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Some emprical equations for predicting standard penetration test blow counts in clayey soils: a case study in Mersin, Turkey

Ali Kayabaşı

Abstract The standard penetration test (SPT) is a widespread test used in situ. Soil mechanical properties such as grain-size content, moisture content, atterberg limits, activity of clay, and consistency of clay determine the SPT blow count. In addition, the SPT blow count reflects the soil mechanical and compressibility parameters. The purpose of this study is to determine the clayey soil states that determine the SPT blow count number; 220 data points derived from a foundation investigation of a sewerage project in Mersin City were used in the study. The Mersin sewerage project includes the sewerage station, central pumping station, and west pumping station in Mersin city, Turkey. The corrected SPT blow counts (N_{60}) and corresponding parameters (fine-grain size percent, liquid limit, plastic limit, plasticity index, moisture content, activity, and consistency) of the clayey soil from these locations were used in a series of simple and nonlinear multiple regression analyses. Based on the analyses, the acceptable results have been determined by a simple regression analyses between the corrected SPT blow counts and the following input parameters: activity, moisture content (in percent), fine-grained percent (in percent). The simple regression analyses for N_{60}-LL (in percent), N_{60}-PL (in percent), and N_{60}-PI (in percent) are considered unacceptable so these parameters are not evaluated as input parameters in the multiple regression analyses. Multiple regression analyses with two independent input parameters produces acceptable results. The optimal equation is determined with multiple regression analyses using activity and moisture content as the input parameters; this equation has a coefficient of determination of R^2=0.78. The prediction capability of the developed equations is also acceptable. However, the prediction equations proposed in this study are developed with a limited number of data points. For this reason, a cross-check should be performed before using these empirical equations for design purposes.

Keywords Regression · Activity · Consistency · SPT · Clayey soil

Introduction

The standard penetration test (SPT) is one of the most frequently used tests in geotechnical studies. The SPT was developed in 1927 (Bowles 1988), and this test is currently used and has become one of the most popular in situ tests. The allowable bearing pressure and SPT blow count in sands were correlated and published by Terzaghi and Peck (1948). Gibbs and Holtz (1957) considered the overburden pressure and length of the drill rod as a cause of non productibility between adjacent boreholes, and they suggested an overburden correction. Lacroix and Horn (1973) proposed a correction for correlations of nonstandard penetration resistances with standard penetration resistances. Skempton (1986) defined the corrected blow count (N_{60}) as the SPT N value corrected for field procedures and apparatus, including the hammer efficiency, borehole correction, sample barrel correction and rod length correction. The Gibbs and Holtz overburden correction results in higher blow counts for uniform soil deposits lying over a stiffer material, so Liao and Whitman (1986) suggested an overburden correction termed $(N_1)_{60}$. Numerous researchers have proposed and used empirical correlations between SPT N values and shear wave velocity: Seed and Idriss (1981), Lee (1990), Dikmen (2009), Hasancebi and Ulusay (2007), Ulugerli and Uyanık (2007), Chatterjee and Deepankar (2013), and Ayolabi and Adegbolo (2014). There are also many empirical equations in the literature derived from correlations of SPT blow count and other

A. Kayabaşı (✉)
Department of Geological Engineering, Eskisehir Osmangazi
University, Eskisehir, Turkey
e-mail: akayabasi@ogu.edu.tr

in situ test parameters; Yagiz et al. (2008), Bozbey and Togrol (2010), and Kayabaşı (2012) studied the correlations of SPT blow counts and pressuremeter parameters. Researchers have proposed a series of empirical equations for sandy, silty, and clayey soils. The empirical equations between cone penetration tip resistance and SPT blow count number N values are also common in the literature: Schmertmann (1970), Chang (1988), Emrem and Durgunoglu (2000), and Akca (2003). Standard penetration test is also used in geotechnical researches widely. Topsakal and Topal (2014) studied slope stability of a re-activated landslide on the Artvin-Savsat road in Meydancik, Turkey. They performed SPT test in boreholes. The SPT-N values generally increased with depth but researchers stated that there is a relationship between the failure surface and the SPT-N values, it can be stated that the SPT-N values decreases around the failure surface. The application of the SPT test for geographic information system for the development of soil suitability map was carried out by Stalin and Vurukan Arun (2014). Investigators countered study area with SPT N values and bearing capacity for various depths and they suggested that the potential of GIS to find the solution of phenomena that was very tedious by any other means, not only in terms of accuracy but also by the use of derived information as the input for other correlation analysis.

The related standards for SPT are the ASTM D 1586 (Bowles 1988), British Standard 1377 (1975), ENV (Eurocode 7) 1997, and Türk Standartları Enstitüsü (TSE) 5744 (1988). SPT is a dynamic test performed for the determination of the strength parameters of silty, sandy, and clayey soils and heavily weathered rock units. A standard split sampler is driven into the soil by dropping a 63.5-kg hammer on the drill rod from a height of 76 cm. The sampler is advanced in the soil by 450 mm (Fig. 1). The number of blow counts is recorded for each 150 mm. The blow counts of the first 150 mm are not included in the calculation to avoid disturbances of the soil. The blow counts of the second and third 150 mm advance of the sampler are recorded separately. These blow counts are the raw blow counts of the test. Then, a groundwater correction, rod energy ratio correction, rod length correction, sample barrel correction and borehole diameter correction are applied, and the corrected blow counts (N_{60}) are obtained.

In this study, data from an investigation of the foundations of a sewerage station (Kayabasi and Gürsoy 1995a, b, c), central pumping station (Kayabasi and Gürsoy 1995a, b, c), and west pumping station (Kayabasi and Gürsoy 1995a, b, c) in Mersin City from Turkey are employed. The location plan of stations is shown in Fig. 2. A total of 20 boreholes in the sewerage station, 10 boreholes in the central station, and 4 boreholes in the west pumping station were drilled. One hundred thirty SPTs at the sewerage
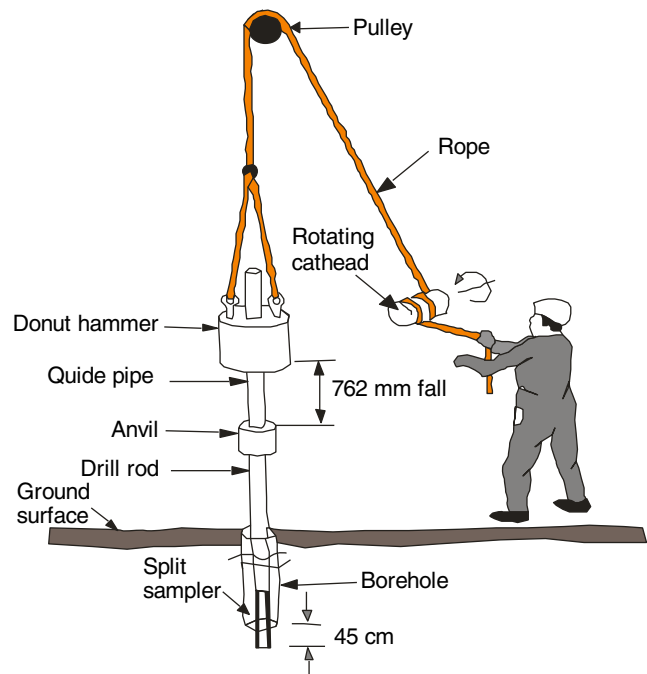

