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ORIGINAL PAPER

Evaluation of the relationships between the laboratory and in situ test results carried out on clayey soils with multiple regression analysis: Van (Turkey) reverse fault area

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Abstract

Many in situ and laboratory tests are being performed to determine the engineering properties of soils. Several relationships can be established between in situ tests and laboratory tests to ensure that both achieve similar results. In this study, in situ standard penetration test and Menard pressuremeter tests were performed on the clayey samples that are in high and low plasticity soil class taken from 6 boreholes reaching to the hanging walls and footwalls of the thrust fault. Disturbed and undisturbed samples were collected in the field, and their physical and mechanical properties were determined in the laboratory. Corrected SPT (SPT-N_{60}), Menard deformation modulus (E_M), and net limit pressure (P_L) values were obtained as part of in situ tests performed. These values were then compared with physical properties like the liquid limit, plasticity index, natural moisture content (w), and mechanical properties like the pre-consolidation pressure (σ_{pc}) and cohesion (c) that were determined through laboratory tests, and linear and non-linear multiple regression analyses were performed on them. The analyses revealed multiple regression equations between dependent variable E_M and independent variables SPT-N_{60}, w, c, and σ_{pc} were obtained with a high degree of determination coefficient. The results also indicate that these multiple regression equations obtained thusly so provided more accurate results compared to simple regression correlations.

Keywords Multiple regression analysis · Pressuremeter · Standard penetration test · Clay soil

Introduction

Various methods and approaches are being performed when trying to determine the bearing capacity and settlement properties of soils where structures will be placed upon. The most widely used methods are the Menard pressuremeter test (MPT), standard penetration test (SPT), cone penetration test (CPT), and the plate loading test. Besides these, laboratory tests are also used for the same purpose. Various factors like potential disturbances in the sample specimens and samples not reflecting the properties of the soil accurately often influence the accuracy of the parameters used in calculations. In situ tests have the significant advantage of providing more reliable and realistic results as the soil is not being disturbed as such (ASTM 1994, ASTM D4318–00 2000, ASTM D1586/D1586M-18 2018). Besides, it is possible in in situ testing to obtain samples from any desired depth among the vertical soil profile. Many statistical relationships between in situ and laboratory tests have been established in the literature for cohesive and non-cohesive soils. The correlations between SPT and cohesion, internal friction angle, and MPT values are frequently in literature. However, no correlation was found in the literature between MPT data and consolidation data, and the studies investigating the relationships between in situ and laboratory findings regarding overconsolidated soils are few in numbers.

A limited number of researchers have performed research on the relationship between SPT and MPT values (Gonin et al. 1992; Yagiz et al. 2008; Bozbey and Togrol 2010;

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Kayabaşı 2012; Kayabaşı and Gökceoğlu 2012; Aladağ et al. 2013; Ağan 2014; Cheshomi and Ghodrati 2015; Anwar 2016; Özvan et al. 2018, 2019). Various empirical correlations have been obtained in the literature between SPT and MPT on sandy and clay soils (Table 1). The study by Chiang and Ho (1980) was performed in Hong Kong in 1980 and evaluated the linear relationship between the SPT-N and E_{PMT} and P_L values of weathered granite, while the study of Ohya et al. (1982) investigated the correlation between SPT-N and E_{PMT} in clayey soils. Meanwhile, Yagiz et al. (2008) investigated the relationship between the corrected SPT blow count (N_{cor}) and E_{PMT} and P_L and revealed that a linear relationship existed between the corrected N_{cor} and E_{PMT} and P_L values for silty sand with clay. Bozbey and Togrol (2010) performed a study and investigated the relationship between SPT-N_{60}, E_{PMT}, and P_L values with a total of 182 tests performed on sandy and clayey soil samples, and have obtained empirical equations with high regression coefficient (R^2) for each soil type, separately. Gonin et al. (1992) have correlated the SPT results for a total of nine different soil types with E_{PMT} and P_L. In some of these studies, high determination coefficients (R^2) were determined between SPT-N and net limit pressure (P_L) and Menard deformation modulus (E_M) for different soil types. The researchers suggest that the equations obtained as part of the study will yield valid results in case they are applied to similar soil types, and they could be taken into consideration during the initial stages of geological projects (Phoon and Kulhawy 1999; Yagiz et al. 2008; Bozbey and Togrol 2010; Kayabaşı 2012; Ching and Phoon 2012, 2013, 2014; Phoon and Ching 2013; Cheshomi and Ghodrati 2015; Shaban and Cosentino 2016; Özvan et al. 2019; Firuzi et al. 2019; Akkaya et al. 2019; Cheshomi et al. 2020; Cheshomi and Khalili 2021). Özvan et al. (2018) have found high determination coefficient between SPT and MPT results for clayey soils. In this research suggested multiple regression analyses be performed on SPT and MPT laboratory tests as future studies to determine the physical and mechanical properties of clayey soils.

Regression analyses based on single variable are generally available in the literature (Table 1). Due to the different physical and mechanical properties of geological structures, it is usual that multivariate analyses give more accurate results. Therefore, the aim of this study is to more accurately describe the geological structure with multiple regression analyses between in situ and laboratory data.

In the present study, SPT and MPT tests were performed on consolidated clayey units that are well-distinguished from weathered clay and that have high (CH) or low plasticity (CL) properties, and on severely weathered claystone and other lithological units that could be classified as overconsolidated units. SPT-N_{60} value was obtained from the SPT test, while E_M and P_L values were obtained from the MPT test. The results of these tests and the data obtained from a series of physical and mechanical tests performed in the laboratory were evaluated using multiple regression analyses, which were then compared to findings obtained from similar soil types in the past.

Geological properties of the study area

The study area consists of Quaternary (Pleistocene) aged old lake and stream sediments that deposited as a result of water movements of the Lake Van (Fig. 1). With different thicknesses and engineering properties, these sediments are particularly present in the wide fields towards the east of Lake Van. Lake Van Basin is a region where rocks of

Table 1 Empirical relationships between E_M, P_L, and SPT-N in the literature

Soil type	E_{PMT}/P_L	E_{PMT}	R^2	P_L	R^2	Literature
Silty clay	12–21	E_{PMT} (kPa) = 388.67 (Ncor) + 4554	0.91	P_L (kpa) = 29.45 (Ncor) + 219.7	0.97	Yagiz et al. (2008)
Sandy soil	7–15	E_{PMT} (Mpa) = 1.33 $(N_{60})^{0.77}$	0.82	P_L (Mpa) = 0.33 $(N_{60})^{0.51}$	0.74	Bozbey and Togrol (2010)
Clayey soil	7–19	E_{PMT} (Mpa) = 1.61 $(N_{60})^{0.71}$	0.72	P_L (Mpa) = 0.26 $(N_{60})^{0.57}$	0.67	
Sandy soil	-	E_{PMT}/Pa = 9.08 $N^{0.66}$	0.48			Ohya et al. (1982)
Clayey soil	-	E_{PMT}/Pa = 19.3 $N^{0.63}$	0.39			
Clayey soil	-	E_{PMT} (MPa) = 0.2885 $(N_{60})^{1.4}$	0.74	P_L (Mpa) = 0.0425 $(N_{60})^{1.196}$	0.74	Kayabaşı (2012)
Clayey soil	-	E_{PMT} (MPa) = 1.24 $(N_{60})^{0.94} - 11.04\ln(w) + 37.9$	0.72	P_L (MPa) = $2.7\ln PI + 0.00001 (N_{60})^{3.408} + 52.39w^{-0.011} - 58.76$	0.77	
Clayey soil	-	E_{PMT} (MPa) = 0.68PI + 0.014 $(N_{60})^{2.067} - 10.44\ln(w) + 23.82$		P_L (MPa) = 0.03 $(N_{60})^{1.26} - 108.4w - 1.69$		
Silty-sand soil	-	E_{PMT}/Pa = $9.8N_{60} - 94.3$	0.79	P_L/Pa = $N_{60} - 20.8$		Cheshomi and Ghodrati (2015)
Silty-clay soil	-	E_{PMT}/Pa = $10N_{60} - 26.7$	0.85	P_L/Pa = $0.5N_{60} + 42$		
Clayey soil	-	E_{PMT} (MPa) = $2.611N_{60} - 26.03$	0.91	P_L (MPa) = $0.142N_{60} - 1.166$	0.89	Özvan et al. (2018)

