Extracting some soil parameters and estimating elastic settlements from Direct shear box data for a granular C- Ø soil

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Abstract

Direct shear box tests were carried out on some soil samples obtained from some locations around Uyo metropolis in Akwa-Ibom State, Southeastern Nigeria. The dominant geology is the Coastal Plain Sands. From the test results, the C-Ø properties of the soils were determined. These values were used to compute ultimate and allowable bearing capacities for the soil samples. Elastic, and shear modulus parameters were computed for the different shear values readings for each vertical loading scheme. Poisson ratio was also computed for the same loading scheme applying the elastic relationship between shear modulus and elastic modulus. These parameters were used to compute elastic settlement using the classical elastic theory equation. Two other modified equations of the elastic theory equation and a consolidation equation was also used to compute immediate and consolidation settlements respectively. The values of immediate settlement obtained for the classical elastic equation is less than those computed by the other two modified equations. This is attributed to the fact that the other two equations assumed a constant value of Poisson ratio for the different imposed loads for which computations were made. For the allowable load of 138.54 kPa the classical elastic theory gave an immediate settlement value of 1.02 cm; the other two modified forms of the theory gave values of 2.94 cm and 4.0 cm. Elastic modulus and Poisson ratio obtained from the Direct shear test for the load are 5662.86 kPa and 0.49. These values were used in the elastic equation. The same value of 5662.86 kPa was used in the other two modified elastic formulas, but Poisson ratio values of 0.15 and 0.2 were assumed for all loads. Direct shear box test provides a convenient, but plausible means of determining parameters required to compute immediate settlement for the type of soil investigated.

Keywords Direct shear box · Elastic modulus · Shear modulus · Poisson ratio · Elastic settlement

1 Introduction

The elastic settlement beneath a flexible footing placed on the ground surface is computed as

$$S_e = \frac{qB(1 - \nu^2)I_f}{E} \quad (1)$$

where q = is the surcharge load (kPa), B = is the width of the footing (m), ν = Poisson ratio, I_f = influence factor, which is a function of ratio of length to width (L/B) and 'H' which is the thickness of the compressible layer which is often taken at not more than four times the footing width.

In Eq. (1) above, determining the appropriate values of Poisson ratio and the Elastic modulus is often a difficult problem. For the elastic modulus, two methods of determining is value are

1. Laboratory methods
2. Field methods.

For purely cohesionless soil, due to the problem of obtaining undisturbed samples for laboratory use, the laboratory method is not often used. For cohesive soil, triaxial test is

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often used with some modifications to take account of consolidation state and sensitivity in case of sensitive soil.

For both cohesive and cohesionless soils the field methods have been found to be reliable. These include,

- i. Plate load test (PLT), Standard Penetration Test (SPT) for example [1, 2]
- ii. Static Cone Penetration Test (CPT), for example [3]
- iii. Pressuremeter Tests (PMT) [4, 5]
- iv. Flat Plate Dilatometer test (DMT, [6, 7]).
- v. Geophysical methods (Seismic refraction, spectral analysis of surface waves (SASW) [8], multichannel analysis of surface waves (MASW) [9].

The geophysical method of determination of elastic modulus and Poisson ratio represents small strain methods compared to most laboratory methods. A common laboratory method that is used in determining these parameters is by the use of the Triaxial machine in the Triaxial test. The Triaxial test can be used to determine Poisson ratio for cohesive soils, but not for cohesionless soil because of the difficulty in obtaining undisturbed sample. The Direct shear apparatus as well as the Triaxial device have undergone such a level of refinement, and are now digitalized which allows measurements to a good level of precision that small stress and strain parameters can be measured reliably and accurately. In light of this, this write-up describes how to estimate elastic modulus, Poisson ratio, and computation of elastic settlements from Direct shear box data on C-Ø soil from Coastal Plain Sands obtained from some locations around Uyo, Akwa Ibom State, Nigeria.

2 Study objectives

The study aims to:

1. Determine the values of cohesion and angle of shearing resistance of soil samples collected.
2. Estimate the Ultimate and allowable bearing resistance of the soil samples,
3. Evaluate elastic modulus, Poisson ratio, shear modulus and other parameters,
4. Estimate elastic settlement using the parameters obtained above.

3 Materials and method

3.1 Soil Index tests

Block soil samples were collected using the trial pit method from some construction sites where single, two

and three storey buildings are being constructed. The block samples were taken at the depth where the foundations of the buildings were placed. From the block samples disturbed and undisturbed tests samples were obtained for soil classification and Direct shear tests. Soil classification tests include:- moisture content, sieve analysis, plastic limit, and liquid limit. These tests were carried out in accordance with relevant American Society of Testing and Materials (ASTM) standards.

3.2 Direct shear tests

The direct shear tests were primarily used to determine the C, Ø and property of the soils. The digital direct shear machine was model 30-WF6016 T2 (Wykeham Franchise) attached to a digital data logger which records all parameters automatically during a testing operation and the test were performed in accordance with [10]. The tests were strain controlled. The machine applied strain at the rate of 0.5 mm per minute. The Direct shear test machine is equipped with strain gauges transducers to measure both horizontal and vertical deformations from the start to the end of the test. It has facility to determine both the peak and residual shear strength of soil, but the latter was not determined for the specimens in this study. Normal load applied to the soil samples were 500 N, 1000 N, and 1500 N. Porous plates were placed on top and bottom of the soil sample to allow drainage from soil.

