Design and performance of piled raft foundation supporting a 300-m high building in Osaka

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ABSTRACT

This paper offers a case history of 300-m high building in Japan. Since the building has a five-story basement, a top-down method was adopted to save construction time by simultaneous construction of the upper and the basement floors. As a cost-effective solution, piled raft foundation consisting of large-diameter bottom-enlarged cast-in-place concrete piles and steel H-piles built-in soil-cement wall (TSW) was employed. To corroborate the foundation design, field monitoring on the settlement and the load sharing between the piles and the raft was performed. Consequently, it was found that the foundation design was appropriate.

Keywords: high-rise building, piled raft foundation, top-down method, settlement, load sharing, field monitoring

1 INTRODUCTION

A 300-m high building called Abeno Harukas, located in Osaka City, was completed in November 2013 and started business in March 2014 (Figure 1). The building of sixty stories with a five-story basement is now the tallest building in Japan. To support the large structure load effectively as well as to ensure safety during deep excavation works and save construction time, piled raft foundation using a top-down method was employed. Several case histories of piled rafts supporting high-rise buildings constructed by the top-down method were reported (Katzenbach et al., 2000; Yamashita and Hamada, 2013). However, case histories on the monitoring of the settlement and load sharing between the piles and the raft are very limited. This paper presents design and performance of a piled raft foundation constructed by the top-down method supporting the 300-m high building.

2 BUILDING AND SOIL CONDITIONS

Figures 2 and 3 illustrate the cross-section of the building with a soil profile and the foundation plan, respectively. The building, approximately 71 m by 80m in plan, consists of a low-rise section, a mid-rise section, and a high-rise section. To support the large axial loads, concrete filled steel tube (CFT) columns are used in the low-rise floors (and partly in the mid-rise floors). The construction site is located on the Pleistocene terrace surface of Uemachi plateau of which the Uemachi fault exists near the western end. The site is located on the eastern side of the Uemachi fault, and the Pleistocene deposits were found below depths of 1-7 m from the ground surface. The groundwater table of artesian head in the Pleistocene sand (Ds2), in which the raft is embedded, was found 16.2 m below the ground surface based on the in-situ permeability test result, while the water table was found around 6.7 m using dry boring.

Figure 1 View of 300-m high building (Photo by H. Suzuki)

4 FOUNDATION DESIGN

The gross load in the structural design is 3,166 MN with its basement area of 5362 m^2. The average pressure over the raft is 590 kPa (which is nearly equal to stresses in basement excavation), and 716 kPa under the high-rise section. A piled raft foundation consists of a raft and large diameter cast-in-place concrete piles. The raft, consisting of 4.5-m deep foundation beam and 1.0-m thick mat slab with its bottom at 30.5 m depth, was embedded in the very dense sand layer (Ds2).

It is common in Japan that one column is supported by one pile and bottom-enlarged piles are employed in tall buildings to support the large axial loads. This arises probably because the geotechnical bearing capacity of piles in Japanese building design code

