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Full Length Research Paper

Allowable bearing capacity based on Schmertmann method for sandy soils

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In this study, since the city of Bartin in Northwest Turkey on the Black Sea is in the first-degree seismic zone and its residential area only occupies 7% of the total acreage that is expected to expand with newly flourishing urbanization, soil samples were obtained from a total of five different locations where there are open areas for the construction of dwellings. Engineering properties of the soils were assessed by laboratory experiments and the allowable bearing capacity and elastic settlement (Schmertmann method) values of the soils were calculated. The results showed that calculated settlement values are very high and can damage the foundation systems of any building constructed in future; therefore, allowable settlement value was fixed at 50 mm and allowable bearing capacities of the soils were obtained from back calculations by using the Schmertmann method. The aim of calculating allowable bearing capacity modified by settlement analysis is to propose a procedure about the foundation designs of the building laying on compressible sandy soils.

Key words: Shallow foundations, allowable bearing capacity, silty sand, Schmertmann method, settlement.

INTRODUCTION

Since soils are created by many processes such as mechanical and chemical weathering, soil and rock depositions are heterogeneous, and soils often have properties which are not suitable for a proposed structure. Therefore, geotechnical investigation is the crucial process of gathering information about soil deposits and the influence of the construction or structural performance of a building project. The investigation is normally achieved by boring exploratory holes and carrying out soil and rock testing. In addition, it should consider all the information relevant to site usage, including meteorological, hydrological and environmental information. Geotechnical investigations are mostly conducted for obtaining bearing capacity of foundation soil beneath a building.

Bearing capacity is the ability of a soil to safely carry the pressure placed on the soil from any engineered structure without causing a shear failure with accompanying large settlements. Applying a bearing pressure, safe with respect to total failure, does not give a guarantee that settlement of the foundation will be within acceptable limits. Therefore, settlement analysis should generally be performed. Calculation of the bearing pressure required for ultimate shear failure is functional where sufficient geotechnical data from site investigation are unavailable to carry out a settlement analysis. Based on engineering experience and practice, an appropriate safety factor can be applied always to the calculated ultimate bearing pressure for obtaining allowable bearing pressure (Bowles, 1996; Salgado, 2008).

It should be noted that unless foundations are placed on hard rock, some measurable settlement will always occur, such as total and differential settlements, and distortions etc. In particular, if differential settlements become too large due to high bearing pressure, buildings will suffer damages, for example, tall buildings can tilt. Therefore, in designing foundations, two criteria should be considered and satisfied separately: Firstly, there must be an adequate factor of safety against a bearing capacity failure in the soil. Secondly, the settlements, and

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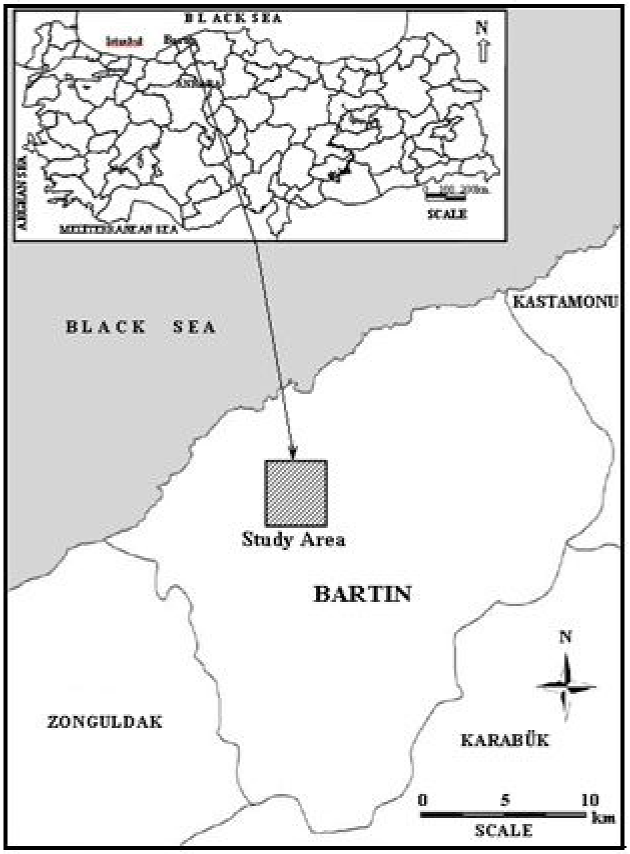

Figure 1. Location map of study area.

particularly the differential settlements must be kept within reasonable limits (Cernica, 1995; Bowles, 1996; Coduto, 2000; Das, 2004).

A great variety of methods have been developed to predict the settlement of shallow foundations on sandy soils. These methods range from purely empirical methods developed originally for conservative footing design to nonlinear finite element methods (Bowles, 1996, Das 1999). Assessments of the performance of various methods have generally been made on the basis of comparisons with measured settlements. There are two significant studies, (Jeyapalan and Boehm, 1986; Tan and Duncan, 1991) have been reported by Poulos, (2000). After examining 76 case studies, Poulos indicated that the method of Schmertmann (Schmertmann, 1970; Schmertmann et al., 1978) gave more dependable calculated settlement results for sandy soils containing fine-grained particles compared to other settlement methods which are Meyerhof (1965), Alpan, (1964), Terzaghi and Peck (1967), Peck and Bazaraa (1969), D'Appolonia and D'Appolonia (1970), Parry (1971), Schultz and Sherif (1973), Peck et al. (1974), Duncan and Buchignani (1976), NAVFAC (1982) and Burland and Burbidge (1985).

In this study, general characteristics of Bartin and the geology of the residential area where soil samples were collected were presented. All soil samples were subjected to some laboratory experiments and geotechnical properties of the soils were calculated. Allowable bearing capacities of the soil samples for different foundation types and sizes were determined based on the experimental results. In addition, the elastic settlement (Schmertmann Method) values of the soils were evaluated. The results showed that the calculated settlement values are very high and can damage the foundation systems; therefore, the allowable settlement value was fixed at 50 mm and the allowable bearing capacities of the soils were obtained from back calculations by using the Schmertmann Method.

MATERIALS AND METHODS

Geological properties of the study area

Bartin is located in the western part of the Black Sea region and its residential area only occupies 7% of the total acreage that is expected to expand with newly flourishing urbanization. Bartin, which is including the study area, is located between 41° 53' northern latitude and 32° 45' eastern longitude. Bartin has 59 km of coastline in the north coast. A site location map of the investigation area is given in Figure 1. The lithological units in the study area from bottom to top can be classified as Yemislicay, Akveren and Caycuma formations with alluvium (Figures 2 and 3) (Keskin et al., 2009).

Yemislicay formation is generally represented with a thin to medium layered volcanogenic sandstone, grayish green, thin to medium layered shale and sandstone intercalation, tuff, tuffite at lower layers, beige and a red-pinkish thin mid-layered pelagic and semi pelagic clayey limestone at medium layers and brown and dark gray agglomerates at upper layers. The age of the unit is Upper Cretaceous. Akveren formation is in the garb of white and beige, sometimes a red-pinkish, thin to medium layered (pelagic, semi pelagic) clayey limestone and grayish green shale intercalation at lower layers (GDMR, 2002). This layer comprises of turbid limestone, sandstone and shale intercalation. Towards upper layers it transforms into grayish green, in some parts pink, thin mid-layered shale with sandstone in mid-level, marl and sandstone. The unit is made up of sandy carbonate at the bottom, clayey limestone, mudstone, marl, turbidity and ebonite towards the top. The age of the unit is Upper Campanian–Lower Eocene. Caycuma formation comprises of sandstone with volcanic intermediate level, siltstone, claystone, shale intercalation. Sandstone is yellowish, with a light green, thin mid layer; in some convolute layered levels.

