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P-Cone: A Novel Cone Penetration Test Device for Deep Foundation Design

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ABSTRACT: A novel cone penetration test device, the P-cone, has been developed to assist in deep foundation design and this P-cone device combines features of CPTU cone sounding technologies with capabilities to perform bidirectional loading at a given soil depth condition. Using two independent systems, the P cone measures shear stress versus movements of cone shaft and stress versus penetration of the cone tip at desired depths. P-cone tests were performed on large compacted clayey silt specimens in a laboratory fabricated chamber. Tests performed showed that the movements to fully mobilize the shaft shear resistance and tip resistance were close to 0.2 mm and 2.0 mm, respectively. The soil failure shapes around the cone tip investigated was found without the horizontal stress build-up around the cone tip and shaft.

KEYWORDS: P-cone, Cone penetration, Head-down test, Bidirectional test, End bearing test

1. INTRODUCTION

Piles that are designed for end bearing are often installed through less competent soil layers to a bearing-competent soil layer. The pile toe movement responses to imposed loads plays a key role in the pile foundation design, where settlement of a piled foundation is large enough to be a problem.

Full-scale static load tests are usually performed to verify a pile design and assumptions involved in the analysis. In a conventional head-down static load test (ASTM D1143-81), however, it is not possible to sufficiently mobilize the pile toe resistance to enable an analysis of the pile toe response. It is difficult to determine the portion of the applied test load that actually reaches the pile toe, and the potential presence of residual load at the pile toe adds a major complexity to the analysis. The bidirectional loading test method (Osterberg, 1984 and 1989) eliminates much of such difficulty if the bidirectional cell can be activated near the pile toe and the shaft resistance is sufficient to supply reaction resistance to the downward push of the cell.

Most piled foundations are designed without the benefit of static loading tests, however, and they rely on information received from the site investigation, particularly results of in-situ tests, such as the CPTU. Comprehensive details of CPT and CPTU methods can be found elsewhere (Eslami, et al., 1997).

This paper presents a novel cone penetration test device (P-Cone) developed to determine the soil resistances at the toe level by allowing it to load in bidirectional mode. This device is capable of performing routine site investigations, improving the penetration depth, and measuring shear movement above and stress penetration below the cone tip at the desired depths. This device is tested in large sized silty soil specimens in laboratory conditions. This device provided valuable data that could aid in the better design of piled foundations. More studies including field studies will be performed in the future to further assess the ability of this device in providing shear resistances at the toe and shaft levels.

2. DESIGN OF P-CONE DEVICE

2.1 Design Concept

A major difficulty with current CPTU devices is that the limited reaction force prevents the sounding from reaching into the depth at which the piles are usually installed. The P-cone device eliminates this restriction by utilizing the pressure in a cell that uses the cone rods and surface anchors on the ground surface as a reaction to push the cone down. This is clearly illustrated in Figure 1. As can be seen from the left diagram of Figure 1, the required reaction load, P, of the current CPT devices is equal to the sum of the shaft resistance, P1, and the cone resistance, P2. The reaction load of the newly designed P-cone (right side of Figure 1) is equal to the sum of the reaction load of the current CPT device and the shaft resistance, P1.

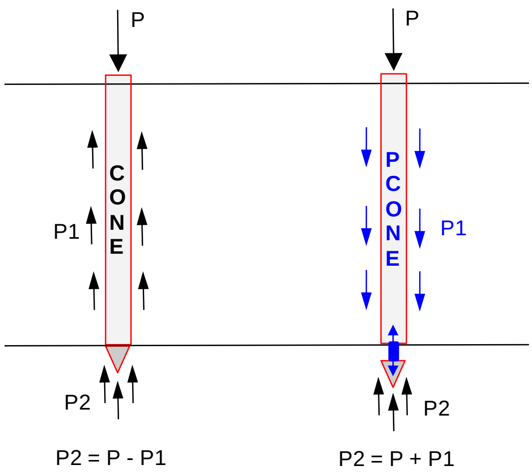


Figure 1 Illustration of the penetrating depth improvement

Figure 2 illustrates the principles of the P-cone used in combination with a hydraulic jack that facilitates the movement-and-force-generating feature at a selected depth. The cone tip resistance is measured by a separate load cell. The other necessary measurements are similar to those of a conventional cone sounding device: the downward and upward movements measured by rod extensometers, the force in the shaft measured by means of strain-gages placed inside the shaft, and the pore pressure acting on the cone shoulder measured by a pressure transducer. In the current tests, pore water pressure transducer is not considered as validation tests are conducted on unsaturated soil specimen.

