Kuwait J. Sci. Eng. **28**(1) 2001

Performance of a ring raft for antenna tower on cemented sand in Kuwait

NABIL F. ISMAEL AND HASAN AL-SANAD

Civil Engineering Department, Kuwait University, P.O. Box 5969, Safat 13060, Kuwait

ABSTRACT

The performance of a 55 m diameter ring shaped raft for the new 370 m high antenna tower in Kuwait is monitored with time by an extensive instrumentation program. The raft is embedded at a depth of 18 m on very dense fine to medium partially cemented sand. Different methods of settlement predictions employed during the design and initial construction stages are compared herein with actual measurements. Most of these methods overestimate the actual settlement. Pore pressures and contact soil pressures beneath the raft were also monitored. Based on the site investigation and the in situ tests carried out, the unique and interesting characteristics of these deposits are presented along with their effect on observed foundation performance. Several problems encountered during the design and construction stages are discussed.

Keywords: Cemented sand; deformation modulus; ring raft; settlement; soil pressure

INTRODUCTION

The new telecommunications center in Kuwait comprises a 370 m tall antenna tower, a 61 m high main telecommunications building, a public office building, a common basement, and underground parking facilities (Fig. 1). Construction work started in early 1987 and was scheduled for completion in 1991. However, it was completed in 1996 after a three-year stoppage period due to the Iraqi invasion of Kuwait.

The antenna tower has a concrete shaft, 308 m high, with the top 62 m taken up by a steel mast. The tower is founded on a ring-shaped raft, 55.5 m in diameter and embedded 18 m below the original ground surface, as shown in Fig. 2. Of particular interest is the settlement behavior of this structure and how it may affect the adjacent buildings. The differential settlement across the foundation was specified not to exceed 55 mm, which corresponds to a tilt of 1:1000. Considering that the differential settlement is usually 0.5–0.75 of the total settlement, the maximum settlement should, therefore, be limited to the range of 75–110 min.

The subsoils in the project area consist predominantly of very dense partially cemented sand strata. As very little is known about the characteristics and the geotechnical behavior of these soils, it was necessary to explore the properties and spatial variability of this deposit and their influence on the response to foundation loads.

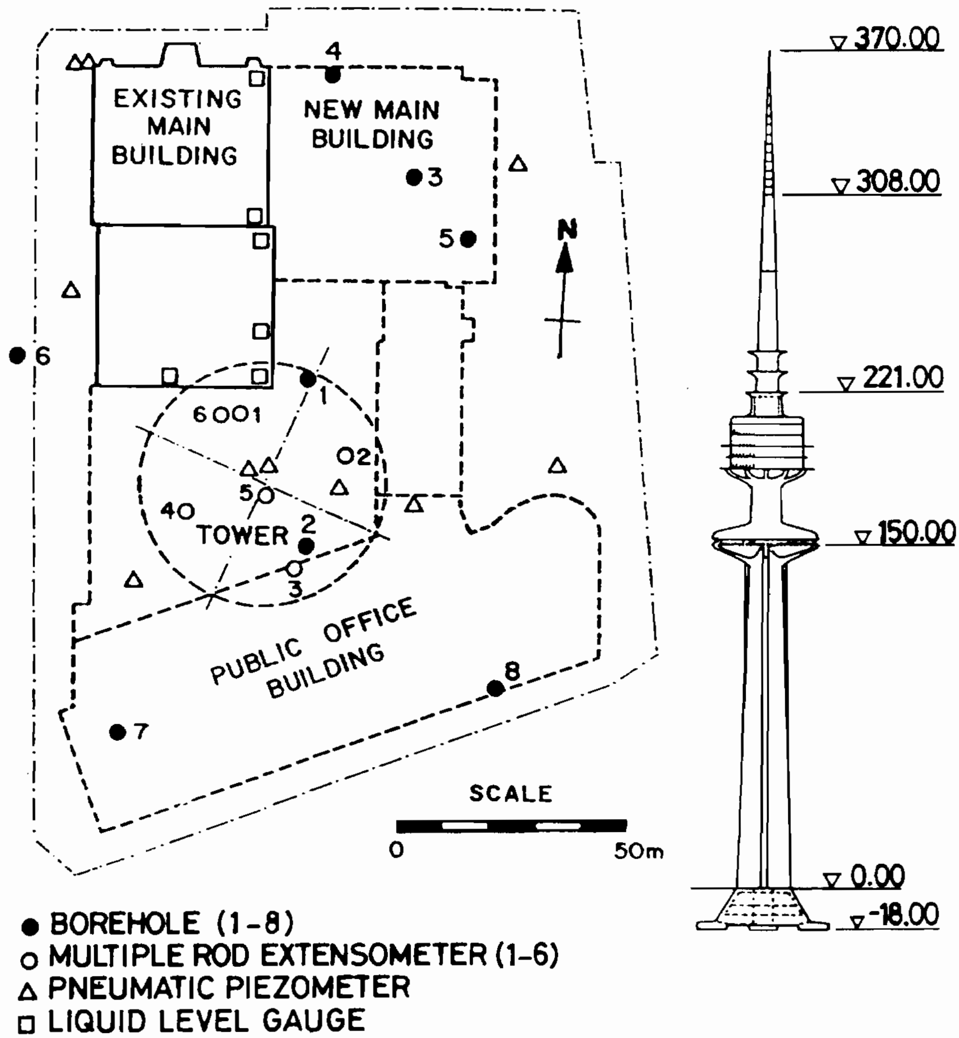

Fig. 1. Tower dimensions and plan of new telecommunications center showing borehole locations and selected instrumentation.

This paper presents the settlement predictions performed at the various project phases and compares them with the results from deformation monitoring. Other measurements including contact earth pressures and pore-water pressures, are also presented and discussed. Emphasis is first placed on the geotechnical characteristics, composition and variability of the soil strata at the site for proper understanding of the performance monitoring data obtained during the various construction stages.

SITE INVESTIGATION AND SUBSURFACE CONDITIONS

Eight boreholes were drilled to depths between 40 and 70 m as shown in Fig. 1 with sampling, standard penetration tests and permeability measurements. Three more

