International Conference on Case Histories in Geotechnical Engineering

(1984) - First International Conference on Case Histories in Geotechnical Engineering

11 May 1984, 8:00 am - 10:30 am

Performance of Preload on Cohesionless Soils

A. F. Youssef
King Saud University, Saudi Arabia

O. Erol
King Saud University, Saudi Arabia

Performance of Preload on Cohesionless Soils

A. F. Youssef

Associate Professor, Civil Engineering Department, King Saud University

O. Erol

Assistant Professor, Civil Engineering Department, King Saud University

SYNOPSIS Common occurrance of loose normally consolidated fine sands and silts extending to considerable depths necessitates the utilization of ground improvement techniques along the coasts of Arabian Peniusula. In the case considered, the soil profile consisted of twenty meter thick compressible sands, and preloading technique was suggested to stabilize the foundation soil. The paper presents the performance of the preloading and the level of soil improvement achieved. The observed settlements and settlement-time behaviour are compared with the values estimated from various methods. The soil parameters back calculated from measured field behaviour are reported.

INTRODUCTION

Realistic prediction of ultimate settlements and settlement-time forecasts are essential for planning and scheduling the projects where preloading technique is used to improve the foundation soils. Settlement predictions, particularly in cohesionless soils, are occasionally unreliable due to the fact that representative soil deformation parameters are difficult to obtain. The widely used penetrometer techniques to obtain such parameters are entirely empirical in nature, and should be improved for more realistic settlement forecasts. The conventional consolidation theory is not appropriate for settlement-time predictions in cohesionless soils because of the discrepancies between behaviour of sand and idealized soil behaviour considered in the theory. Thus there is a continuous need for case records to evaluate the reliability of the current used methods to predict the real soil behaviour.

In the present study the performance of a preloading case encountered in West coast of Kingdom of Saudi Arabia is presented. The comparison of predictions with measured settlement and settlement-time behaviour are documented and summarized.

PROJECT AND SITE CONDITIONS

The project includes construction of a residential and hospital complex covering an area of approximately 100,000 sq.m. and consists of several one to four storey buildings. The location plan of the site and the field investigation program are shown in Fig. 1. The first stage of the field investigation included, 15 boreholes with standard penetration tests performed at 1.5 m intervals, and 4 deep sounding tests. The results of penetration tests and idealized soil profile are shown in Fig. 2.

The profile consists of the following subsequent

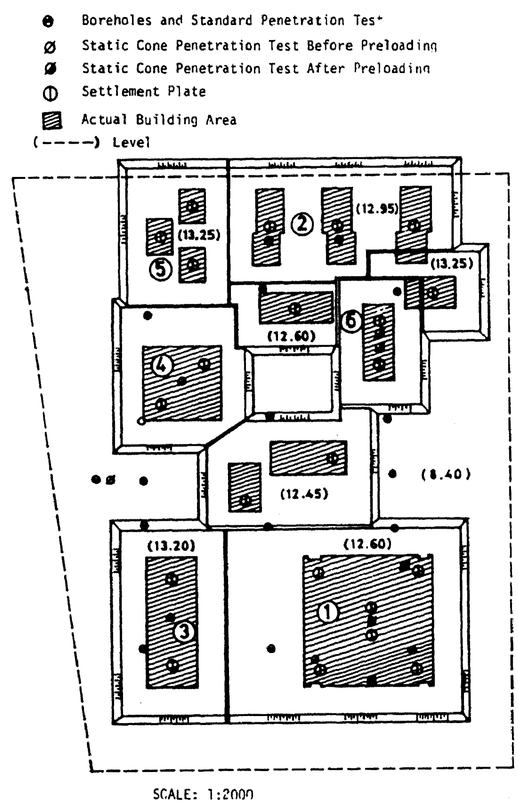

Fig. 1. The Location Plan and the Field Investigation Program

normally consolidated layers: (i) a 9 to 12 m. thick loose to medium dense silty fine sand with average standard penetration (SPT) resistance,