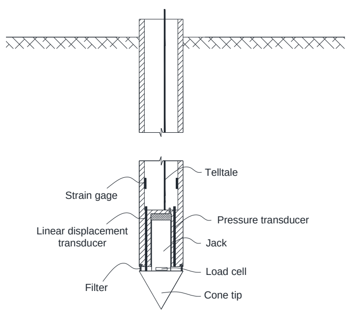
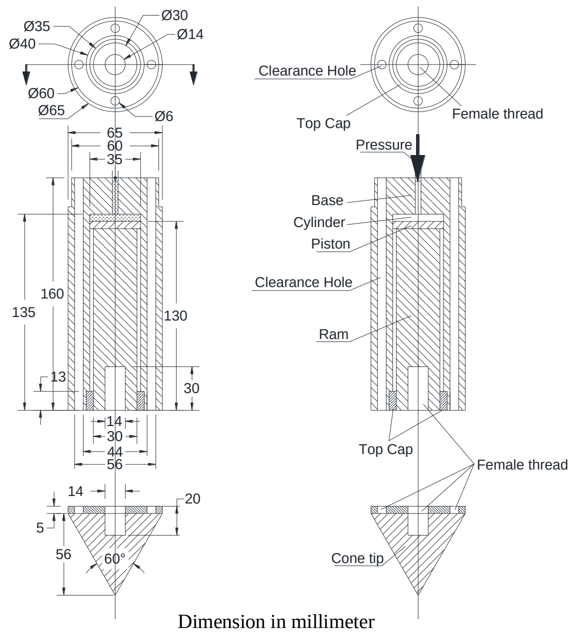


Figure 2 Scheme of novel CPT equipment

2.2 Detail Design

Figure 3 depicts the detailed design of the P-cone. The hydraulic jack has a 65-mm outer diameter (O.D.), 35-mm inner diameter (I.D.), and a 160-mm height. The diameter of the jack in piston is 30-mm, with a 120-mm travel. The capacity of the jack considered in this study is about 20 kN. The pressure inside the jack and piston movements were measured with instruments installed from the jack upper surface. The cone tip has a 60^0apex angle with a 66 cm^2 base area.



Dimension in millimeter

Figure 3 Detailed design of P-cone device

2.3 Manufacturing and Assembly

The main cone devices (jack, cone tip, and shaft) was designed and assembled with required sensors, and strain gages at UTA Geotechnical laboratories. Figure 4 shows the prototype of the P-cone. In order to measure the shaft resistance, the vibrating wire sensors with measureable ranges of 3,000 micro strains (με) were installed 200 mm above the cone tip. The cone stress was measured by a pressure transducer with measureable range of 5 MPa and pressure gauge of 4 MPa. Telltales were also used to measure the upward and downward movements during testing in a laboratory chamber and details are depicted in Figure 2.

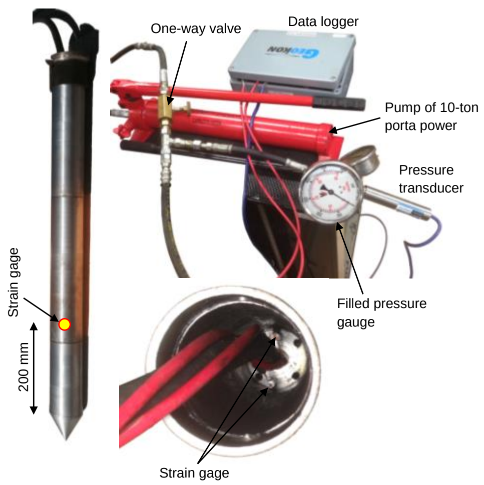


Figure 4 The Prototype P-cone device

2.4 Operation Principle and Measurement

The operation principle and measurement of the P-Cone for the penetration test and bidirectional test are performed according to the following steps:

Step 1: The P-Cone is connected with the jack and the reaction beam to be pushed into the compacted soil chamber. Then, the hose of the P-Cone is connected with hand pump, the pressure gauges and the pressure transducer. Next, the cables of the strain gages and the pressure transducer are connected with computer via datalogger (Figure 5).

Step 2: The LogView software installed in computer is operated to communicate with the strain gages and the pressure transducers. Then, the release valve of the hand pump is closed and the control valve is opened to operate hand pump via handle and to push the cone tip open about 5 mm (Figure 5). At this time, the values of the fluid pressure and the strain gage of the P-Cone are taken as the initial fluid pressure values. It should be noted that the 5 mm expansion of the cone tip is necessary so that the influences of the fluid compressibility and dilatation of hose on the measured cone tip resistances can be eliminated. Moreover, the jack attached with the reaction beam has not had any actions at this period.