Fig. 1 Sketch of standard penetration test instrumentation (Coduto 1994)

station, 72 SPTs at the central pumping station, and 18 SPTs at the west pumping station were performed in the boreholes. The drilling program of the boreholes can be summarized as follows: drill 0.00–1.50 m, perform the SPT test at 1.50–1.95 m, drill 1.95–2.10 m, clear the borehole, sample 2.10–2.70 m undisturbed soil, drill 2.70–3.00 m, clear the borehole, and perform the SPT test at 3.00–3.45 m. This program is conducted for the entire determined depth of the borehole. Constant head permeability tests and pressuremeter tests were also performed in certain boreholes. During the drilling program of the sewerage station, central pumping station, and west pumping station, disturbed and undisturbed samples were collected from boreholes. The disturbed samples from the split-spoon sampler were collected, examined, labeled, and placed in a sealed-waxed jar, and the undisturbed samples are also labeled, waxed, and sent to the General Directory of Electrical Power Resources Survey and Development (EIEI) laboratory daily.

Laboratory testing

A series of laboratory tests were applied for the sewerage project of Mersin. Bulk unit weight tests, sieve tests, Atterberg limit tests, natural moisture content tests, direct shear tests, consolidation tests and triaxial compression tests were conducted on the borehole samples. All of the samples were collected and tested in accordance with the procedures suggested by ASTM (American Society for Testing and Materials) (1994).
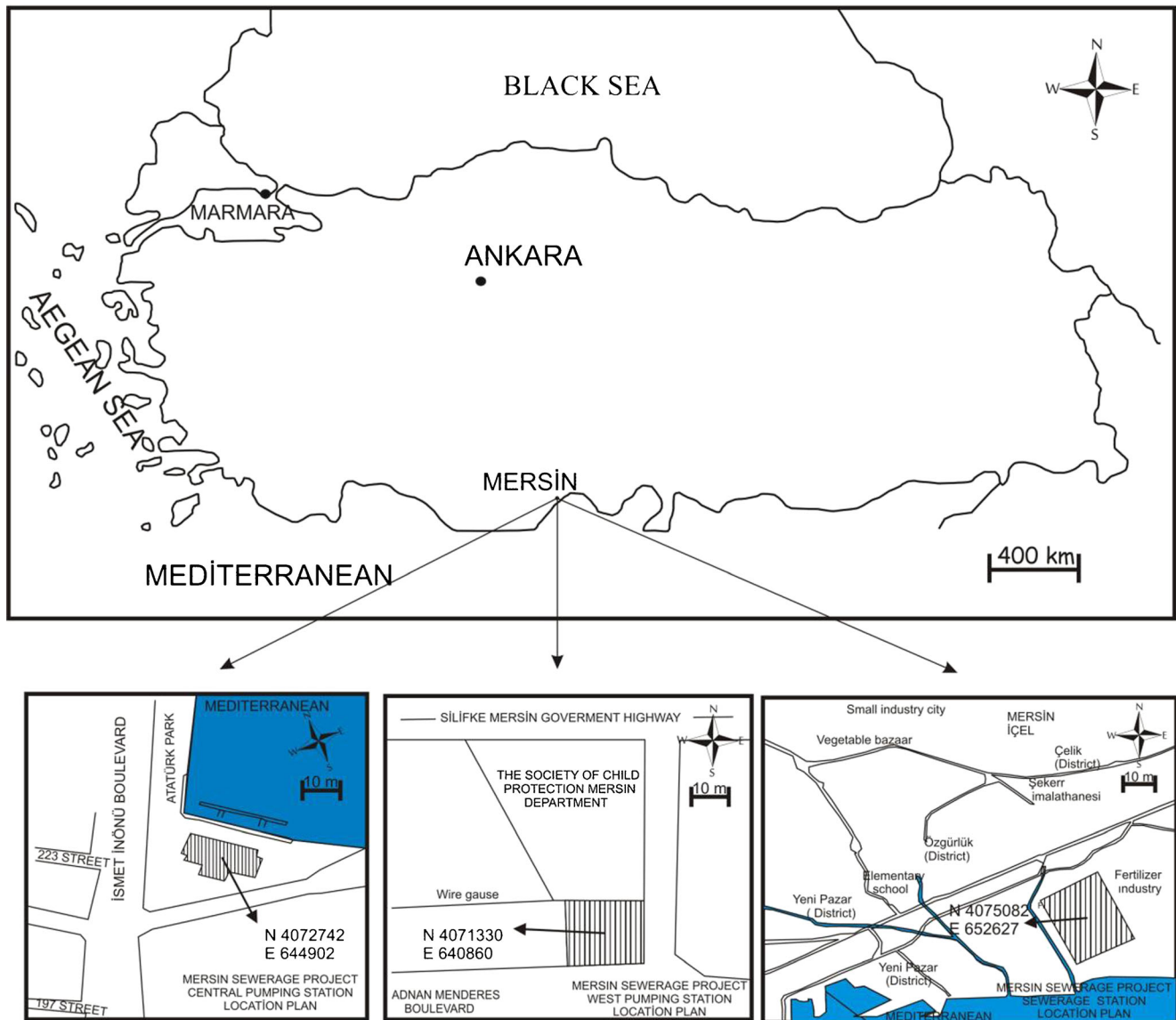
Fig. 2 Location map of the study area

The undisturbed samples were not representative of SPT test levels because of depth differences between the undisturbed samples and SPT test levels. Therefore, the geotechnical properties of the undisturbed samples determined in the laboratory were not correlative with the SPT blow counts; however, the index properties of the samples from the split spoon sampler were correlative with the SPT blow counts.

According to the mean grain size distribution, the soil samples of the sewerage station consist of 74.9 % fine-grained material (no. <200 sieve) and 39.5 % clay-sized material (USBR United States Dep. Int. Bur. Reclamation 1974). The 78 of 130 fine-grained sample locates over the A line. The fine-grained material is mainly highly plastic clay (CH) (Fig. 3). Others locate under the A line, so they can be named as OH or MH soil. The mean values of the liquid limit

(LL) and plastic limit (PL) are 61.5 and 31.0 %, respectively. The plasticity index (PI) mean value is 28.89 (Table 1). The activity (Ac) of the clayey soil is determined by the PI divided by the percent of clay-sized particles (less than 2 μm). The mean activity of the sewerage station clayey soil is 0.57 with a minimum value of 0.22 and maximum value of 3.14 (Fig. 4a). According to the activity classification proposed by Seed et al. (1964), the soil samples are inactive clay. Skempton (1953) classified soils according to activity, and the activity value of montmorillonite changes between 1 and 7. The Ca montmorillonite amount increases to approximately 1, and the Na montmorillonite amount increases to 7 with increasing activity value. The activity of illites is between 0.5 and 1, and the activity of kaolinite is approximately 0.5 (Mitchell 1975). According to this classification, the sewerage foundation soil can be classified as Ca montmorillonite. The consistency (Ic)