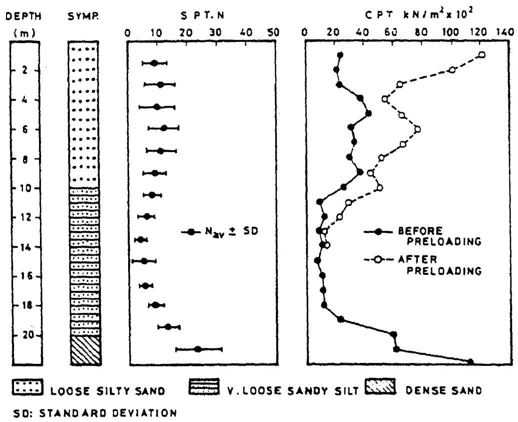

Fig. 2. The Results of Penetration Tests

N, is equal to 10, (ii) a 9 to 11 m. thick very loose sandy silt with an average N value of 5. Typical grain size distribution curve of the fine sand and sandy silt are shown in Fig. 3. The average static cone penetration resistances of the layers are in the range of 2000 to 4000 kN/m^2 and 800 to 1200 kN/m^2 for the sand and the underlying silt, respectively. The underlying material (at approximate depth of 22 m) is a dense to very dense sand. The ground water table is located at 1.4 to 2.0 m below the natural ground level.

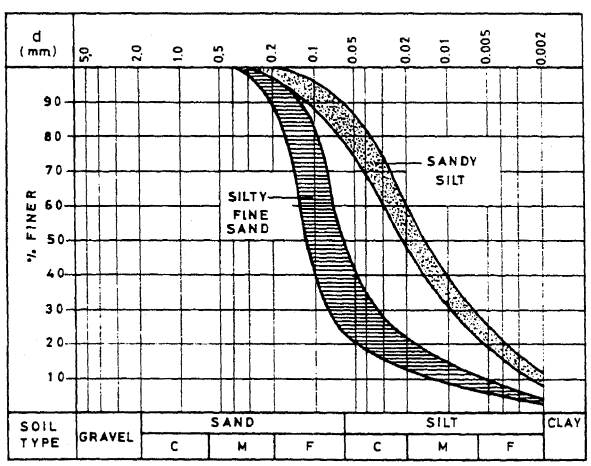

Fig. 3. Grain Size Distribution Curves

An average of 4 m high compacted fill was constructed over the entire area to preload the subsoil and reduce the post constructional settlements. Twenty meter long fabric drains, spaced at a distance of 2.5 m in an hegzagonal pattern, were installed to accelerate the consolidation process. The settlements were monitored by means of settlement plates placed at foundation level which was approximately 1.0 m below the natural ground surface. The location of the plates are shown in Fig. 1.

The placement of the fill was not uniform, and the construction of the full height was reache within 26 to 170 days applying surcharge loads from 79 to 122 kN/m^2 at various locations. I some areas the construction was not continuous and the time of intermission between successiv stages of loadings was sufficient for the settlements to cease. Fortunately, this step loading procedure provided additional data to evalate load versus ultimate field settlement behaviour of the embankment for a wide range of loads. Various loading rates and time-settlement records encountered throughout the projec are shown in Fig. 4. The circuled numbers, in Fig. 4. corresponds to the loading history and related settlements at the locations denoted b the same numbers in the site plan shown in Fig.

