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Full Length Research Paper

Investigation of bearing capacity changes of different clays by using the Menard pressuremeter tests

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Determination of bearing capacities of clay soils always have been a major problem in Geotechnical Engineering, because of the fact that long-term laboratory tests are required on undisturbed samples. But, it is very difficult to get undisturbed sample from sites and the bearing capacities which were calculated with the laboratory test values are always greater than the actual bearing capacities. The Menard Pressuremeter is a reliable method of calculating the bearing capacity, but it is expensive and requires experience and time. So, it is difficult to apply it on all cases. In this study, the Menard pressuremeter test was performed to 13 different test locations in Turkey. A total of 65 test results were taken into account. Bearing capacities were calculated for 5 foundation depths (2, 4, 6, 8 and 10 m) and ranged between 240 to 400 kPa, 400 to 1200, 720 to 1900, 1200 to 2800, and 1700 to 3200 kPa, respectively. Finally, an equation has been developed for calculating the bearing capacity of clay soils depending on different foundation depth. In this manner, the Geotechnical Engineers who have limited facilities can take into account this equation and they will be able to have ideas about the bearing capacity of clay soils by performing only a few experiments on their site investigation.

Key words: Clay, geotechnique, Menard pressuremeter test (PMT), bearing capacity, Turkey.

INTRODUCTION

During engineering activities, time and expenses have become great concern in recent years and they will be more important in the future. This concern can be overcome with choosing appropriate methods, knowledge and experience in geotechnical engineering. In geotechnical engineering, bearing capacity is an important parameter and is the capacity of soil to support the loads applied to the ground. To calculate the bearing capacity, settlement, modulus of elasticity, cohesion, internal friction angle, etc., long-term laboratory tests are required on undisturbed samples. But, it is very difficult to get undisturbed sample each time and the calculated bearing capacity value which are determined with the laboratory tests are always greater than the real bearing capacities.

The Menard pressuremeter test (PMT) is one of the most common types of radial loading tests, which is used to determine bearing capacity in Geotechnical Engineering. PMT was invented by Louis Menard in 1950, during the graduation researches at Illinois University (USA). He did lots of laboratory experiments in Geological Etude Centre to confirm the accuracy of pressuremeter test results. Today, engineers still use these equations (Sols Soils, 1975).

To apply the PMT on every site investigation is difficult because it is expensive and requires experience and time. But it is considered as a reliable method in calculating bearing capacity by the geotechnical engineers (Baguelin et al., 1978).

Isik et al. (2004) performed the PMT for landslide stability and back analysis to collect geotechnical data of the soil of Sinop (Turkey). Isik et al. (2008) also performed PMTs in determining the deformability of jointed rock slope stability and the back analysis of the greywacke of Ankara (Turkey). In their study, they applied the PMTs in 8 boreholes. The deformation modulus from the PMT was evaluated by numerical methods, and the comparisons between the deformation modulus of the greywackes obtained from the PMTs and their geomechanical quality were made. Justo et al. (2008) performed PMTs to collect geotechnical data of