Fig. 3 Distribution of the
foundation clays on a plasticity
chart
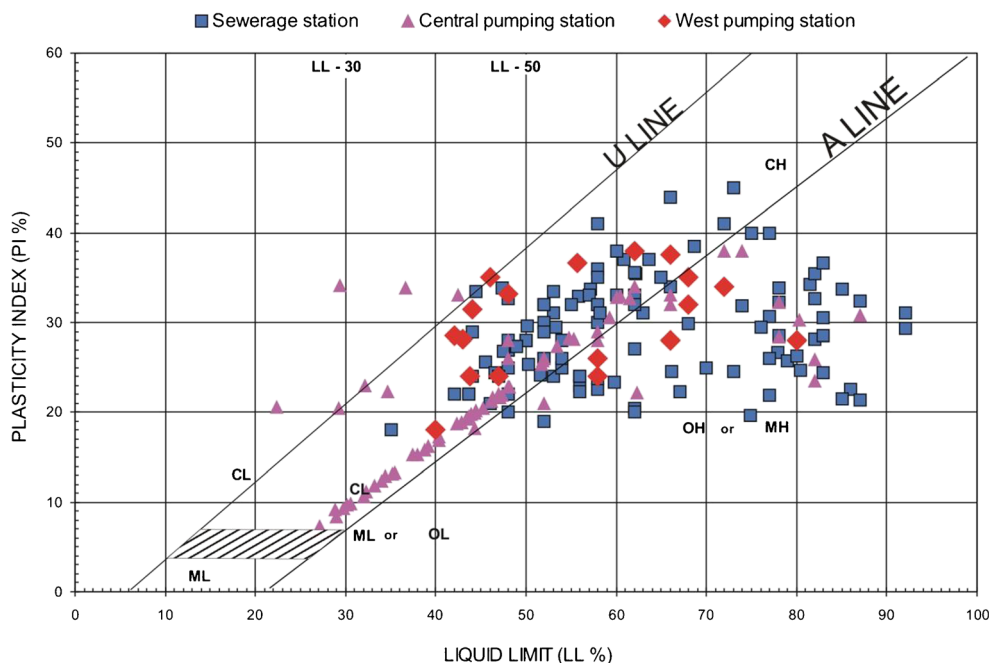
of silt and clay is defined as the difference between the liquid
limit (LL) and moisture content (w) divided by the plasticity
index. Therefore, consistency can be defined as the resistance
of soil to penetration. Peck et al. (1953) proposed a chart
showing the relationship between SPT blow count and soil
consistency (Table 2). Clay and silts can be classified as very
soft to very stiff (British Standart EN ISO 14688-2 2013),
(Table 3), and 63 % of the foundation soils had a consistency
greater than 1, which indicates very stiff or hard soil, and 18 %
of the foundation soil consistency is between 0.75 and 1,
which indicates stiff soil (Fig. 5).

The soil samples of the central pumping station consist of
fine-grained material 47 % (no. <200 sieve) and 18 % clay-
sized material (USBR United States Dep. Int. Bur.
Reclamation 1974); 48 of 72 samples are mainly low plastic
(CL), with the exceptions being13 sample organic clays or
inorganic silts with high plasticity (OH-MH) and 11 samples
that are high plastic clay (CH) (Fig. 3). All of the samples of

the central pumping station separate nearly parallel to the A
line of the plasticity chart. The mean values of the liquid limit
(LL) and plastic limit (PL) are 48 and 28 %, respectively
(Table 4). The plasticity index mean value is 21 %. The mean
activity of the central pumping station clayey soil is 0.34 with
a minimum value 0.096 and maximum value of 1.31
(Fig. 4b). According to the mean value and the classi-
fication of activity proposed by Seed et al. (1964), the
central pumping station soil is inactive clay. According
to Mitchell (1975) clay classification, the clayey soil of
central pumping station can be classified as kaolinitic
clay soil. The 51 % of the foundation soil sample of
central pumping station have consistency greater than 1,
which indicates very stiff or hard soil. 22.5 % of the
central pumping foundation soil consistency is between
0.75 and 1, which indicates stiff soil (Fig. 5b).

The soil samples of the west pumping station consist of fine-
grained material 69 % (no. <200 sieve) and 30 % clay-sized

Table 1 Statistical evaluation of
foundation soil sample properties
from sewerage station

Parameter	N	Range	Minimum	Maximum	Mean	Std. Deviation	Variance
SPT	113	41.00	7.00	48.00	25.50	13.10	171.74
Finerthan no200 (%)	113	85.00	13.00	98.00	66.13	22.63	512.32
LL	113	66.00	33.00	99.00	61.50	12.64	159.81
PL	113	44.70	11.00	55.70	31.00	11.17	124.85
PI	113	27.00	18.00	45.00	28.89	5.74	32.98
Activity	113	2.93	0.22	3.14	0.57	0.47	0.22
w (%)	113	37.30	7.70	45.00	27.88	9.31	86.74
SPT N_{60}	113	21.15	7.20	28.35	17.95	6.09	37.04
Consistency index (Ic)	113	1.99	0.03	2.02	1.14	0.46	0.21

(a)

(b)

(c)

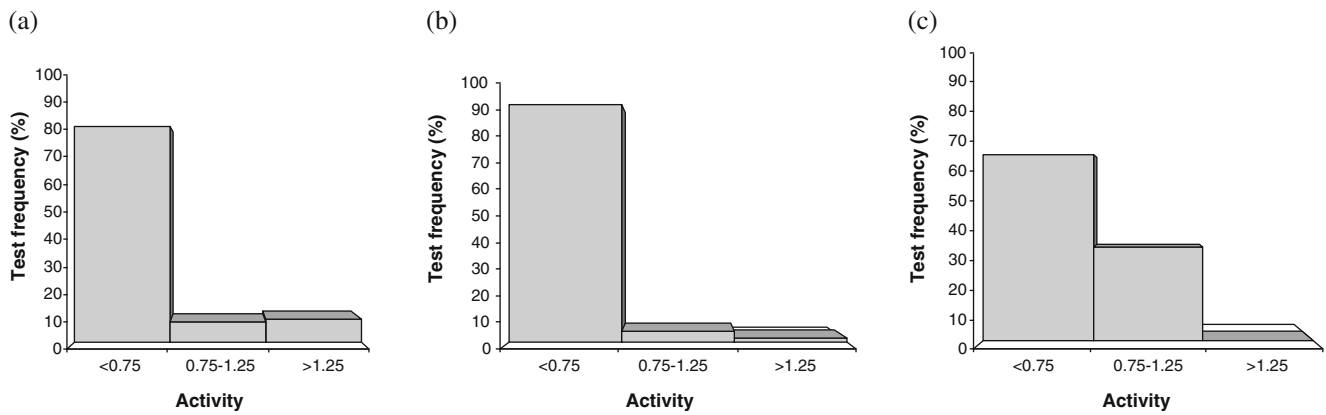Fig. 4　Activity histograms of the study area: **a** central pumping station; **b** west pumping station; and c, sewerage station

material (USBR1974) of 18 samples are low plastic (CL) that accumulates close to 50 % of the liquid limit line and 3 samples over the A line are CH, with the exceptions being 5 samples of organic clays or inorganic silts with high plasticity (OH-MH) close to the A line and 1 sample of organic clays or inorganic silts with high plasticity (OH) (Fig. 3). The mean values of the liquid limit (LL) and plastic limit (PL) is 64.6 and 25.87 %, respectively. The plasticity index mean value is 30.09 (Table 5). The mean activity of the west pumping station clayey soil is 0.62 with a minimum value 0.24 and maximum value of 1.59 (Fig. 4c). According to the classification of activity proposed by Seed et al. (1964), the west pumping station soil is inactive clay. West pumping foundation clayey soil is named as ca monmorillonite according to Mitchell (1975) clay classification. Of the west pumping station soil, 94 % has a consistency greater than 1, which indicates very stiff or hard soil, and 6 % of the foundation soil consistency is between 0.75 and 1, which indicates stiff soil (Fig. 5c).

