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Design and Testing of Piles on a Telecommunications Project in Morocco

Richard L. Kulesza¹⁾, P.E., M.ASCE
Bengt H. Fellenius²⁾, P.Eng., Dr.Tech., M.ASCE

¹⁾Principal Engineer, Bechtel Corporation, 50 Beale St., San Francisco, California 94119.
 <rkulesza@bechtel.com>

²⁾Consulting Engineer, 2475 Rothesay Avenue, Sidney, British Columbia, V8L 2B9.
 <Bengt@Fellenius.net>

ABSTRACT Site investigation, geotechnical analyses, and pile testing were performed in 1990-91 for the design of pile foundation for towers and anchors on a telecommunications project in Morocco. The soil profile consisted of estuarine and lagoonal deposits with a normally-consolidated clay deposit overlying overconsolidated clay (hard basal clay). An estuarine sand deposit of variable thickness and density is located between the clay and basal clay, in places sandwiched within the clay. Consolidation properties of the clay were back-calculated from the records of settlement under an extensive berm constructed earlier at the site. Analyses of pile compression and uplift capacity were performed by several methods. Calculations were also made of deflections and bending moments induced in inclined piles by continuing ground settlement. In most cases, the piles achieved the required tension and compression capacity in the sand and basal clay. However, attention needed to be paid to areas where the sand was relatively thin and underlain by clay. Test piles driven to the calculated depths were tested in compression and tension. The results of the pile testing programme indicated that pile analysis methods employed at the time provided a reliable prediction of the actual pile capacities, even under fairly complex soil conditions.

1. INTRODUCTION

In the late 1980s and early 1990s, a Voice of America (VOA) Morocco Relay Station was designed and constructed in Morocco, near Briech on the Atlantic Ocean, approximately 30 km south of Tangier. The station hub area containing the support facilities, shown on Figure 1, lies approximately 3.6 km east of the coastline. The site lies at an elevation 1.2 m to 2.2 m above sea level in a river flood plain that is subject to frequent inundation during the rainy season. The relay station covers an approximately 2,000 m by 1,600 m area and consists of 26 short-wave antenna towers. To keep the areas above the zone of flooding, the 420 m by 220 m, central hub area was raised in 1985 by about 6.0 m and three, about 50 m wide berms, supporting several antenna arrays (towers and anchorages), were constructed on

approximately 3.5 m to 4.0 m of compacted fill. These berms are marked as West, East, and Southeast Berms in Figure 1. The towers and anchorages are founded on 406 mm diameter pipe piles, driven closed toe. This paper presents the approach used for the project with regard to geotechnical site evaluation, settlement studies, and pile design, and some of the results of pile loading tests and the analyses.

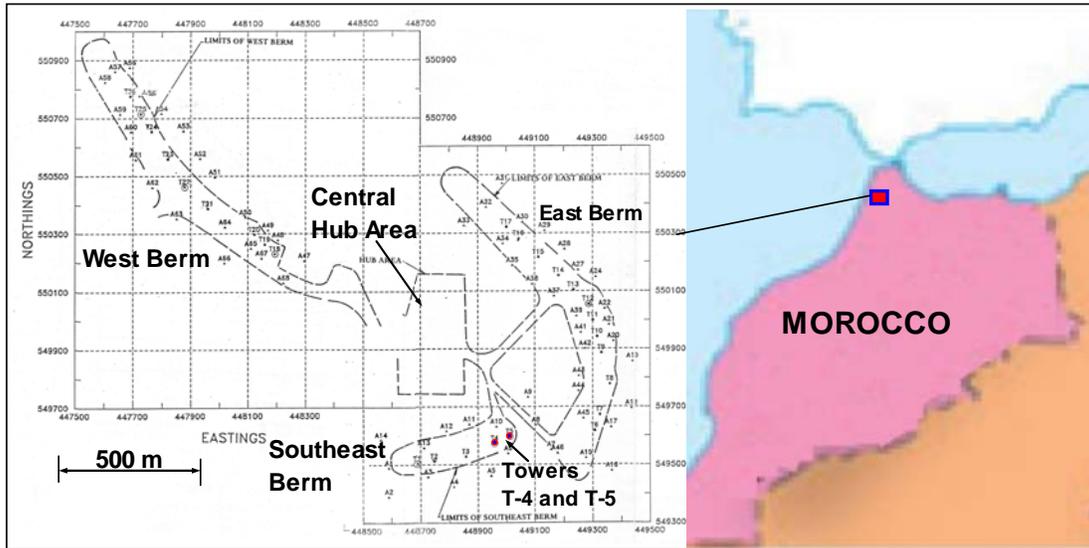


Fig. 1 Project Site Location and Site Layout

2. SUBSURFACE CONDITIONS

2.1 Soil Profile and Properties

Geotechnical investigations were performed by the U.S. Army Corps of Engineers (USACE) between 1984 and 1987, and by TCI/Bechtel in 1990-91 to establish soil properties, including strength and compressibility. The results of the USACE investigation were used by Bechtel in 1990 to develop contours of elevation of upper and lower boundary of the soil deposits, consisting of three major soil types: soft clay, hard basal clay and sand. The contours, as well as preliminary pile lengths computed based on the previous soils data, were used to plan the borehole depths and soil sampling for the 1990-91 site investigation. Because boreholes were put down at every tower and anchor location, in all 96 boreholes, appropriate selection of the borehole depths had a large impact on the cost and schedule of the 1990-91 site investigation.

The site is located in an area varying, in its recent geological past, between estuarian and lagoonal depositional environments. Active sedimentation continues to take place during flooding occurring in the rainy season, with sediments originating in the uplands adjoining the river. The entire site is underlain by a deposit of normally consolidated soft clay (NC Clay), commencing near the ground surface. The NC Clay has a stiff, fissured surficial crust typically 1.5 m thick. Below the crust, the clay is very soft, becoming progressively stiffer with depth.

In some parts of the site, the NC Clay extends to a heavily overconsolidated clay stratum, referred to as hard basal clay. In other parts, sand deposited during estuarian conditions is encountered below the NC Clay, and it either extends to the hard basal clay, or is “sandwiched” within the NC Clay. The thickness of sand is highly variable, ranging from very thin to thicker than 30 m. Except for some loose zones near its upper boundary, the sand is generally dense to very dense and includes some cemented lenses. Where the thickness is adequate, it provides suitable support of pile foundations. Where soft clay reoccurs below the sand, its consistency is similar to that of the upper continuous deposit of the NC Clay. It is therefore referred to as the lower NC Clay. The erratic thickness of the sand layer and the potential presence of the lower NC Clay were critical conditions for the planning of the 1990-91 site investigation and in the selection of pile lengths.