4 Data analysis

4.1 Shear box data

The Direct shear test device acquires principally three parameters, which are horizontal displacement, vertical displacement and shear force values at regular time during the test. The rest of the parameters are computed. The peak shear and, peak normal stress method [11] were used to determine C- Ø property of each sample. For each applied vertical load, a number of computations are made. Computations were carried out with the aid of an excel worksheet. This includes: vertical stress, shear stress, vertical strain, shear strain, 'constrained' modulus, shear modulus, and then Poisson ratio. The latter is computed by applying the relationship

$$E = 2G(1 + \nu) \quad (2)$$

By imposing the relation given by (2) on the data, it is being assumed that both horizontal and vertical deformation are elastic. Therefore a Poisson ratio values of -0.5 – 0.5 obtained will validate the elastic assumption; whereas

other values will indicate non-linear and non-elastic situation in the materials

4.2 Bearing capacity estimates

Modified Terzaghi bearing capacity equation is used in estimating bearing capacity. The equation is of the form;

$$Q_u = 0.67c\bar{N}_c \left(1 + 0.3\left(\frac{B}{L}\right)\right) + D_f\gamma\bar{N}_q + \frac{1}{2}\gamma B\bar{N}_\gamma \left(1 - 0.2\left(\frac{B}{L}\right)\right) \quad (3)$$

4.3 Computations of immediate settlements and consolidation settlement

In computing immediate settlements, two approaches are used. The first is by using Eq. (1). The second is by [12]. They proposed a method to estimate the elastic settlement which takes into account the variation of the modulus of elasticity of soil with the strain level. This method is also described by [13]. According to the method, elastic settlement is computed using the equation

$$\delta_{\text{footing}} = I_s \frac{q_{\text{net}}}{E} B \quad (4)$$

where δ_{footing} = elastic settlement, I_s = Influence factor for a rigid footing, which depends on the ratio L/B obtained from table given by [14]. In the Table, Poisson ratio is taken to 0.15, and $I_s = 0.6$ for the L/B ratio adopted in this study. E = Modulus of elasticity of the soil.

Consolidation settlements were estimated by using the equation

$$S = m_v \sigma H \quad (5)$$

where m_v = is the inverse of one dimensional constrained modulus (E). σ = is the imposed load on the soil, which can be safe, allowable or ultimate load that is significantly effective up to the thickness 'H'; H = is the thickness of compressible stratum where imposed stress value is 0.2σ to 0.3σ or $0.8B$ to $1.8B$ [15]. Since 'B' used in this study is 2.0 m, 'H' translates to 1.2 m to 3.6 m. $H = 4.0$ m was adopted for computations

In using Eq. (1) to compute elastic settlements the factor I_f is defined by the equation

$$I_f = \left[F_1 + \frac{(1 - \nu - 2\nu^2)F_2}{(1 - \nu^2)} \right] \quad (6)$$

where F_1 and F_2 are values from [16] curve and also take on values depending on Poisson ratio values.

If Poisson ratio is 0.5, $I_f = F_1$

If Poisson ratio is Zero $I_f = F_1 + F_2$

If Poisson ratio is between 0 and 0.5, Eq. (6) is used to calculate I_f

Equations (1), (4) and (5) were used to estimate settlements in this study

5 Results and discussion

5.1 Soil indices and types

Table 1 presents some soil indices for the soil samples investigated. Figure 1 presents grain size analysis curves for all the soil samples. The maximum amount passing sieve no 200 for all the samples is 39.82%; making all soil classify as coarse grained soil. The soils are medium to fine grained texture. From Table 1, the soil samples have liquid limit values from 24 to 36%, plastic limit from 13 to 24%, and plasticity index from 8 to 13%. Except for sample 'D' all the soil samples classify as clayey sand (SC), while sample 'D' classifies as clayey sand and silty sand soil (SC-SM). All classification is based on the Unified Soil Classification System (USCS). Compression index (C_c) based on [17] gives values from 0.13 to 0.24

5.2 C- ϕ properties and bearing capacities of samples

Values of cohesion and angle of internal friction determined from Direct shear box ranges from 18.35 to 52.65 kN/m² for cohesion, and 23.1° to 30.74° for angle of internal friction. Typical Direct shear box data is presented in Fig. 2 for samples 'D' and 'E'; while Figs. 3 and 4 presents shear stress values versus horizontal deformation for samples A and B respectively.

Due to the numerical values of C and ϕ , the Ultimate bearing capacities values for the soils were computed the using modified Terzaghi bearing capacity equation (Eq. 3 above). Ultimate bearing capacity values ranged from 415.61 to 688.93 kN/m². Allowable bearing capacity was computed assuming a factor of safety of 3, and values ranges from 139.39 to 229.64 kN/m²

5.3 Computations of elastic modulus, shear modulus and Poisson ratio

For each vertical load sequence, a range of values for vertical strain, shear strain, elastic modulus, shear modulus, and Poisson ratio are calculated. Typical values of these parameters are presented in Tables 2, 3, 4, 5 and 6, for each of the samples whose parameters are employed in settlement computations.

