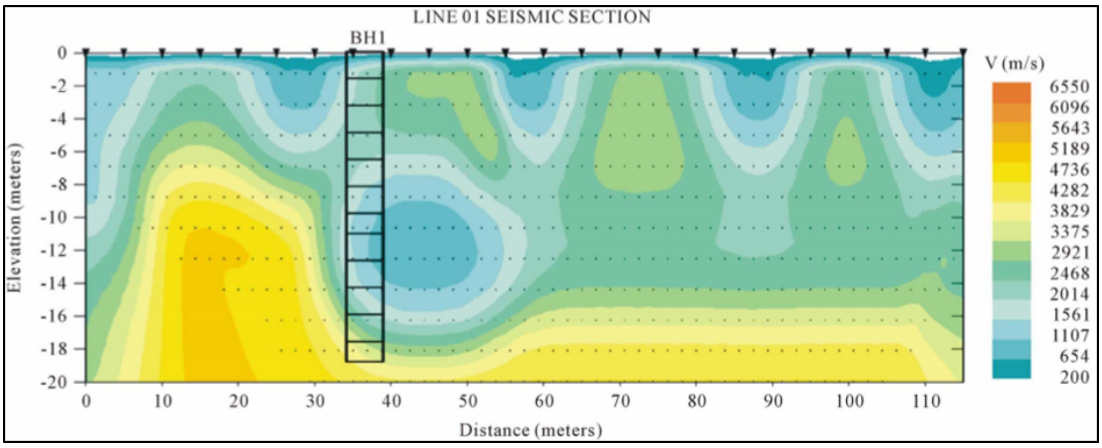

Figure 2.2: Example of line 1 seismic refraction from past research (Bery et al., 2012)

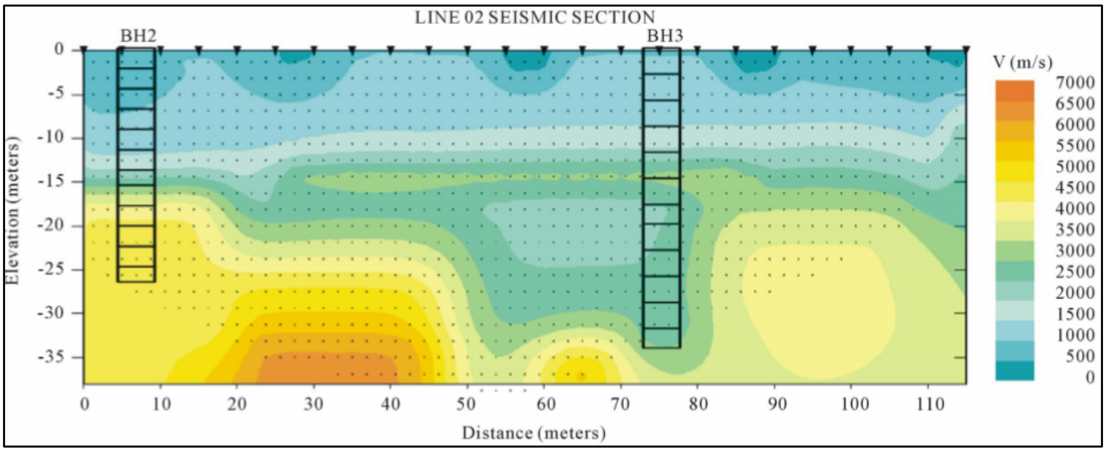

Figure 2.3: Example of line 2 seismic refraction from past research (Bery et al., 2012)

2.4 Seismic Wave using Surface Wave Method

Nowadays, there are many type of seismic wave method that have been used in industry. In this research, the seismic test conducted was used surface wave method. Seismic wave using surface wave method is defined as the geophysical survey using the surface wave that give the output of shear wave velocity of the ground. Shear wave velocity of ground is closely related to the dynamic property of the ground. The previous researchers had proved the higher correlation value between shear wave velocity and SPT-N value. The regression value from the correlation of shear wave velocity and SPT-N is 0.868 (IMAI et al, 1975).

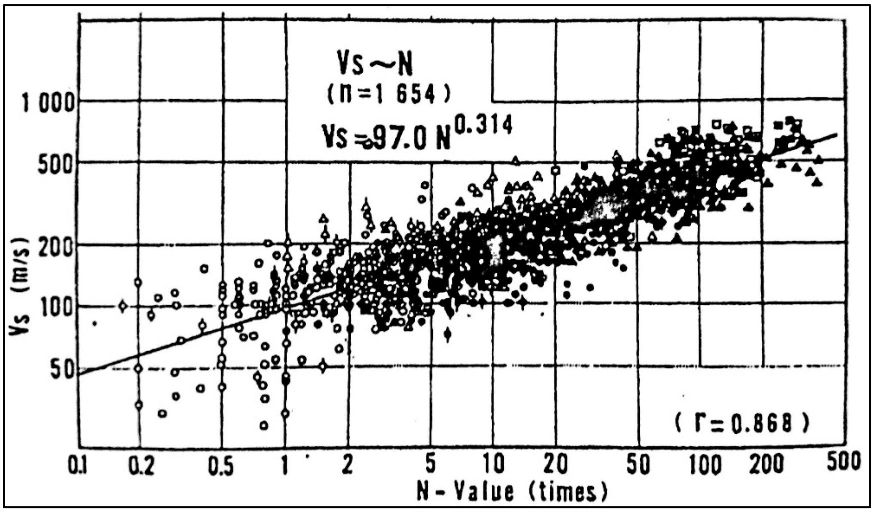

Figure 2.4: Relationship between SPT-N value and shear wave velocity, Vs according to IMAI et al (1975). (Tumwesige, Gidudu, Bagampadde & Ryan)

Basically, the purposes of using surface wave method in industry are to survey the loose area affected by the presence of cavern and to understand the effect on ground improvement. There are a few geophysical surveys using the surface wave such as surface wave method and micro-tremors array measurement. The characteristics of surface wave method are feasible to grasp 2-directions shear wave velocity structure and the target depth span from the surface to the depth is about 20-m. Besides, the characteristics for micro-tremors array managements are feasible to grasp only in 1-direction shear wave velocity structure and the target depths span from the surface from the surface to the depth is about 100-m.

In surface wave method, triggering the ground by using wooden mallet is initiated for measuring the surface wave propagating through the ground. By shifting the trigger point and the receiver, it is feasible to grasp 2-direction wave velocity structure. An approximate criterion for the survey depth is about half of the spread length. An approximate criterion for the trigger point interval is 2 times if the receiver interval in usual use.

Surface wave method analysis can be conducted using SeisImager software that works on McSEIS-SXW or computer. SeisImager is constituted by the following three programs:

- Measurement and Data Preprocessing Software.
- Wave velocity structure analysis software.
- Shear wave velocity structure graphical representation and interpretation software.

2.5 Determination the Strength Parameters (c' and ϕ) by Direct Shear Test

The direct shear test is the oldest and simplest form of shear test arrangement. It is commonly used technique for determining the shear strength parameters (c' and ϕ). The test equipment consists of a metal shear box in which the soil specimen is placed. Normal stress applied on the specimen can be as great as 1050 kN/m^2.

Depending on the equipment, the shear test can be either stress controlled or strain controlled. In stress-controlled tests, the shear force is applied in equal increments until the specimen fails. The failure occurs along the plane of split of the shear box. The shear displacement of the top half of the box is measured by a horizontal dial gauge after the application of each incremental load. The change in the height of the specimen during the test can be obtained from the readings of a dial gauge that measures the vertical movement of the upper loading plate.

Besides, in strain-controlled tests, a constant rate of shear displacement is applied to one-half of the box by a motor that acts through gears. The constant rate of shear displacement is measured by a horizontal dial gauge. The resisting shear force of the soil corresponding to any shear displacement can be measured by a horizontal proving ring or load cell. The volume change of the specimen during the test is obtained in a manner similar to that in the stress-controlled tests.

The advantage of the strain-controlled tests is that in the case of dense sand, peak shear resistance and lesser shear resistance can be observed and plotted. However, stress-controlled tests probably model real field situations better compared with stain-controlled tests.

2.6 Shear Strength of Soil

The shear strength of a soil mass is the internal resistance per unit area that the soil mass can offer to resist failure and sliding along any plane inside it. Generally, soils failed in shear. At failure condition, shear stress along the failure surface reaches the shear strength. Then, soil grains will slide over each other along the failure surface and there is no crushing of individual grains. Besides, shear stress along the failure surface reaches the shear strength at failure condition.