The level at which the hard basal clay commences is also of importance in the design of piled foundations because it constitutes the lower limit of clays having significant compressibility, and provides pile support where the sand layer is absent or of insufficient thickness to support the piles.

Results of site investigation indicate that the engineering properties of each of the three major soil types are reasonably consistent throughout the site. The selected design parameters were therefore considered applicable to each of the deposits at all tower and anchor locations areas with minor modifications where justified by the local conditions.

A summary of the geotechnical characteristics of each of the three major soil types follows.

2.1.1 Normally Consolidated (NC) Clay

The NC Clay has a desiccated, surficial crust, typically 0.5 to 2.5 m thick. An average thickness of 1.5 m was assumed in the analyses. Below the crust, the clay is very soft. Undrained shear strength obtained from unconfined compression tests indicated that the lowest strength, about 7 kPa, occurs immediately below the crust. Hereunder, the strength increases approximately linearly with depth.

The natural water content was about 30 to 45 % in the crust, and reached a maximum of 50 to 60 % just below the crust. It then decreased progressively with depth. The scatter of results is caused by variations in plasticity and occasional sand inclusions.

The liquid and plastic limits ranged from 34 to 93 % and 20 to 25 %, respectively. The plasticity index varied from 17 to 64, with the lower values obtained on samples described as containing sand. On this basis, and also taking into account that the clay fraction generally exceeded 70 %, most of the samples were assigned designation CH (clay of high plasticity per the Unified Soil Classification System). Occasionally, lower plasticity, CL, materials were also identified within this stratum. The natural water content was generally close to the liquid limit.

The unit weight was between 17 and 20 kN/m³ in the crust and reduced to about 16.5 kN/m³ immediately below. The unit weight then increased progressively with depth, reaching about 19 kN/m³ at 30 m depth. Also the distribution of unit weight of the clay with depth is similar over the entire site. The solid density was in the range of 2,690 to 2,780 kg/m³.

Figures 2A and 2B, respectively, present values of undrained shear strength, s_u , and natural water content from areas outside the berms. The distribution of s_u with depth indicates near-normally consolidated condition, with $s_u = 2.0 + 0.20\sigma'_0$, where σ'_0 is the initial vertical effective stress. The value 0.20 is consistent with published data, e.g. Ladd and DeGroot (2003), who state that, for very low OCR (overconsolidation ratio) cohesive soils, the in-situ strength should equal or exceed $s_{u(ave)} = S\sigma'_0$, where $S = 0.22 \pm 0.03$. The uniformity of results indicates that the distributions of undrained shear strength and water content with depth in the off-berm areas are similar across the site.

Due to consolidation of the soft clay under the weight of the berms that took place in the time period between the construction of the berms in 1985-86 and the soil testing in 1991-91, the undrained shear strength under the berms was found to be higher than in the off-berm areas within about 6 m depth below original ground surface. The undrained shear strength under the berms within the depth range of the vane tests was approximately uniform, averaging about 17 kPa, reflecting overconsolidation induced by the berms. Remolded vane tests showed 1.7 average sensitivity, indicating that the clay has low sensitivity.

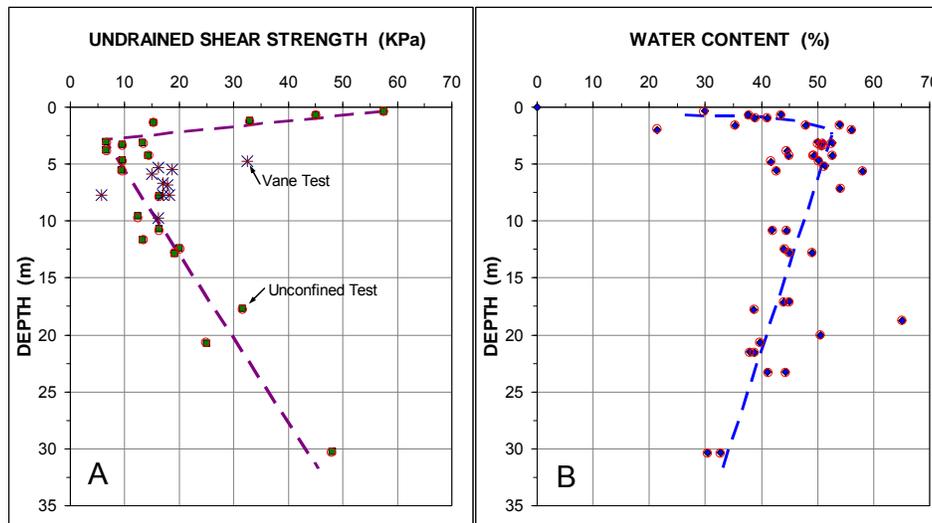


Fig. 2 Undrained shear strength (A) and water content (B) of NC Clay vs. depth

Consolidated-undrained triaxial compression tests performed on samples of the clay from various depths indicated an effective angle of internal friction ϕ' mostly in the range of 18° to 24°. The effective cohesion intercept c' was zero, as expected for the NC Clay. The tests on samples having a natural water content exceeding 40% showed pore pressure coefficient A_f from about 0.6 to over unity, indicating that high pore water pressures can be induced by loading of the NC Clay.

Consolidation and compressibility parameters of the NC Clay were determined by back-analysis of settlements induced by an extensive berm placed in 1985-86 in the hub area, as described later in this paper.

2.1.2 *Sand Deposit*

Particle size distribution tests were not performed, but sample descriptions included on the boring logs indicated that the sand is fine to very fine and consists of well rounded quartz grains. Standard penetration tests (SPT) indicated a large scatter of penetration resistance at all depths, as is common in an estuarine sand deposit. Moreover, clay, silt, and gravel lenses, as well as occasional cemented sand zones were commonly encountered in the sand, causing large variations in SPT results. The cemented sand lenses were about 0.6 m thick and had to be penetrated by rock coring in order to advance the borehole. While 5 out of over 50 SPT N-indices (blows/0.3 m) reported from the 1985-86 USACE investigations, all from depths of 5 to 12 m, indicated loose condition, all the 1990-91 SPT tests gave N-indices exceeding 10, usually by a large margin, indicating medium to very dense condition. The unit weight of the sand was estimated to be 19 kN/m^3 .