Regression analysis

Simple regression analysis

The regression analysis is a widespread statistical method used for the derivation of empirical equations (Bieniawski 1978; Hoek and Brown 1997; Kayabasi et al. 2003; Yagiz and Gokceoglu

2010; Kayabaşı 2012; Chen-Chang et al. 2011; AlHomadhi Emad 2014), and such analyses require comparable data. In this study, the SPT blow count and parameters derived from the laboratory tests on the disturbed samples are compared. Thus, a total of 220 corrected SPT blow counts and the same number of laboratory test results are correlated. A series of simple regression analyses are performed to determine the empirical relationships between the SPT blow count and the following parameters: moisture content (w, in percent), fine-grained material percent (FGP, in percent), liquid limit (LL, in percent), plastic limit (PL, in percent), plasticity index (PI, in percent), consistency index (Ic) of clayey material, and activity (Ac) of clayey material (Table 6).

As shown in Table 6, the acceptable results of the simple regression analysis can be summarized as follows: the equations with the highest coefficients of regression between N_{60} and activity (Ac) are represented by a logarithmic function (Fig. 6); the correlation between N_{60} and moisture content produces the highest regression coefficient with a linear function, and the correlation between N_{60} and FGP (in percent) produces the highest regression coefficient with a linear function also. A power function produces the highest regression coefficient between N_{60} and Ic, and it is also a significant result. A summary of the simple regression analyses results shows that increasing activity and consistency of soil increases the N_{60} value with a logarithmic function and power function, respectively. Decreasing water content and fine-grained percent increases the N_{60} value with a linear function.

Table 2　Penetration resistance, SPT blowcount, and undrained shear strength (Peck et al 1953)

SPT-N_{60}	Consistency	Undrained shear strength Cu (kPa)
<2	Very soft	<12.5
2–4	Soft	12.5–25
4–8	Firm	25–50
8–15	Stiff	50–100
15–30	Very stiff	100–200
>30	Hard	>200

Table 3　Defining of qualitative and quantitative consistency (BS EN ISO 14688-2)

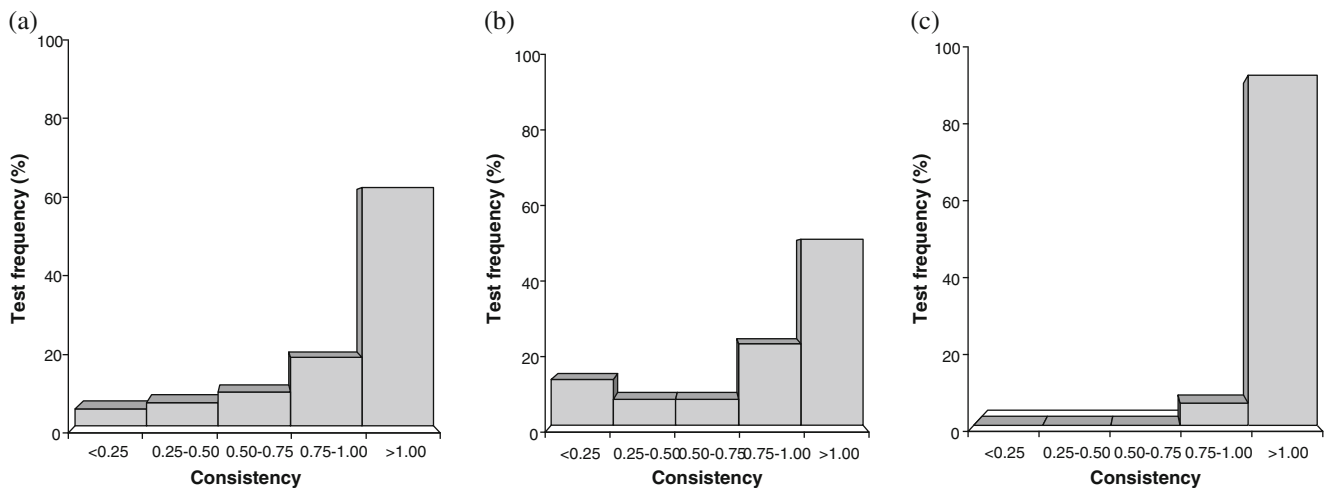| Consistency of silts and clays	Consistency index (Ic)
Very soft	<0.25
Soft	0.25–0.50
Firm	0.50–0.75
Stiff	0.75–1.00
Very stiff	>1.00

Fig. 5 Consistency histograms of the study area: **a** central pumping station; **b** west pumping station; and c, sewerage station

However, the coefficient regressions of the simple regression analyses for N_{60}-LL (in percent), N_{60}-PL (in percent) and N_{60}-PI (in percent) are low, so these regressions are considered to be unacceptable results and are not evaluated as input parameters in the multiple regression analysis.

The prediction performance of the simple regression equations are evaluated with root mean square error (RMSE) and values accounted for (VAF) indices. Determinations of RMSE and VAF have long been used by various researchers (Alvarez Grima and Babuska 1999; Gokceoglu 2002; Dagdelenler et al. 2011; Minaeian and Ahangari 2013). If the RMSE values are closer to 0 and VAF values are closer to 100, the prediction performance of the equation is excellent. The RMSE and VAF indices for each parameter are provided in Tables 7 and 8.

The optimum results are from the equation derived from the correlation between the moisture content and measured SPT blow count; however, the other equations also provide acceptable results.

Multiple regression analysis with two independent parameters

Four parameters of clayey soil properties have been determined from simple regression analyses that could be used as independent input parameters for the multiple regression analyses. These four parameters are soil activity (Ac), fine-grained percent (FGP), soil consistency (Ic) and moisture content (w). A combination of these parameters in two groups results in the following six functions:

$$f(w, Ac) = N_{60} \tag{1}$$

$$f(Ac, FGP) = N_{60} \tag{2}$$

$$f(Ac, Ic) = N_{60} \tag{3}$$

$$f(w, FGP) = N_{60} \tag{4}$$

$$f(w, Ic) = N_{60} \tag{5}$$

$$f(Ic, FGP) = N_{60} \tag{6}$$

Table 4 Statistical evaluation of foundation soil sample properties from central pumping station

Parameter	N	Range	Minimum	Maximum	Mean	Std. deviation	Variance
SPT	71	45.00	2.00	47.00	19.23	11.16	124.48
Finer than no. 200 (%)	71	76.00	22.00	98.00	72.56	18.33	335.89
LL	71	67.90	27.10	95.00	48.99	13.12	172.11
PL	71	49.20	9.30	58.50	25.63	8.62	74.30
PI	71	30.70	7.30	38.00	21.98	7.84	61.50
Activity	71	1.21	0.10	1.31	0.34	0.24	0.06
w (%)	71	30.40	11.60	42.00	26.61	6.15	37.78
SPT N_{60}	71	20.25	7.65	27.90	15.40	5.02	25.21
Consistency index (Ic)	71	1.94	0.07	2.01	0.93	0.44	0.20