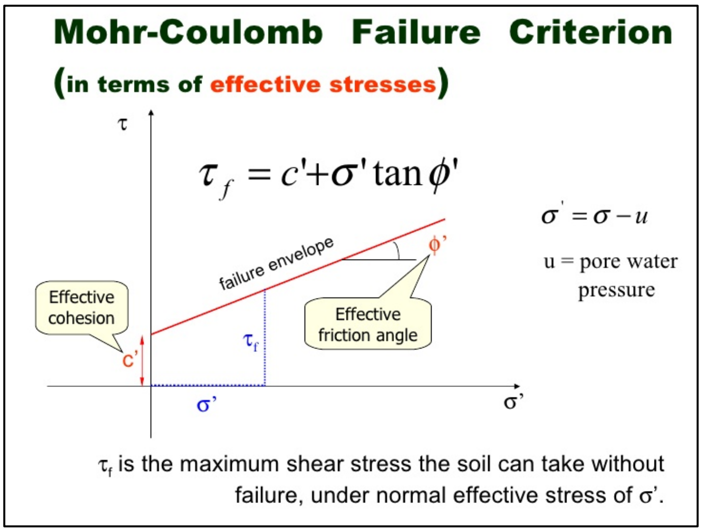

Figure 2.5: Mohr-Coulomb failure criterion

Based on Mohr-Coulomb failure criterion as shown in figure 2.4, we can see that τf is the maximum shear stress of the soil can take without failure and under normal stress. Basically, shear consist of two components which are cohesion and friction (N. Sivakugan, 2001). From equation in figure 2.4, c' refers to the cohesive component while tan ϕ refers to the friction component. Cohesion is the attraction between the soil particles that hold them together without the application of external forces. The results will be in reaction force, R when an object is subjected to both horizontal and vertical forces

CHAPTER 3

METHODOLOGY

3.1 Flow of Study

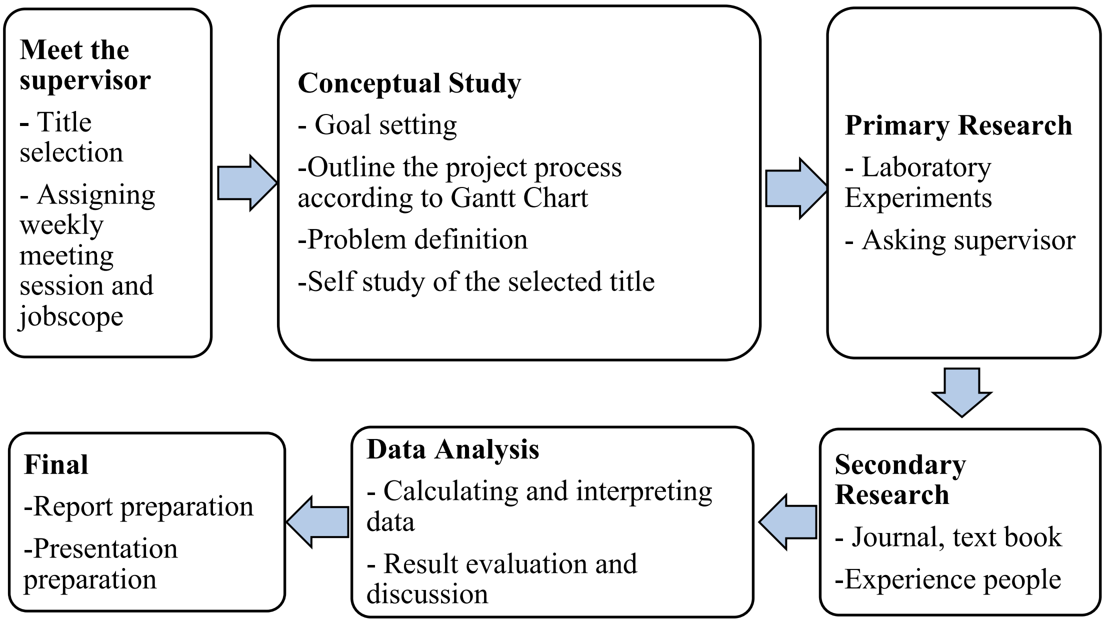

Figure 3.1: Flow of Study

3.2 Research Work Procedures

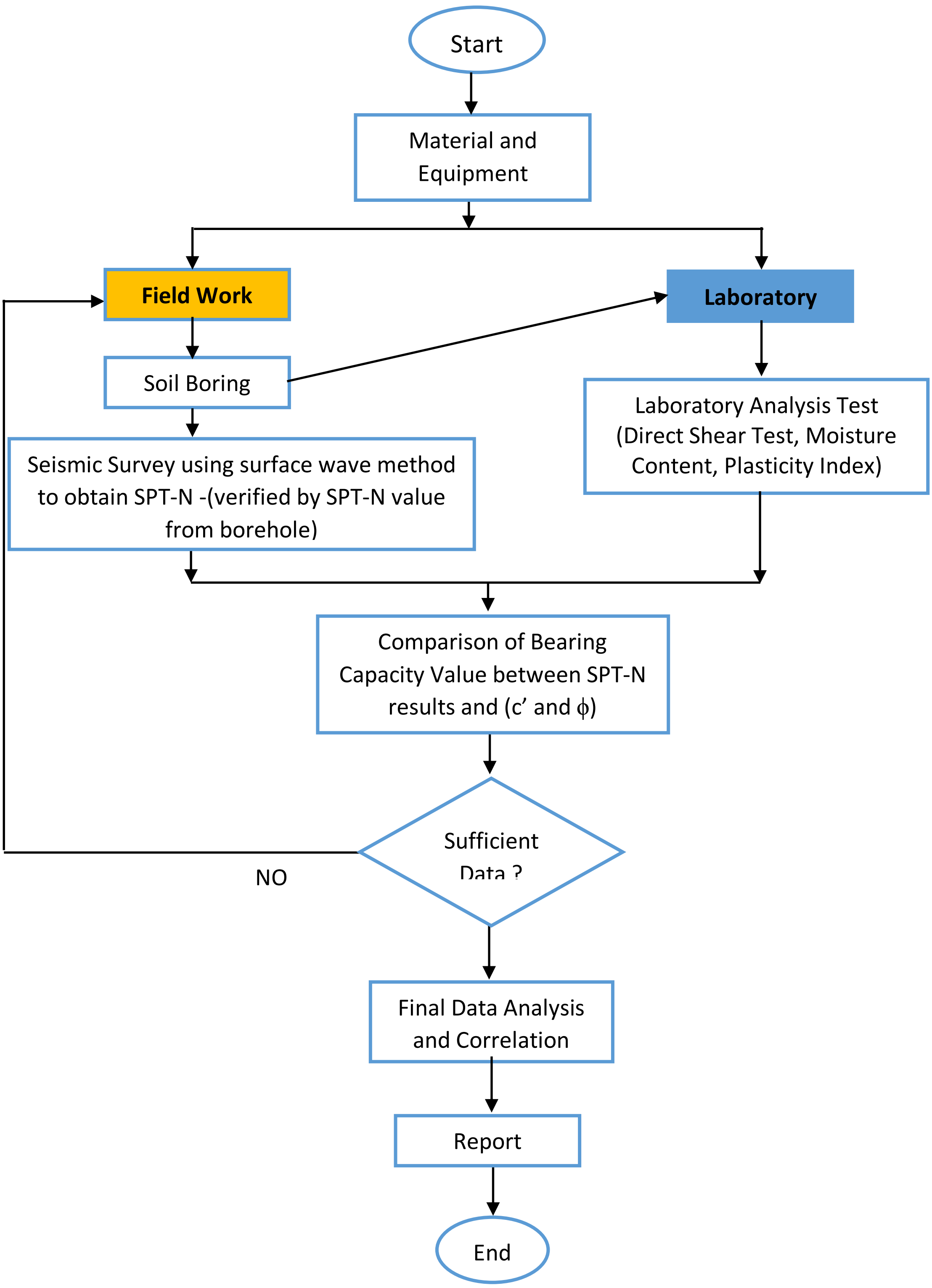

Figure 3.2: Research work procedures

3.2.1 Soil Boring

In this research, several soil samples were collected by using drilling method at six locations in Malaysia such as Cameron Highland, Damansara, Melaka, Pekan and Perlis. The equipment that has been used for soil drilling is fully hydraulic percussion drilling rig as shown in figure 3.3. Usually, the samples were taken at borehole point along the line that have been conducted seismic survey and resistivity test. The fieldwork locations have been chosen randomly and no specifications on the type of soil. The soil samples were brought to the laboratory to conduct a few laboratory tests. Figure 3.4 shows the extruding process of undisturbed sample using extruder machine.