Pa atmospheric pressure

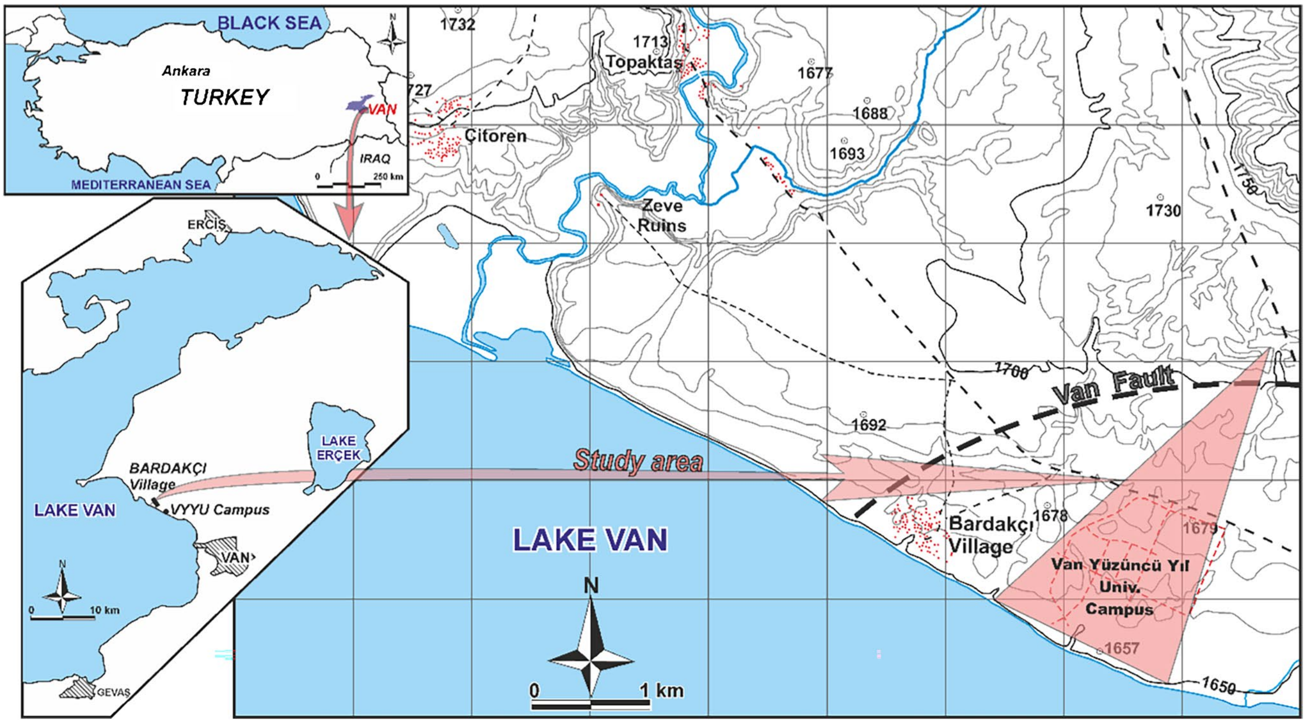

Fig. 1 Location map of the study area

different ages starting with the Paleozoic aged outcrop to the surface and the region has a complex stratigraphy, in particular due to the influence of tectonic activities in the area (Özvan et al. 2005; Akkaya et al. 2015, 2017, 2018; Akkaya and Özvan 2019). The total thickness of old lake sediments is approximately 150 m in the area (Acarlar et al. 1991). According to the previous studies, old and fresh stream sediments were encountered in the study area in addition to the old lake sediments (Acarlar et al. 1991; Selçuk 2003; Koçyiğit 2013). These units are intersected in the north of the study area by the Van thrust fault which ruptured in the destructive earthquake on October 23, 2011 ($M_w = 7.1$), and the units extend from northwest of the study area to the Lake Van (Akkaya et al. 2015, 2017, 2018; Akkaya and Özvan 2019; Sengul et al. 2019). The Van Fault that intersects these units is a thrust type fault inclining towards the north (Fig. 2).

Testing program or experimental testing methods

In addition to the previous data obtained from the study area, 6 additional boreholes were drilled on the hanging wall and footwall of the fault to investigate the influence range of the thrust fault (Fig. 2), and the clayey soils in the area were investigated using both the in situ and laboratory test data.

In situ tests

SPT and MPT represent the most commonly used in situ tests. SPT aims to measure the penetration resistance of the soil and was developed initially in the USA towards the end of the 1920s. Since the test setup is fairly simple and the testing takes a relatively short time, SPT is a widely preferred in situ testing method. In the SPT method performed as part of this study, the test tube was driven into the soil using an automatic pile driver, and the SPT-N blow count was obtained, with which SPT-N_{60} values were calculated (Bowles 1997; Aggour and Radding 2001; British Standards Institution 2007). The SPT was performed in line with the ASTM D1586/D1586M-18 (2018) standards. During the SPT, the blow counts are highly sensitive to the length of rods, hammer energy, sampler type, borehole diameter, and overburden stress (Idriss and Boulanger 2008, 2010). Thus, a corrected penetration resistance is obtained using raw SPT data and a number of correction factors as shown in the following equation:

$$(NI)_{60} = C_N C_E C_R C_B C_S N_m$$

where C_N, C_E, C_R, C_B, and C_S are the correction parameters, whereas N_m is the SPT blow count obtained in situ (Idriss and Boulanger 2008, 2010).

MPT is a test performed using this device and is often performed in areas where the soil is too weak and weathered

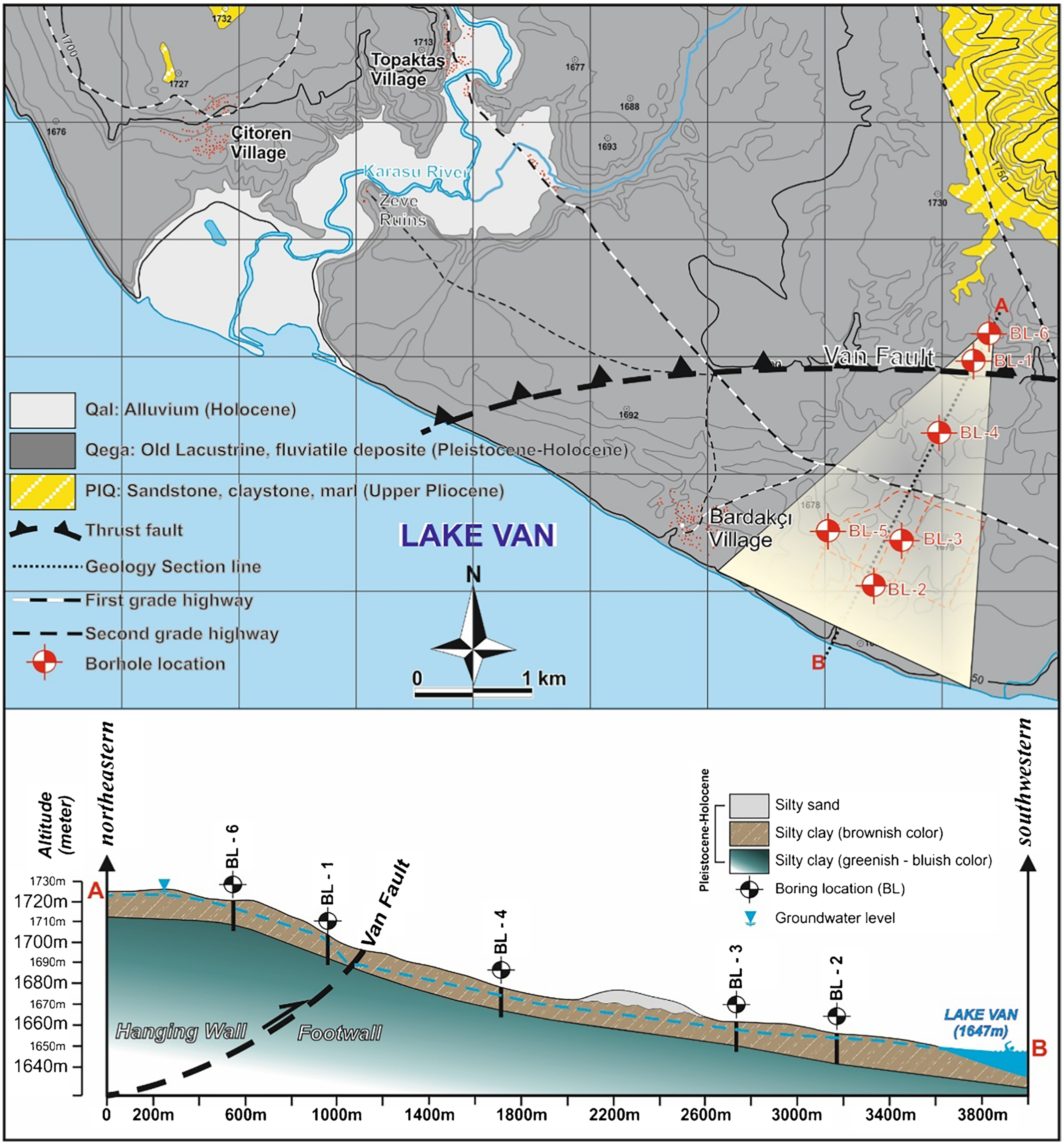

Fig. 2 Geological map of the study area and geological cross section of NE-SW line

soils to obtain proper test specimens for laboratory tests. Furthermore, a self-boring pressuremeter device was also developed to reduce the drilling disturbance in loose soils. The MPT equipment consists of four main parts as the reading unit, probe, pressure air tube, and the pipe section (Fig. 3). The probe through the borehole is either 76 mm or 89 mm in diameter and is made up of three parts consisting of the main body, compressed air cell, and compressed water compartment. The probe diameter is 74 mm. The measuring cell volume (V_c) was taken as 790 cm^3. When the probe reaches the test level within the well, it is inflated using compressed air, and pressure is applied to the well every 60 s in an attempt to deform the soil. If the applied pressure fails the soil, the well walls start to deform and additional water

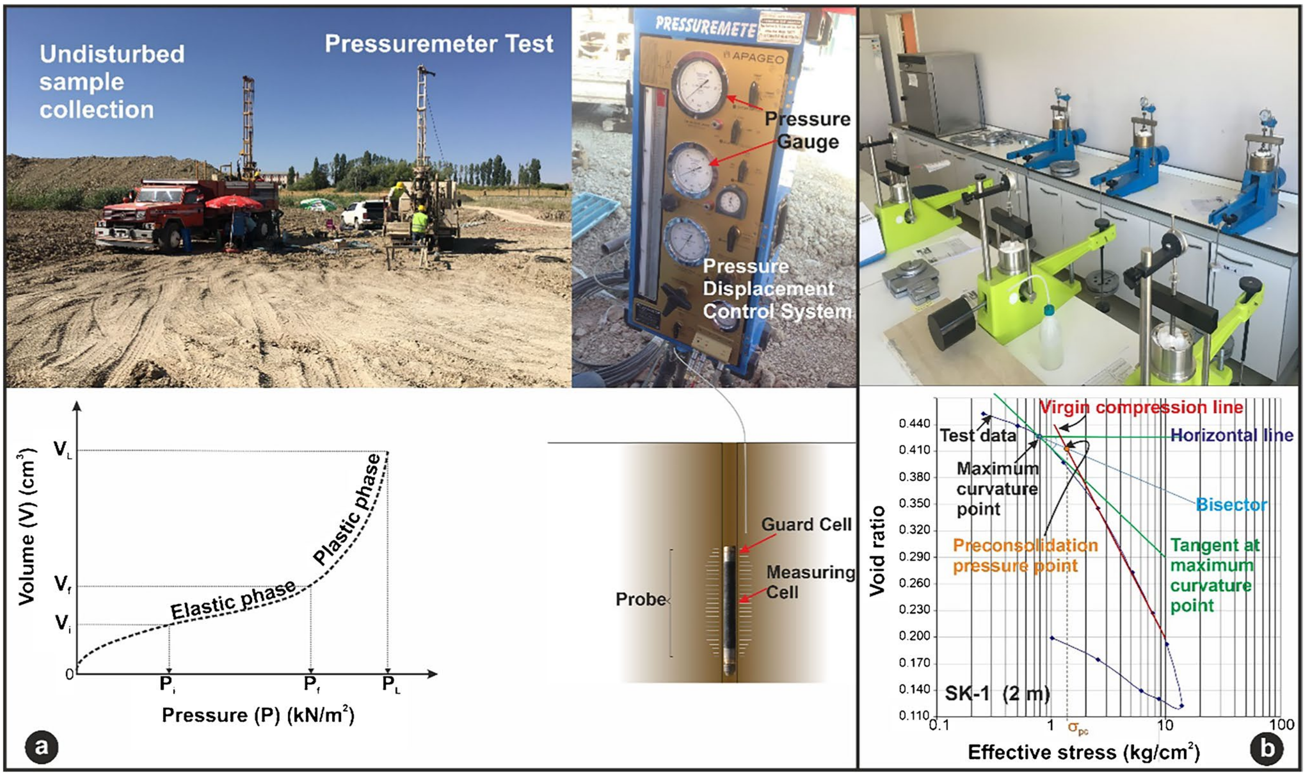

Fig. 3 **a** Simultaneous SPT and MPT measurements from two adjacent boreholes at the same depth: undisturbed sample collection (*left panel*), MPT measurement equipment (*middle panel*), and theorical pressure–volume curve (*bottom panel*). **b** Consolidation measurements: test equipment (*top panel*), and consolidation curve at 2 m in SK-1 borehole (*bottom panel*)

is sent to the compressed air compartment. The amount of water sent to the water chamber is recorded every 15, 30, and 60 s. Here, the pressure level applied corresponds to the soil deformation pressure, and the amount of water sent in corresponds to the amount of deformation under that particular pressure level.