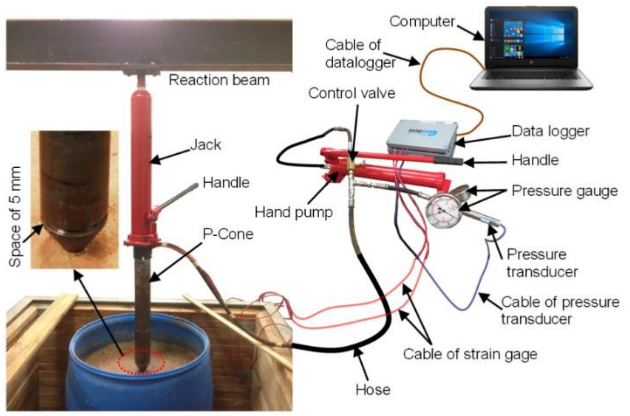


Figure 5 The operating principle and the measurement system of the P-Cone

Step 3: The control valve is closed and then the P-Cone is pushed into the ground by the jack attached with the reaction beam via handle. During penetration, the cone tip resistance and the sleeve friction are measured by pressure transducer and the strain gages, respectively. If the experiment of the P-Cone is performed in saturated soil, additional pore pressure transducer will be installed at the available holes of the jack to measure the pore pressure near the cone tip. The measurements are recorded by data logger and these measurements include the cone tip resistance, the sleeve friction and the pore pressure.

Step 4: After the cone penetration testing has been completed, the bidirectional load test is started. The jack attached with the reaction beam is removed. Then, the telltales and the dial gauges will be installed (Figure 6) to measure the downward and upward movements of the cone tip and the cone shaft, respectively, during the bidirectional load test. Next, the control valve is opened and the loading is performed using the hand pump system.

After each successful load increment, the control valve is closed and then opened after a short holding time period. The process is repeated until the completed load test. During testing, the cone tip and the shaft resistance are also measured by the pressure transducer and strain gages via datalogger system, respectively. The test results obtained from the bidirectional load test at this period are the shear stress versus movement and the cone tip tress versus penetration, the same as the conventionally bidirectional load test.

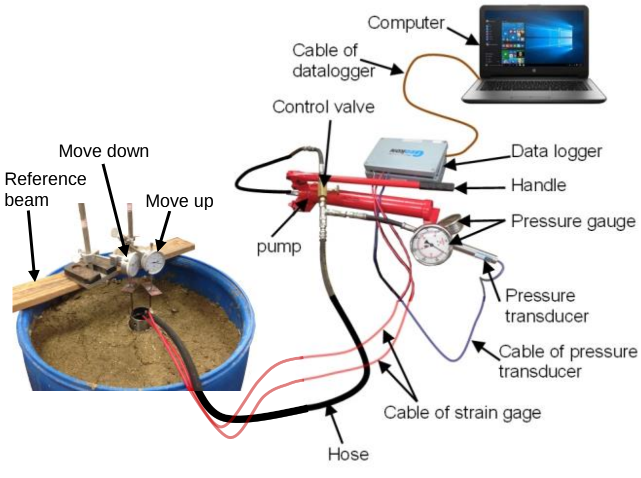


Figure 6 The bidirectional load test and the measurements operating principle and the measurement system of the P-Cone.

3. LABORATORY SETUP

3.1 Chamber

A circular chamber with diameter and height of 590 mm and 889 mm, respectively, was used in the present research, as shown in Figure 7. The ratio of chamber diameter to cone diameter was about 9. Therefore, the effect of the chamber boundaries on the P-cone penetration test will be considered in the interpretation of test results (Ghionna and Jamiolkowski, 1991; Salgado et al., 1998). After placing soil into the chamber, the compaction was performed by using a tamper with a foot diameter of about 180 mm. The weight and drop height of the tamper were about 0.06 kN and 0.5 m, respectively. Each soil layer was subjected to 75 blows to produce a thickness of about 80 mm per lift after compaction. The total thickness of the soil in the chamber was comprised of about ten such compacted layers.

Figure 7 also indicates a loading system set up used for performing the testing. The loading system consisted of a steel frame, a 80-kN hydraulic jack with a travel of about 0.6 m, and a pump system. The cone penetration test can be performed by increasing the fluid pressure inside the jack, via pump handle, to push the cone into the chamber.