It is observed to have intermediate thick layers. The age of Caycuma formation is Lower-Middle Eocene. Quarternary aged alluvium formations are gravel, sand, mud sediments in plain areas that are shaped on streambeds, old grabens. Different materials like clay, alluvion, silt, sand, gravel and blocks are laid on streambeds depending on stream current (GDMR, 2002). There are small scale normal fault lines having net slip between 40 to 50 cm at claystone and sandy claystone levels in Caycuma formation (Jeotek, 2006). The study area is 132 km away at air distance from North Anatolia Fault. Most of the seismic activities that occurred during the last half a century in North Anatolia are related with the North Anatolia Fault and this fault is described as an active strike-slip and right directed. Bartin city center is in a First Degree Seismic Zone (Earthquake Research Dept. 2008).

The geological formations in the study area are divided into two groups for settlement. The first group is all of the lithological units

THEM	System			Series	Formation	Unit	Thickness(m)	Symbol	Lithology	Explanation
Cenozoic	Quaternary							Qal		Alluvium Sand, gravel
Cenozoic	Tertiary	Paleogene	Eocene	Middle Lower	Çaycuma	Kaynarca Unit	350	Teç, Teçk	Teçk	Teç: Sandstone, shale, conglomerate Teçk: Limestone, marl
Cenozoic	Tertiary	Paleogene	Paleocene		Akveren	Çangaza Volcanic Unit	350-400	KTa, KTaç	KTaç	KTa: Semi pelagic limestone, shale, sandstone, conglomerate KTaç: Basalt, andesite
Mesozoic	Cretaceous			Upper	Yemişliçay	Kapanboğazı Unit	100–200	Ky, Kyk	Kyk	Ky: Sandstone, tuff, agglomerate, andesite, basalt Kyk: Pelagic-semi pelagic limestone

Figure 2. Stratigraphic columnar section of the study area (not to scale) (GDMR 2002).

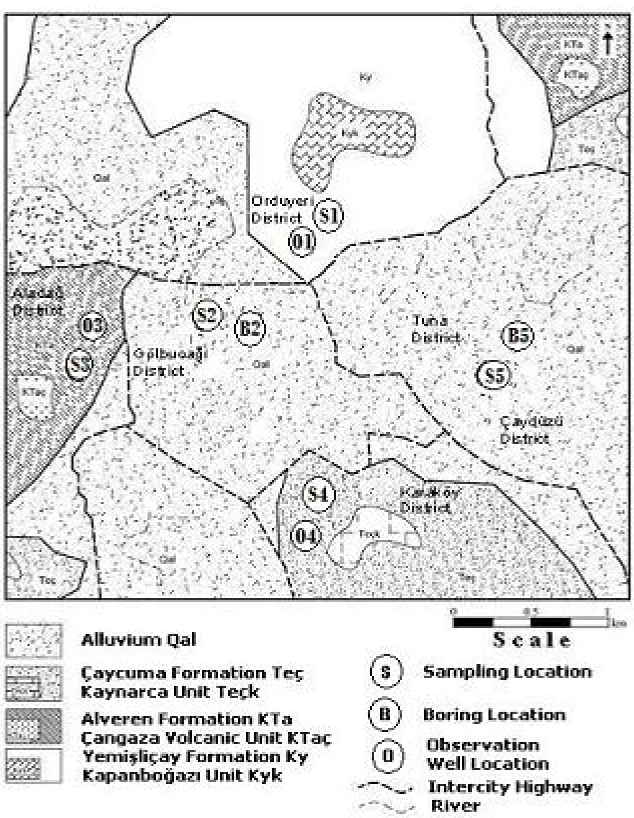

Figure 3. The geological map (GDMR 2002) and sampling location of study area (Keskin et al., 2009).

except for alluvium. The second group is alluvium that covers huge areas in the study area and its surrounding area. The lithological units are also divided into two groups because Akveren formation is generally known to provide a firm foundation because of its high bearing capacity but due to their clayish structure they are prone to mass movements and they have problems such as swelling and heaving. In the units other than this formation, there is no natural instability or mass movement. However, Alluvium is made up of loose gravel, sand, silt and clay (Tuysuz et al., 2001). It has low bearing capacity and high settlement potential.

Sampling and in-situ studies

Soil samples were taken from Yemislicay (S1), Akveren (S3), Caycuma (S4) formations and alluvium (S2 and S5) according to TS 1900 Standard to represent the soil characteristics of the study area. In addition, some geotechnical investigation reports prepared for the study areas containing observation wells and boreholes were obtained from the General Directorate of Utilities and Construction Office at Bartin Municipality. Locations of observation wells (O1, O3, and O4) and boreholes (B2 and B5) and samples locations (S1 to S5) on a geology map are marked (Figure 3). The observation wells in formations of Yemislicay (O1), Akveren (O3) and Caycuma (O4) are to a depth of 6 m. Boreholes (B2 and B5) are in alluvium to a depth of 10 m (Figure 4).

The following observations were made in Figure 4: First, all areas which contain sample locations comprise mainly of sedimentary media consisting of sand, silty sand, clayey sand and gravely sand. Second, alteration gradually decreases for Yemislicay formation (Figure 4a) roughly at 3.0 m, for Akveren (Figure 4b) and Caycuma (Figure 4c) formations at about 4.0 m. Third, alluvium formations (Figure 4d and 4e) consisting of clay, clayey sand, poorly graded sand and gravel and also, roughly from 3.0 m, intercalations with

