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ORIGINAL PAPER

A comparison between undrained shear strength of clayey soils acquired by "PMT" and laboratory tests

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Abstract

A pressuremeter test (PMT) is one of the in situ tests, which is used to evaluate deformation and strength parameters of soils for various projects, including subway projects. The limit pressure (P_L) and undrained shear strength (S_u) are the key parameters that are obtained directly and indirectly from the pressuremeter testing results. This research was carried out using geotechnical information obtained from a subway project in Qom city, Iran. Based on 44 PMT and uniaxial tests on very stiff to hard saturated clayey soils, a linear empirical equation between $S_u - P_L$ and $S_u - P_L{}^* = (P_L - \sigma_H)$ with $R^2 = 0.68$ was proposed and it was found that σ_H had an insignificant effect on the proposed relationship. The effect of physical properties of soil, including plastic index (PI), liquid limit (LL), and water content (ω), was evaluated, and a multivariate equation was proposed between them. A comparison between the equations obtained in this research and those proposed by other researchers reveals that the empirical relationships between S_u and P_L are associated with the consistency of soils; the stiffer the soil is, the slope of relationship between S_u and P_L is less.

Keywords Pressuremeter test · Undrained shear strength · Clay soil · Limit pressure · Physical properties

Abbreviations

PMT	Pressuremeter test
PBP	Pre-boring pressuremeter
SBP	Self-boring pressuremeter
P_L	Limit pressure
E_{PMT}	Pressuremeter modulus
β	Pressuremeter constant
S_u	Undrained shear strength
σ_H	Total horizontal stress
PI	Plastic index
LL	Liquid limit
ω	Water content
CPT	Cone penetration test
FVT	Field vane test
FDT	Flat dilatometer test
CU	Consolidation untrained
SPT	Standard penetration test
N_{SPT}	Standard penetration number
OCR	Over consolidation ratio

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Introduction

A PMT is one of the in situ tests in geotechnical engineering that is used to identify and assess the S_u, the deformation parameters, the in situ σ_H, and the permeability coefficient of soils (Ménard 1957a). It allows engineers to design stable foundations and pavements in various conditions by obtaining several parameters (Foriero and Ciza 2016). This test was developed for the first time by Ménard in 1957b. In this test, a cylindrical balloon expands under the pressure of the fluid within the borehole and changes in volume; hence, pressures can be measured continuously. Then these data are plotted in terms of the applied pressures against the change in volume or the change in the radius of the balloon. In Fig. 1 a and b, a

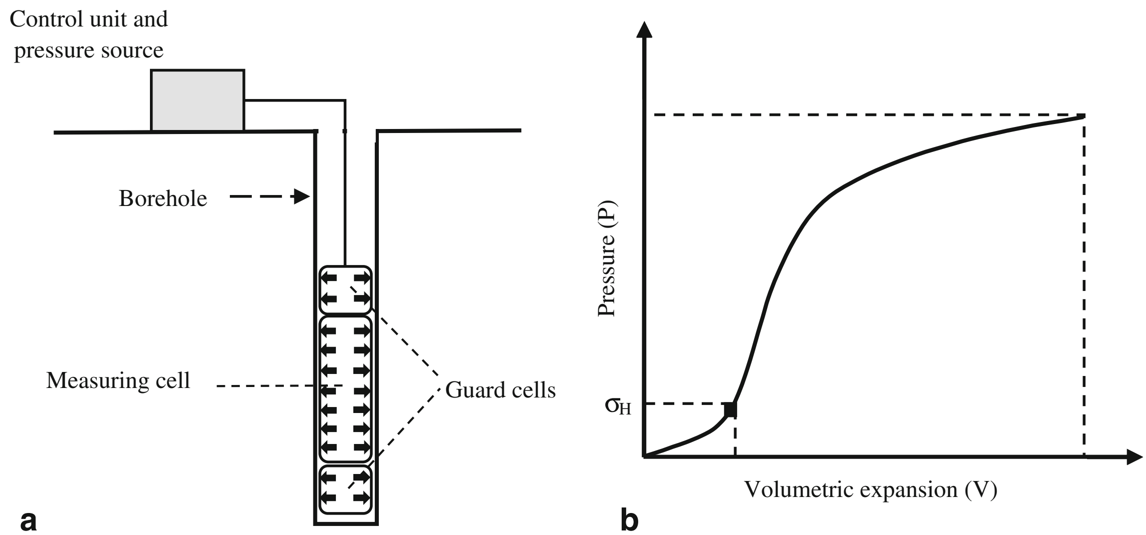


Fig.1 **a** Schematic diagram of a pressuremeter test device and b a typical cavity pressure curve (Soleimanbeigi 2013)

schematic diagram of a pressuremeter test device and a typical cavity pressure curve are shown.

In Fig. 1b, P_L is the limit pressure and σ_H is the total horizontal stress. The limit pressure is defined as the maximum pressure in which the cavity wall expands indefinitely. The σ_H is defined as the static stress of earth. There are several methods to determine the σ_H from a PMT curve, all of which estimate this stress approximately (Clarke 1995). σ_H can be obtained from the initial part of the elastic deformation of pressure–volume diagram of PMT (as shown in Fig. 1b), representing an overcome on the σ_H and also the beginning of the elastic deformation of soil (Briaud 1992).

The standard method of carrying out this test on site is described in ASTM-D4719-16. There are several different types of pressuremeters. The major difference between various categories of pressuremeters lies in the method of installation of the instrument into the ground. The borehole pressuremeter is the most common and has a probe that is inserted into a preformed borehole. One of the problems with this test is that conducting drilling may cause disturbance in soil, and accurate results cannot be achieved. For this reason, Wroth and Hughes (1972) and Baguelin et al. (1972) developed a test called the SBP to reduce the effect of soil disturbance during the drilling operation. A new geotechnical in situ test technique using a self-boring in situ shear pressuremeter (SBISP) was developed by Wang et al. (2018) to evaluate the initial state (horizontal at rest pressure), deformation modulus, and strength characteristics of soil. Briaud and Gambin (1984) proposed a method for drilling borehole for PMT. Elton (1981) and Ohya et al. (1982) investigated the effect of elastic tube strength on the pressuremeter modulus. Haberfield and Johnston (1988) simulated PMT of soft rock under controlled experimental conditions. Agan (2013), Oge, (2018) and Omar et al. (2018) determined the deformation modulus and tensile strength in weak rock mass by using pressuremeter. Kincal and Koca (2019) conducted a research to comparison between the E_{PMT} in andesitic rock mass and the values of elastic modulus of intact rock core specimens. Smith and Rollins (1997) conducted PMT in arid collapsible soil. Kayabasi and Gokceoglu (2018) used PMT in order to modify liquefaction analysis methods. Tu (2018) determined the coefficient of horizontal subgrade reaction with the PMT. Oztoprak et al. (2018) proposed a numerical methodology for capturing the complete curve of a PMT. Silvestri and Tabib (2018) analyzed field test results obtained by PMT in a sensitive clay soil of Quebec.

Some geotechnical parameters of the soil can be measured by the data taken from the PMT, so that these parameters are comparable to the results of other tests. For instance, Tarawneh et al. (2018) estimated E_{PMT} and P_L from CPT for desert sand and compared them with the results of the PMT. Firuzi et al. (2019) presented correlation between SPT, CPT, and pressuremeter data in alluvium soil. In addition to what mentioned above, many researchers were able to demonstrate the empirical relationship between the PMT and the SPT. They provided a number of relationships between the N_{SPT} obtained from the SPT and the values of the E_{PMT} and the P_L obtained from the PMT; and they proposed to use those relationships for similar soils (Yagiz et al. 2008; Bozbey and Togrol 2010; Agan and Algin 2014; Cheshomi and Ghodrati 2015; Anwar 2016; Özvan et al. 2018; Ziaie Moayed et al. 2018).

