局部桩筏基础的设计与施工

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摘要: 以加拿大多伦多市某工程为例, 介绍了在复杂地质条件下采用局部桩筏基础(PPRF)的设计及施工问题; 探讨了设计 PPRF 的决定性因素; 在保持 PPRF 设计的完整性前提下, 提出了单位沉降量的准则, 并用于筏板和桩的设计; 计算了 PPRF 的滑移及转动; 最后, 采用有效方法对该工程采用的局部桩筏基础进行了计算分析。结果表明, PPRF 的设计主要取决于侧向土压力、分布不均的建筑荷载以及地基土的非均匀承载力, 工程桩应主要布置在沉降较大的区域, 即位于筏板基础承受高压力而土体承载力较低的西北部。探讨局部桩筏基础的设计与施工为该类型工程的基础设计提供了一个新的解决途径。

关键词: 地基基础; 设计与施工; 数值模拟; 桩筏基础

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Design and Construction of a Partially Piled Raft Foundation

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Abstract: The paper describes the design and construction of a Partially Piled Raft Foundation (PPRF) adopted under complex geotechnical conditions, in the City of Toronto, Canada. The design of PPRF was governed by lateral soil pressure, unevenly distributed building loads, and non-uniform bearing capacity of foundation soils. Piles were distributed in the area with excessive settlement. Most of the supporting piles were located in the northwest portion of the raft foundation, where high bearing pressure and low soil bearing capacity were encountered. A unit criterion of the proposed settlement has been applied in the design of the raft slab and the piles in order to keep the integrity of the PPRF. Global stability, including sliding and over turning of the PPRF were an integral part of the design. A state of the art computer analysis was utilized.

Key words: foundation engineering; the design and construction; numerical simulation; partially piled raft foundation

0 Introduction

The High Park project is a medium-density condominium development located in Toronto, Ontario, Canada. Site elevations varied from 101.6 to 102.1 m, along Bloor Street West/Ellis Park Road and step down by approximately 11 m towards the south eastern portion of the site (see Fig. 1).

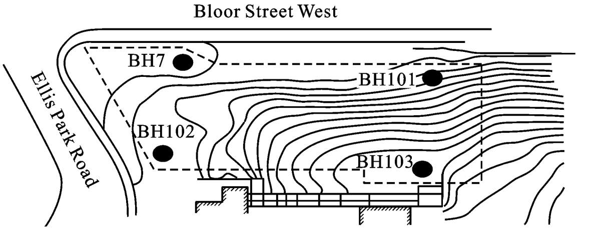

Fig. 1 Site Plan and Borehole Locations

A three level underground parking was constructed under the entire property. General excavation was about 12 m in depth at the north/west side and about 1 m along southeast boundaries.

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Since there were no permanent tiebacks/soil anchors allowed to be installed along Bloor Street West and Ellis Park Road, soil pressure of about 140.4 kPa was distributed on the basement wall, along north and west sides.

1　Subsurface Conditions

Fieldwork for geotechnical investigation consisted of drilling 4 boreholes to maximum depth of 37.4 m. Soil samples were taken using the standard penetration test (SPT) method. A further examination and characterization of soil samples was carried out in laboratory[1]. Borehole locations are shown in Fig.1.

Soil conditions of the project site can be summarized as follows: Fill, about 14.0 to 14.2 m at north side and 1.7 to 7.0 m at south side, of dark brown silty sand to sandy silt was detected on top of loose to compact fine to medium sand. Very stiff, grey silty clay extended to depths ranging from 14.6 to 30.0 m. Compact to very dense sandy silt till, grey and moist to wet, with inclusions of clayey silt/silty sand and gravelly sand layers, continued to depths ranging from 21.9 to 32.9 m. A lower layer of hard clayey silt, grey and moist, was encountered at the top of weathered shale at depths ranging from 22.6 to 34.3 m, see Fig.2. The grey shale of Georgian Bay Formation, inter-bedded with limestone, extended to the maximum depths of borehole explorations.