2.1.3 *Hard Basal Clay*

The NC Clay and the sand deposits are underlain over the entire site by a base stratum of stiff to hard, heavily overconsolidated clay, referred to as hard basal clay. Some of the 1985-86 borehole logs describe it as "indurated", and presence of fissures was noted in some samples. White crystalline inclusions were also reported. This stratum commences at different depths, ranging, at borehole locations, from about 10 m to more than 40 m below the original ground surface. Data from some of the borings indicated that the upper portion of this stratum was in a softened condition. The level of the upper boundary of the basal clay is of importance in the design of foundations because it constitutes the lower limit of significantly compressible clays.

The natural water content ranged from 16 to 29 %. In the upper part of the stratum, which is of interest to pile design, it averaged 26 %. The average value corresponds to the upper range of the natural water contents determined in the hard basal clay within the depths of 15 to 30 m below ground surface, and is a conservative value. The liquid and plastic limits were 42 to 65 % and 20 to 44 %, respectively. The plasticity index ranged from 16 to 27. The water content was generally in the lower part of the plastic range, with some results below the plastic limit. The unit weight was between 19.5 and 21.7 kN/m^3 . The solid density ranged from $2,690$ to $2,780 \text{ kg/m}^3$, same as for the NC Clay.

The undrained shear strength of the basal clay determined in unconfined compression tests on samples from depths of 17 to 32 m ranged between about 100 and 330 kPa. Using Figure 3, and the average water content of 26 %, average undrained shear strength of 130 kPa was selected for the upper part of the stratum, in which some of the piles are supported. Based on the results of consolidated-undrained triaxial compression tests with pore pressure measurement, an effective angle of internal

friction, ϕ' , of 24° and an effective cohesion intercept, c' , of 20 kPa were adopted. The test results indicated a pore pressure coefficient, A_f , of 0.25. This value, consistent with the overconsolidated condition of the clay, suggests that pore pressures induced in the hard basal clay by pile driving and by stress due to foundation loads will be low.

A drained direct shear test was performed on a sample of the hard basal clay, placed in contact with a steel plate, to simulate the interface between the clay and a steel pile. The peak drained friction angle was 14° , and the large-strain friction angle was 12° .

Because some samples of the hard basal clay were observed in the field to exhibit pronounced swelling, a "free swell test" (ASTM D4546) was performed on a sample of the hard basal clay from a depth of 20.3 m. A swell of approximately one percent developed in 27 days.

Mineralogical analysis of a sample of the hard basal clay was performed, mainly intended to determine the nature of white crystalline inclusions prominent in this stratum, and to identify any swelling clay minerals. There was concern that the white inclusions could be gypsum. However, these were found to consist of calcite, which accounted for 60% of the sample. A small (less than 3%) content of smectite was detected. This mineral probably accounts for the observed swelling properties of the hard basal clay.

Figure 3 shows the relation between undrained shear strength and natural water content from amalgamation of the records presented in Figure 2 and a few records from the hard basal clay. The different symbols represent different areas of the site. The distribution indicates that the clays have similar strength properties over the entire site.

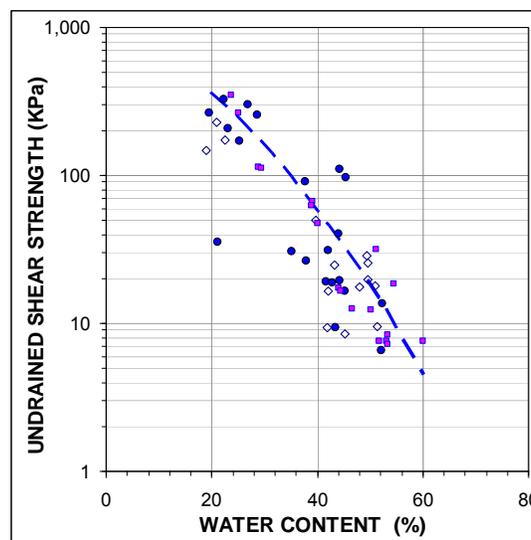


Fig. 3 - Water content vs. undrained shear strength for the clay deposits
 2.1.4 Groundwater Conditions

Piezometric levels were determined using piezometers sealed in the upper NC Clay and in the sand. The groundwater table was typically 1.0 m to 1.5 m below the original ground surface during the dry season. During the wet season, the site is frequently flooded. A water table depth of 1.5 m below natural ground surface with hydrostatic pore pressure distribution throughout was adopted in the analyses for the areas outside the berms. Piezometers inside berm areas indicated that the water table mounded within the berm fill. Hence, an average water table depth of 2.7 m below the berm surface with hydrostatic pore pressure distribution was adopted for analysis in berm areas. One of the piezometers, installed in the sand deposit, was monitored over a period of 8 hours to investigate possible tidal effects, but no significant fluctuations were detected (the site is located over 3 km from the ocean).

2.2. Effect of Soil Profile on Pile Design Development

Because of the loads from the structures are concentrated and because of the presence of thick deposits of soft compressible clay, piled foundations were required. The subsurface conditions were considered suitable for a variety of pile types, provided that the appropriate pile toe depths were selected accounting for the variable levels of soil boundaries. Based on economic and scheduling considerations, 406 mm diameter, 9.5 mm wall thickness, closed-toe, driven steel pipe piles filled with concrete after driving were selected. On the west berm, the towers were supported on one to three piles driven to bearing in the dense sand at depth ranging from 8.9 m through 14.3 m. The anchors were founded on two to five piles, inclined 30° to the vertical and driven to the dense sand or the hard basal clay. The inclined lengths varied from 9.6 m to 24.8 m, reflecting a wide range of design loads as well as variability of soil profile.

For the antennas located on the west berm, the design load for tower-supporting piles ranged from 339 to 738 kN (all in compression), and for anchor piles it ranged from 77 to 427 kN in compression and from 81 to 418 kN in tension. The adopted minimum factors of safety were 2.0 for axial compression and 2.5 for axial tension. As an example, for a sustained (dead) working load of 566 kN and typical transient (live) load of 131 kN, the tension working load was 330 kN. This means that, for these working loads and factors of safety, the piles were designed for an ultimate shaft resistance of 825 kN for piles loaded in tension and an ultimate total resistance of 1,194 kN for piles loaded in compression. Anchor piles to meet both criteria would typically need a toe resistance of 369 kN.