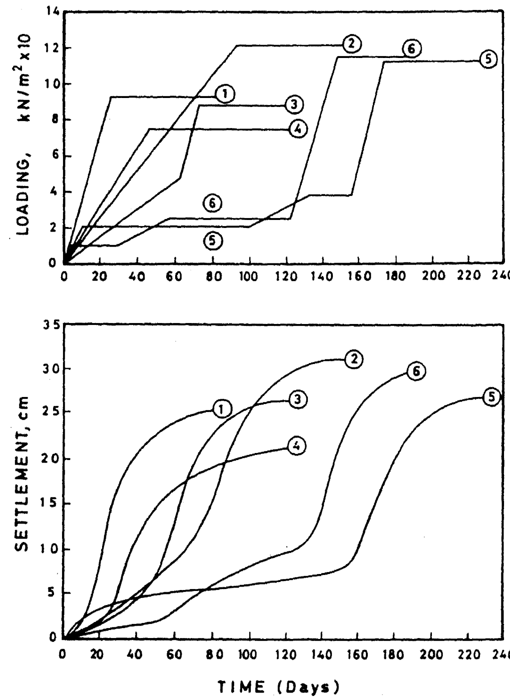

Fig. 4. Loading Rates and Observed Settlements

Deep sounding tests were conducted at 9 locations after the cease of settlements to evaluate the improvement of the subsoil conditions accomplished by the preloading. The average cone penetration resistance of the profile at the en of the preloading period is shown in Fig. 2.

LABORATORY INVESTIGATION

Due to the known difficulties involved in obtaining undisturbed samples from sand and cohesionless silts, oedometer tests were conducted on compacted specimens. Tests performed on few samples recovered in tubes from boreholes revealed unrealistic soil parameters indicating high level of disturbances during sampling.

To resemble the field compressibilities through laboratory tests the following procedure was adopted. Series of cone penetration and oedometer tests were conducted on both the silty sand and the sandy silt soils compacted at various relative densities. The field densities were predicted from laboratory cone penetration resistance versus relative density data presuming that in situ penetration resistances are simulated by the laboratory penetration tests. The deformation moduli to be used in settlement predictions were then estimated from the oedometer test results presented in Fig. 5, as the modulus value corresponding to the estimated in situ relative density and pressure range encountered in the field.

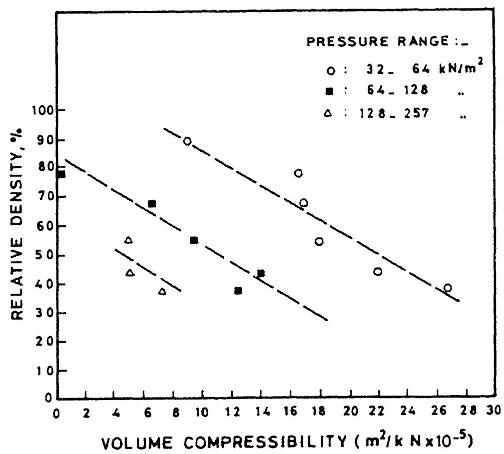

Fig. 5. The Compressibility of the Silty Sand from Oedometer Tests

Although it is not usual to be concerned with time dependent compression of sandy soils, due to the presence of high percentage of silt in the subsoil, an attempt was made to predict settlement-time relationships by using the conventional consolidation theory. The coefficient of consolidations in vertical direction deduced from oedometer tests were as follows:

Soil type:	c_v (10^{-2} cm^2/sec):
Silty sand	4.7 – 24.0
Sandy silt	0.16 – 0.51

PREDICTION OF SETTLEMENTS

The ultimate settlements calculated from oedometer test data and Buisman-De Beers method (Tomlinson, ,1980) based on field cone penetration resistances are summarized in Table I. In Buisman-De Beers method the settlements were evaluated for two different values of soil factors, α, 1.5 and 2.0, for comparison.