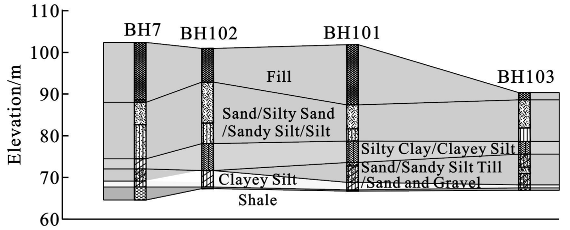

Fig. 2　Stratigraphic Profile of Subsurface Conditions

Groundwater encountered in open boreholes on completion of drilling, was monitored in observation wells. The depth from surface grade to groundwater, on July 17, 1998, ranged from 10 to 18.3 m (Elev. 80.2 m to 82.4 m).

2　Partial Piled Raft Foundation

Based on the encountered subsurface conditions, the partially piled raft foundation was to be constructed on undisturbed native soil and/or on engineered fill, and designed for an allowable bearing capacity of 250 kPa. The strategic use of piles can reduce raft settlements and lead to considerable economy without compromising the safety and performance of the foundation[2].

The raft slab rests on undisturbed natural sand and lean concrete for an area disturbed during the process of fill removal. The underside of the raft slab elevation varies from 87.90 m at the east end of the raft to 92.00 m at the west end, and has been accomplished by series of steps along the length and width of the raft.

Footing Pressures q of the raft, during preliminary design, were computed as

$$q = \frac{\sum P}{A} \pm \frac{M_y}{I_x} \pm \frac{M_x}{I_y} \tag{1}$$

Where P is the vertical load combination; A is the area of raft slab; M_x and M_y are the bending moments around x and y axles, respectively; I_x and I_y are the moment of inertias around x and y axles, respectively. To define the total building load P, six load combinations were analyzed for the ultimate and working state of considered dead, live loading and lateral soil pressure.

Potential settlement of the raft slab (ΔS) was computed as the total of elastic settlement (ΔS_E) and consolidation settlement (ΔS_c) generated from sandy and clayey soils, respectively.

$$\Delta S = \Delta S_E + \Delta S_c \tag{2}$$

$$\Delta S = \frac{1}{E}\int_0^{z1} \sigma_s \mathrm{d}z + \int_0^{z2} m_v \Delta\sigma_c \mathrm{d}z \tag{3}$$

Where z_1 and z_2 are the thickness of sandy and clayey soil layers; E and m_v are Young's modulus and the coefficient of volume compressibility; σ_s and σ_c are load bearing stress acting on sandy soil and clayey soil layer, respectively; z is soil depth. About 80 nodes were laid out on the proposed raft for footing pressure and settlement analysis.

Without piling, the computed settlement for raft foundation was the highest in the northwest corner-about 94.7 mm projected over 20 years. This was due to the footing pressure of about 391 kPa, which was the highest for the entire site.

Piling area was laid out by the comparison of footing pressure and computed potential settlement with the criteria of allowable soil bearing capacity and expected total settlement. The tolerable magnitude of long term settlement was limited as less than 25 mm for 20 years after completion of construction.

Authors then back calculated, footing pressure σ_0 of the raft foundation, with 25 mm total settlement limit. When comparing the required soil bearing pressure resistance σ with σ_0, the difference $\Delta\sigma=\sigma-\sigma_0$ will need to be supported by piles and the area, where $\Delta\sigma>0$ will be the anticipated piling area A. In piling area, the total design load Q will be shared by the raft slab and the driven piles $Q=Q_0+Q_p$, and further

$$Q=\int_0^A \sigma_0 \mathrm{d}a+\int_0^A \Delta\sigma \mathrm{d}a \tag{4}$$

Where, a is per unit area of soil. Amount of piles n was defined by total required load Q_p for piles in the piling area, divided by single pile bearing capacity Q_H, and expressed as $n=Q_p/Q_H$.

Here, it has to be noted that the piles have to be designed with tolerable settlement equivalent to the tolerable settlement of raft slab. Allowable bearing capacity Q_h single pile was designed as:

$$Q_h=\sum_{z=0}^{l} c\sigma_{\mathrm{sh}}\Delta z+A_{\mathrm{t}}\sigma_{\mathrm{t}}-W_{\mathrm{p}} \tag{5}$$

Where l is the soil depth in the research; σ_{sh} is the shear stress along the shaft of driven pile; σ_{t} and A_{t} are the toe bearing capacity and toe area of the pile, and W_{p} is the gravity weight of the pile.