Either the sand or the basal hard clay were considered appropriate for supporting pile loads, whether in tension or compression. In the sand, a minimum penetration of 2.0 m was specified. However, since in some areas, the sand was “sandwiched” within the NC Clay, attention was necessary to ensure that piles embedded in the sand did not approach too closely the surface of the lower NC Clay, where punching failure might develop. A further consideration was the settlement of the NC Clay in the berm areas would continue after the structure had been built. Therefore, the effect was evaluated of ground settlement on the piles, primarily downdrag, but also drag load, and for bending of the inclined piles. Also, in some areas where piles were

founded in the sand above the Lower NC Clay, possible settlement due to pile load transfer into the Lower NC Clay had to be considered.

3. SETTLEMENT

3.1 Back-calculation of compressibility parameters from settlement records

Consolidation properties of the NC Clay were determined from consolidation tests performed during the 1984-87 site investigations. Additional, valuable information on the settlement characteristics was available from the results of a settlement monitoring programme conducted by the USACE at the central hub area, where settlement of approximately 6 m high fills constructed in 1985-86 was monitored over a 40-month period.

Settlement records from six settlement plates located in the central part of the central hub area fill, and thus away from edge effects, were averaged to obtain representative settlement values. While it was known that the fill was constructed over twelve months, between September 1985 and September, 1986, details of the loading procedure and timing were not available. It was simply assumed that the fill had been placed in twelve equal increments to a final height of 6 m over the twelve months period. A total unit weight of fill of 20 kN/m^3 was adopted based on the records of earthwork control density tests done during fill construction. This value was verified by testing of undisturbed samples of berm fill taken during the 1990-91 site investigation. The typical soil profile in the hub area consists of about 1.0 m of clay crust over about 8 m of Upper NC Clay followed by 11 m of sand above about 16 m of Lower NC Clay, deposited on the hard basal clay at a depth of about 36 m.

The observed and the back-calculated settlements are shown on Figure 4. The back-calculation was made in 1991 using the TCON program (Taga Engineering 1991). The Upper NC Clay was assumed to be the source of most of the settlement. Fitting the calculated time vs. settlement curve to the measured settlement values gave a compression index, C_c , of 0.28. Combining this value with the average water content and void ratio of 45 % and 1.2, respectively, the average compressibility expressed in terms of Janbu modulus number, m , is 18 and the corresponding Compression Ratio, CR, is 0.13. The back-calculated coefficient of consolidation, c_v , was $8 \times 10^{-8} \text{ m}^2/\text{s}$ ($2.64 \text{ m}^2/\text{year}$). Figure 4 also shows settlement of the 6 m berm using the UniSettle program (Goudreault and Fellenius 2011).

The settlement estimated for 90 % consolidation was 750 mm, which was reached after about 60 months. Based on similarity of soil classification properties, the back-calculated compressibility parameters and c_v -values were considered applicable to the NC Clay deposits over the entire site.

3.2 Settlement of Berm at Towers T4 and T5

The consolidation parameters of the NC Clay developed as described above were used to generate settlement versus time curves applicable to the sites of two towers, Towers T4 and T5 located in the south-east berm area (see Figure 1). The soil conditions were considered most unfavorable at this site.

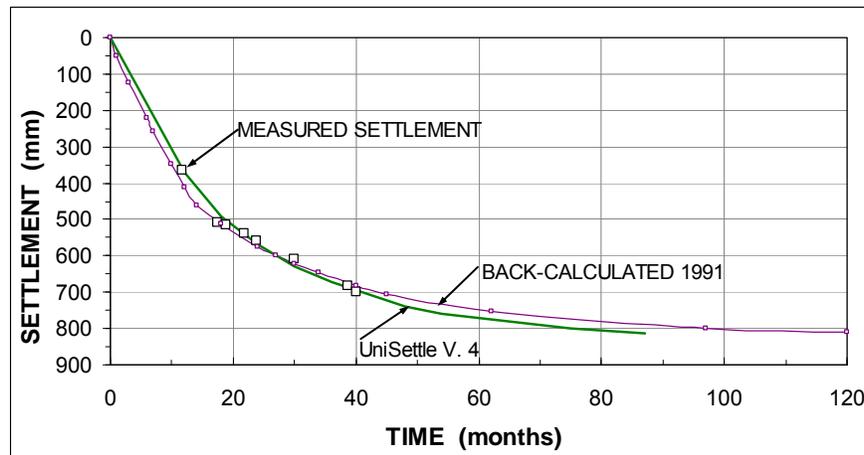


Fig. 4 Measured and back-calculated settlement of a 6 m high berm

The towers are located about 50 m apart and about 25 m from the side of the berm. The soil profiles at the two tower locations are similar: below the fill, the soil consists of 9 m and 6.5 m of NC Clay, respectively, followed by sand. The ground surface after stripping was taken as El. +1.5 m. At the end of construction, the surface of fill was at El. +4.6 m. Allowing for 0.2 m settlement during berm construction, the thickness of the placed fill was about 3.3 m. The berm footprint area was approximately 100 by 300 m. The calculated berm surface elevation versus time in the first twelve months and first seven years after the start of berm construction (September 1985) are shown on Figure 5. Over a 7-year period following start of berm construction, the settlement of the original ground surface was calculated to be 0.50 m.

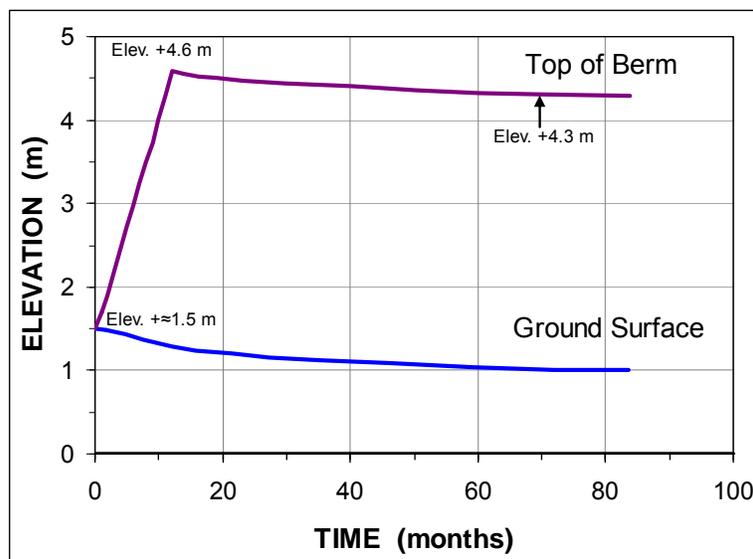


Fig. 5 Calculated settlement for 3.3 m thick berm at Tower T4

The reasonable accuracy of the calculated time vs. fill surface settlement curve was verified during the 1991 site investigation. The berm surface level at borehole T4 was surveyed as El. +4.30 m. This survey was made on 6 June 1991, 70.2 months after the start of berm construction. This is in agreement with the fill surface elevation after 70 months shown in Figure 5. Further, a high-accuracy survey was performed on three benchmarks located in this area on 4 July and 8 September 1991. These showed 7 mm, 4 mm, and 5 mm, respectively, and average of 5 mm, reduction of elevation during this two-month period. This is consistent with the change in surface elevation during this period, observable in Figure 5, and the measurements confirm the validity of the consolidation parameters derived from the back-analyzed settlements (See Figure 4).