The values given in the last two columns of Table I, were predicted by the methods proposed

TABLE I. Settlement Predictions

Station	Load (kN/m²)	Settl. (cm)	Oedometer (cm)	Buisman De Beer α=1.5 (cm)	Buisman De Beer α=2.0 (cm)	Curve fitting. (cm)	Rate dissp. (cm)
1	92	25.4	33.6	52.9	39.7	27.6	26.5
2	122	31.0	44.4	63.8	47.9	32.0	31.9
3	89	26.8	32.4	51.0	38.2	28.2	27.2
4	74	20.6	27.0	44.4	33.3	20.6	21.6
5	39	10.0	14.5	26.3	19.7	10.7	-
5	115	29.4	41.7	61.0	45.8	30.5	31.9
6	35	9.2	14.1	23.8	17.8	11.8	-
6	110	29.0	40.7	58.4	43.9	31.9	-
7	20	6.3	7.3	14.9	11.1	6.1	-
7	112	26.9	40.9	59.8	44.8	27.2	27.6

by Ellstain (1972) and Horn (1983). In the first, curve fitting method, the estimates of time settlement behaviour as well as ultimate settlements are made by fitting a parabola to the initial portion of the measured settlement time records (Ellstain, 1972). Some undergone settlements, preferably more than 20% of the ultimate settlement, must have been observed to have a realistic prediction. Field records of the three stations, stations 1, 5 & 6, extended by using the proposed method are shown on Fig.6. The second method, rate dissipation method, also

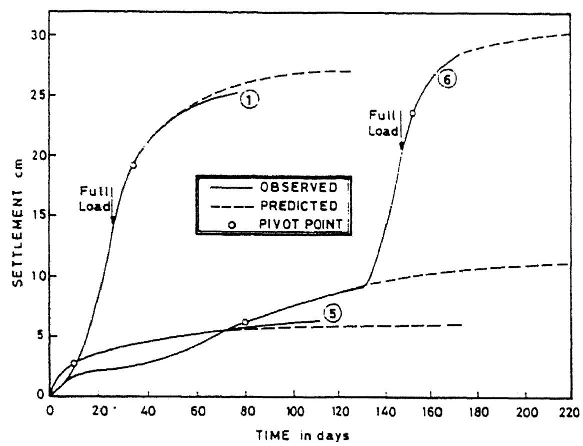

Fig. 6. Settlement Predictions from Curve Fitting Method

utilize the measured settlement-time curves and is based on the premise that, time rate of settlements linearly approaches to zero when plotted against settlements. Then the ultimate settlements are predicted as the settlement intercept of the velocity-settlement plots, as illustrated in Fig. 7. This procedure is slightly different than the method described by Horn (1983). In Table I the final settlement records are the latest measurements made at the end of the preloading period which is proceeded by unloading.

In general the ultimate settlements anticipated by both the curve fitting and rate dissipation

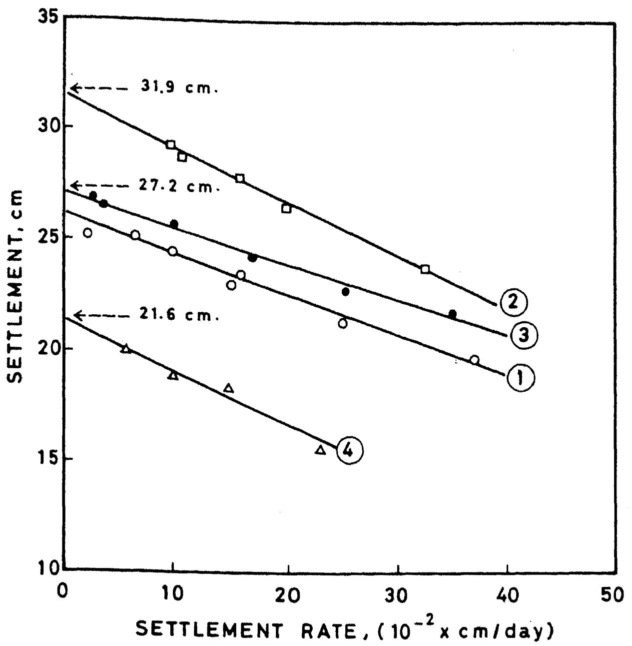

Fig. 7. Estimates of Final Settlements from Rate Dissipation

methods are within ∓ 2% of the final settlement records in all stations, suggesting that real settlements are about to cease at the end of preloading period. This is also indicated by the drastic reduction in the time rate of settlements as shown on Fig. 4. Due to this fact and almost the excellent agreement between the predicted and the measured settlement-time behaviour shown on Fig. 6, it may be considered that the ultimate settlement values given in the last two columns in Table I are the realistic estimates of the field behaviour.