The total required load is about 30 000 kN and the computed bearing capacity for a single H-pile is about 1 000 kN. Hence, 30 piles were required and were installed on the construction site. Piling area basically is located at the zone of northwest corner. Piling area and pile locations are shown on Fig.3.

Piles were driven into a very dense sand or sandy silt with blow count (N) more than 50 per every 300 mm. The in situ pore pressure build up/dissipation was relatively rapid in the encountered cohesionless soil. To reduce potential impact of pile driving on subsurface soil conditions, vibration monitoring was carried out by McClomnt and Rak Engineers. Monitoring results show that there was no adversely side effect on soil bearing capacity, as pile driving further compacted subsurface sandy soil.

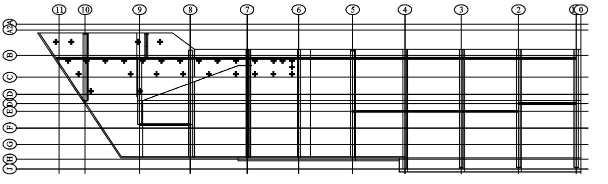

Fig. 3 Piling Area and Pile Distributions

3 Geotechnical Parameters

Geotechnical parameters for raft foundation design were modulus of subgrade reaction, Young's modulus and coefficient of consolidation settlement[3].

The modulus of subgrade reaction is a conceptual relationship between soil pressure and deflection[4]. Considering the encountered consolidation deformation of clayey soils, the modulus of subgrade reaction k_{s}, was defined as: $k_{\mathrm{s}}=\sigma/\Delta S$ and we have:

$$k_{\mathrm{s}}=\frac{\sigma}{\Delta S_E+\Delta S_{\mathrm{C}}} \tag{6}$$

$$k_{\mathrm{s}}=\frac{k_{\mathrm{s}E}\Delta S_E}{\Delta S_E+\Delta S_{\mathrm{C}}} \tag{7}$$

Where $k_{\mathrm{s}E}$ is elastic subgrade-reaction.

Young's modulus E is a function of N value of standard penetration test (SPT) data, and varies with soil type and soil structures. To the same type of soil, Young's modulus' change always follows the N value of SPT. The empirical equation and the correlation with SPT data, for glacial till in the area of Great Toronto Area (GTA), are shown in Fig.4[5].

Coefficient of consolidation settlement m_{v} is also defined as coefficient of volume compressibility[4]:

$$m_{\mathrm{v}}=\frac{1}{1+e_0}\frac{e_0-e_1}{\sigma'_1-\sigma'_0} \tag{8}$$

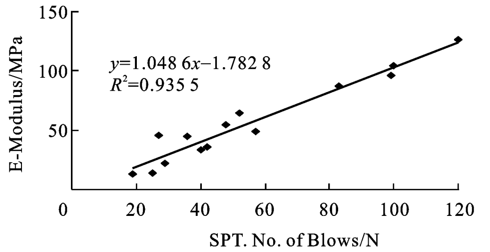

Fig. 4 Correlation of Young's Modulus vs SPT data[5]

Where e_0 and e_1 are void ratios prior to and on completion of settlement; σ'_0 and σ'_1 are stresses before and on completion of settlement. If m_v and $\Delta\sigma' = \sigma'_1 - \sigma'_0$ are assumed to be constant with depth, then consolidation settlement can be calculated as:

$$s_c = m_v \Delta\sigma' H \quad (9)$$

With units of m_v are inverse of pressure (m^2/MN). Where H is soil thickness.

4 Foundation Design

Detailed foundation design was carried out in close cooperation with the project's structural engineers[6]. Based on the results of preliminary foundation design, a total of 30 H-shaped piles was required in the northwest portion of the raft to reduce potential settlement. The total load shared by piles was 30 000 kN, and was to be spread over 30 piles, each of them with bearing capacity of 1 000 kN.

