Pylon foundation of a cable stayed bridge at the motorway ring road of Wrocław

Fondation d'un pylône du pont suspendu du périphérique de l'autoroute de Wrocław

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ABSTRACT: The largest bridge of the motorway ring road of Wroclaw is a cable stayed bridge over the Odra river near Rędzin. Due to hydrogeological conditions of the ground it was not convenient to use standard bored piles of high length. Instead shorter piles were applied with additional improvement by cement injections at the pile toes. This decision was possible by extending ground investigation program and application of numerical modeling of the soil-structure interaction with nonstandard material models of the overconsolidated clayey deposits. Sufficient stiffness and bearing capacity of piles was approved by field tests. Overall performance of the foundation was monitored during construction and after completion of the bridge. Comparison of settlements predicted during design and obtained in the monitoring is presented and discussed.

RÉSUMÉ : La plus grande construction sur le périphérique de l'autoroute de Wrocław est le pont suspendu sur le fleuve Odra à Rędzin. L'utilisation des pieux normalisés de grandes longueurs a été jugée inconvenable à cause des conditions hydro-géologiques. Les pieux de grands diamètres raccourcis avec un renforcement de la base par l'injection de ciment ont été donc utilisés. Une telle solution a été rendue possible grâce à un programme élargi de caractérisation du sous-sol ce qui a permit de réaliser les calculs du projet en utilisant des modèles de comportement non standards adaptés aux matériaux à grains fins surconsolidés. La rigidité et la résistance correspondante des fondations sur pieux ont été confirmées par les essais de chargement in situ. Le comportement de la fondation en entier a été contrôlé durant sa construction et durant la période d'exploitation après la fin des travaux. Les déplacements de la fondation évalués en cours de projet et après, par observation, ont été présentés et discutés.

KEYWORDS: piled-raft foundations, pile foundations, finite element analysis

1 INTRODUCTION

Main section of the bridge of 612m length will be suspended from a single pylon of 122m high. Large loading from the pylon structure causes the design of the foundation to be a challenging geotechnical problem due to high level of loads to be safely transmitted into the subsoil. In the case of the cable-stayed bridge over the Odra river the highest characteristic vertical load, considered among various combinations of loads analysed by the designer, transmitted onto the support was estimated to be 776.0 MN. Dimensions of the bridge makes it the highest object of that type in Poland.

2 SOIL AND WATER CONDITIONS

Designed foundation will be placed in the central part of Rędzin Island within the main Odra river bed. The island is connected with the river banks and mainland through two Rędzin locks, which are located ca. 70 m north and through historical weir, located ca. 110 m south-west. The area designated for foundation is 67.4 m x 28.0 m.

The terrain under foundation is basically flat with mean elevation of 112.8 m a.s.l. From the geomorphological point of view it is located in Wrocław-Magdeburg ice-marginal valley (Odra River valley), 10 km wide and filled with Pleistocene and Holocene river sediments with several terraces at various levels.

The surface layer of fills and normally consolidated river accumulation formations 2.0 m thick lies on dense coarse material of the thickness of 6-7 m with unconfined water table at the mean elevation of 107.6 m a.s.l. Below coarse sediments there are fine soils of tertiary origin represented by clays and locally silty sands. In this continuous, 15 m thick layer local water percolations are observed. Below it, thin layer (2.5 m) of silty sands and sandy silts with confined water table under high pressure was found.

