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Estimation of deformation modulus of gravelly soils using dynamic cone penetration tests

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KEYWORDS

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Gravelly soil

Abstract Estimating the deformation modulus of gravelly soils is a challenging task. The estimate of deformation modulus of cohessionless soils in general relies on availability of correlation between in situ test parameters and deformation modulus back-calculated from field results of pressure settlement relationship based on plate load or footing load tests or observed settlement records. However, such a correlation is rare for gravelly soils. Even if it exists, the correlation is usually constrained with few limitations due to field testing problems associated with presence of gravel size particles. The aim of this paper is to develop a new correlation between deformation modulus of gravelly soils and results of dynamic cone penetration tests. The correlation relies on results of footing load tests carried out in a reclaimed site in Alexandria, Egypt, side by side to dynamic cone penetration tests. The developed correlation is reinforced by settlement records for structures on gravelly soils from literature.

1. Introduction

Extracting undisturbed samples from cohesionless soils is a very difficult task. Accordingly, estimating the deformation modulus of such soils from laboratory testing is a challenging process. The estimate of deformation modulus of cohessionless soils depends on the availability of the correlation between field test parameters and deformation modulus back-calculated from the field results of pressure versus settlement relationship based on plate load tests, footing load tests, or observed settlement records. Although many correlations exist in literature (e.g. [4,25]), such correlation is rare for gravelly soils. Even if it exists, the correlation is usually constrained by a few limitations due to field testing problems associated with the presence of gravel size particles in the ground.

The presence of gravel in a deposit may lead to major problems that prohibit the possibility of using the in situ test parameter in estimating the compressibility of gravelly deposits, and thus, complicate the presence of correlation between the in situ parameter and deformation modulus for gravelly soils. For example, damage could happen to the shoe of the spoon sampler of the Standard Penetration Test (SPT) or to the sensitive tip of the Cone Penetration Test (CPT). If large particles were to become stuck in the shoe of the spoon sampler of SPT, unrealistically high values of SPT N values might be obtained. The presence of large particles under the tip of the

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cone may require unrealistically high axial force to be penetrated or to be pushed. This may sometimes lead to a false refusal case, and thus, shallow termination of the penetration process. The fact that the pressuremeter test is an expensive test that requires special interpretation makes the routine use of the test in site investigation uncommon. In addition, the presence of large particles may cause caving in of the pressuremeter hole before testing.

Some of the above mentioned difficulties led to the introduction of the idea of attaching a solid 60° cone to the end of the SPT shoe [17]. The idea may be started earlier with different elements and input energy [1]. It is believed that this process was the basis for the Dynamic Cone Penetration Test (DCPT) as referenced in British and DIN Standards.

The aim of this paper is to develop a correlation between the blow count of the DCPT as an in situ parameter and the compressibility of gravel deposits determined from footing load tests or settlement records. The results of footing load tests that were carried out at a reclaimed site have been used to develop the relationship. In addition, the developed correlation is reinforced by settlement records of structures on gravelly soils from the database of Burland and Burbidge [4].

The developed correlation shall be an excellent design aid to assist engineers in sizing foundations on gravelly deposit using DCPT results without the need for conservative estimates of the compressibility of such deposits.

2. Reclamation and materials used in reclamation

A major site was reclaimed in Alexandria, Egypt, for the purpose of development of a marina and luxurious residential villas along the developed facility. The site was reclaimed by using underwater filling consisting of a mixture of sand and gravel with a maximum size of about 60 mm. The gradation and classification of the gravel and sand mixture used in the reclamation is shown in Table 1. The thickness of the reclaimed layer was in the range of about 2–6 m.

3. Footing load tests results

Footing load tests were carried out on the reclaimed subsoil formation at several locations across the Alexandria site, as shown in the layout in Fig. 1. The main purpose of the tests was to evaluate the compressibility of the gravelly subsoil formation. The tests were performed using reinforced concrete footings with dimensions of $1.0 \times 1.0 \times 0.30$ m. The footings were used in the tests after allowing enough time to ensure that the concrete had gained enough strength. A steel plate of 30 cm in diameter and 23 mm in thickness was used as a load bearing below the load acting at the center of the footing to ensure load distribution and avoid possible punching due to load concentration. Fig. 2 shows a schematic diagram of the footing load test setup. For each test, the footing was loaded in increments until reaching a contact stress of about 150 kPa. Thereafter, the footing was unloaded in decrements. During each load increment and decrement of the testing, the settlement was measured at five different points on the footing – one point at the footing center and the other four points at the corners – and recorded. The average settlement value has been considered when plotting the resulting stress versus settlement curves. Fig. 3 shows the stress versus settlement relationships for all of the footing load tests.