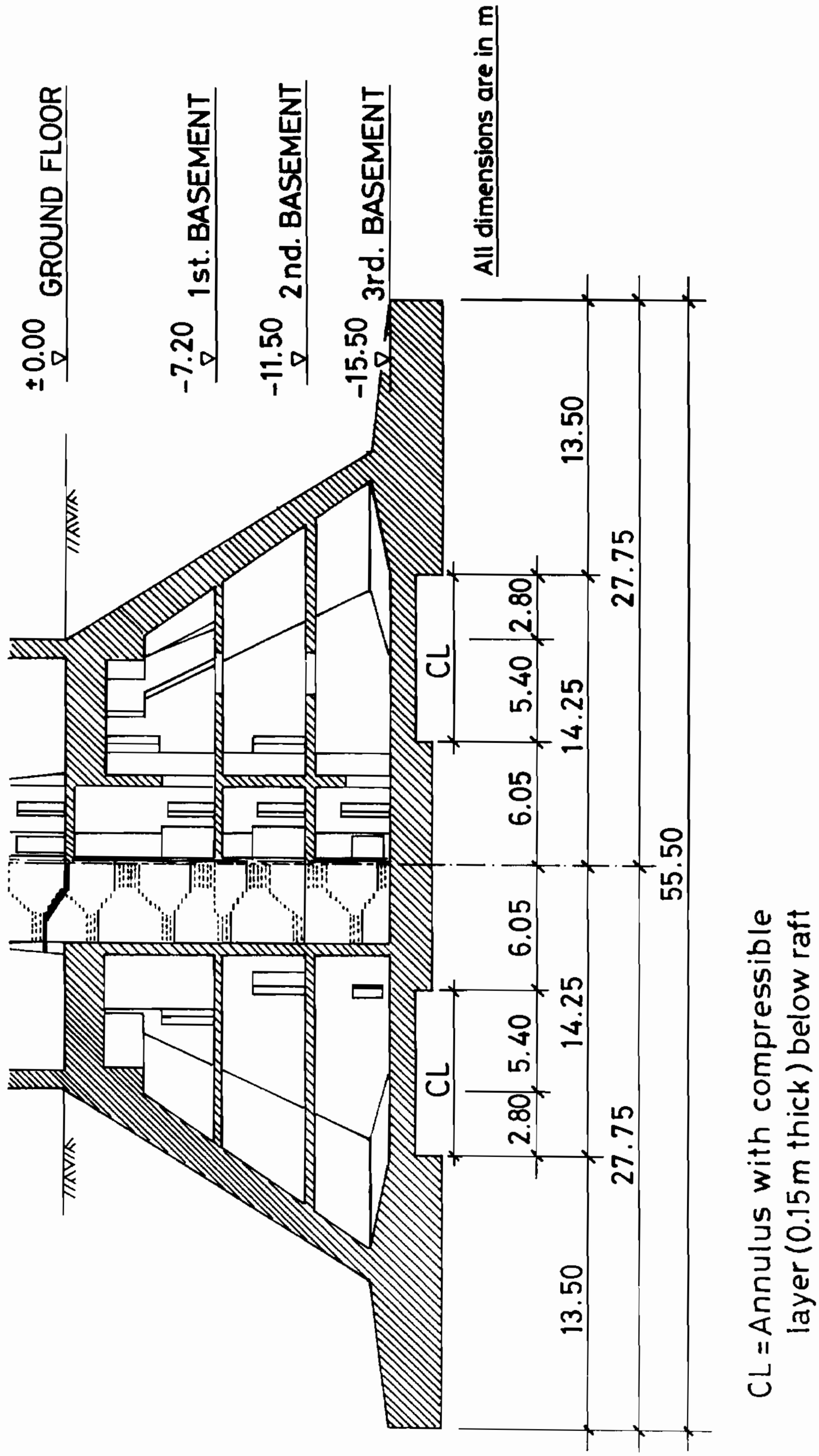

Fig. 2. Cross section of antenna tower foundation.

boreholes were advanced to a depth of 43 m for conducting pressuremeter tests. These test holes (IP, 2P and 5P) were located close to boreholes 1, 2 and 5 respectively. Static cone tests were not possible because of the high soil resistance. Laboratory tests on samples from the boreholes included mineralogical and chemical analyses and the determination of index and engineering properties. Details on the method of boring and sampling, the soil mineralogy, and pressuremeter testing are given elsewhere (Brenner *et al.* 1990).

During construction, when the excavation for the tower foundation had reached its final depth of 18 in, three series of plate load tests were performed. Three sizes of circular plates, i.e., 0.3 m, 0.61 m, and 1.28 m diameter, and a ring plate of 1.28 m outside diameter and 0.68 m inside diameter were employed in each series.

The soil profile at the site consists mainly of dense to very dense calcareous fine to medium sand with little coarse sand and gravel. A special feature encountered is the presence of several layers or horizons of cemented sand which is locally known as "gatch". Moreover, there are other layers of partially or weakly cemented sands where the cementation bonds are not well developed. The cementing agents identified are calcium carbonate, gypsum, and dolomite which are efficient diagenetic cements at certain horizons. Figure 3 shows a summary of the soil conditions at borehole 1 (Al-Sanad *et al.* 1993). Indicated from left to right are the soil description, location of the cemented layers and the degree of cementation as judged by visual examination, bulk unit weight, moisture content, SPT blow count corrected for overburden pressure as per Peck *et al.* (1974) and the soil composition.

Detailed mineralogical analysis of 36 samples from borehole 1 revealed that the major mineral phase in all samples (over 60%) is quartz. The remaining components include calcite (calcium carbonate), dolomite (calcium magnesium carbonate) and gypsum (calcium sulphate). The latter is usually concentrated near the ground surface. Clay minerals were found only in very small or trace quantities which rarely exceeded 5–10% (maximum). The principal component was montmorillonite, with traces of illite and chlorite. Only trace or minor amounts of feldspars were identified.

Although it is premature to make any definite interpretation regarding the sedimentary environment from a single borehole, it is very likely that the sediments originate from a non-marine sequence of deposition. The periodic coarsening and fining nature of the sediments indicate a wadi fluvial system draining the Arabian Shield. The gravel and pebble layers represent a flood water high energy stage in the wadis, with the coarse fraction dropping in place as the velocity decreased. The fining upward of the sediments represents the low energy stage of the wadi channels. This agrees with the recent findings of AI-Sanad and Shaqour (1990) which confirmed that both cemented and uncemented sand layers are of fluvial origin.

From Fig. 3 it is evident that penetration resistance of the subsoil was high throughout the depth tested. The blow count value, N (from SPT), obtained in the eight boreholes usually exceeded 50 and often it was not possible to drive the spoon the required 0.45 m, especially in the regions of cemented horizons. For settlement analysis of structures on sand, the penetration data were therefore of limited use only, because design procedures employing SPT blow count values do not usually consider values greater than 50.

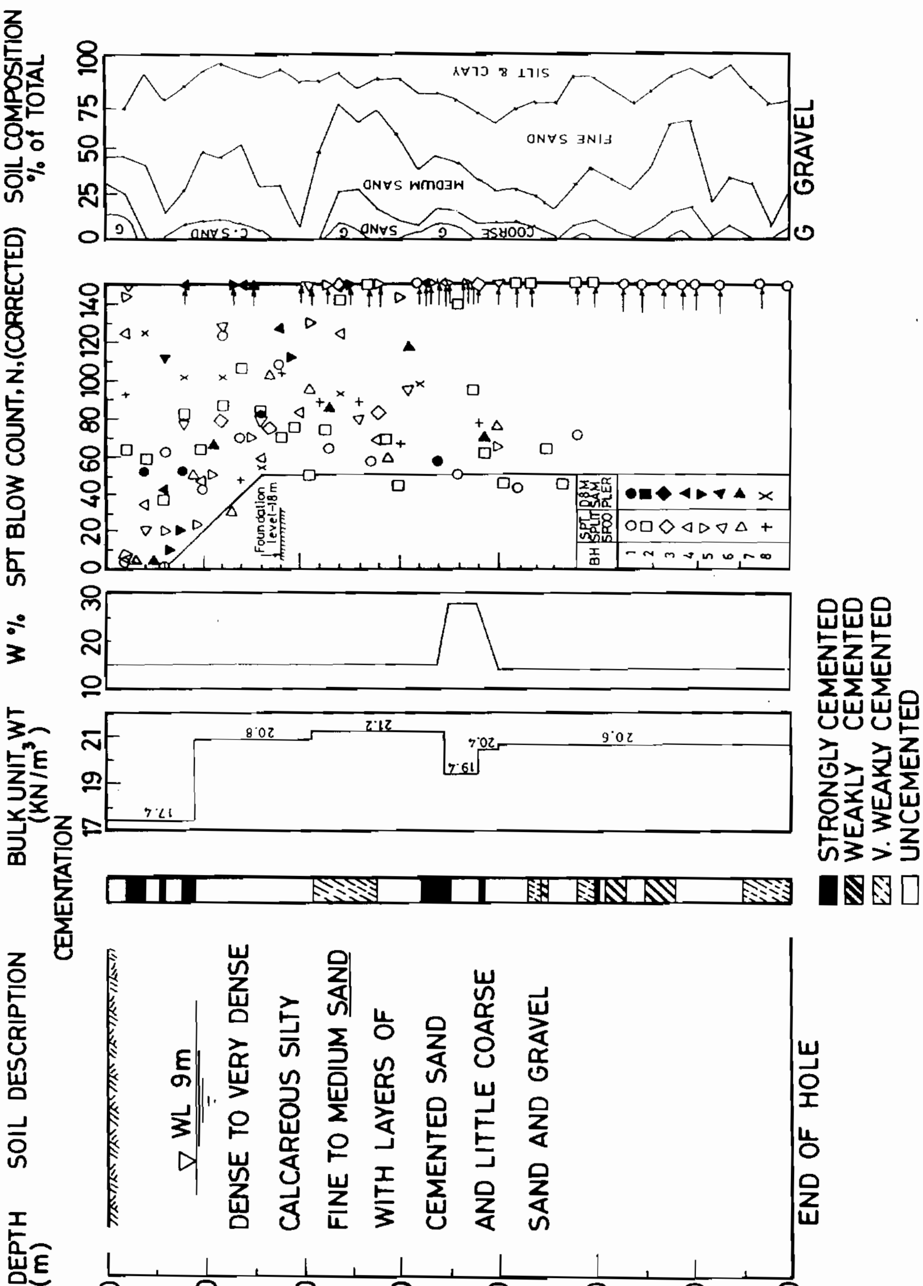

Fig. 3. Soil conditions at the test site (Al-Sanad *et al.* 1993).

