Design of a piled raft foundation for a building at an alpine valley in Switzerland: Soil-structure interaction analysis and comparison with pile test results

Dimensionnement d'un radier sur pieux d'un bâtiment situé dans une vallée des alpes helvétiques. Analyse de l'interaction sol-structure et comparaison avec des résultats des essais de charge statique sur pieu isolé.

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ABSTRACT: The Gotthard Residence, a project consisting of two towers (12 and 9-storey) connected by a common podium, is being built at Andermatt, (Swiss Alps on the Urseren Valley). The subsurface of the valley consists of river and lacustrine deposits with a thickness up to 200m. This geological setting turned an end-bearing piled foundation unfeasible. A floating piled raft foundation was adopted. Static Pile load tests on large diameter bored pile and screw displacement piles have been tested up to failure, and were used to check the calculation rule, stated in the Belgian National Annex of Eurocode 7 and used according to the Swiss code SIA 267. Due to the nature of the soil, the analysis of single pile behavior is not sufficient and a soil structure interaction analysis was performed using a three-dimensional geotechnical FEM program to simulate the interaction between raft, pile and soil, and a structural 3D model. Both programs were used iteratively in turns until convergence of pile head settlement in both was obtained. The results demonstrate large expected settlements, the importance of accounting for loading interference, pile-raft-structure interaction on pile loading and on settlement prediction. Predicted and measured settlement and settlement troughs below the buildings compare well.

RÉSUMÉ: Le Gotthard Résidence, composé de deux tours de 9 et 12 étages reliées par un podium, est en construction à Andermatt (Alpes Suisses, Vallée d'Urseren). Le terrain est composé des dépôts fluviatiles et lacustres très compressibles avec une épaisseur qui peut excéder 200 m. Ce contexte géologique rend inadéquate une fondation sur pieux résistant à la pointe ; un radier sur pieux flottants a été adopté. Néanmoins, même pour cette solution, des tassements significatifs sont prévus. Des essais de charge statique ont été effectués sur des pieux vissés et des pieux forés de grand diamètre, permettant de vérifier les calculs de la capacité portante des pieux utilisant l'annexe nationale belge de l'Eurocode 7 et la norme suisse SIA 267. Vu les caractéristiques des sols, l'analyse du comportement d'un pieu isolé ne suffit pas. Aussi, une analyse de l'interaction sol-structure a été réalisée à l'aide du logiciel FEM 3D Plaxis pour simuler l'interaction entre le radier, les pieux et le sol, et d'un modèle 3D structurel dont les pieux sont modélisés comme des ressorts découplés. Les programmes ont été utilisés itérativement jusqu'à arriver à la convergence du tassement des têtes des pieux dans les deux modèles. Les résultats montrent l'occurrence de tassements excédant 10 cm, démontrant l'importance de tenir compte des interactions pieu-radier-structure sur la prévision des tassements et des efforts sur les pieux. Les tassements et la forme de la cuvette de tassements sous les bâtiments telles que prédits et mesurés correspondent bien.

KEYWORDS: Pile foundations, Pile Raft, Static load test, Soil-structure interaction

1 INTRODUCTION.

Andermatt, located in the Swiss Alps in the Urseren Valley is being developed intensively, including the construction of several multi-storey buildings. The Urseren valley is a high alpine valley with a generally very flat surface surrounded by the slopes of the mountains at the perimeter of the valley. The development is located in the downstream part of the large flat valley formed mainly by very thick loose silt and sand lake deposits.

The largest project in the development is the Gotthard Residence, consisting of a 9-, and a 12-storey tower (Hotel and Residence) located close together, one conference hall, one public pool building and one parking lot, all being interconnected by a one-floor podium. The total footprint area is over 9.500 m². The compressible loose silt and sand deposits require the use of 25 to 30 m long floating piles to control the settlements. Large settlements are expected, as well as differential settlements between the buildings. Therefore a detailed soil-pile-structure interaction is needed to assess settlements and solicitations in the structural members. The present paper describes the selection of pile type, the pile soil-structure interaction design and compares the results to the measured settlements during construction of the reinforced concrete structure of the two tower buildings.