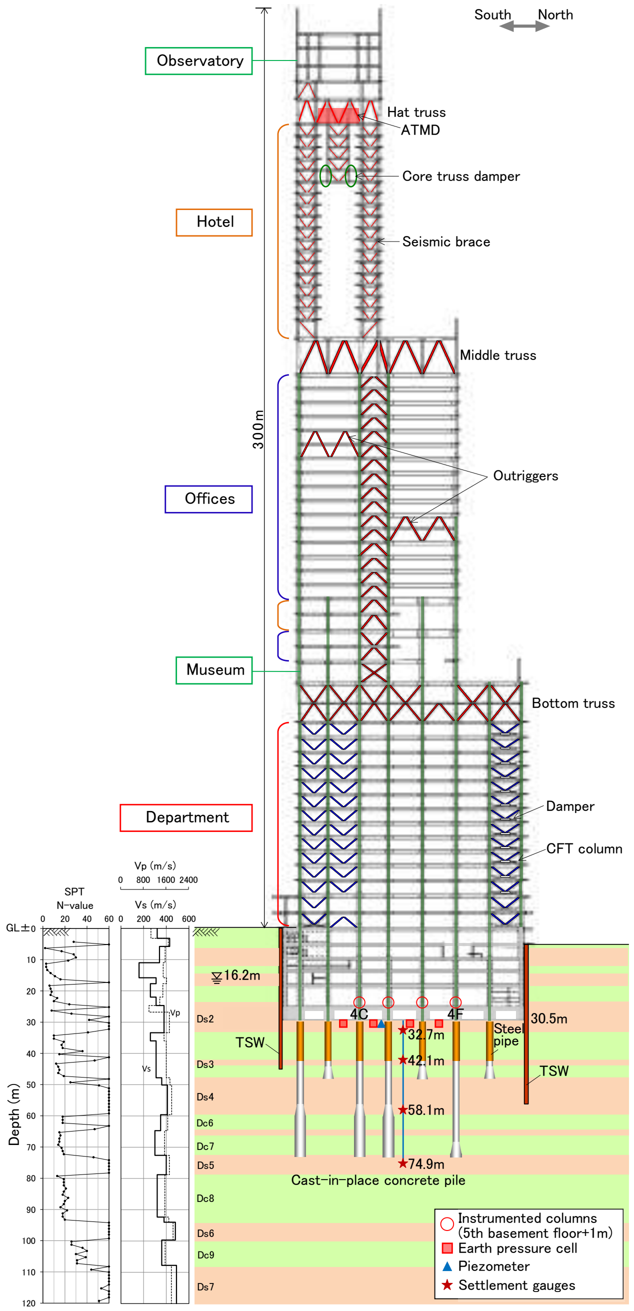

Figure 2 Cross-section of building and foundation (Street 5)

depends significantly on the toe bearing capacity, rather than the shaft frictional capacity. The layout of the cast-in place concrete piles is illustrated in Figure 3. Along the outer perimeter of the basement frame, steel H-piles built-in soil-cement wall (TSW) were placed. The TSW was also used as an earth retailing wall during the underground excavation.

The specifications of the piles and TSW are shown in Table 1. Piles P1, P2 and P3 are placed under the columns supporting the large axial load of 45-80 MN under working load conditions. The pile toes reach the very dense sand (Ds5) below the depth of 70 m from the ground surface, while those of Piles P4 and P5 reach to the very dense sand (Ds4) below the depth of 45 m. The ultimate geotechnical bearing capacity of