As a result of this test, a pressuremeter curve can be plotted which shows the pressure and volume change, and it is possible to calculate net limit pressure (P_L) and Menard deformation modulus (E_M) values for each depth level tested (Menard 1957; Shields and Bauer 1975; Baguelin et al. 1978; Mair and Wood 1978; Clarke 1995; ASTM 1994). P_L represents the difference between the lift-off pressure and limit pressure. P_L is widely utilized to define soil strength for use in design and analysis procedures (Shaban and Cosentino 2016). E_M, on the other hand, is calculated from the pseudo-elastic slope of the corrected pressure–volume curve. These tests were performed as per the standards outlined in ASTM D4719-87 (1994) and AFNOR NF 94–110-1 (2006).

The data from the in situ tests were obtained from a total of 6 boreholes with an approximately 15-m spacing between them. MPT was performed every 1.5 m in the first well, and concurrent SPT measurements were performed in a second well that was approximately 5 m away from the first at the same depths. Furthermore, disturbed and undisturbed (UD) soil samples were collected from the boreholes when possible. Specimens were coated with paraffin to prevent exposure to air, which were then further covered with stretch film. The physical (water content, specific weight, unit volume weight, grain size, and consistency limit tests) and mechanical properties (consolidation and triaxial pressure tests) were determined in the laboratory.

Laboratory tests

The behavior of soils under different water content levels is called "consistency" and it is of extreme importance when trying to determine the physical properties of fine-grained units. Soil consistency is the strength with which soil materials are held together or the resistance of soils to deformation and rupture. Soil consistency is measured for wet, moist, and dry soil samples. The liquid limit (LL), plastic limit (PL), and shrinkage limit values are collectively known as Atterberg limits and are determined based on the water content of the soils. Atterberg limit tests were performed on the disturbed specimens collected from the 6 boreholes at different locations of the study area, adhering to the standards set forth by ASTM D-4318 (2000).

Similar to Cetin (1997, 2000), consolidation tests with ASTM D-2435–2009 standard were performed to determine the consolidation characteristics (e.g., pre-consolidation pressure) of the fine-grained units in the study area including the Van thrust fault. In this test, the constant weight increase period was set to 24 h and its multiples. Each stress increase was sustained until the excessive water pressure in the pores was completely depleted. The minimum diameter was 50 mm and the minimum height was 20 mm for the samples used in this study. The deformation changes in height were measured using a comparator with 0.01 mm sensitivity. In cases where the test was applied to a fully saturated sample or a sample from beneath a groundwater table, water was introduced to the consolidation compartment after the settlement load. In cases where the sample was not covered by water shortly after applying the settlement load, the consolidation device was covered with moist cotton to prevent evaporation, so that the sample volume could be preserved. The sample was then subjected to constant stress increases. To achieve a compression curve with a distinct break in the slope, and in turn, to obtain the pre-consolidation pressures, the final loading pressure was selected as four times the expected pre-consolidation pressure. Loadings were usually initiated so that at least 2.5 kPa stress could be created on the samples. To minimize the heave after the test, the samples were returned to their settlement loads (2.5 kPa) during removal.

Various researchers have developed different methods to determine the pre-consolidation pressure (σ_{pc}) (Casagrande 1936). The most commonly used is the method suggested by Casagrande (1936) and was used in this study to determine σ_{pc} as well. With this method, the maximum effective stress value (σ') that influences a given soil and gives its final structure and fabric is defined as the pre-consolidation pressure. The pre-consolidation pressure (σ_{pc}) was determined using the void ratio (e) – log effective stress (σ') curve and the Casagrande method.

A triaxial pressure test (UU) was also performed on the UD samples under laboratory conditions. This was done by drawing the Mohr circles corresponding to the primary tensions (σ_1, σ_3) of the moment of fracture due to low load impact, the c and ϕ values for the Coulomb's shear equation. In this study, UU (undrained-unconsolidated) triaxial test was performed using the ASTM D(2850)–15 2015 standards.

The index properties of fine-grained soils

Disturbed and undisturbed soil samples were collected from the boreholes as part of the study, and the physical properties of the samples were determined (Table 2). When the grain size ratio of the samples was inspected, it was revealed that the ratio of fine-grained silt and clay amount to total grain size was higher than 80%. When the natural water moisture content of the samples was investigated, it was found out that the fine-grained soil samples usually were not fully saturated with water. Inspection of the water content of these samples has shown that the highest water content was 32%, while the lowest was 11.6%. A great majority of the inspected samples contained 20–24% water, while the average water content among all samples was determined as 21.9% (Table 2).

When the specific gravity and densities of the samples were evaluated, it was found that the highest specific gravity was 2.87 while the lowest specific gravity was 2.60, and the highest density was 2.14 g/cm^3 while the lowest density was 1.82 g/cm^3 (Table 2). Inspection of the consistency curve has shown that the highest liquid limit for these units was 88%, while the lowest was 25%, and the highest plasticity limit was 32%, and the lowest was 15%. When these values are placed in the plasticity chart, the inspected clayey levels were classified either low (CL) or high (CH) clayey soils.

In the in situ tests, N_{30} values of the SPT blow counts were refusal, especially at regions closer to the fault (> 50 blow/30 cm) (Table 2). Differing from previous studies, the present study attempted to continue the penetration after 50 blows in the SPT measurement, so that SPT data could be compared to MPT data (ASTM D1586/D1586M-18, 2018).

In the study area, MPT tests were performed every 1.5 m to make the measurements coincide with that of the SPT tests. This in situ test can be influenced by various factors like in open borehole wall collapses, or groundwater presence. Due to situations like these, when evaluating values obtained as a result of tests, Menard pressuremeter (elastic) modulus (E_M) and net limit pressure (P_L) could not be calculated for some depth levels. The E_M and P_L values were calculated for a total of 33 different depth levels in the study area (Table 2). The calculations show that E_M values change between 58.7 and 658.9 kg/cm^2, while P_L values change between 8.7 and 67.1 kg/cm^2. When these results are compared with values provided for typical E_M and P_L value ranges, it becomes apparent that the soil is very solid—hard clay. When these values are compared to the physical properties of the inspected soils and SPT-N_{60} values, the results were found to be compatible.

The evaluation of the average E_M values obtained as part of this study has shown that the highest E_M values were recorded in areas closer to the Van Fault, while the lowest E_M were recorded in the well that was nearest to the lake (southwest) (BL-2) (Table 2). Similar to the SPT test, it was found out that the higher the depth, the higher the E_M value.

Simple regression analysis

Regression analysis explains a functional relationship between two variables. In such a relationship, if the independent variable is X with e_i representing an additive error

Table 2 Physical properties and soil classes of the investigated clayey levels

SK	Depth (m)	SPT		MPT (kg/cm^2)		Atterberg			Natural moisture content (%)	Specific weight	Density (gr/cm^3)	c (kg/cm^2)	σ_{pc} (kN/m^2)	Soil class
		Raw	N_{60}	E_M	P_L	LL	PL	PI						
1	1.00	38	21	193.1	8.7	66	27	39	16.4	2.70	2.11	0.19	125.6	CH
1	2.00	73	44	315.8	32.5	66	27	39	24.4	2.81	2.03	0.33	127.5	CH
1	2.75	100	64			45	20	24	16.0	2.79				CL
1	4.50	100	71	354.3	25.4	44	20	24	20.2	2.80	2.13	0.26	147.2	CL
1	5.50	105	75	414.9	38.4	44	18	26	12.7	2.76	2.10	0.32	138.3	CL
1	6.50	108	77	387.7	38.3				23.5	2.79	1.98	0.35	137.3	CL
1	7.25	102	73			88	28	60	22.3	2.87				CH
1	8.75	91	68	390.6	31.3	56	26	30	16.3	2.78	2.03	0.37	196.2	CH
1	9.75	102	77	342.8	21.2	60	25	35	21.2	2.84	2.03	0.40	119.7	CH
1	10.50	235	176			36	17	19	12.4	2.72				CL
1	11.25	220	165			25	19	6	20.7	2.79				CL-ML
1	12.50	190	143			46	19	28	20.0	2.70				CL
2	1.00	43	24	149.5	15.8	46	18	29	17.1	2.60	1.93	0.10	147.2	CL
2	3.50	31	20	126.4	16.6	58	20	39	31.2	2.78	1.97	0.07	167.8	CH
2	4.50	47	33	259.0	35.3	72	22	49	29.3	2.78	1.82	0.12	177.6	CH
2	5.50	31	22			72	22	49	30.7					CH
2	6.50	30	21	231.0	26.8	63	22	41	31.1	2.78	1.92	0.10	186.4	CH
2	9.00	42	32			54	21	33	30.2					CH
2	14.00	46	35			76	22	54	28.9					CH
3	1.00	30	17	202.0	21.4	76	26	50	23.3	2.77	1.85	0.13	135.4	CH
3	2.50	49	31	231.8	20.5	80	28	52	23.9	2.74	1.92	0.10	137.3	CH
3	4.25	80	51	328.5	42.2	52	21	31	21.6	2.76	2.07	0.17	206.0	CH
3	5.50	97	69	345.6	31.9	50	20	30	22.3	2.76	2.01	0.24	233.5	CH
3	6.75	117	83	423.6	38.5	51	23	28	22.8	2.73	2.02	0.24	197.2	CL-CH
3	7.25	42	30			51	23	28	21.4	2.76	2.09	0.26	196.2	CL-CH
3	8.25	82	58	397.3	52.6	51	23	28	27.5	2.76	2.00	0.22	206.0	CL-CH
3	9.00	37	28			81	29	52	25.6	2.76	2.03		215.8	CH
3	9.75	44	33			64	32	32	27.6	2.76	1.97	0.17	226.6	CH
3	10.75	127	95	410.8	36.8	65	20	44	32.0	2.76	1.97			CH
3	12.25	189	142	499.7	38.9	49	19	30	21.1	2.74	2.10	0.32	230.5	CH
3	13.75	20	11			41	18	23	24.0	2.76	2.02	0.42	279.6	CL
4	2.50	54	34			65	23	42	22.0					CH
4	3.50	65	47	287.8	31.0	65	23	42	17.4	2.76	2.14	0.32	225.6	CH
4	4.50	103	73			62	22	39	22.2					CH
4	5.75	99	71	406.6	34.5	62	22	40	20.9	2.75	2.10	0.40	230.5	CH