For parameters computed, the early part of the data, usually the first and sometimes the second sets of values are affected by sampling disturbance, and, therefore, may

Table 1 Indices and classification of soil samples

Sample identification	Sampling depth (m)	Natural moisture Content (%)	Liquid Limit (%)	Plastic Limit (%)	Plasticity Index (%)	Sieve sizes in (mm)	Percentage passing sieve no							Unified Soil Classification System (USCS)	Compression index, C_c^a
							7	14	25	36	52	100	200		
						Liquidity Index									
Sample A	1.2	17.18	26	13	13	0.32	95.11	90.59	78.34	67.51	49.10	24.90	20.62	SC	0.14
Sample B	1.4	11.27	26.13	13.7	12.43	-0.20	99.93	97.17	81.61	72.35	61.53	45.51	39.82	SC	0.15
Sample C	1.4	16	36	24	12	-0.67	100.00	99.17	89.48	78.00	57.49	37.54	33.29	SC	0.23
Sample D	1.4	27.02	24	16	8	1.38	100.00	98.87	85.83	72.32	50.26	32.23	27.11	SC-SM	0.13
Sample E	1.4	20.91	27	18	9	0.32	100.00	98.52	83.43	72.60	58.55	42.52	37.03	SC	0.15

^aCompression index by Tezarghi and Peck [17]

results in parameters estimated in that part of data to be distorted.

Elastic and shear modulus generally decrease with increasing vertical stress. The elastic modulus values are reduced due to the shear stress to which the sample is simultaneously subjected to during the shear tests. Values of elastic modulus obtained, therefore, are conservative. Both parameters are also strain dependent. Typical elastic modulus values used in the computations are from 1484.99 to 6910.45 kPa.

Poulos [18] reports that Young's modulus; E can be obtained from the following equations that relates constrained modulus ' D ' with shear modulus ' G '. The equations are $E = 3G/(1 + b.G/D)$, where $b = (1 - \nu)/(1 + \nu)$. These equations were also used to compute E in this study, but there is no significant difference between the values obtained and that obtained directly from the shear box test parameters. For example, for samples 'A' B, and C, the values of E obtained by these equations are 1466.54 kPa, 5844.87 kPa, and 1989.75 kPa respectively whereas the E computed from the shear box test data are 1484.99 kPa, 4112.02 kPa, and 2039.41 kPa respectively for the same samples. Furthermore, [19], has stated that for Poisson ratio of zero, constrained modulus is equal to Young's modulus, and that from the works of [20, 21] which is about triaxial testing involving small strains, Poisson ratio takes on values from 0.1 to 0.2 at usual working loads such that constrained modulus values takes on 1.05' E ' to 1.11' E '. Also the modulus derived in the shear box test is not fully constrained; it can be said to be 'partially constrained'. Based on the preceding considerations, the modulus originally estimated from Direct shear box is retained for all computations.

Poisson ratio takes on (if the early part of data are ignored) values that ranges from -0.94 to 4.25, indicating elastic linear, and nonlinear non-elastic deformation states. In compliance with the elastic theory, Poisson values that indicate linear and elastic values are utilized in the computations of settlement; that is values from -0.5 to +0.5.

5.4 Choice of allowable bearing resistance values from direct shear box data

Parameters associated with values of vertical or normal stress close to the values of allowable bearing resistance computed were adopted in the computation of settlement. The parameters include elastic modulus, and Poisson ratio. The imposed stress, q used was the computed allowable resistance. The values are from 138.54 to 229.64 kPa. The vertical stress developed during the Direct shear test may not necessarily have these values, hence the choice of the values closest to the computed allowable

Fig. 1 Sieve analysis results showing grain size distribution of soil samples used in the study

Fig. 2 Shear stress versus normal stress plot for soil samples 'D' and 'E'

bearing resistance. Vertical stress values after those that give peak shear stress were not selected for computation.

In Foundation engineering practice, it is usual to adopt a factor of safety of three to the ultimate bearing capacity value to obtain the allowable bearing stress. The underlying assumption is that this places the load

in the linear elastic region of load—deformation curve. The Poisson ratio values of the allowable bearing stress obtained in this study is between 0.13 and 0.53. This places the deformation in the soil in the elastic linear range thus asserting the assumption

Fig. 3 Shear stress versus horizontal deformation curves for sample 'A'

Fig. 4 Shear stress versus horizontal deformation curves for sample 'B'

5.5 Elastic settlement values

Three different equations, listed as Eqs. (1), (4), and (5) above were used in computing settlement. Table 7

presents values of computed settlement and the parameters used.