Deformation moduli, E_M, obtained from pressuremeter tests are shown in Fig. 4. The data were limited however, to a depth of 43 m. It is evident that the pressuremeter moduli generally increase with depth and fall within the range of 30 to 50 MPa at the foundation level. With a cyclic test, a reloading modulus which was about four times higher than the virgin modulus was measured. The range of moduli derived from plate load tests is also indicated in Fig. 4. In those tests the reloading modulus was found to be about 2 to 2.5 times larger than the virgin modulus.

The unit weight and moisture content profiles shown in Fig. 3 are average values obtained from the eight boreholes. The bulk unit weight measured by the sand cone at the foundation level ranged between 19.8 and 20.7 kN/m^3, and the moisture content was typically 8–10%. The relative densities determined at the same level ranged between 85–93%. The average angle of internal friction obtained from drained triaxial compression tests was 35° with a cohesion intercept in the cemented layers ranging from 4 to 70 kPa.

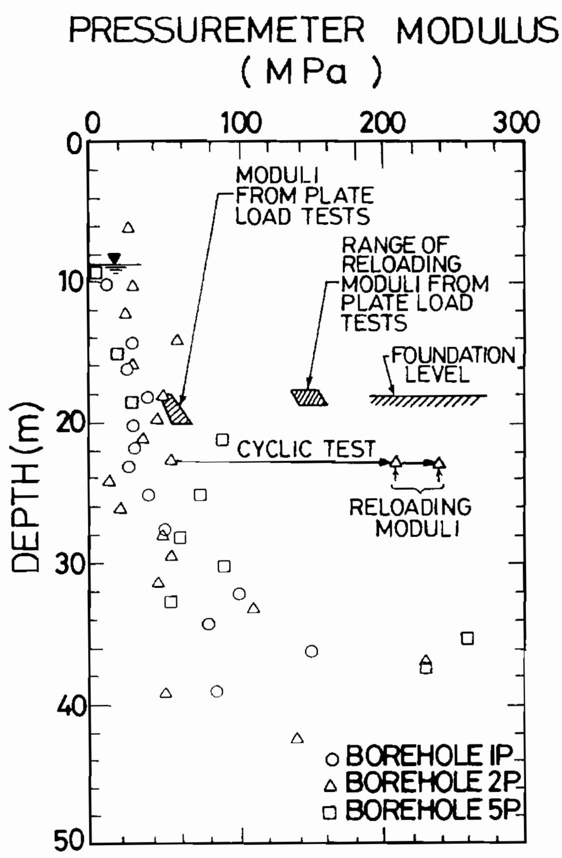

Fig. 4. Variation of pressuremeter moduli with depth and range of moduli obtained from plate load tests (Brenner *et al.* 1990).

Table 1 presents a summary of all relevant data obtained from odometer tests. It is clear that the compressibility is very low with the compression index below 0. 1 for all samples and the term $C_c/(1 + e_o)$ limited to a maximum of 0.05. It can be seen that for the samples originating from the silty-clayey zone between 34 and 46 m, the compressibility is distinctly higher. This is caused by the greater proportion of fines present in these samples.

From the preceding results, it is evident that cemented sand deposits are nonhomogeneous with cemented and uncemented or partially cemented bands or layers. Nevertheless, the soil stiffness increases rapidly with depth as demonstrated by the pressuremeter test results.

The groundwater table in this area varies between about 7.50 and 9 m from the ground surface. Open hole permeability tests were carried out at selected depths in the boreholes. A summary of the results is given in Table 2. The coefficient of permeability decreases with depth. Above 18 m depth, i.e., within the depth range of interest for groundwater table lowering during excavation and foundation construction, the permeability coefficient is typically around 10^{-3} cm/sec. The lowest values ($\sim 5 * 10^{-6}$ cm/sec) occur in the silty fine sand layers between 35 and 40 m depth.

In order to carry out foundation work for the tower and the other buildings, the groundwater table had to be lowered to at least 2 m below the corresponding foundation level ($\sim$10 to 11 m of dewatering). The increased stiffness and the presence of cemented layers with depth caused some problems. The contractor proposed to achieve this lowering by a series of 6 deep wells to a depth of 38 m arranged along the perimeter of the excavation and connected by a 340 mm ring-shaped discharge pipe. However, with this arrangement it was not possible to draw the water table down lower than -14 m. A revised plan was carried out to install an outer ring of sand drains which would penetrate the gatch layers and thus increase the flow in the vertical direction. In addition, five additional wells to a depth of 38 m were installed bringing the number of deep wells to eleven. The arrangement of deep wells, sand drains, and other dewatering facilities is shown in Fig. 5. The first well was installed in April 1987 but the final (target) drawdown level of about -20 m was reached only in March 1988 after many delays and modifications to the original design plan.

INSTRUMENTATION

Many uncertainties remained after site investigation with respect to load-deformation behavior of the subsoil. Moreover, there is a lack of case studies documenting the behavior of cemented sand deposits under heavy foundation loads. For these reasons a comprehensive monitoring program was specified. The devices installed included: (1) six multiple rod extensometers for monitoring ground displacements below the tower foundation, (2) six liquid level gauges in the basement of the existing main building to sense vertical movements during construction, (3) thirteen hydraulic pressure cells to measure the contact pressure at the base of the tower foundation, (4) twelve pneumatic piezometers at depths of 15, 25 and 40 m, (5) stand pipe piezometers to monitor dewatering, (6) six inclinometer tubes

Table 1. Summary of consolidation test results

Test No.	BH No.	Depth (m)	Void ratio e_o	Average compression index C_c	Average swell index C_s	$\frac{C_c}{C_s}$	$\frac{C_c}{1+e_o}$	Method of sampling*	Blow count
A12	5	4.0	0.394	0.052	0.014	3.7	0.040	D+M	-
A9	4	4.2	0.553	0.054	0.010	5.4	0.035	D+M	11-14-18
A15	6	6.2	0.421	0.045	0.011	4.1	0.026	D+M	17-40-50/8
A13	5	7.5	0.523	0.034	0.012	2.8	0.022	D+M	9-14-9
A1	1	8.0	0.380	0.042	0.012	3.5	0.030	D+M	9-24-36
A10	4	14.0	0.475	0.023	0.0085	2.7	0.022	D+M	25-50/11
A2	1	16.1	0.349	0.037	0.010	3.7	0.028	D+M	21-43-50/9
A11	4	18.0	0.465	0.023	0.0095	2.4	0.014	D+M	37-50/10
A16	6	18.0	0.517	0.041	0.012	3.4	0.030	D	-
A3	1	20.6	0.543	0.044	0.013	3.4	0.029	D	-
A8	3	21.0	0.388	0.039	0.010	3.9	0.028	D	-
A20	8	24.0	0.435	0.048	0.011	4.4	0.035	D+M	33-50/12-50/9
A19	7	27.0	0.348	0.026	0.0085	3.1	0.019	D+M	48-50/8-50/7
A14	5	32.6	0.292	0.048	0.014	3.4	0.037	D	-
A17	6	33.0	0.383	0.064	0.021	3.0	0.045	D	-
A4	1	34.15	0.335	0.062	0.013	4.8	0.049	D+M	44-48-50/11
A18	6	38.0	0.615	0.094	0.026	3.6	0.057	D	-
A5	1	39.1	0.612	0.082	0.014	5.9	0.050	D	-
A6	1	45.6	0.389	0.067	0.016	4.2	0.045	D	-
A7	1	59.0	0.310	0.031	0.008	3.9	0.024	D+M	50/12-50/6-50/3