Table 2 (continued)

SK	Depth (m)	SPT		MPT (kg/cm^2)		Atterberg			Natural moisture content (%)	Specific weight	Density (gr/cm^3)	c (kg/cm^2)	σ_{pc} (kN/m^2)	Soil class
		Raw	N_{60}	E_M	P_L	LL	PL	PI						
4	7.00	155	110	479.1	40.7	56	21	34	20.9	2.76	2.09	0.43	269.8	CH
4	8.25	179	128	590.7	31.4	74	26	48	19.8	2.76	2.08		294.3	CH
4	9.50	224	168	658.9	38.6	49	18	31	19.4	2.74	2.08		312.0	CL
4	11.75	165	124			36	16	20	12.1					CL
5	2.50	27	17	132.7	16.7	26	15	11	19.0	2.70				CL
5	5.50	154	110			34	20	14	15.9	2.76				CL
5	6.25	100	71	514.9	55.3	34	18	16	11.6	2.70	2.06		124.6	CL
5	7.25	180	128	620.0	51.5	34	21	13	18.0	2.74				CL
6	2.25	29	17	58.7	9.9	48	18	30	26.1	2.69	2.06	0.08	88.3	CL
6	3.25	39	25	94.3	10.7	48	18	30			1.99	0.09	112.8	CL
6	5.50	50	36	96.7	10.2	46	20	26	23.6	2.76	2.01	0.10	145.2	CL
6	6.50	78	56	173.7	23.6	53	24	29	27.2	2.73	1.96	0.12	196.2	CH
6	8.00	164	117			54	23	31	18.7					CH
6	9.00	189	142	493.0	67.1	45	20	25	22.6	2.75	2.04		233.5	CL
6	10.25	197	148	552.1	54.2	61	21	39	20.6	2.77	2.09		235.4	CH
min		20	11	58.7	8.7	25	15	6	11.6	2.6	1.82	0.07	88.3	
max		235	176	658.9	67.1	88	32	60	32	2.87	2.14	0.43	312	
Average		97.98	70.2	338.3	31.77	54.7	21.7	32.9	21.9	2.75	2.02	0.22	188.3	

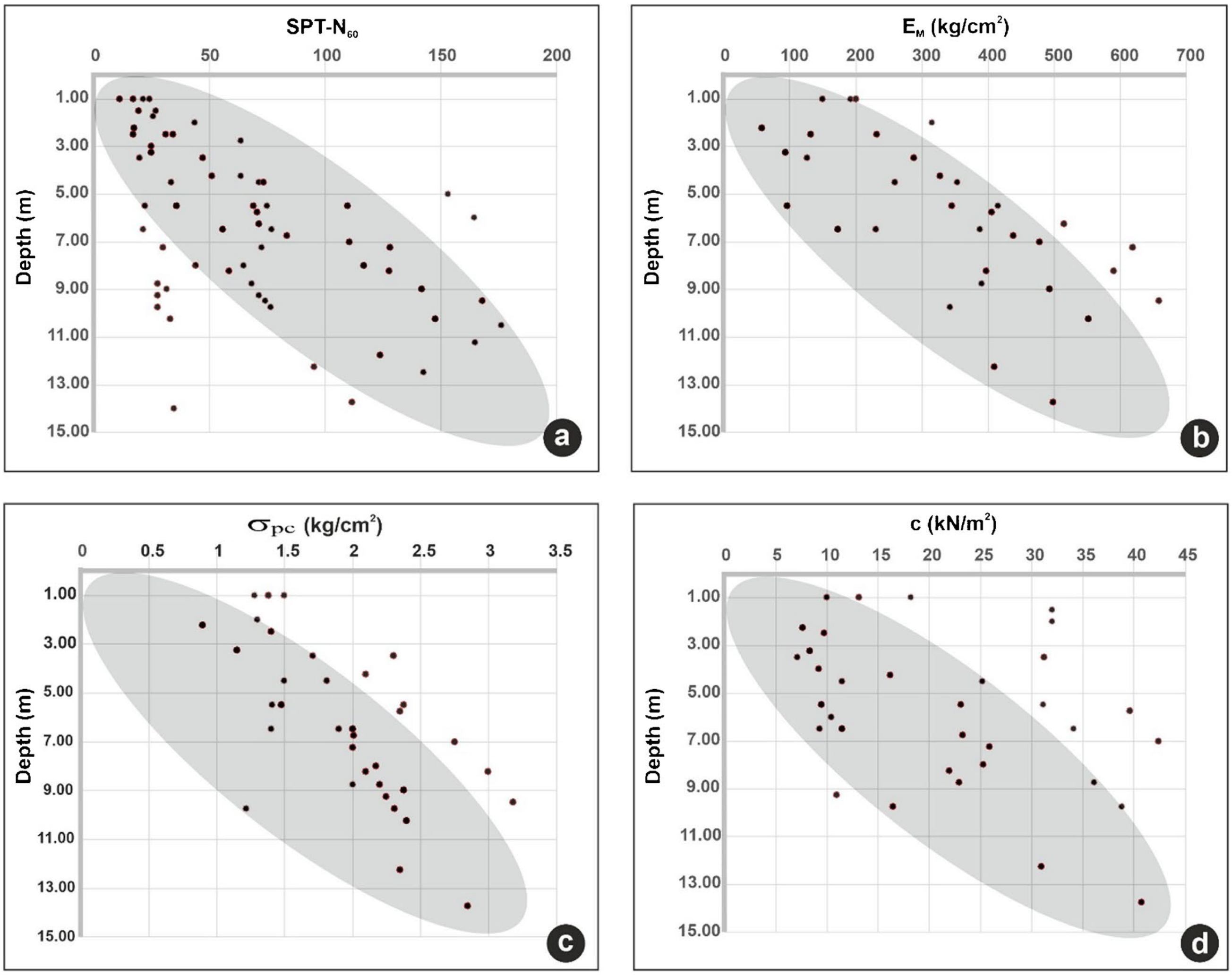

Fig. 4 Depth-dependent changes of data; SPT-N_{60} (**a**), E_M (**b**), σ_{pc} (**c**), and c (**d**)

term and the dependent variable is Y, the functional relationship between the two variables can be written as:

$$Y_i = f(X_i, \beta) + e_i \tag{1}$$

$$f(X_i, \beta) = \beta_0 + \beta_1 X_i \tag{2}$$

The aim of the analysis is to estimate β parameters with different regression analysis types, such as least squares method. The estimations of the parameters with the least squares method and the correlation coefficient formulas are as follows:

$$\beta_1 = \frac{\sum(x_i - \overline{x})(y_i - \overline{y})}{\sum(x_i - x)^2} \tag{3}$$

$$\beta_0 = \overline{y} - \beta_1\overline{x} \tag{4}$$

$$R = \frac{\sum xy - \left(\sum x\right)\left(\sum y\right)/n}{\sqrt{\left[\sum x^2 - \left(\sum x\right)^2/n\right]\left[\sum y^2 - \left(\sum y\right)^2/n\right]}} \tag{5}$$

Statistical evaluations were performed to compare the results of in situ and laboratory tests performed on the fine-grained units of the study area and to reveal any potential correlations between them. It is determined that the physical and mechanical properties of the examined specimens depict dissimilarities in the laboratory and in situ tests. These differences are also recognized in the relationships between these parameters. To start the analysis, the single-variable linear regression of the in situ pressuremeter readings and the parameters obtained through laboratory measurements was performed first.

Firstly, regression analyses were performed to obtain empirical relations between the E_M and the σ_{pc}. The results

