Load Testing and Bearing Capacity of Bored Piles in Limestone

Essai de Chargement et La Charge Limite Des Pieux Forés Ancrés Dans le Calcaire

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ABSTRACT
The objective of this paper is to critically compare the measured shaft resistance with computed values based on unconfined compressive strength of rock. For this purpose the results of load tests conducted for the foundation piles of high rise towers are studied. The main emphasis is given to the Federation Tower complex in Moscow consists of two tower of 62 and 93 floors. The 93 storey Tower, which soars 354 m will become the tallest building in Europe when completed in 2010. The high and concentrated foundation loads necessitated the utilization of pile foundation, which penetrates into limestone. The foundation piles were 1.50 m in diameter with varying lengths depending on the loads. The subsoil consists of 5 m thick medium strong dolomitic limestone, underlain by 5 to 7 m thick marl with thin limestone sub layers. Medium strong to weak limestone underlies these layers. Weak limestone was encountered at a depth of 22 m in which piles were socketed. Ground water table was located at 6 m below pile cut-off level. Core drilling and pressure grouting were performed underneath each pile for possible cavities in the limestone down to 6 m below pile tip level. Pile design loads vary within the range of 25 MN to 35 MN. Two preliminary pile load tests with Osterberg Cell method were carried out for two different pile socket lengths. The maximum applied bi-directional load was 33 MN corresponding to 66 MN top down load. The measured shaft resistance was compared with emprical methods based on the unconfined compressive strength of rock. Load test results were consistent with calculated shaft resistance values of Reese and O Neil (1989) and Rowe and Armitage (1984).

RÉSUMÉ
L'article rapporte les résultats d'un essai de chargement de pieu ancré dans le calcaire pour en déduire les valeurs de résistance latérale dans le rocher. Ces valeurs réelles ont été comparées avec les valeurs de résistance latérale calculées selon les différentes méthodes de calcul utilisant la résistance à la compression uni axiale du rocher pour évaluer leur approximation. Le complexe Tour de Fédérations comprend deux immeubles de grande hauteur de 62 et 93 étages. Complétée en 2010, la tour de 93 étages avec sa hauteur de 354m sera le plus haut immeuble de l'Europe. L'essai de chargement en question a été effectué lors des études des fondations en pieux de ce complexe. Des charges élevées de la structure concentrées sur la fondation nécessite pour leur support l'utilisation des pieux pénétrant dans le calcaire. La profondeur des pieux varie en fonction de ces charges. Les couches traversées par le pieu consistent de calcaire dolomitique de résistance moyenne, d'une épaisseur de 5m qui se trouve en dessus de la couche de marne d'épaisseur variant entre 5m et7m et incluant des nappes de calcaire fine. Une couche de calcaire de résistance faible se trouve à une profondeur de 22 m dans laquelle les pieux sont ancrés. Carottage et injection à pression ont été effectués jusqu'à une profondeur de 6m en dessous de la pointe de chaque pieu pour colmater les cavités possibles. Les charges de fluage des pieux varient entre 25MN et 35MN. Deux essais de chargement ont été effectués utilisant la cellule d'Osterberg pour deux profondeurs d'ancrages de pieu différentes. Les résistances latérales mesurées du fut dans le rocher sont comparés avec celles calculées selon les méthodes utilisant la résistance uni axiale du rocher. Les essais de chargement confirment les résistances calculées selon la méthode proposée par Reese et O Neil (1989), et Rowe et Armitage (1984).

Keywords : O-cell, rock socket, bored pile, shaft resistance, pile load test

1 INTRODUCTION

Moscow International Business Center (MIBC) is situated four kilometers to the west of the Kremlin and is one of the largest and challenging construction and investment projects. It is significantly different from other developments because of its mixed-use, integrating office buildings, hotels and multifunctional retail complexes. The Project area consists of 20 plots and covers an area of about 60 hectares. The construction of MIBC is on the Krasnopresnenskaya embankment on the left bank of the Moscow River.

The goal of the project is to create a new commercial center for the capital including some 2.5 million m^2 of office, shopping and residential space, hotels, parking for 30,000 cars, a huge public square, sports and leisure facilities, and the new 230,000m^2 City Hall.

Figure 1. Major completed and on-going projects in the MIBC

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Major completed and on-going projects in the MIBC undertaken by Kasktaş are shown on the map in Figure 1 and the details of the applications are given in Table 1.