Equation (1) gives values of settlement from 1.02 to 6.78 cm, while Eq. (4) gives settlement values from 2.94 to

Table 2 Direct shear box test results for sample 'A' with normal stress of 138 kPa

Reading No.	Time (mins)	Horizontal displacement q_h (mm)	Vertical displacement q_v (mm)	Shear force T (N)	Corrected area (Ac) (m ²)	Vertical stress (δ_v) (kPa)	Shear stress τ (kPa)	T/ δ_v	Shear strain	Vertical strain	Elastic modulus (kPa)	Shear modulus (kPa)	Poisson ratio -calculated
1	00.00.00.00	0.001	0	0	0.00360	138.89	0.00	0.000	0.00002	0.00000	#DIV/0!	0.00	
2	00.00.00.15	0.001	1.144	0.7	0.00360	138.89	0.19	0.001	0.00002	0.05720	2428.17	11666.86	-0.896
3	00.00.01.00	0.237	1.175	3.2	0.00359	139.44	0.89	0.006	0.00395	0.05875	2373.44	225.93	4.253
4	00.00.02.15	0.533	1.307	104	0.00357	140.13	29.15	0.208	0.00888	0.06535	2144.36	3281.18	-0.673
5	00.00.04.00	1.373	1.547	158.6	0.00352	142.14	45.09	0.317	0.02288	0.07735	1837.64	1970.31	-0.534
6	00.00.06.15	2.461	1.672	175.4	0.00345	144.83	50.81	0.351	0.04102	0.08360	1732.41	1238.67	-0.301
7	00.00.09.00	3.809	1.776	208.3	0.00337	148.30	61.78	0.417	0.06348	0.08880	1670.09	973.22	-0.142
8 ^a	00.00.12.15	5.385	2.055	250.5	0.00328	152.58	76.44	0.501	0.08975	0.10275	1484.99	851.75	-0.128
9	00.00.16.00	7.193	2.286	286.8	0.00317	157.81	90.52	0.574	0.11988	0.11430	1380.64	755.05	-0.086
10	00.00.20.15	9.225	2.482	337.4	0.00305	164.12	110.75	0.675	0.15375	0.12410	1322.50	720.33	-0.082
11	00.00.25.00	11.521	2.59	358.1	0.00291	171.90	123.11	0.716	0.19202	0.12950	1327.38	641.15	0.035
12	00.00.30.15	14.085	2.658	363.6	0.00275	181.49	131.98	0.727	0.23475	0.13290	1365.65	562.23	0.214
13	00.00.34.44	16.243	2.706	335.1	0.00263	190.45	127.64	0.670	0.27072	0.13530	1407.58	471.48	0.493

^aRow whose parameters were used in settlement computation

12.65 cm, values that are slightly more than twice those given by Eq. (1). Equation (4) assumes Poisson ratio of 0.15, whereas Eq. (1) uses values of Poisson ratio existing in the soil at the stress state which is more realistic. The magnitude of settlement values are not dependent only on the magnitude of the ultimate bearing capacity or the allowable values. Sample 'D' has the largest ultimate bearing capacity of 688.93 kN/m² and the allowable, yet it has the second least settlement values of 1.31 cm, and 3.98 cm based on Eqs. (1), and (4) respectively. Furthermore, for all the settlement computations, settlement values are smaller with increasing Poisson ratio.

5.6 Consolidation settlement values

Equation (5) is usually used to compute consolidation settlement. The compressibility values (m_v) in the equation is taken to be approximately equal to the inverse of the elastic modulus. From Table 7, its values ranges from 0.14 to 0.67 per MPa, indicating 'medium' to 'high' compressibility based on [22] compressibility classification which is presented in Table 8. Consolidation settlement values are about 42 cm for the soil samples 'A' and 'C' which has high compressibility values of 0.67 and 0.49 respectively; whereas, samples 'B', 'D' and 'E' with m_v values 0.14–0.24, which represents medium compressibility values have settlement values from 9.76 to 21.45 cm.

5.7 Peak shear strength and computed allowable bearing capacity

The allowable bearing capacity adopted from shear box tests are within the first and second sequence of normal load applied. The shear stress versus horizontal deformation plot (Figs. 3 and 4 being examples) indicate the allowable stress is developed in the soil when the soil consistency is 'loose' to 'medium dense'. Usually the first and second load sequence in this study and the shape of the associated curves indicate 'loose' and 'medium dense' consistency ([23, 24]). The curves of 138 kPa and that of 277 kPa indicate for sandy soils, 'loose' and 'medium dense' respectively. The values indicated by the peak shear strengths for each of the relevant curves are reasonably close to the estimated allowable bearing stress. Samples 'A', 'B', 'C', 'D' and 'E' have computed allowable bearing stress values of 156.58 kPa, 220.51 kPa, 214.92 kPa, 229.64 kPa, and 138.54 kPa respectively. The peak shear strengths for these samples from the plot of shear stress versus horizontal displacement curves respectively are 131.98 kPa, 219.42 kPa, 232.29 kPa, 257.4 kPa, and 129.29 kPa. Sample 'D' has the largest difference of 27.6 kPa between the computed value and peak shear strength read from the plot, followed by Sample, 'A' with a difference in value of 24.6 kPa.