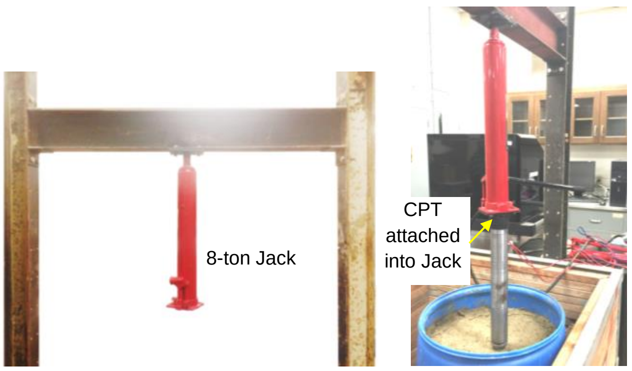


Figure 7 Jack attached to steel frame as reaction system

3.2 Soil Properties

The soil used for testing the P-cone consisted of clayey silt with a plasticity index and liquid limit of 34% and 58%, respectively. The clayey silt contained 0.1% gravel-size particles, 16.4% well-graded sand, 65.7% silt, and 17.8 % clay-size particles. The specific gravity of the clayey silt was 2.69. Proctor compaction tests (ASTM D 698-2007) indicated a maximum dry density of 15.3 kN/m^3 at an optimum moisture content of 21%.

After the p-cone device tests, the compacted soil in the chamber were sampled at different depths to determine the density, water content, void ratio, degree of saturation and the shear strength properties. The sampling ring with dimensions of 25.4 mm in height and in 63.5 mm diameter was used. The average unit weight of the compacted soil layers determined was about 19.3 kN/m^3 at water content of 25%. The average void ratio and degree of saturation were about 0.709 and 95%, respectively.

The strain-controlled direct shear tests were performed on the compacted clay silt soil samples with a shear rate of 0.125 mm/minute. For this shear rate, the strength of soil obtained from shear tests is considered as the undrained strength and is reasonable for correlating into the quick load test results of the P-cone device. The test results showed a cohesion intercept, c, of 34 kPa and an angle of internal friction, ϕ, of 5^0.

It should be noted that the tested soil samples were only partly saturated (degree of saturation 95%) and thus when the total normal stress was increased, the strength of soil was increased because changes in total stress did not cause equal increase in pore pressure. As the total stress applied to a partly saturated specimen was increased, both the pore pressures and the effective stress increased. This occurs because the pore fluid (the mixture of water and air) was not incompressible and only part of the added total stress was carried by the pore fluid. The balance was carried by the soil skeleton, which leaded to an increase in effective stress (Duncan and Wright, 2005). Moreover, it is also noticed that the movements necessary to obtain the peak values of the shear resistance on the tested soil samples were about 1.4 mm.

3.3 Test Procedure

The P-cone device testing program consisted of a cone penetration test, head-down test, bidirectional tests, and the end bearing tests as illustrated in Figure 8. The cone penetration test was performed first by pushing the cone 0.5 m below the ground surface with a penetration rate of 10 mm/s. It is noted that the standard penetration rate of 20 mm/s was not considered due to the penetration performed in the compacted soil.

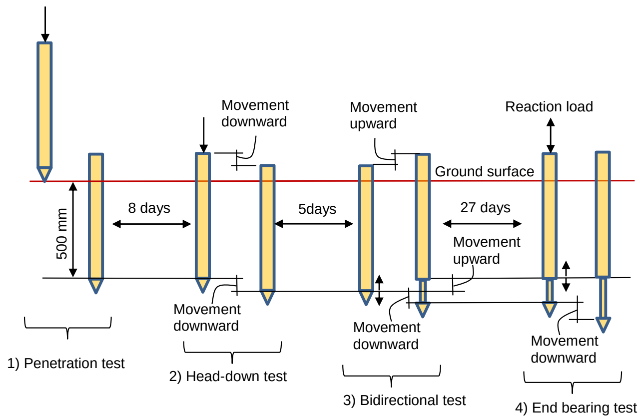


Figure 8 Test procedure of the P-cone performed

The head-down test (Figure 9) was carried out eight days after cone penetration was completed. The head-down loading was performed by the hydraulic jack by placing an additional load at the top of the cone assembly, and the load imposed on the cone head was recorded by using a load cell. The head-down test was performed in a total of 27 load increments, with each increment ranging from about 26 through 318 N. Each of the 27 load increments was held constant for five minutes, and the unloading was performed in three steps.

Five days after the completion of head-down test, the three bidirectional tests were carried out (Figure 10). The first bidirectional test was performed in many load increments, ranging from about 5 through 138 N, and the unloading was performed in seven steps. After the completion of the first bidirectional test, two additional bidirectional tests were performed in 8 load increments, ranging from about 27 through 484 N, and the unloading was performed in four and seven steps, respectively. All load levels were maintained from one to five minutes.