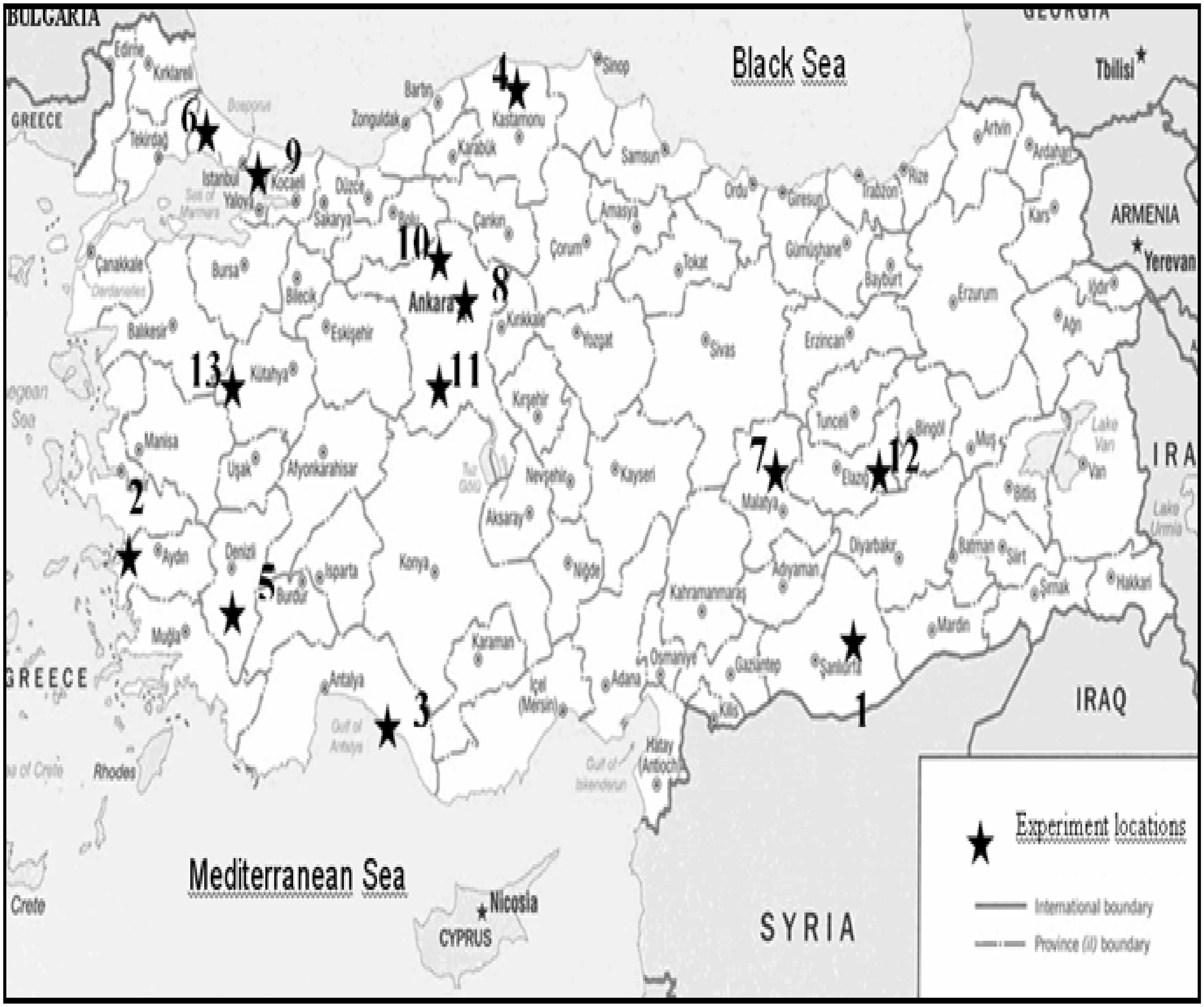

Figure 1. The map of the PMTs locations.

the conglomerate slope of Granada (Spain). Bozbey and Togrol (2010) conducted a study for both sands and clays. They searched for the relationships between N, E_P and P_L based on a study conducted in Istanbul, Turkey. Yagiz et al. (2008) searched for the relationships between N, Menard elasticity module values (E_P) and limit creep pressure (P_L) based on the study conducted in Denizli, Turkey. Mahmutoglu (2011) performed PMT for surface subsidence induced by twin subway tunneling in clayey soft ground conditions in Istanbul.

All of these studies aforementioned were used *in-situ* tests to investigate ground conditions and to collect geotechnical data of the clayey soils. In this study, PMT was used for the generalization of bearing capacity of the clayey soils. Additionally, an attempt was also made to find a relationship between bearing capacity results and foundation depth.

MATERIALS AND METHODS

Test locations

In this study, the PMT was performed for 13 different test locations in Turkey (Figure 1). 5 experiments were performed for each drilling location with 2 m depth interval (2, 4, 6, 8 and 10 m). A total of 65 test results were obtained and taken into account. Bearing capacities were calculated for 5 foundation depths of the 13 locations. At each location, the bearing capacities at different depths are drawn on a graphic. As a result, an equation has been

Table 1. The Atterberg Limits of the samples.