3.3 Settlement of Pile-supported Tower Foundation

Over most the site, the piles were essentially bearing in non-compressible soil with neutral plane located in non-settling soil. Thus, settlement of the pile foundations would be minimal and consist mostly of load transfer movement. Larger settlement could occur in places where significantly thick layers of Lower NC Clay exist between the sand supporting the piles and the hard basal clay, such as at Tower T4. Calculations showed that this tower could experience a 150 mm long-term settlement due combined tower and berm loading. While the actual long-term settlement is not known, information received from TCI is that no problems with any tower or anchor foundations have been noticed.

4. PILE CAPACITY AND LOAD RESPONSE

Pile capacity and load response was calculated using both total and effective stress using software as available in 1990. Reference was made to recommendations in the Canadian Foundation Engineering Manual (1985) and Meyerhof (1976). Most calculations employed the PC program AXIAL/G by Geosoft, which largely relied on recommendations by Meyerhof (1976). Additional analyses were performed using PC program UniPile V. 1 (Goudreault and Fellenius 1990) and PAR (Bea et al. 1984). While some piles were founded in the hard basal clay, most were embedded in the dense sand after penetrating the Upper NC Clay. Where the sand was underlain by the Lower NC clay, special consideration was given to the possibility that the pile toe resistance in the sand may be affected by the softer stratum below. The rule applied in the pile capacity computation, proposed by Meyerhof (1976), was that if the sand was underlain by the Lower NC clay closer than about 10 pile diameters below the pile toe, the calculated toe resistance was proportionally reduced with depth as the pile toe level approached the clay layer.

Because of the settlement brought on by the berms and fills, the piles will be subjected to negative skin friction accumulating to a drag load that will amount to a value close to or about equal to the pile shaft resistance in the NC Clay. For the relatively short piles installed for the project — maximum depth was about 20 m — the maximum load (dead load plus drag load) acting at the neutral plane is well below the safe axial structural pile strength. Because the neutral plane is located in the sand below the Upper NC Clay or close to the hard basal clay, downdrag will be

inconsequential. Therefore, no special precautions were required to address the issue of drag load and downdrag for vertical piles. The project is one of the first major projects where the principles of the Unified Design were applied (Fellenius 1984; 1988; 2004).

As mentioned, in some areas where piles were founded in the sand above the Lower NC Clay, possible settlement due to pile load transfer into the lower NC Clay had to be considered. Potential overstress due to bending of inclined piles caused by settling soil was also analyzed.

Calculations of shaft resistance with depth assumed vertical piles. Because anchor piles were inclined 30° from the vertical, an allowance due to the inclination was included in the form of increased perimeter of the pile calculated by increasing the perimeter by 15 % to allow for the increased pile surface area for each unit length penetrated by the pile.

The design analyses adopted total stress method for length of pile shaft in clay and effective stress method for length in sand. However, in one case (the pull-out test pile at anchor A58), effective stress analysis was also used in clay, to verify the pull-out capacity on long-term basis. The calculated distribution of capacity with depth was very similar to that for the total stress analysis (short-term conditions).

The soil strength parameters input into the pile capacity analyses, described in Section 2.1, were reviewed using the results of boreholes put down at each tower and anchor location, and adjustments were made where appropriate. For example, in loose sand, the friction angle based on the SPT N-index was increased to allow for densification by the driving of the closed-toe piles

4.3 Effect of Settlement on Inclined Piles

Continued ground settlement under berms would induce bending of inclined piles due to enforced lateral movement and, for the inclined piles, by the vertical movement of the soil in contact with the pile. These factors needed to be taken into account in the design of piles driven through the berms, where the NC Clay is thick and could continue to consolidate after anchor construction. Calculations addressing this issue were performed on vertical and inclined piles, using the program PAR (Bea et al. 1984). It is a finite element code that can take into account imposed forces and displacements along the pile, in addition to the axial loads at the pile head. The program computes pile stresses and displacements using non-linear soil deformation parameters. Non-linear springs are used at pile toe (vertical) and along the pile shaft (horizontal and vertical springs, the latter simulating shaft resistance stress-deformation characteristics). In its default mode, the program applies empirical stress-strain relationships recommended by API (American Petroleum Institute), using soil buoyant unit weight and friction angle (sands) and undrained shear strength (clays). The API default relationships were used in the analyses, these are considered conservative. The forces from the settling ground imposed on the pile along its length, calculated manually, were applied at the pile nodes.

The analyses showed that the maximum fiber stress, conservatively estimated, would be smaller than about 60 MPa, well below the maximum allowable fiber stress of about 200 MPa. Hence, no adverse effect was expected to develop on the inclined piles from continuing consolidation of the NC clay.

5. STATIC LOADING TESTS

The calculations of pile capacity were verified in a series of static loading tests comprising four uplift tests and one compression test, all performed on vertical piles in the West Berm area. The locations of the test piles were selected to allow for study of pile response under differing site conditions and included test piles driven through berm fill and where no fill had been placed. The testing programme commenced in September 1991. The uplift piles, labeled A54, A58, A61, and A68, had embedment depths of 14.4 m, 15.0 m, 14.7 m, and 17.5 m, respectively, and the compression pile, labeled T26, had 10.5 m embedment.

The piles were driven with a single-acting Model D30-13 Delmag diesel hammer operating at Fuel Setting 4. Figure 6 shows the pile driving records in terms of penetration resistance, PRES, (blows/0.3m) versus depth (m). The compilation shows that the PRES varied between the piles, primarily due to presence, depth, and thickness of the sand layer. (Association of pile driving log to soil profile is provided in Figures 7 - 11).