The amount of long-term total settlement, expected to be in the range of 20 to 25 mm with differential settlement to be in the range of 5 to 10 mm, was the focus of the design. It was assumed that approximately, 60% to 80% of the expected settlement would occur within the first 2 years.

A commercial software (SAFE), specifically developed for concrete raft system, was utilized in the PPRF design. Results from the vertical and lateral load analysis were used as input into the SAFE model. Structural foundation and shear walls were incorporated in the safe analysis.

Since the piles were distributed throughout the area where excessive settlement was expected, settlement of the proposed PPRF varied, along with the various load distribution configurations between piles and raft; hence the modulus of subgrade reaction had to be varied.

Subgrade response was modeled by the variations of footing pressure and settlement distributions. To determination the proper magnitude of the modulus of subgrade reaction, at different locations within the raft footprint, required a very close collaboration of the structural and geotechnical engineers[7].

Computer analysis for bearing pressure distribution was conducted by structural engineer through 'SAFE'/finite element method (FEM). Initial modulus of subgrade reaction k_s was supplied by geotechnical engineer according to soil conditions:

$$k_s = k_1 \frac{m + 0.5}{1.5m} \quad (10)$$

Where k_s is desired value of modulus of subgrade reaction for the full-size (or prototype) foundation; k_1 is value obtained from a plate-load test using a 0.3 m× 0.3 m or other size of load plate; m is the ratio of length to width of a rectangular footing on stiff clay or medium dense sand.

Using computed bearing pressure, soil settlement was calculated without considering the stiffness of raft foundation. Subsequently, the subgrade reaction was updated based on the footing pressure computed by FEM and calculated raft settlement.

By using the updated modulus of subgrade reaction computed for each node, a revised analysis was carried out and a corresponding bearing pressure distribution was laid out. It should be noted that in the updated analysis, the modulus of subgrade reaction variation within the raft foundation, and the effect of piles were coupled - see the simplification in Fig.5.

Again, using the revised soil bearing pressure, an updated settlement calculation was obtainedand new "k_s" value was generated at each node. Using the new modulus of subgrade reaction an updated distribution of bearing pressure was computed (see Fig.6).

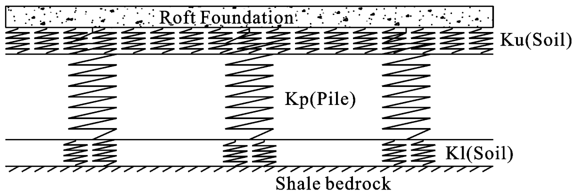

Fig. 5 Plate-on-spring Approach of Piled Raft Foundation

In essence, the modus operandi was that by using the soil bearing pressure to compute settlement, and then updating the modulus of subgrade reaction, and repeating the procedure till the computed soil bearing pressure and expected settlement were brought to acceptable range-see Fig.7.