Table 3 Direct shear box test results for sample 'B' for normal load 277 kPa

Reading No.	Time (s)	Horizontal displacement q _h (mm)	Vertical displacement q _v (mm)	Shear force T (N)	Corrected area (Ac) (m ²)	Vertical stress (δ _v) (kPa)	Shear stress τ (kPa)	T/N	Shear strain	Vertical strain	Elastic modulus (kPa)	Shear modulus (kPa)	Poisson ratio -calculated
1	00.00.00.00	0.005	0	0.2	0.0036	277.80	0.06	0.000	0.0001	0.0000	#DIV/0!	666.72	
2	00.00.00.15	0.008	1.182	4.2	0.0036	277.81	1.17	0.004	0.0001	0.0591	4700.76	8751.17	-0.731
3	00.00.01.00	0.002	1.249	3.4	0.0036	277.79	0.94	0.003	0.0000	0.0625	4448.15	28334.28	-0.922
4 ^a	00.00.02.15	0.916	1.372	200.3	0.0035	282.08	56.50	0.200	0.0153	0.0686	4112.02	3700.97	-0.444
5	00.00.04.00	1.667	1.563	320.9	0.0035	285.72	91.69	0.321	0.0278	0.0782	3655.99	3300.04	-0.446
6	00.00.06.15	2.715	1.796	431.4	0.0034	290.94	125.51	0.431	0.0453	0.0898	3239.90	2773.76	-0.416
7	00.00.09.00	3.943	2.029	526.6	0.0034	297.32	156.57	0.527	0.0657	0.1015	2930.67	2382.45	-0.385
8	00.00.12.15	5.481	2.239	604.7	0.0033	305.70	184.86	0.605	0.0914	0.1120	2730.72	2023.64	-0.325
9	00.00.16.00	7.221	2.39	656.4	0.0032	315.78	207.28	0.656	0.1204	0.1195	2642.53	1722.31	-0.233
10	00.00.20.15	9.313	2.464	663	0.0030	328.82	218.00	0.663	0.1552	0.1232	2668.96	1404.52	-0.050
11	00.00.25.00	11.616	2.506	645.2	0.0029	344.47	222.25	0.645	0.1936	0.1253	2749.13	1147.98	0.197
12	00.00.30.15	14.159	2.603	638.9	0.0028	363.58	232.29	0.639	0.2360	0.1302	2793.51	984.34	0.419
13	00.00.31.33	14.628	2.612	601.6	0.0027	367.33	220.99	0.602	0.2438	0.1306	2812.66	906.43	0.552

^aRow whose parameters were used in settlement computation**Table 4** Direct shear box test results for sample 'C' for normal load 277 kPa

Reading No.	Time (mins)	Horizontal displacement q _h (mm)	Vertical displacement q _v (mm)	Shear force T(N)	Corrected area (Ac) (m ²)	Vertical stress (δ _v) (kPa)	Shear stress, T, (kPa)	T/N	Shear strain	Vertical strain	Elastic modulus (kPa)	Shear modulus (kPa)	Poisson ratio -calculated
1	00.00.00.00	-0.019	0.529	-1.2	0.00360	277.69	-0.33	-0.001	-0.00032	0.02645	10498.67	-231408.20	-1.023
2	00.00.00.15	-0.075	2.465	-1.9	0.00360	277.43	-0.53	-0.002	-0.00125	0.12325	2250.96	-146016.31	-1.008
3	00.00.01.00	0.004	2.554	1.5	0.00360	277.80	0.42	0.002	0.00007	0.12770	2175.38	185197.53	-0.994
4 ^a	00.00.02.15	0.435	2.744	226.2	0.00357	279.81	63.29	0.226	0.00725	0.13720	2039.41	1236.99	-0.176
5	00.00.04.00	1.259	2.887	245.5	0.00352	283.73	69.66	0.246	0.02098	0.14435	1965.58	1155.73	-0.150
6	00.00.06.15	2.309	3.047	323.7	0.00346	288.90	93.52	0.324	0.03848	0.15235	1896.26	892.48	0.062
7	00.00.09.00	3.609	3.448	464.2	0.00338	295.56	137.20	0.464	0.06015	0.17240	1714.36	636.70	0.346
8	00.00.12.15	5.207	3.557	495.5	0.00329	304.18	150.72	0.496	0.08678	0.17785	1710.29	613.88	0.393
9	00.00.16.00	7.031	3.571	579.6	0.00318	314.65	182.37	0.580	0.11718	0.17855	1762.25	542.87	0.623
10	00.00.20.15	9.16	3.575	635.9	0.00305	327.83	208.46	0.636	0.15267	0.17875	1833.99	515.53	0.779
11	00.00.25.00	11.509	3.585	638.4	0.00291	343.71	219.42	0.638	0.19182	0.17925	1917.47	538.39	0.781
12	00.00.27.05	12.502	3.589	598.1	0.00285	350.89	209.87	0.598	0.20837	0.17945	1955.37	586.68	0.666

