13 Aug 2008, 5:15pm - 6:45pm

Case History of Osterberg Cell Testing of a Φ1500mm Bored Pile and the Interpretation of the Strain Measurements for Princess Tower, Dubai, U.A.E.

A. Canan Emrem
Zetas Zemin Teknolojisi, Dubai Branch, Dubai, U.A.E

H. Fatih Kulac
Zetas Zemin Teknolojisi, Dubai Branch, Dubai, U.A.E

H. Turan Durgunoglu
Bogazici University, Bebek, Istanbul, Turkey

Gorkem Icoz
Zetas Zemin Teknolojisi, Alemdag, Istanbul, Turkey

6th International Conference on
Case Histories in Geotechnical Engineering, Arlington, VA, August 11-16, 2008

CASE HISTORY OF OSTERBERG CELL TESTING OF A ϕ1500mm BORED PILE AND THE INTERPRETATION OF THE STRAIN MEASUREMENTS FOR PRINCESS TOWER, DUBAI, U.A.E.

A.Canan Emrem
Vice Manager
Zetas Zemin Teknolojisi
Dubai Branch, Dubai, U.A.E

H.Fatih Kulac
General Manager
Zetas Zemin Teknolojisi
Dubai Branch, Dubai, U.A.E.

H.Turan Durgunoglu
Professor Emeritus
Bogazici University
Bebek, Istanbul, Turkey

Gorkem Icoz
Director of Engineering Group
Zetas Zemin Teknolojisi
Alemdag, Istanbul, Turkey

ABSTRACT

This paper reports a well-documented case history of a successful Osterberg Cell testing performed on a working pile cast for the foundations of the Princess Tower in Dubai. Princess Tower which is considered as the tallest residential tower in the world (currently being registered with Guiness Book & Records) up to an approximate 5000 tons demonstrating the efficiency of the testing methodology when accompanied by high-quality construction technique. The tower is 107 floors high and contains a combination of luxury apartments, offices, sales outlets, car parking spaces, sports and recreational clubs and hotel suites. The Osterberg load test was carried out on a pile referred as P34 on October 22, 2006. The main objective of this load test was to proof-load the test pile to its maximum test load of 4,950 tons which is 1.5 times the safe working load of 3,300 tons. For this purpose 3x900 tons capacity hydraulic jacks were utilized. The test pile was a 1,500 mm diameter bored pile with a total embedded length of 47.6m below the cut-off level. Eight levels of vibrating wire-type strain gauges comprising three units at each level were also installed on the test pile to measure strains at nominated locations. According to settlement values and the satisfactory cross hole sonic logging results, it was concluded that the tested pile can safely carry the design working load in compression with settlements within the allowable limits.

INTRODUCTION

Pile load test is considered as one the best and the most realistic tool to observe the behaviour of the pile under working loads. This paper reports the facts, observation and results of a well-documented application of a successful bi-directional static load test using Osterberg cell performed on a working pile cast for the foundations of the Princess Tower in Dubai, U.A.E.. The test was carried out on the October 22, 2006 on a Ø1500mm pile identified as P34. The objective of the test can be listed as:

- Prove that the pile can safely carry the design load as per the specified code and performance requirement,
- Identify the distribution of mobilized skin friction and determine the portion of the load transferred to the pile tip,
- Further implement the test by checking the integrity of the pile against any possible discontinuities through the pile structure using cross-hole sonic logging.

PROJECT DESCRIPTION

Princess Tower is considered the tallest residential tower in the world (currently being registered with Guiness Book & Records). This tower occupies an area of 37,410 square feet in the Marina area, which is known to be one of the most exceptional and prestigious parts of Dubai. It directly looks on the sea and Palm Island in Jumeirah. The tower will consist of 107 floors having a total height of 414 meters. The tower is also near the Palm Jumeirah which is the world’s largest man-made island and one of the most striking landmarks of Dubai.

Upon completion of the construction works, the tower will serve as a multi-purpose building consisting of luxury apartments, offices, sales outlets, car parking spaces, sports and recreational clubs and hotel suites. The six of the 107 floors were planned as the basement requiring an excavation of approximately 21.3 meters deep.