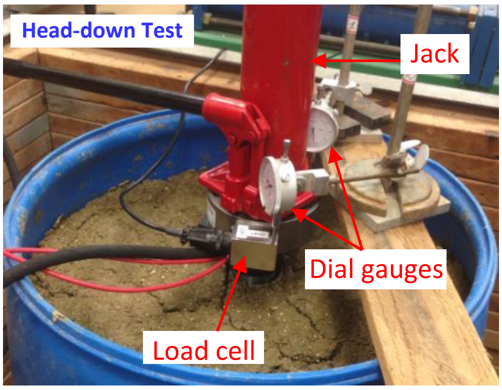


Figure 9 Setup of head-down loading test

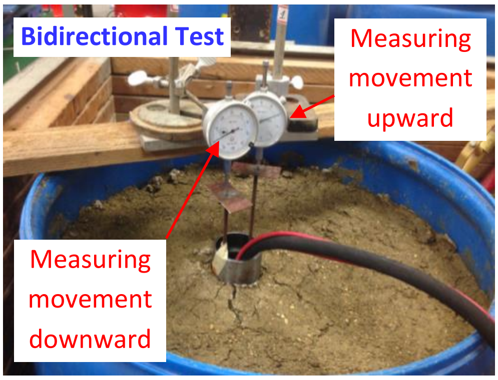


Figure 10 Setup of bidirectional loading test

Twenty-seven days after bidirectional tests, the five end bearing tests were conducted and Figure 11 shows the loading steps followed in the end bearing tests. The end bearing tests were done in five and three loading cycles on the second and third test, respectively. Two loading cycles were performed for both the first and fourth test, and one loading cycle was carried out on the fifth test. The load increments for the tests were performed in 9 through 44 increments, ranging from about 6 through 470 N, and the unloading was performed in four through ten steps, respectively. All load levels were maintained from one to five minutes.

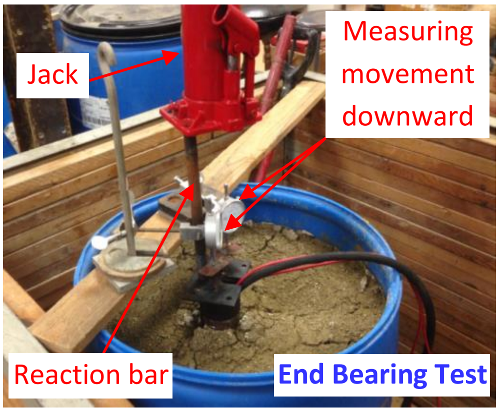


Figure 11 Setup of the end bearing loading test

It should be noted that all of the loading procedures of head-down test, bidirectional tests and end bearing tests were carried out in one loading cycle, until the plunging failure occurred, before starting the next loading cycle. Furthermore, the same sequence of testing was performed on three samples with the different densities and water contents; however, only one sample test results are presented and the following sections cover the analyses of test results.

4. TEST RESULTS AND ANALYSIS

4.1 P-Cone Penetration Test

Figure 12 shows the results of the cone penetration test. As can be seen from the left diagram of Figure 12, the cone tip resistance increased linearly up to 200 mm of depth below the ground surface. At this depth, the cone tip resistance was measured at about 620 kPa. From 200 mm to 300 mm depth, the tip resistance reduced slightly and then decreased linearly to about 400 kPa at 500 mm depth. The right diagram of Figure 14 represents the cone shaft resistances up to 300 mm depth below ground surface. The maximum shaft resistance measured was about 35 kPa at 100 mm depth. Below this depth to 300 mm depth, the shaft resistance reduced gradually to about12 kPa. The results of penetration reflected that the soil layers between 200 mm and 300 mm depth seemed to be more compacted.

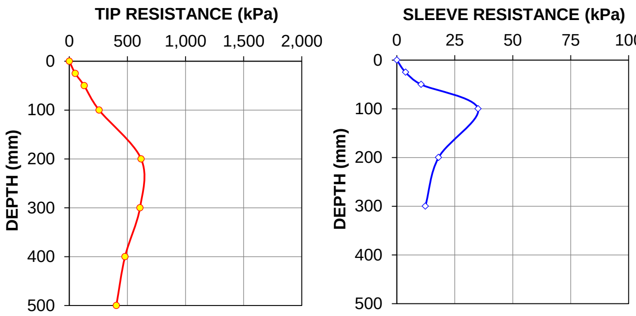


Figure 12 Shaft and tip resistance during P-cone penetration

After penetration, several cracks appeared in the soil surrounding the cone, as displayed in Figure 13. It can be clearly seen from Figure 13 that five main cracks occurred around the cone. The maximum width of the crack found at position no. 1 was about 5 mm at the cone wall. It decreased gradually to 0 mm at a distance of about 130 mm. In short, the influence radius by cone penetration test at this point is about twice the cone diameter. The influence radius of the cracks at the other positions varied from 80 through 120 mm (1.23 – 1.85 times of cone diameter).