S_u of soil is an important parameter in design of geotechnical engineering structures. There is considerable demand to obtain S_u of fine-grained soils in many geotechnical problems, because it is a fundamental property (Bol et al. 2019). The S_u of shallow strata is a critical parameter for a safety design in deep-water operations (Li et al. 2019). S_u can be measured by a variety of in situ tests such as SPT (Sowers 1979), CPT (Lunne et al. 1997; Cheshomi 2018) flat diameter test (Młynarek et al. 2018), VST (Clayton et al. 1995), and PMT (Marshland and Randolph, 1977). In addition, uniaxial, triaxial, and direct shear tests on undisturbed samples are some of the routine laboratory tests for the determination of S_u (Cheshomi 2018; Bol et al. 2019). Belkhatir et al. (2013) studied the relationship between the S_u of the sand–silt mixtures. Cabalar et al. (2018) showed S_u decrease with increasing sand content in clay soil. Chung et al. (2012) presented an estimation of the S_u of Busan clay through a comparison of the results of field vane and laboratory shear test results at two sites in the Nakdong River Delta. Shimobe and Spagnoli (2019) presented a correlation between liquidity index and S_u.

Researchers tried to develop several empirical, analytical, and numerical methods to interpret this parameter through the PMT. Although the shearing plane obtained by pressuremeter differs from those in the conventional laboratory strength (Seah and Shrestha 2006), empirical methods are based on the relationship between properties of soils (S_u, the internal angle friction, the cohesion, OCR, LL, PI etc.) and parameters of pressuremeter (P_L and E_{PMT}) (Ménard 1957a; Amar and Jézéquel 1972; Nasr and Gangopadhyay 1988). S_u of clay soils can be interpreted by empirical equations. However, the analytical methods are based on the plane strain condition, isotropy, soil homogeneity, and undrained conditions, and they can be used to evaluate the behavior of soil under different conditions (Palmer 1972; Ladanyi 1972; Denby 1978; Ferreira and Robertson 1992; Monnet 2007). When a more precise solution of the PMT for soils is required, numerical solutions would be used to estimate the S_u of clays (Zentar et al. 2001; Monnet 2007). Tschebotarioff (1973), Parcher and Means (1968), and Terzaghi et al. (1996) based on S_u of fine-grained soils divided clayey soils into several categories, including very soft, soft, medium, stiff, very stiff, and hard. Table 1 presents the classification of fine-grained soils based on their S_u, proposed by different researchers.

Palmer (1972) provided a theoretical method for interpreting the S_u of clay through the PMT. Using the results of the PMT, they plotted stress–strain graphs of saturated clay and then used the graphs to calculate the S_u of clay. Komornik and Frydman (1969), Amar and Jézéquel (1972), and Marshland and Randolph (1977) conducted numerous tests to suggest a number of empirical relationships to interpret the S_u of clayey soils through PMT. Baguelin et al. (1972) presented a guideline to estimate the strength of clayey soils and the relative density of sandy soils using PMT. The results of studies by Houlsby and Carter (1993) and Bowles (1996) for estimating S_u by PMT showed that the values obtained using PMT was larger than those determined in the field or laboratory (e.g., triaxial compression tests).

Bahar et al. (2012) developed a theoretical model and conducted a numerical analysis on saturated clay in a PMT to determine S_u, and then proposed a model to predict S_u. There was a good agreement with the results of model on the pressuremeter path and the experimental data from other methods (the empirical methods proposed by Ménard 1957a, and Amar and Jézéquel 1972, and the field vane and triaxial tests). Bahar et al. (2012) carried out several pressuremeter tests in clayey soils in different places. They interpreted S_u of soils by three methods. The results of their studies showed that the values of S_u estimated by the Bahar and Olivari (1993) method were greater than those estimated, using the Ménard method (1957a) and Amar and Jézéquel method (1972).

Soleimanbeigi (2013) analyzed the data taken from PMT regarding consolidated organic silt and overconsolidated silty clay to estimate S_u based on the traditional closed-form solution and the finite element method. After comparing the results, he found that the estimated values of S_u predicted from the finite element method were lower than those estimated from the traditional method (Gibson and Anderson 1961). Isik et al. (2015) conducted a numerical study on the effects of the length (L) and radius (R) of the pressuremeter and also the depth of testing on the values of shear strength obtained by the PMT. They proposed a correction coefficient based on the ratio of L/D and observed that the values of S_u obtained by the empirical relationships of pressuremeter were closer to the values obtained through the CPT, FVT, and CU tests.

Alzubaidi (2015a) evaluated the horizontal at rest pressure by five different methods of interpretation (the inflection point method, the numerical iteration method developed by Gibson and Anderson 1961, the graphical

Table 1- Types of fine-grained soils according to undrained shear strength

	S_u (kPa)		
Consistency	Tschebotarioff (1973)	Parcher and Means (1968)	Terzaghi and Peck (1967)
Very soft	15	< 12	< 12.5
Soft	15–30	12–25	12.5–25
Medium	30–60	25–50	25–50
Stiff	60–120	50–100	50–100
Very stiff	120	100–200	100–200
Hard	> 225	> 200	> 200

S_u undrained shear strength

iteration method developed by Marsland and Randolph 1977, the up side-down curve method developed by Van Wambeke and Hericourt 1975, the stress relief method developed by Alzubaidi 2015b).

Alzubaidi (2015b) evaluated the S_u in sandy silt soil by three methods for interpretation (Gibson and Anderson 1961; Palmer 1972; Ladnayi 1972) and five methods of analysis for deducing the horizontal at rest pressure. The results of that study showed that the values of S_u had a considerable difference, when using different values of the horizontal at rest pressure estimated by those five methods.

In Table 2, some relationships proposed by different researchers to calculate the S_u of clayey soils are presented.

Cassan (1972) and Amar and Jézéquel (1972) defined a relationship between S_u and P_L presented in Eq. (1) in Table 2. The effect of total horizontal stress as $P_L - \sigma_H$ was considered in their equation. Ménard (1957b) proposed a factor known as the pressuremeter constant (β). This factor can be obtained using the values of $P_L* = P_L - \sigma_H$ and S_u using this equation $\beta = P_L*/S_u$. Many researchers proposed a number of values for β. There are several factors that can make a difference in the values of β, such as disturbance of soil, anisotropy, lack of sufficient precision in measurement total horizontal stress at rest, and difference in the reference strength (Clarke1995).

In this study, to develop empirical relationships between S_u and P_L for very stiff to hard saturated clayey soils in one of the central cities of Iran, 44 pressuremeter and uniaxial test results were considered to develop proper relationships between S_u and P_L. Then, the effect of σ_H and some key physical properties of soil (such as LL, PI, and ω) on the proposed empirical equations were investigated. In the next step, the proposed relationships were compared with some equations, proposed by other researchers.

Materials and method

Site specification

Qom city is located 148 km south west of Tehran, Iran (34.6416° N, 50.8746° E). For construction of subway, an extensive geotechnical studies were conducted in line with a length of 15 km. This research was carried out using geotechnical information obtained from subway project in Qom city. For this purpose, 18 boreholes with depth between 25 and 40 m were drilled. The study area is underlain by recent alluvium. Figure 2 presents the location area and subsurface soil condition in the route of the study. Based on this figure five layers as follows Fig. 1c are separated. These soils were composed from sandy gravel and gravelly sand, silty clayey sand, silty clay, and clayey silt. In this study, mainly clayey silt and silty clay soils (layer nos. 2 and 3) were taken into account.

Uniaxial test

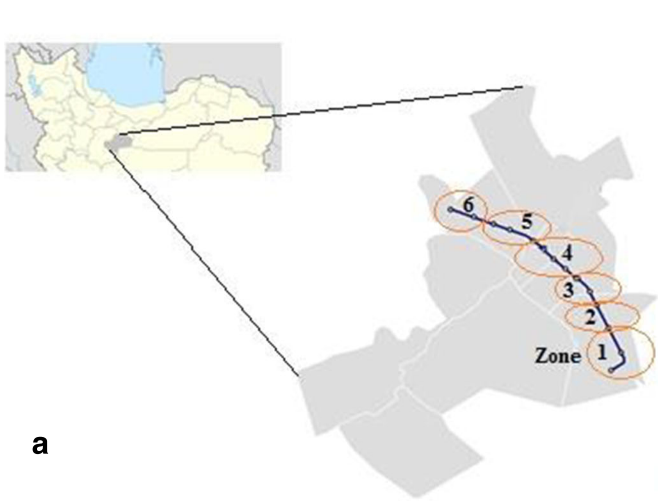
During drilling of boreholes, pressuremeter tests were performed according to the standard (ASTM D4719-16), and undisturbed samples were taken from soils in the sections where the pressuremeter tests were carried out and the undisturbed samples were transferred to the laboratory for testing. Thin-walled tube sampler was used for obtaining undisturbed sample based on ASTM D1587/D1587M-15.