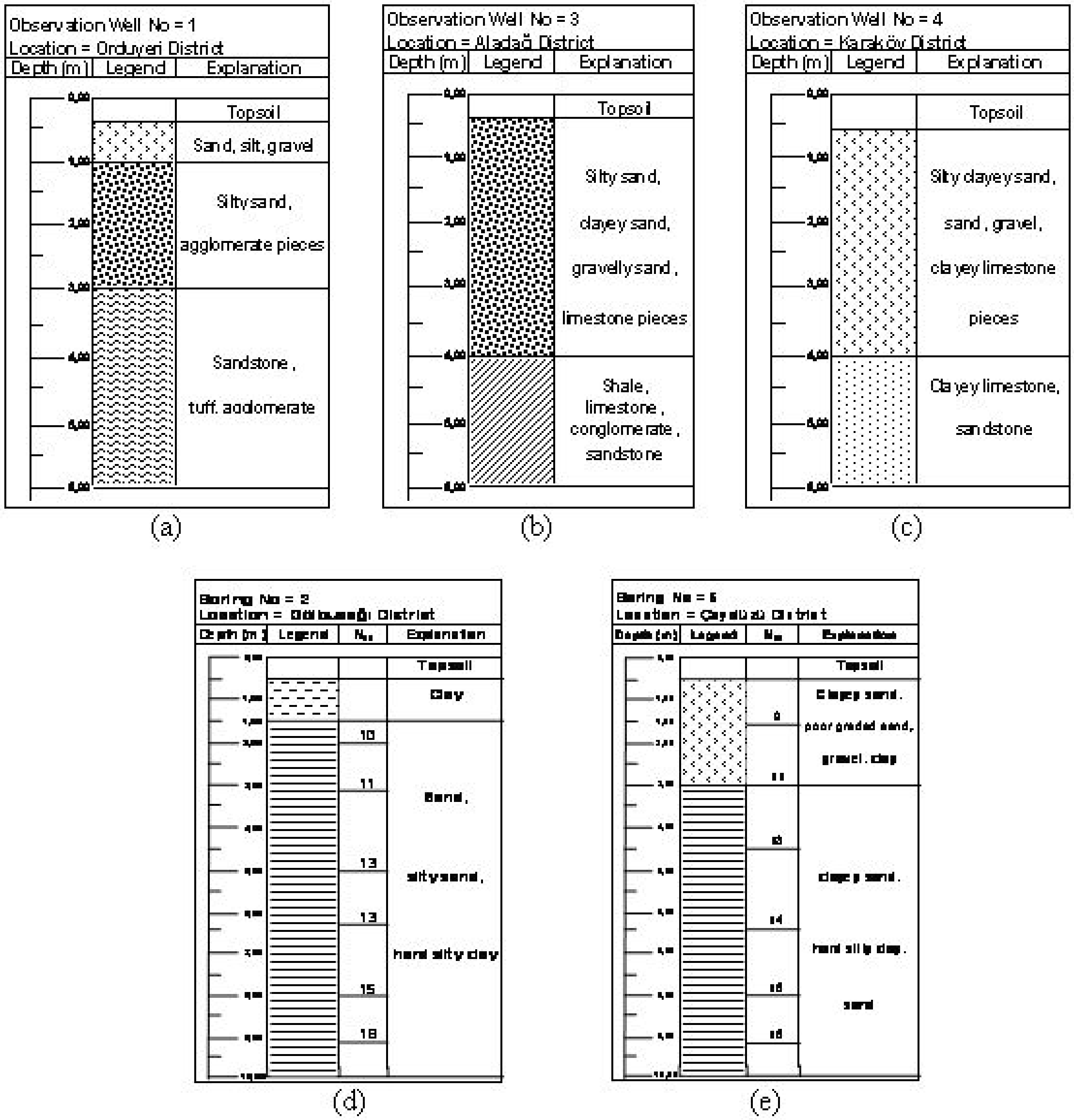

Figure 4. Geological profile at the (a) Yemislicay formation, (b) Akveren formation, (c) Caycuma formation, (d) and (e) alluvium.

soil layers such as sand and silty sand are detected. Consequently, it is confirmed that soil layers are denser with depth.

Experimental studies

The physical properties of soil samples were obtained at Zonguldak Karaelmas University, Geological Engineering Laboratory. All soil samples are sand and silty sand. According to USCS, soil samples fall into well graded silty sand (SW to SM) to poorly graded silty sand (SP to SM).

Engineering properties of soil samples were obtained in the soil mechanics laboratory of the Civil Engineering Department at Middle East Technical University, Ankara Turkey. The engineering properties such as cohesion (c'), internal friction angle (ϕ'), compressibility (m_v) and elastic modulus (E_s) are evaluated by using direct shear and consolidation tests. The tests were conducted according to Turkish Standards (Turkish Standards (TS 1900-1, 2006 and 1900-2, 2006).

Soil specimens were reconstituted in the laboratory because of their low cohesions and granular structures considering their natural water contents and unit weights. Obtained three individually samples from each soil formation were used for direct shear tests conducted under 100, 150, and 200 kPa normal pressures. The cohesion (c') and internal friction angles (ϕ') which are dislocation endurance parameters of samples were investigated. Failure envelopes for soil specimens from direct shear test data are plotted in Figure 5. Later, one-dimensional consolidation (odometer) tests were conducted on soil samples.

In consolidation experiments, samples were loaded under 25, 50, 100, 200, 400, 800, and 1600 kPa and the test results are plotted in Figure 6 and then the compressibility (m_v) parameters and elastic modulus of soil specimens (E_s=1/m_v) obtained in terms of consolidation test results were determined as seen in Figures 7 and

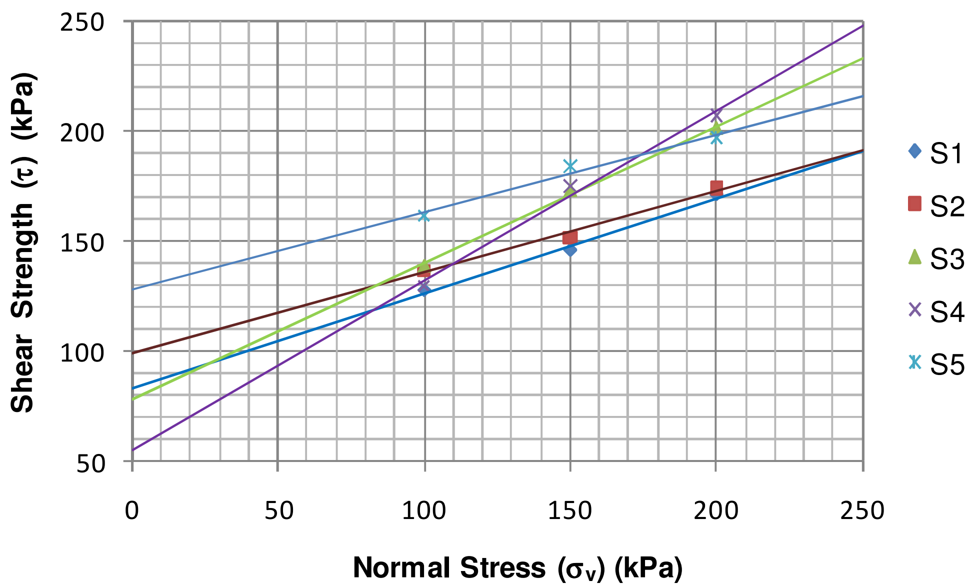

Figure 5. Failure envelopes for soil specimens from direct shear test data.

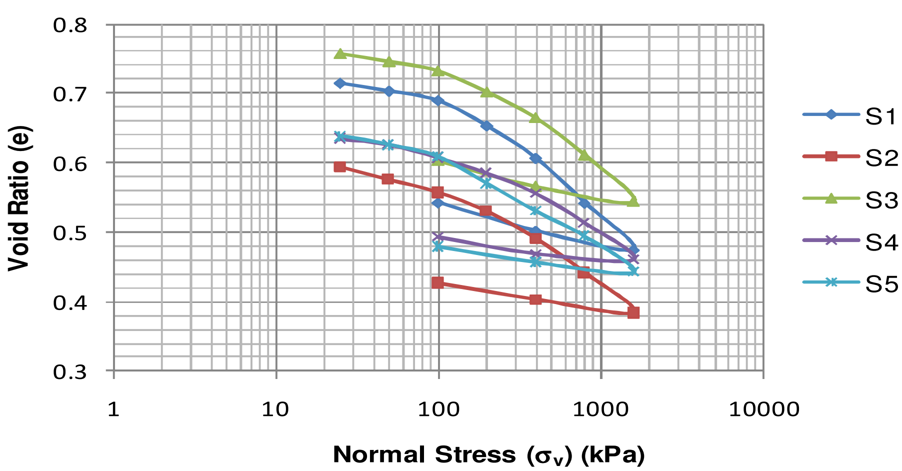

Figure 6. Consolidation test results on soil specimens.

8. Since the foundation depth (D_f) is chosen as 1.0 m, the effective vertical stress level just underneath the foundation system is about 18 to 20 kPa. Therefore, the values of compressibility and elastic modulus were chosen from 0 to 25 kPa stress levels from Figures 7 and 8, respectively. Taking previous studies into consideration, Poisson ratios (μ) of soil samples are assumed as 0.3 for silty sand soils (Bardet, 1997; Das, 1999 and 2004). All physical and engineering parameters are summarized in Table 1.