The observed cracks provided evidence that using the clayey silt with a chamber diameter of 590 mm was sufficient to minimize the effect of the chamber boundaries on the cone tests of 65 mm diameter. However, the scale effects can be important in highly interbedded soils or in stiff heavily over-consolidated clays (Powell and Quarterman, 1988).

Dimension in millimeter

Figure 13 Shaft and tip resistance during P-cone penetration

4.2 Head-Down and Bidirectional Loading Tests

The load-movement curves of the head-down test and bidirectional tests are shown in Figures 14 and 15, respectively. As can been seen from the diagram of Figure 14, the maximum load and movement measured were 3.90 kN and 5.64 mm, respectively. However, the plunging failures occurred at load and movement of about 3.90 kN and 2 mm, respectively. Moreover, the plastic deformation started at the load level and movement of about 3.16 kN and 0.33 mm, respectively.

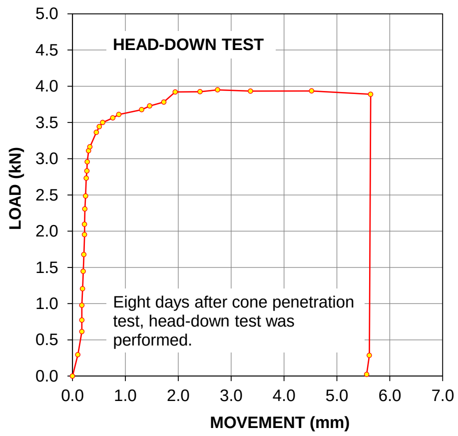


Figure 14 Load-movement curves of head-down loading test

The diagrams of Figure 15 show three results of the bidirectional tests. The maximum load for the three tests was about 1.35 kN. The maximum downward and upward movements recorded were about 0.28 through 3.68 mm, 0.15 through 2.01 mm, and 0.19 through 3.25 mm, respectively. It was noted that a bidirectional test reached the ultimate load in only one of the two resistance components. In this case, the cone shaft resistance reached an ultimate load of only about 1.30 kN because the cone penetration was not deep enough that the cone shaft resistance could be equal or greater than the cone tip resistance. However, in practice, the total cone shaft resistance is often greater than the cone tip resistance, fully mobilizing the cone tip resistance.

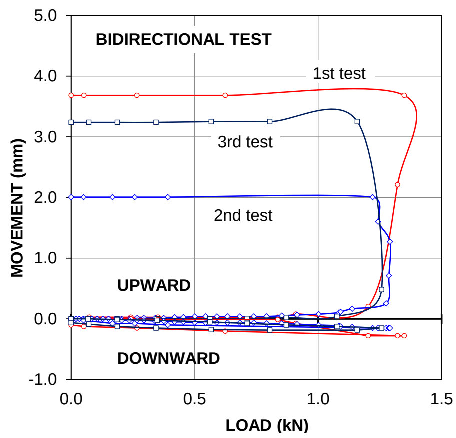


Figure 15 Load-movement curves of bidirectional loading tests

Figure 16 shows the shaft resistances measured from the bidirectional tests. The first striking observation is that the unit cone shaft resistances from the first and second test results were similar: about 15 and 10 kPa, for cone shaft resistance below and above strain gage level, respectively. This demonstrated that the disturbance of the soil along the cone shaft caused by the first had no significant influence on the following test, even though the second test was performed two days after the first test. For the third test results, the diagram of Figure 16a) indicates that the shaft resistance from the cone tip level to the strain gage increased by about 47%. The diagram of Figure 16b) shows that the shaft resistance from the strain gage level to the ground surface decreased significantly, about 50%. It should be noticed that the third test was carried out about 31 days after the second test, and the maximum test load of all three tests at plunging failures was similar. It is likely that the increase of the cone shaft below strain gage level was due to decreasing water content of the soil along cone shaft (increased suction of soil along the cone shaft). Moreover, the decline of the cone shaft resistance above strain gage level was due to cracks in the top soil layer and the upward movements of two previous tests, which reduced the adhesive length between the cone shaft and soil.