*D + M = Dames & Moore sampler; D = Dension sampler

Table 2. Coefficients of permeability measured in situ

Borehole	Depth (m)	k (cm/sec)
1	25.5	8.2×10^{-5}
	25.0	$7.2 \times 10^{-5*}$
2	20.0	1.1×10^{-4}
	30.0	5.2×10^{-6}
3	14.0	1.1×10^{-3}
	40.0	5.8×10^{-6}
4	12.0	4.7×10^{-4}
	18.0	3.4×10^{-4}
5	21.5	8.0×10^{-3}
	32.0	3.4×10^{-4}
6	14.0	3.0×10^{-4}
	28.0	1.5×10^{-3}
7	17.0	1.3×10^{-3}
	27.0	2.7×10^{-4}
8	23.0	5.9×10^{-4}
	36.0	5.2×10^{-6}

* Open hole test

around the tower excavation, and (7) twelve anchor load cells in the shoring system of the excavation. The instruments selected were all supplied and installed under the supervision of Interfels GmbH, Bad Bentheim, Germany.

Figure 1 shows the locations of the extensometers, pneumatic piezometers, and liquid level gauges. In this paper the presentation of monitoring results is focused on extensometer records and settlement obtained from geodetic surveying. The extensometer rods were anchored at 20, 32, 42 and 70 m depth below the original ground surface with the 70 m point used as reference.

PREDICTION OF SETTLEMENT

Final settlements of the tower were first predicted during the design phase based on SPT blow count values and pressuremeter moduli (Brenner *et al.* 1991). Four methods were applied, namely:

1. A method based on Standard Penetration Tests following the relationship proposed by Peck and Bazaraa (1969). The ring raft was modeled as a strip footing of 13.5 m width and the applied pressure was 456 kPa. The effective overburden pressure at 18 m depth was estimated as 256 kPa. For the blow count, N, a value of 50 was selected.
2. A method based on elastic theory considering the deformation of a layer of finite thickness with elastic moduli derived from SPT results based on a relationship recommended by Sherif (1973).
3. A method developed by Menard (1971, 1975) to calculate settlements from pressuremeter test data.

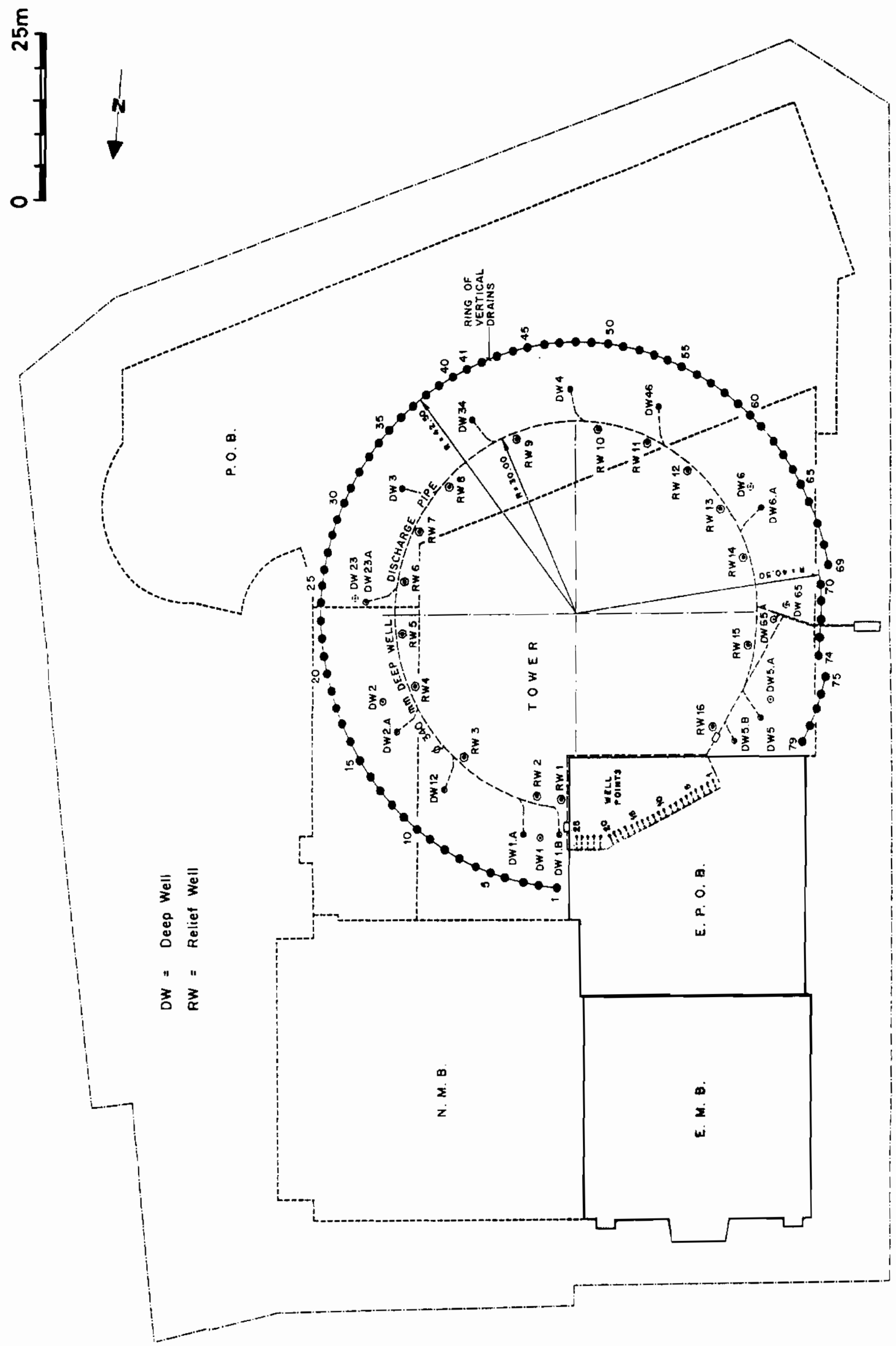

Fig. 5. Layout of dewatering facilities including deep wells, sand drains, well points and relief wells.

Table 3. Predicted final settlement for tower

Method	Test from which soil parameters were derived	Predicted final settlement (mm)
Peak & Bazaraa	SPT ($N = 50$)	35
Sherif	SPT ($N = 50$)	89
Menard	Pressuremeter	25
Linear Elastic Finite Element	Pressuremeter	88

4. A method using a soil-structure interaction finite element approach employing the computer code ANSYS. The soil was modelled as a linear elastic medium with a constant modulus. In the reloading range, the modulus was taken as four times the value of the modulus in the virgin range. The values of the moduli were selected from the results of the pressuremeter tests. The settlement calculated by the four approaches is summarized in Table 3.

Additional settlement predictions were performed during construction when the results of the plate load tests became available. It is possible to make use of Fig. 6 which has been established on the basis of tests with circular and square plates and footings. The width ratio of the antenna tower (diameter = 55 m) with respect to the 300 mm plate is 183. In view of the high density of the soil measured by means of various tests, it is reasonable to assume that the settlement ratio would be represented by a curve which lies below the average curve of Bjerrum and Eggestad (1963) shown in Fig. 6. This average curve may, in the present case, be considered an upper bound.