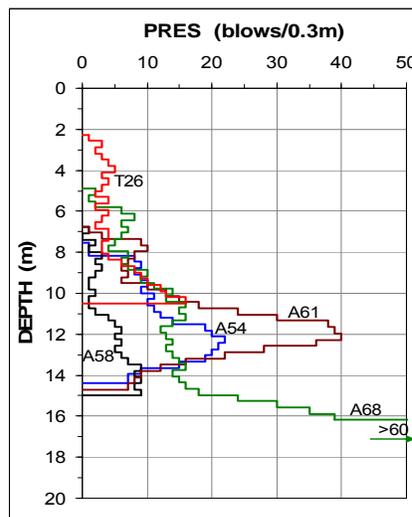


Fig. 6 Pile Driving diagrams for the test piles

Ahead of performing the static loading tests, the distribution of static resistance and capacity for the test piles were calculated using total stress in clay and effective stress in the sand. As mentioned, effective stress calculations were performed both in the sand and in the clay for anchor pile A58, to verify pile capacity under long-term conditions. Each of Figures 7 through 11, shows the load-movement curve of the loading test with the maximum test value marked. To the right of the load-movement curve diagram, a graph is shown with a summary soil profile at the test pile location, pile driving diagram (PRES; scale at bottom axis), predicted load distribution with depth, predicted capacity, and the maximum test load.

The pile capacity vs. depth relationships shown in the figures were used to select the depth of the test piles. These relationships were developed in terms of total stress in clays. The undrained shear strength, s_u , was used, with appropriate reduction factor, α , which varies with s_u (e.g., Tomlinson 1957). The soil unit weight and strength parameters, and the water table level, used in the calculations, are described earlier in this paper. The soil profiles in the figures include typical soil property values.

Table 1 lists the parameters employed in the total stress calculations, as derived from fitting the resistances to the predicted load distributions shown in Figure 7 -11 (the depth values indicate depth of test pile below surface of berm or no-berm). Table 1 also includes effective stress parameters determined in fitting the resistances to an effective stress analysis along the full pile lengths. The beta-coefficients determined in the clay layers are entirely reasonable for the site conditions. Although shaft resistance parameters are usually applied with two decimals, a single-decimal precision, as shown, was imposed for the back-calculation. All calculations are performed using UniPile V. 4 (Goudreault and Fellenius 1999).

To display the degree of agreement between back-calculated and predicted values, Figures 7 - 11 show the load-distribution curves calculated from the Table 1 parameters along with the predicted values of capacity.

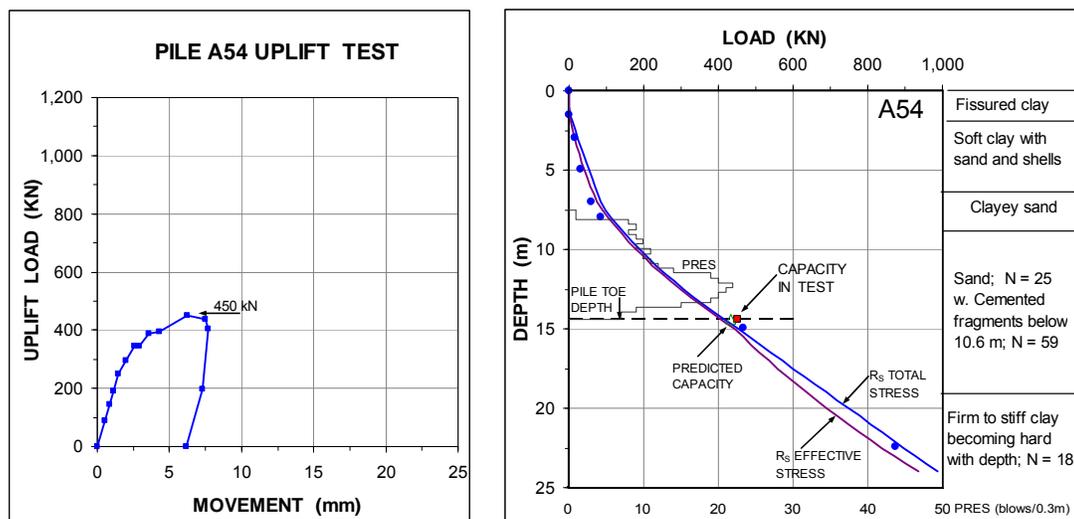


Fig. 7 Pile A54. Results of uplift test and comparison of results to analysis

5.1 Uplift Test Pile A54

Test Pile A54 was driven a depth of 14.4 m outside the berm area. The soil profile (Figure 7) indicates about 7 m of clay on sand and most resistance to driving occurred in the sand. As indicated, the pile reached abrupt failure at a pile head upward movement of about 6 mm for the applied load of 450 kN. The very good agreement between predicted and measured capacity values indicates that the soil parameters (as back-calculated) can be used with confidence for estimating the shaft resistance of other piles driven at places with similar soil conditions.