Table 1. Details of Projects and Type of Applications at MIBC

Plot	Description of the Project	Type of Application
4	Imperia Tower and Aquapark (60 floors, 239m)	Foundation Piles (Dia.: 1.2 m)
9	City of Capitals (Moscow Tower:73 floors, 274m; St.Petersburg Tower: 62 floors, 235m)	Foundation Piles (Dia.: 1.2 m)
10	Naberezhnaya Tower (Tower A:17 floors, Tower B:27 floors, Tower C:60 floors 260 m)	Foundation Piles (Dia.: 1.2 m)
11	Transport Terminal (Three Towers; 42, 33 & 21 floors)	Foundation Piles (Dia.: 1.2 m); Anchored Secant Piled Wall
12	Eurasia Tower (75 floors; 300 m)	Foundation Piles (Dia.: 1.5 m); Anchored Diaphragm Wall
13	Federation Towers (Tower A:93 floors, 354m; Tower B:62 floors, 242 m)	Foundation Piles (Dia.: 1.5 m)
16	Office & Hotel Complex (78 floors & 80 floors)	Anchored Diaphragm Wall

The foundation engineering works at MIBC were commenced in 2003 and has been under construction as of December, 2008. A total of ~45,000 m of bored piles were installed as foundation piles under the foundations of the Towers and different structures; 28,000 m^2 of diaphragm wall with trench cutter system; ~12,600 m of secant piled wall and ~36,000 m of ground anchors were executed as shoring system.

The unconfined compressive strength of limestone (q_u) and the details of preliminary pile load tests performed with O-cell method are summarized in Table 2.

The centerpiece of Moscow City will be the 93 and 62 storey Federation Towers, which will be Europe's tallest building at a height of 354 m.

This paper mainly presents the design, construction and testing of large diameter bored piles constructed for the foundations of Federation Towers in Moscow, Russian Federation.

2 SUBSOIL AND GEOLOGICAL CONDITIONS

The construction area extends over the left bank of the Moscow River and is formed by combination of flood plain and above-flood-plain terraces. The territory of site is a part of old stone quarries in which in the XVIIth – XIXth centuries limestone was extracted. In the XXth century after induced rise of the water level in the Moscow River, these stone quarries were backfilled.

Extensive soil investigation programme was carried out to investigate the geological and hydrological conditions of the site, to define physical, mechanical properties, corrosion characteristics and to assess karst suffusion danger of the foundation soil. Approximately, 200 boreholes were drilled to a depth of 10 to 100 meters in the construction area of MIBC for various purposes.

The whole area of construction of MIBC is referred to as potentially hazardous territory of the karst-suffusion processes. The conducted soil investigations make it possible to verify that the formation is safe in terms of karst-suffusion phenomena. 25 boreholes from 35 to 100 m in depth (Total 1,700m) have been drilled at the site.

Table 2. Summary of preliminary pile load tests

Plot No	4	9	10	11	13
Test No (no.)	2	2	2	2	2
Test Pile Diameter (m)	1.2	0.9	1.2	1.2	1.2
Pile Length (m)	28	19	24	22	29
Max. Test Load (MN)	26.5	21.4	29.7	20.1	33.3
q_u Limestone (MPa)	22	52	61	8.1	13.4

The subsoil below pile cut-off level consists of 5 m thick medium strong and weak Ratmirovsky series dolomitic limestone (Layer No : 10 and 11 respectively), underlain by 5 to 7 m thick Voskrensky series marl with thin limestone sub layers. Medium strong to weak Suvarovsky series limestone (Layer No : 13 and 14 respectively) underlies these layers. Weak Moscovian stage limestone (Layer No : 15) was encountered at a depth of 22 m in which piles were socketed. Medium strength limestone (Layer No : 16) underlies this layer. Groundwater table was located at 6 m below pile cut-off level. Strength and deformation properties of subsoil are summarized in Table 3 and the geological section of soil and rock layers is shown in Figure 2.

3 FOUNDATION DESIGN

The footprints of the Federation Towers (Plot-13) are triangular, covering an area of 3,600 m^2 and 2,700 m^2 for the 93 and 62 storey towers respectively. The foundation level was 22 m below ground surface. A reinforced concrete perimeter diaphragm wall with tie-backs was chosen for the support of the excavation.

The loads are high and concentrated, which necessitated supporting the Towers on deep foundations, penetrating into the limestone. The foundation piles were selected as 1.50 m in diameter with varying lengths depending on the loads

The soil resistance and modulus characteristics provided by a local geotechnical consultant were used in the initial foundation design. Most of the high-rise buildings in Moscow are supported on massive reinforced concrete structural mats that are placed on grade which generally rests on the upper Perhurosky limestone formation.