Extrapolating now from the average curve for the width ratio $B/B_1 = 183$ yields a settlement ratio $S_B/S_1 = 15$. The settlements of the 1280 mm plate at an applied pressure of 456 kPa were 5.3, 6.5 and 6.2 mm, respectively, for series I, II and III. Hence, an upper bound of the foundation settlement is estimated to be in the range of 80 to 100 mm which is the same order of magnitude as predicted during the preconstruction phase. Assuming a curve in Fig. 6 corresponding to a relative density higher than 35% considerably reduces the expected range of settlements.

An elastic analysis was performed after the soil stiffnesses were adjusted, i.e., the virgin (E_i) and reloading (E_R) moduli were somewhat increased and decreased, respectively. The E_i values ranged between 70 and 160 MPa and the E_R values between 100 and 250 MPa. The program SETTL/G was employed for analysis. It is based on an assumed vertical stress distribution in an elastic medium and computes the vertical displacements under uniformly loaded areas. The settlement of the tower computed by this approach amounted to 115 mm, not taking into account the rigidity of the foundation.

SETTLEMENT FROM EXTENSOMETERS AND GEODETIC LEVELING

Settlement data from the extensometers are plotted in Figs. 7 to 9. These measurements were made at a depth of 42 m, 32 m, and 20 m from the original ground level. During excavation, the displacements were read by means of a dial gauge. Before the concrete of the raft was poured, the extensometer heads were fitted with

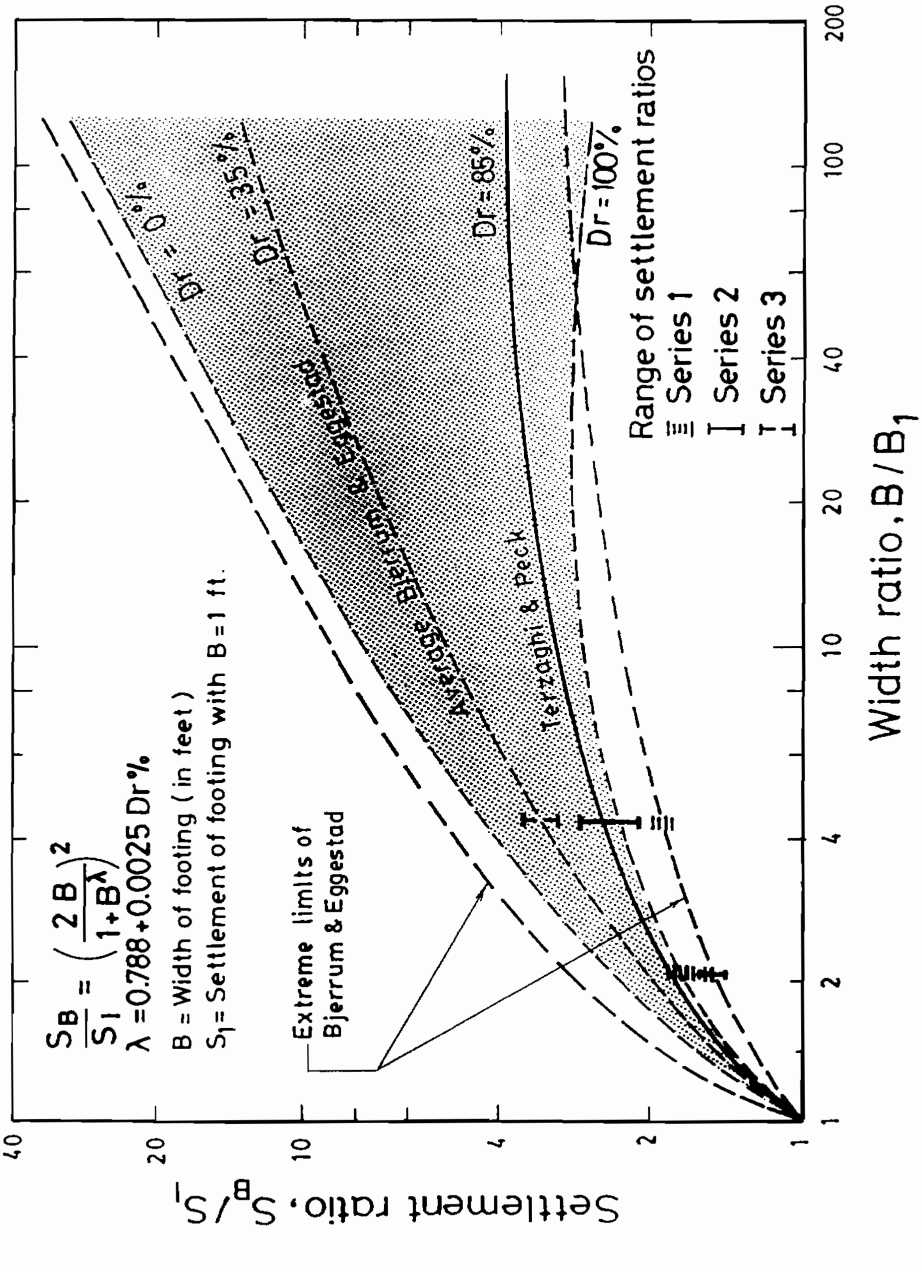

Fig. 6. Influence of relative density on settlement ratio with present test results (Arnold 1980).

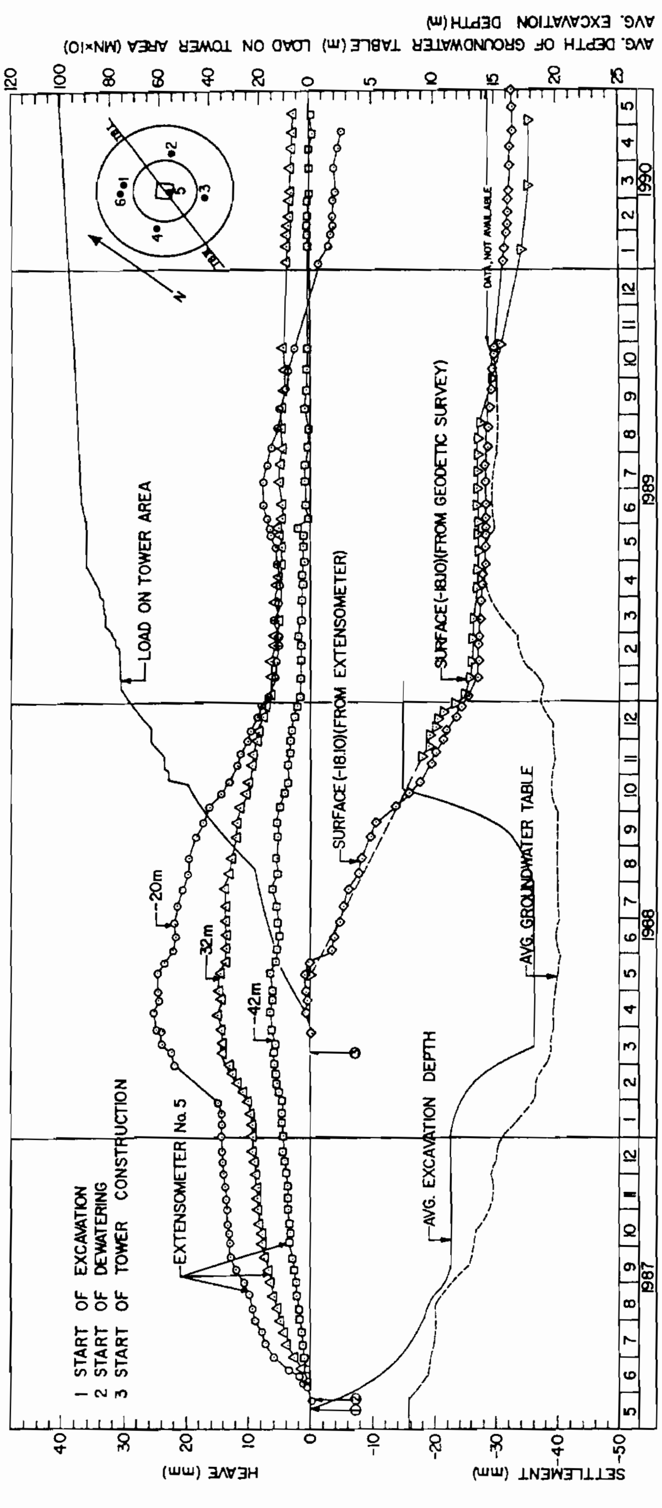

Fig. 7. Time variation of: displacements measured by extensometer no. 5, tower settlements determined by geodetic survey, average excavation depth, average groundwater table position, and load on tower foundation.