Uniaxial tests were performed according to the standard (ASTM D2166-16) on undisturbed samples. An array of tests was performed to determine the S_u parameter of the soil. Although there is a shortcoming in the uniaxial test due to the lack of confining pressure, its application in geotechnical studies of projects is common, because of its

Table 2 Relationships provided by different researchers to obtain S_u by pressuremeter test

No.	Equations	Clay type	Reference	Remarks
1	$S_u = (P_L - \sigma_H)/5.5$	Soft to firm clays	Cassan (1972) and Amar and Jézéquel (1972)	Empirical method by pre-boring pressuremeter
2	$S_u = (P_L - \sigma_H)/8$	Firm to stiff clays		
3	$S_u = (P_L - \sigma_H)/15$	Stiff to very stiff clays		
4	$S_u = (P_L - \sigma_H)/10$	Stiff clays	Martin and Darhos (1986)	Empirical method by pre-boring pressuremeter and unconsolidated undrained shear test
5	$S_u = (P_L/10) + 25$	Soft to stiff clays	Johnson (1986)	Empirical method by pre-boring pressuremeter and undrained shear test
6	$S_u = 0.35(P_L - \sigma_H)^{0.86}$	Soft clays	Bozbey and Toghrol (2010)	Empirical method by pre-boring pressuremeter and unconfined compression strength test
7	$S_u = 0.67(P_L)^{0.75}$	Firm to stiff clays	Baguelin et al. (1972)	Analytical method by shear stress strain curve

P_L limit pressure, S_u undrained shear strength, σ_H total horizontal stress



Layer No.	Description
L-1	Filled Soil
L-2	Silty CLAY ((Passing 200 > 65% and PI>7)
L-3	Clayey SILT (Passing 200 > 65% and PI<7)
L-4	Sandy GRAVEL and gravelly SAND (Passing 200 = 5-20%)
L-5	Silty clayey SAND with gravel

c

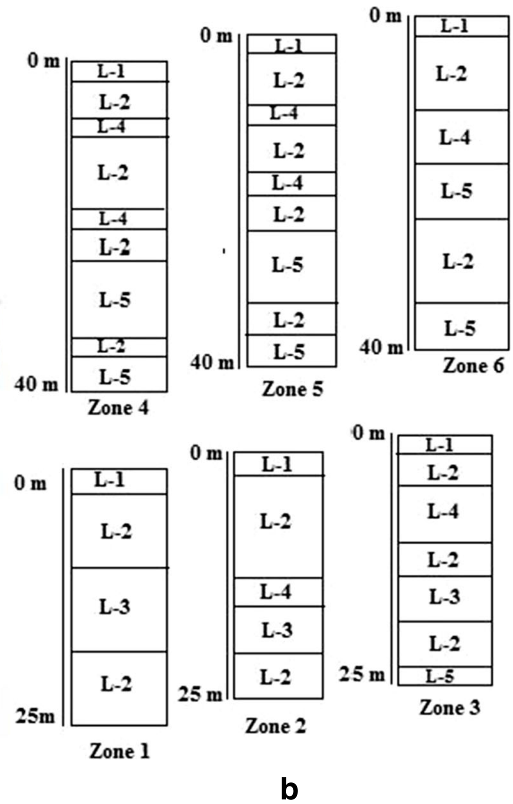


Fig. 2 **a** Location of site and subway rout. **b** Subsurface soil condition in the subway rout. **c** Soil layers' description

simplicity, low cost, and high efficiency. In this study, since clayey soils were saturated and the loading rate was relatively high, the shear strength values obtained from the uniaxial tests were regarded as S_u. In Fig. 3a, b and c, uniaxial test apparatus with a sample photo before and after test with an unconfined compressive strength (q_u) versus axial strain curve for the sample is shown respectivly.

Pressuremeter test

Pressuremeter tests were performed according to the standard (ASTM D4719-16). The pressuremeter, used in this research, was a PBP of type GC. The reason for choosing GC pressuremeter was its suitability for dense and hard soils (Baguelin et al. 1972). It can be noted that soils in the study area were very stiff to hard clays. In addition, it is widespread and more acceptable to use this type of pressuremeter in Iran. Depending on the type of employed pressuremeter, the shear modulus, the E_{PMT}, and the P_L can be obtained directly from results of pressuremeter tests. In Fig. 4, PMT apparatus with a pressure versus volume curve is shown. Equation (1) proposed for determining the E_{PMT} (Murthy 2008; Agan 2013):

$$E_{\mathrm{PMT}} = \frac{2(1+u)(V_0+V_m)\Delta P}{\Delta V} \tag{1}$$

where E_{PMT} (kPa) is the pressuremeter modulus, ν is the Poisson's ratio (equal to 0.33), and V_0 is the volume of the uninflated probe at the ground surface; ΔP, ΔV, and V_m in Fig. 4b are presented.

One derived parameter from the PMT is the limit pressure (P_L, pressure at which failure occurs); one of the ways to determine a limit pressure is to extrapolate the pressure–volume curve (Baguelin et al. 1972). In this method P_L is defined as the pressure where the probe volume reaches twice the original soil cavity volume ($V_0 + 2\mathrm{V_i}$), where V_i is the corrected volume reading at the pressure where the probe makes contact with the borehole. Based on ASTM D4719-16, if the test was performed to read sufficient plastic deformation, P_L can be determined by a 1/V versus pressure plot, as shown in Fig. 4c. Points from the plastic range of the test generally fall in an approximate straight line. The extension of this line to twice the original probe volume gives the P_L on the plot. In this research, this method is used to determine the P_L. Figure 5 illustrates the flowchart of the present research along in situ and laboratory variables for proposing empirical equations.

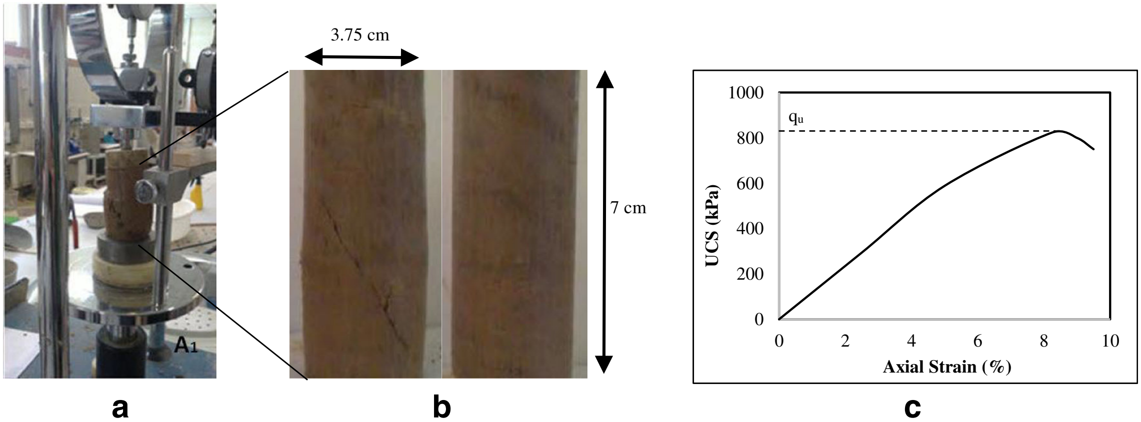


Fig. 3 **a** Uniaxial test apparatus. **b** A sample before and after test. **c** Unconfined compressive strength (q_u) versus axial strain for a sample

Test results

In this study, 44 pressuremeter tests and 44 corresponding uniaxial tests were performed. S_u values of soil samples were obtained by the uniaxial tests. Using the results of pressuremeter tests, P_L and σ_H were calculated. P_L and σ_H were used to interpret the S_u of clayey soils.