Atterberg limits	Locations			
	8, 10 and 11	4, 6, and 9	1, 7 and 12	2, 3, 5 and 13
Natural unit weight, **γ** (kN/m^3)	18-20	17-20	17-21	17-21
Liquid limit, **LL** (%)	50-57	55-67	25-34	35-42
Plastic limit, **PI** (%)	23-26	24-28	10-13	15-17

developed for calculating the bearing capacity of clay soils depending on different foundation depth. In this manner, the Geotechnical Engineers who have limited facilities can use this equation and they will be able to have ideas about the bearing capacity of clay soils by doing only a few experiments on their site investigation.

The PMTs, performed to 13 different locations in Turkey, can be seen in Figure 1. In all experiment locations, ground is considered to be consisting of clay.

For the test locations 8, 10 and 11, Kaolinite-rich clay layers interbedded with low quality lignite seams are found within a depositional basin surrounded by Volcanic and Pyroclastic rocks at the locations. Kaolinitic and Bentonitic clay deposits are encountered both in Cretaceous Flysch and in the Tuffs of Miocene Pliocene age. Clays of the locations belong to the group CH. At test locations 4, 6 and 9, the clay soil was formed from alteration of Perlite and Pyroclastic rocks of Pliocene age. The clay is Montmorillonite type and belongs to the group CH. At test locations 2, 3, 5 and 13, the clay was formed from alteration of Limestone Eosene age. The clay of the locations belong to the group CL. For test locations 1, 7 and 12, the clay was formed from alteration of Eosene-Miosene Limestone and Basaltic rocks of Quarterner age. The clays of the locations belong to the groups CL and ML. The Atterberg Limits of the samples which are determined from the test locations are shown in Table 1.

METHODOLOGY

The PMT can be performed on soils and weak rocks which have uniaxial comprehensive strength not exceeding 10 MPa (Yıldırım, 2002). Limit creep pressure (P_L) and Menard elasticity module (E_P) of soil can be determined by the application of PMT in the drilling holes (ASTM D4719, 2007). Thus, bearing capacity (q), settlement (S) and stress-strain characteristics of foundation ground are exposed (Sols Soils, 1975). The PMT device consists of 4 main parts as seen in Figure 2.

To obtain more realistic soil characteristics, the PMT must be performed right after the drilling. Test-depth intervals should be determined by the concerned engineer according to the properties of the project.

Probe is sent to test level and filled with pressured air. Water volume changes on the control panel are recorded. We usually wait for one minute for stabilization of the pressure level. Water volume changes are recorded for all predetermined pressure level. The decreasing in water volume level means, there is volume expansion in the drilling hole. Pressure level must be increased till the ground yields. The reason of pressure increasing is to measure the yielding pressure point (P_L) and the modulus of deformation (E_P) of the hole side walls (Kumbasar and Kip, 1999). By the application of the PMT, 2 important data are obtained. The pressuremeter modulus (E_P) can be used in estimating the coefficient of subgrade reaction and calculating the settlement beneath a foundation. Limit pressure (P_L) can be used in calculating the ultimate bearing capacity of a foundation. Then the stress-strain graph is drawn as in Figure 2. X axis represents the applied pressure level (kPa) and Y axis represents the volumetric changes (cm^3). As shown in Figure 3, turning point of the curve to the infinite means the bearing capacity of ground (Agan, 2009).

This stress-strain graph is drawn for each test depth levels. Finally, all yielding pressure points (P_L) and Menard elasticity module values (E_P) are carried on a logarithmic graph as shown in Figure 4 (Agan, 2009).

Bearing capacity and settlements are calculated with the help of this logarithmic graph. The bearing capacity is calculated by the following formula

$$q = q_o + k * P_L \quad (1)$$

(Sols Soils, 1975)

q, Bearing capacity (kPa); q_o, Surcharge load (kPa); P_L, limit creep pressure (kPa); k, coefficient depends on soil type and foundation shape; “q”, value is divided to safety factor.

$$q_a = q / SF \quad (2)$$

(Sols Soils, 1975)

q_a, Safe bearing capacity (kPa); SF, safety factor.