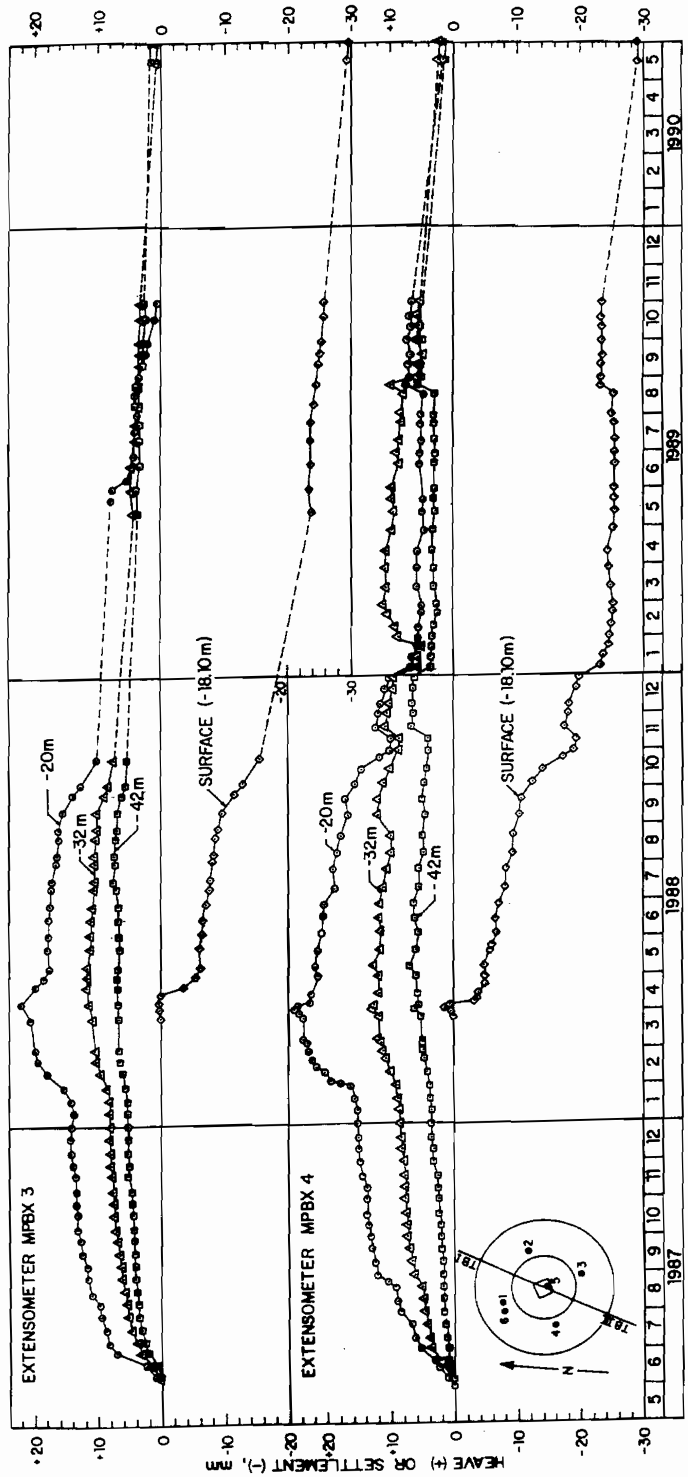

Fig. 8. Time variation of vertical displacements measured by extensometers no. 3 and 4.

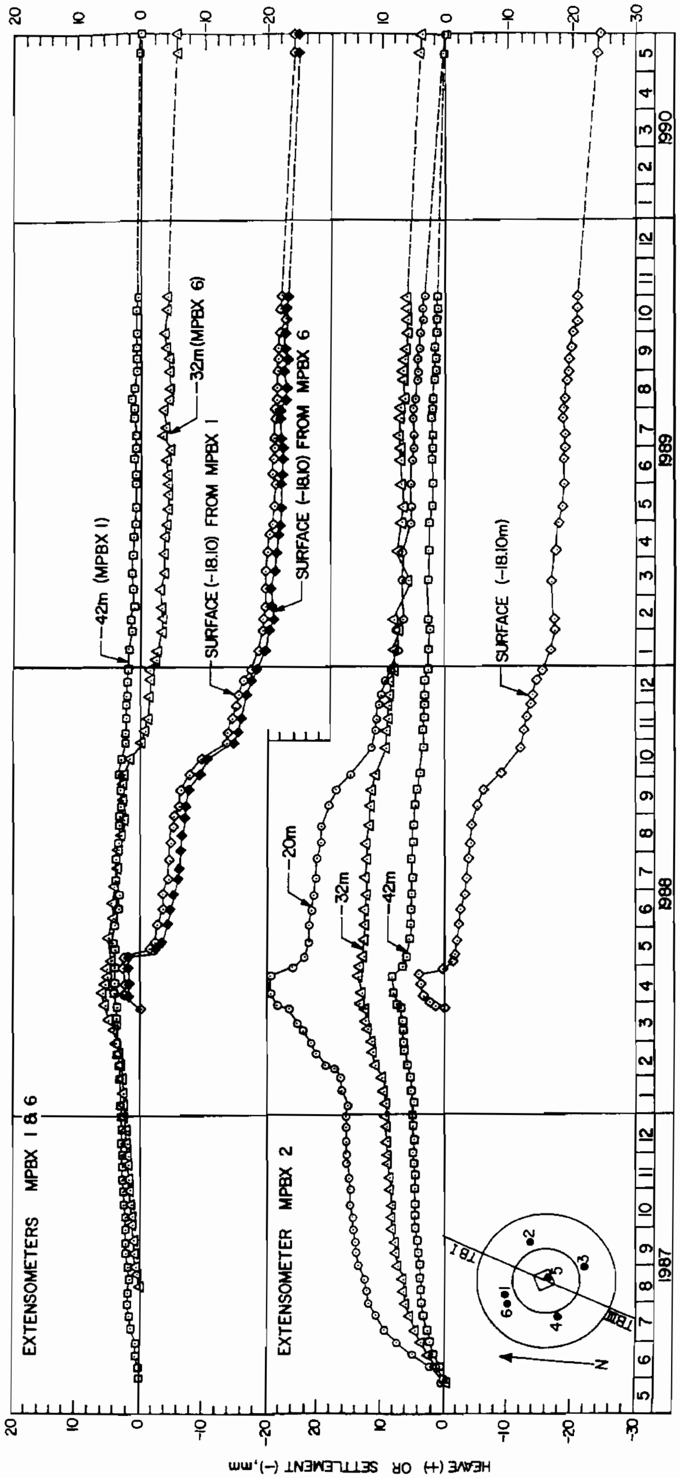

Fig. 9. Time variation of vertical displacements measured by extensometers nos. 1, 6 and 2.

transducers and protected by a concrete box. Also given in Fig. 7 is the progress of excavation and the average position of the groundwater table as measured by standpipe piezometers. During excavation the extensometer points indicated heave, which at 2 m below the bottom of the excavation (20 m from the original ground surface) reached a maximum of 25 min at extensometer 5. Heave still continued for some time when the blinding concrete was poured and the reinforcing steel placed. The load application due to tower construction is also shown. It includes the partial backfill around the conical foundation structure. With the transducer installed, it was possible to determine surface settlements (i.e., the compression of the layer between the base of the excavation (−18 m) and the reference point at (−70 m). In addition, selected points of the tower foundation were surveyed geodetically. The average settlements of these points are plotted and they agree well with the extensometer 5 measurements.

