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Design and Fabrication of a Geo Probe for Determining the q_u/s_u of Soft Soil

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Abstract

Aim of Study

This study is to design and fabricate a geo probe that is used to determine the basic properties of soft soil.

Need of Study

Deep foundation on soft is difficult as it is vulnerable to lateral movement when it is subjected to vertical load. Due to this problem, a geo probe is designed and constructed with foam concrete to determine the condition of the soil which is based on the ultimate bearing capacity, *qu* and the undrained shear strength, *Su*.

Research Approach

Designed geo probe comprises a cylindrical hollow PVC pipe of 50mm for internal diameter and 100mm for the external diameter in filled with foamed concrete of 1200kg/m3 density. It was installed on a test site in RECESS and loaded to compute the value of ratio ultimate bearing capacity over undrained shear strength in the particular study of soil. As commonly said ultimate bearing capacity is more than undrained shear strength by five times which only happen in a certain condition.

Research Findings

This study formed a basis design of a geo probe which can be used to determine basic properties of soft soil. Experimental work showed that at shear strength of 1kpa the ratio of qu/Su is 6.7 and at 19kpa, the ratio of qu/Su is 3.1.

Limitations

This study is to evaluate and design a scale model to be tested under static loading on soft soil. The study will cover the experimentation of the scaled down the probe for field trials in UTHM at the soft soils that are located at RECESS. It will also consist of determining the ratio of qu/Su from the results that have been obtained from the experiment.

Importance and Contribution

The result of this study is expected to improve the understanding on the methods of determining the bearing capacity and the ratio of qu/Su of soft soil.

Keywords: soft soil, bearing capacity, undrained shear strength

I. INTRODUCTION

Significant proportions of land surface in some region are covered with soft deposits such as clay. These soft deposits frequently exist in areas where developments have taken place or are projected. Construction of a structure over soft ground is crucial and will have few problems to the soil.

Soft soil which include peat and organic soils commonly occur as extremely soft, wet, unconsolidated surficial deposits that are integral parts of the wetland systems [1]. Peat, which is also often described as a problematic soil, has been identified as one of the major groups of soils found in Malaysia, which, in fact, that over 3 million hectares or 8% of the area in Malaysia is covered with peat [1]. Substantial pear swamp forests have been cleared for agriculture and are under plantation of crops such as oil palm and rubber. On the west coast of Malaysian Peninsular, deposits of peat soils are the formed in the depression consisting of marine clay deposits or mixtures of marine and river deposits which especially in areas along the river coarse.

Stable growth of the economy a decade ago quickly generated various booming development in Malaysia. This growth is demanding more soil usage including land area of soft soils. Due to economic reasons, currently utilization of peat soil land in Malaysia is quite low although the construction marginal land on peat has become increasingly necessary. Engineers are reluctant to construct on peat because of the difficulty to access the site and other problems related to unique characteristics of peat [2]. Engineers are trying to find the best methods in construction and to determine the condition of the soil.

Peats and organic soils are highly compressible and organic which also has low shear strength for accommodating loading from the upper side. Peat has a volume change that is 10 times higher than in a swelling clay soil [3]. Thus, peat is a rather complex material that contains fibrous organic matter as major constituents. Peats are always taken to calculate and predict for their deformation behavior. A peat soil poses low undrained strength and high compressibility where the settlement will be very large.

Since, soft soil such as peat and clay is hard to predict where wrong assumption will result to excessive settlement that is dangerous. Peat soils generally contain fibers and minerals that tend to be high in water permeability where it is highly compressible that will cause large settlement. This makes peat soils unpredictable soil that normally been avoided during construction. Hence, peat is very much unsuitable for supporting foundation in its natural state [2].

For the construction in soft soil, many factors will influence the method of the construction. This includes the characteristics of the peat and underlying soil, construction material equipment required, available time location of structures and drainage requirement [4].

Deep foundation on soft soil is difficult as it is vulnerable to lateral movement when it is subjected to vertical load. Due to this problem, a geo probe was designed that can perform in this type of soil which is constructed by foamed concrete to have a density that will fit the environment. Geo probe is used to determine the condition of the soil which is based on the ultimate bearing capacity, qu and the undrained shear strength, Su. Hence, this project aims to design and fabricate geo probe to use in soft soil for determining the in situ properties of the soft under static loading. This method can be an alternative to engineers to know and determine the condition and properties of the peat. However, the study will cover the experimentation for field trials in UTHM at the soft soils that are located at RECESS. It will also consist of determining the ratio of qu/Su from the results that have been obtained from the experiment.

II. LITERATURE REVIEW

A. *Geo Probe*

Geo probe is a simplified version of a probe which is designed based on the criteria and important aspects that are to be tested. Shape and density of the geo probe is carefully designed to match the criteria for the soil to maximize the performance of the probe. As from the research of Fellenius [5] and Paikowsky [6], selection of location and gages are the important aspects to be considered for design. Other than that, ensure that the gages and instrument that is used will survive when the experiment is conducted. There are a number of gages that have been used and tested for geo probe instrumentation such as piezometers, pressure gages, strain gages, accelerometers, tell-tales, and thermostat. As for this study, it consists only the hollow probe and a bearing plate with rods connecting to the surface of the probe. The probe will be tested with different depth inside the soil repeatedly. Fig 2. include the prototype of geo probe.