Limit pressure

Using pressure–volume curves acquired from the pressuremeter tests (as shown in Fig. 4c as a sample curve), the P_L was calculated for all 44 pressuremeter tests. The variation of P_L values with depth and frequency are shown in Fig. 6 a and b. Upper and lower bounds for P_L–depth graph have been drawn, indicating an increasing trend of P_L values with depth. As can be seen, most of the data are gathered between 2000 and 6000 kPa. The upper and lower bounds were drawn as visually so that most of the data fall within the range between the two lines.

Total horizontal stress

Values of σ_H were determined by the initial part of the elastic deformation of pressure–volume curve of the PMT (as shown in Fig. 1b). In Fig. 7 a and b, variations of σ_H with depth and frequency of σ_H are presented. As shown in Fig. 7a, the upper bound and the lower bound of σ_H are plotted and the trend of σ_H with depth is similar to that found in P_L; with increasing in depth, the σ_H increase, and based on Fig. 7b most of the data are located between 200 and 600 kPa.

Undrained shear strength

The undisturbed samples were taken at the same location, where the pressuremeter tests were conducted. The samples were sent to the laboratory for conducting uniaxial tests. Since the clay samples were saturated and not allowed to be drained in the uniaxial test apparatus, the angle of internal friction angle was assumed to be zero, similar to UU triaxial tests. Therefore, in this case, the S_u can be obtained by Eq. (2):

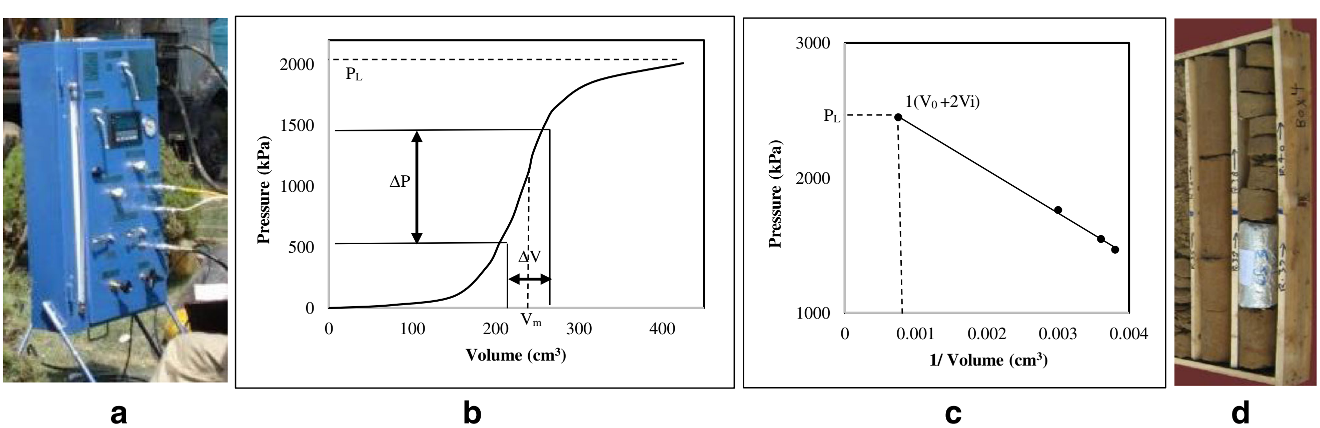


Fig. 4 **a** PMT apparatus. **b** Pressure versus volume curve. **c** Pressure versus 1/V to calculate the P_L. **d** The clay layer that has been tested

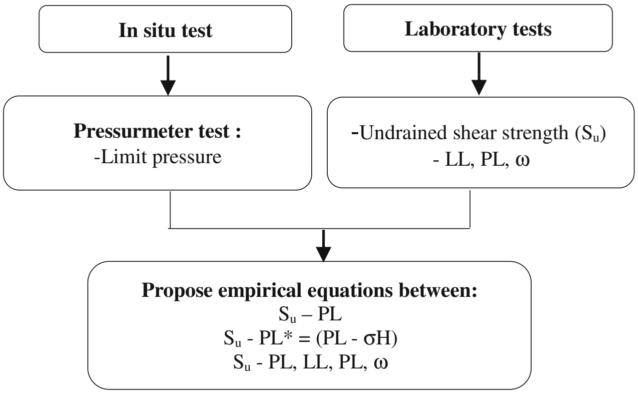


Fig. 5 The flowchart of the present study along with the measured parameters

$$S_u = {}^{q_u}/_2 \tag{2}$$

where S_u is the undrained shear strength and q_u is the uniaxial compressive strength (UCS), which is the maximum axial compressive stress that a upright-cylindrical sample of material can withstand before failure.

Figure 8 a shows the variation of S_u obtained by the laboratory tests against the depth. After drawing the upper and the lower bounds of the graph, it can be observed that with increasing the depth, S_u increases too, but with a slight slope. The frequency of S_u values is shown in Fig. 8b. The frequency range of S_u data used in this study was between 56 and 618 kPa.

As seen in the histogram shown in Fig. 8b, the values of S_u for most tested samples were more than 100 kPa. Therefore, with regard to the values of S_u and also according to classifications presented in Table 1, it can be concluded that the target clays were very stiff to hard clay.

Physical properties (PI, LL, and ω)

Using samples, taken during drilling, the plasticity index (PI), the liquid limit (LL), and the water content (ω) of soil samples were determined. The variation of PI, LL, and ω (%) with depth are shown in Fig. 9 a, b, and c, respectively. Upper and lower bounds for all 3 graphs are plotted in Fig. 9 a, d and c. As can be seen, PI, LL, and ω have an increasing trend with depth.

Empirical relationships between S_u and P_L

In Fig. 10, based on the data presented in previous section, the variation of S_u against the P_L is shown for all 44 tests.

Referring to Fig. 10, Eq. (3), a linear relationship between P_L and S_u, can be obtained with a determination coefficient of 0.68.

$$S_u = 0.55\ P_L + 58\ n = 44\ R2 = 0.68 \tag{3}$$

S_u and P_L are in kilopascal. This equation is valid for very stiff to hard clays with a P_L greater than 2000 kPa.

Many researchers have considered σ_H as a variable in the proposed relationship between the P_L and the S_u (Ménard 1957a; Amar and Jézéquel 1972; Lukas and De Bussy 1976; Marshland and Randolph, 1977; Martin and Drahos 1986). They used $P_L{-}\sigma_H$ instead of just P_L. In this study, in order to investigate the effect of σ_H, the value of this parameter was determined according to the results of the pressuremeter tests. Similar to the work of previous researchers, the values of σ_H were contracted from the values of P_L and their effects were captured in the proposed empirical equation. Figure 11 shows the variation of $P^*_L = P_L{-}\sigma_H$ against S_u.

According to Fig. 11, Eq. (4) is obtained between the $P^*_L = (P_L{-}\sigma_H)$ and S_u with a determination coefficient of 0.68,

$$S_u = 0.58\ (P_L{-}\sigma_H) + 75\ n = 44\ R2 = 0.68 \tag{4}$$

The units of S_u and P_L and σ_H are in kilopascal. This equation is valid for very stiff to hard clays with P^*_L greater than 1600 kPa.

The determination coefficients of Eqs. (3) and (4) showed that σ_H had insignificant effect on the relationship between the S_u and P_L, since the soils of the studied area were very stiff to hard clays. This finding was similar to the one previously proposed by Clarke (1995). Therefore, it seems that it is not necessary to implement σ_H in the relationships obtained.

