O-Cell assembly and reinforcing cage hoisted for insertion into the barrette slurry trench

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O-CELL TESTING AND FE ANALYSIS OF 28-M-DEEP BARRETTE IN MANILA, PHILIPPINES

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ABSTRACT: Alfaro's Peak is to be a 28-story residential building located in Makati, Manila, Philippines. The loads are high and concentrated, which necessitated supporting the building on deep foundations, penetrating into a residual soil called the Guadalupe Tuff formation encountered at a depth of approximately 15 m. The foundation chosen consisted of a perimeter diaphragm wall combined with rectangularly shaped, 2.4-m^2 cross section, barrettes to support interior columns. A static loading test using the Osterberg-cell (O-cell) test method was performed to study the barrette capacity and deformation behavior. This paper describes the O-cell test, summarizes a finite-element (FE) analysis performed to assist interpretation of the results, and indicates foundation design change adopted as a result of the test. The maximum applied O-cell load during the tests was 11,600 kN. The accumulated upward movement of the top plate was about 10 mm. The accumulated upward movement of the bottom plate was 58 mm, corresponding to about 6% of the barrette width. The results of testing and analyses performed show that the shaft resistance (side shear) acting on the barrette is proportional to the effective stress distribution. This means that any design based on the parameters established from the analysis of the test must include the unloading consequence of basement excavation at the site. The FE computations enabled a comparison between the O-cell test and a conventional head-down test, which indicated that the O-cell test results are representative for the behavior of the barrette in a conventional head-down test and gave insight in the overall load-transfer behavior of the barrette. The O-cell test, strain gauge instrumentation, and FE analysis gave reliable results of decisive importance for the design of the barrettes and other foundation units at the site.

INTRODUCTION

Alfaro's Peak is to be a 28-story residential building located in Makati, Manila, Philippines. The building footprint is a rectangular, 23 by 35 m area and the basement is 12 m below the ground surface. Most of the high-rise buildings in Makati are supported on massive reinforced concrete structural mats that are placed on grade in the Guadalupe Tuff formation, a residual soil considered as bedrock in the Makati area. Typically, four to eight basement levels are excavated. Such deep excavations result in large compensation of the loading, as well as having the benefit of transferring the net foundation loads down to competent soil. However, for the Alfaro's Peak, only two basement levels were required, resulting in a smaller load compensation (larger net loads) and in the bottom of the excavation being placed in relatively weak and compressible overburden soils. The loads are high and concentrated, which necessitated supporting the building on deep foundations, penetrating into the tuff, which was encountered at a depth of approximately 15 m below ground surface.

A reinforced concrete perimeter diaphragm wall with tiebacks was chosen for the support of the excavation. Adoption of the diaphragm wall provided economic and schedule incentives for using barrettes as the deep foundations to support the structure. Barrettes are rectangularly shaped bored piles (drilled-shafts) constructed as individual short-length panels of cast-in-place reinforced concrete. The barrettes are excavated using a clam shell, which results in a rectangular shape, as opposed to the round shape usually considered for bored piles. Both barrettes and walls would be constructed by the same specialty contractor, using the slurry panel technique. The same clamshell, excavating a panel of a nominal 0.85-m width, would be used for both the diaphragm wall and the barrettes.

Discussion with local consultants disclosed that there was no documented experience in the Makati area with comparable types of foundation. Moreover, only limited factual data were available regarding shaft and toe resistances and values of elastic modulus for deep foundations supported in the volcanic tuff. These factors, together with a review of the results of geotechnical exploration of the volcanic tuff (discussed later), suggested caution in the adoption of the proposed foundation design. It was therefore necessary to perform a static loading test to study the barrette capacity and deformation behavior. However, with the very high design loads (6,100 kN for the smallest of the barrettes), a conventional static loading test would have been prohibitively costly and, also difficult to arrange in the available time and space. Hence, the Osterberg-cell (O-cell) test method was selected, and a O-cell test was conducted on October 18, 1996. A retest was carried out on December 5, 1996, 48 days later, at the discretion of the barrette installation specialty contractor.

This paper describes the O-cell test, summarizes a finite element (FE) analysis performed to assist interpretation of the results, and indicates a foundation design change that was adopted as a result of the test. The information on the geotechnical parameters of the Guadalupe Tuff and results of the loading test are considered to be of general interest for the design of similar deep foundations and, specifically, in the Makati area.

INITIAL FOUNDATION DESIGN

For use in the initial foundation design, soil resistance and modulus characteristics were provided by a local consultant. The values were based on judgment because no test data for heavily loaded barrettes or caissons supported at depth in the volcanic tuff were available at the time. Thus, the barrettes were designed as rock-socketed piles using a total stress analysis with allowable values in the volcanic tuff of 150 kPa for unit shaft resistance and 2,000 kPa for unit toe resistance. The

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dead plus live column loads applied to the foundations ranged from 12,200 to 49,500 kN. The barrette width of 0.85 m was controlled by the size of the excavating clamshell, and the design calculations indicated barrette lengths ranging from 2.85 to 8.0 m and extending to depths of 28–34 m below the original grade, or 16–22 m below the basement slab.

Settlement calculations employed a Young's modulus assumed to be 1,000 times the unconfined compressive strength of 1.0 MPa; that is, 1,000 MPa resulted in calculated settlement values ranging from 4 to 7 mm.

SUBSURFACE CONDITIONS

The soils encountered in a borehole located close to the test barrette consisted of soft to stiff clay extending to 11 m below the ground surface, followed by 4 m of clayey silty sand or clayey sandy silt. At a depth of 15 m, the material consists of the Guadalupe Tuff, believed to be of volcanic origin, also locally known as the Adobe formation (Fig. 1). The Guadalupe Tuff is a weakly cemented sandy clayey silt, but includes layers or lenses of weak sandstone with local strong zones. Results from several borings at the site indicated that the levels at which the stronger layers occur were quite variable across the site. Nearest the test barrette, the tuff profile consisted of siltstone from 15 to 20 m and sandstone from 20 to 26 m, followed by siltstone with seams of sandstone extending to well below the depths considered for the barrettes.

The geotechnical investigation performed at the Alfaro's Peak site was typical of the normal coring and sampling, and testing practice of the Guadalupe Tuff in the Makati area, employing NQ-size coring with preservation of selected cores wrapped in plastic. The core recovery was typically 40–60%. In accordance with local practice, the testing consisted of determining unconfined compressive strength, dry density, and water content. The unconfined compressive strength of the recovered cores ranged from 800 to 6,000 kPa, with two-thirds of the results in the 800–2,100 kPa range. Because of the porous structure of the volcanic tuff, the dry density values were low, ranging from 1,100 to 1,400 kg/m^3. The water content of the core samples was reported to be generally about 20%, with a few values about 40%.