Figure 3.3: Fully Hydraulic Percussion Drilling Rig

Figure 3.4: Extruding sample process using extruder machine

3.2.2 Moisture Content Reading

The moisture content reading was taken for each soil sample using oven drying method when the collected samples were brought to the laboratory. Generally, it is defined as the ratio of the weight of water to the weight of the dry soil grains in a soil mass. Besides, the water content is a good indication of the strength of clay soils.

Figure 3.5: Weighing soil sample to obtain moisture content value

3.2.3 Atterberg Limit Test

Atterberg Limit Test or also known as plasticity index (PI) consist of liquid limit and plastic limit tests and both were performed to obtain plasticity index values of soil samples. The liquid limit of a soil is determined by Casagrande's liquid device and is defined as the moisture content at which a groove closure of 12.7 mm occurs at 25 blows while the plastic limit is defined as the moisture content at which the soil crumbles when rolled into a thread of 3.2 mm in diameter.

Figure 3.6: Plastic Limit Test

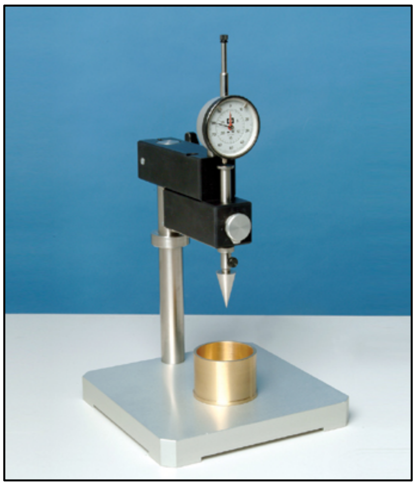

Figure 3.7: Fall cone apparatus used in Liquid Limit Test

Figure 3.8: Liquid Limit Test

The plasticity index is the difference between the liquid limit and the plastic limit of a soil. The equation to obtain the plasticity index is as follow:

$$PI = Liquid\ Limit - Plastic\ Limit$$

3.2.4 Seismic Survey Test to Determine the SPT-N value

Seismic Survey Test has been conducted at all fieldwork locations such as Cameron Highland, Damansara, Melaka, Pekan, Perlis and Universiti Teknologi PETRONAS (UTP). Before going to fieldwork, all the seismic survey equipment has been prepared and checked for their functionality as shown in figure 3.9. The tools and equipment required for seismic survey are McSEIS-SXW, takeout cables, receivers, mallet and battery.

Basically, in this research, the total number of geophones that being used is 24 with a constant spacing of 3-m for each profile. Illustration on the seismic wave acquisition are shown in figure 3.8. Sledge hammer with a weight of 8-kg was used to slam on the steel plate with dimension of 20-cm x 20-cm x 5-cm. There are total of three shot along the seismic line with 10 to 15 slam were carried out for each location of shot. The first and second of the shot point were 25m offset from two ends of the geophone array while the third shot point was located at the middle of the array. P wave and s wave were generated from the source point due to the slam of the steel plate by the sledge hammer.

As shown in the figure 3.8, 24 channel of ABEM Terraloc MK8 seismograph recording system and 4.5-Hz of vertical component geophones are being used to obtain the data on surface waves (Rayleigh waves), incident waves, reflected waves and refracted waves. The duration of each slot was set to 1024-ms with a sampling interval of 0.5-ms and the number of samples per trace was 2048. The detector is place along the straight line with difference in the distance from the source of wave. The velocity of the wave will increase as the wave travel deeper of the subsoil surface.

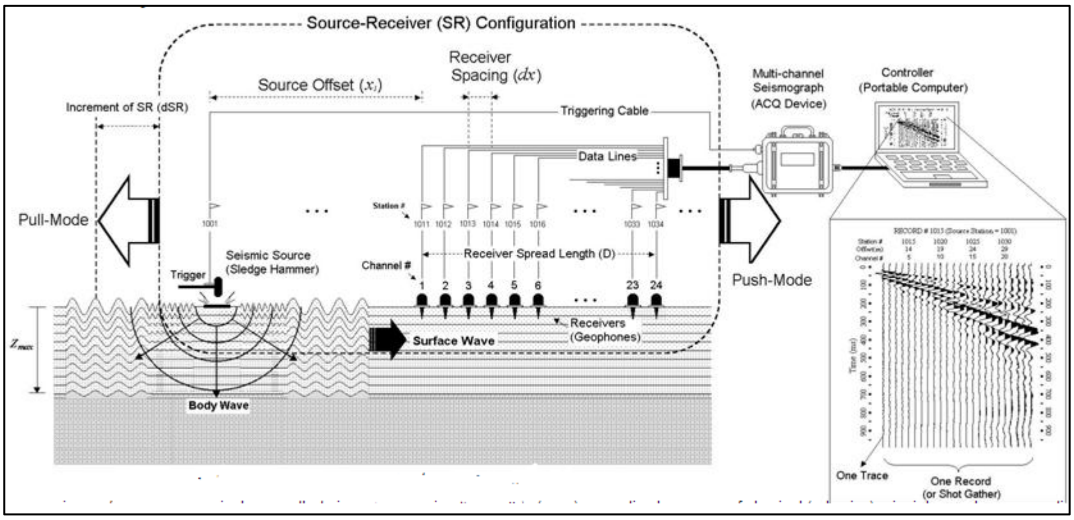

Figure 3.8: Schematic of all field set up of seismic survey


Figure 3.9: Equipment of Seismic Survey

Figure 3.10: Seismic Survey Test

Figure 3.11: Seismic Survey Test conducted at UTP

3.2.5 Direct Shear Test

The purpose of conducting direct shear test in this research is to obtain the shear strength parameters (c' and ϕ) value. The test equipment consists of a metal shear box in which the soil specimen is placed. The soil specimens may be square or circular in plan but for this research, circular soil specimen has been used. The size of the specimen is 102-mm x 102-mm across and about 25-mm high. The box is split horizontally into halves. Normal force on the specimen is applied from the top of the shear box. Shear force is applied by moving one-half of the box relative to the other to cause failure in the soil specimen. The incremental loads have been applied three times for each sample which are 2-kg, 4-kg and 6-kg. The direct shear test was conducted for all the samples from fieldwork.