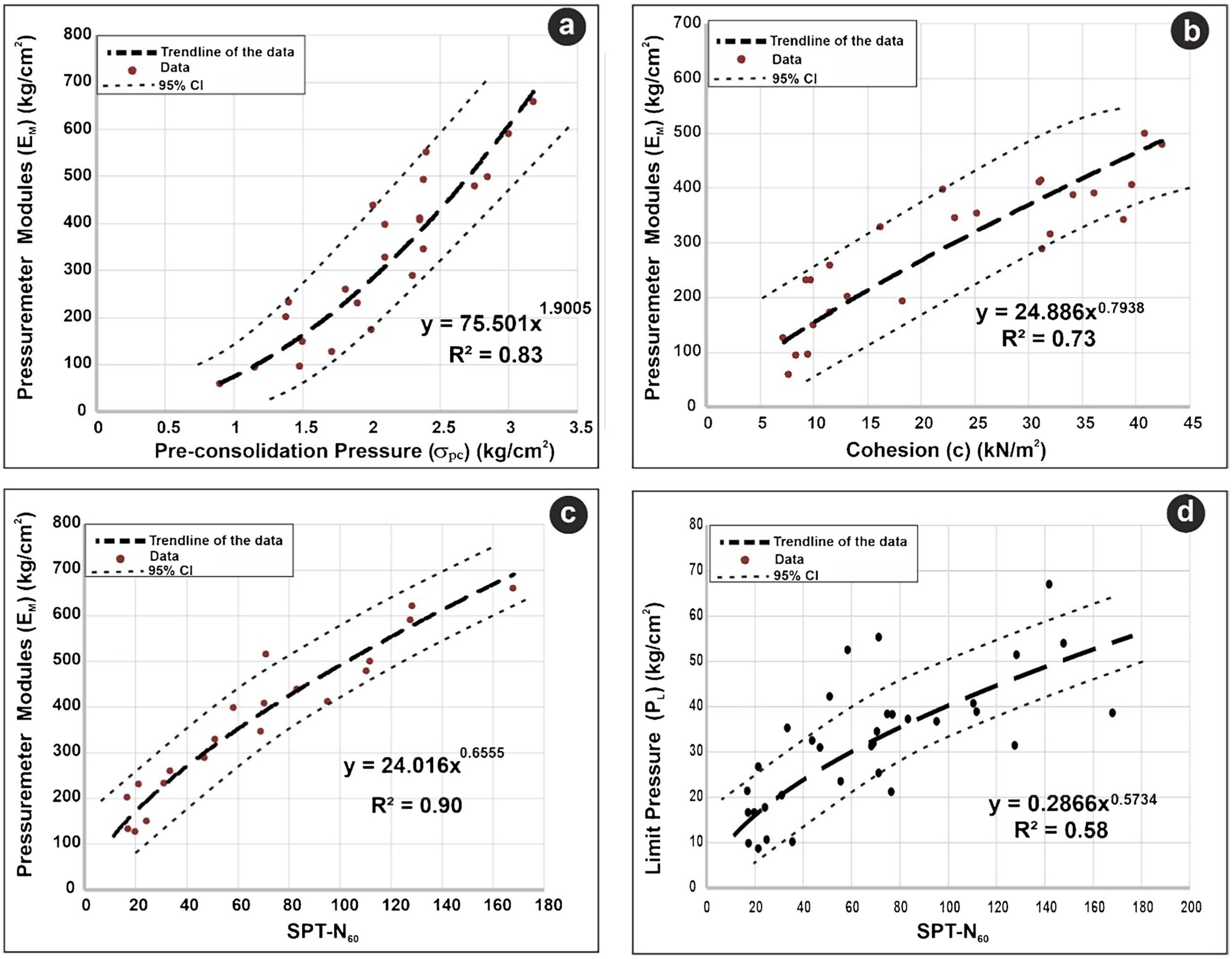

Fig. 5 Relationships between in situ and laboratory data; E_M and σ_{pc} (**a**), E_M and c (**b**), E_M and SPT-N$_{60}$ (**c**), and P_L and SPT-N$_{60}$ (**d**)

of the regression analysis are shown in Table 5 and Fig. 5a. The equation with the highest coefficient ($R^2 = 0.83$) of the regression between E_M and σ_{pc} is represented by a power function (Eq. 9). In this equation, E_M and σ_{pc} values are in kg/cm^2. Evaluation of the changes in σ_{pc} and E_M values with depth has shown that as depth increased, both of these values increased as well (Fig. 4b–c).

The other regression analyses were then performed to obtain empirical relations between the E_M and the cohesion (c) derived from triaxial compressive strength (Fig. 5b). The equation with the highest coefficient ($R^2 = 0.73$) of the regression between E_M and c is represented by a power function (Eq. 10). Cohesion (c) values are influenced by factors like the grain size and water content of the soil, making it challenging to obtain correlations between in situ and laboratory data. Evaluation of the c value with depth has revealed that in general, as the depth increased, the c value increased as well (Fig. 4d).

Evaluation of the relationship between the P_L and SPT-N$_{60}$ values has shown only a low determination coefficient between the parameters ($R^2 = 0.58$) (Fig. 5d). In general, the P_L value was found to increase as the depth increased. Similarly, the SPT-N$_{60}$ value increased as the depth increased in most cases (Fig. 4a).

When the correlation between E_M and SPT-N$_{60}$ values was investigated, a high determination coefficient was determined ($R^2 = 0.90$) (Fig. 5c). Evaluating the relationship between depth and E_M and SPT-N$_{60}$ values has shown that, in general, these values increased as the depth increased (Fig. 4a–b). We considered that the data from the point closer to the hanging-wall side of the Van Fault (BL-1 and BL-6) affect the correlation between all the data, due to the presence of deformation structures in the soil caused by the fault. When the data from these boreholes are ignored, the regression coefficient increases from $R^2 = 0.75$ to $R^2 = 0.90$ (Eq. 11).

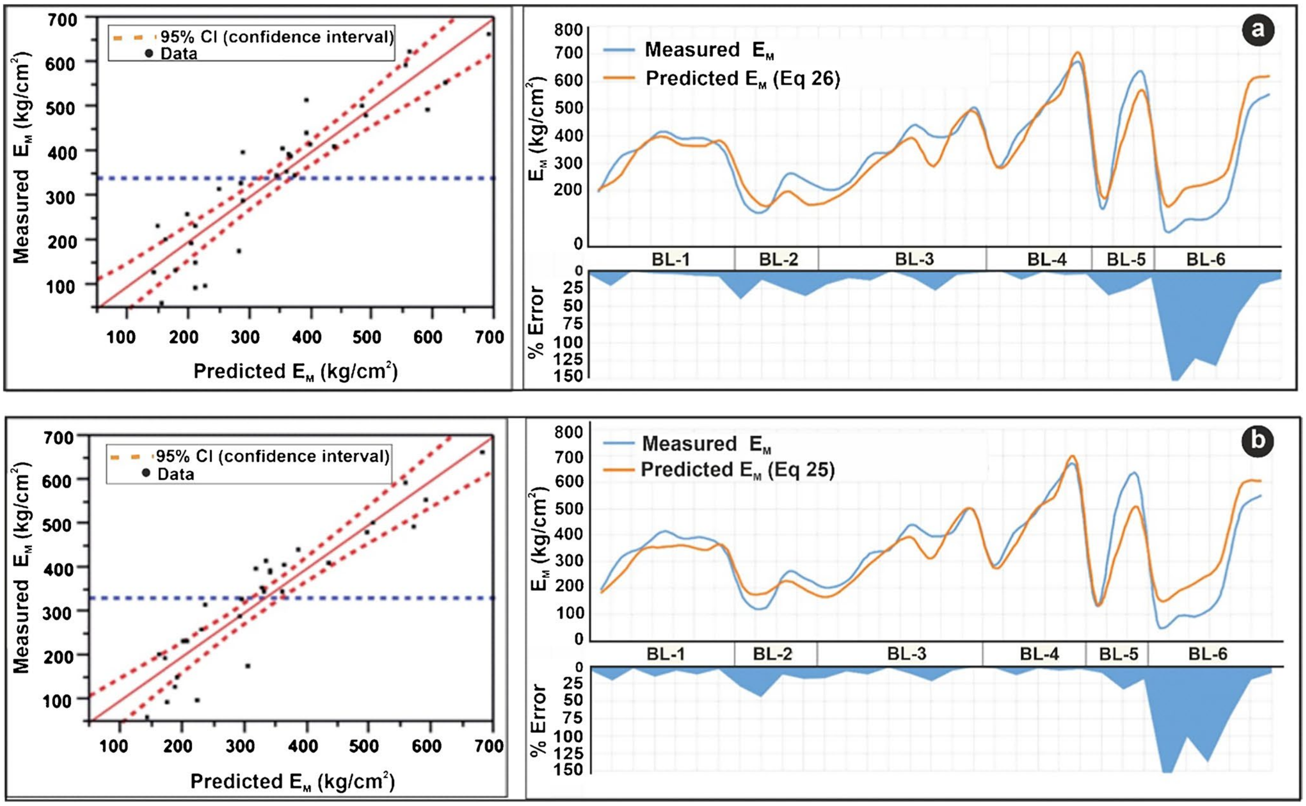

Fig. 6 Correlation between the dependent variable E_M and the E_M value calculated using the independent variables SPT-N_{60} and w (Eq. 26) and the distribution of error margins (**a**), the independent variables SPT-N_{60} and σ_{pc} (Eq. 25), and the distribution of error margins (**b**)

Adaptation between the measured and E_M values computed exponentially through Eq. 11 was determined, except for data from BL-6. Similarly, the margins of error between measured and calculated values were also found to be low, once again with the exception of BL-6. The fact that deformations related to the fault nearby the BL-6 point are high is causing the margins of error in these data to rise beyond thresholds.

When all data groups are evaluated overall, it was found that the data nearby the hanging-wall side of the fault (BL-1 and BL-6) show increased variation due to deformation structures in the soil, which influence the regression results. In almost every variable inspected, ignoring these data resulted in higher harmony and determination coefficients. This is indicative of the significance and importance of data set selection, particularly in thrust fault deformation areas.

Multiple regression analysis

The relationship between one dependent variable and more than one independent variable can be examined in the regression model. The multiple linear regression model has the form:

$$Y_i = \beta_0 + \sum_{j=1}^{n} \beta_j X_{ij} + e_i \tag{6}$$

Y_i is the real-valued response for the ith observation, β_0 is the regression intercept, β_j is the jth predictor's regression slope, X_{ij} is the jth predictor for the ith observation, and e_i is an error term.

The statistical analysis indicates that the non-linear multiple regression approach is more suitable than the linear regression analysis. In the multiple regression steps of the statistical studies, the relationships between the E_M and P_L with the SPT-N_{60}, σ_{pc}, c, w, PI, and PL values were evaluated together. Generally, the equations with the high determination coefficients were obtained from multiple regression analysis.