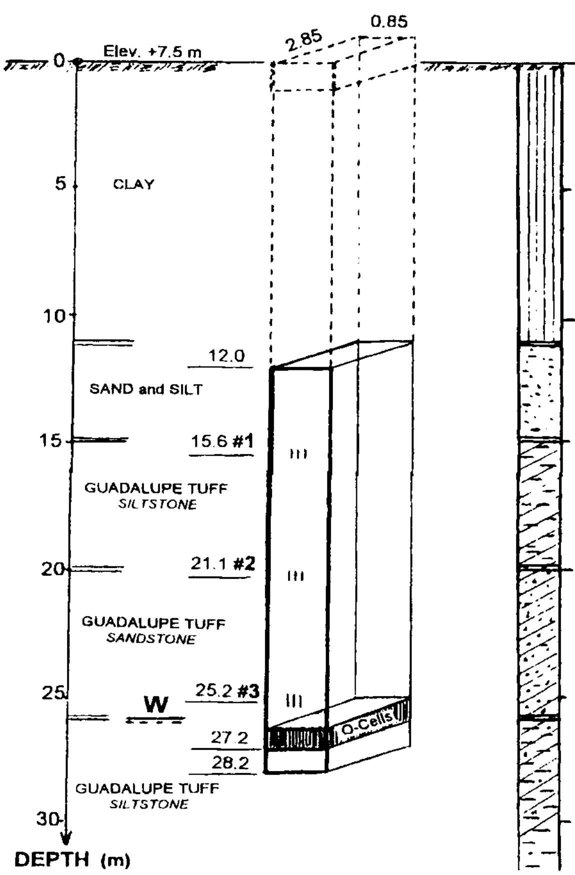

FIG. 1. Schematic Profile of Site and Barrette

The reported range of the water content values was considered inconsistent with the reported high compressive strength. During the foundation design evaluation, therefore, the solid density ("specific gravity") was determined for a few samples from the excavation of the test barrette, which enabled phase-systems calculations of the degree of saturation of the tested samples. (No solid density tests were included in the site investigation.) The solid density was 2,670 kg/m^3 and calculations indicated that, for most samples, the reported values of water content corresponded to a low degree of saturation, most likely caused by partial drying of the core samples prior to testing. The reported values of unconfined compressive strengths and Young's modulus were therefore questionable. The natural water content at full saturation was about 45% and the actual void ratio was about 1.2.

Two standpipe piezometers were installed close to the location of the test barrette to provide the pore pressure data necessary for an effective stress analysis of the results of the barrette test. One standpipe was sealed at a depth of 15 m, and the second at a depth of 30 m below the ground surface. As measured in the piezometer installed at the depth of 15 m, before constructing the test barrette, a perched water table existed at a depth of about 8 m. The piezometer installed at a depth of 30 m showed a phreatic height of only 4 m, indicating a downward gradient at the site. During construction of the test barrette, the perched water table disappeared (i.e., the construction drained the water from the soil). The piezometric level at the time of the test was therefore approximately 2.0 m above the bottom of the barrette (and 1.0 m above the O-cell level).

Review of the above summarized geotechnical data for the Alfaro's Peak site raised concern regarding the initial design of the barrettes. The low density of the siltstone appeared to indicate a compressible, brittle structure that could be viewed to conflict with the relatively high values of unconfined compression strength and Young's modulus, originally proposed for this deposit, but now questioned. The indicated high strength of the material also appeared to conflict with local experience with deep excavations of the Adobe deposit, which can be excavated by a backhoe without the need for rippers or hydraulic breakers. These considerations contributed to the decision to proceed with a barrette loading test by the O-cell method.

When the review of the test results at the Alfaro's peak site was in progress, barrette or circular caisson foundations in the same Guadalupe formation were considered for another site in Makati. In view of the limited data from the Alfaro's Peak site investigation and the smaller than desired capacity of the test barrette established by the testing (as detailed below), a more comprehensive investigation was performed. Although at that site, the Guadalupe Tuff commenced at a greater depth (about 25 m), the measured characteristics are considered to also be applicable to the formation at the Alfaro's Peak site. That investigation was carried out to a depth of 80 m below ground surface and included, in addition to NQ-size coring, undisturbed sampling by a Pitcher triple-barrel sampler, which was brought from the United States to Manila for the investigation. This sampling method resulted in excellent recovery of the soft zones of the volcanic tuff, which were largely lost during the previous NQ coring.

Laboratory testing on samples obtained from the other site indicated void ratios in the tuff, averaging about 1.1 to a depth of 75 m. The particle size distribution averaged 54% of sand, 42% of silt, and 4% of clay. Some more sandy layers as well as more clayey layers, including occasional lenses with gravel-size particles were also present. The liquid limit of the finer zones ranged from 30 to 60%, the plastic limit was between 26 and 32%, and the plasticity index ranged from 0 to 29, averaging about 12. The solid density was 2,670 kg/m^3.

Oedometer tests on Pitcher samples on the softer (siltstone) portions of the tuff indicated a quasi-preconsolidation stress close to the effective overburden pressure, and a compression index C_c ranging from 0.04 to 0.17, averaging about 0.1. The compression indices and void ratio values correspond to a Janbu modulus number of about 50. The initial slope of stress-strain curves in unconsolidated-undrained compression tests gave an "undrained elastic" modulus that ranged from 7 to 27 MPa, averaging about 17 MPa. The average slope of unloading-reloading curves gave values ranging from 17 to 54 MPa, averaging about 30 MPa (the values are much smaller than the elastic modulus of 1,000 MPa used in the initial design). These values of "elastic" modulus correspond to a Janbu modulus number of about 200–300 (Fellenius 1996).

Triaxial tests on Pitcher samples on the softer portions of the tuff resulted in an undrained shear strength ranging from 69 to 147 kPa, with an average of 115 kPa.

Unconfined compression tests on the NQ cores indicated an unconfined compressive strength ranging from 250 to 2,500 kPa, averaging about 750 kPa. While it is possible that the strength of the core samples increased during the period between sampling and testing due to inadvertent partial drying in the hot Manila climate, the core samples are also undoubtedly indicative of the stronger portions of the Guadalupe formation. However, much of the formation consists of the softer materials not recovered by NQ coring. Also, the levels at which the harder and the softer portions of the formation occur vary considerably in adjacent boreholes. For these reasons, the lower-strength, more compressible materials may control the bearing capacity and settlement of deep foundations.

PRINCIPLES OF O-CELL TEST

The O-cell method (Osterberg 1998) incorporates a sacrificial hydraulic jack-like device (Osterberg-cell) placed at or near the toe (base) of the pile (drilled-shaft or barrette) to be tested. When hydraulic pressure is increased, the O-cell expands, pushing the shaft upward and the base downward. The upward movement of the O-cell top plate is the movement of the shaft at the O-cell location and it is measured by means of telltales extending from the O-cell top plate to the ground surface. In addition, the separation of the top and bottom O-cell plates is measured by displacement transducers placed between the plates. The downward movement of the O-cell base plate is obtained as the difference between the upward movement of the top plate and the cell plate separation. It is important to realize that the upward and downward load movements are not equal. The upward load movement is governed by the shear resistance characteristics of the soil along the shaft, whereas the downward load movement is governed by the compressibility of the soil below the pile toe.