Table 1 Gradation and classification information of the sand and gravel mixture used in reclamation.

Effective size, D_{10} (mm)	0.40
Mean particle size, D_{50} (mm)	15
Maximum size (mm)	60
Clay fraction (%)	–
Fines content (%)	3
Sand content (%)	30
Gravel content (%)	67
Uniformity coefficient C_U	65
Coefficient of curvature C_C	1.54
Plasticity of fines	Non-plastic
Classification (USCS)	GW
	Well graded gravel with sand

4. Dynamic cone penetration test results

Dynamic Cone Penetration Tests (DCPTs) were performed on the subsoil formation at the site. The tests were performed using a split-barrel sampler with a 50 mm outside diameter and 35 mm inside diameter and about 600 mm length. The toe of the sampler was connected to a solid cone. The split-barrel sampler was connected to a string of drilling rods. The sampler was driven into the bottom of the borehole by means of a 63 kg hammer falling freely along a guide from a height of 760 mm and onto an anvil at the top of the drilling rods. The number of blows required to advance the sampler with the solid cone a distance of 10 cm in the soil is known as the N_{100}(SH). It represents the super heavy dynamic cone penetration blow count within 100 mm penetration. Fig. 4 shows the N_{100}(SH) profiles measured across the site.

5. Calculation of deformation moduli

It is well known that the stress versus strain relationship of gravelly soil is nonlinear and thus it is expected that the secant deformation modulus is strain dependent (e.g. [15]). Thus, the stress versus settlement relationship might be judged to be nonlinear as well. However, since the shear strength of the gravelly soils is high (e.g. [25]), it is expected that the ultimate bearing capacity on the gravelly deposits is relatively high. The stress level expected from most of structures founded on shallow foundations is very small compared to the ultimate bearing capacity of gravelly deposits. Thus, for all practical purposes, the stress versus settlement relationship under footings can be assumed to be linear and in the stress level range expected from buildings founded on shallow foundations. Such an assumption is used in the development of most of the methods used to predict settlement of shallow foundations on granular soils (e.g. [24,4,19]).

The deformation modulus of the gravely deposit at the Alexandria site was back-calculated from the results of footing load tests using the following equation adapted from Burland and Burbidge [4] and Terzaghi et al. [25]:

$$E_s = \frac{qZ_I}{S}\left[\frac{1.25(L/B)}{(L/B)+0.25}\right]^2 \qquad (1)$$

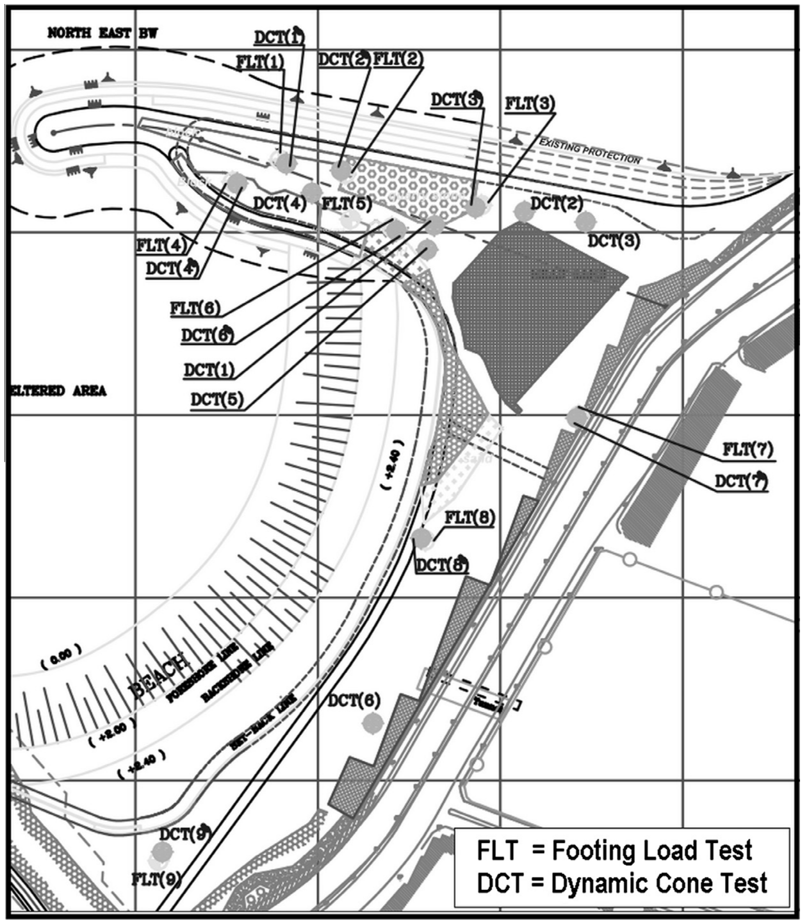

Figure 1 Site layout and field testing plan.

