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Settlement of a ring foundation using cone data

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ABSTRACT: This paper presents a comparison between predicted and measured settlement for a 12-ft (3.7 m) wide, 4-ft (1.2 m) thick, and 80-ft (24.4 m) outside diameter ring foundation. Soil conditions at the site were investigated by five cone penetration tests. The soils consist of alternating layers of hard clay and dense to very dense sands and silts. A reasonable estimate of settlement was made using only cone data and available correlations between cone resistance and soil compressibility.

1 INTRODUCTION

Settlements of shallow foundations on dense sands and hard clays may be estimated by use of empirical methods or elastic theory. In empirical methods, the settlement is related directly to a parameter such as the N-value of the Standard Penetration Test combined with the load intensity and footing size. In methods based on elastic theory, it is necessary to calculate the increase in stress due to the applied loading and to estimate soil compressibility. The increase in stress is calculated by formulae or charts. The soil compressibility is based on laboratory data such as triaxial and consolidation tests or on correlations with in situ tests such as blow count, pressuremeter tests, cone penetration tests, and seismic velocity measurements.

For large and critical projects such as power plants, high-rise buildings, and offshore platforms, extensive soil investigation and analyses to assess settlement behavior can usually be justified. For more routine projects, the engineer is forced to base his assessment on either laboratory data from borings or in situ tests such as cone penetration tests or blow counts. Extensive testing and crosschecking by different methods is generally not possible due to budgetary constraints. Due to the inevitable sample disturbance in very stiff soils, soil compressibility based on laboratory tests yields settlements which may be several times the measured values (Peck, 1974 and Konstantinidis et al. 1986). Consequently, greater reliance should be placed on estimating compressibility from in situ tests or other indirect methods. The success of this approach depends on the data base supporting the correlations between soil compressibility and in situ tests. This points out the need to perform settlement measurements on full-scale structures and to compare predicted and measured settlements for improving the correlations between in situ tests and soil compressibility.

This paper presents the results of settlement measurements for a 12-ft (3.7 m) wide, 4-ft (1.2 m) thick, and 80-ft (24.4 m) outside diameter ring foundation. The ring supports a 69-ft (21.1 m) diameter butane storage sphere on 11 equally spaced legs. Gross loading below the 12-ft (3.7 m) wide ring during the hydrotest was 5.28 ksf (252.8 kPa). Settlements were calculated during design using cone penetration tests and published correlations between cone resistance and elastic modulus. By comparing measured settlements during the hydrotest with the calculated values, it is shown that a reasonable estimate of settlement was made using only cone data and available correlations.

2 SPHERE FOUNDATION

The sphere support legs each carry about 1075 kip (4784 kN) maximum load. The 11 steel legs are supported on 6 ft by 6 ft by 3 ft (1.8 m by 1.8 m by 0.9 m) high concrete piers equally spaced on a 12-ft (3.7 m) wide, 4-ft (1.2 m) thick reinforced concrete ring foundation. The bottom of the foundation is 6 ft (1.8 m) below the ground. The foundation plan and locations of the cone and soil borings are shown in Fig. 1. Foundation loads of the various components are provided in Table 1.

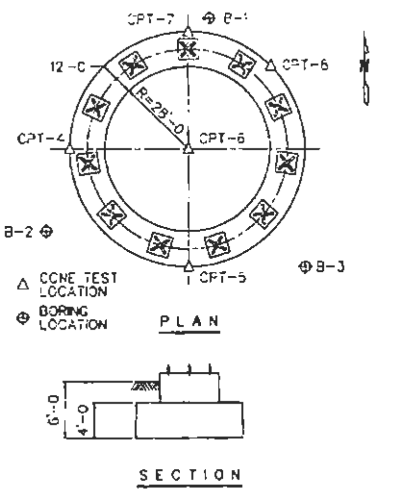

Fig. 1 Foundation and site investigation plan (1 ft = 0.305 m)

Table 1. Foundation loading

Component	Load kips	Pressure ksf
foundation and piers	1,720	0.67
steel tank	1,080	0.42
water during hydrotest	10,730	4.19
total during hydrotest	13,530	5.28

1 kip = 4.45 kN, 1 ksf = 47.9 kPa.

3 SUBSURFACE CONDITIONS

Soil conditions were investigated by performing five cone penetration tests to depths of 20 to 41 ft (6.1 to 12.5 m) below the existing ground surface. Three previously performed soil borings to depths of 50 ft (15.2 m) were also reviewed.

A typical cone penetration test and soil boring log is shown in Fig. 2. A summary of all five cone penetration tests is presented in Fig. 3.