At the start of the test the pressure in the O-cell is 0 and the self-weight of the barrette at the location of the O-cell is carried structurally by the O-cell assembly. The test consists of applying load increments to the barrette by means of incrementally increasing pressure in the O-cell and recording the resulting plate separation and telltale movements. The first pressure increments transfer the barrette self-weight from the assembly to the O-cell fluid. The O-cell load determined from the hydraulic pressure reading at completed transfer is the self-weight value and it is reached at minimal movement (i.e., separation of the O-cell plates). The self-weight consists of the buoyant weight of the barrette plus any residual load in the pile at the location of the O-cell.

When the full self-weight of the barrette has been transferred to pressure in the O-cell, a further increase of pressure expands the O-cell; that is, the top plate moves upward and the base plate moves downward. (The assembly is built with an internal bond between the plates, a construction feature, which breaks when the separation starts.)

The O-cell load versus the upward movement is the load-movement curve of the barrette shaft. The O-cell load versus the downward movement is the load-movement curve of the barrette base. This separation of the load-movement behavior of the shaft and base is not obtainable from a conventional static loading test. Of course, the self-weight must be subtracted to obtain the load-movement of the pile shaft. The self-weight should be included in the load movement for the pile toe, however.

TEST BARRETTE AND TESTING PROGRAM

To minimize the magnitude of the required test load, one of the two smallest planned barrettes was selected for the O-cell test. The barrette was produced during the diaphragm wall construction in advance of the planned barrette construction. Fig. 2 shows a photograph of the reinforcement cage, O-cell assembly, and some of the instrumentation of the test barrette while lying on the ground prior to insertion into the slurry trench. The test barrette had a cross section of 2.85 by 0.85 m (area 2.42 m^2) and was constructed to a depth of 28.2 m (base level) below the ground surface. The upper 12 m of the barrette was made from a very weak concrete to facilitate the future excavation to this depth at the site.

The barrette was constructed by excavating the soil and soft rock using a 0.85-m-wide, mechanically operated clamshell, while supporting the sides of the excavation with bentonitic slurry. Advancement of the barrette excavation through the sandstone layer encountered at a depth of about 20 m below the surface required the use of a chopping bit, and some concerns were expressed that, when the material changed again to siltstone near the bottom of the barrette, some disturbance of the siltstone below the base of the barrette might have occurred as a result of this method of loosening the material. On reaching the full depth, a reinforcing cage was inserted into the slurried hole. The O-cells and the strain gauges for the loading test were attached to the reinforcing cage. Sleeves for later insertion of telltale rods were also included. The slurry was then displaced by concrete, poured by tremie method. The construction was completed on October 12, 1996.

A 0.40-m-thick O-cell assembly was placed with the bottom O-cell plate 1.0 m above the base. The assembly embodied two 550-mm (o.d.) O-cells (the sacrificial jacks) connected in a series to a common pressure hose and pressure gauge in an

FIG. 2. O-Cell Assembly and Reinforcing Cage

assembly of a total plan area of 0.55 by 2.48 m and a height of 0.40 m. The maximum stroke, that is, the separation of the O-cell plates, was 150 mm.

To obtain information on the shaft resistance distribution, the barrette was instrumented with strain gauges installed in groups of four separate gauges at depths of 15.6 m (Level 1), 21.1 m (Level 2), and 25.2 m (Level 3). The highest level was at the approximate level of the boundary between the overburden soils and the Guadalupe Tuff. The second level was approximately at the boundary between siltstone and sandstone layers within the tuff. Level 3 was placed 1.6 m above the O-cell top plate and 3.0 m above the barrette base.

The strain-gauge readings at the start of the test were taken to be zero. Therefore, strain recorded during the test corresponds to the increase of load (for each gauge level) and reflects the load generated by the O-cells after the transfer of self-weight is completed.

The initial test was performed on October 18, 1996, 6 days after completion of the construction. The testing procedure followed the quick test schedule, applying small, approximately equal increments of load at equal short time intervals. The load increments were applied by increasing the pressure in the O-cells in steps corresponding to about 600 kN. The increments were applied approximately every 5 min until the observations showed excessive movement of either the shaft (upward) or the base (downward), or until the maximum capacity of the particular O-cell combination (i.e., 36,000 kN) had been reached. Readings of all gauges were taken at 1, 2, and 4 min into each load increment. A graph of total cell separation versus applied hydraulic pressure was plotted as the test progressed, and the loading was terminated when distinct steepening of the load-displacement curve took place, characteristic of imminent failure. At that time, there was difficulty in maintaining the hydraulic pressure. On December 5, 1996, the O-cell test was repeated using the same loading schedule.

TEST RESULTS

The primary results of the O-cell test are the measured O-cell load versus the recorded plate movements. These results are plotted in two diagrams shown in Fig. 3, presenting the recorded load-movement data for the top and bottom O-cell plates for both the initial test and the retest. Notice that the top-plate diagram is not adjusted for the self-weight of the barrette. The bottom diagram shows the measured expansion of the O-cell, that is, the separation of the O-cell plates occurring below the 6,400-kN O-cell load during the initial test. Because of leaks at the connection of the hydraulic hose to one of the pressure gauges, the test had to be interrupted twice, first in the very beginning of the test and a second time at an O-cell load of 6,860 kN. The interruptions do not seen to have affected the general appearance of the load-movement curves.

During the initial test, at an applied load of 1,060 kN, separation of the O-cell plates had not yet occurred. At the load of 1,664 kN, the next load level, a separation of 1.7 mm was measured. Intersection of the straight portion of the line with the abscissa at 1,400 kN identifies the self-weight to be approximately equal to the calculated buoyant weight of the barrette.

The maximum applied O-cell loads during the initial test and retest were 10,300 and 11,600 kN, respectively. The accumulated upward movements of the top plate at these loads were about 7 and 10 mm, respectively. The downward movement of the bottom plate, presented in the lower diagram, was much larger than the upward movement. The accumulated movements at the maximum loads were 45 and 58 mm, corresponding to about 6% of the barrette width. The shape of the load-displacement curve for the bottom plate is essentially linear below 10,000 kN. Beyond this load, a progressive increase of the bottom-plate displacement is indicated; however, no ultimate load can be defined from the curve.