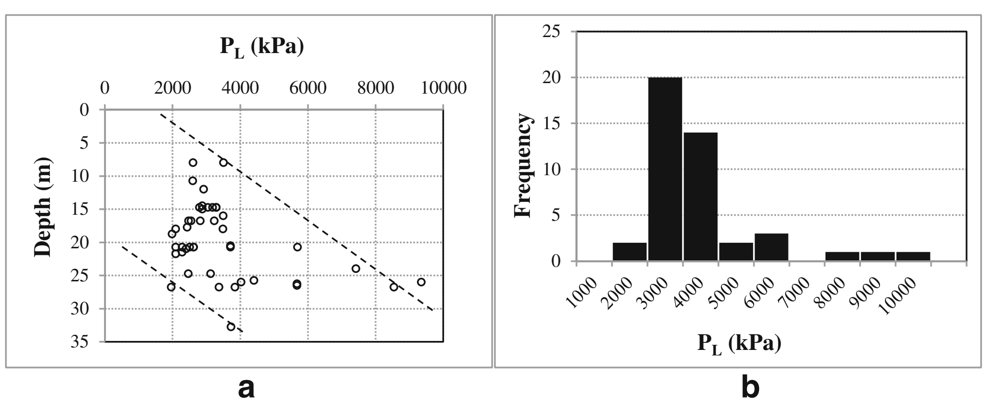
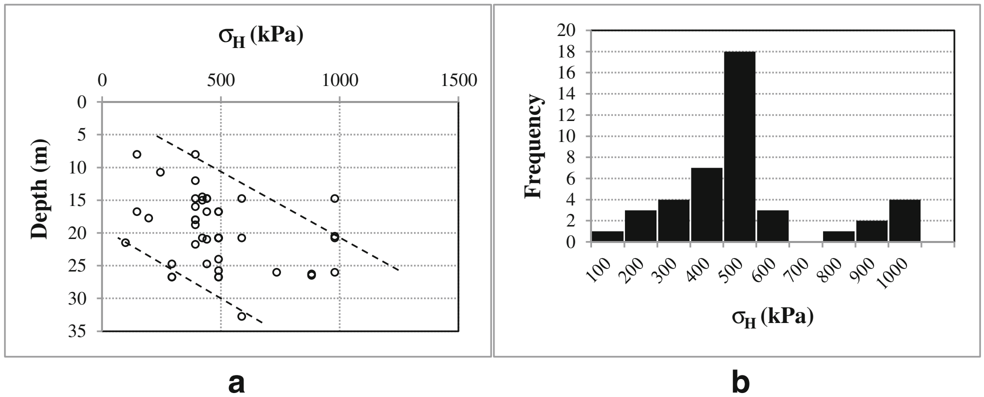


Fig. 6 **a** Variation of P_L with depth based on 44 pressuremeter tests. **b** Frequency range of P_L values

Fig. 7 **a** Variation of σ_H with depth based on 44 pressuremeter tests. **b** Frequency range of σ_H values



In order to study the effect of physical parameters of soil such as the PI, LL, and ω, a comparison between S_u and P_L, S_u, P_L, PI, LL, and ω were carried out through a multivariate analysis in SPSS statistical software. In this regard, S_u was considered as a dependent variable and other parameters including P_L, PI, LL, and ω were considered as independent variables. Equation (5) can be used to interpret S_u values for very stiff to hard clays.

$$S_u = 0.031 + 2.29\,LL + 0.54\,PI{-}1.77\,\omega + 0.0593\,P_L\ n = 44\ R2 = 0.73 \tag{5}$$

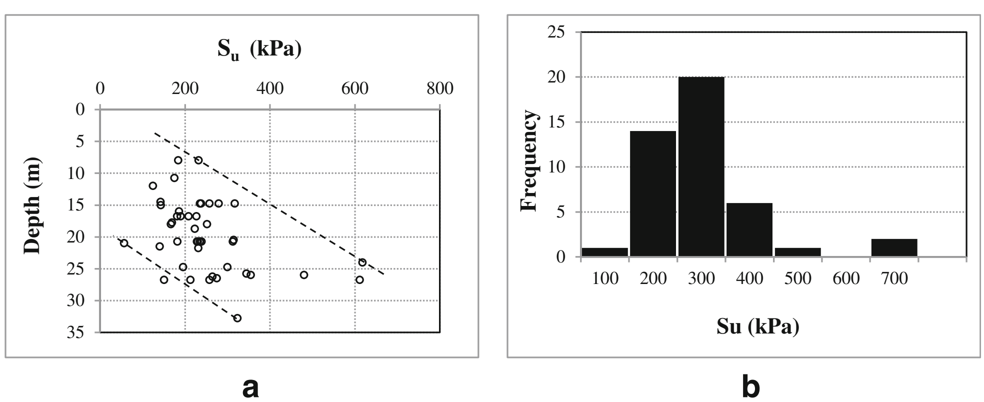
It is worth mentioning that S_u and P_L are in kilopascal and LL, PI, and ω are in percent. According to the observed determination coefficient, using physical parameters such as PI, LL, and ω of the soil can have a positive impact on results because the engineering properties of fine-grained soils depend on their physical properties. As can be seen in Eq. (5), the determination coefficient in multivariate method is 4% higher than that of the single variable method. Due to these enhanced determination coefficients obtained in the case of a single variable and a multivariate mode, S_u can be determined using either Eq. (3) or Eq. (5).

Discussion

In order to evaluate the proposed empirical relationships between S_u and P_L, a comparison between the proposed relationships in this research and the equations provided by other researchers (presented in Table 2) was made. For this purpose, the values of P_L obtained from the pressuremeter tests in this research were placed in relationships 1 to 7 and the values of S_u were estimated. Then the values of S_u estimated through the empirical equations were compared with the values measured from the laboratory tests. Figure 12 shows the data points and regression lines for comparison of proposed relations 1 to 7 by previous researchers with the proposed relationships of the present research (Eq. 3).

In Fig. 12, the line $Y = X$ (the black solid line) representing the determination coefficient of 1 is drawn up to determine the range of variations of values obtained from Eqs. 1 to 7 and the proposed equation of the present study. Comparison showed that all equations have the same trend, but slopes of the lines increased with softening of soil. The estimated values of S_u from the proposed equation of this study were within the range of values estimated by Eq. 3 that proposed by Cassan 1972 and Amar and Jézéquel 1972 for stiff to very stiff clays. The proposed

Fig. 8 **a** Variation of S_u with depth based on 44 uniaxial compressive strength tests. **b** Frequency range of S_u values



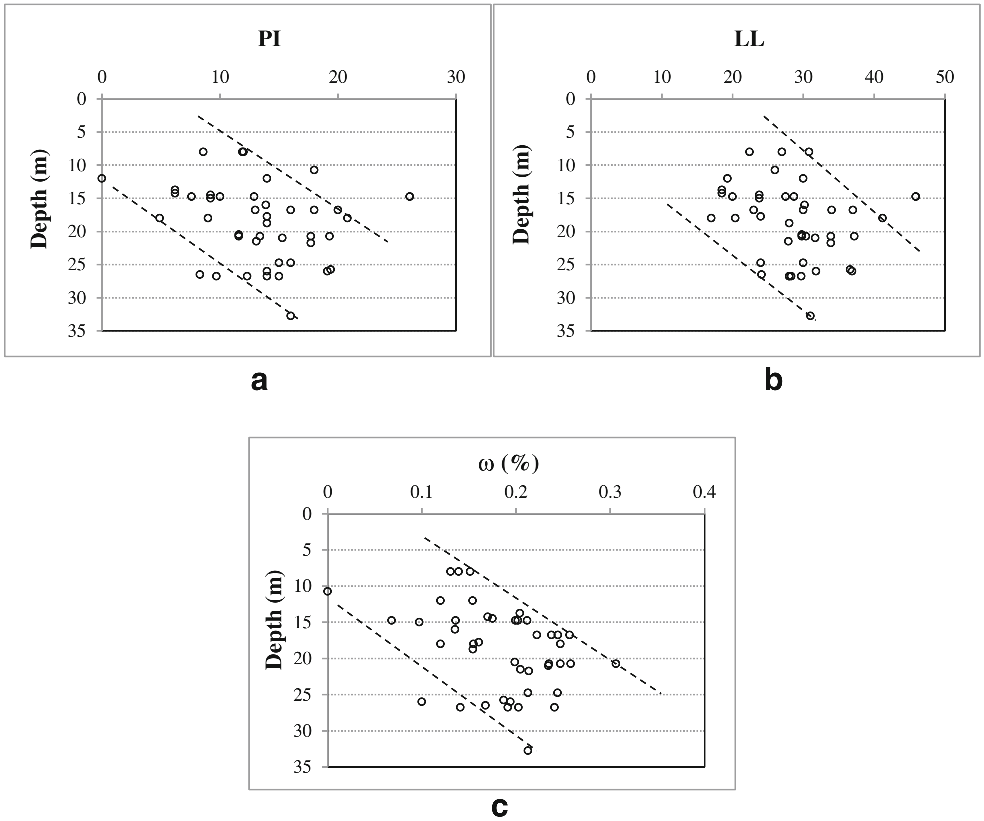


Fig. 9 Variation of soil properties with depth. **a** Plasticity index. **b** Liquid limit. **c** Water content