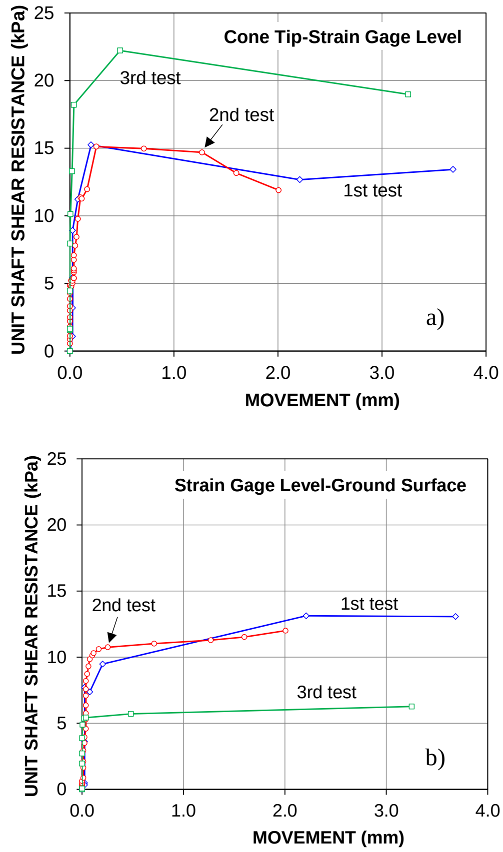


Figure 16 The unit shaft resistance-movement curves of bidirectional tests

By comparing the cone shaft resistances obtained from the cone penetration test (right diagram of Figure 12) with the bidirectional tests, it can be seen that the cone shaft resistances of the first and third bidirectional test results below strain gage level increased by about 25 through 83%, and above strain gage level decreased by about 18 through 50%, respectively. The causes leading to the decline of the cone shaft resistances above strain gage level were explained earlier.

It is necessary to consider the direct shear test results of soil in the laboratory with the cone shaft resistances measured from the cone tip to strain gage level (Figure 16a). At the 0.3 m depth of the strain gage placed in soil box, the vertical pressure of soil, estimated based on the density of compacted soil, was about 6 kPa. Based on the direct shear test results, the undrained shear resistance of soil was about 34.4 kPa, 2.3 and 1.5 times greater than the cone shaft resistances for the first two and third bidirectional test results, respectively. However, it should be noted that the movements to fully mobilize the cone shaft resistances were about seven times less than the direct shear resistances.

Figure 17 demonstrates that the failure surface of soil along the cone shaft during tests occurred at the interface between the cone and the soil. Moreover, if correlating the cone shaft resistance of the third bidirectional test results with the direct shear resistance of soil sampled from the chamber, the interface factor was about 0.65.

Figure 17 The failure surface of soil along the cone shaft

4.3 End Bearing Loading Test

Figure 18 shows the load-movement curves of the five end bearing tests measured during the loading cycles. The loadings were only terminated when the plunging failures occurred, and all of the test results indicated that the plunging failures occurred at movement of about 2 mm. The first test results showed the maximum load of about 2.70 kN, which is greater than the other test results by about 17%. It is likely that cone tip resistance differences measured at the tests is due to disturbance of soil or the different densities of the compacted soil layers.

Upon comparing the cone tip resistances measured by the cone penetration test (left diagram of Figure 12), the cone tip resistance of the first end bearing test was greater by about 4%, and the cone tip resistance of the other end bearing tests was smaller by about 11%. An average load-movement curve showed a maximum load of about 2.30 kN at 2 mm of movement.

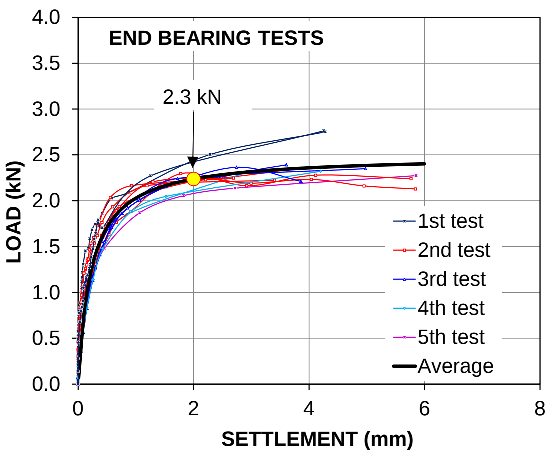


Figure 18 Load-movement curves of end bearing tests

Figure 19 represents the failure shape of the soil below the tip cone, investigated by excavating half of a soil chamber after completing the end bearing tests. The investigation shows that the soil failure shape around the cone tip was found without the horizontal stress build-up around the cone tip and shaft as the conventional assumed failure modes of soil below pile toe (Terzaghi 1943, Meyerhof 1951, Vesić 1972, Janbu 1976).