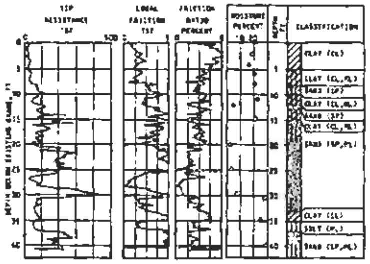

Fig. 2 Typical subsurface exploration log (1 ft = 0.305 m; 1 tsf = 95.8 kPa)

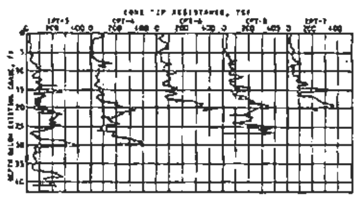

Fig. 3 Summary of cone penetration test results (1 ft = 0.305 m; 1 tsf = 95.8 kPa)

The upper 6 ft (1.8 m) of soils consist of variable fill with some soft clay in localized areas. This fill was excavated and the foundation was placed at a depth of 6 feet (1.8 m). Therefore, the fill is not relevant to the analysis. The fill soils are underlain by hard clays and dense to very dense sands to the 50-ft (15.2 m) depth explored. Four of the five cone penetration tests reached refusal at depths of 20 to 29 feet (6.1 to 8.8 m). Only one cone penetration test in the south end of the tank could be completed to a depth of 41 feet (12.5 m).

Primarily clayey soils are present between depths of 6 and 20 ft (2.1 and 6.1 m) and between 33 and 37 feet (10.1 and 11.0 m). The clays are of medium plasticity with liquid limits between 30 and 50 and plasticity indices between 10 and 25. The natural water contents of the clays range between 16 and 25 percent. The undrained shear strength of the clays was estimated from the cone data by using the relationship:

$$q_c = S_u N_k + p_o$$

where q_c is the cone penetration resistance, S_u is the soil undrained shear strength, N_k is the cone factor, and p_o is the total overburden pressure.

We selected N_k as 15 and estimated the undrained shear strength of the clays as ranging between 5 and 10 ksf (239.4 and 478.8 kPa) with an average value of 8 ksf (383.0 kPa).

Primarily sands and silty sands are present between depths of 20 and 33 ft (6.1 and 10.1 m), and between 37 and 50 feet (11.3 and 15.2 m). The sands have cone penetration resistance of 300 to 800 ksf (14.4 to 38.3 mPa). No free groundwater was encountered in any of the borings in the area within the upper 50 feet (15.2 m).

Seismic downhole shear wave velocity measurements in similar soil conditions at a nearby project provided shear wave velocities of 830 fps (253 m/s) to depths of 30 ft (9.1 m) and 1300 fps (396.2 m/s) between 30 and 65 feet (9.1 and 19.8 m).

4 SOIL COMPRESSIBILITY

Soil compressibility during the design stage was estimated by published correlations between constrained modulus and cone resistance (Mitchell and Gardner, 1975, Robertson and Campanella, 1984).

For sands the constrained modulus is defined as:

$$D = 1/m_v = \alpha q_c$$

where D is the constrained modulus, m_v is the coefficient of volume compressibility, and α is the factor relating cone resistance q_c to the constrained modulus.

The values of α vary widely and depend on the density, degree of overconsolidation, stress and strain level, and differences in theoretical methods used for obtaining the correlations. For very dense and apparently overconsolidated sands, we conservatively selected an α value of 4.

The value of preconsolidation load for the on-site clays was conservatively selected as 40 ksf (1915.2 kPa). This preconsolidation load was estimated on the basis of an average undrained shear strength of 8 ksf (383.0 kPa) and plasticity index of 18 by means of the following correlation (Skempton, 1957):

$$S_u/\bar{p}_o = 0.11 + 0.0037(PI)$$

where S_u is the undrained shear strength and $\bar{p}_o$ is the effective overburden pressure, and PI is the plasticity index.

The behavior of overconsolidated clays under load is largely determined by the ratio:

$$dp / (\bar{p}_c - \bar{p}_o)$$

where dp is the increase in stress and $(\bar{p}_c - \bar{p}_o)$ is the difference between the preconsolidation pressure and present overburden pressure.

This ratio, for an applied maximum surface load of 5.28 ksf (252.8 kPa), is less than 0.1 at the center of the clay layers. This indicates that the applied stress is less than 10 percent of the difference between the present overburden and the preconsolidation pressure. Under such conditions, the clay would behave almost elastically and it is appropriate to calculate the settlement using an elastic modulus. Since the thickness of the clay layers is small with respect to the size of the foundation, settlement would be primarily one dimensional and constrained modulus should be used in settlement calculations. The constrained modulus can be determined from the expression:

$$D = E\,(1-\mu) / (1+\mu)(1-2\mu)$$

where D is the constrained modulus, E is the elastic modulus and μ is the Poisson's ratio.