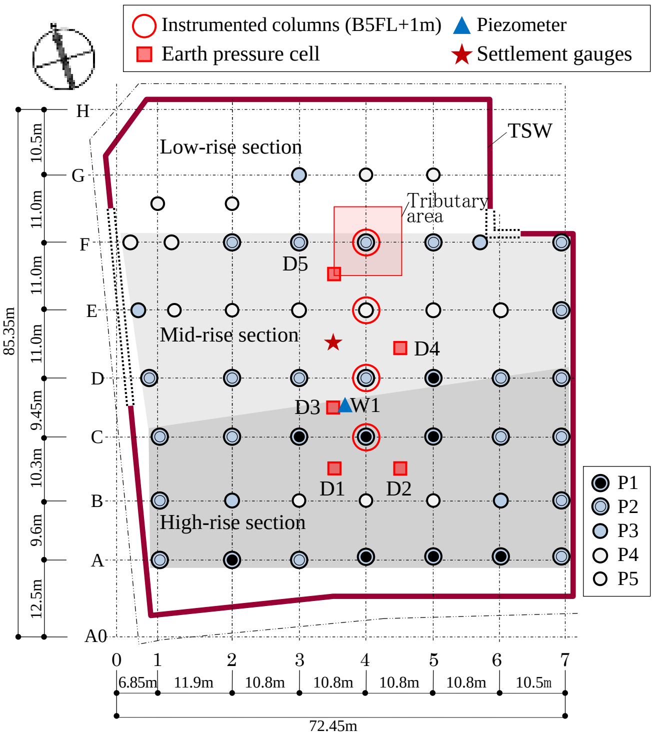

Figure 3 Foundation plan with layout of piles and TSW

Pile P1 was 159 MN, hence, a bottle-shaped enlarged pile toe (4.2 m in diameter) was employed to ensure the large bearing capacity by making use of frictional resistance of the hard clay layers (Dc6 and Dc7) as shown in Figure 2. Piles P2 to P5 have a normal bottom-enlarged shape. The toe bearing capacity of Piles P1 to P3 was determined considering the bearing capacity of a clayey soil below the pile toe (Dc8). For the seismic design, a factor of safety under Level 2 earthquake motions (strong ground motions defined in Japanese code) was set to 1.5 against the ultimate capacity. To cope with the large bending moments caused mainly by the structure's inertial force, a steel pipe having an outer diameter of 2.3-2.5 m (14-25 mm in thickness) and a length of 12.5 m was provided for reinforcement of the top portion of the pile shaft.

In the top-down method, piles support a preceding load which means a temporary construction load before the construction of raft at a bottom of the basement, thereafter both the piles and raft support the subsequent load. Hence, the load carried by the piles and those carried by the raft are evaluated as follows (Yamashita and Hamada, 2013): For piled rafts, the equilibrium equation is given by the equation (1).

$$W = P_p + P_r \tag{1}$$

where W: gross load of structure, P_r: load carried by raft, P_p: load carried by piles

In the top-down method, the equilibrium equations for P_p and P_r are expressed by the equations (2) and (3).

$$P_p = W_1 + \alpha_p'(W - W_1 - U_w) \tag{2}$$

Table 1 Specifications of piles and TSW

	Column load (MN)	Shaft diameter (m)	Toe diameter* (m)	Toe depth (m)	Ultimate capacity (MN)	Concrete strength (N/mm^2)
P1	71.9-79.6	2.5	4.2 (4.1)	72.7-70.9	159	60
P2	46.7-74.0	2.5	4.2 (4.1)	73.1-70.5	140	60
P3	44.6-59.0	2.5	3.5 (3.4)	72.7-70.9	120	48
P4	33.5-48.1	2.5	3.5 (3.4)	48.2	94	48
P5	25.2-42.3	2.3	3.3 (3.2)	48.2	84	48
TSW	―	1.1 (wall width)	―	45.0-55.0 (steel H)	7.2-12.6 (MN/m)	2.0 (soil cement)

* Values in parentheses indicate those used in design.

$$P_r = (1 - \alpha_p')(W - W_1 - U_w) + U_w \quad (3)$$

where W_1: preceding load, U_w: groundwater buoyancy acting on raft bottom, α_p': ratio of load carried by the piles to subsequent net load (net load means gross load minus the buoyancy)

Based on the construction process, the preceding load was estimated to be 60% of the gross load considering that the superstructure frame would be constructed up to 55th floor at that time. For the subsequent load (40% of the gross load), the settlement and the load sharing between the piles and the raft were evaluated using a basement-raft frame model with springs of the piles and the soil. The vertical stiffnesses of the piles and the soil were determined using the simplified analysis method in consideration of the interaction among piles, soil and raft proposed by Yamashita et al. (1998). The soil shear modulus was set at small strain shear modulus, obtained from the shear wave velocities shown in Figure 2, with degradation factor of 0.5-0.7 (which was determined empirically).

The ratio of the load carried by the piles (α_p') was computed as 0.66 (average value) using the simplified method, and the design value of α_p' was set to 0.75 by adding some margin to the computed value. Hence, the ratio of the load carried by the piles to the gross load was assumed to be 0.90 (i.e., 0.60+0.40x0.75) where the groundwater buoyancy acting at the raft bottom was neglected in the pile design on a conservative side. On the other hand, although the ratio of the load carried by the raft to the gross load was given as 0.14 (i.e., 0.40x(1-0.66)) when the buoyancy was neglected, the foundation slab should be designed considering the water pressure acting on the raft bottom at 30.5 m depth. Using the water table of 6.7 m depth from the dry boring, the hydrostatic water pressure was assumed to be 235 kPa at the raft bottom.