relationship in this research is also for very stiff to hard clays. Therefore, it can be concluded that the relationship proposed in this study is closer to Eq. 3 due to similarity in their consistency. Thus, clay consistency has a significant impact on the proposed empirical relationship; hence, for clay with different consistency, practitioners should use distinct experimental relationships. Accordingly, stiffness of clayey soil is dependent on physical properties of soil, especially the moisture content, as shown in Eq. (5). In other words, taking into account the physical properties (PI, LL, ω) may increase the determination coefficient.

The values of pressuremeter constant (β) in this study are plotted in Fig. 13a. The rang of β values was between 7 and 35 with an average of 12.6. In Fig. 13b, the β value in the present study was compared with the values presented by previous researchers in clay with different consistencies.

Cassan (1972) and Amar and Jézéquel (1972) proposed β values of 5.5 for soft to firm clays, 8 for firm to stiff clays, and 15 for stiff to very stiff clays. Marsland and Randolph 1977 recommended β value of 6.8 for stiff clays, and Lukas and De Bussy (1976) suggested the value of 5.1 for all clay soils. Martin and Drahos (1986) proposed 10 as a value for β when the soil is

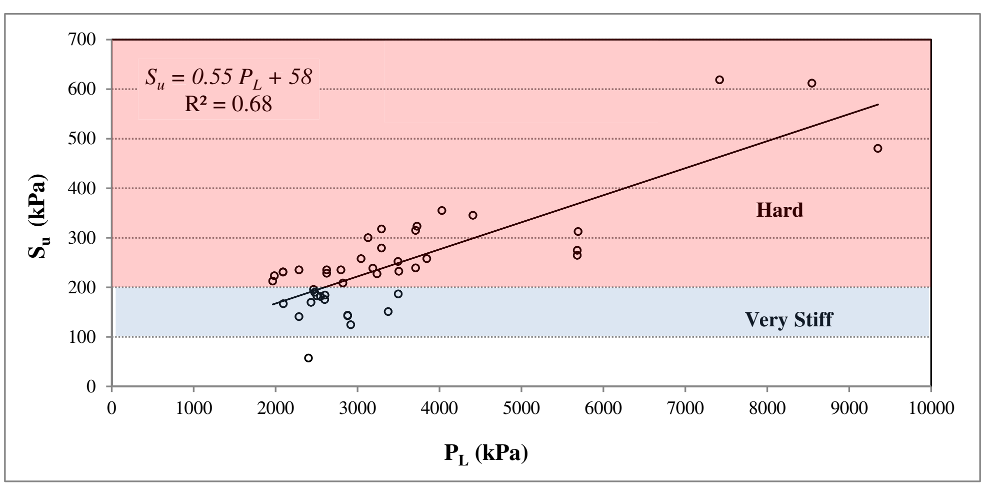


Fig. 10 Variation of S_u against the P_L for very stiff to hard clays

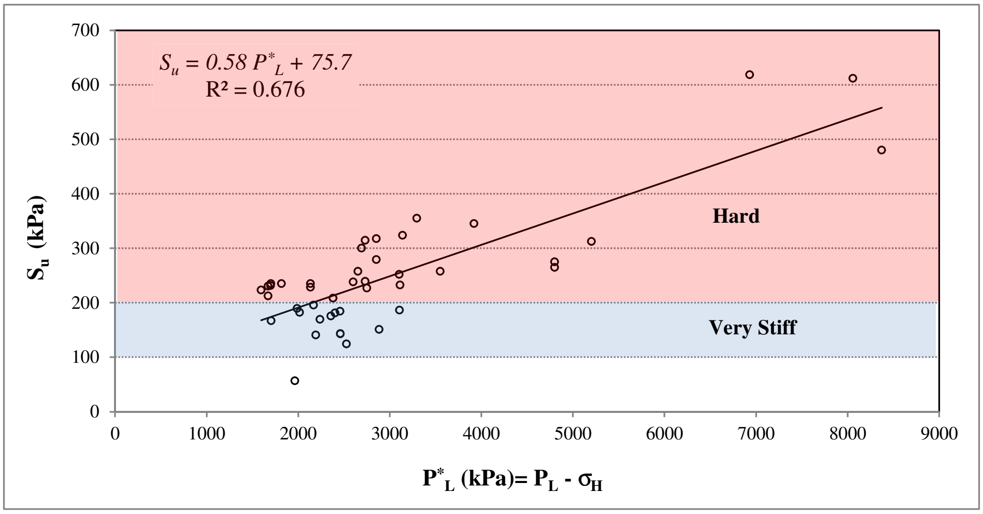


Fig. 11 Variation of Su against the $P^*_L = (P_L\text{-}\sigma_H)$ for very stiff to hard clays

stiff clay. The value of β, obtained in the present study, has a good agreement with those proposed by Cassan (1972) and Amar and Jézéquel (1972) for stiff and very stiff clays. Therefore, as discussed, the value of β is a function of clay consistency. Anisotropy, lack of sufficient precision in measurement, the total horizontal stress, and difference in the reference strength can affect the value of pressuremeter constant (Clark 1995).

Conclusions

This research was carried out based on 44 pressuremeter and uniaxial tests on very stiff to hard saturated clay samples. The key conclusions drawn from the findings of this study can be summarized as follows:

An empirical relationship between S_u–P_L for very stiff to hard saturated clay with $R^2 = 0.68$ was suggested. The estimated S_u from the PMT was less than the measured S_u obtained from laboratory tests. To evaluate impact of total horizontal stress, an empirical equation between S_u–$P^*_L = P_L$–σ_H with $R^2 = 0.68$ was proposed. By comparing the relationship between S_u–P_L and S_u–P^*_L it was found that σ_H had an insignificant impact on the proposed relationship.

In order to evaluate the effect of PI, LL, and ω on S_u and P_L, a multivariate equation was proposed. The finding revealed that incorporation of these variables can affect the proposed