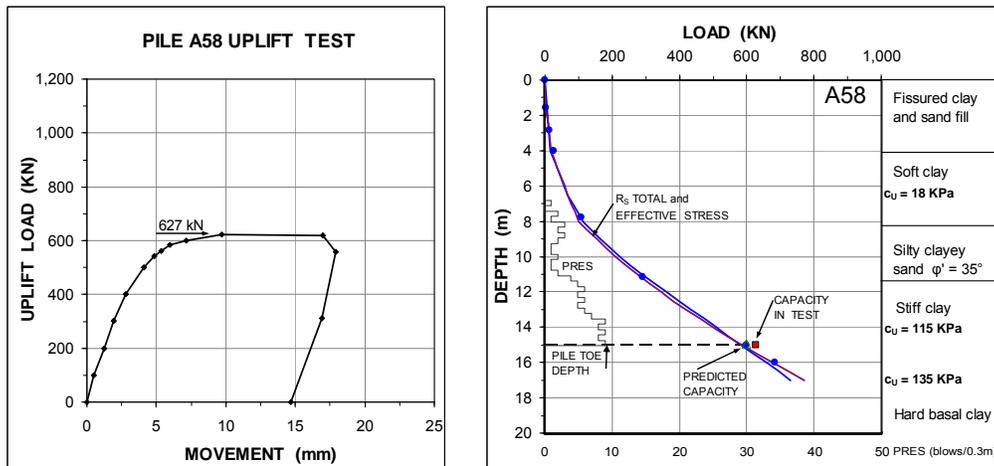


Fig. 8 Pile A58. Results of uplift test and comparison of results to analysis

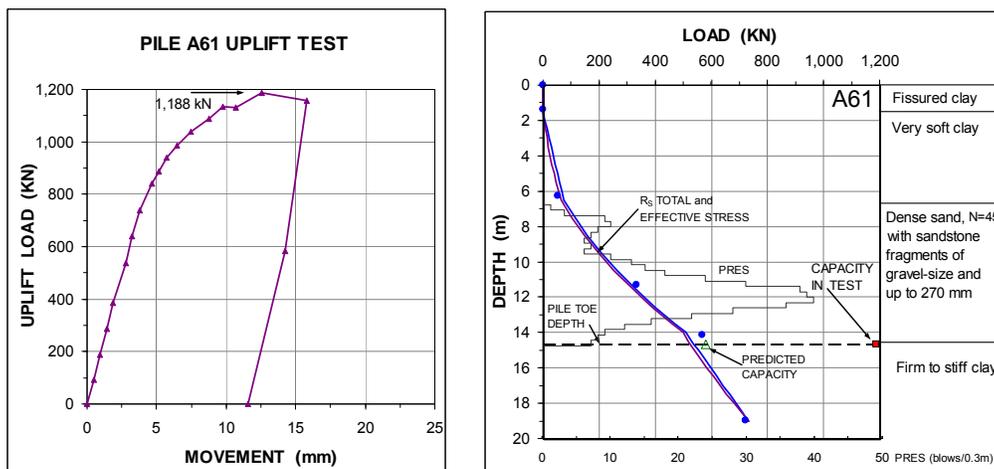


Fig. 9 Pile A61. Results of uplift test and comparison of results to analysis

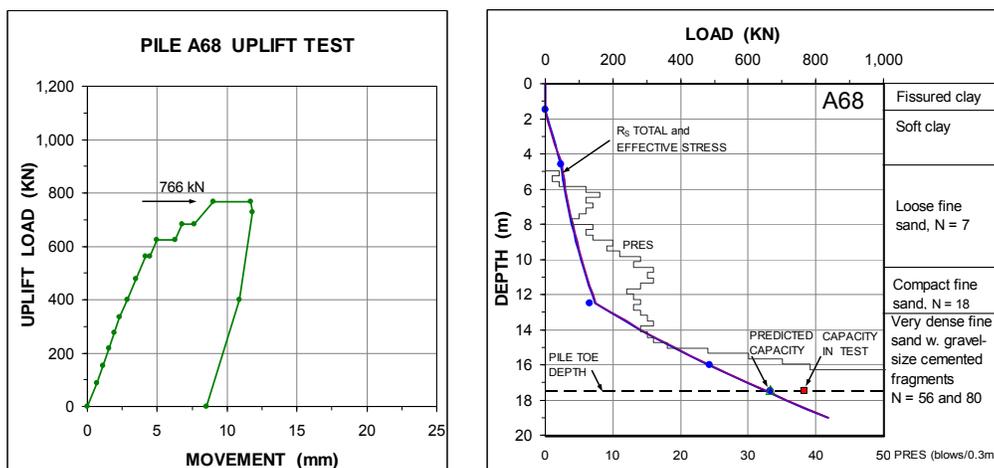


Fig. 10 Pile A68. Results of uplift test and comparison of results to analysis

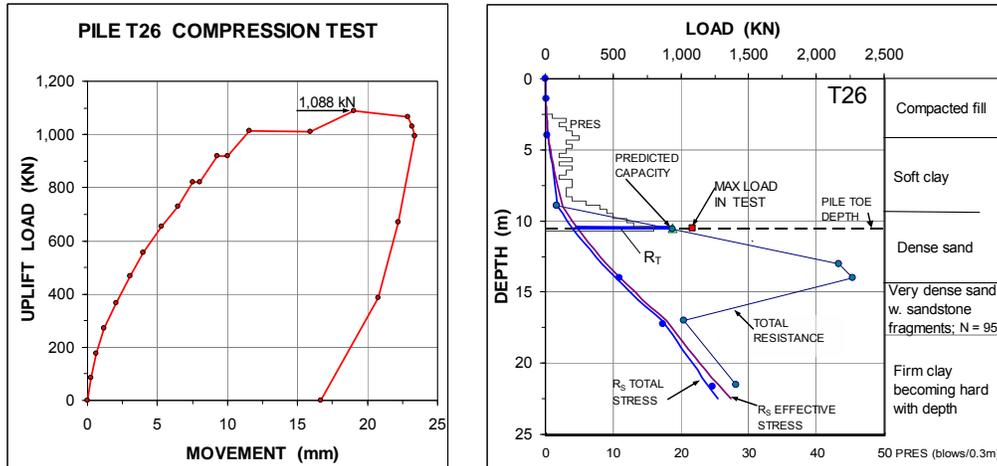


Fig. 11 Pile T26. Results of compression test and comparison of results to analysis

Table 1 Test pile data with predicted and back-calculated values

Test Pile ID	On-berm / Off berm	Depth Upper / Lower (m)	Soil Type	Unit Shaft Resistance (kPa)	Beta-Coefficient (---)	r_T (MPa)	Beta Coefficient (---)	N_T (MPa)
				Total Stress Analysis			Effective Stress Analysis	
A54	Off berm	0.0 - 1.5	Clay crust	0			0	
	Depth (m) = 14.4	1.5 - 7.0	Soft clay w. sand and shells	10			0.2	
	R_{PRED} (kN) = 434	7.0 - 15.0	Clayey sand		0.3		0.3	
	R_{TEST} (kN) = 450	15.0 -	Firm to stiff clay	40			0.2	
A58	On berm	0.0 - 4.0	Fill (sand and clay)		0.1		0.1	
	Depth (m) = 15.0	4.0 - 8.0	Soft clay, $c_u = 18$	20			0.2	
	R_{PRED} (kN) = 600	8.0 - 11.0	Silty clayey sand, $\phi' = 35^\circ$		0.4		0.4	
	R_{TEST} (kN) = 623	11.0 -	Stiff to very stiff clay	50			0.4	
A61	Off berm	0.0 - 1.5	Clay crust	0			0	
	Depth (m) = 14.7	1.5 - 6.5	Very soft clay	10			0.2	
	R_{PRED} (kN) = 580	6.5 - 14.0	Dense sand, $N = 45$		0.4		0.4	
	R_{TEST} (kN) = 1,188	14.0 -	Firm to stiff clay	30			0.2	
A68	Off berm	0.0 - 1.5	Clay crust	0			0	
	Depth (m) = 17.5	1.5 - 4.5	Soft clay	10			0.3	
	R_{PRED} (kN) = 655	4.5 - 10.0	Loose sand		0.1		0.1	
	R_{TEST} (kN) = 766	10.0 - 12.5	Compact sand		0.1		0.1	
		12.5 -	Very dense sand		0.5		0.5	
T26	On berm	0.0 - 4.0	Fill		0.1		0.1	
	Depth (m) = 10.5	4.0 - 9.0	Soft clay	10			0.2	
	R_{PRED} (kN) = 940	9.0 - 14.0	Dense sand		0.5	13	0.5	100
	R_{TEST} (kN) = 1,088	14.0 - 17.0	Very dense sand		0.5	1	0.6	7
	Compression Test	17.0 -	Firm to hard clay	50		1	0.3	7