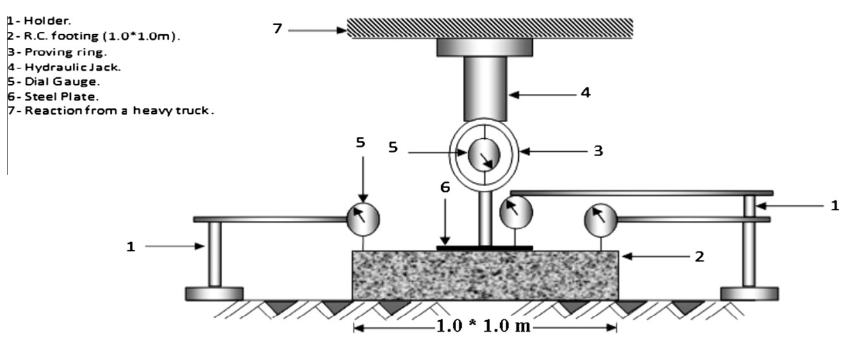

Figure 2 Schematic diagram of the footing load test setup.

where E_s is the loading deformation modulus in MPa; q is the net applied load on the footing in kPa; Z_I is the depth of the zone influenced by the load taken as $B^{0.75}$; B is the width of footing in m; L is the length of footing; and S is the settlement in mm. The slope of the unloading part of the curve was used to back-calculate the unload–reload deformation modulus $E_{s\text{-}ur}$. Table 2 shows a summary of the back-calculated values of the moduli. Shown also in the same table is the average value of N_{100}(SH), over a depth Z_I under the footing, from the DCPT carried out at the same location of the footing load test.

6. Correlations between dynamic cone and Standard Penetration Tests

The Dynamic Cone Penetration Test (DCPT) is a simple soil investigation used for in situ testing. In this test, a cone attached to the base of a small diameter rod is driven into the soil by means of regular blows from a hammer, and the number of blows required to drive the cone a distance "d" are counted. Accordingly, the DCPT N value is named N_d. In the literature, the distance "d" over which the blows are counted could be

