Observed Settlements of a Piled Raft

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ABSTRACT
This paper presents a case study for the performance of a piled raft for a high-rise office complex located in Moscow. The piling system is unique due to both been a part of the interactive piled raft and the columns of the top-down construction – which enables safe and feasible erection of deep basement levels in a heavily urbanized area. The construction methodology and the design approach are summarized while the settlement of the piled-raft is discussed with graphical presentations of predicted and measured displacement values. It is worth mentioning that the direction of construction activity for the three-part complex considerably influenced the behavior of the piled raft.

RÉSUMÉ
L'article décrit la performance de la fondation en radier renforcé par pieux d'un complexe de tours d'office à sous-sols à Moscou. Les étages en-sous sols ont été construits par la méthode dite du haut en bas; méthode particulièrement efficace pour la construction des sous-sols profonds en site extrêmement urbanisé. La particularité du système de fondation consiste en ce que les pieux supportent aussi les planchers des sous-sols assurant la stabilité des diaphragmes de soutènement durant la construction des étages en sous-sols. La méthode de construction des sous-sols selon cette méthode a été décrite. La méthode de calcul du radier renforcé par pieux est décrite, les tassements calculés par cette méthode sont comparés avec les tassements mesurés. L'article attire l'attention sur l'influence de la phase de construction des trois tours sur le comportement du système de fondation.

1 INTRODUCTION

An interactive piled raft design has been implemented for the deep foundation system of a high-rise office complex, where piles are utilized to reduce the settlement of the raft rather than solely carrying the entire structural load – thereby sharing the load bearing function with the raft. Due to the heavily urbanized location of the complex, the top-down construction methodology has been selected.

This paper presents the recorded performance of the piled raft for the time period starting with the piling works in December 2006 and ending with the latest displacement measurement of the raft dated May 2008. As of the writing date of this paper, the complex has been already completed and is serving as one of the prestigious business centers of Moscow.

2 DESCRIPTION OF THE PROJECT

Located in central Moscow, this Class A office complex is comprised of three independent buildings sharing a three level basement. Two of these blocks – Building A & B – have sixteen floors rising up approximately sixty five meters from the zero level. The other block – Building C – is seven stories high rising up to thirty meters. The basement footprint is 9,704 square meters, while the total development area is 108,548 square meters. The site overview has been provided below in Figure 1, and the completion dates of piling works and slabs are given in next page in Figure 2.

Figure 1. Site overview of the complex.

3 SOIL CONDITIONS

The mean natural ground elevation is 157.48. The subsoil consists of 1.1 to 4.3 meters thick fill with sandy clay and debris, 1.3 to 3.6 meters thick water-bearing glacial sand with sandy clay, 2.5 to 6.6 meters thick morainic deposits of hard sandy clay (bottom elevation 148.48~149.75), 6.5 to 8.8 meters thick water-bearing glacial sand with gravel (bottom elevation 140.64~142.61), 6.1 to 10.8 meters thick Jurassic deposits of Tithonian sandy clay (bottom elevation 134.91~139.30), and 7.9 to 13.3 meters thick Oxfordian hard clay (bottom elevation 126.61~127.65). Low strength limestone and marl underlies these layers. The groundwater elevation varies between 150.05 and 151.95

and its mean elevation is 151. The engineering properties of subsoil underneath the piled raft are tabulated below:

Table 1. The engineering properties of subsoil.

Soil Type	Elastic Modulus E, MPa	Cohesion c, kPa	Friction angle φ, deg	Bulk weight γ, kN/m^3
Glacial sand	30	0	38	19.7
Sandy clay	18-24	70	21	19.4
Hard clay	45	91	17	17.5
Limestone	250	300	26	21.9