Bearing capacity

An ultimate bearing capacity (q_u) of a shallow foundation system is given in Equation (1) (Vesic, 1973):

$$q_u = c' N_c F_{cs} F_{cd} F_{cc} + q N_q F_{qs} F_{qd} F_{qc} + \frac{1}{2}\gamma B N_\gamma F_{\gamma s} F_{\gamma d} F_{\gamma c} \quad (1)$$

N_c, N_q, N_γ are bearing strength coefficients; F_{cs}, F_{qs}, $F_{\gamma s}$ are shape factors; F_{cd}, F_{qd}, $F_{\gamma d}$ are depth factors; F_{cc}, F_{qc}, $F_{\gamma c}$ are compressibility factors; q (γ D_f) is surcharge load; c' is the cohesion of foundation soil; γ is the unit volume weight of the soil beneath the foundation (Vesic, 1973; Bowles, 1996; Cernica, 1995; Das, 1999 and 2004; Coduto, 2000; Salgado, 2008).

In this study, shallow foundation types classified as shallow foundations; square (B=L), rectangle (L>B) and continuous type (L>>B) shallow foundation systems were considered. B and L are foundation width and length, respectively. Building codes suggested that the width of the footing should not be less than 1.0 m if it is made of reinforced-concrete and it is not economical and causes internal stability problems if it is larger than 3.0 m. Therefore, B values were chosen at least 1.0 and the most 3.0 m. In addition, to investigate the effects of magnitude of B on bearing capacity and settlement values of the soil deposits, B values were assigned as increasing starting from 1.0 with 0.25 m increments till 3.0 m. Furthermore, based on B values, rectangle (B/L=0.8, 0.6, and 0.5)

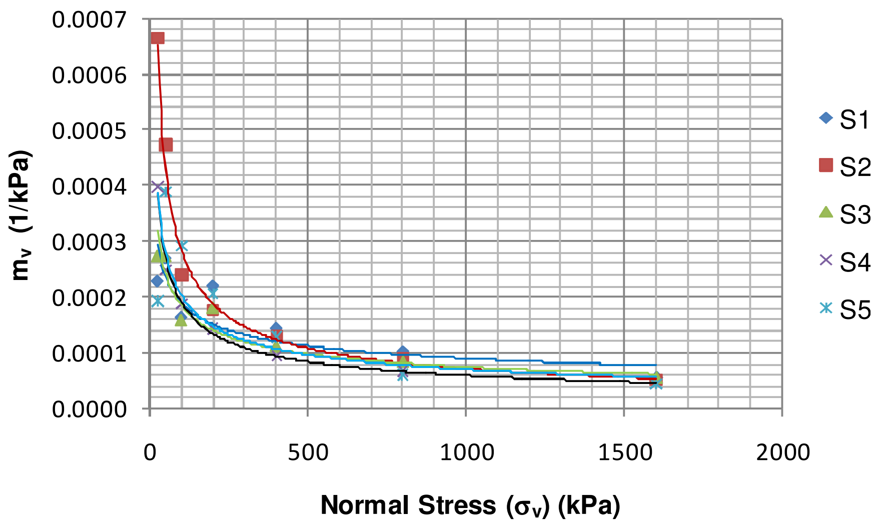

Figure 7. Variation of the compressibility of the specimens with stress level and loading direction.

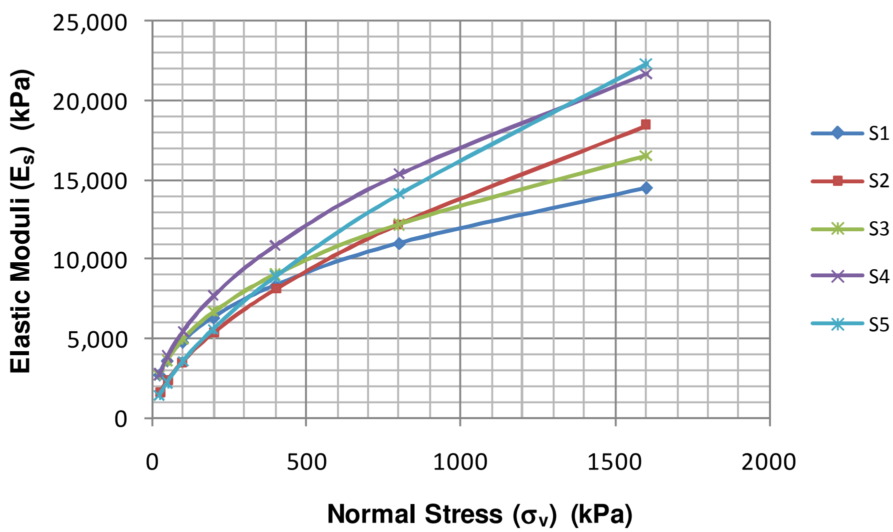

Figure 8. Variation of elastic modulus obtained from the consolidation tests results.

Table 1. Index and strength parameters of research soil deposits.

Sample No.	Formation	USCS	W_n (%)	γ_d (kN/m^3)	γ_n (kN/m^3)	c'(kPa)	ϕ'(°)	m_v m^2/kN	E≈1/m_vkPa	μ
S1	Yemislicay	SP-SM	21.0	16.26	19.81	85	23	2.68E-04	3730.04	0.3
S2	Alluvium	SW-SM	18.0	16.50	19.46	99	20	4.69E-04	2132.61	0.3
S3	Akveren	SP-SM	20.2	16.31	19.62	78	32	2.74E-04	3646.84	0.3
S4	Caycuma	SP-SM	26.5	14.88	18.82	56	38	2.49E-04	4020.49	0.3
S5	Alluvium	SP-SM	21.9	15.70	19.15	128	19	3.88E-04	2574.80	0.3

and continuous (B/L=0) foundation systems were chosen for the bearing capacity and settlement analyses.

The depth of the foundation systems (D_f) was selected as 1.0 m in terms of frost action. The city of Bartin does not have very cold weather but still experiences frost action during winter time. Because of this, foundation system should be constructed under frost depth. 1.0 m as a foundation depth from the ground surface is enough magnitude to protect the foundation systems from frost

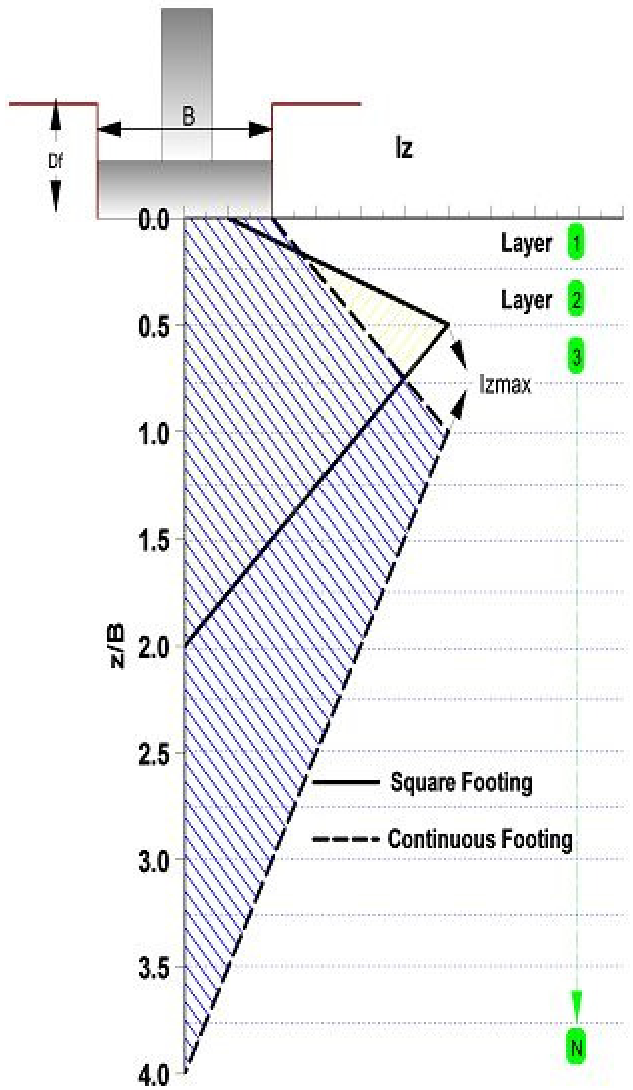

Figure 9. Distribution of strain influence factor with depth under different types of footings.