Table 5 Statistical evaluation of foundation soil sample properties from west pumping station

Parameter	N	Range	Minimum	Maximum	Mean	Std. deviation	Variance
SPT	18	37.00	13.00	50.00	35.92	10.66	113.74
Finer than no. 200 (%)	18	70.00	22.00	92.00	56.55	20.56	422.89
LL	18	55.00	40.00	95.00	64.60	13.69	187.44
PL	18	41.10	10.90	52.00	25.88	11.25	126.48
PI	18	20.00	18.00	38.00	30.09	5.59	31.29
Activity	18	1.35	0.24	1.59	0.63	0.32	0.10
w (%)	18	28.40	8.90	37.30	18.30	8.52	72.65
SPT N_{60}	18	16.65	12.60	29.25	22.91	4.80	23.03
Consistency index (Ic)	18	1.24	0.89	2.13	1.53	0.36	0.13

When developing an empirical equation, the input parameters should not be interrelated each other. Functions 2, 3, and 5 consist of related parameters. Activity and fine-grained percent of clayey soil are the input parameters of function 2. FGP is a determining parameter of activity, so there is relation between input parameters of Function 2. If we examine the input parameters of Function 3, plasticity index is determining parameter both activity and consistency index, so there is also relationship between activity and consistency index. Lastly, water content is determining parameter consistency index, there is relationship between input parameters of Function 5 also. Therefore, functions 2, 3, and 5 are eliminated in the multiple regression analyses. The Functions 1, 4, and 6 are accepted for multiple regression analysis due to there is not interrelationship between input parameters of functions.

The simple regression equation between N_{60} and activity (Ac) is a logarithmic relationship, whereas the simple regression equation between N_{60} and moisture content (w, in percent) is a linear relationship. The nature of these equations shows that the nonlinear multiple regression is more suitable than the linear multiple regression.

$$N_{60} = 7.92\ln(Ac) + 24.70 \tag{7}$$

$$N_{60} = -0.52(w) + 31.20 \tag{8}$$

The combination of Eqs. 7 and 8 can be defined as follows:

$$N_{60} = A\ln(Ac) + B(w) + C \tag{9}$$

A, B, and C are the coefficients of the multiple regression equation. The following equation is obtained for the predictions of the SPT blow count N_{60} from a nonlinear regression analysis using a statistical program (SPSS 2002).

$$N_{60} = 5.55\ln(Ac) - 0.32(w) + 31.7 \tag{10}$$

The coefficient of determination (R^2) between $N_{60 \text{ (measured)}}$ and $N_{60 \text{ (predicted)}}$ from Eq. 10 is 0.78 (Fig. 7).

$$N_{60(\text{measured})} = 1.0002 N_{60(\text{predicted})} - 0.0117 \left(R^2 = 0.78\right) \tag{11}$$

Subsequently, the combination of equations N_{60}-moisture content (w) and N_{60}-fine-grained percent (in percent) equation

Table 6 Estimating emprical equations for N_{60} versus soil parameters

| Parameters	Linear	Logarithmic	Power	Exponential
N_{60}-Ac	N_{60}=9.78 (Ac)+12.66 (R^2=0.446)	N_{60}=7.92ln (Ac)+24.70 (R^2=0.619)	N_{60}=24.96 (Ac)$^{0.458}$ (R^2=0.446)	N_{60}=12.60e$^{0.54}$(Ac) (R^2=0.379)
N_{60}-w (%)	N_{60}=−0.52(w)+31.20 (R^2=0.56)	N_{60}=−11.8ln (w)+55.56 (R^2=0.55)	N_{60}=142.73(w)$^{-0.67}$ (R^2=0.49)	N_{60}=36.34e$^{-0.030}$(w) (R^2=0.51)
N_{60}-FGP (%)	N_{60}=−0.21(FGP)+31.03 (R^2=0.52)	N_{60}=−0.22ln(FGP)+59.95 (R^2=0.51)	N_{60}=164.59(FGP)$^{-0.56}$ (R^2=0.41)	N_{60}=34.5e$^{-0.011}$(FGP) (R^2=0.379)
N_{60}-LL(%)	N_{60}=0.188(LL)+6.74 (R^2=0.20)	N_{60}=10.8ln(LL)−25.90 (R^2=0.22)	N_{60}=1.0974(LL)$^{0.67}$ (R^2=0.23)	N_{60}=8.47e$^{0.011}$(LL) (R^2=0.21)
N_{60}-PL(%)	N_{60}=−0.15(PL)+21.81 (R^2=0.072)	N_{60}=−4.62ln(PL)+32.62 (R^2=0.078)	N_{60}=35.42(PL)$^{-0.233}$ (R^2=0.056)	N_{60}=20.57e$^{-0.0078}$(PL) (R^2=0.054)
N_{60}-PI (%)	N_{60}=0.44 (PI)+5.76 (R^2=0.30)	N_{60}=9.60ln(PI)−13.56 (R^2=0.28)	N_{60}=2.26(PI)$^{0.61}$ (R^2=0.32)	N_{60}=7.91e$^{0.028}$(PI) (R^2=0.32)
N_{60}-Ic	N_{60}=8.90(Ic)+7.72 (R^2=0.50)	N_{60}=4.69ln(Ic)+17.89 (R^2=0.35)	N_{60}=16.87(Ic)$^{0.30}$ (R^2=0.40)	N_{60}=9.1166e$^{0.537}$(Ic) (R^2=0.51)
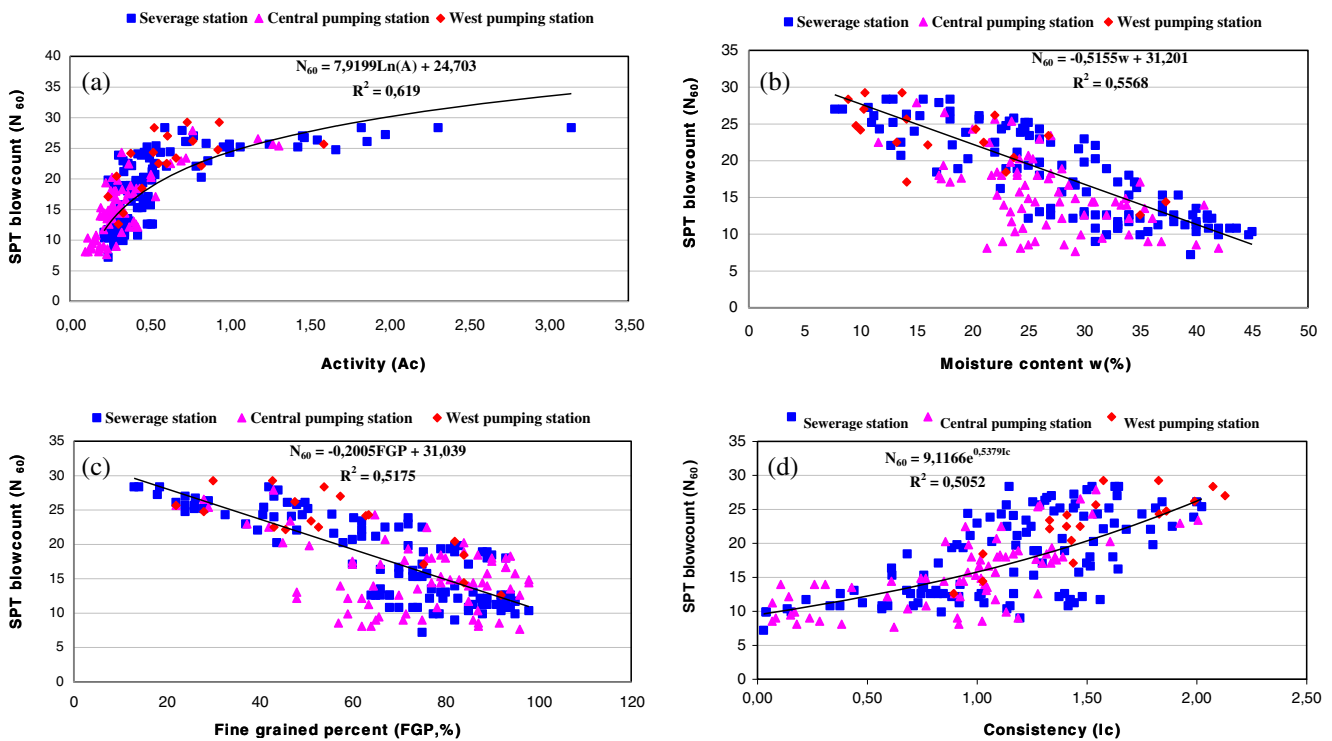
Fig. 6 Graphs showing the simple regression analyses: **a** between N_{60} and activity; **b** between N_{60} and moisture content; **c** between N_{60} and fine-grained percent; **d** between N_{60} and consistency