^aRow whose parameters were used in settlement computation

5.8 Comparison of computed settlement values with those from previous works

There is paucity of literature on examples of settlement either computed or observed of structures. Mayne [19] reported settlements of a raft footing for a 13 storey building of between 25 cm at the center of the structure to 12.7 cm at the corners. The raft size has a length of 104 m, width of 18 m, and thickness of 1.1 m. The building was constructed on Piedmont residual silty soils. The soils are silty, and clayey, but have a clay fraction of less than 20%. Load on the mat footing is estimated at about 150 kPa. Soil borings on the site investigated indicate the presence of sandy silts (ML), and silty sands (SM). Ground water table is at between 6.0 and 7.0 m below the ground surface. Settlement calculations were made using the equation provided by 'elastic continuum theory'. The equation is given as.

$$\rho = (q)(d)I_h \frac{(1 - \nu^2)}{E_o} \quad (7)$$

where ρ = is the settlement at the center of a footing, Q = the imposed load, D = width of the footing, ν = Poisson ratio taken to be between 0.1 and 0.2, E_o = equivalent elastic soil modulus beneath the foundation base determined for this case by Dilatometer, and I_h = displacement influence factor that accounts for the simplified subsurface geometry, layering, foundation rigidity, layer thickness, and soil stiffness variation with depth obtained from chart prepared by [25].

The above equation was used to estimate settlements at different positions beneath the raft of the building 'B' reported in the article. The results matched the settlement observed. It was reported that consolidation tests over-predicts settlements on Piedmont soils, hence the adoption of the elastic continuum theory method. Compared to the study, the soils under investigation are essentially sand, although silty sand (SM) is common to the two cases. The loads used in estimating settlement in Piedmont soil which 150 kPa is similar to that of sample 'A' in the present study. The consolidation settlements estimate for the soil in this study appears to be excessive for the particular load, although there is a paucity of data from the study area that will enable effective comparison. Applying Eq. (7) to calculate elastic settlement for the samples in this study, gives values that are higher than those computed with Eqs. (1) and (4). Settlements estimated by Eq. (1) seem more plausible.

Oweis and Scagnelli [26] reported on settlements of foundations for seven bridges that forms part of a 6.1 km Highway by-pass Route 31 Section 1A project. Site investigation was carried out at the piers, and east and west abutment locations of the bridges.

The foundations were founded on overconsolidated clays which have silty sand (SM), poorly graded sands (SP), and SP-SM as intercalating layers. The clays include inorganic clay (CL), inorganic silt of low to medium plasticity (ML), inorganic silt of high plasticity (MH), Inorganic clay (CH), and CL-ML.

Field investigation tools consist of Standard penetration test borings, piezocone soundings, and Pressuremeter testing. Settlement of the bridge structures was monitored from between 300 and 500 days after construction. Settlement estimation was made using the software, SETTLE/G (1987). It was reported that both immediate and consolidation settlements were calculated. Typical contact pressures on the center piers were about 123.8 kPa while that of the abutment is about 264.9 kPa. All the foundations were spread footings. At bridge 1, and at a boring designated No C1-1, a silty sand (SM) occurred at 3.66 m from the ground surface. Its Pressuremeter modulus was listed as 1.41 MPa. An average of four moduli was the modulus used in the settlement computations. The moduli are Pressuremeter modulus, unconfined compression modulus, Oedometer modulus, and undrained triaxial compression tests with pore pressure measurements modulus. In this example, though triaxial tests represent one of the tests used to estimate modulus, it must be that it cannot give a value of Poisson ratio reliable enough to be employed in the computation of settlements; hence, a value of 0.33 was assumed. This could have effect on all the values of settlements computed. The method proposed by the present study estimate Poisson ratio reasonably.

It was reported that while the settlements for all the piers were over-predicted, the predictions for the abutment settlements were in accord with predictions except for one bridge. The East abutment of bridge 3 has an imposed pressure of 232.7 kPa, with average estimated elastic settlement of 2.70 cm, and a total estimated settlement of 7.46 cm. while average measured settlement is 5.95 cm. In this study, the nearest load to the above is that of sample 'D' which is 229.64 kPa. The least value of elastic settlement computed for this load is 1.31 cm while consolidation settlement is 13.29 cm. Expectedly the elastic settlement values is less than that quoted for bridge 3 above since the soil here is sandy with some clays. No purely clayey or silty layer is assumed to be within the 4.0 m compressible stratum used in calculation in this study. Pure clay or silt will deform more than a sandy soil.

Dafalla [27] reports on settlements associated with soil which is similar both in type and geology to the one in this study. The study area is on the south west coast of Saudi Arabia, along the Red Sea coast (Coastal Plain Sands). The site is reported to be underlain by medium dense silty sand followed by very loose to medium dense silty sand to sandy silt; with Standard penetration test blow count as low as 4