Figure 1. The Princess Tower

SUBSURFACE PROFILE

Lying in the heart of the world's arid zone, the regional geology of United Arab Emirates (UAE) has been substantially influenced by the deposition of marine deposits associated with the continuous sea level changes during the relatively recent geological time**.** Superficial deposits consisting of beach dune sands with marine sands and silts are underlined by alternating layers of calcarenite, carbonate sandstone, sands as well as cemented sand layers. Based on the soil investigations performed for the subject site, subsoil profile consisted of medium to very dense sands and silty sands existing down to approximately 24.0m depth. The denseness of the silty sand layer increasing with depth and becoming very dense at the bottom. The sand layer is underlained by the layers of very weak to moderately weak calcareous Sandstone/ conglomeritic Calcisiltite/ calcerous Siltstone/ calcerous Conglomerate/ Calcarenite interbedded with sandstone bands, silty gravel and cemented gravelly sand-calcerous. The average unconfined compressive strength (UCS) for the upper 10m thickness of rock Layer No.3 was found by laboratory tests as 800 kPa and below that average UCS of rock Layer No.4 was found as 1700 kPa. The UCS distribution vs. depth is presented in Figure 2 and the typical soil profile is presented in Figure 3. The ground water table was encountered at approximately 7.0m depth.

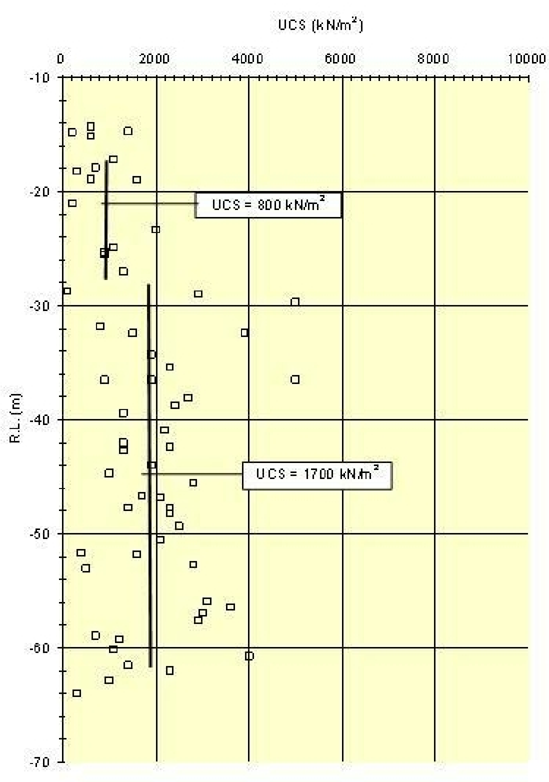

Figure 2. The variation of UCS values of bedrock with depth

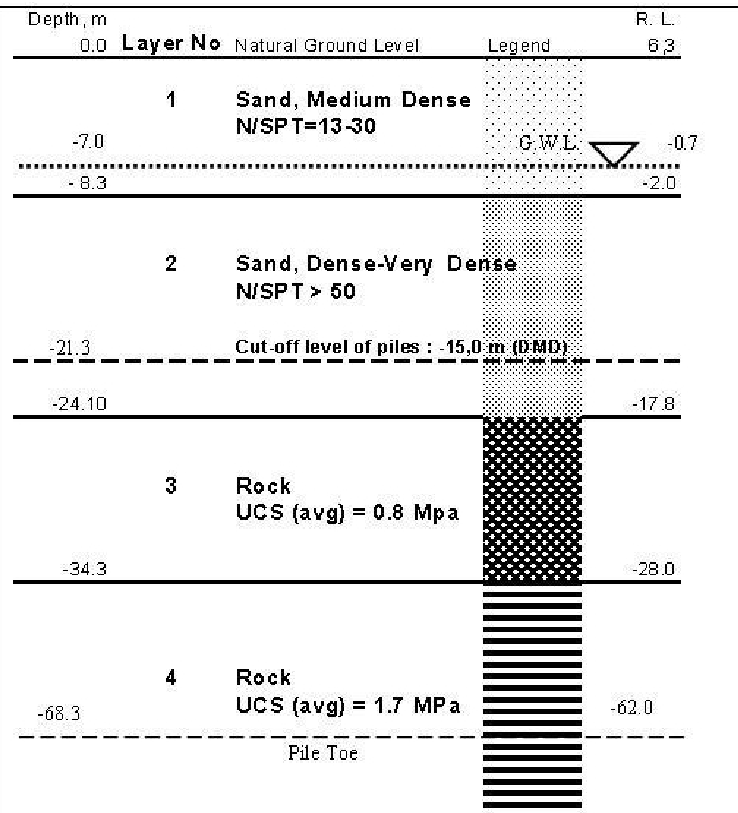

Figure 3. Soil Profile Modeling

FOUNDATION SYSTEM

High structural loads and the existing subsurface conditions required a deep foundation system well socketed within the bedrock for the support of the tower. The deep foundation system consisted of bored piles of different sizes and lengths.

Design Parameters and Methodology

For the design of the piles to achieve design vertical load capacities, basic strength parameters utilization and pile capacity approaches were used together with the soil profile given in Figure 3. Since the pile cut-off level is at -15m (relative to Dubai Municipality Datum:DMD), Layer No.1 will have no contribution and Layer No. 2 will have partial contribution for the pile capacity.