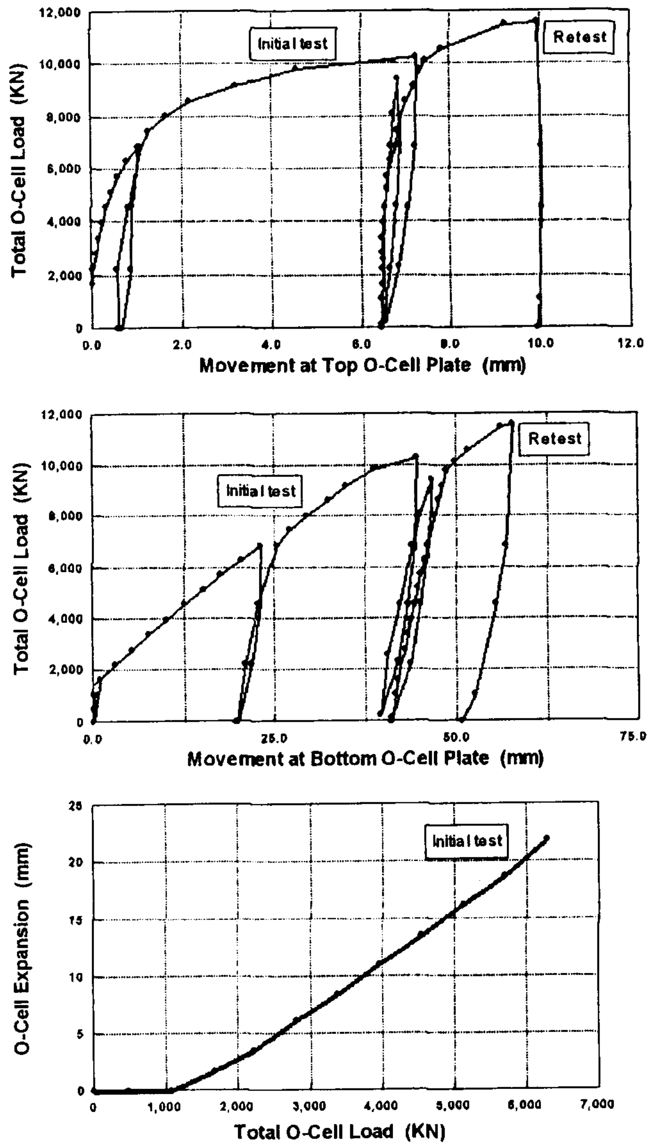

FIG. 3. Load versus Movement of O-Cell Plates

Adjusting for the 1,400-kN self-weight, the maximum force in side shear is 8,900 kN during the initial test and 10,200 kN during the retest. The 7-mm upward movement of the O-cell top plate at the maximum load includes the elastic compression of the barrette, which is estimated to be about 3 mm. Thus, the shaft resistance was fully mobilized at or before a 4-mm relative movement at the top of the barrette—top of sound concrete 12 m below grade. The observation confirms the general wisdom that shaft resistance is mobilized at very small relative movement and is independent of the size (diameter) of the shaft.

The strains recorded by means of the individual strain gauges at Levels 1–3 in the barrette are presented versus the recorded O-cell pressure in Fig. 4 (with the initial reading taken as the "zero" reading). Notice, the three diagrams employ different scales for the ordinates. At gauge Level 1, the four gauges show similar development with increasing cell pressure. However, this is not the case at Levels 2 and 3. At Level 2, the strain gauges agree and disagree in pairs; whereas all strains recorded at Level 3 differ from each other.

The differences in strain gauge values suggest that the loading induced by the two O-cells is uneven. This is what would

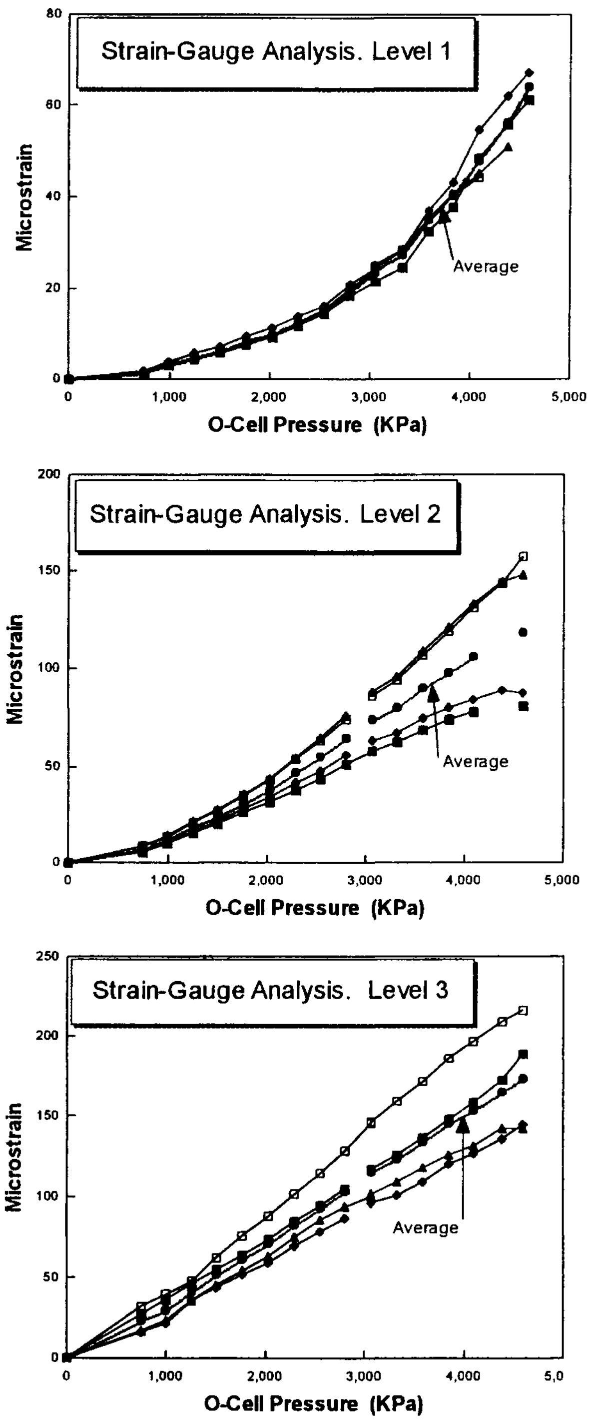

FIG. 4. Strain Measured for Individual Gauges

be expected because it is not possible to place the cells absolutely concentric with the barrette (even if placed concentric to the geometric center of the barrette, the force center may not necessarily be the same as the geometric center). Variation of the soil characteristics around the perimeter of the barrette could also contribute to the appearance of uneven loading, as well as variations of the cross section. Furthermore, as the O-cells are hydraulic jacks and subject to some small inside friction, it is possible that the friction may differ between them. Therefore, uneven stress distribution (bending, in fact) is concluded to be the cause of the observed inconsistent distribution of strain within the cross section of the barrette. As would be the case, this effect is most obvious closest to the O-cell assembly, Level 3, and diminishes farther up the barrette. However, the average strains for each level of gauges are quite consistent.

To determine the stress represented by the strain readings requires input of the Young's modulus of the concrete (in combination with the steel reinforcement cage). According to information from LOADTEST Inc., the concrete cylinder strength corresponds to an E-modulus of 3,500 ksi, that is, 24 GPa. The modulus can also be determined by plotting each increment of load divided by the corresponding increase of strain, i.e., the tangent modulus, against the total strain. This method (Fellenius 1989) builds on that, if the side shear (shaft resistance) has been fully mobilized, the strains measured for the subsequent load increments reflect the strain induced by these load increments as acting on a free-standing column. For cast-in-situ units, it also provides the advantage of including an automatic compensation for the actual cross-sectional dimensions being different to those designed. Fig. 5 shows this plot made for the average value of strain at each strain gauge level. The method is sensitive to small inaccuracies in the values of load and strain.