Assuming a Poisson's ratio, μ, of 0.35, the constrained modulus D is 1.6 times the elastic modulus E. For heavily overconsolidated clays of medium plasticity, we selected a conservative value of E/S_u of 100 (Ladd et al. 1977 and Robertson and Campanella, 1984), which leads to a constrained modulus of 160 S_u.

These correlations yield constrained modulus values in the range of 800 to 1600 ksf (38.3 to 76.6 mPa) for clays and 1200 to 2400 ksf (57.4 to 114.9 mPa) for sands. For sandy silt layers, modulus values using sand correlations are about 400 ksf (19.1 mPa) while using the clay correlations are about 1000 ksf (47.9 mPa). We used an intermediate value of 700 ksf (33.5 mPa) for such soils.

We compared these values with the values of modulus obtained from seismic shear wave velocity measurements. Assuming a shear wave velocity of 830 fps (253 m/s) for soils to a depth of 30 ft (9.1 m), and 1300 fps (396.2 m/s) between 30 and 65 ft (9.1 and 19.8 m), low strain Young's modulus values of 7000 ksf and 17000 ksf (335.2 and 813.9

mPa) are obtained. The elastic modulus was converted to a constrained modulus by multiplying with a factor of 1.6 as obtained previously. The low strain modulus must also be corrected for strain level. Anticipated axial strain is about 0.1 percent (shear strain of 0.135 percent). To correct for strain level, we used the relationship between shear strain and shear modulus shown in Fig. 4 (Seed and Idriss, 1970). Using these relationships we multiplied the low strain modulus with an average 0.15 reduction factor. This yielded a value of 1680 ksf (80.4 mPa) for the upper 30 ft (9.14 m) and about 4080 ksf (195.4 mPa) below that depth. These values are significantly higher than the values selected from the cone data.

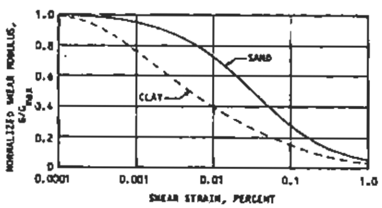

Fig. 4 Variation of shear modulus with shearing strain for sands and saturated clays (from Seed and Idriss, 1970)

For settlement calculations, we used the cone penetration test C-5 as the typical profile representing more compressible soil conditions and developed the generalized soil profile shown in Table 2.

5 SETTLEMENT CALCULATION PROCEDURE

To estimate settlements under the tank foundations, we used the computer program SETTL/G which performs settlement and stress distribution calculations under uniformly loaded rectangular areas (Geosoft, 1984). The program incorporates the effects of all loaded areas under all points. Loaded areas can be at or below the ground surface and Boussinesq, Mindlin, and Westergaard theories can be specified. The ring foundation was approximated by 64 rectangular areas and the soil profile was represented by nine layers shown in Table 2. The increase in stress at the center of each layer was calculated by using both Boussinesq and Mindlin stress distribution theories. The settlement below the selected points was calculated by:

$$S = \int_0^H \varepsilon_v \, dH = \sum_{i=1}^{n} (dp_i \, H_i)/\ D_i$$

where S is the settlement, ε_v = vertical strain, n is the number of soil layers, H is total soil thickness, dp_i is the change in stress at the center of layer i, H_i is thickness of layer i, and D_i is the constrained modulus of layer i.

The constrained modulus for soils below 41 ft (12.5 m) (maximum depth of cone penetration test) was conservatively assumed to be 2000 ksf (95.8 mPa). Settlement calculations were carried to a depth of 100 ft (30.5 m) because stress increase below this depth is less than 5 percent of the overburden pressure.

Table 2. Soil Profile

Depth, ft	Soil type	D, ksf
0– 6	Clay (Fill)	N.A.
6– 8	Clay	655
8– 10	Silty Sand	695
10– 19	Clay/Sand	1270
19– 33	Sand	1270
33– 35	Clay	1420
35– 37	Sandy Silt	700
37– 41	Sand	1680
41–100	Sand/Clay	2000

1 ksf = 47.9 kPa; 1 ft = 0.305 m

6 MEASURED SETTLEMENTS

Settlement measurements were made at 11 points on the foundation during the hydrotest. Settlement readings include readings during filling in about 24 hours, holding full load for four days, emptying in 24 hours and final reading after 18 days. The settlement readings for four diagonally opposite points are summarized in Fig. 5. These data indicate an average settlement of 0.6 in. (15.2 mm) with maximum and minimum values of 0.84 and 0.36, inch (21.3 and 9.1 mm). The maximum measured differential settlement under full load was about 0.5 inch (12.7 mm). The maximum differential settlement between any two adjacent legs was about 0.12 inch (3 mm). The permanent settlement after unloading ranges between 0.24 and 0.6 in. (6.1 and 16.2 mm) with an average value of 0.45 inch (11.4 mm).