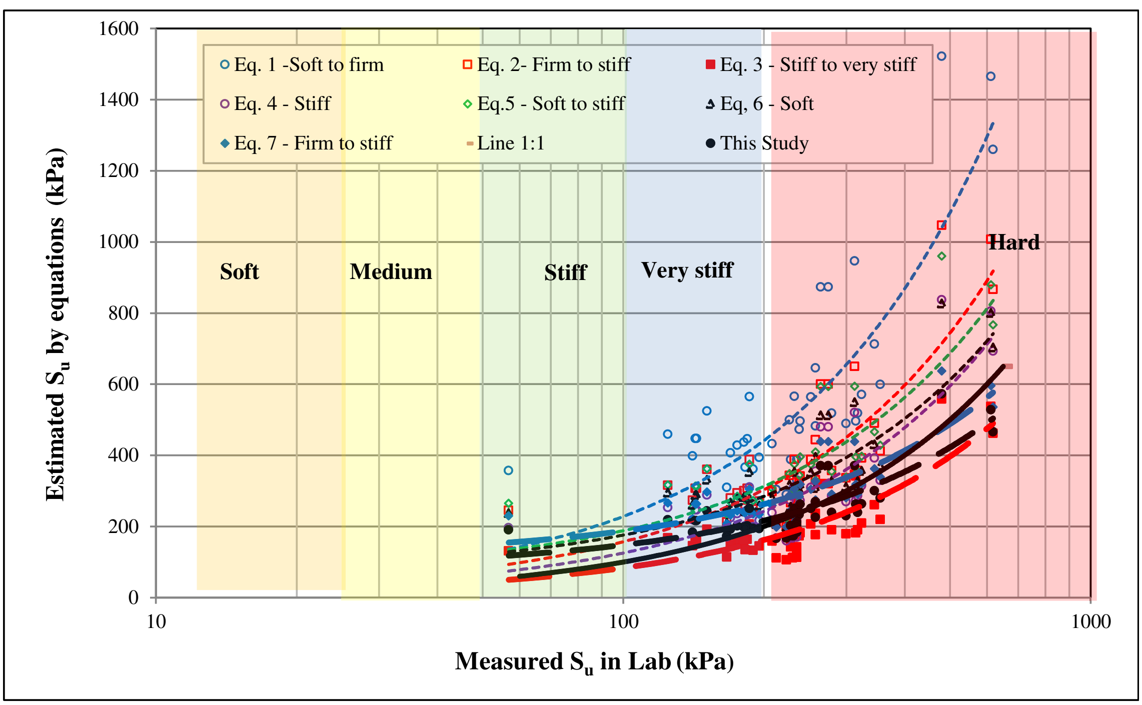


Fig. 12 Comparison of the proposed equations in present research to interpret S_u by pressuremeter tests with those equations proposed by other researchers

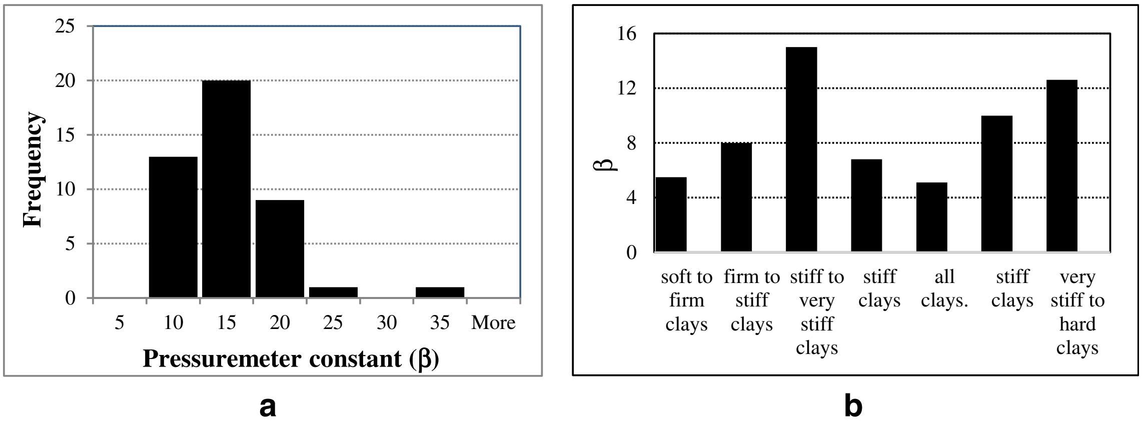


Fig. 13 **a** Histogram of Pressuremeter constant (β). **b** β value in this study and previous research

relationship and would increase the coefficient of determination.

The proposed equation in this research was compared with a number of equations proposed by other researchers. Those relationships were functions of consistency of soil, so that obtained values of S_u from the proposed equation developed in this study were within the range of values estimated by the equations proposed by Cassan 1972 and Amar and Jézéquel 1972 due to the similarity in their consistency.

The values of the pressuremeter constant (β) were dependent on clay consistency. In this study the average of β was 12.6, and it had a good agreement with those proposed by Cassan (1972) and Amar and Jézéquel (1972) for stiff and very stiff clays.

The applicability of proposed relationships in this research is for very stiff to hard clayey soils, with P_L between 2000 and 6000 kPa.

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References

Agan C (2013) Determination of the deformation modulus of dispersible-intercalated-jointed cherts using the Menard pressuremeter test. Int J of Rock Mech and Min Sci 65:20–28

Agan C, Algin HM (2014) Determination of relationships between Menard pressuremeter test and standard penetration test data by using ANN model: a case study on the clayey soil in Sivas Turkey. G T J 37(3):412–423

Alzubaidi RM (2015a) A new method for interpreting pressuremeter data to estimate in situ horizontal stress. Arabian J of Geo Sci 8(7):5295–5302

Alzubaidi RM (2015b) A new approach for interpretation strength sensitivity to Po in pressuremeter testing. Geotech and Geo Eng 33(4): 813–832

Amar S, Jézéquel JF (1972) Essais en place et en laboratoire sur sols cohérents: comparaison des résultats. Bull des Laboratoires des Ponts et Chaussées 58:97–108

American Society for Testing Materials (ASTM) ASTM D1587 / D1587M - 15 (2015) Standard practice for thin-walled tube sampling of fine-grained soils for geotechnical purposes. ASTM International West Conshohocken www.astm.org

American Society for Testing Materials (ASTM) D2166/D2166M-16 (2016) Standard test method for unconfined compressive strength of cohesive soil. ASTM International West Conshohocken www.astm.org

American Society for Testing Materials (ASTM) D4719-16 (2016) Standard test methods for prebored pressuremeter testing in soils. ASTM International West Conshohocken www.astm.org

Anwar MB (2016) Correlation between PMT and SPT results for calcareous soil. HBRC J 14(1):50–55

Baguelin F, Jezequel JF, Le M, Lemehaute A (1972) Expansion of cylindrical probes in cohesive soils. J. of Soil Mech & Found Div 98(Sm 11):1129–1142

Bahar R and Olivari G (1993) Analyse de la réponse du modèle de prager généralisé sur chemin pressiométrique. In actes du 6ème colloque franco-polonais de mécanique des sols. Appliquée: 97-104

Bahar R, Baidi F, Belhassani O, Vincens E (2012) Undrained strength of clays derived from pressuremeter tests. Eur J of Environ and Civil Eng 16(10):1238–1260

Belkhatir M, Schanz T, Arab A (2013) Effect of fines content and void ratio on the saturated hydraulic conductivity and undrained shear strength of sand–silt mixtures. Environ Earth Sci 70(6):2469–2479

Bol E, Önalp A, Özocak A, Sert S (2019) Estimation of the undrained shear strength of Adapazari fine grained soils by cone penetration test. Eng Geology 261:105277

Bowles LE (1996) Foundation analysis and design. McGraw-hill, New York

Bozbey I, Togrol E (2010) Correlation of standard penetration test and pressuremeter data: a case study from Istanbul, Turkey. Bull Eng Geol Environ 69(4):505–515

Briaud JL (1992) The pressuremeter. Balkema, AA

Briaud JL, Gambin G (1984) Suggested practice for drilling boreholes for pressuremeter testing. G T J 7(1):36–40

Cabalar AF, Khalaf MM, Karabash Z (2018) Shear modulus of clay sand mixtures using bender element test. Acta geotechnica Slovenica 15(1):3–15

Cassan M (1972) Correlation entre in situ en mechanique des sols. Internal report, Fondasol, Avignon

Cheshomi A (2018) Emperical relationships between CPTu results and undrained shear strength. J GeoEng 13(2):49–57