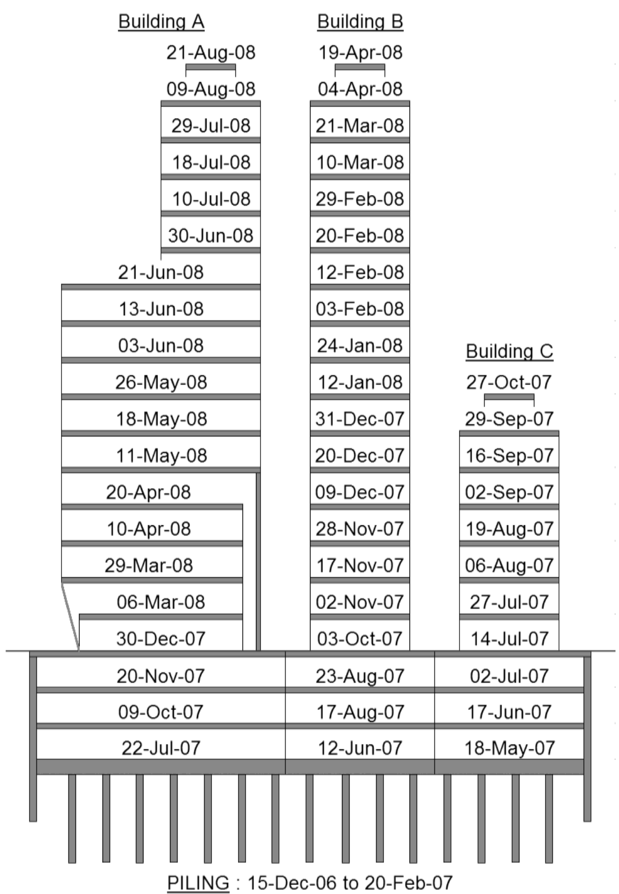

Figure 2. Cross-section of the complex.

4 PILED RAFT

A worldwide popular engineering application for more than two decades now, piled rafts enable cost effective solutions for foundation engineering without compromising the long-term performance and safety of the foundation. (Poulos, 2001)

Due to the fact that a raft alone did not satisfy the design requirements, the foundation system of the complex has been selected to be a piled raft: Structural loads are resisted both by the ground underneath the raft and by a special configuration of piles which limits the overall settlement of the building and also optimizes the raft thickness by reducing the bending forces acting on it.

4.1 Dimensions

The piled raft comprises of a reinforced concrete raft with a varying thickness from 1.0 to 2.5 meters acting together with 17 meters long, 1.20 meter diameter cast-in-place reinforced concrete piles, typically bearing at elevation 130 (in meters). The top of raft elevation is 147.60. At its perimeter, the piled raft is confined by a 0.8 meters thick reinforced concrete slurry wall. The "Koltsovaya" ring line of Moscow metropolitan runs underneath one corner of the raft.

A sectional view of the piled raft together with the soil profile and the location of the metro line in relation to the building are illustrated in Figure 3.

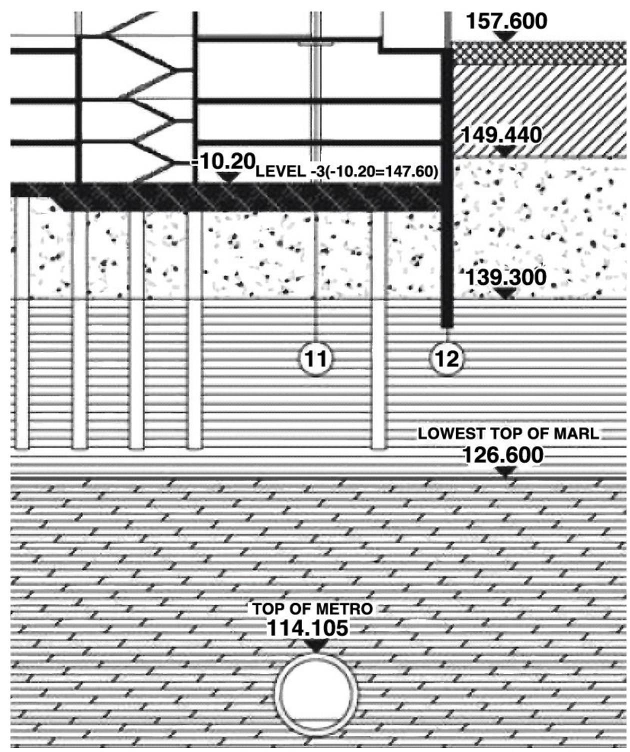

Figure 3. Piled raft foundation and subsoil profile, all elevations in meters.