Layer No.2

$f_s = K_s \cdot \sigma'_{v0} \cdot \tan\delta$

where;

K_s	:	Average coefficient of earth pressure on pile shaft and taken as Ks=1.0 according to Meyerhof (1976)
σ'_{v0}	:	Average effective overburden pressure along shaft
γ	:	Unit weight of soil taken as 19 kN/m^3
Ø	:	Internal friction angle of soil taken as 40^o
δ	:	Angle of internal friction between soil and concrete taken as 0.9Ø

Layers No.3 and 4

The unit skin friction and unit tip resistance values in Layer 4 at the pile tip are the two basic parameters that contribute to axial pile capacity. The correlation between these parameters and UCS is suggested by several authors. The values of unit skin friction resistance obtained using these relationships are summarized in Table 1.

Pile Sizes and Design Capacities

The pile cut-off level is at approximately 21m deep (-14.4m DMD) below the existing ground level considering an architectural design with six basement floors. The elevation of the existing ground level prior to excavation was approximately at +6.5m DMD.

Based on the required pile capacities, six different pile types were examined using the pile capacity calculation procedure explained above. Allowable capacities of the piles are estimated using a constant factor of safety of 2.5 both in skin friction and tip resistance reaching up to 33,000 kN per pile with a diameter of Ø1500mm having cut-off level of -14.4m DMD (Dubai Municipality Datum) and toe level of -62.0m DMD.

Table 1. Design Unit Skin Friction Values

Emprical Method	α	β	$fs=\alpha.\sigma_c^{\beta}$, MPa	
			Layer No. 3	Layer No. 4
Horvath and Kenney (1979)	0.21	0.50	0.19	0.27
Carter and Kulhawy (1988)	0.20	0.50	0.18	0.26
Williams (1980)	0.44	0.36	0.41	0.53
Rowe and Armitage (1984)	0.40	0.57	0.35	0.54
Rosenberg and Journeaux (1976)	0.34	0.51	0.30	0.45
Reynolds and Kaderbek (1980)	0.30	1.00	0.24	0.51
Gupton and Logan (1984)	0.20	1.00	0.16	0.34
Reese and O'Neil (1987)	0.15	1.00	0.12	0.26
Toh et al. (1989)	0.25	1.00	0.20	0.43
Meigh and Wolski (1979)	0.22	0.60	0.19	0.30
Horvath (1982)	0.20	0.50	0.18	0.26

Note: In the table, eleven different empirical equations are utilized. The range of values of fs were 0.12 to 0.41 MPa for layer No.2 and fs=0.26-0.54 MPa for layer No.4. The average values of 0.23 and 0.38 MPa are utilized in calculations.