Table. 1: Characteristic values of basic geotechnical parameters

Layer	Soils	(γ/γ') [kN/m^3]	ϕ' [^{o}]	c' [kPa]	E_{oed} [kPa]	ν [–]
IIa/IIb	Cl, siCl, Si	21,0/11,0	15,0	5,0	30 000	0,20
IIIa	MSa, CSa, FSa	19,0/10,0	33,0	1,0	85 000	0,20
IIIb	MSa, CSa, grCSa	20,0/10,0	35,0	1,0	150 000	0,15
IIIc	grCSa, Gr	20,0/10,0	35,0	1,0	220 000	0,15
Va	Cl, siCl,	21,5/11,5	23,0	18,0	40 000	0,20
Va*	Cl, siCl,	21,5/11,5	23,0	18,0	100 000	0,20
Vc	siSa	20,5/11,0	32,0	1,0	85 000	0,15

To the depth of 50 m below the subsoil surface there were no weak soils found. Tertiary clays are characterized by good strength and stiffness. The only problem related to the depth of the pylon foundation are local water percolations and high water pressures in confined aquifers. The most unfavorable foundation and execution conditions have been assumed for the further calculations and analyses, see Figure 1 and Table 1.

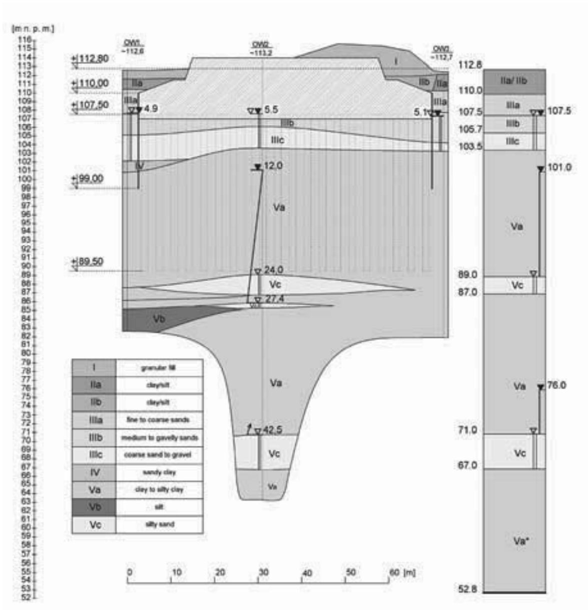

Figure 1: Characteristic geotechnical cross-section with the simplified system of layers.

3 TECHNOLOGICAL SOLUTION OF THE FOUNDATION

Four alternatives regarding the foundation of the pylon were being considered i.e.: shallow foundation in the layer of dense coarse soils, foundation on the diaphragm walls, foundation on the block made of jet-grouting columns and foundation on large diameter bored piles. Finally, the latest concept has been assumed for design. Additionally bored piles were strengthened by the injection under the pile base. Such solution was found to be optimal technologically in the soil conditions. The concept of shallow foundation was rejected due to small thickness of coarse material below the foundation level. In the case of diaphragm walls the problem might be low shaft bearing capacity. The foundation on the block made of injection columns has been rejected due to large volume and mass of the block.

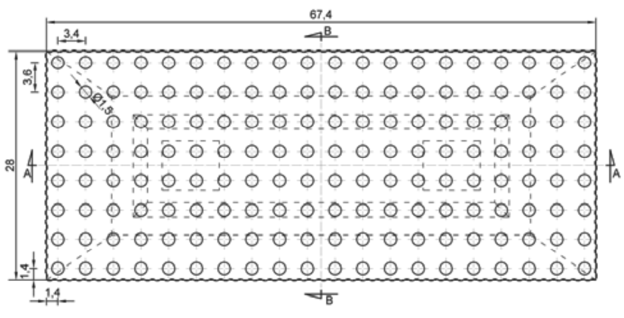

Figure 2: Projection of the foundation with the lay-out of pile heads.

The foundation foot has size of 67.4 m x 28.0 m and is founded on 160 large diameter bored piles. The piles have a rigid connection with the slab. The piles of diameter D=150 cm and length of 18 m are spaced in the rectangular grid 3.4 m x 3.6 m, (Fig. 2). The bottom of the foundation slab is localized at the elevation of 107.5 m a.s.l. and rests on the 0.5 m thick layer of blinding concrete. The perimeter of the foundation foot was protected by sheet pile wall of the length of 11.0 m (between elevations of 99 m to 110 m a.s.l.). The sheet pile wall is not a foundation element transmitting the loads into the subsoil but it is used as erosion protection.