4.2 Design

This piled raft is rather unique compared to the local piling practice, where the piles are generally designed to resist whole structural load with a certain factor of safety as either rock socketed piles or friction piles – even though local regulations (item 7.4.10 of SP 50-102-2003) allow the implementation of piled raft design.

The design procedure of the piled raft is as follows: Spring stiffness coefficients for different portions of the foundation – raft, piled raft, pile, slurry wall – are estimated with respect to the settlement of all underlying soil layers under the influence of the raft and the settlement of the Oxfordian hard clay under the influence of piles, where the total settlement is limited by the serviceability of the superstructure. Together with the various structural load combinations, these coefficients are inputted into finite element structural analysis software in order to investigate the soil pressure and the settlement under the raft. One can refer to the paper by Kulhawy & Prakaso (2001) for a detailed discussion of the displacement-based design methodology of piled rafts.

The result of the finite element analysis is as follows: The total structural load is calculated to be 2250 MN. With a varying load from 3500 kN to 6800 kN on each, 45% of the overall load, that is approximately 1000 MN is resisted by piles. While 315 MN is acting on the slurry wall, the remaining load of 935 MN (42%) is supported by the ground underneath the raft. The maximum settlement of the entire building is anticipated within the region of 35 millimeters. The settlement estimation is depicted in Figure 4.

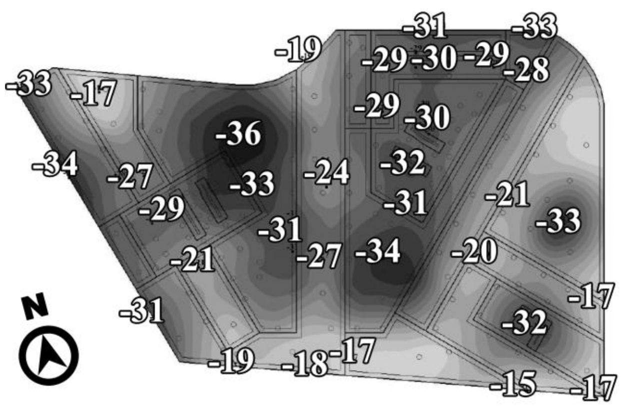

Figure 4. Predicted settlements, in millimeters.

5 PILE TESTS

Two preliminary axial static pile load tests have been conducted to evaluate the capacity and deformation behavior of piles. Two 1.2 meter diameter test piles have been installed beneath Block A and B with the elevation of drilling platform at 158.60 and pile tips resting at 128.10 and 130.00, respectively. The pile boring diameter from elevation 158.60 down to the bottom of raft is 1.3 meters, with 1.2 meter diameter permanent steel casing being installed in this part. The annulus between the permanent casing and boring was filled with sand-bentonite mixture to eliminate shaft friction between the platform level and the bottom level of the raft. Actual working lengths of test piles are 17.75 and 15.85 meters. The axial test loads are 9.0 MN and 9.6 MN and corresponding maximum recorded displacements are 40.1 mm and 46.6 mm, for Blocks A and B respectively. Test results are plotted below in Figure 5, where triangular data points are for the test under Block A, and squares are for Block B.

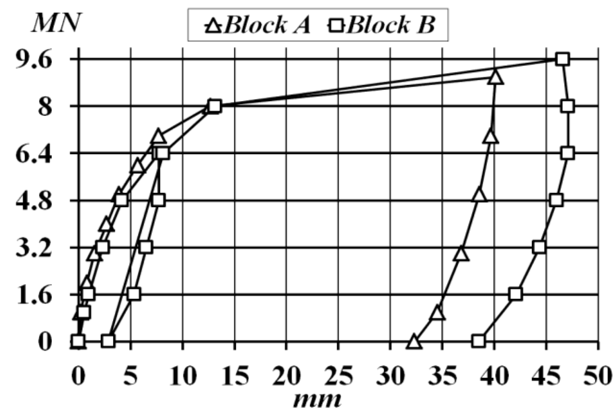

Figure 5. Load-displacement curves of test piles.