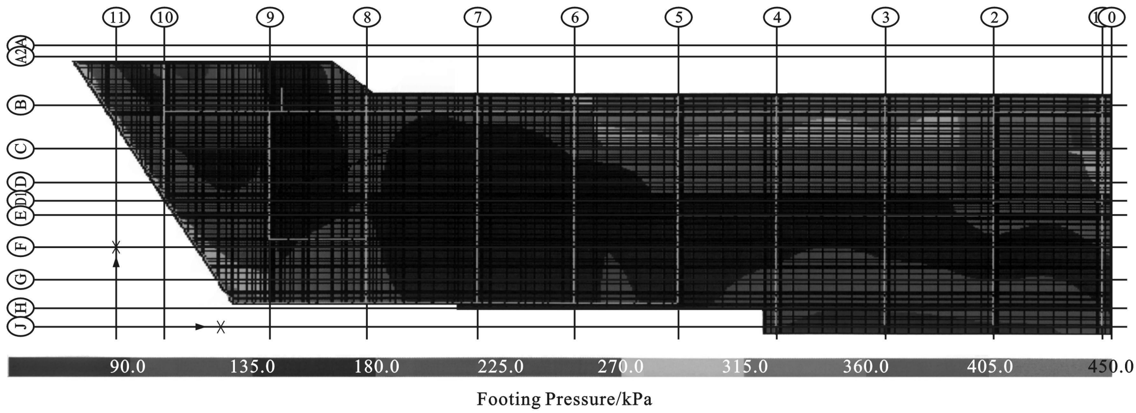

Fig. 6 Computed Footing Pressure by Using FEM Program SAFE

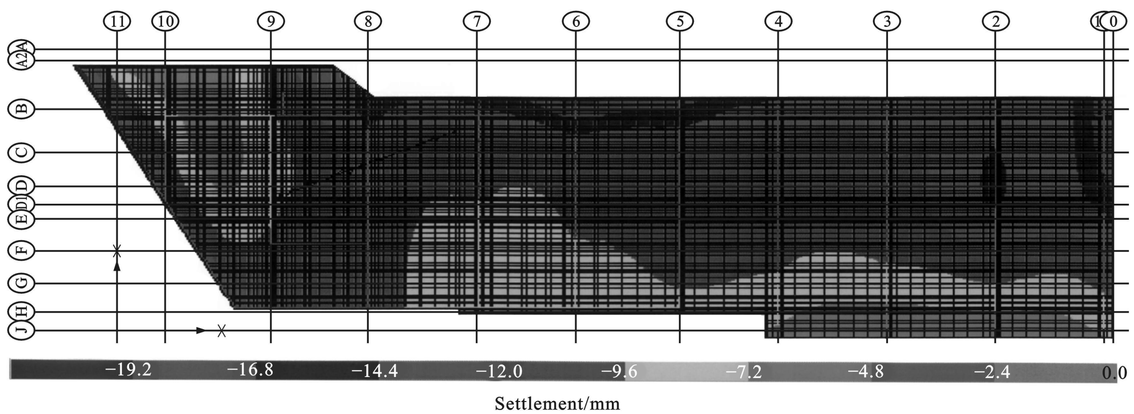

Fig. 7 Computed Potential Settlement by Using FEM Program SAFE

This convergence was achieved when the soil pressure and displacements, predicted by the finite element analysis, matched the soil bearing capacity and the settlement predicted by geotechnical engineer[8].

Global stability for the critical sections, incorporating lateral pressure and the proposed structure, was analyzed using the simplified bishop method for circular slip surfaces. A licensed commercial program, SB-slope, developed by Von Gunten Engineering Software, Inc., Fort Collins, Colorado, was primarily used for the two-dimensional analyses. Deep seated failures were evaluated using radius options with minimum depth and elevation search. The minimum factor of safety (FS) calculated was about 2.95, which was acceptable according to Canadian Foundation Engineering Manual. Results of global stability computation are shown in Fig.8.

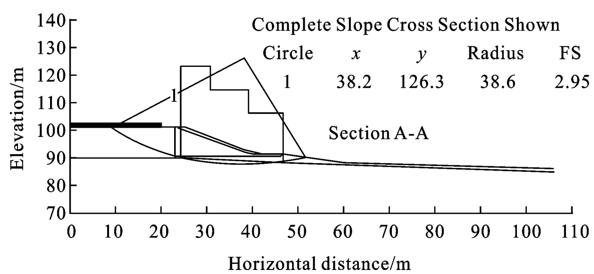

Fig. 8 Global Stability by Using Simplified Bishop Slope Stability Analysis

5 Foundation Construction

The general sequence of construction operations has an important influence on the soil displacement and the corresponding pile movement. The sequence of pile installation was arranged as those piles along the perimeter of the foundation were driven first. As the result, the heave of the soil surface in the central area of the foundation was increased and that of the surrounding area correspondingly decreased.

In addition, the piles installation was carried out at a virtually uniform spacing throughout the foundation area, hence the stresses produced by pile driving were distributed uniformly throughout the foundation area, and the soil displacement had been minimized.

In-situ monitoring of vertical and horizontal foundation soil movement was carried out during pile driving/foundation construction. The localized depression surrounding the H pile has a circular or an oval shape, with a diameter of about 600 to 900 mm. The dimensions and the shape of depression "doughnut" change with the soil composition/density.

Results of the on site monitoring and experience of pile foundation construction show that in fine grained/cohesive soils (such as the lower deposits on site, i.e. in the silts and clayey tills), approximately half the volume of displaced soil appeared as surface heave, within the area of the pile foundation, while the remaining half was took place outside of the building area. However, in the upper sands, the volume displacement within the building area was reduced by about 20% due to the sand densification.