action. For a given foundation to perform at its optimum capacity, one must ensure that the load per unit area of the foundation does not exceed a limiting value, thereby causing shear failure in soil. This limiting value is the ultimate bearing capacity q_u. Considering the ultimate bearing capacity and the uncertainties involved in evaluating the shear strength parameters of the soil, the allowable bearing capacity q_a can be obtained by using a chosen factor of safety (G_s) as follows:

$$q_a = \frac{q_u - q}{G_s} \tag{2}$$

Previous studies and foundation design codes also indicated that the magnitude of ultimate bearing capacity should be reduced by a factor of safety which is generally chosen between three and four. This factor was chosen as 4.0 due to not only Bartin being located in an earthquake zone but also considering the uncertainties of engineering behavior of the research soil deposits. However, based on limiting settlement conditions, an allowable bearing capacity for a foundation q_{all} can be given by Equation 3.

$$q_a \xrightarrow{\text{SETTLEMENT ANALYIS}} q_{all} \tag{3}$$

Settlement analysis

Generally the settlement of foundations may be regarded as consisting of three separate components which are immediate settlement (δ_e) resulting from the constant volume, consolidation settlement (δ_c) resulting from water discharge with time from loaded area under the influence of the load and last one's secondary settlement or creep (δ_s) which is also time dependent may occur ear essentially constant effective stress. In this study, all soil samples investigated are sandy soils and previous studies showed that elastic settlement due to increment of vertical stress from constructions are expected (Bowles, 1996; Coduto, 2000). As discussed before, the Schmertmann method is mostly used for elastic settlement analysis of sandy soils. This method was originally suggested by Schmertmann (1970) and further was improved by Schmertmann et al. (1978). The Schmertmann method was developed primarily for spread footings, so the various empirical data used to calibrate the method have been developed with this type of foundation in mind.

The method suggested that the greatest strains do not occur immediately below the footing, as one might expect, but at a depth of 0.5 B to B below the bottom of the footing, where B is the footing width. This distribution is described by the strain influence factor, I_z which is a type of weighting factor. The distribution of I_z, with depth has been idealized as two straight lines, as shown in Figure 9 (Schmertmann, 1978; Vlught and Rosenthal, 1989; Poulos and Mayne, 1999; Lee et al., 2008). The peak value of the strain influence factor I_{zmax} is calculated as follows:

$$I_{z_{max}} = 0.5 + 0.1\sqrt{\frac{q_a - \sigma'_{zD_f}}{\sigma'_{z_{max}}}} \tag{4}$$

Where;

$I_{z_{max}}$: Maximum peak strain influence factor.

q_a : Bearing pressure.

σ'_{zD_f} : Soil vertical effective stress at a depth D_f below the ground surface.

$\sigma'_{z_{max}}$: Maximum vertical effective stress at depth of peak strain influence factor.

The exact value of I_z at any given depth may be computed using the following equations (Coduto 2000):

Square footings;

$$z = 0 \sim \frac{B}{2} \rightarrow I_z = 0.1 + \left(\frac{z}{B}\right)\left(2I_{z_{max}} - 0.2\right) \tag{5}$$

$$z = \frac{B}{2} \sim 2B \rightarrow I_z = 0.667\, I_{z_{max}}\left(2 - \frac{z}{B}\right) \tag{6}$$

Continuous footings; $\left(\frac{L}{B}\right) \geq 10$;

$$z = 0 \sim B \rightarrow I_z = 0.2 + \left(\frac{z}{B}\right)\left(I_{z_{\max}} - 0.2\right) \quad (7)$$

$$z = B - 4B \rightarrow I_z = 0.333 I_{z_{\max}} \left(4 - \frac{z}{B}\right) \quad (8)$$

Rectangular footings, $\left(1 \langle \frac{\mathrm{L}}{\mathrm{B}} \langle 10\right)$

$$I_z = I_{zs} + 0.111\left(I_{zc} - I_{zs}\right)\left(\frac{L}{B} - 1\right) \quad (9)$$

Where;

z : Depth from bottom of foundation to midpoint of the layer.

I_z : Strain influence factor.

I_{zc} : I_z for a continuous foundation.

I_{zs} : For square footing I_z > 0.

The procedure for computing I_z beneath rectangular foundations requires computation of I_ε for each layer using the equations for square foundations (based on the I_z) and I_z for each layer using the equations for continuous foundations (based on I_{zmax}), then combining them using Equation 9. Schmertmann method also includes empirical corrections for the depth of embedment (C_1), secondary creep (C_2) in the soil, and footing shape (C_3) then Schmertmann`s equation can be given as below:

$$\delta = C_1 \times C_2 \times C_3 \left(q_a - \sigma'_{zD_f}\right) \sum_{i=1}^{N} \frac{I_{z_i H_i}}{E_{s_i}} \quad (10)$$

$$C_1 = 1 - 0.5 \left(\frac{\sigma'_{zD_f}}{q - \sigma'_{zD_f}}\right) \quad (11)$$

$$C_2 = 1 + 0.2 \log\left(\frac{t}{0.1}\right) \quad (12)$$

$$C_3 = 1.03 - 0.03 \frac{L}{B} \geq 0.73 \quad (13)$$

Where H and E_s are thickness and equivalent modulus of elasticity of soil layers respectively and *t* is time-passed after the pressure applied on the soils layers. It should be used in year units.

In this study, considering soil layers are getting denser with depth, all research soils are divided into individual layers having 20 cm thickness as seen in Figure 9. All reconstituted soil specimens were subjected to a one dimensional consolidation test and their compressibility (m_v) values were obtained. Actually, the consolidation test is used for fine grained soils such as silty and clayey soils. The reason for using this test for sandy soils is to obtain the elastic modulus of the samples indirectly for each soil layer. Previous studies suggested that the constrain modulus (M) is $1/m_v$ and can be practically assumed as the elastic modulus (E_s) (Bowles, 1996; Salgado, 2008). The consolidation test also allows one to find out compressibility values at different vertical effective stresses (σ'). When increasing the vertical effective stress on soil specimens are consolidated more and become denser materials. In this case, higher effective stress for consolidation gives denser soil material or higher elastic modulus values. Equation 12, secondary creep (C_2), corresponds to time effect on the settlement values.

In this study, elastic settlement values are evaluated for 1, 5, 10, 20, 30, 40 and 50 years. The reason of using different time values for settlement analysis in C_2 investigates time effect on the settlement analysis. In addition, a foundation engineer should pick the time duration of a building based on its service life time and important category. For example, if a building was planned for temporary use then it would be acceptable to calculate using one year in the C_2 factor or if the building were used for long time and had high priority such as a hospital or an industrial plant, fifty years should be picked to put into C_2 formulation. For these reasons, different time intervals were used.