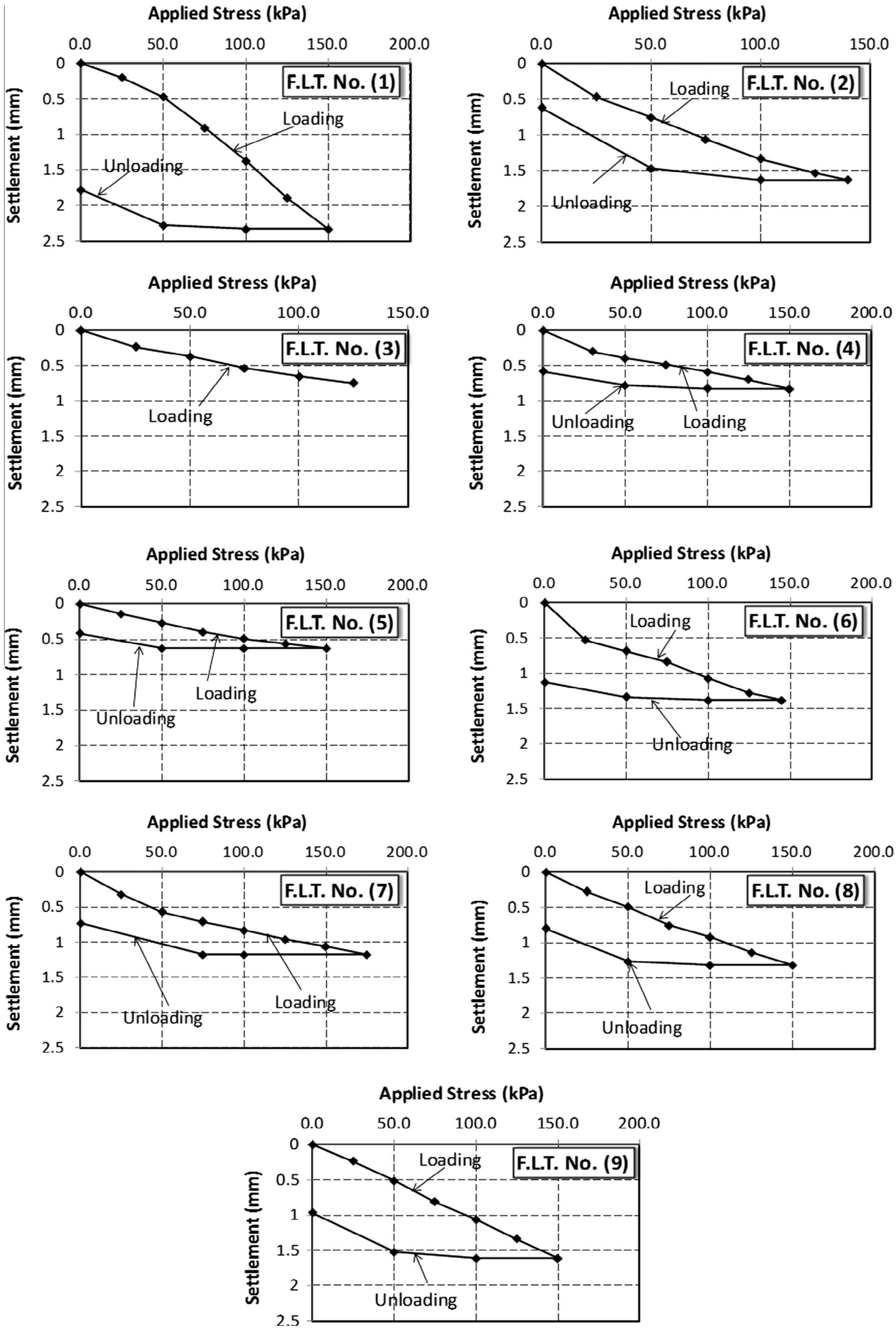

Figure 3 The stress versus settlement relationship for the footing load test.

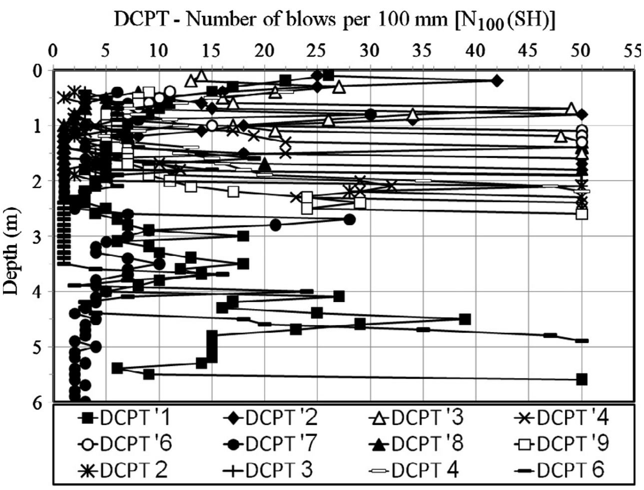

Figure 4 Dynamic cone test results across the site.

Table 2 Summary of back-calculated values of moduli and average DCPT over depth Z_I under the footing.

FLT no.	$N_{100}(\text{SH})_{\text{avg}}$	E_s (MPa)	$E_{s\text{-}ur}$ (MPa)
1	12	64.4	272.7
2	15	85.9	138.6
3	20.6	166.7	–
4	23.7	180.7	600.0
5	21.5	240.0	714.3
6	11.1	141.2	568.6
7	8.1	148.9	393.3
8	8.6	114.1	291.3
9	7.3	93.2	232.6

100 mm, 200 mm or 300 mm. The DCPT N_d value could be converted to N_{300} and vice versa using the following equation:

$$N_{300} = \frac{300}{d} N_d \tag{2}$$

The advantages the DCPT has over other penetration tests are its simplicity, portability, and low cost. There are four main types of dynamic cone penetrometers that are commonly used depending upon to the relation between the diameter of the cone and the diameter of the attached extension rod [29]. In the current study, the used cone has the same diameter of the extension rod.