The unit tip resistance is estimated using:

$q_{max} = 4.83\ \sigma_c^{0.51}$ (MPa) (Reese and O'Neill, 1987). The corresponding value is 6.33 MPa.

OSTERBERG CELL (O-CELL)

System

The O-cell is a hydraulically driven, high capacity, sacrificial loading device installed within the foundation unit. Working in two directions, upward against side-shear and downward against end bearing, the O-cell automatically separates the resistance parameters, i.e. the side resistance from end bearing.

The O-cell derives all reaction from the soil and/or rock system. Load testing with the O-cell continues until one of three things occurs: ultimate skin friction capacity is reached upwards, ultimate capacity downwards is reached, or the maximum O-cell capacity is reached. Each Osterberg Cell is specially instrumented to allow for direct measurement of the expansion so with compression and top of pile shaft measurements, the downward end bearing movement and the upward skin friction movement are known.

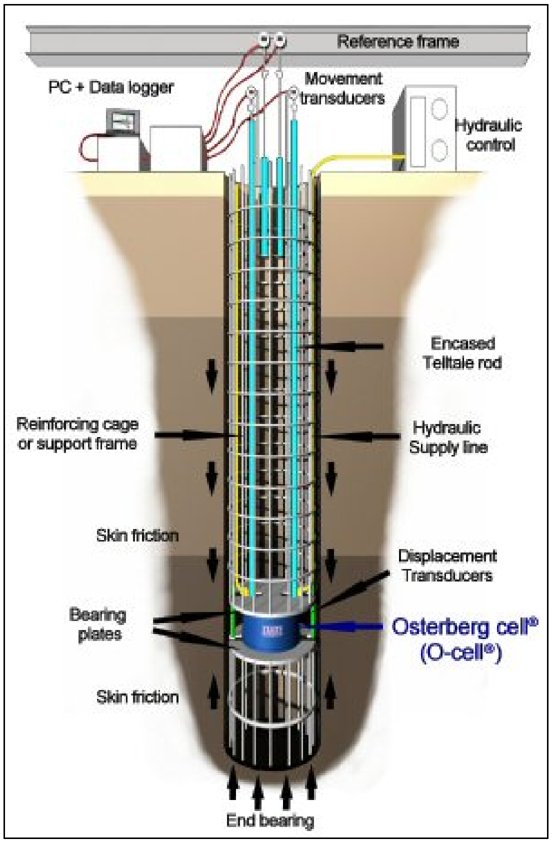

Figure 4: Typical Osterberg Load Cell Setup (www.loadtest.com)

O-cells range in capacities from 0.7 MN to 27 MN. By using multiple O-cells on a single horizontal plane, the available test capacity can be increased to more than 200 MN. The O-cell is attached to the reinforcing steel cage, and after the concrete reaches a minimum strength, the test may be started. Separation of the pile into two elements is induced by hydraulic pressure applied at the cell. Instrumentation is included in each test pile for direct measurement of O-cell expansion and shaft movements. The test pile often includes monitoring strain gauges embedded in the shaft. From the test, the load-upward deflection curve in side shear and the load-downward deflection in end bearing are obtained. (www.loadtest.com)

The load testing system that was used for testing the subject pile consisted of bi-directional expanding hydraulic loading cell (Osterberg cell), two rigid bearing plates, telltale rods, vibrating wire strain gauges, sonic pipes (to implement cross hole sonic logging). Figure 4 is a schematic figure of a typical Osterberg load cell setup and Figure 5 is a picture taken after the load cell was installed within the steel caging of the designated pile when the system is ready to be lowered down to the pre-drilled shaft.

Figure 5: Osterberg Load Cell Setup

Osterberg Cell

The main part of the testing system is the Osterberg cell. A photograph taken during the installation of the cell is given in Figure 6.