The diagram of Figure 16 shows that the maximum load measured in the head-down test was about 3.90 kN, while the sums of the maximum shaft and tip resistance from the bidirectional tests and the end bearing tests, respectively, were less than the maximum load from the head-down test by about 8% at movement of 2 mm. However, loads at movements from 1 to 2 mm are similar. It was noticed that at movements from 1 to 2 mm, the capacity of the model pile was reached as shown in Figures 14, 15 and 18. This means that the head-down test load shows a good agreement with the sum of the bidirectional and end bearing test loads at plunging failure. Furthermore, if ignoring the shortening of cone material, it is recognized that movement to fully mobilize the soil resistance of the head-down test depends only on the movement of the cone tip.

Figure 19 Failure shape of soil below cone tip

5. DISCUSSION

Currently-available cone penetration devices are unable to provide response to imposed loads in terms of movement into soils at desired depths. The penetrating depth is also limited by the reaction load system, and the shear resistance measurement is performed only with a single system. Therefore, it is difficult to avoid the unexpected errors of measurements.

The design and manufacture of a novel cone penetration test device, the P-cone, is presented in this study. Successful experiments in a laboratory chamber of clayey silt soil provided the first compelling evidence of its use in measuring the shear resistances of soil versus movements along the cone shaft and below cone tip at the desired depths, where the pile toe will be placed. The most interesting finding from the test results is that the movements to fully mobilize the cone shaft resistance is about 0.2 mm, which is dramatically smaller than the movements obtained from full-scale pile tests (Fellenius and Nguyen 2013; Nguyen and Fellenius 2014). It is likely that these differences are because of the different types of shear failure of soil along the cone and pile shaft. In the subject case, the shear failure occurred at the interface between the soil and cone wall, while the shear failure of full-scale pile tests are rarely observed and possibly take at places outside pile wall. The differences in soil type and water content of the soil have possibly caused the difference of movements required to mobilize the full shaft resistance. It should be noted that the movement required to fully mobilize the shaft resistance is independent of the diameter of pile (Poulos, 2011); therefore, the variations of the diameters of the model pile and full-scale piles were not considered. The soil failure shape below the cone tip investigated was found without the horizontal stress build-up around the cone tip and shaft; this has reflected the uncertainties of the conventional assumed failure modes of soil below pile toe.

It has become clear that the greatest advantage of the P-cone device over existing cone penetration devices is that it combines two technologies in a single device. The P-cone not only consists of the advances of the present cone and bidirectional test technologies, but also offers more useful advances as the tip resistance measurement with two independent measurement systems, the penetrating depth improvement, and the stress-movement measurement of soil at the desired depths. Moreover, the P-cone is able to offer a potential application for in-situ consolidation compression.

Preliminary experimental findings provide promising future applications of the P-cone device to a wide range of soil investigations. Consequently, a field test program should be undertaken to make any desired improvements and to fully understand the advantages of this device.

6. CONCLUSION

The design and manufacturing specifics of the new cone penetration test device, the P-cone, have been presented. The successful experiments of the P-cone in a laboratory chamber of compacted clayey silt soil provided the first compelling evidence for its advantages in measuring the shear resistance of soil versus movements. The findings from the test results provided a better understanding of soil behavior along the cone shaft and below the tip cone. The following conclusions can be drawn from the present study.

The maximum load and movement measured from the head-down test were about 3.9 kN and 5.6 mm, respectively. The necessary movement to fully mobilize the cone resistance under head-down test was about 2.0 mm.

The maximum load and movement measured from the bidirectional tests were about 1.3 kN and 3.6 mm, respectively. The necessary movement to mobilize fully the cone shaft resistance from bidirectional tests was about 0.2 mm.

The maximum load and movement measured from the end bearing tests were about 2.7 kN and 5.8 mm, respectively. The necessary movement to mobilize fully the cone tip resistance from end bearing tests was about 2.0 mm.

The head-down test load shows a good agreement with the sum of the bidirectional and end bearing test loads at the plunging failure.

The interface factor between the soil and the cone was established based on the results of direct shears test and bidirectional tests and was about 0.65.

The failure surface of the soil along the cone shaft was found at the interface between the soil and the cone wall. The observed response of the soil failure shape below the cone tip show no horizontal stress build-up around the cone tip and shaft as the conventional assumed failure modes of soil below pile toe.

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