The first step is to define the independent variables of the SPT-N_{60}, σ_{pc}, c, PI, PL, and the w value as the function of E_M dependent variable:

$$\left.\begin{aligned} E_M &= f\left(\mathrm{SPT}-\mathrm{N}_{60}, w\right) \\ E_M &= f\left(\mathrm{SPT}-\mathrm{N}_{60}, \sigma_{pc}\right) \\ E_M &= f\left(\mathrm{SPT}-\mathrm{N}_{60}, \sigma_{pc}, w\right) \\ E_M &= f\left(\mathrm{SPT}-\mathrm{N}_{60}, PI, w\right) \end{aligned}\right] \tag{7}$$

Table 3 Statistical values of linear multiple regression analysis including E_M dependent variable and SPT-N_{60} and w independent variables (Eq. 26)

Regression statistics									
Multiple R	R^2	Adjusted R^2	Standard Error RMSE	Average	Observation				
0.919	0.845	0.835	6.605	33.87	33				
ANOVA test results									
	df	*Sum of squares*	*Square mean*	*F ratio*	*Significance F*				
Regression	2	7147.069	3573.535	81.920	0.000				
Difference	30	1308.673	43.622						
Total	32	8455.742							
	Estimation	Standard error	*t ratio*	*P value*	*Low 95%*	*High 95%*	*Low 95%*	*High 95%*
Intercept	18.893	6.523	2.897	0.007	5.572	32.21	5.57	32.21
SPT-N_{60}	0.341	0.029	11.967	0.000	0.283	0.40	0.28	0.40
w	−0.365	0.262	−1.394	0.174	−0.899	0.17	−0.89	0.17

After this definition, the relationships between the different combinations of SPT-N_{60} and w% values were evaluated. A, B, C, and D represent the coefficients of the equations. Multiple regression experiments were carried out by creating linear and non-linear different equation groups. The non-linear multiple regression equations obtained are given as follows;

$$\left.\begin{array}{r} E_M = A + BN_{60} + Cw \\ E_M = A + B\sigma_{pc} + CN_{60} \\ E_M = A + B\sigma_{pc} + Cw \\ E_M = A + Bw + CN_{60} \\ E_M = AN_{60}^{B} + C\ln(w) + D \end{array}\right] \quad (8)$$

When the differences between the measured E_M values and E_M values that were calculated through linear multiple regression analyses that contained these parameters were evaluated, it was found that the values are close to each other at the 95% confidence interval (CI) (Fig. 6a). The margin of error is small in all points except BL-6. We considered that the errors associated with BL-6 data are due to the deformation in soil influencing SPT values. Table 3 presents the results of the statistical analyses that were performed with these parameters.

Investigation of Table 3 reveals that the R^2 value between the E_M variable and the independent variables SPT-N_{60} and w is 0.919. However, adjusted R^2 value should be considered valid for multiple regression analyses. Accordingly, the R^2 value should be taken as 0.845. This means that 84.5% of the change that occurs in the dependent variable (E_M) can be explained by the independent variables (SPT-N_{60} and w). Furthermore, the results of the ANOVA (variance analysis) test have revealed the meaningfulness of the model as a whole through the results of F tests. The significance value here is essential. In case the F test finds a value meaningful, this means that our model is statistically meaningful as a whole. The significance value for this analysis was found as 0.000, meaning it is smaller than 0.05, and even 0.01,

Table 4 The statistical values for the linear multiple regression analysis containing the dependent variable E_M and the independent variables SPT-N_{60} and σ_{pc} (Eq. 25)

Regression statistics								
Multiple R	R^2	Adjusted R^2	Standard error RMSE	Average	Observation			
0.904	0.817	0.804	6.869	31.03	31			
ANOVA test results								
	df	*Sum of squares*	*Square mean*	*F ratio*	*Significance F*			
Regression	2	5917.22	2958.61	62.711	0.000			
Difference	28	1320.98	47.178					
Total	30	7238.21						
	Estimation	Standard error	*t ratio*	*P value*	*Low 95%*	*High 95%*	*Low 95%*	*High 95%*
Intercept	9.032	4.418	2.044	0.005	−0.017	18.081	−0.017	18.081
SPT-N_{60}	0.323	0.045	7.124	0.000	0.230	0.416	0.230	0.416
σ_{pc}	1.614	3.149	0.513	0.312	−4.835	8.064	−4.835	8.064

Table 5 The results of simple and multiple variable regression analyses performed in the study (units for the variables are as follows: E_M: kg/cm^2, P_L: kg/cm^2, σ_{pc}: kg/cm^2, c: kN/m^2, w: %, LL: %, PI: %)

	Equation no	Equation	R^2	p values		
				val. 1	val. 2	val. 3
Single regression	9	$E_M = 75.501^*(\sigma_{pc})^{1.9005}$	0.83	0.000	-	-
	10	$E_M = 24.886^*(c)^{0.7938}$	0.73	0.000	-	-
	11	$E_M = 24.016^*(\text{SPT-N}_{60})^{0.6555}$	0.90	0.000	-	-
	12	$E_M = 14.592^*(\text{SPT-N}_{60})^{0.7513}$	0.75	0.064	-	-
	13	$E_M = 24.016^*(\text{SPT-N}_{60})^{0.6555}$	0.90	0.058	-	-
	14	$E_M = 21.266^*\ln(\text{SPT-N}_{60}) - 50.956$	0.84	0.003	-	-
	15	$E_M = 10.322 + 0.3513^*\ (\text{SPT-N}_{60})$	0.84	0.003	-	-
	16	$P_L = 2.8661^*(\text{SPT-N}_{60})^{0.5734}$	0.58	0.092	-	-
	17	$P_L = 1.55 + 0.0242^*(\text{SPT-N}_{60})$	0.51	0.081	-	-
Multiple regression	18	$E_M = 17.017 + 0.344^*\text{SPT-N}_{60} + 0.112^*PI - 0.458^*\text{w}$	0.85	0.000	0.338	0.111
	19	$E_M = -6.43 + 3.7778^*w + 9.562^*c/98.1$	0.72	0.131	0.000	-
	20	$E_M = 142.883 + 198.218^*\sigma_{pc} - 8.667^*w$	0.66	0.000	0.039	-
	21	$E_M = -50.363 + 207.706^*\sigma_{pc} + 8.375^*\text{P}L - 9.106^*w$	0.69	0.000	0.125	0.028
	55	$E_M = 91.917 + 201.693^*\ \sigma_{pc} + 1.761^*PI - 9.465^*w$	0.67	0.000	0.371	0.029
	23	$E_M = -8.163 + 74.739^*\ \sigma_{pc} + (7.457^*c/98.1)$	0.79	0.009	0.000	-
	24	$E_M = 101.232 + 51.053^*\sigma_{pc} + 2.772^*\ \text{SPT-N}_{60} - 2.394^*w$	0.86	0.102	0.000	0.395
	25	$E_M = 9.032 + 1.614^*\ \sigma_{pc} + 0.323^*\ \text{SPT-N}_{60}$	0.82	0.312	0.000	-
	26	$E_M = 18.893 + 0.3413^*\ \text{SPT-N}_{60} - 0.365^*w$	0.85	0.000	0.174	-
	27	$E_M = 67.31 + 2.547^*\ \text{SPT-N}_{60} + 3.896^*c/98.1$	0.83	0.001	0.027	-
	28	$E_M = 43.925^*(\text{SPT-N}_{60})^{0.519} + 78.104^*\ln(w) - 86.031$	0.87	0.000	0.001	-
	29	$E_M = 148.208 + 3.438^*\ \text{SPT-N}_{60} + 1.147^*LL - 4.758^*w$	0.85	0.000	0.19	0.095
	30	$P_L = 14.789 + 0.245^*\ \text{SPT-N}_{60} - 0.0702^*LL + 0.204^*w$	0.52	0.000	0.631	0.645
	31	$P_L = 1.2297 + 0.0247^*\ \text{SPT-N}_{60} + 0.0136^*w$	0.51	0.000	0.426	-
	32	$P_L = -5.4817 + 0.82^*w + 0.6825^*c/98.1$	0.42	0.070	0.000	-
	33	$P_L = 1.112 + 10.994^*\sigma_{pc} + 0.332^*c/98.1$	0.52	0.007	0.046	-
	34	$P_L = 10.792 + 3.844^*\ \sigma_{pc} + 0.191^*\ \text{SPT-N}_{60}$	0.48	0.006	0.541	-
	35	$P_L = -1.183 + 2.241^*\sigma_{pc} + 0.224^*\ \text{SPT-N}_{60} + 0.578^*w$	0.55	0.064	0.004	0.205
	36	$P_L = 2.183 + 14.136^*\ \sigma_{pc} + 0.071^*w$	0.37	0.000	0.883	-
	37	$P_L = 3.396 + 14.0536^*\ \sigma_{pc} - 0.0418^*PI + 0.0898^*w$	0.38	0.000	0.856	0.858
	38	$P_L = -2.591 + 14.371^*\ \sigma_{pc} + 0.207^*PL + 0.0599^*w$	0.38	0.000	0.756	0.901
	39	$P_L = 14.211 + 0.244^*\ \text{SPT-N}_{60} - 0.114^*PI + 0.231^*w$	0.52	0.000	0.535	0.605
	40	$P_L = 13.459 + 0.243^*\ \text{SPT-N}_{60} + 0.071^*c/98.1$	0.45	0.042	0.794	-

indicating that our regression model as a whole is statistically significant.

When the significance level of the regression model is inspected in Table 3, the intercept coefficient was determined as 18.893, and p value was determined as 0.007. This means that the constant term is also significant. The regression model coefficient for SPT-N$_{60}$ was calculated as 0.341. T test results show that the significance level is 0.000, and since this value is below 0.05, the analysis is meaningful at a significance level of 5%. The regression model coefficient for the natural moisture content (w) was calculated as -0.365, and the p value for it was 0.174. Here, the regression model coefficient is negative and w has an inverse relationship with the variables.