At Level 3, the side shear was mobilized early in the test and most strain readings, therefore, are uninfluenced by side shear. The average tangent modulus value for the last 11 load increment values is 24.7 GPa. The strains recorded at Levels 1 and 2 are influenced by side shear, and the Level 1 and 1 tangent modulus values are not representative for the E-modulus of the barrette material.

The 24.7-GPa modulus value determined from the tangent-modulus method was combined with the average of the recorded strains to evaluate the stress induced at Levels 1–3. The loads were then obtained as the stress value times the barrette cross-sectional area. Because of the variations between the different gauges, the load values are very approximate. However, the strain gauge data indicate that the resistance to the upward movement of the barrette shaft is proportional to the overburden stress.

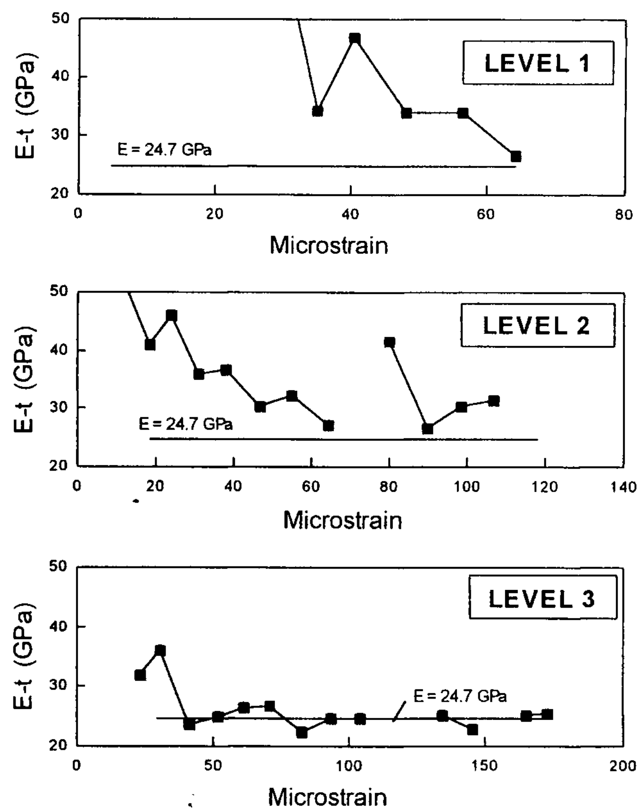

FIG. 5. Tangent-Modulus Plot

EFFECTIVE STRESS ANALYSIS

The strain gauge results are not accurate enough for use in determining the load in the barrette at the gauge levels. However, they do indicate quite clearly that the shaft resistance increases with increasing depth. It is reasonable to assume that the shaft resistance is proportional to the distribution of overburden effective stress. An effective stress analysis of the load transfer for the conditions at the barrette base was made using the UniPile Program (Fellenius 1996). The analysis assumed that the draining of the perched water had been complete and that the ground-water table was located at a depth of 26 m with hydrostatic pore water pressure below this depth.

With regard to shaft resistance, effective stress analysis indicates that the shaft resistance of 10,000 kN found in the O-cell test corresponds to an average Bjerrum-Burland coefficient (β) of 0.2. This is a low value and suggests that a film of bentonitic slurry remains between the concrete and the soil governing the shear transfer from the barrette to the soil. For this reason, there is little sense in assigning different β-coefficients to the various layers of soil and tuff.

With regard to toe resistance, the effective stress analysis indicates that a base load of 10,000 kN (which is approximately the same value as found in the O-cell test), corresponds to a toe-bearing coefficient (N_t) of 9. Based on the soft rock (siltstone) geological designation of this material, a higher value had been expected. However, an N_t-value of 9 is representative for a silt with a 45% natural water content.

For reference to the allowable total stress values of shaft and toe unit resistances applied in the initial foundation design (150 and 2,000 kPa, respectively, as quoted in the section on the Initial Foundation Design of the project), the 10,000-kN loads correspond to an average shaft resistance of 50 kPa and a toe resistance of 4,100 kPa. Moreover, the 45-mm movement and a base load of about 10,000 kN, combined with a Boussinesq stress distribution of the load, correlates to an elastic modulus of 150 MPa, which is only 15% of the originally assumed value for the site. The original values clearly overestimated the site conditions.

DISCUSSION

The total maximum resistance of the test barrette at the initial test was taken as the sum of the net shaft and net base resistances at the 10,000-kN O-cell load, that is, 17,200 kN. This value was taken as representative for the barrette capacity and served as a base for a factor-of-safety assessment of the design. (For correlating the test results to a conventional head-down loading, the barrette self-weight is subtracted from the O-cell load.)

The barrettes of the size tested were intended for a load of 12,200 kN (beyond the self-weight) to be distributed on two barrettes. This indicates a factor of safety of about 2.8 on the net pile capacity of 17,200 kN taken as the barrette head-down capacity. However, the actual foundation barrettes will be working under a smaller effective overburden stress because of the 12-m-deep excavation. A repeat effective stress analysis for the actual conditions and applying the β-coefficient and N_t-coefficient as calibrated from the test results shows that the net capacity of a single barrette would be no more than about 11,000 kN—shaft resistance of 4,000 kN and base resistance of 7,000 kN—that is, the actual factor of safety would be 1.8, which was not considered adequate. However, more important than the factor of safety is that a design must safeguard against excessive settlement for the applied loads. The O-cell test results indicate that the barrette movement for the loads from the structure would be larger than acceptable. Moreover, applying the back-calculated shaft and toe-resistance values to the larger, much more heavily loaded barrettes planned for the site, resulted in lower factors of safety and larger calculated settlement than indicated for the test barrette. Very substantially deeper barrettes, or substantially more numerous barrettes, would have been required to provide a satisfactory design.

As mentioned, a repeat test was carried out 2 months after the initial test. The purpose was to investigate if the barrette capacity would increase with time after installation and the O-cell test was repeated. The test was brought to a maximum total O-cell load of 11,600 kN, which is 1,300 kN (12%) more than the maximum value of the initial test. The maximum load occurred at an additional movement of the top plate (upward) of about 3 mm. The maximum additional movement of the bottom plate (downward) was 13 mm.

The diagrams shown in Fig. 3 suggest that the increase of load found in the retest is due to the total increase of the barrette movement. This is clearly indicated by the load movement for the bottom plate, though less so for the load movement of the top plate, unless the last load value of the initial test is neglected. The implied increase in shaft resistance could be due to continued setup after the construction. It could, possibly, also be due to the fact that the draining out of the perched water had not quite been completed at the time of the initial test. At the retest, therefore the effective stress would have increased, resulting in a corresponding increase of shaft resistance. Most probable, however, is that the retest simply continued where the initial test finished, and the small increase is quite simply due to a strain-hardening effect.

The increase of capacity was not material enough to affect the conclusion that the as-designed barrette foundation was inappropriate for this building. After a comparison of the economics involved, it was decided to replace the barrettes by a mat foundation placed on the tuff, requiring some increase in the depth of excavation.

While not considered for the subject project, it would not be unrealistic to place O-cell devices in each barrette and preload the base of each barrette to control the movements due to the subsequent construction of the building. This solution could have ultimately made the barrette foundation appropriate.