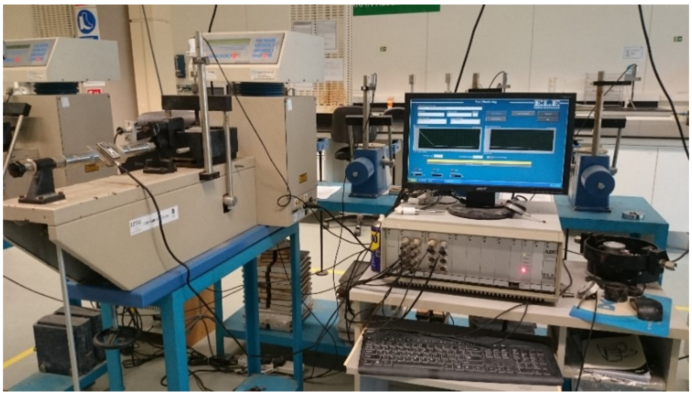

Figure 3.12: Direct Shear Test Machine

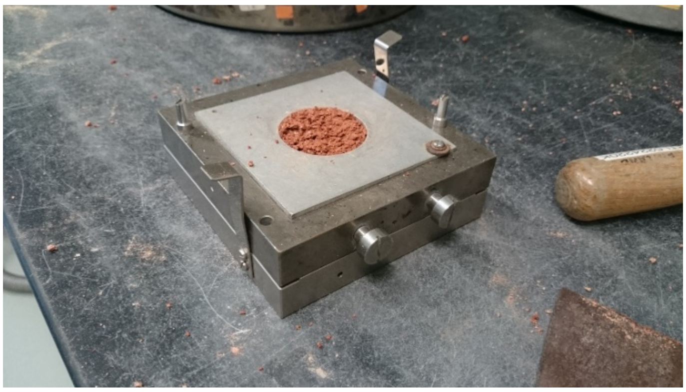

Figure 3.13: Circular soil specimen has been used for all samples

Figure 3.14: Sample and material preparation before start the test

3.2.6 Bearing Capacity Calculation Spreadsheet based on SPT-N value

MAYERHOF Formula :					K1	K2					
Skin, Ps =		K1 x Nav x As		SAND	2.0	400					
Bearing, Pb =		K2 x Nb x Ab		SILT	2.5	250					
Ult. Load, Pu =		Ps + Pb		CLAY	3.0	100					
All. Load, Pa =		Ps/Fs + Pb/Fb									
Factor of safety :			Skin, Fs =	2							
			Bearing, Fb =	3							
-	-										
To calculate allowable soil bearing capacity :											
Pile used :		Type :		s							
(c = circular)		Width, b =		0.4	m Perimeter:	1.600					
(s = square)		(= dia. for type c)			Ab:	0.160					
					Qa(conc):	1800					
=	=	=	=	=	=	=		=	=	=	=
Depth	Soil Type	SPT N	As	Skin Fric	Total Skin	Base Load	Working Load Fs = 2.0 Fb = 3	Ultimate Load	Allowable Load	Total Length	No of Pile
(m)			(m²)	(kN)	(kN)	(kN)	(KN)	(kN)	(kN)	(m)	
-	-	-	-	-	-	-		-	-	-	-
0.0	CLAY	0	0.0	0.0	0.0	0.0	0.0	0.0	0.0		
1.0	CLAY	3	1.6	14.4	14.4	48.0	23.2	62.4	23.2	77.6	77.6
2.0	SILT	5	3.2	40.0	54.4	200.0	93.9	246.7	86.2	41.8	20.9
3.0	SILT	5	4.8	60.0	114.4	200.0	123.9	302.9	112.3	48.1	16.0

For a designed load of 100 KN, the estimated depth is 16.5m.

Note : 1. The clay skin friction for the top soil layer is assume to be 7.5kN/m2

2. The pile is designed as floating pile where the pile would settle together with the soil and therefore there is no effective negetive skin friction.

3. The final depth of piles penetration shall be decided on site by load test.

Figure 3.15: The bearing capacity calculation spreadsheet based on SPT-N value

The spreadsheet of bearing capacity calculation based on SPT-N value has been created to easier calculate the bearing capacity of soil as shown in figure 3.15. Besides, the spreadsheet has been created using Meyerhof formula. The pile size is assumed to be square RC pile and the dimension is about 0.4-m x 0.4-m. The factor of safety used for skin friction, Fs is 2 and for bearing, Fb is 3. This spreadsheet was used to calculate the bearing capacity at all fieldwork locations.

3.2.7 Bearing Capacity Calculation Spreadsheet based on Strength Parameters (c’ and ϕ)

BEARING CAPACITY CALCULATION BASED ON STRENGTH PARAMETERS (C AND φ)							
MEYERHOF FORMULA:							
$q_u = c'N_c F_{cs} F_{cd} F_{ci} + qN_q F_{qs} F_{qd} F_{qi} + 0.5\ \Upsilon B N_{\Upsilon} F_{\Upsilon s} F_{\Upsilon d} F_{\Upsilon i}$							
Strength Parameters:			Data:				
C'	12.21		ϒ (kg/m3)	12.42		L	2
φ	23.08		B (m)	2		FS	3
φ (rad)	0.402823		Df (m)	3.0			
			q (q=Df x ϒ)	37.26			
Bearing Capacity Factors:							
N_q =	8.73						
N_c =	18.15						
N_{Υ} =	8.29						
Shape, Depth, Inclination Factors:							
				Df/B > 1	for φ > 0;		
Shape :	F_{cs} =	1.4812	Depth :	F_{qd} =	1.3096		
	F_{qs} =	1.4261		F_{cd} =	1.3496		
	$F_{\Upsilon s}$ =	0.6		$F_{\Upsilon d}$ =	1		
q_u (kN/m2) =	1112.472						
q_{all} (kN/m2) =	370.8239						
Q (kN) =	1483.296						

Figure 3.16: The bearing capacity calculation spreadsheet based on (c’ and ϕ)

Figure 3.16 shows the spreadsheet of bearing capacity calculation based on strength parameters (c’ and ϕ). The spreadsheet has been created using Meyerhof formula. This spreadsheet was used to calculate the bearing capacity based on strength parameters (c’ and ϕ) obtained from direct shear test.

The size of foundation was assumed to be square with the dimension of 2-m x 2-m. For this research, the inclination factor can be ignored since all the samples were taken at the flat area. Besides, the length of foundation used is approximately 3-meter since the samples were taken at maximum depth of 3-meter.

CHAPTER 4

RESULT AND DISCUSSION

4.1 Moisture Content

Table 4.1: Original moisture content at fieldwork locations

SAMPLE			MOISTURE CONTENT (%)
CAMERON HIGHLAND	BH1	1-m	24.88
		2-m	22.45
		3-m	23.24
	BH2	1-m	20.92
		2-m	24.84
		3-m	26.69
DAMANSARA	BH1	1-m	16.48
		2-m	15.38
		3-m	16.48
	BH2	1-m	18.95
		2-m	16.46
		3-m	21.4
MELAKA	BH1	1-m	74.11
		2-m	90.68
		3-m	100.43
	BH2	1-m	24.31
		2-m	71.61
		3-m	91.72
PERLIS	BH1	1-m	71.52
		2-m	116.17
		3-m	86.30
	BH2	1-m	69.99
		2-m	91.86
		3-m	95.65
PEKAN	BH1	1-m	57.34
		2-m	93.90
		3-m	52.22

	BH2	1-m	69.97
		2-m	69.10
		3-m	46.77

Table 4.1 shows the original moisture content values obtained once the soil samples were collected from a few fieldwork locations. There are ten samples have been collected from five locations. From the observation, the location that has the highest moisture content is Perlis with the moisture range around 69.99% to 116.17%. Besides, the location that has the lowest moisture content is Damansara with the moisture range around 15.38% to 21.4%.

4.2 Plasticity Index

Table 4.2: Liquid Limit, Plastic Limit and Plasticity Index of all samples

Sample			Liquid Limit (%)	Plastic Limit (%)	Plasticity Index (%)
Cameron Highland	BH1	1-m	50.00	38.5	11.50
		2-m	52.00	41.99	10.01
		3-m	52.00	42.16	9.84
	BH2	1-m	52.00	37.23	14.77
		2-m	52.00	36.48	15.52
		3-m	54.00	40.48	13.52
Damansara	BH1	1-m	47.83	28.71	19.12
		2-m	53.82	31.2	22.62
		3-m	46.91	25.86	21.05
	BH2	1-m	58.56	35.71	22.85
		2-m	56.59	33.99	22.60
		3-m	60.18	39.68	20.50
Melaka	BH1	1-m	46	26.55	19.45
		2-m	74	43.33	30.67
		3-m	66	27.37	38.63
	BH2	1-m	39.00	21.14	17.86
		2-m	62.00	38.45	23.55
		3-m	61.00	38.48	22.52

Perlis	BH1	1-m	51.00	28.68	22.32
		2-m	51.00	24.72	26.28
		3-m	41.00	21.28	19.72
	BH2	1-m	55.00	32.47	22.53
		2-m	49.00	35	14
		3-m	45.00	26.7	18.3
Pekan	BH1	1-m	67.00	41.2	25.8
		2-m	54.00	35.9	18.1
		3-m	38.00	29.26	8.74
	BH2	1-m	53.00	32.47	20.53
		2-m	54.00	30.53	23.47
		3-m	37.00	27.12	9.88

Plastic limit and liquid limit tests were done to all fieldwork samples after the samples were oven dried. Plasticity Index test values were obtained from the equation below: -

$$PI = Liquid\ Limit - Plastic\ Limit$$

The plasticity index is expressed in percent of the dry weight of the soil sample. It shows the size of the range of the moisture contents at which the soil remains plastic. Generally, the plasticity index depends only on the amount of clay present and it indicates the fineness of the soil and its capacity to change the shape without altering its volume. Thus, high plasticity index indicates an excess of clay in the soil

Table 4.2 shows the result of liquid limit, plastic limit and plasticity index for all the fieldwork locations. The plasticity index in Cameron Highland was in a range of 9.84% to 15.52%. Besides, the range of plasticity index for other locations are 19.12% to 22.85 in Damansara, 17.86% to 38.63 in Melaka, 14% to 26.28 in Perlis and 8.74% to 25.8 in Pekan. Based on the plasticity index value, the locations that have silty soils are Cameron Highland and Pekan as they have the plasticity index value less than 10%. Since the other locations have plasticity index more than 11%, therefore they contain of clayey soils.