RESULTS

In this study, ultimate bearing capacities of each soil deposit for various shallow foundation systems were primarily calculated. Elastic settlement values were obtained due to q_a assumed to be the net allowable load per unit area of the foundation systems. The allowable bearing capacities (q_{all}) of sandy soils were derived based on settlement. Details and results of the research conducted are given step by step below.

According to the results of direct shear tests, all soil specimens have considerable cohesion (c') since all soil samples contain about 10% to 12% finers (particle size smaller than 0.075 mm). Another result from direct shear tests, the internal friction angles (ϕ') of the samples vary between 19° and 32°. By using all other soil parameters, the ultimate and allowable bearing capacity values of the deposits for various shallow foundation systems were calculated by using Equations 1 and 2. Due to the considerable calculation steps in Equations 1 for 5 different soil layers underneath a total of 40 different sizes and dimensions of foundation systems which are different 8 square, 24 rectangle and 8 continuous footings, an excel-macro programming technique was coded to obtain the bearing capacities. All results are summarized in Table 2 and Figures 10, 11 and 12.

Observing Table 2 and Figure 10a, the highest and the lowest allowable bearing capacities of S1 soil were calculated to be 559.04 (B/L= 1 and B=1 m) kPa and 228.20 (continuous foundation, B=3 m) kPa, respectively. For S2 soil sample, the highest and lowest allowable bearing capacity values are calculated to be 380.70 (B/L ratio is 1 and B=1 m) kPa and 147.31 (continuous foundation, B=3 m) (Table 2 and Figure 10b). For S3 soil, the highest and lowest allowable bearing capacity values were determined as 764.0 (B/L ratio is 1 and B=1 m) and 279.94 (continuous foundation, B=2.5 m) kPa (Table 2 and Figure 11a), respectively. When the values for S4 soil are examined, the highest allowable value is found as 882.19 kPa, B=1 m and B/L ratio =1 square foundation.

Table 2. Results of allowable bearing capacity and settlement analyses for research soil deposits.

Sample	Foundation Type	B (m)	B/L (m)	q_a (kPa)	□ (mm) 1 year	□ (mm) 5 years	□ (mm) 10 years	□ (mm) 20 years	□ (mm) 30 years	□ (mm) 40 years	□ (mm) 50 years	q_{all} (kPa) 1 year	q_{all} (kPa) 5 years	q_{all} (kPa) 10 years	q_{all} (kPa) 20 years	q_{all} (kPa) 30 years	q_{all} (kPa) 40 years	q_{all} (kPa) 50 years
1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19
	Square		1	559-452	190-312	212-348	221-364	231-380	236-389	240-395	244-400	201-113	186-105	180-102	174-100	171-98	169-97	168-96
			0,8	488-398	171-279	183-302	189-314	196-326	200-333	203-338	205-342	193-111	182-104	177-101	172-99	170-97	168-96	166-96
S1	Rectangle	1-3	0,6	424-349	157-253	159-264	163-271	167-280	169-285	171-289	173-292	182-107	177-102	173-100	170-98	168-97	166-96	165-95
			0,5	394-326	153-244	150-248	151-253	154-260	156-264	157-267	158-271	174-104	173-101	171-99	168-97	166-96	165-95	164-94
	Continuous		0	265-228	91-156	101-174	106-182	111-190	113-194	115-198	117-200	169-99	156-92	151-90	146-87	144-86	142-85	141-84
	Square		1	381-307	193-289	216-322	225-337	235-351	241-360	245-366	248-370	139-86	128-81	125-79	121-77	119-76	118-75	117-75
			0,8	330-268	170-253	184-276	190-287	197-298	201-304	204-309	207-313	134-85	126-80	123-78	120-76	118-76	117-75	116-74
S2	Rectangle	1-3	0,6	285-233	153-223	157-236	161-243	165-251	168-256	170-259	172-262	128-83	124-79	121-78	119-76	117-75	116-74	115-74
			0,5	264-217	146-212	146-219	148-224	151-231	153-235	155-238	156-240	124-81	122-78	120-77	118-76	116-75	115-74	115-74
	Continuous		0	173-147	82-124	92-138	96-144	100-150	102-154	104-157	105-159	121-80	112-75	109-73	106-71	104-70	103-69	102-69
	Square		1	764-624	289-466	322-520	337-544	351-567	360-581	366-591	371-598	199-113	183-105	177-102	172-99	169-98	167-97	165-96
			0,8	649-538	252-408	269-441	278-458	288-475	294-485	298-493	302-498	190-111	180-104	175-101	170-99	168-97	165-96	164-96
S3	Rectangle	1-3	0,6	547-461	224-362	226-375	230-385	235-396	239-403	241-409	244-413	179-107	174-102	172-100	168-98	166-96	164-96	163-95
			0,5	501-426	215-345	208-348	209-354	213-362	215-368	217-372	219-375	172-104	171-101	169-99	166-97	164-96	163-95	162-94
	Continuous		0	309-280	111-200	124-223	130-233	136-243	139-249	141-253	143-256	168-100	155-93	150-90	145-88	143-87	141-86	140-85
	Square		1	882-733	345-548	385-611	403-639	420-666	430-682	437-694	443-703	199-116	184-107	178-104	172-101	169-100	167-99	166-98
			0,8	734-624	293-471	313-509	323-528	335-548	341-560	346-568	350-575	192-113	180-106	176-104	171-101	168-99	166-98	164-97
S4	Rectangle	1-3	0,6	606-529	253-412	255-427	259-438	265-450	269-458	272-464	275-469	180-109	176-105	172-102	169-100	166-98	165-98	163-97
			0,5	549-486	239-423	231-392	233-399	236-408	239-415	241-419	243-423	174-107	172-103	170-101	167-99	165-98	164-97	163-96
	Continuous		0	318-303	113-214	126-239	139-250	137-260	140-267	143-271	144-275	170-104	157-96	152-93	147-91	145-89	143-88	142-88
	Square		1	447-360	261-375	292-419	305-438	318-457	326-468	331-475	336-481	129-83	120-78	116-76	113-74	111-73	110-72	109-72
			0,8	387-314	230-328	248-358	257-372	266-386	272-395	276-401	279-406	125-82	118-77	115-75	112-74	111-73	109-72	108-72
S5	Rectangle	1-3	0,6	333-271	206-287	211-303	216-312	222-323	226-329	229-334	231-337	120-80	116-76	113-75	111-73	110-72	109-72	108-71
			0,5	308-252	197-272	196-281	199-288	203-296	206-301	208-305	210-308	116-78	114-76	112-74	110-73	109-72	108-71	108-71
	Continuous		0	200-168	107-153	120-171	125-178	131-186	134-191	136-194	138-196	115-78	106-73	103-71	100-69	99-68	98-68	97-67

The lowest value is found as 302.79 kPa for B=1.75 in continuous foundation (Table 2 and Figure 11b). For S5 soil sample, the highest and lowest allowable bearing capacity values are calculated as 446.53 (B/L ratio 1 and B=1 m) kPa and 168.28 (continuous foundation, B=3 m) kPa