NUMERICAL SIMULATION

After the O-cell test was completed, it was suggested that the method of testing (the O-cell test), could have caused the capacity to be smaller than that found in a conventional test where the load is applied to the pile head. As mentioned, for several reasons, performing a conventional test would not be possible. To yet clarify the issue, it was decided to calibrate the site and barrette conditions by means of an FE simulation of the O-cell test and then to use the calibration results to simulate a conventional head-down test. The simulation was completed before the more detailed soil data from the other Manila site was available.

The Advanced Geotechnical Analysis Code (AGAC) FE program developed by Altaee (1991) was used for the numerical analysis. The Advanced Geotechnical Analysis Code program has been used in the analysis of several full-scale situations including cases of instrumented piles subjected to repeated axial loading [e.g., Altaee et al. (1992)]. The program treats the soil as an elastoplastic material and uses the bounding surface plasticity model (Bardet 1986; Altaee 1991) to model the stress-strain-strength response of the soil. The barrette was modeled as a linear elastic material with an elastic modulus of 25 GPa. the E-modulus in the upper 12-m portion of the barrette was reduced to 2.5 GPa to account for the weak concrete in this portion. A Poisson's ratio of 0.2 was used for all of the barrette.

The ground-water table was set at a depth of 26 m and the

TABLE 1. Soil Parameters

Layer (1)	Γ (2)	φ (3)	e (4)	C_e (5)	C_r (6)
0–28 m	1.6	30	1.2	0.120	0.003
28–30 m	1.6	35	1.2	0.120	0.010
30–50 m	1.6	35	1.2	0.120	0.003
Shaft band	1.6	12	1.2	0.120	0.003

pore pressure was assumed to be zero above this depth. Below the ground-water table, the pore water pressure distribution was assumed to be hydrostatic.

The soil profile was assumed to consist of three soil layers. An initial parametric analysis indicated that the conventional soil parameters given in Table 1 would be representative for the soil profile. In a 0.1-m-thick zone nearest the barrette side, a band of weaker soil (the slurry film) was assumed to exist. In Table 1 the symbol Γ stands for critical void ratio at 1-kPa mean effective stress. The symbol C_c stands for the slope of the steady-state line in an e-log(p) plane during loading, where e is the void ratio and p is the mean stress [$p = (\sigma_1 + \sigma_2 + \sigma_3)/3$]. The symbol ϕ stands for the soil angle of friction and is assumed equal in compression and extension. No strain-softening response was assumed; that is, the angles of friction at peak and postpeak are equal. [For more details about the parameters, see Altaee and Fellenius (1994a; 1994b).]

RESULTS OF MATCHING MEASURED SIMULATED LOAD MOVEMENT TO LOAD MOVEMENT

The FE computed load movements for the top and bottom O-cell plates are shown in Figs. 6 and 7, respectively, together with the measured values. The agreement between the computed and measured curves is forced by trial-and-error procedures. To obtain the agreement, the soil below the barrette base had to be assigned a less stiff response than that used for the soil above the base. This is consistent with the conditions encountered in the nearest boring, which indicated the presence of the more compressible siltstone commencing 2 m above the bottom of the barrette and continuing below the barrette.

The FE analysis indicated that the soil movements diminished rapidly with the horizontal distance from the barrette and were very small beyond a distance of one barrette width. The effect of the barrette movement diminished less rapidly below the base. At a distance of 1.7 m, two diameters (barrette widths), the magnitude was still about 50% of the movement of the base. At a distance of 4 m, about five diameters, below the base, the movement was bout 10% of the base movement.

The FE analysis also computed the load at the three strain gauge levels. Fig. 8 presents the computed total loads plotted against the measured total O-cell loads. The 1:1-sloping dashed line represents the O-cell load plotted against itself. Thus, the distance between the dashed line and the gauge-level curves represents the calculated reduction of load due to shaft resistance over the distance from the O-cell and the respective gauge levels. At the O-cell load of 1,400 kN, which is the self-weight of the barrette, the barrette started to move, upward, and downward, against the soil. Because the O-cell load must overcome both the weight of the barrette and some small side shear along the 1.6-m distance from the O-cell to gauge Level 3, the load at Level 3 is slightly smaller than the O-cell load.

Fig. 9 illustrates the computed response of the strain gauges to the O-cell loads at the strain gauge levels in terms of "measured" and "computed" loads. The comparison shows that the "zero" values of the strain gauge measurements are off for Levels 2 and 3. To be a correct measure of the load in the barrette, the measured strain gauge loads should have started at or close to the abscissa, similar with the FE computed val-