According to the International Symposium of Penetration Tests, there are four different methods for dynamic probing DP [23]: DPL, DPM, DPH and DPSH. The abbreviation L, M, H and SH stand for the weight of the equipment, which is described as Light, Medium, Heavy and Super Heavy, respectively. The input energy for each type of probing is dependent upon the weight of the hammer and the drop height. According to the specific energy per blow ([9], [1]), the blow count of the dynamic probing of any weight category can be converted by the ratios of specific energy per blow to the Super Heavy dynamic probing blow using the following equation:

$$N_d(\text{SH}) = 0.7N_d(H) = 0.63N_d(M) = 0.21N_d(L) \tag{3}$$

Using the same concept of specific energy ratio per blow, the Standard Penetration Test blow count N can be theoretically converted to super heavy blow count using the following equation:

$$N_d(\text{SH}) = (1 \text{ or } 2)\frac{d}{300}N \tag{4}$$

The use of multiplier 1 or 2 depends on the assumption of the area of the shoe of the SPT; the ratio of 1 corresponds to soil plugging the shoe of the SPT while the ratio of 2 corresponds to the transmission of the energy through the annulus area of the shoe.

To determine a correlation between SPT N values and dynamic probing N_d, it is useful to use the experience accumulated over the years based on SPT N. Many correlations

Table 3 Summary of correlations developed or modified from the literature.

No.	Correlation	Soil	Reference
1	$N_{100}(\text{SH}) = 0.38N$	Sandy soils (Japan)	Muromachi and Kobayashi (1982)
2	$N_{100}(\text{SH}) = 0.2N$	Sandy-silty gravels	Tissoni [26]
3a	$N_{100}(\text{SH}) = 0.33N$	Alluvial gravel (UK)	Card and Roche [5]
3b	$N_{100}(\text{SH}) = 0.37N$	Flood Plain Gravel (UK)	
3c	$N_{100}(\text{SH}) = 0.47N$	Sands (UK)	
4	$N_{100}(\text{SH}) = 0.013N^2 + 0.009N$	Coarse grained soils	Cearns and McKenzie [6]
5a	$N_{100}(\text{SH}) = 0.6N$	Fine sand	
5b	$N_{100}(\text{SH}) = (0.1 - 1.0)N$	Medium sand	
5c	$N_{100}(\text{SH}) = 0.27N$	Coarse sand	
5d	$N_{100}(\text{SH}) = 0.33N$	Gravel	
6	$N_{100}(\text{SH}) = 0.2N$	Coarse soil (Italy)	Cestari [7]
7	$N_{100}(\text{SH}) = \frac{0.15\left(\frac{\sigma'_o}{Pa}\right)+0.083I_c+0.262}{\left(\frac{2}{\sqrt{N}}-0.36\right)}$ σ'_o is effective overburden pressure, Pa is a reference pressure taken as 100 kPa and I_c is a soil type factor	All soils (Egypt)	Abu-ElNaga [1]
8	$N_{100}(\text{SH}) = 0.5N$	Coarse soil (Germany)	DIN [9]
9	$N_{100}(\text{SH}) = 0.17N$	Sandy-silty with fine gravel (Italy)	Spagnoli [22]
10	$N_{100}(\text{SH}) = 0.3N$	Highly weathered limestone (Sudan)	Kassim and Ahmed [11]
11	$N_{100}(\text{SH}) = \frac{0.267N}{1-0.02N}$	Sandy soils (South Africa)	MacRobert et al. [13]

between SPT N values and dynamic probing N_d are based on comparative field measurements. In this paper, for ease of comparability the super heavy dynamic probing is used, because it corresponds to almost the same input energy of SPT. Furthermore, N_{100} is used instead of other values introduced in the literature. Therefore, any dynamic probes used within another weight category with a blow count over a distance "d" shall be converted to Super Heavy N_{100} using Eq. (2). Table 3 summarizes the correlations developed or modified from the literature. Fig. 5 shows a graphical summary of the correlations in Table 3. Shown also in the same Figure is the correlation proposed in this study expressed in the following equation:

$$N_{100}(\mathrm{SH}) = \frac{0.18N}{1 - \sqrt{0.012N}} \tag{5}$$

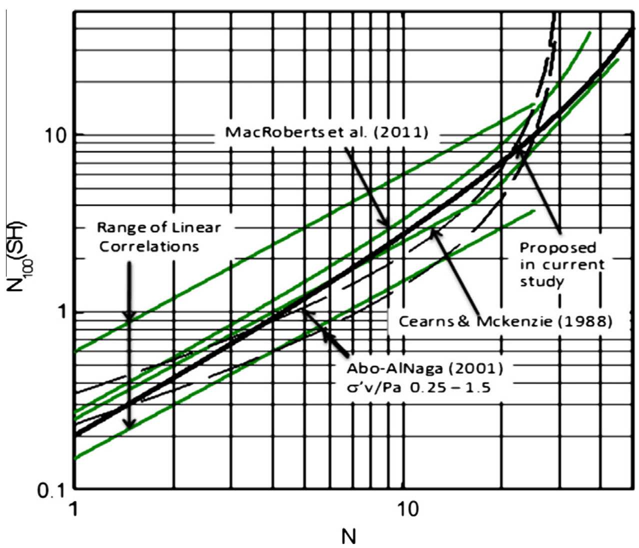

Figure 5 Summary of correlations in the literature and the proposed correlation.