Figure 6: Osterberg Load Cell Installation

For this project, three bi-directional expanding Osterberg cell's were used each having a capacity of 900-tonnes. In case the loading is applied from or near the bottom of the pile, test using Osterberg cell automatically separates the end bearing and side shear components (John Burlands and John Mitchell, 1989). Consequently, the Osterberg cell effectiveness is doubled as compared to a system where the load is applied from the top of the pile. As a result, the effective testing capacity of the each cell is twice the nominal Osterberg cell capacity (i.e. 1800-tons per cell). The estimated side resistance may be considerably greater than the estimated ultimate end bearing. In that case, it may be advantageous to place the cell at some determined distance above the bottom of a drilled shaft so that the upward ultimate resistance of the portion of the shaft above the cell is approximately equal to the ultimate downward resistance of the portion of the shaft below the cell (Osterberg, 1995).

Bearing Plate

The function of the rigid steel bearing plate is to transfer the load from Osterberg cell to the pile. The thickness of the bearing plate is 50mm and the diameter is 1200mm. The bearing plates used for the test are shown in Figure 7.

Figure 7: Steel Rigid Bearing Plates

The bearing plates were welded on top and bottom of the Osterberg cell and also to the steel caging as shown in Figures 8 and 9.

Figure 8: Attaching Steel Plates to the Osterberg Cell


Figure 9: Attaching Steel Plates to the Steel Caging

Dial Gauges

Dial gauges are installed on extensometers and on top of the pile to measure pile head movement.

Vibrating Wire Strain Gauges

Vibrating wire gauges are installed within the pile body at eight different levels with 120^o separation to measure strains at nominated locations as shown in Figure 10.

Figure 10: Installation of Vibrating Wire Gauges

Sonic Pipes

The presence of any defect in a drilled shaft such as cracks, waists or voids, may cause a serious decrease of load capacity and increase of displacement (Lianyang Zhang, 2004). It is therefore important to conduct pile integrity test to detect defects in drilled shafts. Both pile integrity test and cross-hole sonic logging were performed to determine the integrity of the concrete body of the tested pile. For this purpose sonic pipes were installed during shaft construction along the pile body.

LOAD TESTING WITH OSTERBERG CELL

The Osterberg load test was carried out on a pile identified as P34 on October 22, 2006. The main objective of this load test was to proof-load the test pile to its maximum test load of 4,950 tons which is 1.5 times the safe working load of 3,300 tons estimated previously. For this purpose 3x900 tons capacity bi-directional hydraulic jacks were utilized. The test pile was a 1,500 mm diameter bored pile with a total embedded length of 47.6 m below the cut-off level. The Osterberg cell was installed 28.6 m below the cut-off level and vibrating wire gauges were installed at 1.6m, 7.6m, 13.6m, 17.6m, 23.6m, 25.6m, 33.6m, 38.6m below the pile cut-off level. The geomety of the testing apparatus is shown in Figure 11.

Load was applied to the hydraulic jacks using air-driven hydraulic pumps. This bi-directional loading system acts in two opposite directions, above the jack assembly which is resisted by side shear and below the jack assembly which is resisted both by end bearing and side shear.