4.3 Direct Shear test

The purpose of conducting direct shear test in this research is to determine the value of strength parameters (c' and ϕ) of soil from all fieldwork locations. The results of strength parameters (c' and ϕ) obtained from direct shear test are shown in figure 4.3

Table 4.3: Strength Parameters obtained from direct shear test

SAMPLE	BH	DEPTH (m)	Strength Parameters	
			C (kPa)	ϕ
Cameron Highland	1	1	87.06	41.48
		2	46.23	13.48
		3	42.06	48.75
	2	1	88.52	43.80
		2	44.50	44.70
		3	40.08	25.78
Damansara	1	1	22.3	47.81
		2	31.9	47.43
		3	50.93	48.96
	2	1	27.72	48.03
		2	46.67	68.82
		3	5.79	68.76
Melaka	1	1	33.22	29.22
		2	26.51	21.78
		3	10.36	12.31
	2	1	33.83	13.06
		2	25.62	24.41
		3	17.73	21.02
Perlis	1	1	25.28	19.37
		2	16.47	47.35
		3	10.69	9.08
	2	1	32.70	23.34
		2	17.30	19.97
		3	11.32	9.08
Pekan	1	1	39.01	13.10
		2	26.86	7.78
		3	26.03	10.22

	2	1	17.63	8.12
		2	19.70	10.01
		3	17.62	8.58

4.4 Seismic Survey Test

Seismic survey test has been conducted at a few fieldwork locations to determine the SPT-N value to be used in bearing capacity calculation. Table 4.4 shows the results of SPT-N obtained from seismic survey test using surface wave method.

Table 4.4: SPT-N value obtained from seismic survey test

SAMPLE	BH	DEPTH (m)	SPT VALUE, N	CONSISTENCY
Cameron Highland	1	1	3	Soft
		2	5	Firm
		3	5	Firm
	2	1	3	Soft
		2	5	Firm
		3	6	Firm
Damansara	1	1	14	Stiff
		2	14	Stiff
		3	14	Stiff
	2	1	12	Stiff
		2	12	Stiff
		3	17	Very stiff
Melaka	1	1	3	Soft
		2	4	Soft
		3	4	Soft
	2	1	3	Soft
		2	4	Soft
		3	4	Soft
Perlis	1	1	4	Soft
		2	3	Soft
		3	3	Soft
	2	1	4	Soft
		2	3	Soft
		3	3	Soft

Pekan	1	1	2	Very soft
		2	1	Very soft
		3	2	Very soft
	2	1	2	Very soft
		2	1	Very soft
		3	2	Very soft

From the table 4.4, we can conclude that the soils at Damansara has highest SPT value for 3-meter depth compare to other locations. The value of SPT at Damansara are in a range of 12 to 17. The soils at other locations have a close range of SPT value which is around 1 to 5. The soils at Pekan has the lowest value of SPT at depth of 2-meter which is 1. It means that the soils for 3-meter depth in Pekan are very soft compare to other locations.

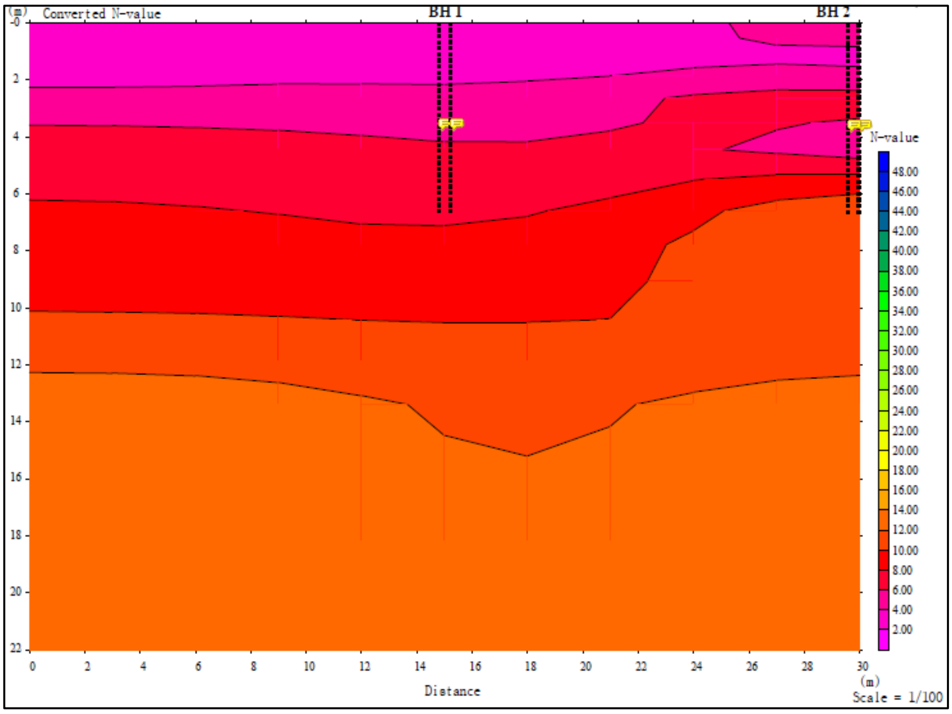

Figure 4.1: Graph of SPT-N value obtained from seismic survey test at Cameron Highland