Figure 1. Prototype of Geo Probe

B. *Cohesive Soil*

Cohesive soil is the type of soil which has higher water content and in small soil particles form which consist of silts, clays, and organic materials. Cohessionless soils which are highly permeable compared to the cohesive soils. Settlement characteristics of cohesive soils are considered only under completely saturated conditions. On the other hand, clay which is a cohesive soil is low in strength and high incompressibility. Clay consists of several minerals such as Silica Tetrahedron and Alumina Octahedrons which are the basic units to compose the clay minerals. The particle size of clay is very small which is less than 2μm and electrochemically very active. Clay minerals are produced mainly from the chemical weathering and decomposition of feldspars, such as Orthoclase and Plagioclase and some Mica.

C. *Ultimate Bearing Capacity*

Ultimate bearing capacity is the shear failure that happens in the soil when it is directed with the maximum capacity of loading that can be sustained by the soil. When a load is gradually applied on the foundation, the foundation will experience settlement. This settlement will turn to sudden failure in the soil supporting the foundation will take place, and the failure surface in the soil will extend to the ground surface. The load per unit area that causes this failure is referred as the ultimate bearing capacity. Based on Fleming (1985), two simple criteria which can be used to define the ultimate load are “the load at which settlement continues to increase without further increase in load” and “ the load causing settlement of 10% of the probe diameter (base diameter)” which is the loading that will cause the shear failure of the ultimate bearing capacity. This theory is supported with a statement from British Standard where the ultimate bearing capacity is the load which the resistance of the soil become mobilized and at a load greater than the ultimate bearing capacity the soil undergoes shear failure, allowing the probe to penetrate into the ground.

Based on British Standard 8004-1986, the ultimate bearing capacity of foundations on non-cohesive soils depends not only on the angle of shearing resistance of the soil but also increases with the breadth and the depth of embedment of the foundation.

D. *Undrained Shear Strength*

Determination of shear strength parameters for organic soils is important and somehow a difficult job in geotechnical engineering. For organic soils, several methods have been used to determine the undrained shear strength in laboratory which is the triaxial test, shear box test and vane shear test and for a field test which include the field vane and Dutch Cone Penetration test [1].

Shear strength of soil is divided into two conditions which is drained and undrained. In this study, undrained shear strength is the one of the main properties that are taken into consideration.

Undrained shear occurs when the pore water is unable to drain out of the soil as the rate of the loading is much quicker than the rate at which the pore water is able to drain out of the soil. As a result, most of the external loading is taken by the pore water which increases the pore water pressure. Existence of undrained condition of the soil depends on the soil type, geological formation and the rate of the loading. The shear strength of soil under undrained condition is called the undrained shear strength.

The undrained shear strength is very important parameter in geotechnical properties of the soil. In theoretical view of an undrained shear strength of the probe is based on the skin friction of the probe that is in contact with the soil. In this study, the undrained shear strength will be obtained from both vane shear strength and calculation based on the theoretical equation.

E. Vane Shear Test

Field vane shear test is one of the most common in-situ methods for the estimation of the undrained shear strength of peat soil. Field vane shear test is conducted according to the BS1377.

The vane shear apparatus consists of four blades on the end of the rod as the height, H, of the vane of the apparatus is twice the diameter, D. The vane can be either rectangular or tapered. The vanes of the apparatus are pushed vertically into the soil without disturbing the soil appreciably. Torque is applied at the top of the rod to rotate the vanes at a standard rate of 0.10 per second. This rotation will give the result of the failure in the soil of cylindrical shape surrounding the vanes. The maximum torque, T, that is applied to cause shear failure of the soil is measured and using the following equation to determine the undrained shear strength of the soil [7]

This test is used primarily in fine grained soils. In addition, the soil should be free of gravel or large shell particles which would influence the test results. Vane shear test attempts to provide a direct measurement of undrained shear strength. Hanzawa et al, states that field vane shear test do not provide the appropriate shear strength of peaty soil for design use, Su and it is widely known that the vane shear strength is unsafe.

III. PREPARATION OF TEST SITE

The location of the testing is determined after a visual survey on the soil around UTHM area. The test site will be located in UTHM, which is located at the RECESS research center. The preparation of the in situ site to be tested will be digging out a hole until the ground water level of the soil to have a reluctantly soft soil that will be suitable to be tested using the probe. The ground water level of the test site will be identified first before the preparation of the site take place. A test will be done in a day or today two day time so that the result that will be taken will not vary much due to changes of the soil condition due to rain. Test is also done on other sample of soil such as sand and prepared sample that is located in the RECESS to have a comparison of the result the will be obtained. Figure 2 shows the sample of soil which is clayey soil type and figure 3 is a sample of sand that is tested.

IV. DESIGN OF THE SCALED DOWN MODEL OF GEO PROBE

Geo Probe is designed to take the loading that is to be transferred into the soil to obtain the load resistance. The design of the scaled down composite probe is not fully based on the prototype of the probe; it is designed to be smaller in size and will be in a circular design which is same as a spun probe. The outer and inner circle of the probe is protected by a layer of PVC pipes that are use as a mold and protection for the foamed concrete which has lower strength capacity. Figure 4 shows the outline of how the probe is designed with the dimension and details. The sizes of the PVC pipe are of 50mm and 1000mm diameter.