Table 5 Direct shear box test results for sample 'E' for normal load 277 kPa

Reading No.	Time (mins)	Horizontal displacement q _h (mm)	Vertical displacement q _v (mm)	Shear force T(N)	Corrected Area (A _c) (m ²)	Vertical stress (δ _v) (kPa)	Shear stress † (kPa)	T/N	Shear strain	Vertical strain	Elastic modulus (kPa)	Shear modulus (kPa)	Poisson ratio -calculated
1	00.00.00.00	0	0.000	0.3	0.0036	277.78	0.08	0.000	0.0000	0.0000	#DIV/0!	#DIV/0!	
2	00.00.00.15	0.002	0.446	13.9	0.0036	277.80	3.86	0.014	0.0000	0.0223	12457.23	115841.06	-0.946
3	00.00.01.00	0.211	0.689	165	0.0036	279.74	46.16	0.165	0.0035	0.0345	8120.23	13125.33	-0.691
4 ^a	00.00.02.15	0.699	0.823	301.2	0.0035	284.36	85.65	0.301	0.0117	0.0412	6910.45	7351.99	-0.530
5	00.00.04.00	1.274	1.034	387.3	0.0034	289.96	112.30	0.387	0.0212	0.0517	5608.52	5288.94	-0.470
6	00.00.06.15	2.141	1.189	457.1	0.0033	298.72	136.54	0.457	0.0357	0.0595	5024.66	3826.52	-0.343
7	00.00.09.00	3.633	1.322	519.8	0.0032	314.74	163.60	0.520	0.0606	0.0661	4761.55	2701.92	-0.119
8	00.00.12.15	5.347	1.583	563.9	0.0030	334.79	188.79	0.564	0.0891	0.0792	4229.81	2118.44	-0.002
9	00.00.16.00	7.048	1.777	596.7	0.0028	356.64	212.81	0.597	0.1175	0.0889	4014.00	1811.66	0.108
10	00.00.20.15	9.321	1.908	609.7	0.0026	389.35	237.39	0.610	0.1554	0.0954	4081.27	1528.09	0.335
11	00.00.25.00	11.643	1.988	601.9	0.0023	427.64	257.40	0.602	0.1941	0.0994	4302.24	1326.45	0.622
12	00.00.30.15	14.279	2.000	534.1	0.0021	478.38	255.50	0.534	0.2380	0.1000	4783.75	1073.61	1.228
13	00.00.31.12	14.756	2.020	503.4	0.0020	488.52	245.92	0.503	0.2459	0.1010	4836.78	999.94	1.419

^aRow whose parameters were used in settlement computation**Table 6** Direct shear box test results for sample 'E' with normal stress of 138 kPa

Reading No.	Time (mins)	Horizontal displacement q _h (mm)	Vertical displacement q _v (mm)	Shear force T (N)	Corrected Area (A _c) (m ²)	Vertical stress (δ _v) (kPa)	Shear stress † (kPa)	T/N	Shear strain	Vertical strain	Elastic modulus (kPa)	Shear modulus (kPa)	Poisson ratio -calculated
1	00.00.00.00	0.005	0	-0.2	0.00360	138.90	-0.06	0.000	0.00008	0.00000	#DIV/0!	-666.72	
2	00.00.00.15	0.02	0.182	10.3	0.00360	138.94	2.86	0.021	0.00033	0.00910	15267.60	8586.20	-0.111
3	00.00.01.00	0.039	0.223	23.9	0.00360	138.98	6.64	0.048	0.00065	0.01115	12464.50	10220.32	-0.390
4	00.00.02.15	0.33	0.237	31.4	0.00358	139.66	8.77	0.063	0.00550	0.01185	11785.40	1594.63	2.695
5 ^a	00.00.04.00	1.019	0.499	114	0.00354	141.29	32.21	0.228	0.01698	0.02495	5662.86	1896.79	0.493
6	00.00.06.15	2.08	0.722	184.7	0.00348	143.88	53.15	0.369	0.03467	0.03610	3985.50	1533.12	0.300
7	00.00.09.00	3.41	0.83	207.6	0.00340	147.26	61.14	0.415	0.05683	0.04150	3548.39	1075.80	0.649
8	00.00.12.15	4.951	1.094	265.6	0.00330	151.38	80.41	0.531	0.08252	0.05470	2767.46	974.51	0.420
9	00.00.16.00	6.784	1.301	302.7	0.00319	156.59	94.80	0.605	0.11307	0.06505	2407.29	838.46	0.436
	00.00.20.15	8.847	1.488	341.6	0.00307	162.91	111.30	0.683	0.14745	0.07440	2189.65	754.83	0.450
11	00.00.25.00	11.145	1.581	378.9	0.00293	170.57	129.26	0.758	0.18575	0.07905	2157.78	695.88	0.550
12	00.00.30.15	12.7	1.614	322.4	0.00284	176.18	113.60	0.645	0.21167	0.08070	2183.15	536.70	1.034

^aRow whose parameters were used in settlement computation

Table 7 Bearing capacity values, Poisson ratio, elastic modulus, computed elastic and consolidation settlement values