5 MONITORING

The location of monitoring devices is illustrated in Figures 2 and 3. The axial loads of CFT columns were measured at 1 m above the 5th basement floor. The settlements of the 1st floor columns were measured using an optical level. Five earth pressure cells and one piezometer were installed underneath the raft to measure the contact pressure and pore-water pressure.

Figure 4 shows the development of the vertical ground displacement measured by the differential settlement gauges. Here, a negative sign means a rebound. The rebounds occurred as the excavation for the basement construction proceeded, and a maximum of 47 mm was observed at 32.7 m depth just below the raft. After the casting of the foundation slab, the settlement of the piled raft was approximately equal to that of the ground just below the raft and 7 mm in April 2013 when about 85% of the gross load in the design was imposed on the foundation. Unfortunately, the settlement gauge at 32.7 m depth ceased functioning. Thereafter, the ground displacements at depths of 42.1 and 58.1 m were quite stable. Figure 5 shows the measured settlements of the 1st floor columns at four points (3D, 4E, 4F and 7D) on February 22, 2013 when about 85% of the gross load was imposed on the foundation. The settlements were 28-33 mm. These settlements correspond to the sum of the vertical displacement of the piles and the axial shrinkage of CFT columns under the 1st floor, which occurred after the construction of the 1st floor. The computed

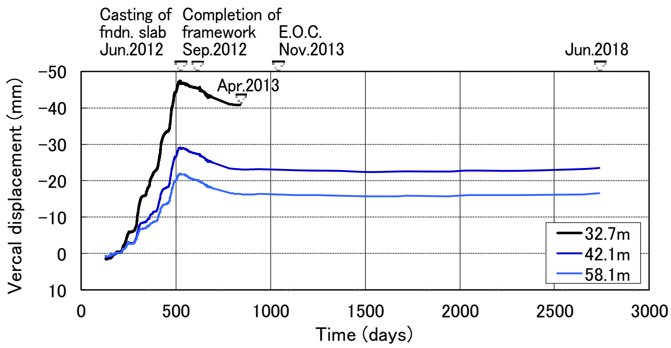

Figure 4 Measured vertical ground displacements

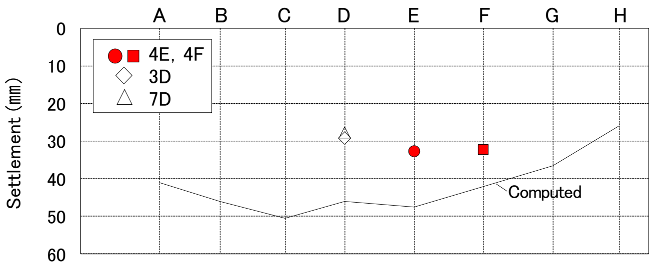

Figure 5 Measured and computed settlement profiles at 1st floor along Street 4 (Feb. 22, 2013)

settlements at the 1st floor (along Street 4) in the design phase are also shown in Figure 5. The computed settlements roughly agreed with the measured ones while the former was larger than the latter.

Figure 6 shows the development of the measured contact pressure and pore-water pressure underneath the raft. The contact pressures increased sharply due to the increase in the pore-water pressure caused by the cease of pumping up after the casting of the foundation slab. The contact pressures were stable after the end of the construction in November 2013 (denoted as E.O.C.). The contact pressures around Column 4C (D1, D2 and D3) were 265-303 kPa and that around Column 4F (D5) was 231 kPa in June 2018, 55 months after E.O.C. The pore-water pressure was 150 kPa at the beginning of March 2013 and seemed to be almost stable. The measured value was consistent with the artesian water pressure in the sand layer (Ds2) from the in-situ permeability test result (140 kPa at 30.5 m depth).