Another multiple regression analysis was performed between the dependent variable E_M and independent variables SPT-N$_{60}$ and σ_{pc} (Fig. 6b, Table 4). The evaluation of the difference between the measured E_M value and the E_M calculated in the linear multiple regression analysis that contains SPT-N$_{60}$ and σ_{pc} independent variables has shown that these values are close to each other (Fig. 6b). The margin of error between measured and calculated values is low in all points except for BL-6. Like the case was in other analyses, we believe

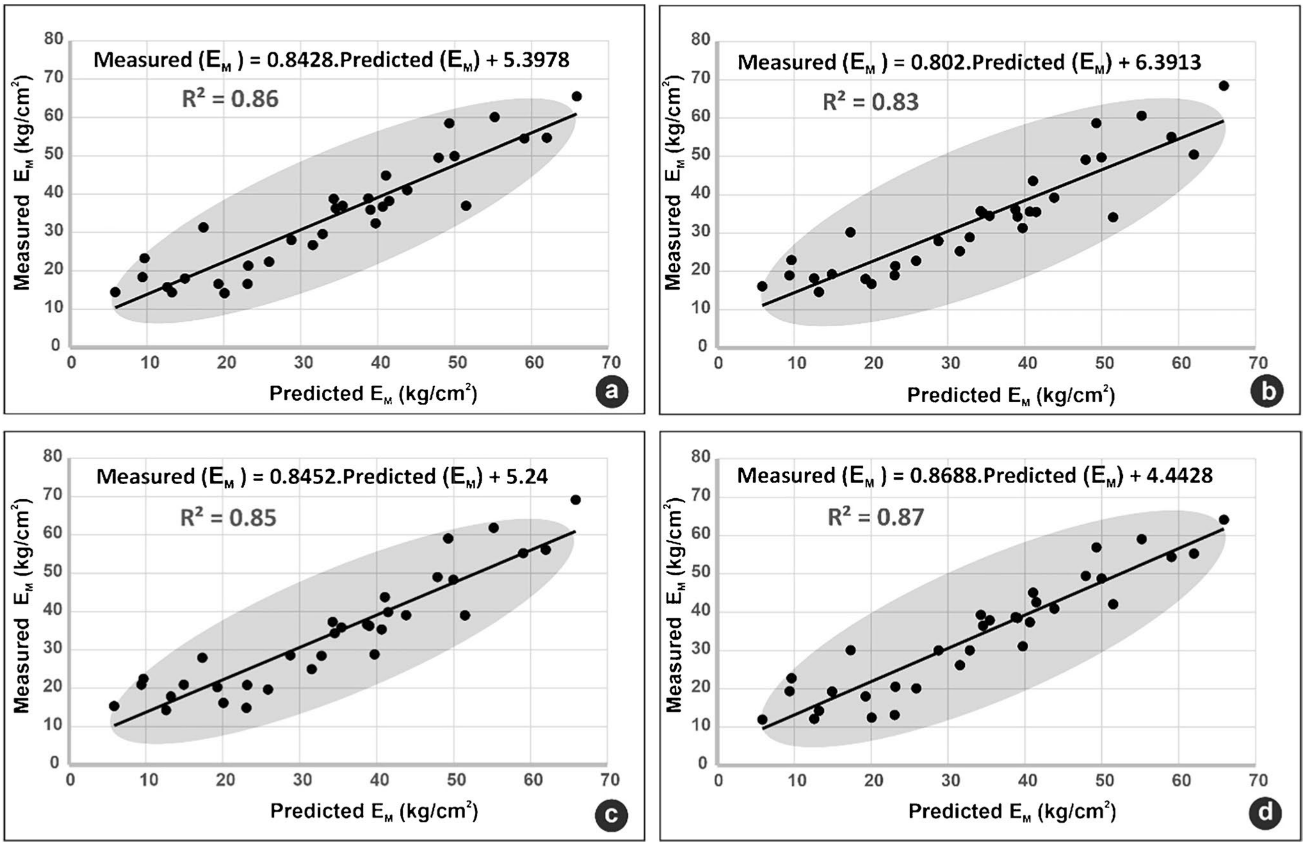

Fig. 7 Graph showing the relation between E_M(measured)–E_M(predicted) with two variables in Eq. 11 (**a**), Eq. 25 (**b**), Eq. 26 (**c**), and Eq. 28 (**d**)

the errors with BL-6 are caused by the deformation constructs in the soil or the stress release beyond the fault area influencing the SPT values.

Investigation of Table 4 has revealed the R^2 value between the dependent E_M value and independent SPT-N_{60} and σ_{pc} variables as 0.904. Yet, the adjusted R^2 value should be considered in multiple regression analyses, and the R^2 value should thereby be considered 0.804. This can be interpreted as 80.4% of the change in the dependent variable (E_M) can be explained by the independent variables (SPT-N_{60} vs. σ_{pc}). ANOVA test also has revealed the F test results which display the statistical meaningfulness of the model as a whole.

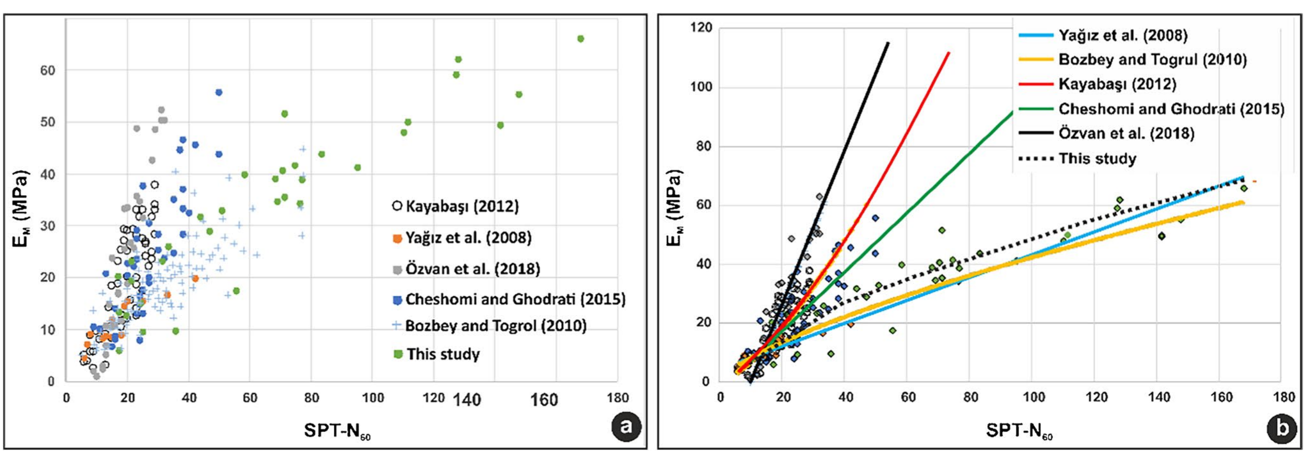

Fig. 8 SPT-N_{60} and E_M data obtained in the literature and in this study (**a**), comparison of the regression model between E_M and SPT-N_{60} developed in this study with those proposed in the literature (**b**)

Here, the significance value is of importance, as in case the *F* test finds this value significant, this would mean our model would be statistically significant as a whole. The significance value for this analysis was found to be 0.000, which is smaller than 0.05, and even 0.01. This is indicative that our regression model is statistically meaningful.

Evaluation of the coefficients of the regression model in Table 4 and the significance value has shown that the coefficient for the intercept term was 9.032, and the *p* value was found as 0.005. This means that the intercept term is also meaningful. The SPT-N_{60} regression coefficient was calculated as 0.323. *T* test results show the significance level as 0.000, and since this value is lower than 0.05, the analysis is meaningful at a 5% significance level. σ_{pc} regression model coefficient was calculated as 1.614, and the *p* value was found as 0.312. The regression coefficient is positive, indicating that the σ_{pc} variable is directly proportional with other variables. In this analysis, when σ_{pc} as the weakest independent variable (with the lowest *p* value) was removed from the model and the regression was calculated again, the model was not suffered any additional weakening, and the independent variable was integrated back into the model.

All simple and multiple regression analysis results obtained as part of this study have been presented in Table 5. As can be seen from their inspection, certain correlations with high determination coefficients were determined between in situ and laboratory tests. The correlations with relatively lower determination coefficients are around the thresholds that are suitable for future tests with increased data volume.

The highest correlation level in this study was determined between E_M and SPT-N_{60} values, which are also frequently worked in literature. However, the correlations between the laboratory tests are unusual in literature. The reason for this could be the fact that soil structure usually contains factors that influence the laboratory test results like the grain size, plasticity, and water content, which were also encountered in the study. Variables like consistency and water content, which are used in multiple regression analyses, often yield *p* values that are higher than 0.05. While this situation has lowered the level of correlation, it had a minor impact on the determination coefficients. Of the equations presented in Table 5, Eqs. 1, 3, 15, 17, 18, 20, and 23 have been determined as equations with relatively higher correlations with more statistically significant relationships.

All data obtained from in situ and laboratory tests were analyzed and put into graphs using spreadsheets prepared in Excel. The statistical correlations between the parameters obtained from the analyses were evaluated in the JMP-8v and SPSS-22v software (2020).

The E_M (predict) data derived from the equation (Fig. 7) and the E_M (measured) values correlated with the basic regression analysis results in a regression coefficient (R^2) of 0.83–0.87, which is greater than the coefficient of determination of Eqs. 9–15, except for Eq. 11 (Table 5).

While similar coefficients were obtained in certain simple regression analyses, the high consistence results obtained in multiple regression analyses where multiple variables were considered are of great importance. Equations that have been produced based on a higher number of variables reflect a concordance in data. Due to this concordance in multiple regression analyses, the results obtained are quite suitable. The nature of this relationship shows that the linear and non-linear multiple regression analyses are more suitable than the simple regression analyses.

When some literature equations were examined, it was determined that there were different distributions (Fig. 8a). The samples taken in this study, unlike other studies, generally have overconsolidated clay properties. After, 50 blow counts in the SPT values were obtained using automatic pile driver.

Considering the trend between SPT-N_{60} and E_M data obtained in this study, Yagiz et al. (2008) and Bozbey and Togrol's (2010) studies were observed to be closer to their trends (Fig. 8b). It can be said that the equations obtained between SPT-N_{60} and E_M in this study can give the best result for overconsolidated clays among other equations in the literature.

Conclusions and recommendations

In this study, the changes in physical and mechanical properties of clays in different depths in the study area including the Van thrust fault have been investigated with in situ and laboratory tests, and the test results were statistically evaluated.

In this study, some statistically significant empirical equations with high determination coefficient and 95% confidence interval between in situ and laboratory tests are proposed. Thus, new contributions and suggestions were made to the relations between in situ and laboratory findings of overconsolidated soils, which are limited in the literature. In addition, there is no correlation between MPT data and consolidation data in the literature, and a correlation obtained between these parameters was suggested in the study.

The statistical analysis indicates that the non-linear multiple regression approach is more suitable than the simple regression analysis. In the multiple regression steps of the statistical studies, the relationships between the E_M and P_L with the SPT-N_{60}, σ_{pc}, *c*, *w*, *PI*, and *PL* values were evaluated together. Generally, the equations with the high determination coefficients were obtained from multiple regression analysis.

Linear multiple regression analyses containing the dependent variable E_M and independent variables SPT-N_{60}, natural water content (w), c, and σ_{pc} have been performed, and the suggested equations are provided below. In the suggested equations, significant results were obtained when there are parameter value intervals for all SPT values in the 95% confidence interval, w is greater than 10, σ_{pc} is greater than 50, and c values are between 0.1 and 0.4.