7. Settlement of gravely deposits

Burland and Burbidge [4] developed an extensive database of settlement records from all over the world. The database is comprised of more than 200 records, and includes settlement records of footings over sand and gravel. The purpose of using the database was to develop a correlation between a compressibility parameter and average SPT N values, and thus, a method for estimating settlement of footings on cohessionless soils. In spite of the fact that the number of records is large, only a limited number of cases (about 20) were recorded for deposits that include gravel. Based on these data, Burland and Burbidge [4] statistically attempted to introduce a correction factor for gravelly soils. The correction factor was to increase the measured SPT N values by 25%. Because such a correction did not seem to make physical sense, Burland and Burbidge [4] recommended neglecting such a correction factor, stating the need for further data collection due to the limited number of

Table 4 Settlement records on gravelly deposits (from Ref. [4]).

Structure	SPT (N)	B (m)	L (m)	Depth (m)	Pressure (kPa)	S (mm)	N_{100}(SH)	E_s (MPa)	$E_{s\text{-}ur}$ (MPa)	Reference
Nuclear reactor	47	60–0∅	–	5.2	417	45	34.0	199.8		Breth and Chambosse [3]
Silo	33	2.4	Strip	–	490	14	16.0	100.4		Bjerrum and Eggestad [2]
Nuclear reactor	60	135	179	20.9	500	15	71.3		1460.2	Fischer et al. [10]
Footing load test	29	1.2	1.2	2.6	215	2.5	12.7	98.6		Bazaraa
	26	1.2	1.2	2.6	215	1.5	10.6	164.3		Bazaraa
	18	1.2	1.2	2.6	215	8.6	6.1	28.7		Bazaraa
12 Storey building	37	4	7	5	518	7.6–11.9	20.0	189.0		Levy and Morton [12]
Footing load test	50	1.2	1.2	0.5	300	4.5	39.9	76.4		Levy and Morton [12]
	50	1.4	0.9	3.7	300	1.5	39.9	257.4		Levy and Morton [12]
	30	0.9	0.9	1.2	300	4.0	13.5	69.3		Levy and Morton [12]
	20	0.9	0.9	3.1	300	6.7	7.1	41.4		Levy and Morton [12]
	20	0.9	0.9	1.2	300	2.7	7.1	102.7		Levy and Morton [12]
Factory building	13	1.1	1.1	1.2	78	2.0	3.9	41.9		Meigh and Nixon [14]
	13	1.5	1.5	1.2	77	2.1	3.9	49.7		Meigh and Nixon [14]
	13	1.5	1.5	1.2	77	1.3	3.9	80.3		Meigh and Nixon [14]
Plate tests	25	1.2	1.2	0	320	2.8	9.9	131.0		Oweis [16]
Building	36	41.2	41.2	10.0	158	10	18.9	256.9		Sanglerat et al. [18]
30 Storey building	20	17.6	84.0	10.7	240	21.2	7.1	137.2		Schultze [20]
20 Storey building	14	16.0	43.0	7.3	228	17.9	4.3	133.3		Schultze [20]
Chimney	10	20.5∅	–	3.5	173	8.0	39.9	208.3		Schultze [20]
Chimney	26	14.5	14.5	3.5	255	15.5	10.6	122.2		Schultze [20]
Nuclear reactor	34	33.0∅		5.3	216	43.8	16.9	67.9		Schultze [20]
Building	37	2.6	10.7	1.0	293	10.9	20.0	76.4		Schultze and Sherif [21]
5 Storey building	50	3.8	Strip	7.0	383	4.8	39.9	323.0		Tschebotarioff [27]

records on gravelly soils. The back-calculated deformation moduli of these records are shown in Table 4. The average SPT N values of each of the published case records were converted to N_{100}(SH) as shown in Table 4 using Eq. (5) that was developed in the current study.