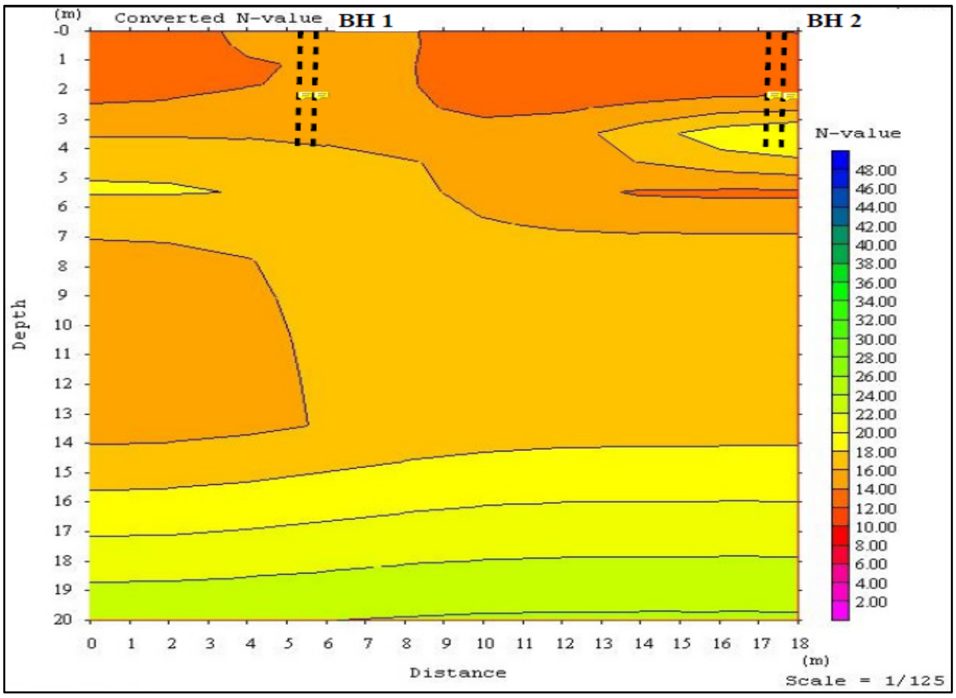

Figure 4.2: Graph of SPT-N value obtained from seismic survey test at Damansara

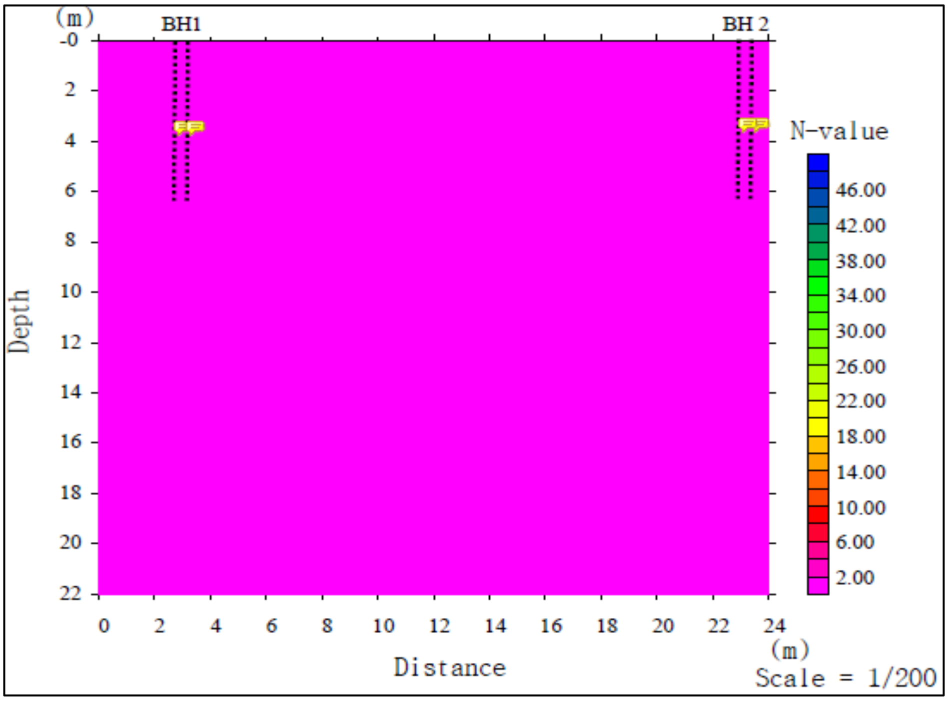

Figure 4.3: Graph of SPT-N value obtained from seismic survey test at Melaka

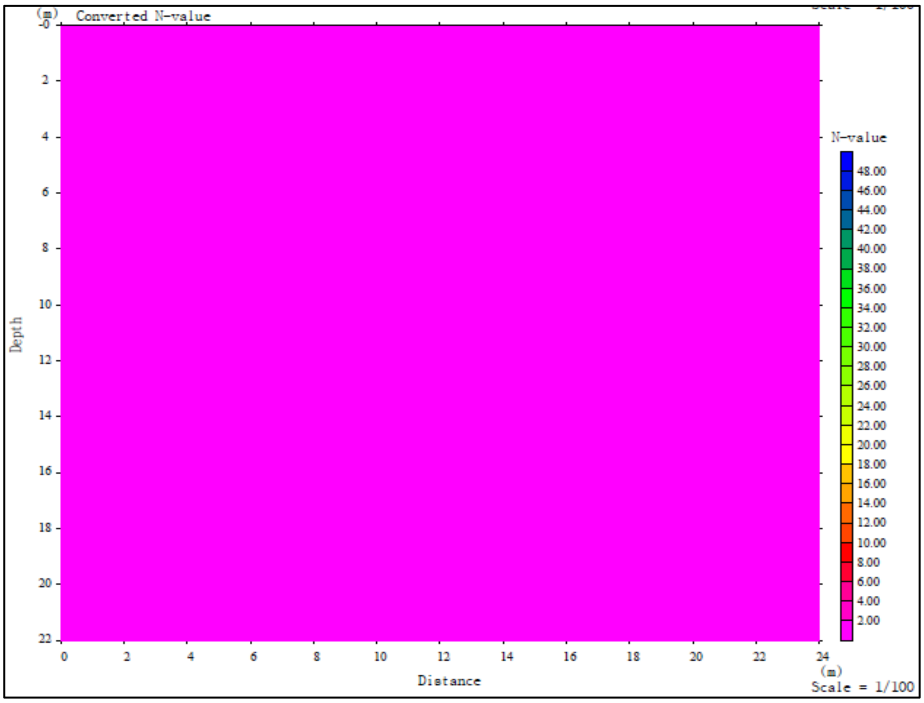

Figure 4.4: Graph of SPT-N value obtained from seismic survey test at Perlis

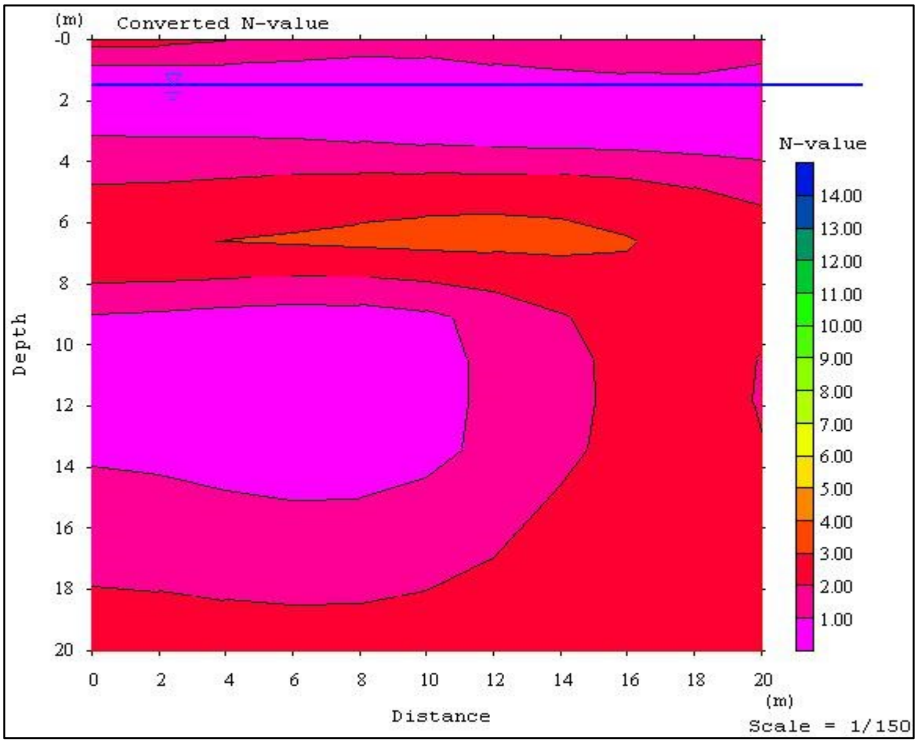

Figure 4.5: Graph of SPT-N value obtained from seismic survey test at Pekan

Figure 4.1 to figure 4.5 show the graph of SPT-N value obtained from seismic survey test at a few fieldwork locations. The consistency of soils at Cameron Highland is soft and firm with the value of SPT-N in a range of 3 to 6. Besides, the consistency of soils at Melaka and Perlis are soft. The soils at Damansara are stiff to very stiff compare to other locations since it has the SPT-N value in a range of 12 to 17. Unfortunately, the soils at Pekan is very soft since the value of SPT-N in a range of 1 to 2.

4.5 Correlation of Bearing Capacity between SPT-N from Seismic Survey and Strength Parameters (c' and ϕ)

Table 4.5: Bearing Capacity obtained based on SPT-N from seismic and (c' and ϕ)

Sample	Q SPT-N Seismic (kN)	Q (c' and ϕ)
Cameron Highland, Damansara, Melaka, Perlis and Pekan	93.9	64.81
	106.4	156.32
	47.7	69.80
	76.5	16.47
	23.2	52.30
	47.7	91.63
	76.5	49.94
	30.9	59.05
	61.6	13.49
	30.9	100.46
	40	44.66
	61.6	14.09
	15.5	50.85
	14.9	24.79
	48.3	30.72
	15.5	18.57
	14.9	23.42
	48.3	20.51
	98	32.52
	95.3	32.52
	98	151.69
	153.3	151.69
	350.3	151.69
	98	47.32
	134	47.32