Pile driving was governed by the encountered foundation conditions and the large difference in elevation, between the northern and southern site boundary, the actual pile driving could typically displace soil laterally towards the lower elevation (open slope). However, the preferred sequence of pile driving on the subject site was arranged that the piles along the perimeter of the building had been driven first. This assured that the heave of the bottom of the excavation in the central area of the site was increasing and that of the surrounding (i.e. build up) area correspondingly decreased.

The surface heave of the bottom of the excavation and the surrounding area (prior to, during and upon completion of pile driving) was estimated by the following steps[9]:

(1) The volumetric displacement ratio (V_r) was calculated by dividing the total volume of the inserted piles by the volume of soil enclosed by the pile foundation.

(2) The normalized soil heave (h_n), equal to the soil heave divided by the pile length, was estimated empirically. For the low displacement H piles used on this project, the pile dimensions and the proposed foundation plan, the normalized soil heave was as expected, approximately one-half the volumetric displacement ratio obtained in step A (see Fig. 9). This can be described by an empirical equation $h_n = 0.5V_r$.

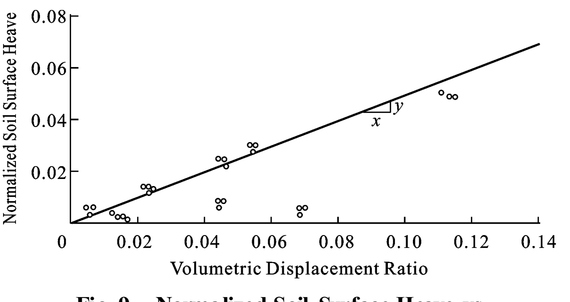

Fig. 9 Normalized Soil Surface Heave vs Volumetric Displacement Ratio[9]

(3) The heave of the soil surface was estimated to be the product of the normalized soil heave and the average length of the piles.

In order to monitor the soil displacement/heave during the pile driving, precision survey and monitoring work (including cyclops type total work station system) was carried out. The monitored maximum heave, settlement/deflection and bearing pressure, both in magnitude and range of distribution, for the PPRF were all in acceptable range based on the reference criteria derived from the initial foundation analysis and design.

Vibration monitoring was carried out during pile installation and foundation construction by the measurement of peak particle velocity (PPV). State-of-the-art equipment measurement and precision survey-such as the cyclop type total work station, was used. Great care was used in the selection of monitoring locations and sensor/target placement as they are important factors in site monitoring/vibration measurements. Ground vibration trigger levels was programmed, in the practical range, to eliminate the readings outside the pre-set lower and the upper acceptable limits.

In consultation with the pile driving contractor, the actual pile installation technique was discussed and verified in the field during the initial work phase. The type/weight of hammer and driving energy selection were based on the results of pile analyzer test and the results of vibration monitoring. The measured peak particle velocity versus distance is plotted in Fig. 10.

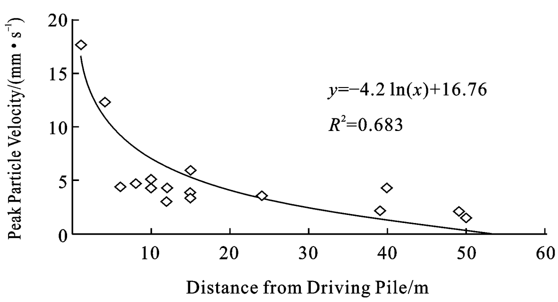

Fig. 10 Peak Particle Velocity Versues Distance from Driving Piles

6 Discussion and Conclusion

As soil composition was non uniform and bearing pressure from the structure varied, the computed settlement distribution was also variable. Adding piles, in a critical/heavily loaded area of raft foundation, was an effective way to limit the differential settlement within a tolerable criterion.

Soil conditions, load distribution, quantity and location of piles were important factors in raft foundation design. Subgrade reaction, Young's modulus and coefficient of consolidation settlement prove to be very sensitive parameters in the computer modeling for partial piled raft foundations.

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