Figure 2. Test site at RECESS

Figure 3. Geo Probe on test

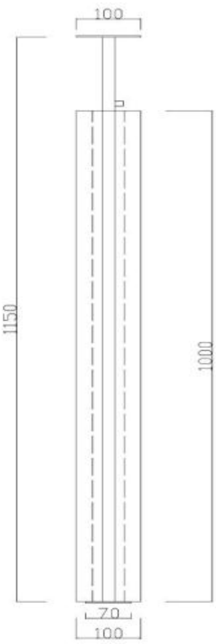

Figure 4. Typical cross section of geo probe

The scaled down probe will be much smaller than the prototype as it will be only about 1m long and 100 mm in diameter which is designed based on the effect of density and shape of the probe. The probe will be in a controlled density which is 1200kg/m^3 as shown in Fig 5 and Fig 6. A probe key is designed so that the loading can be alternated to measure the skin resistance and the tip resistance of the probe. A steel bar will be located under the key to make the alternate adjustment for the two tested value as shown in Fig 7. The bearing probe which is the steel bar in the middle of the probe which connect the bearing plate and loading plate is using a 20mm reinforcement bar which is chosen to resist any bending that will affect the result that should be obtain. Bearing plate and loading plate is using a 3mm thickness plate which is sufficient enough to resist any deformation that will be caused when it is loaded. This is an alternative method to obtain the undrained shear strength and bearing capacity based on the soil mechanics principles.

Figure 5. Foamed concrete for casting

Figure 6. Concrete casted into pipe

Figure 7. Modeled Geo Probe

V. TESTS ON GEO PROBE

The scaled down composite probe will be tested on the test site to obtain the value of Su/qu ratio which is commonly mentioned that qu is 5 times higher than Su for soft soil. The test will be repeated twice on every site to find similarity between the tests results and to get an average reading of the test. The procedure of the testing will be divided into two which is testing for the tip resistance which will be used to calculate the bearing capacity and skin resistance which is for undrained shear strength. The testing procedure is as follows.

A. Tip Resistance:

i. As the movement is only the bearing probe, a paper is stick to the upper of the bearing probe and the initial distance between the loading plate to the upper end of the probe is recorded. The upper end of the probe will be the marking to the paper.

ii. The probe was set vertically on the soft soils, and the depth of the initial penetration of the probe is recorded.

iii. The bearing plate was tested by loading it with 10kg every session and the settlement of the bearing plate is recorded. The testing continues after 5 minutes to wait until the movement of the instrument stopped. With increasing the loading by 10kg every session and the depth of settlement is recorded as shown in Fig 8.

Figure 8. Testing of Geo Probe

A steel bar is placed in between the probe lock and the probe so that the whole probe will be absorbing the load that will be applied as shown in Fig 9.

Figure 9. Placing the lock on geo probe key

B. *Skin Resistance:*

i. Marking is done on the outer section of the probe by every 50mm starting from the lowest part of the probe.

ii. The probe was set vertically on the soft soils, and the depth of the initial penetration of the probe is recorded.

iii. The probe was tested by loading it with 10kg every session and the settlement of the bearing plate is recorded. The testing continues after 5 minutes of wait until the movement of the instrument is stopped.

With increasing the loading by 10kg every session and the depth of settlement is recorded as shown in Fig 10.

Figure 10. Loading the probe

VI. DATA ANALYSIS

Data that was collected from the testing is analyzed by calculating on the ultimate bearing capacity and the undrained shear strength of the soft soil that will be used to find the qu/Su ratio. Both the result will be taken together on the same day to have an accurate result that will not be affected by the moisture of the soil. Calculation will be made based on the settlement of the probe for the end bearing which is the tip resistance and the skin friction which is the skin resistance that will be calculated to obtain the ultimate bearing capacity and the undrained shear strength of the probe.

A. *Ultimate Bearing Capacity*

Ultimate bearing capacity will be calculated based on the force, F_b that will cause the failure of soil due to the surface acting on the bearing plate to the peat soil which is the area of the bearing plate, as shown in Fig 11.

$$q_u = \frac{F_b}{A_b}$$

Where,
q_u – Ultimate Bearing Capacity
F_b – Load
A_b – Area of bearing plate

B. Undrained Shear Strength

The undrained shear strength is taken as the force that push the probe to penetrate the soil, F_u over the area of the probe in the soil as shown in Fig 11.

$$S_u = \frac{F_u}{A_u}$$

where,

S_u – Undrained Shear Strength

F_u – Shear force

A_u – Area of probe that is in contact with soil

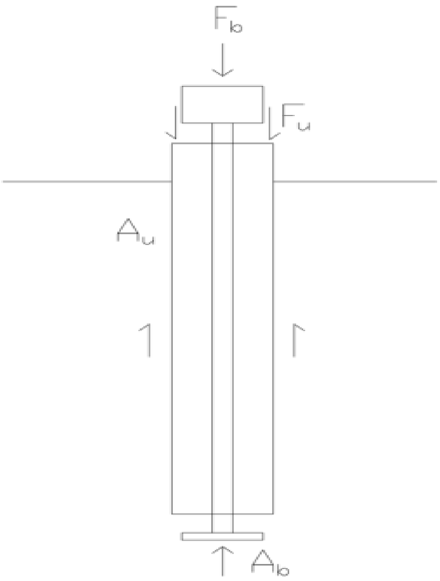

Figure 11. Ultimate Bearing Capacity and Undrained Shear Strength

VII. RESULTS AND DISCUSSION

The experimental work was carried out at RECESS UTHM on two different types of soils which include sand sample and one clay sample. Both tested sites are on disturbed sample as digging of the holes and sample location was prepared and transferred from other sites. The test only aimed the observation and determination of the ratio of the ultimate bearing capacity and the undrained shear strength. The results of tests are discussed in following sections.