The settlements calculated from oedometer tests are invariably in excess of the measured settlements. On the average, the oedometer data overestimates the settlements 29.5 ∓ 10 percent. This observation possibly reflect non-representativeness of the compacted laboratory specimens in resembling real soil fabric of cohesionless soils (Rowe, 1972).

The settlement predictions from field cone penetration data is found to be highly conservative. The particular method overestimates the settlements 108 ∓ 21% and 56 ∓ 16% for α values of 1.5 and 2.0, respectively. The correct magnitude of soil constant α is found to be in the order of 2.9 for this particular soil conditions as back calculated from the measured settlements. These considerations are in agreement with the generally accepted fact that the particular method gives an upper limit for the anticipated settlements (Sanglerat, 1972).

A plot of the total settlement versus applied surcharge is shown on Fig. 8. It is noted that the actual relationship between the load and the settlement can reasonably be approximated by a straight line. The magnitude of the average constrained modulus, E_s, of the particular soil profile, is found as 7,520 kN/m^2 from the slope of the load-settlement curve on Fig. 8. This value of E_s suggest a relationship between the deformation modulus and the average field cone resistance, q_c, as $E_s = 4.4q_c$ which is in agreement with the correlations given for normally consolidated sands (Chapman and Donald, 1981).

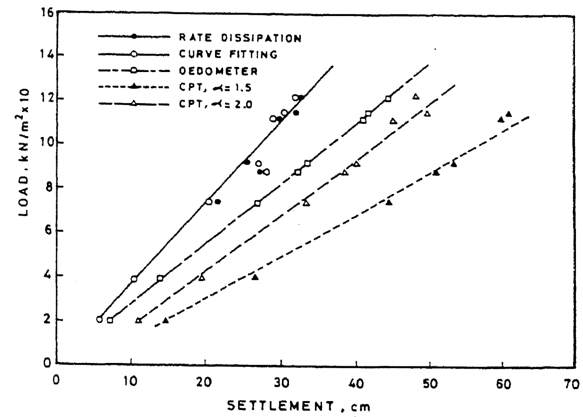

Fig. 8. Load-settlement Behaviour

SETTLEMENT TIME BEHAVIOUR

Conventional consolidation theory is applicable for fine grained soils, and as the soils becomes coarser the theory becomes less valid. Although the theory is not reliable for sands, it does constitute a method that is generally accepted. The application of the theory to the present case was not straight forward because of the following facts. The soil profile consists of two subsequent layers of cohesionless soils to which the conventional theory is not appropriate. The analysis is further complicated by the existance of drains, and various loading rates with occasional intermissions. Therefore the estimated values presented herein is to be considered as an approximation.

Typical results of settlement-time analysis are shown on Fig. 9. The observed settlement in