In this study, bearing capacity values (q) are not divided to safety factor. Because, according to the importance of the project, safety factor can be changed by engineer (Tosun, 1988).

The PMT device, used in this study, had a prebored Menard GC type having a 60 mm probe diameter. The PMT probe was decreased to test level and filled with pressured air. The pressure level was immobilized and waited for 1 min (Coduto, 1999). The water volume changes were noted for all pressure levels. Pressure levels were increased till the ground yields. Then, the stress-strain graph was drawn for each hole. By this way, the limit creep pressure points (P_L) of the drilling-hole side walls were determined. Then, all yielding pressure points (P_L) were carried on a logarithmic graph. Finally, bearing capacities were calculated with this logarithmic graph and Equation 1.

In this study, surcharge loads (q_o) have been ignored because only the generalization of bearing capacity of clay soil was investigated. Surface loads are insignificant at shallow levels, but significant at deep levels. They are affected by unit volume weight of the ground, and naturally they change from place to place (Canadian Geotechnical Society, 1975).

The coefficient k, which depends on soil type and foundation shape, was taken as 0.8 for clay invariably and singular foundation (Smith, 2006). It is assumed that all the foundation characteristics are the same (Foundation length B = 10 m, Foundation width L = 5 m). By this way, only the bearing capacity of clay soil is the decisive.

RESULTS AND DISCUSSION

The limit creep pressure results (P_L) of the drilling-hole side walls are presented in Table 2. All the calculated q values are presented below in Table 3.

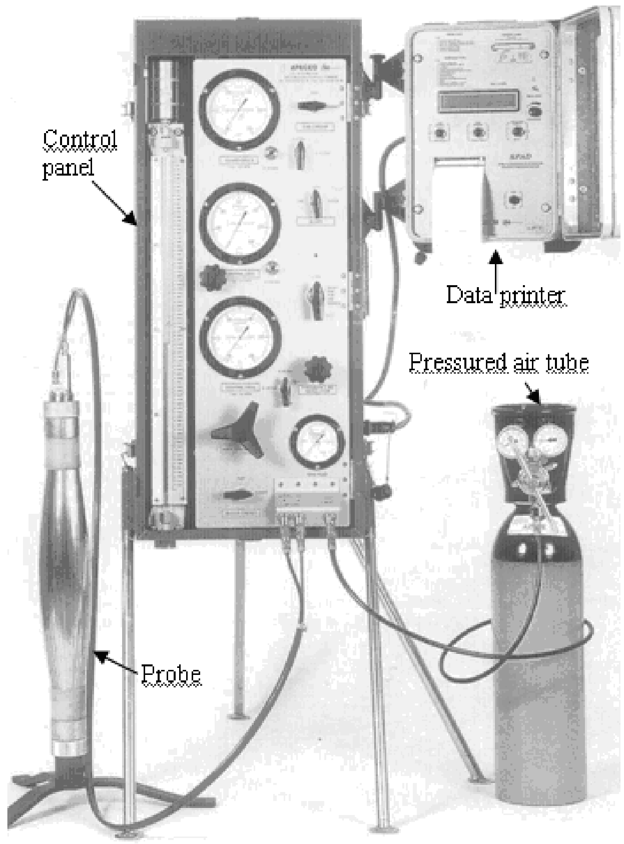

Figure 2. The PMT device (Sols Soils, 1975).