Figure 7 shows the time-dependent load sharing among the pile, the soil and the buoyancy in the tributary area of Column 4F shown in Figure 3. In June, 2018, the measured axial load of Column 4F was 65.4 MN. The gross load in the tributary area was estimated by adding a weight of the raft below the monitoring point at the 5th basement (which was estimated to be 8.0 MN) to the column load, and the gross load was calculated as 73.4 MN. The estimated gross load roughly agreed with the design column load (67.1 MN). The load carried by the raft (27.4 MN) was obtained using the measured contact pressure (D5) in the tributary area by assuming a uniform distribution of the contact pressure on the raft bottom. Then, the axial load of the pile was calculated as 46.0 MN by subtracting the raft load from the gross load. Note that the pile load just before the casting of the foundation slab was approximately equal to the column load (51.6 MN) while the pile load in the design was 60.4 MN (67.1x0.90).

The ratio of the load carried by the pile to the gross load in the tributary area was estimated to be 0.63, while that to the net load was 0.83 in which the pore-water pressure was assumed to be constant after March 2013 as indicated in Figure 7. The ratio of the load carried by the pile to the net load roughly agreed with the design value (0.90) in which the groundwater buoyancy acting on the raft bottom was neglected. Consequently, it was confirmed that the load of the pile in the design was fully greater than those estimated based on the field monitoring from the beginning of the construction to 55 months after E.O.C.

6 CONCLUDING REMARKS

(1) The maximum rebound of the ground just below the raft during the excavation was 47 mm. After the casting of the foundation slab, the settlement of the piled raft due to the subsequent load in the top-down construction was 7 mm when about 85% of the gross load in the design was imposed on the foundation. At that time, the settlements at the 1st floor were 28-33 mm which correspond to the sum of the vertical displacement of the piles and the axial shrinkage of CFT columns under the 1st floor.

(2) The ratio of the load carried by the pile to the net load in the tributary area 55 months after E.O.C. was estimated to be 0.83 and roughly agreed with the design value (0.90) in which the groundwater buoyancy was neglected on a conservative side. Consequently, it was confirmed that the load of the pile in the design was fully greater than those estimated based on the field monitoring.

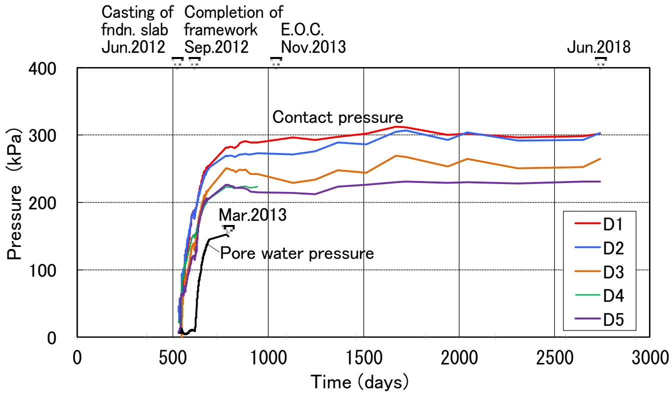

Figure 6 Measured contact pressure and pore-water pressure

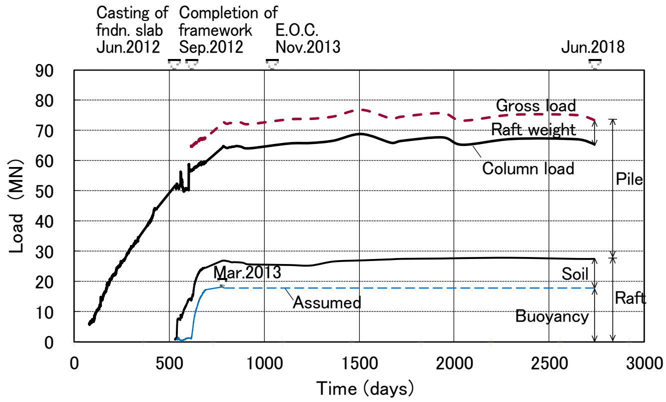

Figure 7 Load sharing between pile and raft in tributary area of column 4F

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