is conducted. The moisture content and fine-grained percent are defined as a function of the standard penetration blow count N_{60} in Eqs. 8 and 12.

$$N_{60} = -0.52(w) + 31.20 \tag{8}$$

$$N_{60} = -0.21(FGP) + 31.03 \tag{12}$$

Equations 8 and 12 are both linear, so the multiple linear regression analysis can be conducted. The combination of Eq. 8 and 12 can be expressed as follows:

$$N_{60} = A(w) + B(FGP) + C \tag{13}$$

A, B, and C are the coefficients of the equations. Equation 14 is obtained by employing a linear multiple regression analysis:

$$N_{60} = -0.344(w) - 0.122(FGP) + 34.879 \tag{14}$$

The coefficient of determination (R^2) between $N_{60\ (measured)}$ and $N_{60\ (predicted)}$ from Eq. 14 is 0.69 (Fig. 8).

The last stage of the multiple regression analysis is the combination of the fine-grained percent and consistency as a function of the standard penetration blow count N_{60} in Eq. 6.

The previously performed simple regression analyses produce Eqs. 12 and 15:

$$N_{60} = -0.21(FGP) + 31.03 \tag{12}$$

Table 7 The values of the root mean sguare errors (RMSE) and values account for (VAF) for the predicted N_{60} equation derived from simple regression analyses

Independent parameter	Predicted N_{60} equation	RMSE	VAF	n
Activity	$N_{60}=7.92\ln(Ac)+24.70$	4.24	45.23	220
Moisture content	$N_{60}=-0.52(w)+31.20$	3.39	64.30	220
Fine-grained percent	$N_{60}=-0.21(FGP)+31.03$	4.31	43.25	220
Consistency	$N_{60}=8.90(Ic)+7.72$	4.40	39.35	220

Table 8 The values of the root mean square errors (RMSE) and values account for (VAF) for the predicted N_{60} equation derived from multiple regression analyses with two independent variables

Independent parameter	Predicted N_{60} equation	RMSE	VAF	n
Ac–moisture content	$N_{60}=5.55\ln(Ac)$ $-0.32(w)+31.7$	3.14	64.60	220
Moisture content–FGP	$N_{60}=-0.344(w)$ $-0.122(FGP)+34.879$	4.02	63.30	220
FGP–Ic	$N_{60}=2,559e^{0.002}(Ic)$ $-0.147(FGP)-2539$	4.03	39.35	220

Fig. 7 Graph showing the relationship between N_{60} (measured) and N_{60} (predicted) with two variables: activity and moisture content
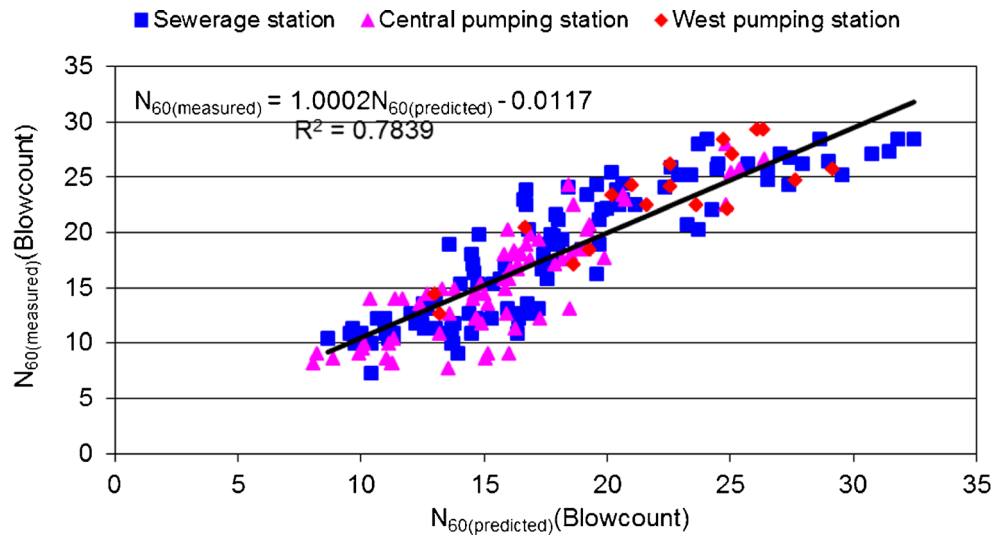
$$N_{60} = 9.1166 e^{0.537(Ic)} \qquad (15)$$