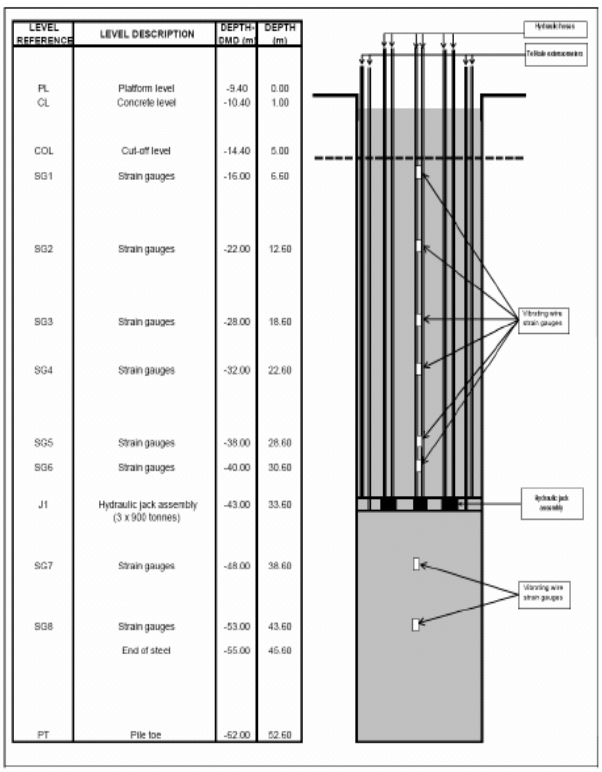

Figure 11: Test Pile and Testing System Geometry

Loading and unloading was carried-out in two cycles, one of which was loading to the 100 per cent design load and then unloading, second of which was loading to 150 per cent of design load and unloading. Each load step was kept for 10 minutes except the maximum load at Cycle 1 which was kept 60 minutes and the maximum load at Cycle 2 kept for cycle 120 minutes.

Interpretation of Strain Measurements and Test Results

The strain measurement was recorded for each load increment. Then the load at each gauge level is calculated using:

Load(P)=strain×(Area of Steel Reinforcement×E_s+Area of Concrete×E_c), where E_s is the modulus of steel and E_c is the modulus of concrete.

The difference between the loads at any two gauge levels represents the shaft load carried by that specific portion of the pile. Therefore the load transfer between each strain gauges is given by the formula:

$\Delta P=\Delta_{VSWG}$ x AE_P

where Δ_{VSWG} is the change in strain gauge readings and AE_P is the equivalent pile stiffness. In this formula, elastic modulus of the concrete was determined by the laboratory tests as 35 kN/mm^{2}and the elastic modulus of the reinforcing steel was given by the manufacturer as 210 kN/mm^2. The subject pile was reinforced with 22Ø32 bars for the top 12 meters and 22Ø25 bars for the following 12 meters. Dividing the load transferred by the circumferential area gives the average unit skin friction mobilized between the subject gauges. The formula can be generalized as:

Skin Fricion= |Lower Load-Upper Load|/Circumferential Area Between Two Gauge Levels.

Table 2: Strain and Load at Each Gauge Level

			(%100=33,000 kN)		(%150=49,500 kN)	
ID	Level - DMD m	Depth Below Pile Cut-Off m	Microstrain at 100% Load Increment	Loads at Respective Strain	Microstrain at 150% Load Increment	Loads at Respective Strain kN
Gauge 1	-16	1.6	1	71	0	12
Gauge 2	-22	7.6	2	131	2	119
Gauge 3	-28	13.6	6	414	10	637
Gauge 4	-32	17.6	16	1015	30	1954
Gauge 5	-38	23.6	88	5701	160	10424
Gauge 6	-40	25.6	184	11977	318	20621
Cell	-43	28.6		16500		24750
Gauge 7	-48	33.6	70	4413	141	8886
Gauge 8	-53	38.6	25	1601	50	3139

Table 3: Load Distribution and Mobilized Unit Skin Friction

	Effective Cell Load=33,000 kN		Effective Cell Load=49,500 kN	
Location	Load Between Levels kN	Mobilized Skin Friction kN/m2	Load Between Levels kN	Mobilized Skin Friction kN/m2
1 to pile top	71	3.0	12	0
2 to 1	60	2.0	107	4
3 to 2	283	10.0	518	18
4 to 3	601	32.0	1317	70
5 to 4	4686	166.0	8469	300
6 to 5	6276	666.0	10198	1082
Cell to 6	4523	320.0	4129	292
7 to Cell	12087	513.0	15864	673
8 to 7	2812	119.0	5746	244
End Bearing	1601	906.0	3139	1776

Using the above formulations, the load at each strain level is determined and is given in Table 2; the mobilized unit skin friction, and the distribution of the load carried by the shaft and the pile tip are given in Table 3.

Based on the results obtained from the load test, the equivalent top load vs. settlement chart is obtained and presented in Figure 12.