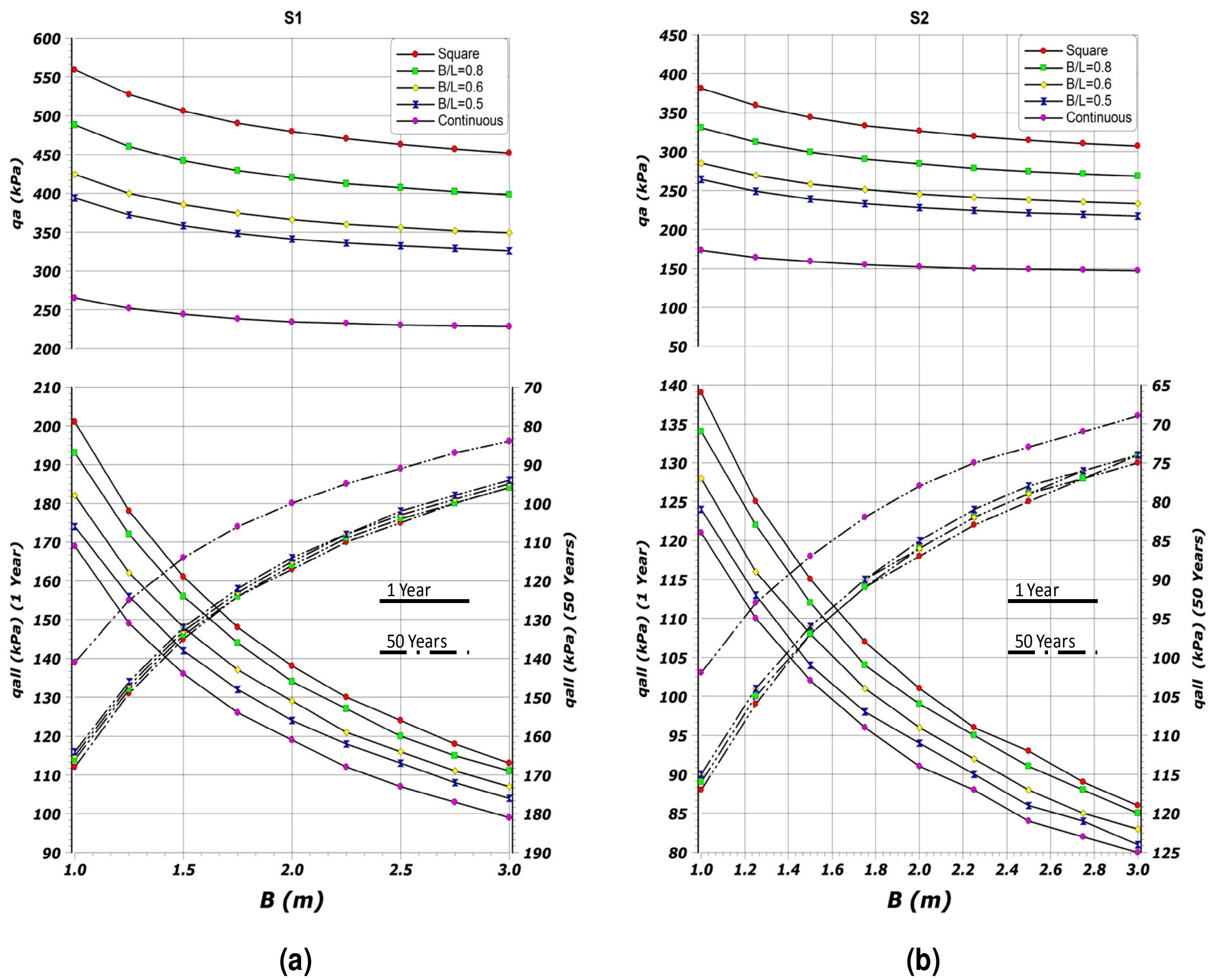

Figure 10. Values of q_a and q_{all} for different B and foundation systems (a) Yemislicay Formation (b) Alluvium.

(Table 2 and Figure 12).

The results of elastic settlements analysis are also presented in Table 2. When calculating elastic settlement values with Schmertmann method, first the results of q_a (Table 2, Column 5) were put into Equation 10 then elastic settlement values were evaluated and given in Table 2 (Column 6 through 12) for 1, 5, 10, 20, 30, 40 and 50 years. All soil deposits were assumed multi-layered with 20 cm thickness. As seen in Table 2, the highest and lowest settlement values of S1 soil for one year time interval were calculated to be 312 (B/L= 1 and B=3 m) mm and 91 (continuous foundation, B=1 m) mm, respectively. Although the highest bearing pressure (q_a = 559 kPa, B/L= 1& B=1 m) are assumed to cause the highest settlement (δ=312 mm), the highest settlement is occurred is occurred at q_a=452 kPa (B=3 for square footing). According to Schmertmann method, larger B values give a higher stress influence depth causing as plotted in Figure 9. Previous studies also proved that the amount of settlement is proportional to the influence depth.

All q_a values computed in Equation 2 were reduced to cause 50 mm total settlement for each foundation system to obtain q_{all}. An excel macro program was coded for these computations and the Newton-Raphson method was applied to solve the back calculation of allowable bearing capacity (q_{all}) modified by settlement analysis. All results of q_{all} are summarized in Table 2 (Column 13 through 19) for 1, 5, 10, 20, 30, 40 and 50 years.

DISCUSSION

Lithologically, Akveren (S3) formation is generally known to provide a firm foundation because of its high bearing capacity but due to its clayish structure, it is prone to mass movements and it has problems such as swelling and heaving. In the units other than this formation, no natural instability or mass movement is observed. Namely, the other formations (S1 and S4) should have higher bearing capacity values compared to Akveren formation. Although Caycuma (S4) formation gave the