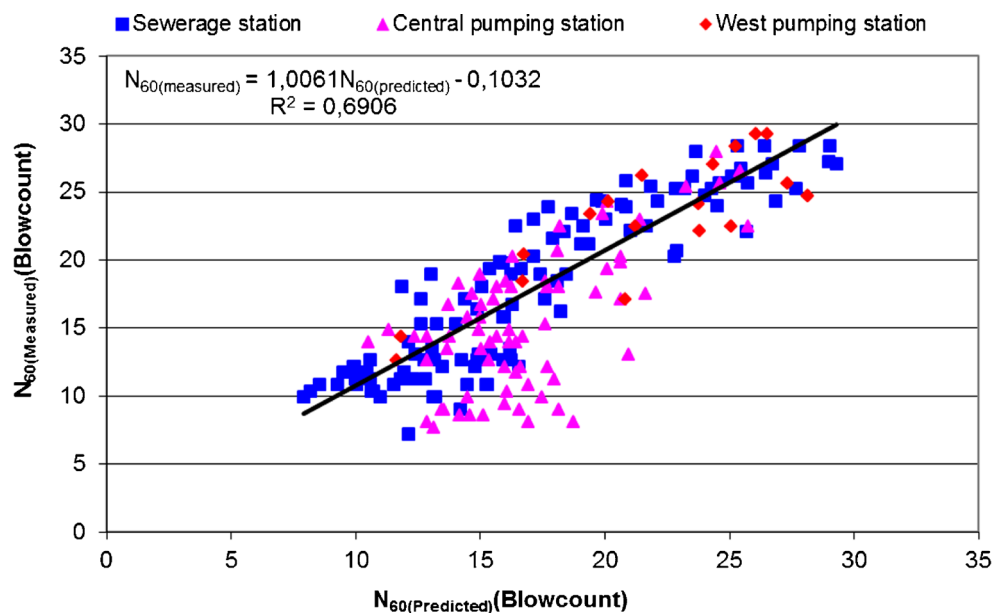The combination of the Eqs. 12 and 15 can be expressed as follows:

$$N_{60} = A e^{B(Ic)} + C(FGP) + D \qquad (16)$$

Performing a nonlinear analysis with two variables versus the standard penetration blow count N_{60} produces Eq. 17.

$$N_{60} = 2559 e^{0.002(Ic)} - 0.147(FGP) - 2539 \qquad (17)$$

The coefficient of determination (R^2) between $N_{60\ (measured)}$ and $N_{60\ (predicted)}$ from Eq. (17) is 0.73 (Fig. 9).

All of the empirical equations derived in this study are provided in Table 9. The high coefficient of regression in all of the equations is noteworthy. The equation derived from the input parameters activity and moisture content produces the highest determination coefficient.

Results and discussion

Soil mechanical properties are known to determine the standard penetration blow count number. The purpose of this study is to explain the relationship between the corrected SPT blow counts (N_{60}) and fine-grained size content, atterberg limits, activity, and consistency. For this purpose, data were obtained from foundation investigations of a sewerage station,

Fig. 8 Graph showing the relationship between N_{60} (measured) and N_{60} (predicted) with two variables: fine-grained percent and moisture content

Fig. 9 Graph showing the relationship between $N_{60 \text{ (measured)}}$ and $N_{60 \text{ (predicted)}}$ with two variables: consistency index and fine-grained percent
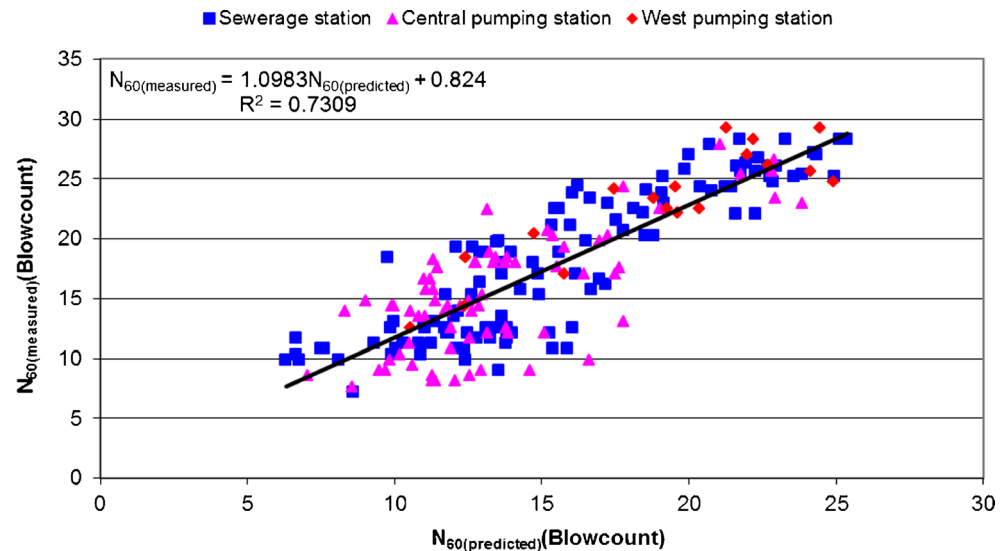
central pumping station, and west pumping station in Mersin City, Turkey. The foundation areas of three projects mainly consisted of clayey soils. A total of 20 boreholes in the sewerage station, 10 boreholes in the central station, and 4 boreholes in the west pumping station were drilled. There were 220 N_{60} values in the statistical correlation with the laboratory test results of the split spoon samples from the same standard penetration tests.

A series of simple regression analyses were conducted between N_{60} and the following parameters: liquid limit, plasticity limit, plasticity index, activity, and consistency index of soil samples. The determination coefficient of the simple regression analyses for N_{60}-LL, N_{60}-PL, and N_{60}-PI were considered unacceptable, so these soil index properties were not used as input data in subsequent regression analyses. The highest determination coefficient was found between N_{60} and activity of soils. The simple regression analysis between N_{60} and moisture conent, N_{60} and fine-grained percent content also resulted in high determination coefficients. The prediction performance of the simple regression equations were tested with RMSE indices and VAF values. All of the accepted empirical equations resulted in accurate prediction performances.

In a second step, multiple regression analyses were conducted. The moisture content and activity of clayey soils were used as the initial input parameters, the moisture content and fine-grained percent of clayey soil were used as the second input parameters, and the fine-grained percent content and consistency index of the clayey soils were used as the final input parameters. All of the derived empirical equations had high regression coefficients, especially the equation derived from the independent input parameters activity and moisture content. The prediction performances of the equations with two independent parameters derived from the multiple regression analyses were more accurate than the equations derived from simple regression analyses.

It is concluded that the performance of the empirical equations obtained in this study is satisfactory. However, all empirical models depend on the number and quality of the data employed. For this reason, before using the empirical equations developed in this study for design purposes, they must be checked. The amount of data limited in this study, so this study must be supported by additional studies. In order to be general equations, the empirical equations of this study must be supported by countless data.

Table 9 Emprical equations derived in this study

Dependent parameter	Independent parameter	Predicted N_{60} equation	R^2	n
N_{60}	Ac	$N_{60}=7.92\ln(Ac)+24.70$	0.62	220
N_{60}	W	$N_{60}=-0.52(w)+31.20$	0.56	220
N_{60}	FGP	$N_{60}=-0.21(FGP)+31.03$	0.52	220
N_{60}	Ic	$N_{60}=9.1166e^{0.537(Ic)}$	0.51	220
N_{60}	Ac,W	$N_{60}=5.55\ln(Ac)-0.32(w)+31.7$	0.78	220
N_{60}	W, FGP	$N_{60}=-0.344(w)-0.122(FGP)+34.879$	0.69	220
N_{60}	FGP, Ic	$N_{60}=2559e^{0.002(Ic)}-0.147(FGP)-2539$	0.73	220

References

Akca N (2003) Correlation of SPT-CPT data from the United Arab Emirates. Eng Geol 67:219–231

AlHomadhi Emad S (2014) New correlations of permeability and porosity versus confining pressure, cementation, and grain size and new quantitatively correlation relates permeability to porosity. Arab J Geosci 7:2871–2879. doi:10.1007/s12517-013-0928-z

Alvarez Grima M, Babuska R (1999) Fuzzy model for the prediction of unconfined compressive strength of rock samples. Int J Rock Mech Min Sci 36:339–349

ASTM (American society for testing and materials) (1994) Annual book of ASTM Standarts. Section 4, Construction, V. 0408 Soil and Rock; Building Stones. ASTM Publication, pp 978.