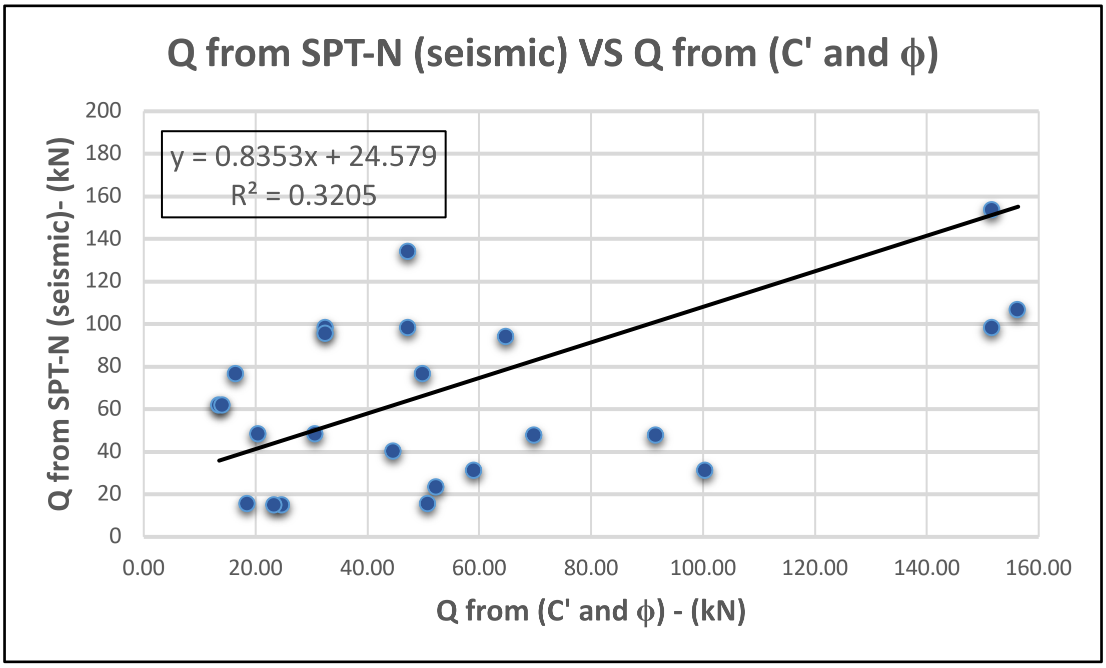

Figure 4.6: Correlation of Bearing Capacity, Q from Seismic Survey and (c' and ϕ)

Figure 4.6 shows the correlation of bearing capacity between SPT-N from seismic survey and (c' and ϕ) from soil drilling. The bearing capacity value from both parameters are directly proportional. The linear trend between bearing capacity from seismic survey and (c' and ϕ) is established but the regression value is relatively low (R^2=0.3205).

Table 4.5 shows the bearing capacity obtained based on SPT-N from seismic survey and (c' and ϕ). The data used for this correlation have been gathered from all fieldwork locations except Universiti Teknologi PETRONAS (UTP) since soil boring has not been conducted at UTP. Thus, there are no sample from UTP for conducting laboratory test. Besides, some of the data have been isolated from the graph since they were not behaved appropriately.

4.6 Correlation of SPT-N from Seismic Survey and SPT-N from boreholes

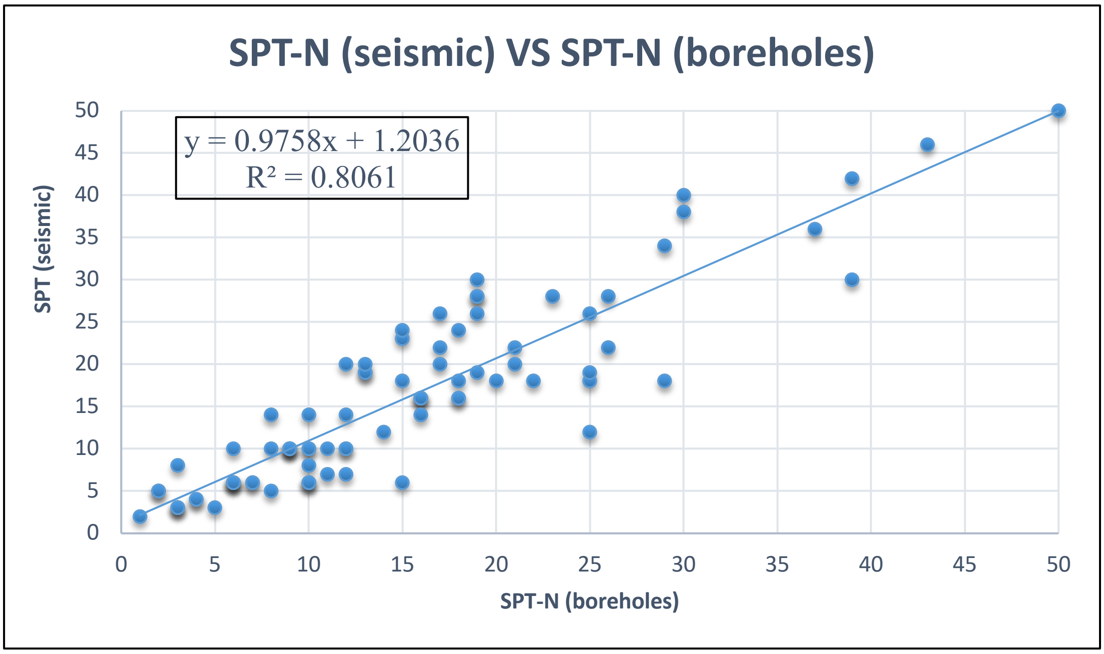

Figure 4.7: Correlation of SPT-N value from seismic survey and SPT-N boreholes

The correlation of SPT-N value from seismic survey and SPT-N from boreholes only used the data from UTP since the data of SPT-N from boreholes only can be gathered from UTP site. The SPT from boreholes have been conducted by the previous contractor and those data were used in this research for verification purpose. The main reason of conducting seismic survey at UTP is to verify the reliability of SPT-N value of seismic survey from SPT-N value of boreholes.

Figure 4.7 shows the correlation of SPT-N value from seismic survey and SPT-N from boreholes. The SPT-N values from both methods are directly proportional. Besides, the correlation of SPT-N from seismic and boreholes show high regression value which has verified the reliability of SPT-N from seismic survey (R^2=0.8061).

4.7 Correlation of Bearing Capacity between SPT-N from Seismic Survey and SPT-N from boreholes

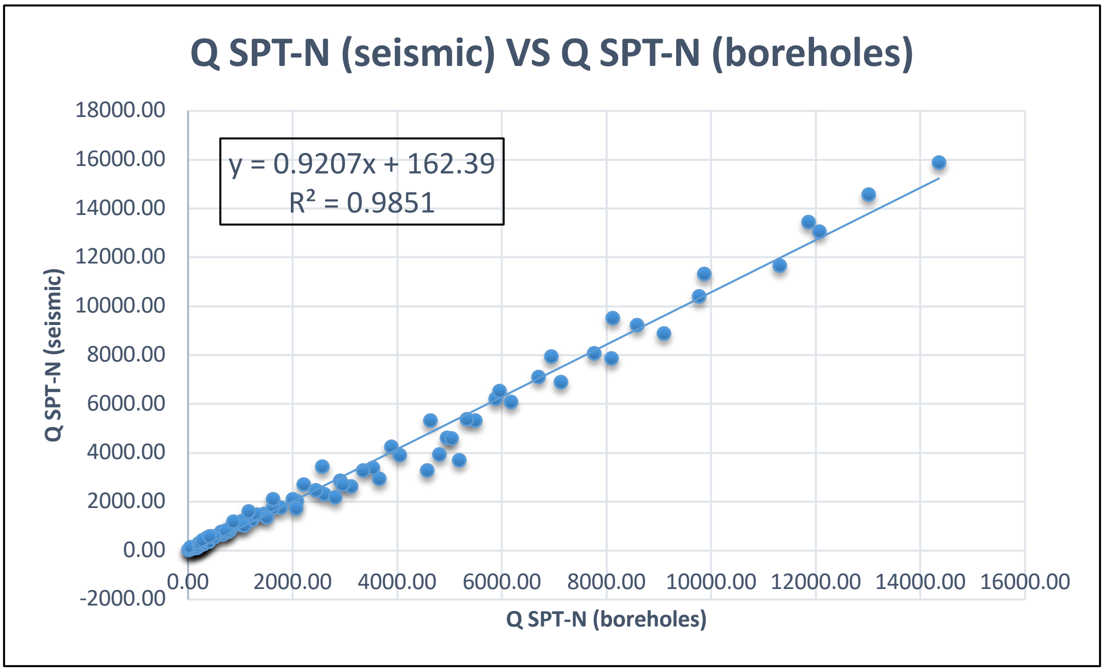

Figure 4.8: Correlation of bearing capacity,Q between SPT-N (seismic) and SPT-N (boreholes)

Figure 4.8 shows the correlation of bearing capacity between SPT-N from seismic and SPT-N from boreholes. Bearing capacity value from both different SPT method are directly proportional. The linear trend between bearing capacity based on SPT-N from seismic and SPT-N from boreholes is established and show high regression value (R^2=0.9851). The high regression value from this graph also proved the reliability of bearing capacity calculation based on SPT-N from seismic survey.