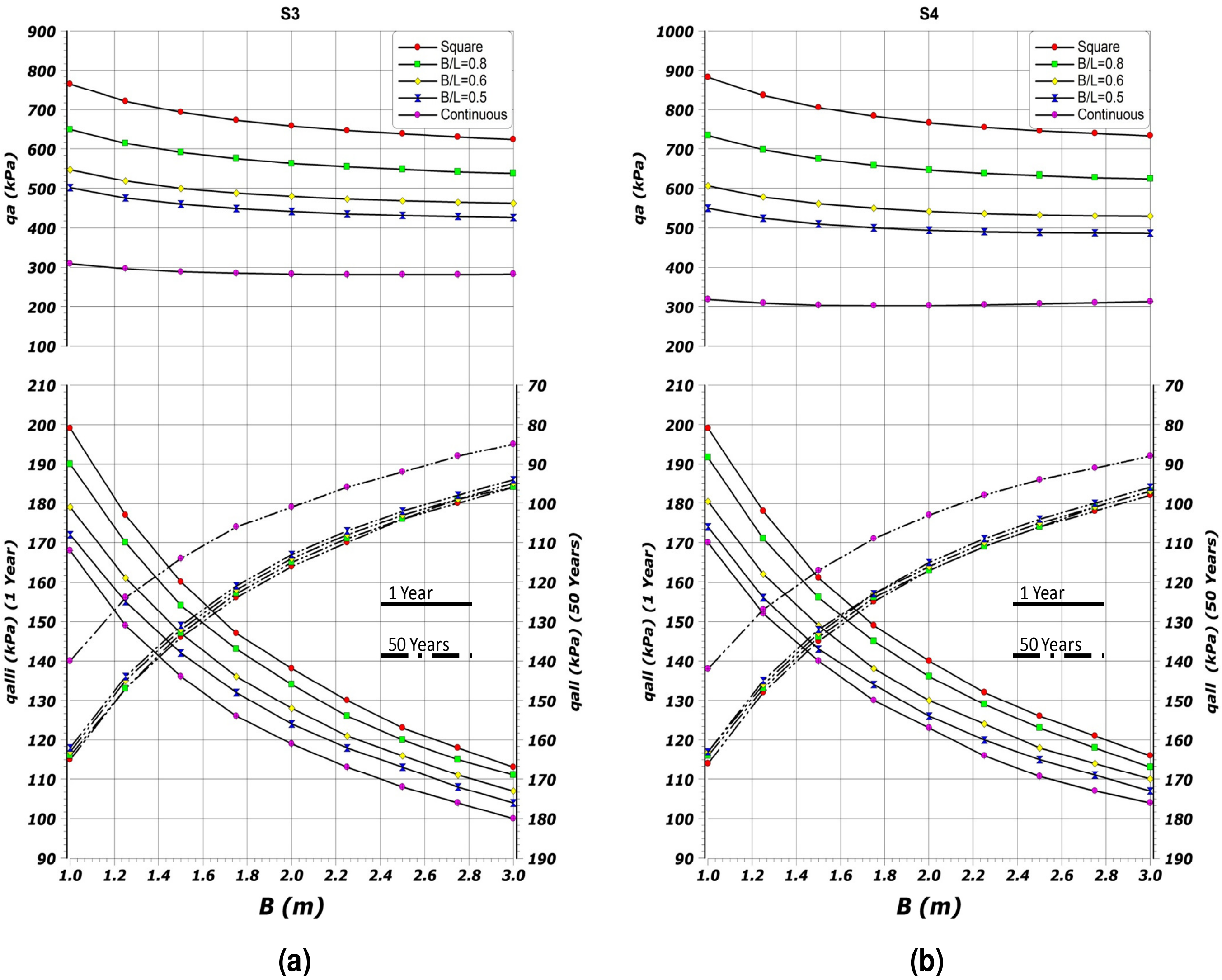

Figure 11. Values of q_a and q_{all} for different B and foundation systems (a) Akveren Formation (b) Caycuma Formation.

highest bearing capacities as given Table 2, Akveren formation has the second highest values among the other formations. The reason for this is apparently the experimental results of direct shear and consolidation tests in Table 1. All formations have almost same elastic modulus but big differences in internal friction angles (ϕ'). N_c, N_q, N_γ, bearing strength coefficients, are functions of ϕ' and they increase considerably with the increment of ϕ'.

Alluvium (S2 and S5) is made up of loose gravel, sand, silt and clay. It has low bearing capacity and high settlement potential. The results of bearing capacity and settlement analysis for alluvium confirmed it as given in Table 2. S1 and S5 soil samples have the lowest values of E_s and ϕ' compare to the values of all formations in Table 1. The aforementioned maximum allowable settlement for sandy soils for commercial buildings should be limited to 50 mm (EN 1997-1 2004; Frank et al., 2004; Bond and Harris, 2008). It can be concluded that all settlement values are higher than 50 mm in Table 2 and these values inevitably cause big deformation and stability problems for the building lay on the soil deposit. In this case, the foundation-soil can tolerate q_a values with high amount of settlement but the construction of the building is forced considerable structural-deformation. This is not acceptable and q_a values must be reduced till settlement analysis results obtain 50 mm settlement value. All formations (S1, S3, and S4) have different values of q_a for square footings (B=L=1.0 m) such as 559, 764, and 882 kN/m^2, respectively. These allowable bearing pressures correspond to 190, 289, and 345 mm settlement (Table 2). When q_a values of the formations for the same square footing are reduced to cause 50 mm total settlement to obtain q_{all}, these values interestingly change to 201, 199 and 199 kN/m^2, respectively. This indicates that Elastic

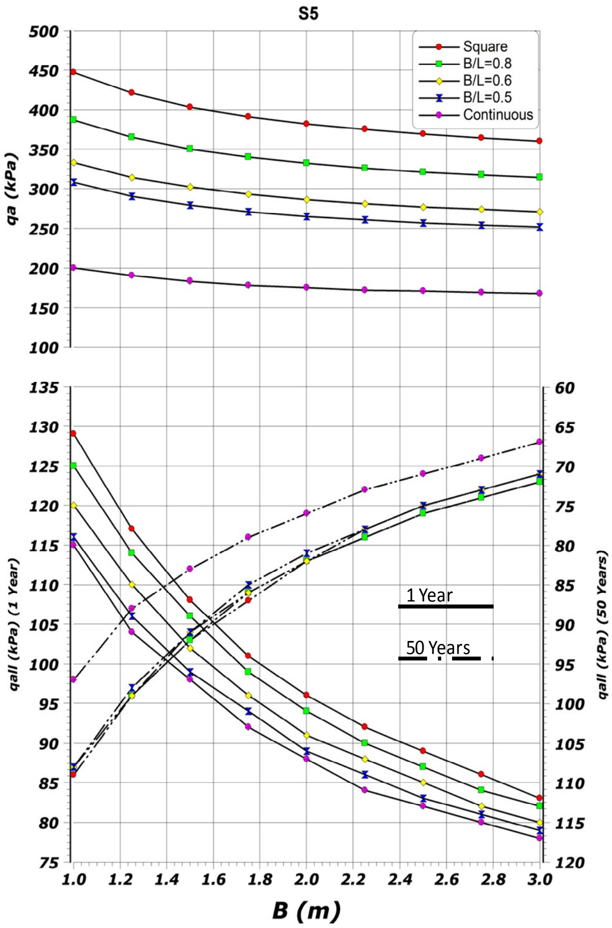

Figure 12. Values of q_a and q_{all} for different B and foundation systems in Alluvium.

Modulus (E_s) is the key factor of the settlement analysis because S1, S3 and S4 formations have almost the same E_s values as seen in Table 1.

As seen in Figures 10, 11 and 12, all q_a values of the formations are plotted first and then only q_{all} modified with 50 mm settlement for 1 and 50 years are plotted. The settlement values of all foundation systems for one year time interval are about 77% of the total settlement values for 50 years after construction. These results indicate that the other time intervals such as 10, 20, 30 years etc. did not give significant differences between settlement values.

Conclusions

The following conclusions may be drawn based on the results of this study:

i) Since the variability of the soil conditions cause different behavior, detailed geotechnical investigations are very important for an accurate prediction of foundation settlement. Selection of the most effective geotechnical investigation method dealing with the inevitable uncertainties of soil deposits depends on geotechnical background and experience. There is always some doubt whether the borings accurately present the subsurface conditions. Therefore, engineers attempt to compensate for these uncertainties by applying factors of safety in our analyses. Additionally, construction has to be safe and economical. Unfortunately, this solution also increases construction costs. The incremental cost of additional investigation and testing does not produce an equal or larger reduction in construction costs.

ii) The aim of calculating allowable bearing capacity values modified by the settlement analysis based on time factors for different foundation types and B values is to propose information about the foundation designs of any building to be constructed in the future. After choosing the convenient foundation system for the building, engineering companies which make building designs in the area, can obtain allowable bearing capacity from Table 2 depending on the soil type. In addition, if the material parameters from the experimental results are different from Table 1, they must comprise these two tables conveniently.

iii) According to geotechnical and foundation investigation, regulations approve that the allowable bearing capacity value must be unique for foundation soils. In practical applications, allowable bearing capacity is calculated for B value as 1 m, and this value is thought as the allowable bearing capacity value for that soil. But as seen in the calculations, when value B increases, the bearing capacity value decreases and the magnitude of settlement values increases. The main reasons of this are shape, depth, compressibility factors in Equation 1 and the influence strain depth factor in Equation 10. Consequently, if the compressibility potential of the soil is high, allowable bearing capacity values must be recalculated considering settlement analysis.

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