According the test results, the expected settlement of the pile under 100% design load (33,000 kN) is to be 10.2 mm and the expected settlement of the pile under 150% design load (49,500 kN) is to be 15.6 mm.

The mobilized unit skin friction values were compared with the unit skin friction values utilized in the design and the resulting chart is presented in Figure 13. It could be seen from Figure 13 and Table 3 that the larger portion of the load below the jack area along 10m length, is carried by the pile skin friction leaving the pile tip only a minor fraction of the total load applied (1601 kN out of 16500 kN applied in downward direction). This could be explained by the fact that "the ultimate side shear occurs at much smaller deformations than end bearing" and therefore in many situations at working loads the great majority of the load is taken by side shear (unless there exists a soft shaft bottom). In addition, at the loading level corresponding the 150% of the working load, no

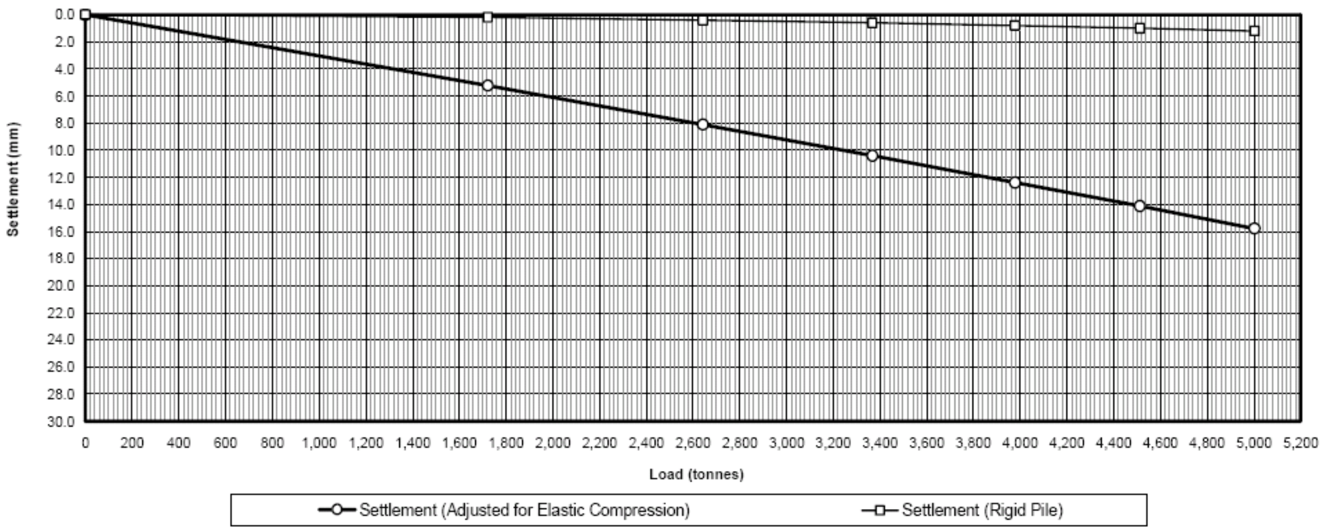

Figure 12: Equivalent Top Load vs. Settlement Chart

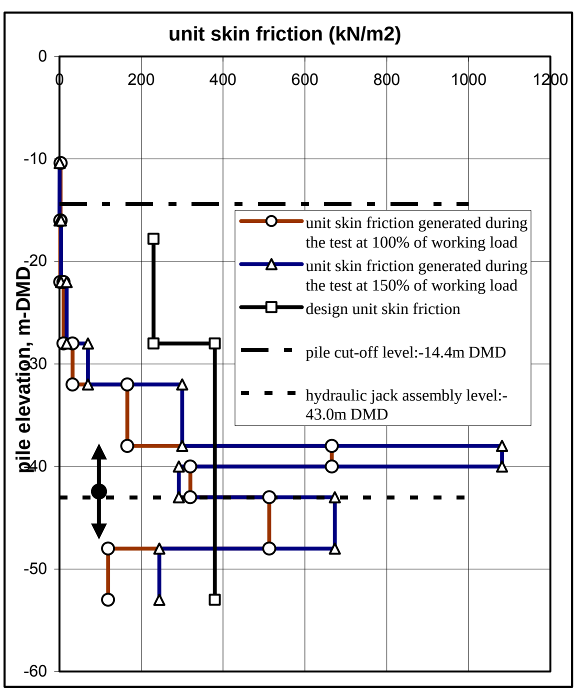

Figure 13: Design vs. Mobilized Skin Friction Values

significant skin friction was generated above the level of -32.0m DMD.

According to settlement values and the satisfactory cross hole sonic logging results, it was concluded that the tested pile can safely carry the design working load in compression with settlements within the allowable limits.

ACKNOWLEDGMENT

The Osterberg Cell load testing had been conducted within the scope of the quality plan for the construction of Princess Tower at plot No 392-192 at Marsa, Dubai, UAE. We would like to thank all the engineers involved on behalf of the Client TAMEER Holding and to Mr. J. K. Al Janabi on behalf of the Consultant ENG. ADNAN SAFFARINI for their valuable collaboration during the design and the application of this testing.

REFERENCES

Meyerhof, G. 1976. Bearing Capacity and Settlement of Pile Foundations, Journal of Geotechnical Engineering Division, ASCE, Vol 102, No. GT3.