Sam- ples	Unit weight (kN/ m ³)	Depth (m)	Cohes- ion, C (kN/ m ²)	Angle of internal fric- tion, (ϕ) degrees	Ulti- mate bear- ing capac- ity, (Q_u) (kN/ m ²)	Allow- able bear- ing capac- ity, q_{all} (kN/ m ²)	Stress values from direct shear box test	Numeri- cal value of Poisson ratio, ν	Width of foot- ing, B' (m)	Thick- ness of com- press- ible layer, $H=2B$ (m)	F_1 , Fac- tors from Stein- bren- ner curve	F_2 Factors from Stein- bren- ner curve	Influ- ence factor, I_f	Elastic Settle- ment due to allow- able load (cm) using Eq. (1)	Elastic settle- ment com- puta- tion (cm) using Eq. (4)	Elastic settle- ment com- puta- tion (cm) using Eq. (7) ^a	I-D con- strained Modulus (kPa)	Com- press- ibility, Mv. Per kPa	Consoli- dation settle- ment from Eq. (5) (cm)
Sam- ple A	16.71	1.2	30.12	23.1	469.73	156.58	152.58	0.13	2	4	0.275	0.06	0.327	6.78	12.65	17.21	1484.99	0.00067	42.18
Sam- ple B	15.21	1.4	50.10	24.8	661.53	220.51	282.08	0.44	2	4	0.275	0.06	0.290	2.51	6.44	8.75	4112.02	0.00024	21.45
Sam- ple C	15.21	1.4	44.52	25.2	644.75	214.92	279.81	0.18	2	4	0.275	0.06	0.322	6.56	12.65	17.20	2039.41	0.00049	42.15
Sample D	15.21	1.4	52.65	23.9	688.93	229.64	284.36	0.53	2	4	0.275	0.06	0.275	1.31	3.99	5.42	6910.45	0.00014	13.29
Sam- ple E	16.01	1.4	15.78	30.74	415.61	138.54	141.29	0.49	2	4	0.275	0.06	0.275	1.02	2.94	3.99	5662.86	0.00018	9.79

^aInfluence factor, $I_f=0.85$, Poisson ratio, $\nu=0.2$

Table 8 Typical values of compressibility for clays^a

Soil description	Compressibility	M_v (m ² MN) ⁻¹	Cc
Heavily over consolidated clays	very low	< 0.05	< 0.10
Very stiff to hard clays	Low	0.05–0.10	0.10–0.25
Medium clays	Medium	0.10–0.30	0.25–0.80
Normally consolidated clays	High	0.30–1.5	0.80–2.50
Very organic clays and peats	very high	> 1.5	> 2.50

M_v = Coefficient of volume compressibility; Cc = Coefficient of compression

^aBy D. G. McKinlay [22]

blows per 300 mm. Groundwater table is at 2.4–3.8 m below the ground surface. The materials are predominantly classified as SM, ML and SC. Clayey materials are very limited and are encountered as thin films. The soils are reported to be very compressible and liquefiable. Buildings for a technical college were planned for the site. To minimize settlement to tolerable limits, it was decided to preload the soil with sand embankment as surcharge, which will impose a minimum load of 100 kN/m². The preloading was carried out for a total of 32 weeks. A settlement rate of 2.5–5 cm/week was observed during first 4 weeks, followed by a rate 0.25–0.5 cm/week from 4 to 12 weeks, and a very slow rate of settlement of 0.1–0.25 cm/week in the last 12–32 weeks. This gives a total settlement in the range of 14–29 cm over the 32 weeks. With regard to the present study, the nearest load value close to the applied surcharge load is sample 'E' with a load of 141.29 kPa, which gives a total settlement of 10.81 cm by Eq. (1) and 12.73 cm by Eq. (4). These values are not significantly different from the lower value of the settlement observed in this example which is 12 cm. It was also reported that changes occur within the 3–4 m from the ground surface, which represents approximately the thickness of compressible layer assumed in using Eqs. (1) and (4). This study however did not predict an upper limit of 29 cm observed settlement.

5.9 Limitations of the method and future work

1. Sample disturbance, due to removal of overburden pressure, which causes the top of the soil sample to expand.
2. The fact that the bearing capacity is not known when the Direct shear test is carried out may result in normal stress applied to sample being out of range. To address this, the usual doubling of the vertical load on the sample during Direct shear box test should not be used. Instead a stepwise increase in the vertical load should be adopted. For example, 100 N, 200 N, 300 N, 400 N, 500 N.... to 1500 N. This loading sequence also depends on the type of soil being investigated. The loading sequence suggested will be appropriate for the type of soil investigated in this study, which are

clayey and silty sands of medium to high compressibility.

The method hold good prospect for estimating settlement as noted in the first two cases cited where the Poisson ratio used for estimating settlements were assumed, whereas it is determined with this method.

6 Conclusion

A method using the Direct shear box test is put forward. It has been demonstrated that it is possible to obtain reasonable estimates of parameters that are required to estimate elastic settlement for the type of soil investigated in this study. Such parameters include, shear modulus, elastic modulus, and Poisson ratio.

Elastic settlement estimated using these parameters is realistic and reasonable for the type of soil.

Elastic settlement computed using basic elastic theory equation (Eq. 1) gave values that are least conservative compared to other equations used, but are plausible.

There is the possibility of deducing allowable bearing capacity of a soil by reading off the value of the peak shear resistance from the shear stress versus horizontal displacement curve of the loading sequence which develops the vertical stress close to the estimated value, which is the first and second load sequence in this study.

The modulus used in computing the settlements in this study was not adjusted as is normally done for the ones obtained from oedometer test as the modulus is partially constrained while that of oedometer is fully constrained.

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Compliance with ethical standards

Conflict of interest On behalf of all authors, the corresponding author states that there is no conflict of interest.

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