Table 3. Strength and deformation properties of subsoil

No	Description	γ	e	c'	q_u	E	ϕ'
		kN/m³	%	kPa	MPa	MPa	°
10	Dolomitic Limestone	23.2			38.9	8730	39.6
11	Dolomitic Limestone	21.3		3600	8.01	1940	31.5
12	Voskrensky Clay	22.3	0.54	4		89	24.6
13	Suvorovsky Limestone	24.4	0.18		20.6		
14	Suvorovsky Limestone Clay Interbed	23.4	0.31		8.3		
15	Moscovian Limestone, Dolomite, Clay interbed	21.6	0.37		13.4		
16	Medium Strength Limestone	25	0.23		24		

In accordance with Code of Regulations on Designing and Construction of Pile Foundations of the Russian Federation (SP 50-102-2003), the load bearing capacity F_d , of the bored piles resting on the bedrock shall be determined as follows:

$$F_d = \gamma_c \cdot R \cdot A \qquad (1)$$

Where; F_d is load bearing capacity (kN); γ_c is the pile working conditions factor in the soil to be taken as equal to 1; R is the design resistance of soil under the tip of the end bearing pile (kPa) and A is the area of the pile resting on the soil (m^2).

For bored piles embedded into the unweathered rock (without weak interlayers) for at least by 0.50 m, the design resistance R, under the tip of the end bearing pile shall be determined as follows :

$$R = \frac{R_{c,n}}{\gamma_g}\left(\frac{l_d}{d_f} + 1.5\right) \qquad (2)$$

Where; $R_{c,n}$ is mean value of q_u of rock in the saturated state (kPa); γ_g is the reliability factor according to the soil, to be accepted as equal to 1.4; l_d is design depth of rock socket of the bored piles (m); and d_f is external diameter of the bored pile, embedded into the rock (m).

Rock socketed piles designed in accordance with the above given equations, carry the load by both shaft resistance and base resistance. At the time of the design, there was no documented experience in the Moscow area with high capacity large diameter piled foundation socketed in the limestone. Therefore, the design of piles was such that the length of rock socket to be capable of carrying pile load on side friction in limestone. 1.50 m diameter piles are designed to resist 20 MN to 35 MN design load.

4 TEST PROGRAM

Pile design loads varied within the range of 25 MN to 35 MN. Since no test data for heavily loaded large diameter piles socketed in the limestone were available at the time of design, it was necessary to perform static loading test to evaluate the pile capacity and deformation behavior. However, with the very high design loads a conventional static loading test would have been costly and also it was difficult to provide such a kentledge. Hence, the Osterberg-cell (O-cell) test method was selected to perform static bi-directional load tests.

Two preliminary pile load tests with O-cell method were carried out for different pile socket lengths. To minimize the magnitude of the required test load, test piles with a diameter of 1.20 m were constructed. These test piles were constructed under water with tip elevations of 80.63 m and 83.30. Construction of the piles commenced by open boring through an upper limestone layer followed by rock auger within the underlying cohesive soil stratum until ground water was encountered. Pile excavation was completed using a rock bucket within the remaining soil stratum and underlying limestone bedrock to final pile tip elevation. After cleaning the base by air lift, the reinforcing cage with attached O-cell assembly was inserted into the excavation and spliced with the upper rebar cage section. Each pile was fitted with two 540 mm diameter O-cells at one level. The upper part of the socketed length of the test piles were filled with sand to eliminate shaft friction.

Embedded tell tales were installed at three sections of each pile. The O-cell assembly were located 3.10 m and 6.0 m above the tip of pile for Tower A and Tower B. For Tower A, test pile tip level and concrete top level was 80.63 and 87.53 whereas test pile tip level was 83.30 and concrete top level was 96.65, for Tower B respectively. O-cells were installed at elevations of 83.73 and 89.30 for Tower A and Tower B respectively (Figure 2). The general sub-surface stratigraphy at the location of the test piles is consisted of marly clay from the Voskrensky series overlying limestone from the Suvorovsky Series. The test piles and generalized subsurface profile are given in Figure 2. The maximum applied bi-directional load was 33 MN corresponding to 66 MN top down load.