5.2 Uplift Test Pile A58

Test Pile A58 was driven a depth of 15.0 m inside the berm area. The soil profile (Figure 8) shows a fill thickness of 4.0 m on 4.0 m of soft clay followed by 11.0 m thick sand layer deposited on stiff clay (the 115 kPa and 135 kPa values correspond to very stiff condition). The lowest 4.0 m length of Pile A58 is embedded in the hard basal clay. The pile driving penetration resistance in the basal clay is larger than in the sand, but smaller than observed in the sand at Test Pile A54. As indicated, the pile reached plunging failure at a pile head movement of about 7 mm for the applied load of 627 kN. Again, a very good agreement is shown between predicted and measured capacity values, and, therefore, the soil parameters were considered applicable when estimating the shaft resistance of other piles driven at places with similar soil conditions.

5.3 Uplift Test Pile A61

Test Pile A61 was driven to a depth of 14.7 m outside the berm area. The soil profile (Figure 9) shows 6.5 m of soft clay followed by dense sand to 14.0 m depth on firm to stiff clay, that is, the lower 0.7 m length of Pile A58 is embedded in basal clay. The pile driving penetration resistance in the sand was about twice that recorded for Pile A54. However, the penetration resistance started to reduce at 12 m depth, about 2 m (5 pile diameters) before the pile toe entered the basal clay. The reducing resistance is probably due to the pile toe 'feeling' the impending clay layer, which is more compressible than the sand. The penetration resistance for the last 0.3 m of driving was 7 blows, which is about the same value, 9 blows/0.3 m, as recorded for the last 0.3 m of driving of Pile A58 into the basal clay.

During the drilling of Borehole A61, a 0.6 m thick layer of cemented sand was noted between depths of 11.4 m and 12.0 m. The layer was hard enough that penetrating it required rock coring during the advancement of borehole. The occurrence is local as two reaction piles, driven open-toe, did not experience similarly high penetration resistance in this depth interval.

As indicated, the pile reached plunging failure at a pile head movement of about 13 mm for the applied load of 1,188 kN, twice the predicted capacity of 580 kN. The reason for the discrepancy between predicted and measured capacity values is believed to be due to the presence of the mentioned layer of cemented sand. It is possible that the pile was "jammed" within the hard, cemented layer.

In view of the above information, including the fact that the two reaction piles encountered significantly lower penetration resistance, the test result for T61 is considered unrepresentative of the typical conditions. Such condition is not predictable and the larger resistance cannot be relied upon for the uplift design.

5.4 Uplift Test Pile A68

Test Pile A68 was driven to a depth of 17.5 m outside the berm area. The soil profile (Figure 10) shows 4.5 m of soft clay followed by loose to compact to very dense sand below 14.0 m depth. Above 15 m depth, the pile driving penetration resistance in the sand was slightly smaller than that recorded for Pile A54. However, below 15 m, an abrupt increase of resistance was observed, from a driving resistance (PRES) of about 15 bl/0.3 m at 15 m depth to over 60 bl/0.3 m at 16 m through end-of-driving. The two anchor piles, driven open ended, also encountered an increase in penetration resistance in the same depth range, albeit smaller, from about 8 bl/0.3 m to about 13 bl/0.3 m.

As indicated, the pile reached plunging failure at a pile head movement of about 9 mm for the applied load of 766 kN about 100 kN (15 %) larger than the predicted value, which is still a good agreement between predicted and measured capacity values, and the soil parameters were considered applicable when estimating the shaft resistance of other piles driven at places with similar soil conditions.

5.5 Compression Test Pile T26

Test Pile T26 was driven to a depth of 10.5 m inside the berm area. The soil profile (Figure 11) shows a fill thickness of 4.0 m on 5.0 m of soft clay followed by sand at 9.0 m. The pile driving penetration resistance in the clay was about 3 bl/0.3 m. From 8 m depth, the resistance increased linearly to 16 bl/0.3 m at the end-of-driving pile depth. The penetration resistance in the sand is approximately similar for the piles driven in dense sand between about 9 m and 12 m depth, Piles A54, A61, and A68. The back-calculated parameters applied to shaft resistance are also quite similar. Because of the short length in the sand, the predicted shaft resistance is only about 200 kN and the predicted capacity, 940 kN, is made up mostly of toe resistance.

As indicated, the pile reached a plunging failure at a pile head movement of about 16 mm for the applied load of 1,088 kN. Capacity was interpreted to be the load applied immediately before, 1,010 kN, about 70 kN (7 %) larger than the predicted value. Again, this is a very good agreement between predicted and measured capacity.

Despite the good agreement between predicted and measured resistance, the plunging trend of the load-movement curve implies a pile response dominated by shaft resistance, which contrasts with the predicted large contribution of toe resistance.

CONCLUSIONS

The results of the pile loading test programme confirmed the validity of the pile uplift capacity calculation methods used on this project and of the soil strength parameters selected for design.

The pile driving records of Pile A61 demonstrated that a pile toe resistance in dense sand was affected by the proximity to a softer layer located below the pile toe, starting when the distance was about 5 pile diameters.

Intermittently occurring very dense cemented sand layers or lenses — so strong that diamond bit coring was necessary in order to drill through them — resulted in a considerable increase of pile driving resistance. As no surprise, the presence of the cemented sand also caused a large shaft resistance, as in the case of Pile A61. However, as demonstrated by the results of the tests on Piles A54 and A68, the larger shaft resistance was not guaranteed by the presence of a such layer or lens.

The data from monitoring of berm settlements in the hub area prior to the finalizing of the design were useful in determining the consolidation time and the magnitude of settlement to expect for the berms at the site and enabled the final design to include reliable settlement estimations.

The results gave evidence that, if a comprehensive site investigation is performed resulting in an adequate understanding of soil properties, available pile design methods, in conjunction with carefully selected soil strength parameters, can provide reliable estimates of the pile capacity even under complex soil conditions.

Information from the response of the structure over twenty years of operation is that the foundations have performed well and no issues have been noted with regard to foundation movement.

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