A. Test 1

Test 1 was carried out on a sand sample that is located at RECESS. Test 1 result may defer from other test as sand is considered as granular soil and is not cohesive. Data was gathered in two trials of experiments which is summarized in table I and II.

Results in table I and II show that the settlement caused by the loading varied from 4mm to 18mm for the tip resistance and 1mm to 8mm for skin resistance of the probe. This is acceptable as the tip resistance has smaller size that is in contact with the sample. The undrained shear strength is lower as the settlement is lower than the ultimate bearing capacity. Relationship between depth and loading is illustrated in figure 12 and 13.

Both the first trial and the second trial give similar results with a low percentage of difference for both the ultimate bearing capacity and the undrained shear strength. Both the ratio of ultimate bearing capacity over undrained shear strength shows an average of 2.29 and 2.39. The shear strength value that was obtained for this sample using vane shear test is at range of 8kpa to 10kpa which can be concluded as an average of 9kpa. The qu/Su ratio for both trials is illustrated in figure 14 and 15.

TABLE I. EXPERIMENTAL DATA (FIRST TRIAL)

Loading (kg)	Force (kN)	Tip Resistance			Skin Resistance			
		Penetration (mm)	Cumulative	q_u (kN/m^2)	Penetration (mm)	Cumulative	A_u (m^2)	S_u (kN/m^2)
0	0	0	35	0	0	35	0.011	0
10	0.0981	6	41	20.02041	2	37	0.01162	8.43951
20	0.1962	6	47	40.04082	4	41	0.01288	15.2323
30	0.2943	10	57	60.06122	6	47	0.01477	19.9316
40	0.3924	4	61	80.08163	8	55	0.01728	22.71
50	0.4905	14	75	100.102	5	60	0.01885	26.0218
60	0.5886	18	93	120.1224	5	65	0.02042	28.8242

TABLE II. EXPERIMENTAL DATA (SECOND TRIAL)

Loading (kg)	Force (kN)	Tip Resistance			Skin Resistance			
		Penetration (mm)	Cumulative (mm)	q_u (kN/m^2)	Penetration (mm)	Cumulative (mm)	A_u (m^2)	S_u (kN/m^2)
0	0	0	36	0	0	36	0.01131	0
10	0.0981	4	40	20.02041	1	37	0.01162	8.43951
20	0.1962	4	44	40.04082	5	42	0.01319	14.8696
30	0.2943	6	50	60.06122	4	46	0.01445	20.3649
40	0.3924	12	62	80.08163	7	53	0.01665	23.5669
50	0.4905	16	78	100.102	5	58	0.01822	26.9191
60	0.5886	15	93	120.1224	8	66	0.02073	28.3875

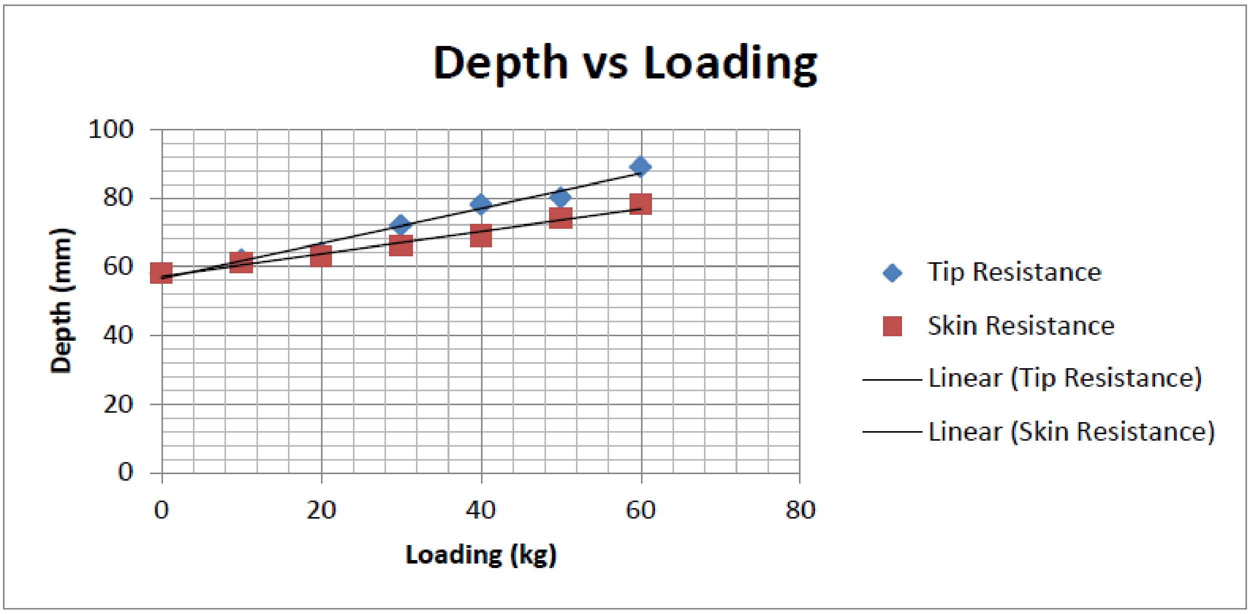

Figure 12. Graph of Depth vs Loading (First Trial)