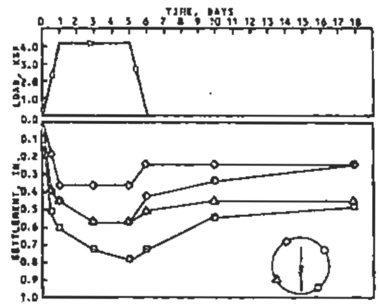

Fig. 5 Summary of measured settlements during the hydrotest
(1 in. = 25.4 mm; 1 ksf = 47.9 kPa)

7 SETTLEMENT COMPARISONS

We performed settlement calculations under the water load of 4.2 ksf (201.1 kPa) using the soil compressibility values shown in Table 2 and the calculation procedure described in the preceding section. Results of these calculations indicate maximum settlement under the center of the 12-ft (3.7 m) wide footing assuming a flexible loaded area as 1.02 inches (25.9 mm). The average settlement using a rigid foundation is 0.83 inch (21.1 mm). These values are obtained by using Boussinesq stress distribution. The maximum settlements using Mindlin stress distribution to include embedment effects are about 80 percent of the Boussinesq values. Based on analyses similar to those described here, we predicted total settlements in the range of 1 to 1.5 in. (25.4 to 38.1 mm) and settlements during hydrotest in the range of 0.75 to 1.1 inches (19.1–27.9 mm). The average predicted settlement during hydrotest of about 0.95 in. (24.1 mm) is about 60 percent greater than the average observed value of 0.6 inch (15.2 mm).

We also performed settlement calculations using the constrained modulus of 1680 ksf (80.4 mPa) in the upper 30 ft (9.1 m) and 4080 ksf (195.4 mPa) below that depth. These values were obtained after applying the strain adjustment to the low strain modulus values from the seismic velocity data. These calculations indicate an average settlement of 0.43 in. (10.8 mm) which is about 70 percent of the average measured settlement.

8 DISCUSSION AND CONCLUSIONS

Two of the methods generally used to estimate soil modulus for very stiff soils include correlations with cone penetration resistance and modulus obtained from seismic shear wave velocity data corrected for strain levels. Available correlations range from α values of 1.5 to 3.5 for normally consolidated sands and from 4 to over 20 for overconsolidated sands (Robertson and Campanella, 1984). Most of the high values of α appear to be based on laboratory data. Data from one site reported by Konstantinidis et al. (1986) indicate that high values in the range of 9 to 15 may be applicable for very dense sands with cone penetration resistance of 600 to 1200 ksf (28.7 to 57.5 mPa). Measured settlements for the tank foundation appear to support α values of about 4 to 6.

For overconsolidated clays, proposed E/S_u values range between 100 and over 600 depending on degree of overconsolidation, plasticity index, ratio of the applied stress to the undrained shear strength, and the method used in obtaining the value of S_u (Ladd et al., 1977). For shear strength of heavily overconsolidated clays of medium plasticity, obtained by using N_k of 15 from the cone data, an E/S_u of 100 to 200 appears to be justified.

If sand and clay layers contribute equally to the settlements, the constrained modulus D is 6 times q_c for sands and 16 times q_c for clays (D=1.6 E).

A reduction factor of 25 to 50 percent is generally recommended to convert the low strain seismic moduli for use in settlement calculations (Swiger, 1974). Based on the backcalculated modulus from the settlement measurements, lower reduction factors of 10 to 20 percent are also possible.

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NOTATIONS

dp	= increase in stress,
dp_i	= increase in stress at center of layer i,
D	= constrained modulus,
D_i	= constrained modulus for layer i,
E	= elastic modulus,
H	= total soil thickness,
H_i	= thickness of layer i,
m_v	= coefficient of volume compressibility,
n	= number of soil layers,
N_k	= cone factor,
p_o	= in situ total overburden pressure,
$\bar{p}_o$	= effective overburden pressure,
$(\bar{p}_c-\bar{p}_o)$	= difference between the effective preconsolidation and the present overburden pressures,
PI	= plasticity index,
q_c	= cone point resistance,
S	= settlement,
S_u	= undrained shear strength,
α	= factor relating cone resistance to constrained modulus,
ϵ_v	= vertical strain,
ν	= Poisson's ratio.