From Fig. 7 it is evident that the largest vertical displacements took place by the end of 1988. Then the rate decreased markedly, because pumping of some of the deep wells was discontinued and the groundwater table rose, which caused a decrease in effective stress. By the end of October 1989, the total settlement, including recompression, had reached about 30 mm with nearly 90% of the final load applied (the total weight of the tower is estimated as 1100 MN). The final settlement after application of the full load as measured on 30/7/95 was 40 mm. This is the average geodetically measured value from four points on the tower raft which recorded settlements of 38, 42, 41 and 41 mm. These points are located in the vicinity of extensometers 4, 3, 2 and 1/6 respectively. The extensometer records demonstrate that the contribution to settlement by the soil strata below about 40 m depth from the original ground surface is very small.

By comparing the data from the different extensometers, the following interesting features are observed:

1. The amount of heave varied between about 22 and 27.5 mm.
2. The surface settlement (at level −18 m) under the ring attained at the end of May 1990 varied between 24 and 30 mm. The surface settlement at the center (extensometer 5) was 36 mm. This difference is somewhat unexpected. The average settlement of the four geodetic bolts agrees extremely well with the surface settlement obtained from extensometer 5.
3. Extensometer records are not available after 1990, but the geodetically measured settlements of the four points at the end of July 1995 given above differ by a maximum, of 4 mm. According to these measurements the tower foundation appears to tilt slightly towards SSE. This direction coincides more or less with the predominant wind direction in Kuwait.

The settlement records reveal that the predicted settlements from elastic analysis were considerably overestimated. In order to illustrate the main reasons, the settlements in the virgin range has been analyzed using a simplified approach. The virgin range starts when the load has reached about 500 MN, which occurred around the middle of October 1988. Extensometer records are now considered from October 15, 1988 to January 23, 1989. During this time interval the water table remained more or less constant and below the level of the tower foundation. On January 23, 1989 the total load had increased by about 290 MN. The calculated settlement for a load increase from 500 MN to 1100 MN in the virgin range

Table 4. Settlements between 15/10/88 and 23/1/89

Depth (m)	Measured from extensometer 5 (mm)	Calculated assuming a flexible foundation (mm)
18–20	3.3	2.5
20–32	4.2	11.2
32–42	1.9	4.5
42–70	2.3	7.8
> 70	–	9.8

Table 5. Back-calculated and predicted moduli for virgin loading

Depth (m)	$E_{i_{av}}$ Back-calculated from extensometer 5 data (MPa)	$E_{i_{av}}$ Assumed for settlement prediction (MPa)
18–20	29	70
20–32	142	70–120
32–42	250	120–135
42–70	433	135–140

was 74.0 mm. Assuming now, for the purpose of an approximate analysis, a linear relationship between load and settlement, the settlement for a load increase of 290 MN would be 35.8 mm. Table 4 gives a comparison between measured and computed (from elastic analysis) settlements for the various soil layers which could be sensed by the extensometer rods.

The settlement at the center of a rigid foundation is about 85% of that for a flexible foundation. Elastic analysis produces a settlement below 70 m depth which is about 27% of the total settlement. Hence, assuming a rigid foundation and considering only the layers between 18 and 70 m depth, a calculated settlement of 22.1 mm, is obtained which is about twice that observed.

From the observed strains the average modulus for each layer can be computed. The corresponding stresses were estimated from Butterfield & Banerjee (1971) for a rigid disc embedded in a half-space. Table 5 presents the back-calculated values together with the values assumed in the predictions. The deformation moduli at greater depths were considerably underestimated. Below 70 m the strata can, for practical purposes, be considered as rigid.

CONTACT PRESSURE MEASUREMENTS

The records obtained from the earth pressure cells are presented in Fig. 10. From these graphs the following observations can be made:

1. The highest pressures were recorded by cells EP6, EP9 and EP13. EP13 is located below the central pedestal of the raft and is expected to yield the highest pressures.
2. The development of the contact pressures follow the loading history of the tower raft without any phase lag.

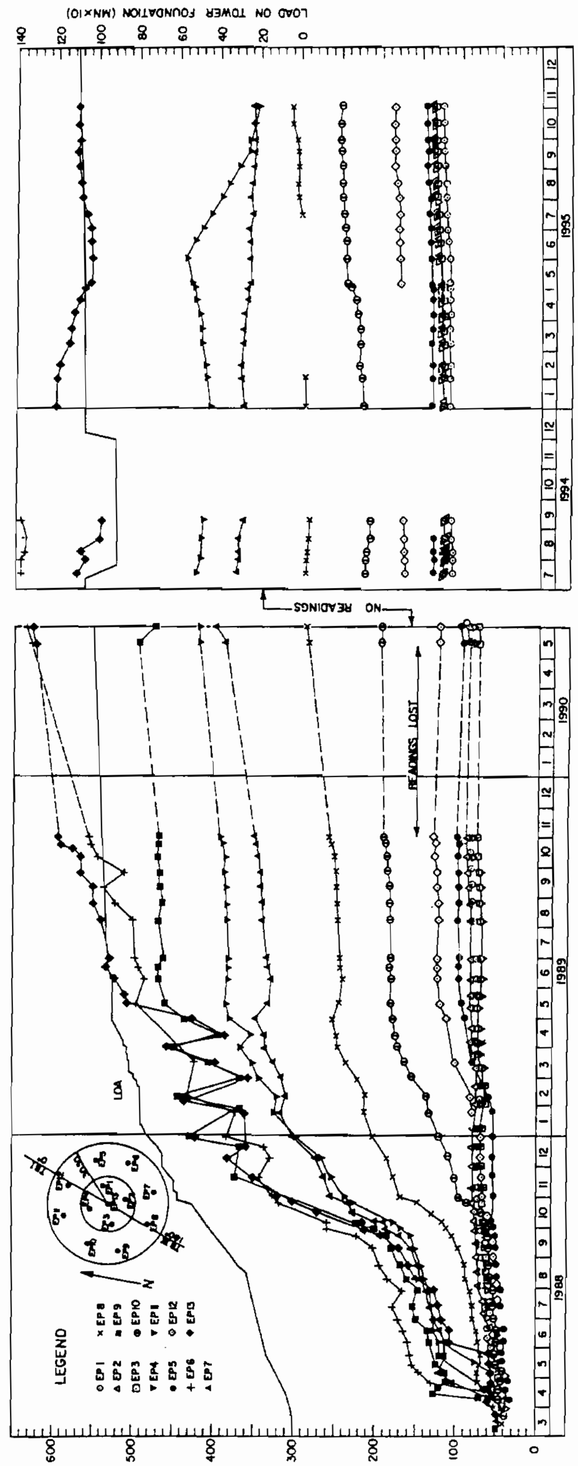

Fig. 10. Variation of contact pressures below tower raft with time as measured by earth contact pressure cells.

3. The cells, EPI to EP4 which were placed under the inner annulus with a compressible layer under the raft (area CL on Fig. 2), indicated very low pressures, which are in the order of 70 to 80 kPa increasing after liberation to about 120 kPa. These measurements demonstrate that the load is transferred to the soil mainly through the outer ring of the raft.
4. The variation of the contact pressure along the ring foundation of the raft shows some scatter and doesn't seem to be related to the differential settlements observed. However, high values are associated with low values on the diametrically located cell. For example, cells EP9 and EP6 indicate high pressures, whereas the corresponding diametrically located cells EP5 and EP1O indicate low values. Average pressures are typically 400 to 450 kPa. It can be inferred that the majority of the cells produced reliable data.
5. The contact pressures measured during 1995 are more or less stationary. Small variations may be associated with groundwater fluctuations.