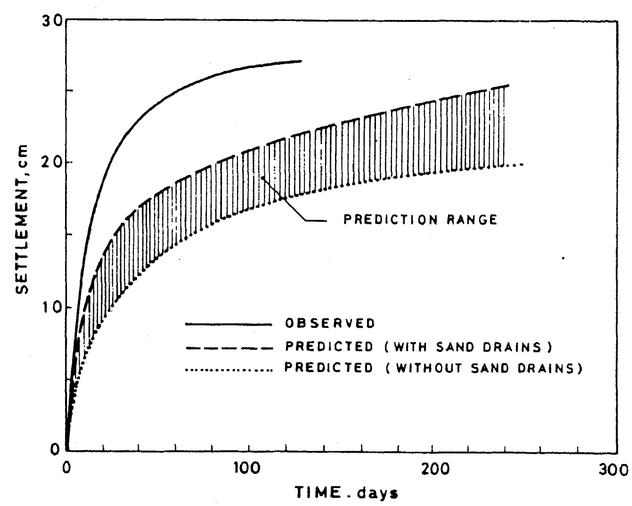

Fig. 9. Settlement Rate Predictions

Fig. 9 were modified to account for the gradual loading encountered during the construction of the fill (Terzaghi, 1943). The upper bound of the prediction range correspond to the case where radial drainage is taken into consideration. For this case the coefficient of consolidation in horizontal direction, C_h, was assumed as twice of that in vertical direction (Ladd et al, 1972).

The discrepancies between real and predicted behaviour in Fig. 9, reflects the uncertainities involved in the settlement-time analysis of cohesionless soils, and illustrate the considerable overestimations of preloading time. On the contrary the predictions based on the method by Ellstain (1972) are in good agreement with the time settlement records, as illustrated in Fig. 6.

The settlement curves of station 5 and 7 fit well to the theoretical shape of the square root time plot, thus it was possible to evaluate the average field coefficient of consolidation. Backcalculations reveal that, C_v values are in the order of 0.170-0.256 cm^2/sec. However, a direct comparison of field and laboratory C_v calues, is not relevant for this case since the field C_v values includes the overall effect of both radial and vertical drainage.

SOIL IMPROVEMENT ATTAINED BY PRELOADING

The comparison of cone resistance before and after the preloading shown on Fig. 2 reflects the level of ground improvement achieved by the preloading. The cone resistance of the silty sand layer increased by about 150 to 300%, whereas only slight increases in cone resistances were detected in the underlying sandy silt. The effect of preloading was not noticed below 14 m depth.

CONCLUSIONS

1. The analysis made by using the soil parameters obtained from oedometer tests does not reveal realistic estimations for either the ultimate settlements or the preloading time.

2. The observed settlements do not compare with the values predicted by Buisman-De Beer's **method based** on field cone penetration resistances. The method was found highly conservative for the presumed values of the soil factor $\alpha = 1.5$ and 2.0. Back calculations from observed settlements suggest an average value of α as 2.9 for the particular soil conditions.

3. The methods, referred as the curve fitting and the rate dissipation methods in this study, give favorable predictions of ultimate settlements. The settlement-time relationship anticipated by the curve fitting method is found to be in good agreement with the settlement records.

4. Comparison of the deformation modulus back calculated from in-situ load-settlement data and the field cone resistances suggests the relationship, $E_s = 4.4\ q_c$.

REFERENCES

Chapman G.A. and I.B. Donald, (1981), Interpretation of Static Penetration Tests in Sands, 10th International Conference on Soil Mechanics and Foundation Engng., Stockholm, Vol.2, 455-458.

Ellstein, A. (1972), Settlement Development: Observed vs. Predicted, Purdue Conf. on Performance of Earth and Earth Supported Structures, Proc. of ASCE 1.2, 1073-1085.

Horn, A. (1983), Determination of Properties of Weak Soils by Test Embankments, Int. Symp. on Soil and Rock Investigations by In-Situ Testing, Paris, Vol.2, 61-66.

Ladd C.C., J.J. Rixner and Gifford D.G., (1972), Performance of Embankments with Sand-Drains on Sensitive Clay, Purdue Conf. on Performance of Earth and Earth Supported Structures, Proc. of ASCE, 1.1, 211-242.

Rowe, P.W. (1972), The Relevance of Soil Fabric to Site Investigation Practice, Geotechnique, Vol. 22, 195-300.

Sanglerat G., (1972), The Penetrometer and Soil Exploration, Elsevier Scientific Publication Comp., 4888 pp.

Terzaghi, K., (.943), Theoretical Soil Mechanics, John Wiley and Sons, Inc.

Tomlinson K., (1980), Foundation Design and Construction, Pitman Advanced Publishing Program, London, 45h Ed., 793 pp.