According to item 3.10 of SNiP 2.02.03-85, a single pile shall be designed with respect to the load bearing capacity of base according to the following condition:

$$N \leq F_d / \gamma_k \quad [1]$$

where N is the design load transferred to the pile; F_d – the design load carrying capacity of base soil of a single pile which is defined as the pile load carrying capacity; γ_k – reliability coefficient which is taken equal to 1.2 when the pile carrying capacity is determined by the results of static load tests in field.

The pile load carrying capacity F_d, is calculated by the following formula found in item 5.3 of SNiP 2.02.03-85:

$$F_d = \gamma_c \,.\, F_{u,n} / \gamma_g \quad [2]$$

where γ_c is the working condition coefficient which equals 1.0 for compressive loads; $F_{u,n}$ – the normative value of pile limit resistance; γ_g – soil reliability coefficient which can be taken equal to 1.0 for this case.

In building and structure foundations, for the particular value of pile limit resistance F_u for compressive loading, the load under which the tested pile shall have settled equal to s, shall be adopted. s is calculated with the following equation:

$$s = \varsigma \,.\, s_{u,mt} \quad [3]$$

where $s_{u,mt}$ is the limit value of average foundation settlement which is established according to the instructions of SNiP 2.02.01-83; ς is the coefficient of transition from the limit value of average settlement of the foundation of buildings or structures to the pile settlement obtained at the static test with conventional attenuation of settlement.

The values of coefficient ς may be refined on the basis of observation of settlement of the buildings erected on pile foundations in similar soil conditions.

According to item 5.5 SNiP 2.02.03-85, for cases when the increase in settlement during one stage of loading (with total value of settlement being more than the 40mm) which is 5 times and more than the increase in settlement obtained at the previous stage of loading or pile settlement does not stabilize without increasing the load; a load which is one degree less than that which caused above mentioned consequences, is taken as the particular value of pile limit resistance, F_u.

If at the maximum test load, which is equals to or greater than 1.5 times the pile load bearing capacity found from calculation, the pile settlement is less than 40mm, then the maximum test load can be taken as the particular value of pile limit resistance F_u.

For the testing case at hand, the limit resistance, F_u, as derived from load-displacement curves in Figure 5, is 8000 kN. Therefore the pile load carrying capacity is calculated using Equation 2 as:

$$F_d = 1 \cdot 8000 / 1 = 8000 \text{ kN} \quad [4]$$

And the allowable design load of piles is calculated using Equation 1 as:

$$N = 8000 / 1.2 = 6667 \text{ kN} \quad [5]$$

It is concluded that working load of piles can be taken as 6667 kN, which is utilized by the designer during the calculation of spring stiffness coefficient of piles for the analysis of piled raft.

6 PILING AND TOP-DOWN CONSTRUCTION

As a result of the water-bearing upper soil stratum, 0.8 meter thick reinforced concrete slurry wall down to a depth of 19 meters has been executed prior to the construction of basement floors to ensure safe and dry conditions.

To minimize the disturbance to the heavily urbanized setting of the site location, the top-down construction methodology has been chosen. This method is based on the introduction of an internal perimeter ring slab, initially cast on soil and subsequently supported by piles in which top-down columns made of I-beam steel profile are installed during piling works.

The ring slab supports the perimeter slurry wall and resists the lateral soil thrust. The advantage of this method is the unrestricted construction of central part of the basement down to the final excavation depth.

1.2 meter diameter cast-in-place bored piles have been drilled from the first basement floor level (153.30) with an average depth of 24 meters. The total number of piles is 189 (with 86 of them having I-beam steel profile installed as shown in Figure 6) and the total drilling length is 4,506 meters.

Figure 6. Pile with Steel Beam Profile

After piling, the erection of the structure has started with the -1 floor slab, which has been partially poured as a ring adjacent to the slurry wall, thus allowing gaps in center of construction area to allow excavated material out and construction materials in.

With the installation of steel struts at some critical locations (see Figure 7 in next page), the lateral load is transferred between each side of the ring and from the wall to the ring.