The lack of high strength soil layer, did not allow to design base bearing piles, hence the raft foundation system was designed. In such system transmission of loads takes place both by piles as well as by foundation slab. The main layers deciding of bearing capacity and settlements of the foundation are layers of over-consolidated tertiary clays No. Va and Vc. The base level of the piles is designed in the upper part of geotechnical layer No. Va above the confined aquifer Vc. The confined aquifer is not considered as weak layer from the strength point of view, nevertheless it should be protected against any perforation due to high water pressure occurring in it. It was recommended to concrete the piles by dry method however this recommendation could not be fully achieved. The piles have been strengthen by injection under their bases. During the injection both grout pressure and pile heave were controlled.

4 CALCULATIONS

At the preliminary stage of the foundation design several schemes of pile foundation system were analyzed. In the simplest scheme no direct soil - foundation slab interaction as well as infinite stiffness of slab were assumed (rigid foundation method). Here, maximal compression force in pile was estimated at the level of 7200 kN for the envelope of maximum moments acting at the top of foundations, whereas minimum compression force was 5300 kN. In the next calculation stage the foundation was analysed as boundary-value problem solved by finite element method. The discretization was based on structural elements such as shells and beams resting on elastic supports. The characteristics of elastic supports have been calculated based on the stiffness of soil layers. The soil response under the slab was assumed as uniform passive ground pressure equal to 100 kPa. It allowed to assess the values of internal forces in the foundation slab and in the piles. These forces were necessary for design of the reinforcement. The maximum calculated axial force in the pile was 7367 kN and meets standard bearing capacity condition just on the edge of safety whereas maximum bending moment was 4742 kNm.

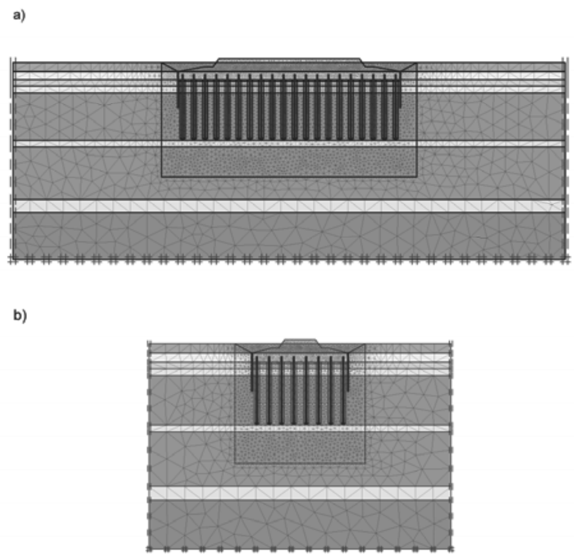

Figure 3: Foundation model with FEM mesh in plane strain state: a) longitudinal section, b) cross-section. Piles and sheet pile walls are modeled by beam elements with averaged stiffness.

Final calculation stage regarded numerical simulation of boundary-value problem by finite element method with discretization of the foundation body and geotechnical layers by continuum elements. The piles were discretised by beam elements directly interacting with soil elements. Calculations were carried out for simplified schemes in plane strain state with averaging stiffness of piles in rows and in the complex three dimensional model. In the later case alignment of beam elements is independent of the mesh representing soil (so called embedded piles). Bearing capacity of the pile base is modelled by equivalent force. Its value is calculated based on the stress state and bearing capacity of a soil at the level of pile base. Schemes of plane strain and three dimensional models are shown in Figs. 3 and 4.