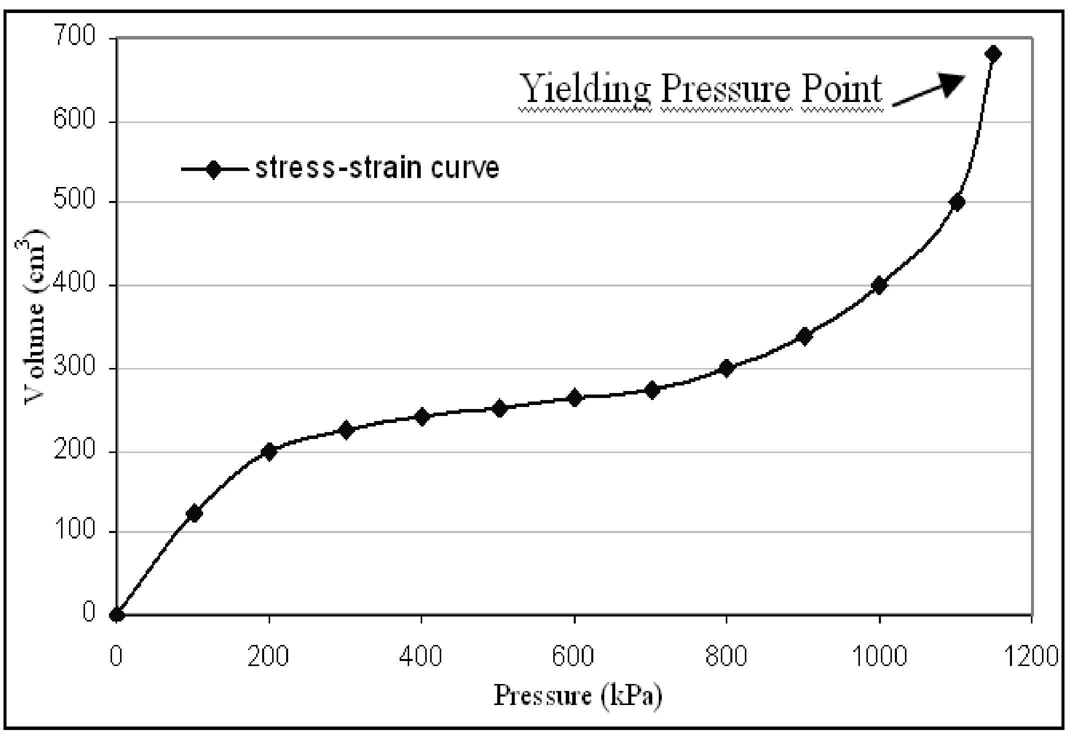

Figure 3. Stress-strain graph for a test depth level (Agan, 2009).