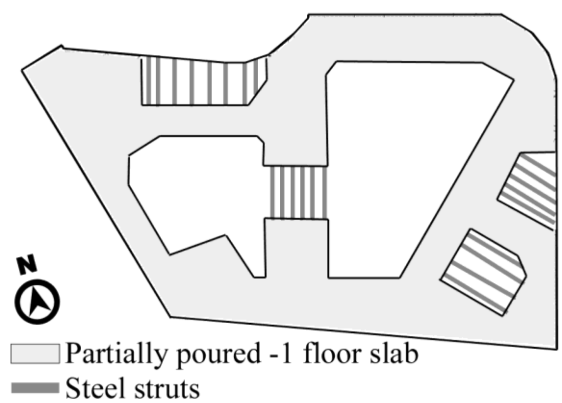

Figure 7. Ring slab.

The excavation of underneath the -1 floor slab for the construction of foundation has been executed afterwards. After pouring the foundation, the core and structural columns are casted and they are then followed by the construction of -2 floor slab, the completion of -1 floor slab, and zero level floor slab and the above ground level floor slabs, respectively. A photograph which is taken during the excavation under the ring slab is presented in Figure 8.

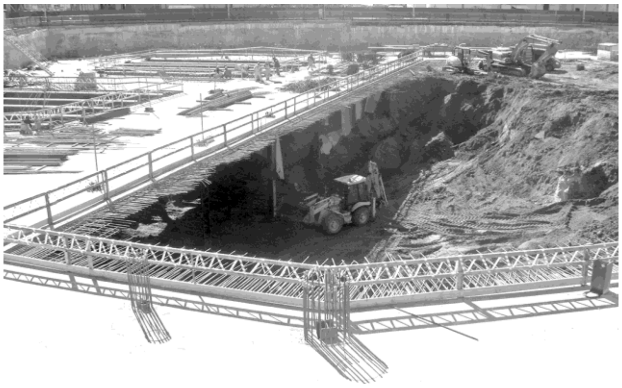

Figure 8. Excavation under the ring slab.

Figure 9 depicts another photograph, which is taken from beneath the ring slab under Block A. In this photograph, the I-beam steel profiles carrying the ring slab can be spotted on the middle right side in the foreground, while the on-going construction of upper floor slabs of Block C is on the further to the left in the background.

7 MONITORING OF PILED RAFT

Thirty displacement survey points under the raft foundation and additional sixteen displacement survey points under the super structural columns at zero level floor, has been employed for the verification of the load-settlement behavior of the piled raft. In Figure 10, survey points are shown, where points from 1 to 30 are of the raft foundation and points from 31 to 46 are of columns.

Figure 9. View from under beneath the ring slab.

The plot of all measurements is graphed in Figure 11 for the time period starting with the installation of monitoring equipment in August 2007 and ending with the latest displacement measurement of the raft dated May 2008. The data is separated into three groups representing the performance of the raft under each independent building.

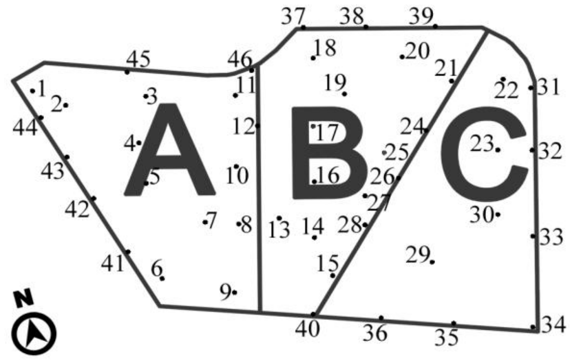

Figure 10. Survey points.