8. Proposed equation to estimate deformation modulus

The deformation modulus together with the unload–reload deformation modulus back-calculated from the results of footing load tests in this study were plotted against N_{100}(SH) in Fig. 6. Plotted also in Fig. 6 are the data from settlement records presented in (Table 4). In spite of the presence of expected scatter, it is interesting to note that the two ranges of data coincide with each other. Fig. 6 also shows the following proposed expressions for the correlations developed in this paper to estimate the compressibility of gravelly deposit from DCPT results:

$$E_s = 20[N_{100}(\mathrm{SH})]^{0.65} \tag{6}$$

$$E_{s\text{-}ur} = 80[N_{100}(\mathrm{SH})]^{0.65} \tag{7}$$

It is known in the literature that the ratio between the unload–reload deformation modulus and the deformation modulus during loading is constant and in the range between 2 and 4 [4]. Eqs. (6) and (7) suggest that the ratio is in the range between 3 and 4. However, the ratio seems to be slightly dependent upon the value of N_{100}(SH). Such a trend is similar to that reported by Vaughan [28] who presented the experimental data of Daramola [8] that suggest that the mentioned ratio is dependent upon the state of denseness of the soil.

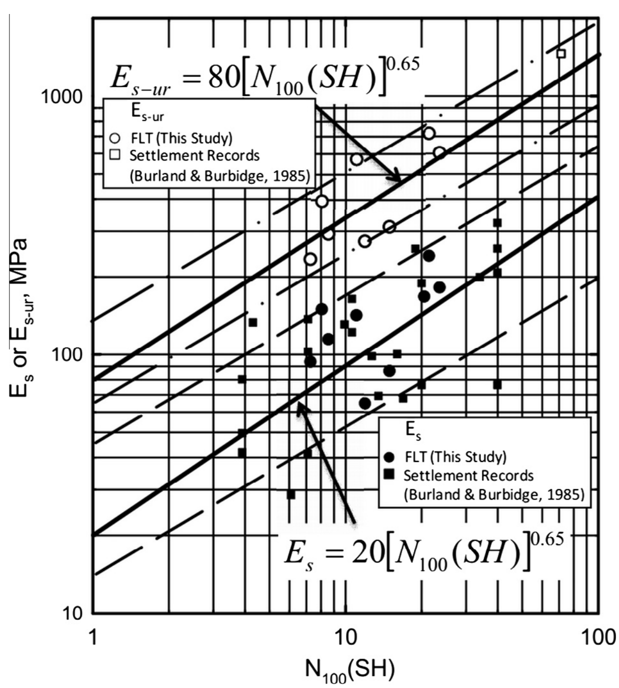

Figure 6 Deformation modulus and unload–reload modulus from footing load test in the study (FLT) and those from settlement records from Burland and Burbidge [4].

9. Conclusions and concluding remarks

Based on a review of available data and relationships in the literature, a new correlation is proposed between the Standard Penetration Tests blow count and that of the dynamic cone penetration tests.

The results of footing load tests carried out on the reclaimed site of gravelly deposit are used to back calculate the deformation modulus and the unload–reload deformation modulus of the gravelly deposit.

The results of the dynamic cone penetration tests that were carried out side by side to the footing load tests were interpreted and used to develop the intended correlation in this paper.

Settlement records on gravelly deposits from the Burland and Burbidge [4] database were used to reinforce the data obtained from the footing load tests. As Burland and Burbidge [4] used SPT N values as a basis for the correlation, the SPT N values of the selected case records were converted to DCPT N_{100}(SH) using the SPT–DCPT correlation developed in this paper.

The data from both footing load tests and settlement records were used to develop correlations to estimate both the deformation modulus and the unload–reload modulus from DCPT N_{100}(SH).

The proposed correlations are a useful aid to help engineers in the practice to size foundations on gravelly deposits.

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References

[1] Abu-ElNaga HM. Calibration of dynamic cone results in cohesionless soils using artificial neural network. MSc thesis, Faculty of Engineering, Cairo University; 2001. p. 243.
[2] Bjerrum L, Eggestad A. Interpretation of a loading test on sand. In: Proceedings of the European conference on soil mechanics and foundation engineering, Wiesbaden, vol. 1; 1963. p. 199–204.
[3] Breth H, Chambosse G. Settlement distribution in the subsoil underneath a nuclear reactor. In: Conf. settlement of structure. Cambridge: Pentech Press; 1974. p. 10–3.
[4] Burland JB, Burbidge MC. Settlement of foundations on sand and gravel. Proc Inst Civil Eng (Part 1) 1985;78(December):1325–81.
[5] Card GB, Roche DP. The use of continuous dynamic probing in ground investigation. Proceedings, geotechnology conference: penetration testing in the UK, Birmingham. London: Thomas Telford; 1988.
[6] Cearns PJ, McKenzie A. Application of dynamic cone penetrometer testing in East Anglia. In: Proceedings, geotechnology conference: penetration testing in the UK, Birmingham. Thomas Telford, London; 1988. p. 123–7.
[7] Cestari F. Prove Geotecniche in Sito. Ed. GEO-GRAPH, Segrate; 1996.
[8] Daramola O. The influence of stress history on the deformation of sand. PhD thesis, University of London; 1978.
[9] DIN – Deutsches Institut für Normung. Erkundung und Untersuchung des Baugrunds. DIN 4094-2, 4094-3; 2002.