Horvath. R. G., Kenney. T.C., 1979. Shaft Resistance of Rock Socketed Drilled Piers, Proceedings Symposium on Deep Foundations, ASCE National Convention, Atlanta.

Carter, J. P., Kulhawy, F.H., 1988, Analysis and Design of Drilled Shaft Foundations Socketed into Rock, Report EL-5918, Electric Power Research Institute, Paolo Alto, CA.

Williams, A. F., 1980, The Design and Performance of Piles Socketed into Weak Rock. Ph.D. dissertation, Monash University, Clayton, Victoria, Australia.

Rowe, R. K., and Armitage, H. H., 1984, The Design of Piles Socketed into Weak Rock. Faculty of Engineering Science, The University of Western Ontario, London, Ont. Research Report GEOT 11-84

Rosenberg, P., and Journeaux, N. L., 1976, Friction and Bearing Tests on Bedrock for High Capacity Socket Design, Can. Geotech. J., Ottowa, Canada, 13 (3), 324-33.

Reynolds, R. T., and Kaderbek, T. J., 1980, Miami Limestone Foundation Design and Construction, ASCE, New York, N.Y.

Gupton, C. and Logan, T., 1984, Design Guidelines for Drilled Shafts in Weak Rocks of South Florida, Proc. South Florida Annual ASCE Meeting, ASCE.

Reese, L. C., and O'Neill, M. W., 1987, Drilled Shafts: Construction Procedures and Design Methods, Design Manual, U.S. Department of Transportation, Federal Highway Administration, McLean, VA.

Toh, C. T., Ooi, T. A., Chiu, H. K., Chee, S. K. And Ting, W. N., 1989, Design Parameters for Bored Piles in a Weathered Sedimentary Formation, Proc. 12th ICSMFE, Rio De Janerio, 2, 1073-1078

Meigh, A.C. and Wolski, W., 1979, Design Parameters for Weak Rocks, Proc. 7th European Conf. On Soil Mech. and Found. Engrg., Brighton, British Geotechnical Society, 5, 57-77

Horvath, R. G., 1982, Drilled Piers Socketed into Weak Shale-Methods of Improving Performance, Ph.D. Dissertation, University of Toronto, Ontario, Canada

Lianyang Zhang , Drilled Shafts In Rock Analysis and Design, A.A. Balkema Publishers, 2004.

John Burlands and John Mitchell/Editors, Piling and Deep Foundations, A.A. Balkema Publishers, 1989

Osterberg, J.O., Osterberg Cell For Load Testing Drilled Shafts and Driven Piles, U.S. Department of Transportation, Federal Highway Administration, 1995