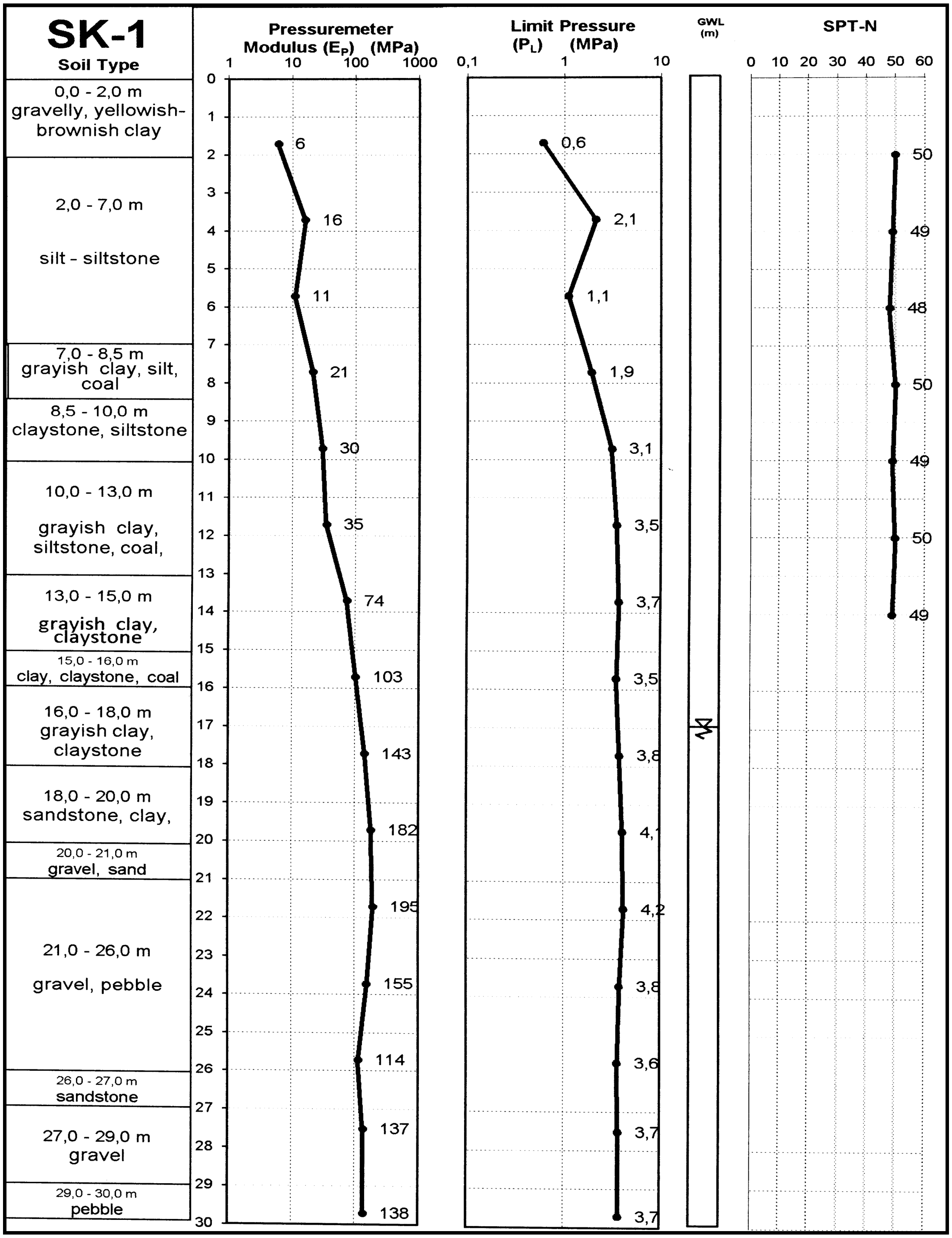

Figure 4. Logarithmic test graph (Agan, 2009).

To examine the bearing behaviour of clays at different foundation depths, all of the 13 calculated q values were drawn together in a graph, as it shown in Figure 5.

The 3rd, 6th and 7th test results are explicated as too much changeable. The instability of 7th test result is thought to result from underground water. Underground water decreases ground strength during the test and causes insubstantial low parameters. The instability of 3rd and 6th test results is thought to result from large gravel pieces in the clay ground. Gravel piece increases the ground strength during the test and causes insubstantial high parameters. For this reason, 3rd, 6th and 7th test

Table 2. Limit creep pressure values for different foundation depths at 13 test locations.

Test locations	P_L values (kPa) at the below test depth levels				
	2 m	4 m	6 m	8 m	10 m
1	300	700	1500	2000	4000
2	100	300	500	600	700
3	1000	1600	2400	3000	5500
4	300	600	700	800	1400
5	200	800	1000	1800	2200
6	600	1200	3300	3500	4000
7	500	1000	2300	3500	2100
8	500	1500	1600	2000	3000
9	200	400	900	1400	1800
10	300	500	900	1700	2700
11	300	1000	1400	1900	2300
12	500	700	900	1500	2300
13	300	700	1100	2300	3500

Table 3. Bearing capacity values for different foundation depths at 13 test locations.

Test locations	q values (kPa) at the below test depth levels				
	2 m	4 m	6 m	8 m	10 m
1	240	560	1200	1600	3200
2	80	240	400	480	560
3	800	1280	1920	2400	4400
4	240	480	560	640	1120
5	160	640	800	1440	1760
6	480	960	2640	2800	3200
7	400	800	1840	2800	1680
8	400	1200	1280	1600	2400
9	160	320	720	1360	2160
10	240	400	720	1360	2160
11	240	800	1120	1520	1840
12	400	560	720	1200	1840
13	240	560	880	1840	2800

results were decided not to be taken into account. To examine the bearing behaviour of clay soils at different foundation depths, 10 stable q values and their average ($q_{average}$) were drawn together in a graph, as it shown in Figure 6.

Equation 3 was produced to calculate the average bearing capacity as follows.

$$y = 176.17 * e^{0.2475 * x} \quad (3)$$

In this equation, y represents the bearing capacity and x represents the foundation depth level. And finally, Equation 4 was formed as

$$q = 176.17 * e^{0.2475 * D_f} \quad (4)$$

D_f, Foundation depth level (m); e, exponential function (approximately 2.718281828)

It is anticipated that this study will make an important addition to the current literature and the obtained equation may help the practicioners in dealing with the evaluating, comparing, interpreting or cross checking of the soil parameters obtained from this important *in-situ* test.