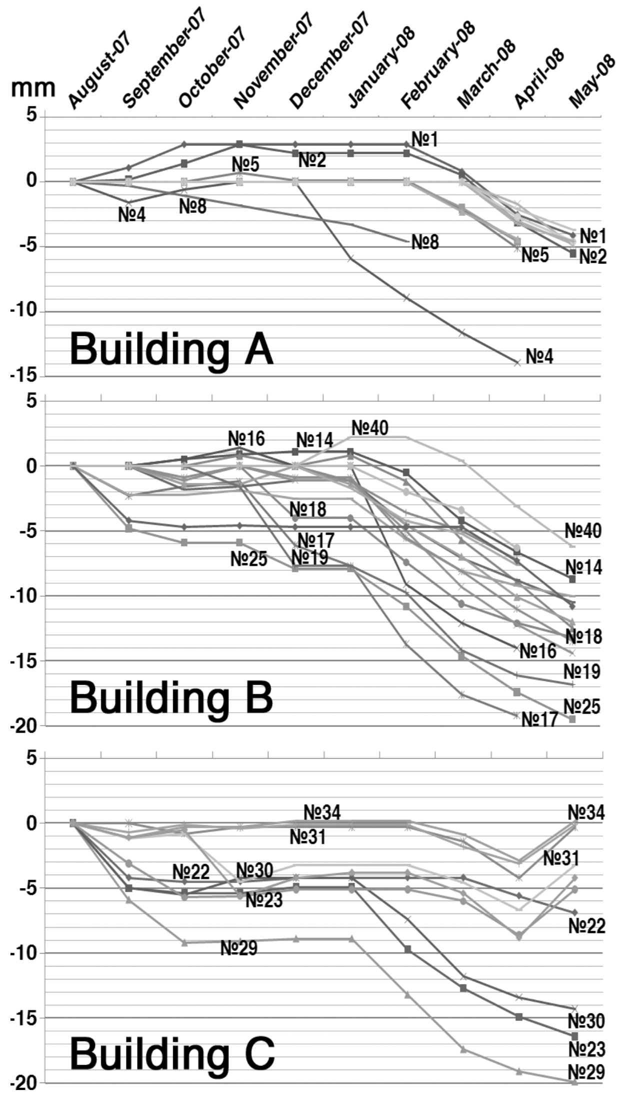

Figure 11. Settlement survey of buildings.

In reference to Figure 1, it can be observed that concreting of all slabs and the construction of upper level blocks has always been started from east side and finished at the west side for every new level of the complex. Displacement measurements on the raft during the building activity, clearly reflects this situation, where the points on the west is slightly lifted temporarily up to an utmost value of +3.0 mm –afterwards settled down to -4.0~-5.0 mm– and maximum downwards settlements of -20 mm being occurred on the east side of the complex. Final displacement measurements as of May 2008 are shown below in Figure 12.

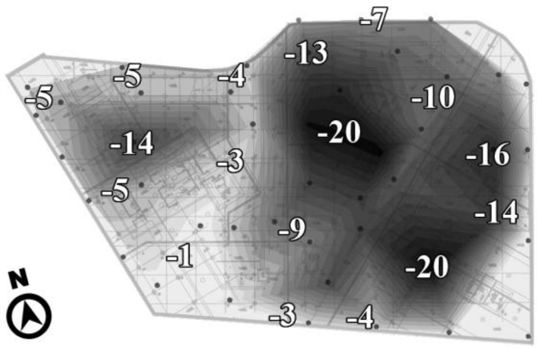

Figure 12. Measured settlements, in millimeters.

8 CONCLUSIONS

The following conclusions have been drawn from this case study:

1) The piled raft have enabled an economical and still safe alternative to the conventional foundation piling practice.

2) With a pile raft coefficient of 0.45 (Katzenbach et al, 2010), piles still play the major role bearing the structure while the raft carries 42% and the slurry wall takes on 13% of the total load.

3) Until the construction of last nine floors of Building A, the design limit for the vertical displacement of the piled raft, which is 35mm, has not been exceed, and a maximum value of 20mm, has been achieved at three survey points: two points (№17 and №25) are under the core of Building B and the other point (№29) is under the core of Building C. All floors of these two structures have already been completed at the time of last measurement.

4) It is also worth mentioning that the 29th survey point has reached close to the extreme displacement value earlier than elsewhere as the result of east-to-west wise construction activity.

5) Total settlements of Buildings A, B and C monitored until May 2008, are in the order of 14 to 20 mm, which had so far stayed lower than the predicted final values of 15 to 36mm.

6) With only half of the high-rise floors of Building A remaining to be constructed, the time-settlement curves of completed Buildings B and C suggest that even though the final values of displacement have not been reached at the time of the last measurement, they are likely to stay lower than the anticipated values.

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