[10] Fischer JA et al. Settlement of a large mat on sand. In: Proceedings of the conference on performance of earth and earth-supported structures. Purdue University, June 1972, American Society of Civil Engineers special conf., 1, No. 2; 1972. p. 997–1018.
[11] Kassim BA, Ahmed OBEM. Comparison of continuous dynamic probing with the standard penetration test for highly weathered limestone of Eastern Sudan. Electron J Geotech Eng 2010;15(Bundle B):235–49.
[12] Levy JF, Morton K. Loading tests and settlement observations on granular soils. In: Conf. settlement of structures. Cambridge; 1974. p. 43–52.
[13] MacRobert C, Kalumba D, Beales P. Correlating standard penetration test and dynamic probe super heavy penetration resistance values in sandy soils. J S Afr Inst Civil Eng 2011;53(1).
[14] Meigh AC, Nixon IK. Comparison of in situ tests for granular soils. In: Proceedings of the 5th international conference on soil mechanics and foundation engineering, Paris, vol. 1; 1969. p. 499–507.
[15] Modoni G, Koseki J, Dan LQ. Cyclic stress strain response of compacted gravel. Géotechnique 2011;61(6):473–85.
[16] Oweis IS. Equivalent linear model for predicating settlements of sand bases. J Geotech Eng Div Am Soc Civil Eng 1979;105(GT12):1525–44.
[17] Palmer DJ, Stuart JG. Some observations on the standard penetration test and a correlation of the test with a new penetrometer. In: Proceedings of 4th international conference on soil mechanics and foundation engineering, London, vol. 1; 1957. p. 231–6.
[18] Sanglerat G et al. Control in situ des previsions de tassments basees sssur les essays de penetration statique (pour 79 ouvrages sur 17 sites differents) (In situ check of settlement predictions based on static penetration tests). Ann. Inst. Techn. Bar. Trav. Publ. 369; 1979. p. 31–50.
[19] Schmertmann JH. Static cone to compute static settlement over sand. J Soil Mech Found Div – ASCE 1970;96(3):1011–42.
[20] Schultze E. Problem bei der Auswertung von Setzungsmessnger (Problems in the analysis of settlement measurements). Essen: Vortrage Der Baugrundtagung; 1962, p. 343–81.
[21] Schultze E, Sherif G. Prediction of settlements from evaluated settlement observations for sand. In: Proceedings of the 8th international conference on soil mechanics and foundation engineering, Moscow, vol. 1.3; 1973. pp. 225–30.
[22] Spagnoli S. An empirical correlation between different dynamic penetrometers. Electron J Geotech Eng 2008(Bundle C):13.
[23] Stefanoff G, Sanglerat G, Burgdahl U, Melzer KJ. International reference test procedure. In: Proceedings of the 1st international symposium on penetration testing, dynamic probing, Document 2, Orlando, I; 1988. p. 53–70.
[24] Terzaghi K, Peck RB. Soil mechanics in engineering practice. 2nd ed. John Wiley & Sons Inc.; 1967.
[25] Terzaghi K, Peck RB, Mesri G. Soil mechanics in engineering practice. 3rd ed. John Wiley & Sons Inc.; 1996, 549 p..
[26] Tissoni A. La prova SPT e SPTC a confro nto nei terreni fluvioglaciali della pianura torinese. Geologia Tecnica, no. 20; 1987.
[27] Tschebotarioff GP. Soil mechanics, foundations and earth structures. New York: McGraw-Hill; 1951, p. 44 and 155.
[28] Vaughan PR. Discussion of "Settlement of foundations on sand and gravel" by Burland JB and Burbidge MC (1985). Proc Inst Civil Eng (Part 1) 1986;80:1627–30.
[29] Waschkowski E. Dynamic probing and site investigation. In: Proceedings of the European symposium on penetration testing, vol. 2, Amsterdam; 1982.

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