Conclusions

Results of this study indicate that, application of the PMT in 13 different clay formations in Turkey has meaningful conclusions. In this respect, the key point is the bearing capacity of clay soils. The bearing behaviours of different

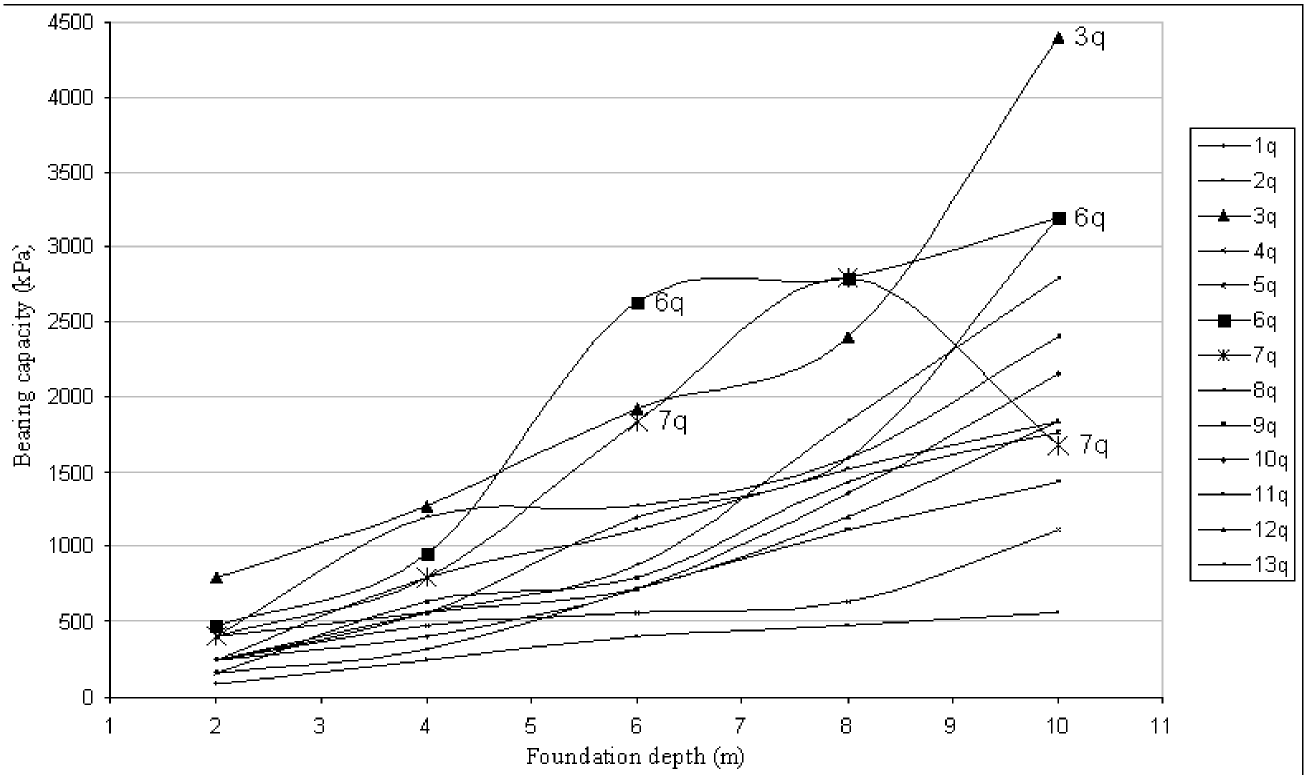

Figure 5. Bearing capacity (q) graph for different foundation depths at 13 test locations.