Cheshomi A, Ghodrati M (2015) Estimating Menard pressuremeter modulus and limit pressure from SPT in silty sand and silty clay soils A

case study in Mashhad, Iran. Geomechanics and Geoengineering 10(3):194–202
Chung SO, Hong YP, Lee JM (2012) Evaluation of the undrained shear strength of Busan clay. KSCE J of Civil Eng 16(5):733–741
Clarke BG (1995) Pressuremeters in geotechnical design. Blackie, London
Clayton C R I, Matthews M C, Simons N E (1995) Site Investigation. 2nd Edition, Wiley-Blackwell, 592 pp.
Denby GM (1978) Self-boring pressuremeter study of the San Francisco bay mud, Ph. D. Thesis the Stanford Univ.
Elton DJ (1981) The effect of elastic tube strength on the pressuremeter modulus. G T J 4(3):130–134
Ferreira RS, Robertson PK (1992) Interpretation of undrained self-boring pressuremeter test results incorporating unloading. Can Geotechn J 29(6):918–928
Firuzi M, Asghari-Kaljahi E, Akgün H (2019) Correlations of SPT, CPT and pressuremeter test data in alluvial soils, case study: Tabriz Metro Line 2. Iran Bull Eng Geol Environ 78:5067–5086. https://doi.org/10.1007/s10064-018-01456-0
Foriero A, Ciza F (2016) Confined pressuremeter tests for the assessment of the theoretically back calculated cross-anisotropic elastic moduli. G T J 39(2):181–195
Gibson RE, Anderson WF (1961) In-situ measurement of soil properties with pressuremeter. Civil engineering and public works review 56(658):615–618
Haberfied CM, Johnston IW (1988) Model studies of pressuremeter testing in soft rock. G T J 12(2):150–156
Houlsby GT, Carter JP (1993) The effects of pressuremeter geometry on the results of tests in clay. Geotechnique 43(4):567–576
Isik NS, Ulusay R, Doyuran V (2015) Comparison of undrained shear strength by pressuremeter and other tests, and numerical assessment of the effect of finite probe length in pressuremeter tests. Bull Eng Geol Environ 74(3):685–695
Johnson LD (1986) Correlation of soil parameters from in situ and laboratory tests for building 333—In Use of in situ tests in geotechnical engineering ASCE 635-648
Kayabasi A, Gokceoglu C (2018) Liquefaction potential assessment of a region using different techniques (Tepebasi, Eskişehir, Turkey). Eng Geol 246:139–161
Kincal C and Koca MY (2019) Correlations of in situ modulus of deformation with elastic modulus of intact core specimens and RMR values of andesitic rocks: a case study of the İzmir subway line. Bull Eng Geol Environ in press
Komornik A and Frydman S (1969) A study of in situ testing with pressuremeter. Proceedings of the conference organized by the British Geotechnical Society in London 145–154
Ladanyi B (1972) In-situ determination of undrained stress-strain behavior of sensitive clays with the pressuremeter. Can Geotech J 9(3): 313–319
Li Y, Hu G, Wu N, Liu C, Chen Q, Li C (2019) Undrained shear strength evaluation for hydrate-bearing sediment overlying strata in the Shenhu area, northern South China Sea. Acta Oceanol Sin 38(3): 114–123
Lukas RG, De Bussy BL (1976) Pressuremeter and laboratory test correlations for clay. J of Geotech and Geoenviron Eng 102:954–963
Lunne T, Robertson PK, Powell JJM (1997) Cone penetration testing in geotechnical practice. Blackie Academic, EF Spon/Routledge New York
Marsland A, Randolph MF (1977) Comparisons of the results from pressuremeter tests and large in situ plate tests in London clay. Geotechnique 27(2):217–243
Martin RE and Drahos EG (1986) Pressuremeter correlations for preconsolidated clay. In use of in situ tests in geotechnical engineering. ASCE 206–220
Ménard LF (1957a), Mesures in situ des propriétés physiques des sols-Annales des Ponts et Chaussées. l (3): 357-376
Ménard LF (1957b) An apparatus for measuring the strength of soils in slace. Doctoral dissertation, University of Illinois
Młynarek Z, Wierzbick J, Stefaniak K (2018) Interrelationship between undrained shear strength from DMT and CPTU tests for soils of different origin. G T J 41(5):20170365. https://doi.org/10.1520/GTJ20170365
Monnet J (2007) Numerical validation of an elastoplastic formulation of the conventional limit pressure measured with the pressuremeter test in cohesive soil. J of Geotech and Geoenviron Eng 133(9):1119–1127
Murthy S (2008) Geotechnical engineering: principles and practices of soil mechanics, 2nd edn. CRC, UK
Nasr AN, Gangopadhyay CR (1988) Study of su predicted by pressurmeter test. J Geotech Eng 114(11):1209–1226
Oge IF (2018) Determination of deformation modulus in a weak rock mass by using Menard pressuremeter. Int J of Rock Mech and Min Sci 112:238–252
Ohya S, Nagura M, Hosono M (1982) Discussion of the effect of elastic tube strength on the pressuremeter modulus by D. J Elton G T J 5: 101–103
Omar H, Ahmad J, Nahazanan H, Ahmed Mohammed T, Yusoff ZM (2018) Measurement and simulation of diametrical and axial indirect tensile tests for weak rocks. Measurement: J of the Int Measurement Confederation 127:299–307
Oztoprak S, Sargin S, Uyar HK, Bozbey I (2018) Modeling of pressuremeter tests to characterize the sands. Geomechanics and Engineering 14(6):509–517
Özvan A, Akkaya İ, Tapan M (2018) An approach for determining the relationship between the parameters of pressuremeter and SPT in different consistency clays in eastern Turkey. Bull Eng Geo Environ 77:1145–1154
Palmer AC (1972) Untrained plane-strain expansion of a cylindrical cavity in clay: a simple interpretation of the pressuremeter test. Géotechnique 22(3):451–457
Parcher JV, Means RE (1968) Soil mechanics and foundations. Charles E. Merrill, Columbus, Ohio
Seah TH, Shrestha D (2006) Simulation of pressurmeter shearing mode by true triaxial apparatus. G T J 30(2):141–151
Shimobe S, Spagnoli G (2019) Some relations among fall cone penetration, liquidity index and undrained shear strength of clays considering the sensitivity ratio. Bull of Eng Geo and the Environ 78(7): 5029–5038
Silvestri V and Tabib C (2018) Application of cylindrical cavity expansion in MCC model to a sensitive clay under Ko consolidation. J of Materials in Civil Engineering 30 (8): Article number 04018155
Smith TD, Rollins KM (1997) Pressuremeter testing in arid collapsible soils. G T J 20(1):12–16
Soleimanbeigi A (2013) Undrained shear strength of normally consolidated and overconsolidated clays from pressuremeter tests: a case study. Geotech and Geol Eng 31(5):1511–1524
Sowers GF (1979) Introductory soil mechanics and foundations. 4th, Macmillan, New York, 621 pp.
Tarawneh B, Sbitnev A, Hakam Y (2018) Estimation of pressuremeter modulus and limit pressure from cone penetration test for desert sands. Cons and Building Materials 169:299–305
Terzaghi K, Peck RB, Mesri G (1996), Soil mechanics in engineering practice. John Wiley & Sons
Tschebotarioff GP (1973) Foundations, retaining and earth structures, McGraw-Hill Book
Tu QZ (2018) Research on the testing method for determining the coefficient of horizontal subgrade reaction with the pressuremeter test. J of Railway Engineering Society 35(10):20–26
Van Wambeke AD, Hericourt J (1975) Coubed pressiometriques inverses: méthode d iterpretation de lessai pressiometrique. Soil-Soils 25:15–25

Wang K, Xu G, Wang J, Wang C (2018) Self-boring in situ shear pressuremeter testing of clay from Dalian Bay, China. Soils Found 58(5):1212–1227

Wroth CP and Hughes JMO (1972) An instrument for the in situ measurement of the properties of soft clay. Technical report soil TR13. University of Cambridge

Yagiz S, Akyol E, Sen G (2008) Relationship between the standard penetration test and the pressuremeter test on sandy silty clays: a case study from Denizli. Bull Eng Geol Environ 67(3):405–410

Zentar RHPY, Hicher PY, Moulin G (2001) Identification of soil parameters by inverse analysis. Comput Geotech 28(2):129–144

Ziaie Moayed R, Kordnaeij A, Mola-Abasi H (2018) Pressuremeter Modulus and limit pressure of clayey soils using GMDH-type neural network and genetic algorithms. Geotech and Geo Eng 36(1):165–178