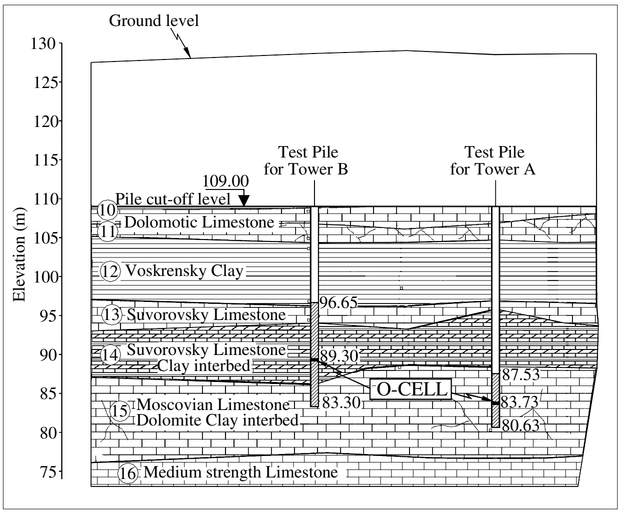

Figure 2. Preliminary Test Piles for Tower A and Tower B

5 TEST RESULTS

5.1 *Combined End Bearing and Lower Side Shear (Tower A)*

The maximum downward applied load was 33.33 MN. At this loading, the average downward movement of the O-cell base was 21.15 mm. The side shear capacity of the 3.10 m pile section below the O-cell is calculated to be 26.96 MN assuming a unit side shear value of 2,307 kPa and a nominal pile diameter of 1.20 m. The maximum applied load to end bearing is then 6.37 MN and the unit end bearing at the base of the pile is calculated to be 5,631 kPa at the above noted displacement.

5.2 *Upper Side Shear (Tower A)*

The maximum upward applied net load was 33.05 MN. At this loading, the upward movement of the O-cell top was 43.32 mm. Assuming a nominal pile diameter of 1.20 m, the average unit side shear capacity of the 3.80 m pile section above the O-cells is calculated to be 2,307 kPa.

5.3 *Combined End Bearing and Lower Side Shear (Tower B)*

The maximum maintained downward applied load was 31.90 MN. At this loading, the average downward movement of the O-cell base was 3.78 mm. The side shear capacity of the 6.00 m pile section below the O-cell is calculated to be 25.81MN assuming a unit side shear value of 1,141 kPa and a nominal pile diameter of 1.20 m. The maximum applied load to end bearing is then 6.09 MN and the unit end bearing at the base of the pile is calculated to be 5,389 kPa at the above noted displacement.

5.4 *Upper Side Shear (Tower B)*

The maximum maintained upward applied net load was 31.61 MN. At this loading, the upward movement of the O-cell top was 5.83 mm. Assuming a nominal pile diameter of 1.20 m, the average unit side shear capacity of the 7.35 m pile section above the O-cells is calculated to be 1,141 kPa. These results are plotted in the diagrams shown in Figures 3 and 4, presenting the recorded load-movement data for the top and bottom O-cell plates for both the Tower A and Tower B.

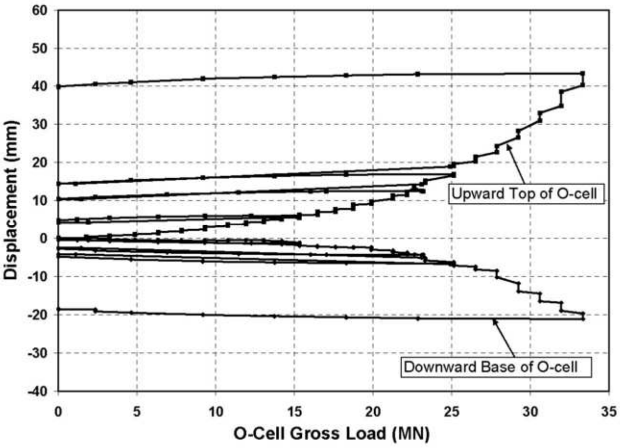

Figure 3. Load Movement Curve for Tower A

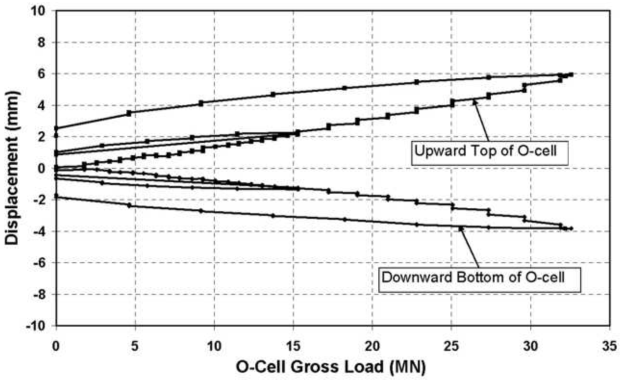

Figure 4. Load Movement Curve for Tower B

6 CONSTRUCTION AND QUALITY CONTROL

A total of 314 no of 1.50 m diameter and 22 to 29 m deep cast in situ bored piles were implemented for the foundations of Tower A and Tower B. Skin and base grouting; and core drilling were employed for each pile upon completion of concreting. A comprehensive quality control and verification testing program were incorporated in the project.