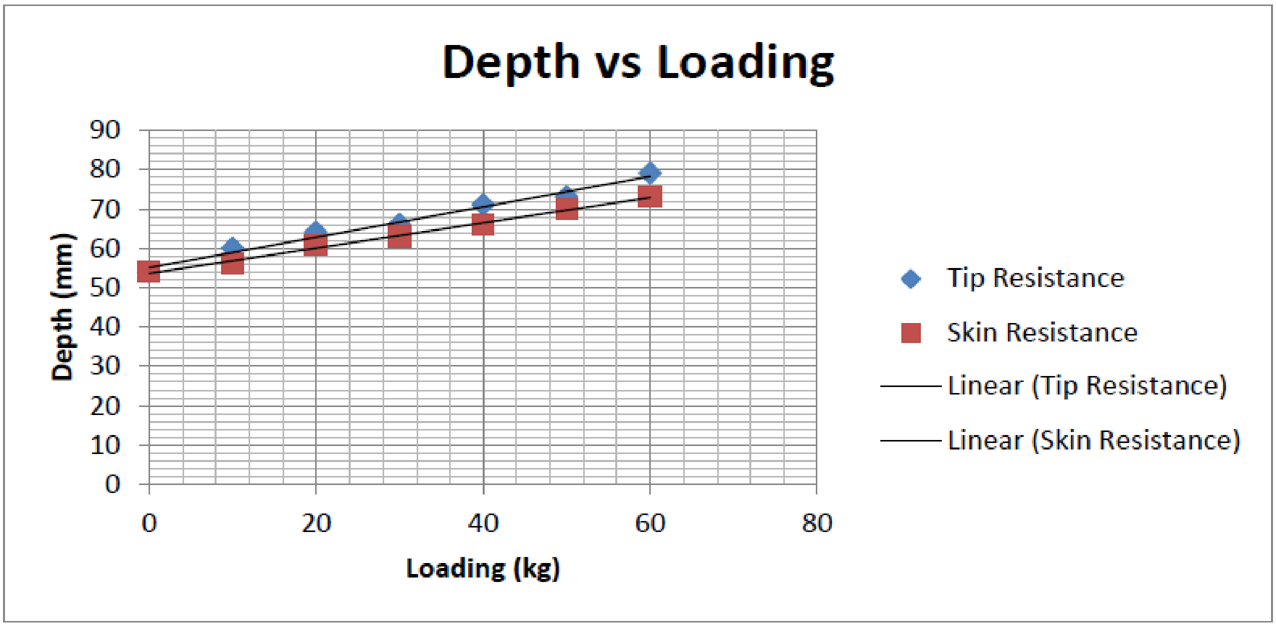

Figure 13. Graph of Depth vs Loading (Second Trial)

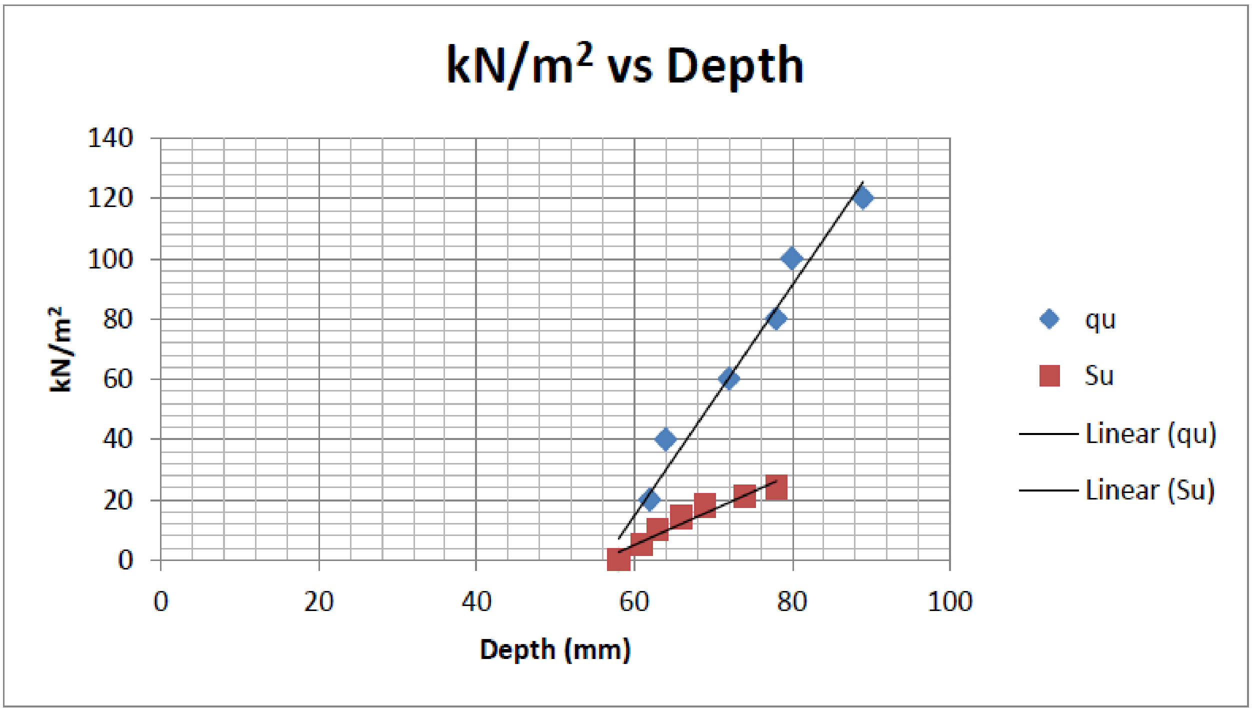

Figure 14. Graph of qu and Su vs Depth (First Trial)