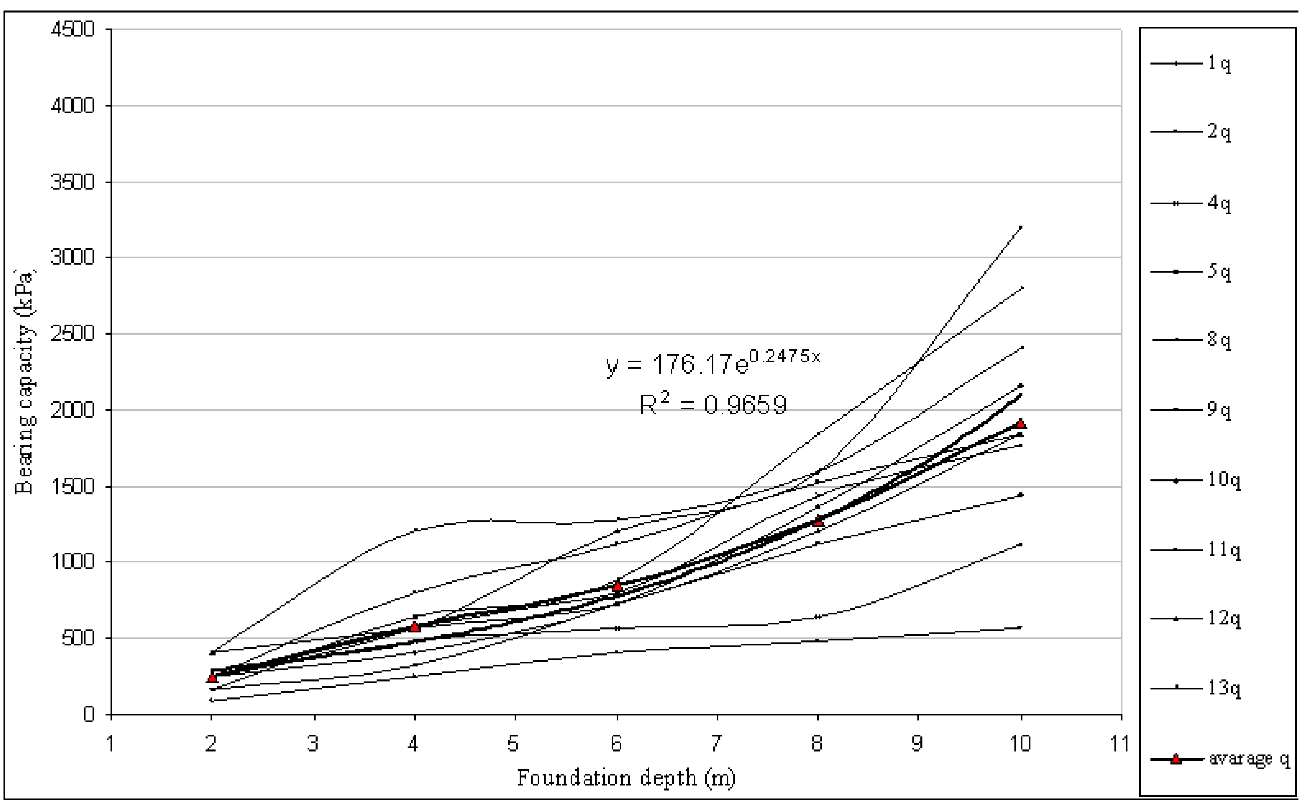

Figure 6. Average bearing capacity ($q_{avarage}$) graph for different foundation depths at 10 test locations.

clays at different foundation depths were investigated. And all of the 13 calculated q values were drawn together in a graph. Bearing capacities were calculated for 5 foundation depths (2, 4, 6, 8 and 10 m) and ranged between 240 to 400, 400 to 1200, 720 to 1900, 1200 to 2800 and 1700 to 3200 kPa, respectively.

The equation of the average bearing capacity trend line was also generated. By this equation, by replacing the foundation depth value with (x), the bearing capacities of clays can be compared in Turkey.

The importance of determining the characteristics and behaviors of formations with limited experiments will inevitably increase in the near future. By Equation 4, developed in this study, Geotechnical Engineers can determine the data which is required for their site investigations with a few *in situ* tests.

It is believed that, by the application of Equation 4 in different site investigations and comparison of the outcomes, Equation 4 could be performed in a more efficient manner. This may lead Geotechnical Engineers to a supply of practical bearing capacity data.

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