Averaging the pressures recorded on the gauges yields a measured load of about 762 MN which is only 70% of the estimated total load. Other users of pressure gauges, skeptical of the results they give, have reported similar experiences (Weiler & Kulhawy 1978). Using the average of the pressure recorded by the cells in Fig. 10, the load transmitted to the ring raft is about 614.5 MN, the load transmitted by the central pedestal is about 69 MN and the load transmitted by the annulus is 78.5 MN. This confirms the preceding statement that the load is transferred to the soil mainly through the outer ring of the raft.

PORE WATER PRESSURE CELLS

Excess pore water pressure induced by loading of the ground by tower construction were insignificant. The pressure cells responded mainly to the groundwater table variations. Future settlements due to excess pore water pressure dissipation can, therefore, be excluded.

DISCUSSION

Monitoring by multiple rod extensometers enabled the researchers to follow the development of vertical displacements below the tower foundation and to ensure that they were both uniform and within the specific tolerable limits. The data revealed that the increase in soil stiffness with depth was much more than originally expected from the results of site investigation. Hence, the predicted settlements were too high. A similar increase in stiffness with depth has also been recorded for raft foundations on the bouldery clay of Singapore (Wong *et al.* 1996). Although the deformation modulus could be determined at the bottom of the excavation from plate load tests, the increase with depth had to be largely inferred based on available information from the literature and from pressuremeter test data (Fig. 4), but the latter were available only to a depth of 24 m below the foundation level.

Unlike the elastic methods requiring modulus values, the empirical method based on SPT values predicted a final settlement of only 35 mm which is close to the actual settlement of 40 mm. Considering the variability and irregular cementation of the soil fabric, and the fact that most SPT values were larger than 50, this agreement is considered fortuitous.

The subsoil was found to be competent and very dense, however, there was no previous experience in the area of the project with large-sized embedded rafts, and there was a tendency to use conservative parameters for settlement prediction. In such a case construction monitoring is of great help. The monitoring results enable the designer to make corrections to his prediction at a relatively early stage. This, in turn, will generate a better overall control of the structure and of possible adjacent buildings.

CONCLUSIONS

Comprehensive site investigation and monitoring programs which were initiated at the Antenna tower site in Kuwait, allowed the researchers to explore ground conditions and to check on the appropriateness of the design assumptions and on the accuracy of the predictions made. Based on the test results and on the monitoring data the following conclusions are made:

1. Cemented sand deposits are not homogeneous. Cemented, partially cemented and uncemented layers or bands were encountered with depth. Regardless of the degree of cementation the soil increased in stiffness sharply with depth.
2. Deformation (settlement or heave) of the ground below the tower raft was quite uniform. Differences were in the order of a few millimeters.
3. Excavation of the tower foundation caused a considerable amount of heave amounting to 25 to 30 mm next to the ground surface (at −18 m). The recompression of the heave movement accounted for 60 to 75% of the total settlement.
4. The recorded settlements due to loading by the tower structure were in the order of 40 mm with respect to the condition of the ground after excavation. This was well below the predictions based on elastic methods because of underestimation of the selected moduli values with depth.
5. The ground differed in its response to unloading, reloading and virgin loading. The reloading stiffness was highest while virgin loading gave the lowest stiffness.
6. Dewatering activities including the lowering and subsequent restoration of the water table had a pronounced effect on ground deformations under tower loading. The rate of settlement decreased as the groundwater table rose causing a decrease in the effective stress.
7. Contact pressures measured at the base of the raft indicated some non-uniform distribution. The variations from the mean value were 40% along the ring foundation. The measured values remain, however, stationary and should be of no concern.
8. Pore water pressure cells responded only to ground water table variations. Future settlements due to excess pore water pressure dissipation can, therefore, be excluded.

ACKNOWLEDGMENTS

The writers wish to thank the Ministry of Public Works, Kuwait for granting permission to publish this paper.

REFERENCES

Arnold, M. 1980. Prediction of footing settlement on sand. Ground Engineering **13(2)**: 40–49.

Al-Sanad, H.A., Ismael, N.F. & Brenner, R.P. 1993. Settlement of circular and ring plates in very dense calcareous sand. Journal of Geotechnical Engineering, ASCE **119(4)**: 622–638.

Al-Sanad, H.A. & Shaqour, F. 1990. Effect of groundwater level changes on the engineering properties of desert sands in Kuwait. Final Report Project No. EV036, Kuwait University Research Administration, Kuwait.

Bjerrum, L. & Eggestad, A. 1963. Interpretation of loading tests on sand. Proceedings of 3rd European Conference on Soil Mechanics and Foundation Engineering. Wiesbaden, Germany **1**: 199–203.

Brenner, R.P., Kunzi, B., Jeragh, A.M. & Gunatilaka, A. 1990. Site investigation for tower structure on ring-shaped raft on sand. Proceedings of 6th International IAEG Congress, Amsterdam **3**: 1813–1820.

Brenner, R.P., Wetterwald, V. & Jedelhauser, P. 1991. Ground displacements caused by antenna tower on ring-shaped raft. Proceedings of 10th European Conference on Soil Mechanics & Foundation Engineering, Florence **1**: 337–340.

Butterfield, R. & Banerjee, P.K. 1971. A rigid disc embedded in an elastic half space. Geotechnical Engineering **2(l)**: 35–52.

Menard, L. 1971. Le tassement des fondations et les techniques pressiometriques—Bilan apres dix ans de resultats experimentaux. Annales de l'Institut Technique du Batiment et des Travaux **288**: 105–124.

Menard, L. 1975. Interpretation and application of pressuremeter test results. Sols-Soil **26**: 1–43.

Peck, R.B. & Bazaraa, A.R.S. 1969. Discussion of settlement of spread footings on sand. Journal of Soil Mechanics and Foundations Division, ASCE **95**: 905–909.

Peck, R.B., Hanson, W.E. & Thornburn, T.H. 1974. Foundation Engineering. 2nd edition, John Wiley and Sons, New York, NY, USA.

Sherif, G. 1973. Setzungsmessungen an industrie und hochbauten und ihre auswertung. Mitt. Inst. Verkehrswasserbau, Grundbau und Bodenmechanik (VGB), TH Aachen, **Heft 57**.

Weiler, W.A. & Kulhawy, F.H. 1978. Behavior of stress cells in soils. Geotechnical Engineering Report No. 78-2, School of Civil and Environmental Engineering, Cornell University, Ithaca, NY, USA.

Wong, I.H., Ooi, I.K. & Broms, B.B. 1996. Performance of raft foundations for high-rise buildings on the bouldery clay in Singapore. Canadian Geotechnical Journal **3**: 219–236.

(Accepted 12 November 2000)

أداء أساسات اللبشة الخرسانية لبرج المواصلات على التربة الإسمنتية في الكويت

نبيل فتحي إسماعيل و حسن عبدالعزيز السند
قسم الهندسة المدنية – جامعة الكويت
ص.ب.5969 الصفاة 13060 – الكويت

خلاصة

تمت متابعة قاعدة اللبشة الخرسانية المجوفة لبرج المواصلات في الكويت والبالغ قطرها 55 متراً . اشتملت القياسات على الهبوط وضغط التربة وضغط المياه الجوفية وغيرها.

ولتقدير الهبوط المتوقع تم عمل فحوصات موقعية ومختبرية تفصيلية للتربة في الموقع وذلك لمعرفة خصائص التربة تحت منسوب التأسيس. واستناداً إلى نتائج هذه الفحوصات تم تقدير الهبوط المتوقع بعد الإنشاء ثم مقارنته بالقياسات الفعلية التي بينت هبوطاً يعادل 42 مليمتراً بعد اكتمال الإنشاء.