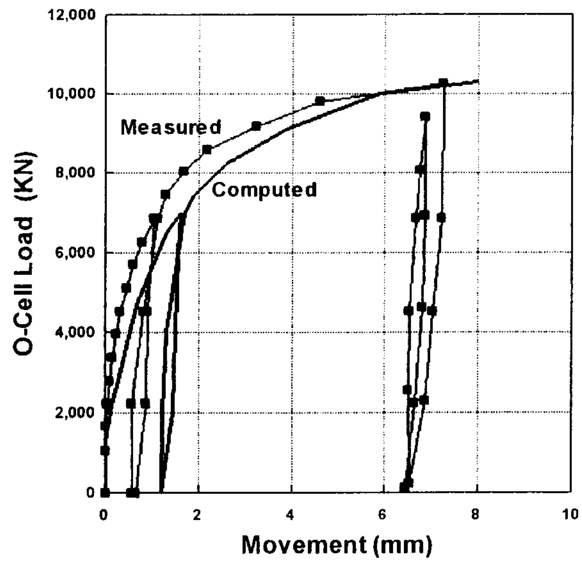

FIG. 6. Barrette Movements at O-Cell Top Plate

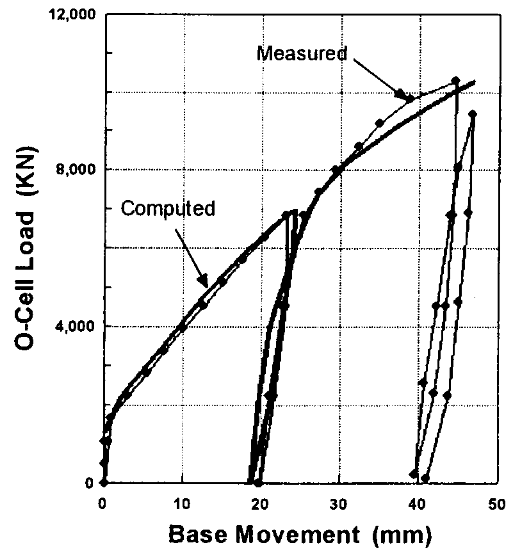

FIG. 7. Barrette Movements at O-Cell Bottom Plate

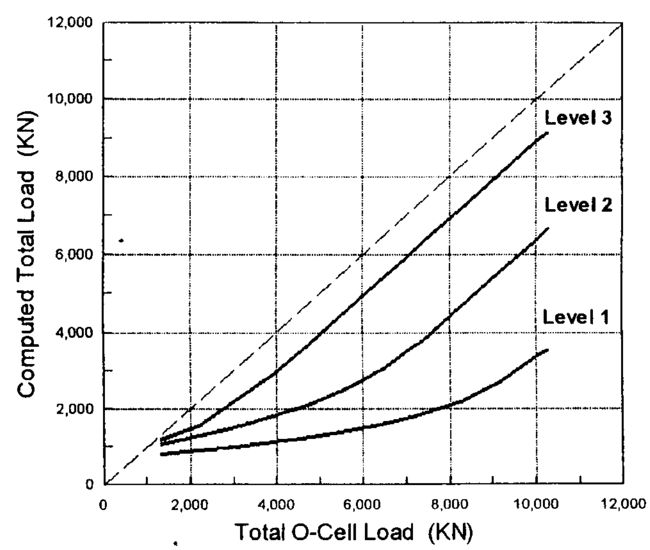

FIG. 8. Computed Strain Gauge Loads

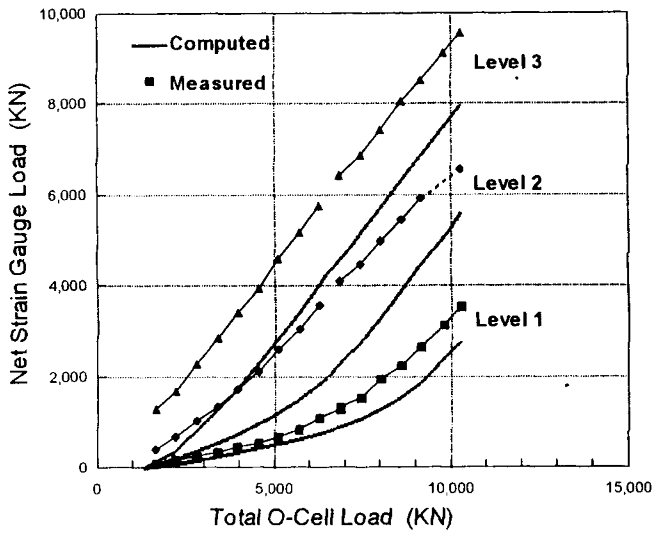

FIG. 9. Strain Gauge Loads versus O-Cell Loads

ues. The cause of the overestimation is not known. It could be due to an uncertain "zero-reading" or, an effect of uneven loading inducing variable stress and strain (bending) over the barrette cross section. It is not possible that friction and other resistance in the O-cell could have caused the O-cell values to be underestimated, because such an occurrence would have resulted in smaller O-cell loads. The trend of the strain gauge values appears to be correct, however, and there is good agreement between the computed and measured values in this regard. Bending stresses are therefore considered to be the most plausible cause. Qualitatively, the diagram confirms that the loads at the strain gauge levels reduced in approximate conformity with the change of effective stress between the levels.

RESULTS OF SIMULATED CONVENTIONAL TEST

To respond to the mentioned suggestion that the O-cell test would be fundamentally different to a conventional head-down static loading test, a conventional static loading test was simulated in a repeated FE computation. The weak concrete in the 12-m upper portion of the barrette was made equal to that of the rest of the barrette (same E-modulus, 25 GPa), to enable the barrette to resist the loading, but all other input parameters for the barrette and the soil were kept the same as that used for the O-cell test simulation.

Fig. 10 presents the distribution of axial load in the barrette for the two types of tests, the O-cell test and the head-down test ("head test"). Two head test curves are shown. The left of the two curves is for the case of a maximum load applied to the barrette head equal to twice the net O-cell test load during the initial test (i.e., 17,800 kN). The right of the two curves is for the case of equal base movement, which required a slightly larger total load (19,100 kN) to be imposed at the barrette head.

All three distributions exclude the self-weight of the barrette. When comparing the O-cell test to a head-down test, the net loads must be used, because in an O-cell test, the pressure corresponding to the self-weight value cancels out the self-weight. In contrast, in conventional head-down test, the self-weight is already in the pile at the start of the test and stays in the pile during the entire test. The self-weight is usually a small portion of the maximum load, and the issue of self-weight is normally only of "academic" interest. In the subject test, however, the self-weight of 1,400 kN is a considerable portion of the maximum load and cannot be neglected.

The similarity between the two test results is evidenced by the parallel behavior of the load distribution curves of the two tests and that the same amount of load acts at the barrette base.

Fig. 10 also presents the loads determined at the three strain gauge levels at the maximum O-cell load. The loads are shown as a horizontal bar indicating the range of values determined separately for each of the four strain gauges at each level.

Additional results of the FE analysis of the head-down test are that, in contrast to the O-cell case, the computed movements in the soil outside the barrette side do not diminish appreciably with the distance from the barrette, but extend more than one barrette width beyond the barrette side. The reason for this is that the relative movement between the barrette and the soil is larger in the head-down test (due to the fact that the movement is the sum of shaft and base movements as opposed to the movements in the O-cell test, which