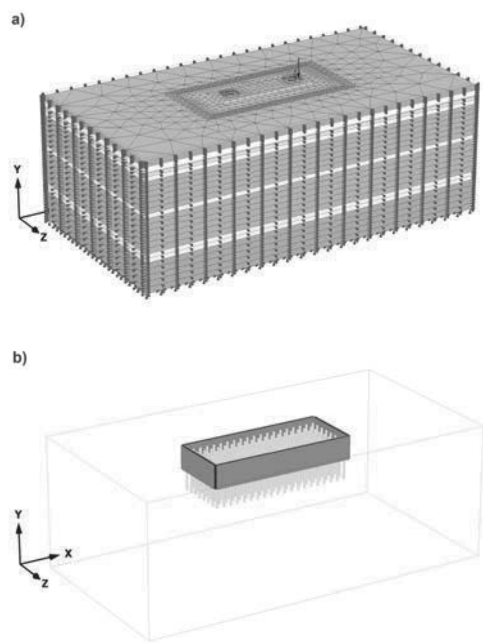

Figure. 4: 3D FEM mesh of foundation model: a) general view with boundary conditions, b) detail of pile elements (so called embedded piles) and sheet pile wall elements (plate elements). Foundation body modelled by continuum elements.

The main goal of the last calculation stage was a determination of the subsoil resistance and an assessment of the foundation settlements during the construction phase as well as during its operation. Besides strength of the soil layers which was modelled by either Coulomb-Mohr or Matsuoka-Nakai strength criteria, special attention was paid to possibly accurate description of the soil stiffness. For that purpose non-linear stiffness model including description of small strain behaviour of soil was adopted. Conventional implementation of constant stiffness with secant parameters (Table 1) leads to unrealistic overestimations of the settlements in majority of practical cases of raft foundations. It is well known from the laboratory and in situ investigations that the stiffness increase with the mean effective stress, however on the other hand it degrades due to accumulation of large shear strains (Atkinson, 2000), (Santos and Correia, 2000). The nonlinear small strain stiffness was included in the model for fine grained soils. Small strain stiffness moduli for the soil layers were determined in the triaxial tests equipped with bender elements, (report (2), 2008).

The value of current shear modulus G was calculated according to the following formulas:

$$G = G^{ref}\left(\frac{\sqrt{\frac{1}{3}\sum_{r=1}^{3}\sum_{s=1}^{3}\sigma_{rs}\sigma_{rs}}}{p_{ref}}\right)^{1-\beta} \qquad (1)$$

$$G^{ref} = G_0\left(\frac{\gamma_{0.7}}{\gamma_{0.7}+\frac{3}{7}\gamma}\right)\ , \qquad (2)$$

where G and G^{ref} are current and reference tangent shear moduli respectively, $G^{ref} = E^{ref}/(2(1\text{-}\nu))$ at mean reference stress $p_{ref} = 200$ kPa; σ is the stress tensor; $\beta = 0.5$ is power parameter expressing the dependence of the stiffness on effective stress and $\gamma_{0.7}$ is shear strain at which shear modulus G^{ref} decreases about 30% with respect to G_0 – initial modulus at reference stress p_{ref} (see Fig. 5).

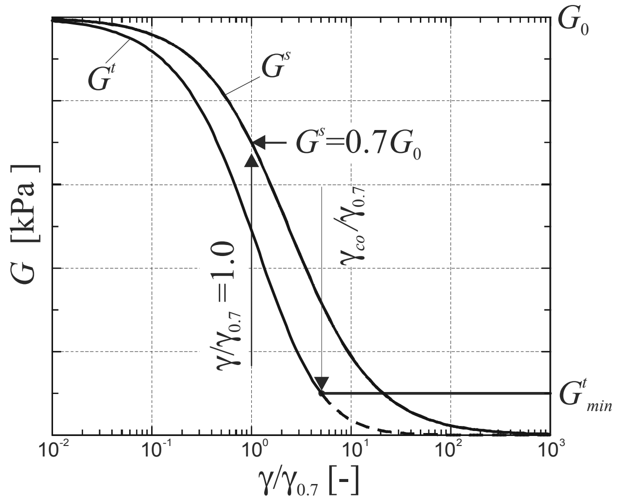

Figure 5: Dependence of shear modulus on shear strains. G_t=G_{ref} and G_s are tangent and secant shear moduli, respectively and G_{tmin} is a minimal shear modulus at the accumulation of shear strain γ_{co}.