4.7 Correlation of Bearing Capacity between SPT-N from boreholes and (c' and ϕ)

Table 4.6: Bearing Capacity from SPT-N boreholes and (c' and ϕ)

SAMPLE	Q SPT Boreholes (kN)	Q (C and φ) (kN)
UTP	114.33	32.52
	98.33	32.52
	268.33	32.52
	363.00	32.52
	498.00	32.52
	954.33	151.69
	89.33	73.75
	170.00	73.75
	723.33	73.75

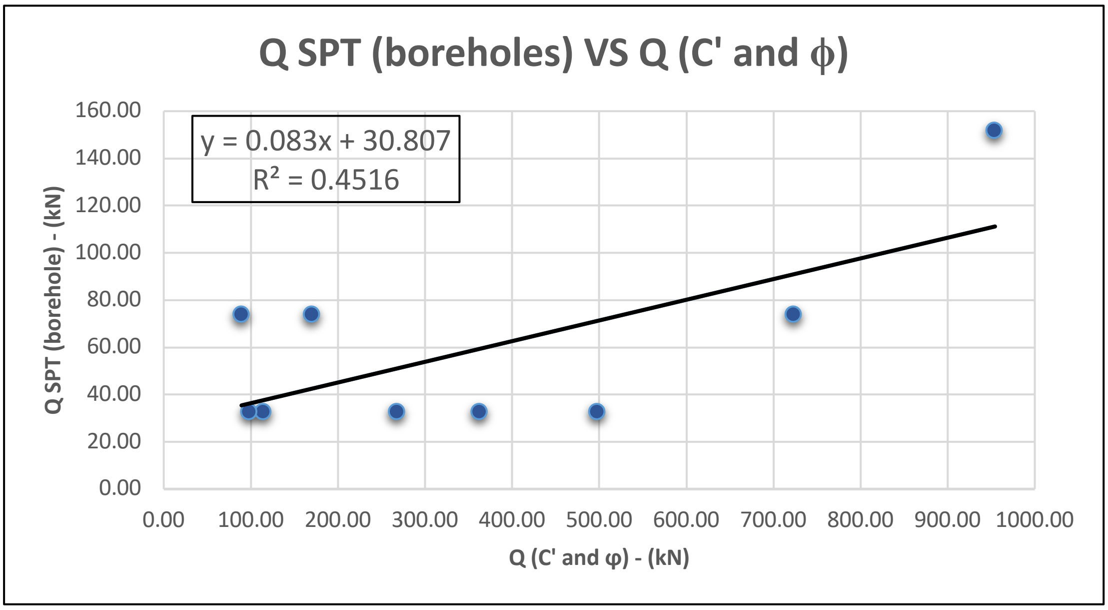

Figure 4.9: Correlation of Bearing Capacity, Q from SPT boreholes and (c' and ϕ)

Table 4.5 shows the bearing capacity value obtained based on SPT-N values from boreholes and strength parameters (c' and ϕ). This correlation was used the data from UTP site only. The data of SPT-N boreholes and (c’ and ϕ) has been determined by the previous contractor. The purpose of plotting the graph in figure 4.9 is to see the correlation value between bearing capacity from SPT-N (boreholes) and (c’ and ϕ).

Figure 4.9 shows the correlation of bearing capacity value based on SPT-N from boreholes and (c’ and ϕ). The correlation of bearing capacity from both different parameters is directly proportional. The linear trend was established on the graph but it shows the low regression value (R^2=0.4516).

CHAPTER 5

CONCLUSION AND RECOMMENDATION

5.1 Conclusion

Overall, the objectives of this research are fulfilled. Seismic survey and direct shear test were done at all selected fieldwork locations to gather the strength parameters value. SPT-N value from seismic survey and (c' and ϕ) were used in two different equation to determine the bearing capacity value.

Therefore, the overall conclusions of the research are as follow:

- The correlation of bearing capacity calculation obtained based on SPT-N from seismic survey and strength parameters (c' and ϕ) was established and the value is relatively low ($R^2 = 0.3205$).
- The correlation of SPT-N (seismic survey) and SPT-N (boreholes) shows high regression value which verified the reliability of SPT-N value from seismic survey ($R^2 = 0.8061$).
- Therefore, the correlation of bearing capacity, Q from SPT-N (seismic) and SPT-N (boreholes) also shows high regression value ($R^2 = 0.9851$).

5.2 Recommendation

There are a few recommendations can be suggested for a better result of this research. The recommendations are as follow:

- More fieldworks test should be conducted to get more data for verification of SPT-N (seismic) from SPT-N (boreholes).
- Require more samples for conducting direct shear test to get more data in correlation of bearing capacity from SPT-N (seismic) and bearing capacity from (c’ and ϕ).
- Unconsolidated Undrained (UU) Trixial Test should be conducted in the research to obtain the better results of strength parameters (c’ and ϕ).

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APPENDICES

APPENDIX A: MOISTURE CONTENT SPREADSHEET TEMPLATE

- Sample: Damansara (BH1 -1 meter)

Moisture Content				
Location - Damansara				
Soil Description				
Test Method BS 1377: Part 2: 1990: 3.2				
Related Test				
Specimen ref.				
Container no.	1	2	3	Average
Mass of wet soil + container (m_2) -g	52.30	52.10	54.90	
Mass of dry soil + container (m_3) -g	47.70	47.50	49.90	
Mass of container (m_1) -g	18.98	19.06	20.91	
Mass of moisture (m_2 - m_3) -g	4.60	4.60	5.00	
Mass of dry soil (m_3 - m_1) -g	28.72	28.44	28.99	
Moisture Content, w = [m_2 - m_3/ m_3 - m_1] 100 %	16.02	16.17	17.25	16.48

APPENDIX B: PLASTIC LIMIT CALCULATION

- Sample: Damansara (BH1 -1 meter)

PLASTIC LIMIT	Test no.	1	2	3	Average
Container no.		1	2	3	28.7145
Mass of wet soil + container	g	28.30	27.70	27.40	
Mass of dry soil + container	g	26.70	25.70	25.70	
Mass of container	g	21.15	18.98	19.53	
Moisture content	%	28.8288	29.7619	27.5527	

- Sample: Damansara (BH1 -2 meter)

PLASTIC LIMIT	Test no.	1	2	3	Average
Container no.		1	2	3	31.204
Mass of wet soil + container	g	27.10	28.60	27.10	
Mass of dry soil + container	g	25.30	26.50	25.30	
Mass of container	g	19.70	19.60	19.50	
Moisture content	%	32.1429	30.4348	31.0345	

- Sample: Damansara (BH1 -3 meter)

PLASTIC LIMIT	Test no.	1	2	3	Average
Container no.		1	2	3	25.8551
Mass of wet soil + container	g	29.90	31.00	25.40	
Mass of dry soil + container	g	28.20	29.10	24.10	
Mass of container	g	21.90	21.20	19.20	
Moisture content	%	26.9841	24.0506	26.5306	

- Sample: Damansara (BH2 -1 meter)

PLASTIC LIMIT	Test no.	1	2	3	Average
Container no.		1	2	3	35.7064
Mass of wet soil + container	g	30.00	25.80	27.80	
Mass of dry soil + container	g	27.50	24.00	25.70	
Mass of container	g	20.60	18.90	19.80	
Moisture content	%	36.2319	35.2941	35.5932	

- Sample: Damansara (BH2 -2 meter)

PLASTIC LIMIT	Test no.	1	2	3	Average
Container no.		1	2	3	33.9853
Mass of wet soil + container	g	29.00	28.10	28.00	
Mass of dry soil + container	g	27.10	25.80	26.30	
Mass of container	g	21.50	18.90	21.40	
Moisture content	%	33.9286	33.3333	34.6939	

- Sample: Damansara (BH2 -3 meter)

PLASTIC LIMIT	Test no.	1	2	3	Average
Container no.		1	2	3	39.6758
Mass of wet soil + container	g	30.90	26.10	29.50	
Mass of dry soil + container	g	28.10	24.30	27.20	
Mass of container	g	21.30	19.50	21.50	
Moisture content	%	41.1765	37.5	40.3509	

APPENDIX C: LIQUID LIMIT CALCULATION

- Sample: Damansara (BH2 -3 meter)

LIQUID LIMIT	Test no.	1			2			3		
Gauge reading	mm	9	9.4	8.9	16.2	16.4	16.7	23.4	23.6	23.9
Average penetration	mm	9.10			16.43			23.63		
Container no.		1			2			3		
Mass of wet soil + container	g	51.4			49.9			56.5		
Mass of dry soil + container	g	41.5			38.2			41.1		
Mass of container	g	21.4			19.3			18.9		
Moisture content	%	49.25373134			61.9047619			69.36936937