Ayolabi EA, Adegbolo RB (2014) Application of MASW in road failure investigation. Arab J Geosci 7:4335–4341

Bieniawski ZT (1978) Determining rock mass deformability: experience from case histories. Int J Rock Mech Min Sci Geomech Abstr 15: 237–247

Bowles JE, (1988) Foundation analysis and design. 4th edition. Mc Graw-Hill International Editions

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

British Standard 1377 (1975) Method of test for soils for civil engineering purposes. British Standard Institution, London

British Standart EN ISO 14688-2 (2013) Geotechnical investigation and testing. Identification and classification of soil. Principles for classification. British Standards Institution, London

Chang MF (1988) In-situ testing of residual soils in Singapore. Proceedings 2nd International Conference Geomechanics in Tropical Soils (Singapore) 1:97-108

Chatterjee K, Deepankar C (2013) Variations in shear wave velocity and soil site class in Kolkata city using regression and sensitivity analysis. Nat Hazards 69:2057–2082. doi:10.1007/s11069-013-0795-7

Chen-Chang L, Cheng-Haw L, Hsin-Fu Y, Hung-I L (2011) Modeling spatial fracture intensity as a control on flow in fractured rock. Environ Earth Sci 63:1199–1211

Coduto DP (1994) Foundation design principles and practices. Prentice Hall, Englwood Cliffs

Dagdelenler G, Sezer EA, Gokceoglu C (2011) Some non-linear models to predict the weathering degrees of a granitic rock from physical and mechanical parameters. Expert Syst Appl 38:7476–7485

Dikmen U (2009) Statistical correlations of shear wave velocity and penetration resistance for soils. J Geophys Eng 6:61–72

Emrem C, Durgunoglu HT (2000) Türkiye CPT very tabanı ve mevcut amprik bağıntılar ile karşılaştırma. Zemin Mekanigi ve Temel Mühendisliği Sekizinci Ulusal. Kongresi, İstanbul (in Turkish)

ENV (Eurocode 7) (1997) Geotechnical design-Part 3: Design assisted by field testing

Gibbs HJ, Holtz WG (1957) Research on determining the density of sands by spoon penetration testing. In Proceedings of the 4th International Conference on Soil Mechanics, London, 1:35–39

Gokceoglu C (2002) A fuzzy triangular chart to predict the uniaxial compressive strength of the Ankara agglomerates from their petrographic composition. Eng Geol 66(1–2):39–51

Hasancebi N, Ulusay R (2007) Empirical correlations between shear wave velocity and penetration resistance for ground shaking assessments. Bull Eng Geol Environ 66:203–213

Hoek E, Brown ET (1997) Practical estimates of rock mass strength. Int J Rock Mech Min Sci 34(8):1165–1186

Kayabaşı A (2012) Prediction of pressuremeter modulus and limit pressure of clayey soils by simple and nonlinear multiple regression techniques: a case study from Mersin, Turkey. Environ Earth Sci 66: 2171–2183. doi:10.1007/s 12665-011-1439-4

Kayabasi A, Gürsoy N (1995) Mersin municipality, Mersin sewerage project, sewerage station, geotechnical investigation report. EIEI Publication. No 95-4 (unpublished)

Kayabasi A, Gürsoy N (1995) Mersin municipality, Mersin sewerage project, west pumping station, geotechnical investigation report. EIEI Publication. No 95-43 (unpublished)

Kayabasi A, Gürsoy N (1995) Mersin municipality, Mersin Sewerage Project, Central Pumping Station, Geotechnical Investigation Report. EIEI Publication. No 95-44 (unpublished)

Kayabasi A, Gokceoglu C, Ercanoglu M (2003) Estimating the deformation modulus of rock masses: a comparative study. Int J Rock Mech Min Sci 40(1):55–63

Lacroix Yves, Horn Harry (1973) Direct determination and indirect evaluation of relative density and its use on earthwork construction projects: in evaluation of relative density and its role in geotechnical projects involving cohesionless soils: ASTM Special Technical Publication 523: 251-280

Lee SHH (1990) Regression models of shear wave velocities. J Chin Inst Eng 13:519–532

Liao SSC, Whitman RV (1986) Overburden correction factors for SPT in sand: Journal of Geotechnical Engineering. ASCE 112(3):373–377

Minaeian B, Ahangari K (2013) Estimation of uniaxial compressive strength based on P-wave and Schmidt hammer rebound using statistical method. Arab J Geosci 6:1925–1931. doi:10.1007/s12517-011-0460-y

Mitchell JK (1975) Fundamentals of soil behaviour. Wiley, Newyork, p 422

Peck RB, Hanson WE, Thornburn TH (1953) Foundation Engineering John Wiley Sons, Newyor, p 410

Schmertmann JH (1970) Static cone to compute static settlement over sand. ASCE J Soil Mech Found Div 96(3):1011–1043

Seed HB, Idriss IM (1981) Evaluation of liquefaction potentials and deposits based on observation of performance in previous earthquakes. Preprint 81–544, in situ testing to evaluate liquefaction susceptibility, ASCE National Convention, Missouri, pp 81–544

Seed HB, Woodward RJ, Lundgren R (1964) Fundamental aspects of the Atterberg limits. J Soil Mech Found Div ASCE 90(SM6):75–105

Skempton AW (1953) The colloidal activity of clays. In: 3rd International Conference on Soil Mechanics and Foundation Engineering, Switzerland 1: 57

Skempton AW (1986) Standard penetration test procedures and the effects in sands of overburden pressure, relative density, particle size, aging and over consolidation. Geotechnique 36(3):425–447

SPSS (2002) Statistical Package for the Social Sciences (v.11.5). SPSS Inc, Chicago

Stalin VK, Vurukan Arun BR (2014) Geographic information system for the development of soil suitability map in south Chennai, India. Arab J Geosci 4. Doi: 10.1007/s12517-014-1323-0

Terzaghi K, Peck RB (1948) Soil mechanics in engineering practice, 1st edn. John Wiley&Sons, New York, p 566

Topsakal E, Topal T (2014) Slope stability assessment of a re-activated landslide on the Artvin-Savsat junction of a provincial road in Meydancik, Turkey. Arab J Geosci 4. Doi: 10.1007/s12517-013-1210-0

Türk Standartları Enstitüsü (TSE) 5744 (1988) In situ measurement methods of the properties of foundation soils in civil engineering (in Türkish)

Ulugerli EU, Uyanık O (2007) Statistical correlations between seismic wave velocities and SPT blow counts and the relative density of soils. J Test Eval 35(2):1–5

USBR (United States Dep. Int. Bur. Reclamation) (1974) Earth Manual: a Water Resources Technical Publication. Denver, Colo, p 810

Yagiz S, Gokceoglu C (2010) Application of fuzzy inference system and nonlinear regression models for predicting rock brittleness. Expert Syst Appl 37:2265–2272

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 67:405–410