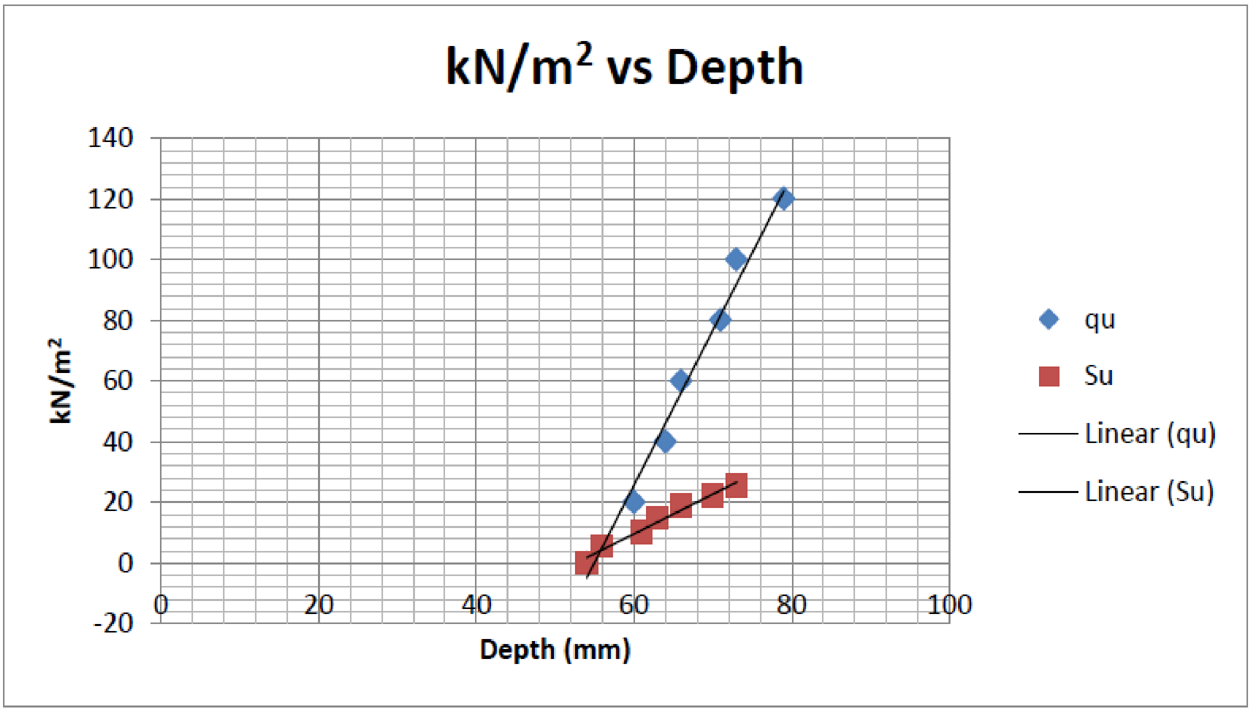

Figure 15. Graph of qu and Su vs Depth (Second Trial)

B. Test 2

Soil that was used for testing in test 2 is a clayey soil. From visual observation; soil was found as red brownish soil which is a cohesive soil. The shear strength of the soil that was tested using a vane shear apparatus gives a result ranging from 20kpa to 23kpa. Penetration that resulted from the loading applied to the probe was in the range of 2mm to 10mm for tip resistance as shown in table III and table IV.

Tables III and IV highlight that there is a drastic increase of the penetration which is 83mm. It can be said that there is a void section in between the soil at the bottom of the probe which caused the failure of the soil when it was loaded with 60kg of load. Cumulative penetration that is recorded for tip resistance is 131mm and 52mm. For the skin resistance, the penetration of the soil is in the range of 1mm to 6mm and cumulative penetration of 38mm and 44mm as shown in the figures 16 and table 17.

The drastic increase of the penetration for tip resistance in trial 1 is ignored when plotting graph. As shown in figures 18 and 19, ultimate bearing capacity value is higher than the undrained shear strength. The qu over Su ratio for both trial was in average of 1.09 and 1.68 as shown in table 4.11 and 4.12 which almost the same and prove that the result taken is acceptable.

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TABLE III. EXPERIMENTAL DATA (FIRST TRIAL)

Loading (kg)	Force (kN)	Tip Resistance			Skin Resistance			
		Penetration (mm)	Cumulative (mm)	q_u (kg/mm^2)	Penetration (mm)	Cumulative (mm)	A_u (m^2)	S_u (kN/m^2)
0	0	0	20	0	0	20	0.00628	0
10	0.0981	5	25	20.02040816	1	21	0.0066	14.8696
20	0.1962	3	28	40.04081633	2	23	0.00723	27.1532
30	0.2943	2	30	60.06122449	4	27	0.00848	34.6958
40	0.3924	8	38	80.08163265	3	30	0.00942	41.6349
50	0.4905	10	48	100.1020408	5	35	0.011	44.6089
60	0.5886	83	131	120.122449	3	38	0.01194	49.3045

TABLE IV. EXPERIMENTAL DATA (SECOND TRIAL)