The calculation results show that for estimated strength parameters of the soil layers stability of the foundation is preserved. In the numerical simulations the foundation was stable even for doubled loads. Minimum factor of safety for the foundation model received by ϕ-c reduction method was $F = 1.35$. The uniform settlement of the foundation was 0.08 m, whereas maximum settlement difference was 0.01 m. Exemplary distribution of vertical displacements for the calculation scheme assuming maximum horizontal loads has been shown in Fig. 6.

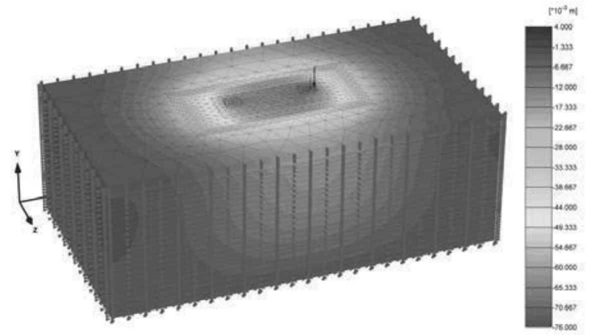

Figure. 6: Distribution of vertical displacement obtained for maximum lateral load calculation scheme.

5 LOAD TESTS OF INDIVIDUAL PILES.

In order to verify the design assumptions as well as to meet general recommendations regarding pile foundations, the bearing capacity of piles was investigated by static load tests. Due to importance of the construction as well as from the design assumptions, the load tests program was extended (after the agreement of the designer and investor). In order to examine the effectiveness of the pile base improvement by injection as well to control its quality, one of piles designated for testing was installed without the injection. Additionally, in three of the test piles, extensometer measuring system to control the distribution of force along the pile was installed. The goal of these measurements was to investigate the distribution of load transmitted by the pile shaft and base. It is also useful to assess the force generated at the pile base due to injection. Additionally, extensometer measurements were planned for the case of unfavorable bearing capacity test results. The measurement results would be useful for the analysis of the reasons of too low bearing capacity (shaft of pile or its base). The extensometers were installed at seven levels distributed along the pile length.

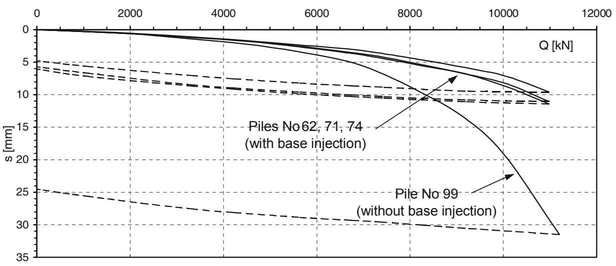

Figure 7: Pile load test results

Pile load tests were carried out according to the standard procedure i.e. to the maximum force Q_{max} = 11000 kN with intermediate unloading at the force Q_1 = Q_r = 7400 kN. The results of load tests confirmed the need of extended measuring program. The gained information proved the safety of the current and future foundation work. The important observation was proper work of the piles with injection and their advantage over the pile without base improvement. It is well visible in Fig. 7 where respective load-settlement curves are shown. The results of extensometric measurements in the form of distribution of force along the piles are shown in Fig. 8. They allowed for the examination of the work of the piles in the existing soil conditions. The essential portion of force was transmitted by the pile shaft what corresponded to the assumed concept of piles in the raft foundation system.