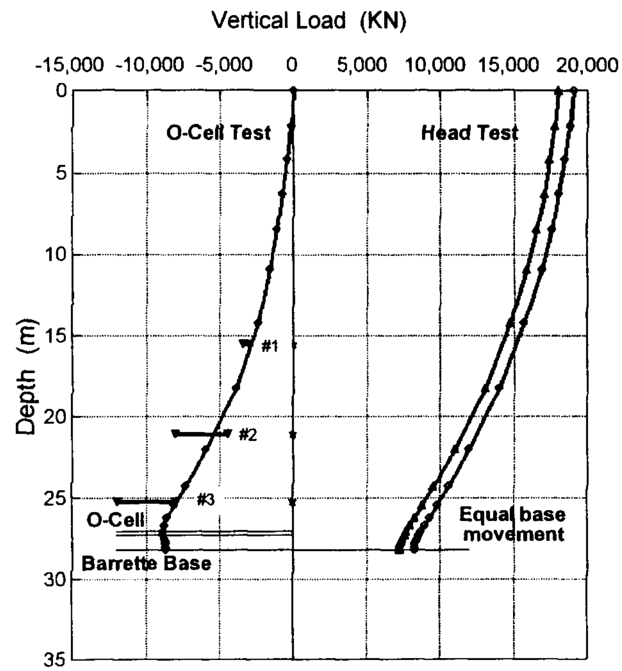

FIG. 10. Vertical Load versus Depth for O-Cell and Head Test

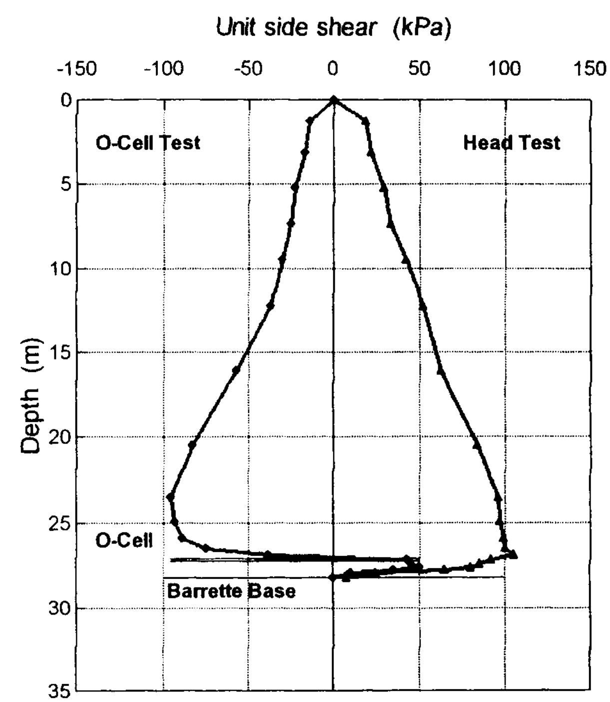

FIG. 11. Unit Side Shear versus Depth for O-Cell and Head Test

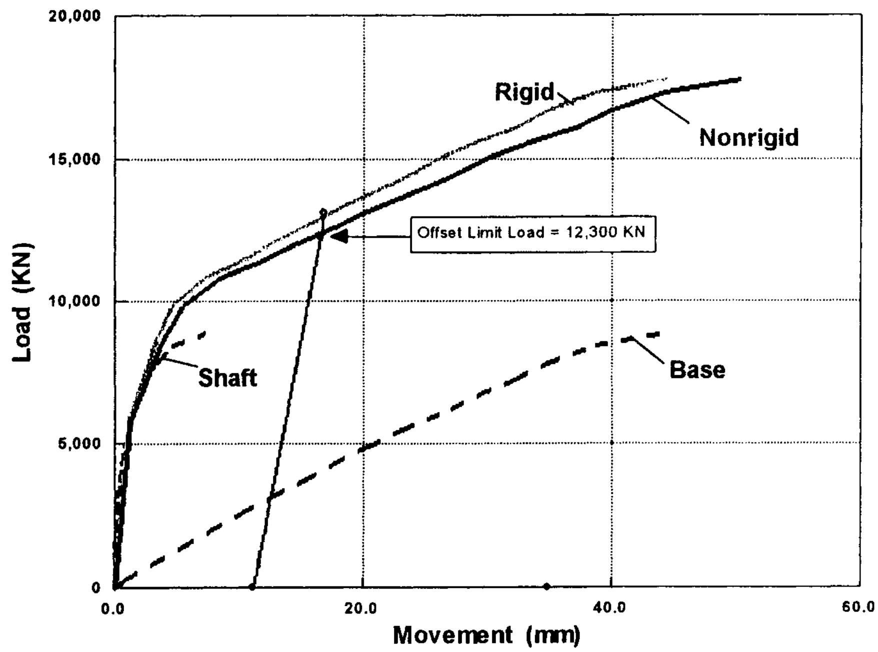

FIG. 12. Load-Movement for Equivalent Head-Down Test. Base = Recorded Load Movement of Base Plate; Shaft = Recorded Load Movement of Top Plate; Rigid = Head-Down Load Movement for Rigid Shaft; Nonrigid = Head Down Load Movement for Nonrigid Shaft

occur separately). As to movement below the barrette, the analysis does not indicate any appreciable difference between the two methods of loading.

Fig. 11 presents the unit shaft resistance distribution (side shear) for the barrette as computed for the two types of tests. Only one distribution is shown for the head test because the mobilized shaft resistance is the same for both cases of applied head load. The shaft resistance acts in the negative direction for the O-cell test and in the positive direction for the head test. Generally, there is very little difference between the computed unit side-shear values for the two types of tests. Notice that for the O-cell test the unit resistance reduces above and below the cell level and that, similarly, it reduces near the barrette base for the head-down test.

SIMULATING HEAD-DOWN CONVENTIONAL TEST USING O-CELL TEST DATA

The O-cell load movement curves are sometimes used to construct an equivalent head-down test. This then normally assumes that the tested pile is rigid and does not experience any shortening for the load. The head-down curve is obtained by adding the loads measured for the base and the shaft at equal movement values. However, the omission of the shortening is not necessary because it can easily be calculated. Fig. 12 shows the recorded base and shaft O-cell curves together with the equivalent head-down curves for both rigid and nonrigid conditions. When comparing the rigid and nonrigid curves, the importance of including the elastic shortening of the pile is obvious.

The construction of the equivalent head-down load movement curve is a simple exercise aiming toward producing a conventional load movement curve to use for determining a bearing-capacity value and a factor-of-safety reasoning. For example, as is commonly used for a conventional static loading test, the Davisson offset-limit load construction is also included in Fig. 12 (Davisson 1972; Fellenius 1996). The offset limit load is often used as a lower-bound capacity in a factor-of-safety consideration. The value determined for the simulated head-down test is 12,300 kN. Using this value instead of the 17,200-kN net load, the calculated factor of safety reduces from 2.9 to 2.0. The simulated head-down test does not include the effect of the excavation mentioned above. After excavation, the factor of safety is further reduced. Thus, the analysis of the simulated head-down test confirms the assessment that the barrette foundation as originally designed would not be suitable for the structure.

The head-down curve may be useful to engineers who are more comfortable with using results from a conventional loading test. However, the main advantages of the O-cell test are that the shaft and toe behaviors are determined separately and the load movement of the pile toe is directly obtained. This is important information on which a conventional test only provides an allusion. Commonly, pile design does not include settlement analysis. Instead, settlement is assumed satisfactory if a static loading test shows that a good factor of safety is at hand. This is not always correct, nor safe. In contrast to the conventional head-down test, the O-cell test results can be of significant value to a settlement analysis.

CONCLUSIONS

1. The highly successful O-cell test provided for the first time factual data on shaft and toe resistances for deep foundations in the Guadalupe Tuff formation of the Makiti area.
2. The measured load movement for the barrette base could be directly applied to determine the settlement potential of the barrette foundation.
3. The O-cell test and the strain gauge instrumentation gave reliable results of decisive importance for the design of the barrettes and other foundation units at the site.
4. The FE computation was forced to an excellent agreement with the O-cell test data, making it highly credible that the FE simulation of the head-down test is equally representative.
5. The FE computation indicated that the O-cell test results are representative for the behavior of the barrette in a conventional head-down test.
6. The shaft resistance (side shear) is proportional to the effective stress distribution. This means that any design based on the parameters established from the analysis of

the test must include the unloading consequence of the excavation at the site.

7. The back-calculated β-coefficient is about 0.2. It is not known if this low value is due to slurry remaining between the concrete and the tuff or to the loose structure (high void ratio) of the tuff.
8. The back-calculated toe-bearing coefficient for the maximum base load is about 9, which is representative for a soft silt. Nota Bene, the barrette base did not reach failure at the maximum test load. However, because of the large movements, a higher capacity could not be employed in the foundation design.
9. The soil below the barrette base is more compressible than the soil along the side of the barrette. This could be due to differences in soil composition; the soil at the base was siltstone, whereas half the barrette length in the tuff was in sandstone. However, it could also be due to installation disturbance.
10. The strain gauge values are influenced by uneven stress distribution across the barrette.

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