Loading (kg)	Force (kN)	Tip Resistance			Skin Resistance			
		Penetration (mm)	Cumulative (mm)	q_u (kg/mm^2)	Penetration (mm)	Cumulative (mm)	A_u (m^2)	S_u (kN/m^2)
0	0	0	23	0	0	23	0.00723	0
10	0.0981	4	27	20.02040816	2	25	0.00785	12.4905
20	0.1962	6	33	40.04081633	4	29	0.00911	21.5353
30	0.2943	2	35	60.06122449	2	31	0.00974	30.2189
40	0.3924	4	39	80.08163265	3	34	0.01068	36.7367
50	0.4905	8	47	100.1020408	6	40	0.01257	39.0327
60	0.5886	5	52	120.122449	4	44	0.01382	42.5812

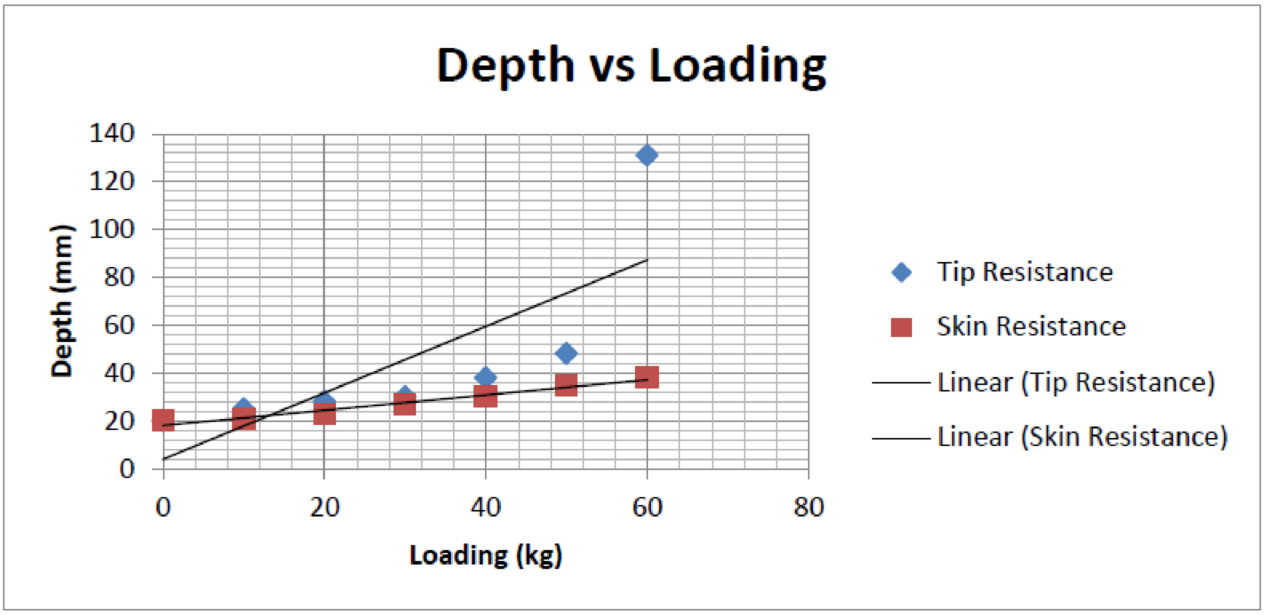

Figure 16. Graph of Depth vs Loading (First Trial)

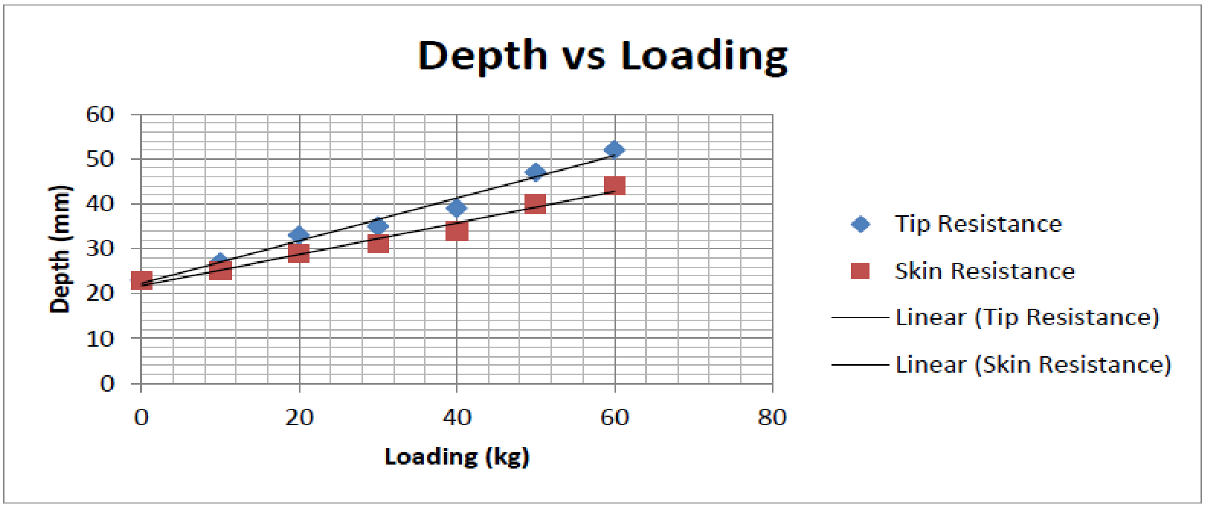

Figure 17. Graph of Depth vs Loading (Second Trial)