Suggested equations	R^2	Equation no
$E_M = 24.016\ (\text{SPT-N}_{60})^{0.6555}$	0.90	Equation 11
$E_M = 75.501\ (\sigma_{pc})^{1.9005}$	0.83	Equation 9
$E_M = 24.886\ (c)^{0.7938}$	0.73	Equation 10
$P_L = 2.8661(\text{SPT-N}_{60})^{0.5734}$	0.58	Equation 16
$E_M = 18.893 + 0.3413\ \text{SPT-N}_{60} - 0.365w$	0.85	Equation 26
$E_M = 43.925\ (\text{SPT-N}_{60})^{0.519} + 78.104\ \ln(w) - 86.031$	0.87	Equation 28
$E_M = 9.032 + 1.614\ \sigma_{pc} + 0.323\ \text{SPT-N}_{60}$	0.85	Equation 25
$E_M = 17.017 + 0.344\ \text{SPT-N}_{60} + 0.112\text{PI} - 0.458w$	0.85	Equation 18
$E_M = -8.163 + 74.739\ \sigma_{pc} + (7.457c/98.1)$	0.79	Equation 23
$P_L = 1.2297 + 0.0247\ \text{SPT-N}_{60} + 0.0136w$	0.51	Equation 31
$P_L = 1.112 + 10.994\ \sigma_{pc} + 0.332\ c/98.1$	0.52	Equation 33

Correlations with high determination coefficients are found in this study as the results of various simple and multiple regressions. The authors suggest that the correlations with relatively lower determination coefficients be tested again with the increased number of data. We suggest increased borehole and number of data for future studies so that more accurate evaluations with lower error margins could be performed. We also suggest further analysis for clay levels of different properties.

Declarations

Competing interests The authors declare no competing interests.

References

Acarlar M, Bilgin ZA, Erkal T, Güner E, Şen AM, Umut M, Elibol E, Gedik İ, Hakyemez Y, Uğuz MF (1991) Van Gölü Doğu Ve Kuzeyinin Jeolojisi. MTA Raporu, Ankara, No 9469:94s (in Turkish)

Ağan C (2014) Examination of relationships between Menard pressuremeter, standard penetration and laboratory tests data on the silty soil (Kastamonu, Turkey). İMO Technical Journal 415:6679–6698

Aggour MS, Radding WR (2001) Standard penetration test (SPT) correction, Research report submitted to Maryland Department of Transportation, Report No. SP007B48, State Highway Administration

Akkaya İ, Özvan A, Tapan M, Şengül MA (2015) Determining the site effects of 23 October 2011 earthquake (Van province, Turkey) on the rural areas using HVSR microtremor method. J Earth Syst Sci 124(7):1429–1443

Akkaya İ, Özvan A, Akın M, Akın M, Övün U (2017) Kayma Dalgası Hızı (Vs) Kullanılarak Erciş (Van) Yerleşim Alanının Sıvılaşma Potansiyelinin Değerlendirilmesi. Çukurova Üniversitesi Mühendislik Mimarlik Fakültesi Dergisi 32:55–68 (in Turkish)

Akkaya İ, Özvan A, Akın M, Akın MK, Övün U (2018) Comparison of SPT and Vs-based liquefaction analyses: a case study in Erciş (Van, Turkey). Acta Geophys 66(1):21–38

Akkaya İ, Özvan A, Özvan EE (2019) A new empirical correlation between pressuremeter modules (EM) and shear wave velocity (Vs) for clay soils. J Appl Geophys 171:103865

Akkaya İ, Özvan A (2019) Site characterization in the Van settlement (Eastern Turkey) using surface waves and HVSR microtremor methods. J Appl Geophys 160:157–170

Aladağ CH, Kayabasi A, Gokceoglu C (2013) Estimation of pressuremeter modulus and limit pressure of clayey soils by various artificial neural network models. Neural Comput Applic 23:333–339

Anwar MB (2016) Correlation between PMT and SPT results for calcareous soil. Housing Building Nat Res Center. https://doi.org/10.1016/j.hbrcj.2016.03.001

ASTM (1994) Annual book of ASTM Standarts-Section 4, Construction, vol 0408. Building Stones. ASTM Publication, Soil and Rock, p 978

ASTM D4318–00 (2000) Standard test methods for liquid limit, plastic limit, and plasticity index of soils, ASTM International, West Conshohocken, PA, www.astm.org

ASTM D2850–15 (2015) Standard test method for unconsolidated-undrained triaxial compression test on cohesive soils, ASTM International, West Conshohocken, PA, www.astm.org

ASTM D1586 / D1586M-18 (2018) Standard test method for standard penetration test (SPT) and split-barrel sampling of soils, ASTM International, West Conshohocken, PA, www.astm.org

Baguelin F, Jezequel JF, Shields DH (1978) The Pressuremeter and foundation engineering, Trans Tech Publications

British Standards Institution (2007) BS EN ISO 22476–3: geotechnical investigation and testing-field testing (part 3: standard penetration test). British Standards Institution, London, p 2007

Bowles JE (1997) Foundation analysis and design, 5th edn. McGraw-Hill, Singapore

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

Casagrande A (1936) Characteristics of cohesionless soils affecting the stability of slopes and earth fills. Jour Boston Soc Civ Eng 23:13–32

Cetin H (1997) How did the Meers fault scarp form? Paleoearthquake or aseismic creep? A Soil Mech Perspective: Eng Geol 47:289–310

Cetin H (2000) An experimental study of soil memory and preconsolidation adjacent to an active tectonic structure: the Meers fault, Oklahoma, USA. Eng Geol 57:169–178

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. Geomech Geoeng 10(3):194–202

Cheshomi A, Khalili A (2021) Comparison between pressuremeter modulus (E_{PMT}) and shear wave velocity (Vs) in silty clay soil. J Appl Geophys, 192 https://doi.org/10.1016/j.jappgeo.2021.104399

Cheshomi A, Bakhtiyari E, Khabbaz H (2020) A comparison between undrained shear strength of clayey soils acquired by "PMT" and laboratory tests. Arab J Geosci 13:640. https://doi.org/10.1007/s12517-020-05660-9

Chiang YC, Ho YM (1980) Pressuremeter method for foundation design in Hong Kong. In: Proceedings of sixth Southeast Asian Conference on Soil Engineering 1 1980; 31–42

Ching JY, Phoon KK (2012) Modeling parameters of structured clays as a multivariate normal distribution. Can Geotech J 49(5):522–545

Ching JY, Phoon KK (2013) Multivariate distribution for undrained shear strengths under various test procedures. Can Geotech J 50(9):907–923

Ching JY, Phoon KK (2014) Correlations among some clay parameters - the multivariate distribution. Can Geotech J 51:686–704

Clarke BG (1995) Pressuremeters in geotechnical design. Blackie/Chapman and Hall, London

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

Gonin H, Vandangeon P, Lafeullade MP (1992) Correlation study between standard penetration and pressuremeter tests. Rev Fr Geotech 58:67–78

Idriss IM, Boulanger RW (2008) Soil liquefaction during earthquakes. Monograph MNO-12, Earthquake Engineering Research Institute, Oakland, CA, 261 pp

Idriss IM, Boulanger RW (2010) SPT-based liquefaction triggering procedures, Report No. UCD/CGM-10/02, department of Civil & Environmental Engineering College of Engineering University of California, pp 259

Kayabaşı A (2012) Prediction of pressuremeter modulus and limit pressure of clayey soils by simple and non-linear multiple regression techniques: a case study from Mersin, Turkey. Environ Earth Sci 66:2171–2183

Kayabaşı A, Gökceoğlu C (2012) Taşıma Kapasitesi ve Oturma Miktarının hesaplanmasında Yaygın Kullanılan Yöntemlerin Mersin Arıtma Tesisi Temeli Örneğinde Uygulanması. TMMOB Jeoloji Mühendisleri Odası, Jeoloji Mühendisliği Dergisi 36(1):1–22 (in Turkish)

Koçyiğit A (2013) New field and seismic data about the intraplate strike-slip deformation in Van region, East Anatolian plateau E Turkey. J Asia Earth Sci 62:586–605

Mair RJ, Wood DM (1978) Pressuremeter testing methods and interpretation. CIRIA

Menard L (1957) An apparatus for measuring the strength of soils in place. University of Illinois, Thesis

Ohya S, Imai T, Matsubara M (1982) Relationship between N value by SPT and LLT pressuremeter results. In: Proceedings of the 2nd European Symposium on Penetration Testing, vol 1, Amsterdam, The Netherlands, 24–27 May 1982, 125–130

Özvan A, Akkaya İ, Tapan M, Şengül MA (2005) Van yerleşkesinin deprem tehlikesi ve olası bir depremin sonuçları, Deprem Sempozyumu Kocaeli 2005, 23–25 Mart 2005. Kocaeli (in Turkish)

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 Geol Env 77:1145–1154

Özvan A, Özvan EE, Akkaya İ, Akın M, Akın M (2019) A study of the relationship between the pressuremeter modulus and the preconsolidation pressure around a thrust fault. Environ Earth Sci 78:596. https://doi.org/10.1007/s12665-019-8597-1

Phoon KK, Kulhawy FH (1999) Evaluation of geotechnical variability. Can Geotech J 36:625–639

Phoon KK, Ching J (2013) Multivariate model for soil parameters based on Johnson distributions. In: Proceedings of Geo-Congress 2013. Foundation Engineering in the Face of Uncertainty Geotechnical Special Publications (GSP) GSP 229, San Diego, California, United States, pp. 337–353

Selçuk L (2003) Yüzüncü Yıl Üniversitesi Zeve Kampüsü Yerleşim Alanının Mühendislik Jeolojisi. Yüzüncü Yıl Üniversitesi, Fen Bilimleri Enstitüsü, Van, s.118 (in Turkish)

Sengul MA, Gürboğa Ş, Akkaya İ, Özvan A (2019) Deformation patterns in the Van region (Eastern Turkey) and their significance for the tectonic framework. Geol Carpath 70:193–208

Shaban, A. M., Cosentino, P. J. (2016) Development of the miniaturized pressuremeter test to evaluate unbound pavement layers. J Test Eval, 45(2). https://doi.org/10.1520/JTE20150322

Shields D, Bauer G (1975) Determination of the modulus of deformation of a sensitive clay using laboratory and in situ tests. In: Proceedings of the specialty conference on in situ measurement of soil properties, vol 1. American Society of Civil Engineers, Raleigh, North Carolina, 1975, 395–421.

SPSS (2020) Statistical package for the social sciences (v. 23). SPSS Inc., Chicago, IL

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 Env 67:405–410