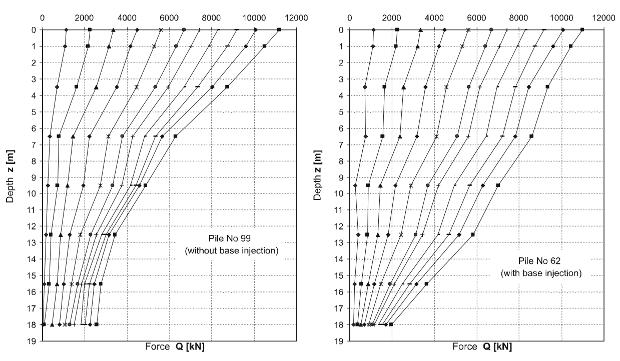

Figure 8: Distribution of forces along the pile form the extensometric measurements.

6 FIELD MEASUREMENTS

After piling and pylon slab construction, six benchmarks were placed on the slab surface in characteristic points (see figure X). Since 11.09.2009 till now the vertical displacements of foundation are geodetically monitored. After every stage of bridge loading vertical displacement were measured. Currently bridge is fully loaded, and settlements are in a stabilisation phase. Good agreement between measured displacements and numerical analysis results is observed (see Fig. 9).

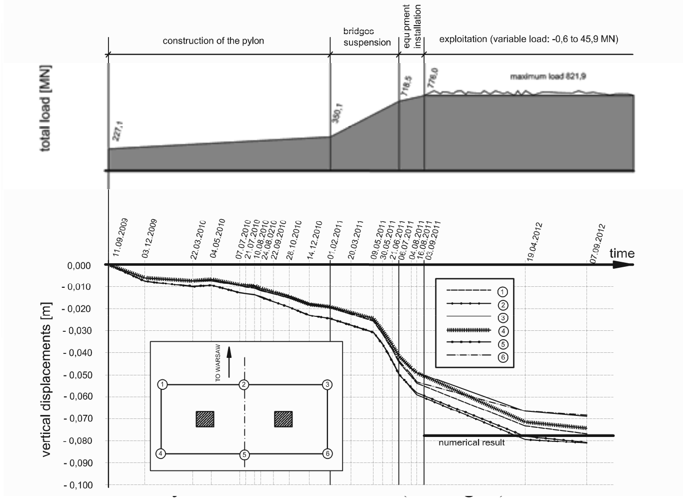

Figure 9: Settlements of foundation slab measured in 6 points in different phases of pylon loading.

7 SUMMARY

In the paper analysis of the raft foundation supporting the pylon of cable-stayed bridge in Rędzin being the part of Wrocław motorway A8 ring is briefly presented. Important decision related to the selection of the pile length was their shortening due to occurrence of confined aquifers which would cause the liquefaction of soils under the pile bases during installation. The decision could be undertaken after comprehensive numerical analyses as well as additional, non-standard investigations of soil parameters. Load tests with the measurement of the distribution of force along the piles with extensometers were very useful, allowing the control of design assumptions. The field measurements and their agreement with numerical analyses results are the best proof that assumptions made in the design process were correct.

8 REFERENCES

Atkinson, J.H. *Non-linear Soil Stiffness in Routine Design*, 40th Rankin Lecture, "Geotechnique", 2000, vol. 50 No. 5, 487-508.

Geological report (1), A-8 Motorway, AOW, GeoTech Ltd. Rzeszów 2006, (in Polish).

Kosecki M.: *Static of piling structures*, PZTB Szczecin branch, 2006, (in Polish).

Elaboration the way of foundation of the P14 support of cable-stayed bridge pylon in the motorway ring (A8) over the Odra river in Wrocław, Geosyntex Ltd., Gdynia 2008, (in Polish).

Santos, J.A, Correia A.G.: *Reference Threshold Shear Strain of Soils, Its Application to obtain a Unique Strain-Dependent Shear Modulus Curve for soil.* Proc. The 15th Int. Conf. On Soil Mech and Geot. Eng., Instanbul, 2000, vol. 1, 267-270.

Geological report (2) on the geotechnical and laboratory investigations under the pylon and permanent supports for construction of cable-stayed bridge over the Odra River in Wroclaw, Geoteko - Design and Geotechnical Consulting Ltd., Warszawa, 2008, (in Polish).