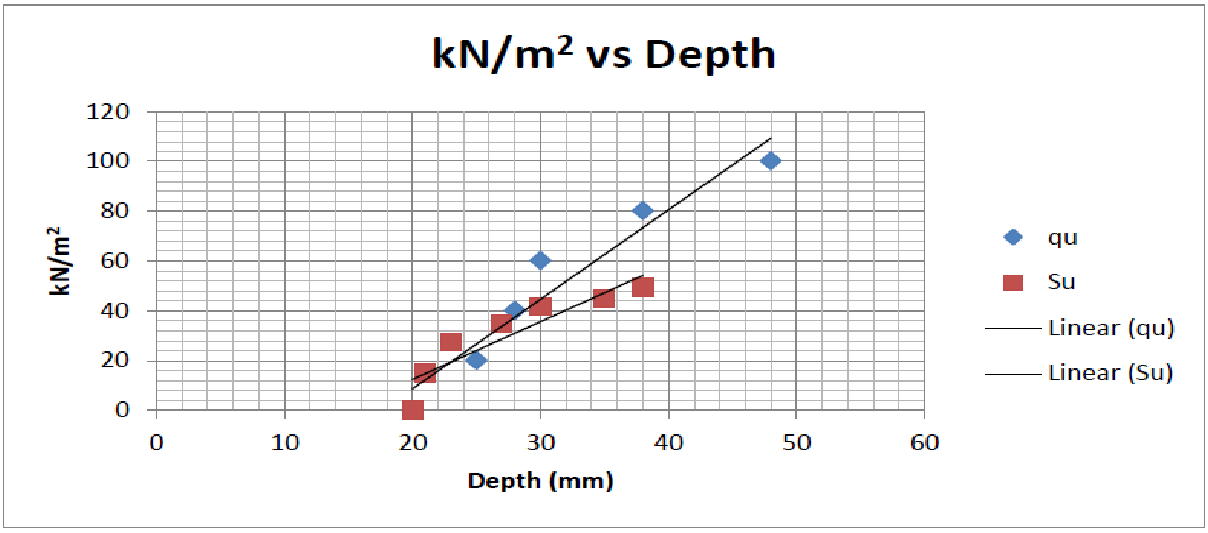

Figure 18. Graph of qu and Su vs Depth (First Trial)

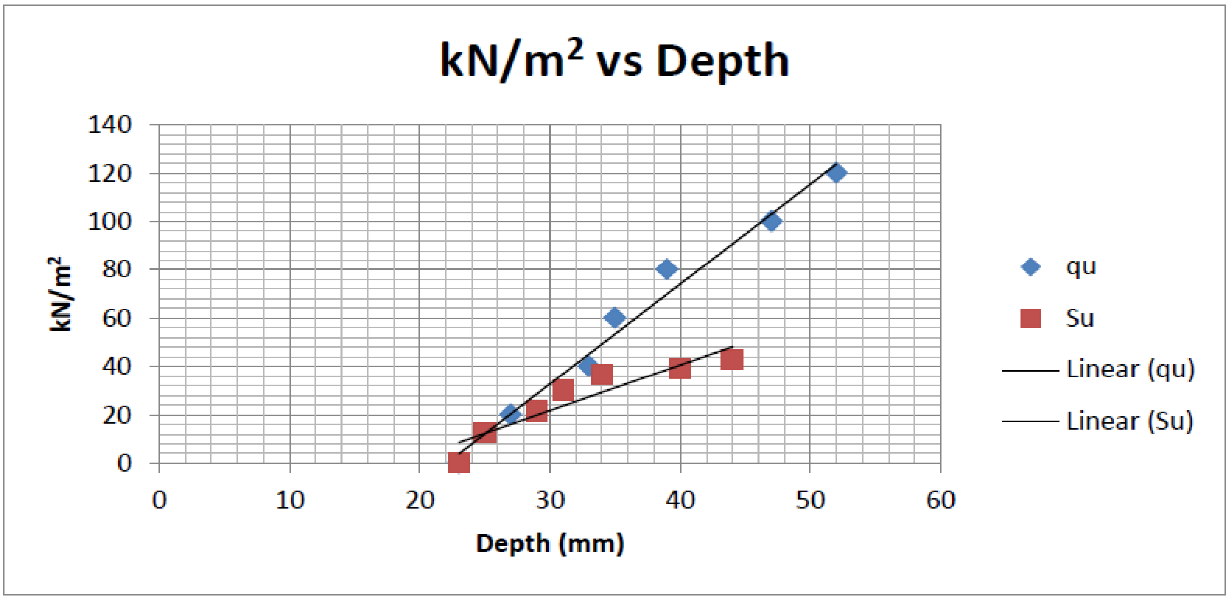

Figure 19. Graph of qu and Su vs Depth (Second Trial)

VIII. CONCLUSION

This study presented a new approach that will be an alternative approach to conventional investigation on soft soil. The data had been obtained from carrying out test and calculation based on the fundamental theory of soil engineering. Total of two tests with two trials each were done to obtain the data that was necessary. Based on the analyses carried out, the conclusion of the study can be summarized as follow:

1. The modeled probe is designed and fabricated and is working as how it is expected to be with the ability to obtain the value for tip resistance and skin resistance.

2. The ratio of qu over Su decreases as the strength of the soil increase that can be observed.

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