Before pile installation works, probing was performed to detect the possible cavities in the limestone. The depositional process of the limestone strata is such that there is also risk of karst. For the purpose of investigating of the potential presence of karst and clay lenses, rock coring was performed with double core barrel 6 m below the toe of each pile.

Base grouting was implemented under each pile for eliminating the potential risk of possible cavities in limestone down to 6 m below the pile tip level. The water / cement ratio by volume varied to meet the characteristics of each hole and was normally within the range between 3 / 1 and 0.6 / 1. The grout pressure was maintained around 10 MPa. A double packer system was used at top and bottom of the rock zone to be grouted. The unconfined compressive strength of the grout samples were range between 14 MPa and 20 MPa. The grout mixture was designed to expand 2-4 % .

Coring for pile concrete-base contact was performed by a double core barrel through a 150 mm internal diameter duct from just above the pile toe level to within the limestone to demonstrate a clean base. The base was considered clean where clean concrete to limestone contact was observed.

Verification testing of concrete quality was performed on all piles by Cross-hole Sonic Integrity Test (CSL). For conducting CSL tests four no's of steel tubes were attached to the inside of the pile reinforcing steel cage. This arrangement allowed four profiles around the perimeter of the pile and two diagonal traces to be obtained.

7 EVALUATION OF TEST RESULTS AND CONCLUSION

The measured shaft resistance was compared with shaft resistances computed by emprical methods based on the unconfined compressive strength of rock. For this purpose a total of 10 O-cell tests performed at MIBC were studied, however; neither the ultimate shear resistance nor the ultimate end bearing had been reached at the maximum capacity of O-cell for the 9 piles tested. Therefore, only the result of Tower A test pile was considered in this study.

For Tower A the maximum upward applied net load was 33.05 MN. At this loading, the upward movement of the O-cell top was 43.32 mm (Figure 3). The failure of the pile socket was clearly observed. The observed creep limit for upper part of this pile was 29 MN.

Table 4 summarizes the results of ultimate shaft resistances calculated with several methods as proposed by various researchers. Unconfined compressive strength of limestone is considered as 13.38 MPa in calculating shaft resistance. The ultimate unit side shear capacity of the 3.80 m pile section above the O-cells is measured to be 2,307 kPa in Moscovian stage limestone. The calculated shaft resistance values are compared with the actual values measured during the load tests.

The results of O-cell tests showed that the shaft resistances were under predicted by methods 1, 2, 3 and 5. However, methods 6, 7, and 9 over predicted shaft resistances. Load test results are consistent with calculated shaft resistance values by Reese and O Neil (1989) and Rowe and Armitage (1984) methods. The calculated skin friction values with methods 4 and 8 are consistent with the actual results of load test performed on bored piles socketed in to Moscovian stage Limestones (Layer No:15).

The load bearing capacity of 1.20 m diameter rock socketed pile calculated as 44.2 MN for 3.10 m socket length in layer no. 15 by equations (1) and (2) in accordance with SP 50-102-2003 is in good conformity with the downward movement trend of O-cell test result obtained from Tower A as shown in Figure 3.

Table 4. Calculated shaft resistances for Tower A

DESIGN METHOD (Seidel & Collingwood, 2001)	α	β	Ultimate Shaft Resistance $F_{su} = \alpha\, q_u^{\beta}$ (MPa)
1. Horvath & Kenney (1979)	0.21	0.50	0.77
2. Carter & Kulhawy (1988)	0.20	0.50	0.73
3. Williams et al. (1980)	0.44	0.36	1.12
4. Rowe & Armitage (1984)	0.40	0.57	1.76
5. Rosenberg & Journeux (1976)	0.34	0.51	1.28
6.Reynolds & Kaderbeck (1980)	0.30	1.00	4.02
7. Gupton & Logan (1984)	0.20	1.00	2.68
8. Reese & O'Neill (1988)	0.15	1.00	2.01
9. Toh et al. (1989)	0.25	1.00	3.35

REFERENCES

Load Test Int. Inc. 2005, Report on drilled pile load testing (Osterberg Method). Unpublished Report no LTI 2299-1 and LTI 2299-2

Seidel J.P.,and Collingwood,B, 2001 A new socket roughness factor for prediction of rock socket shaft resistance, Canadian Jeotechnical Journal 38: 138-153

SP 50-102-2003; Russian Federation State Committee on Construction and Housing Utility Complex (Gosstroy of Russia), 2004, Code of Regulations on Designing and Construction of the Pile Foundations, Moscow