2 GEOLOGICAL AND GEOTECHNICAL SETTING

2 .1 *General geological setting*

The subsurface of the valley at the project location consists of loose sand, silt –clay lake deposits over a thickness that may exceed 200m. It has probably been formed in a way that is similar to the forming of a dredge deposit. This might lead to coarse material deposited in the upstream part of the area, while fines are transported much further. The planned resort is in the downstream area, thus where the amount of fines is high. The valley has not been used for construction works before creating the development.

2 .2 *Geotechnical conditions and soil profiles*

The in-situ fieldwork conducted to assess the geological and geotechnical conditions consisted of 3 boreholes of 30 m, in which 15 SPT blowcounts were performed, 39 dynamic penetration tests (DPT) and 19 Cone Penetration Tests (CPT) to depths of 45 m to 60 m. The figure 1 shows the lithography and a typical CPT result (CPT02 close to static test on Fundex pile is shown).

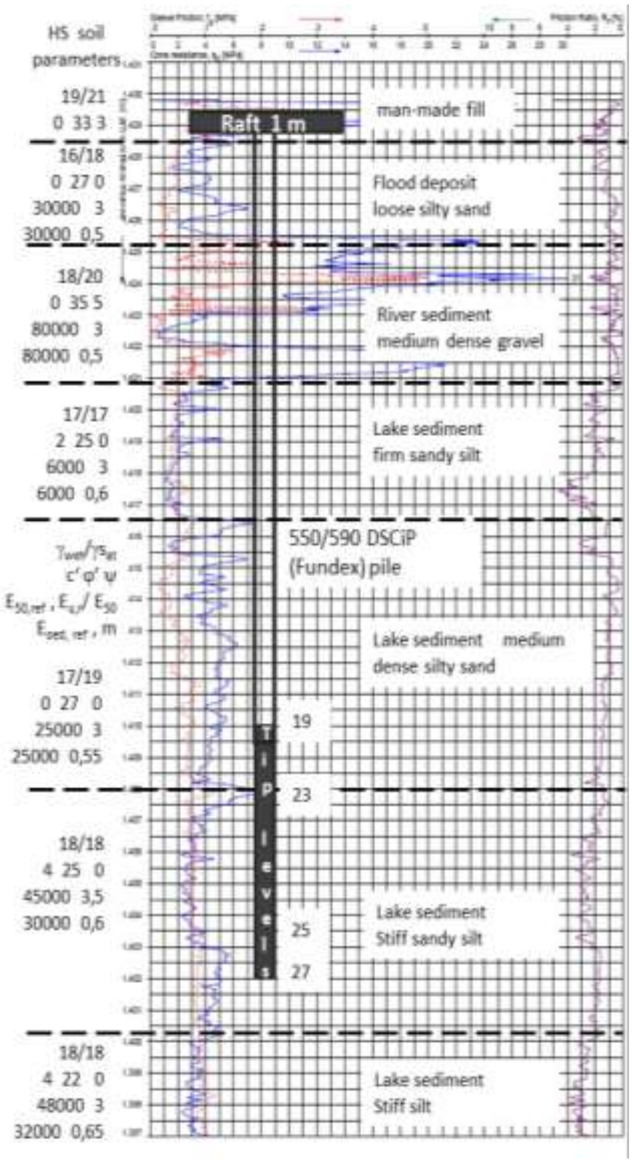

Figure 1. Lithological description, typical CPT and Hardening Soil parameters used for FEM

The lake sediments are quite homogeneous over the project area; the more recent river sediments and flood deposits however are variable in thickness and in packing.

The groundwater level is close to the soil surface and subject to seasonal variations. Flooding of the valley occurs quite frequently.

For the calculations, the soil has been considered as Normally Consolidated although some preconsolidation has probably developed due to ice and snow load and fluctuations of the groundwater level.

The figure 1 also provides the values of the material parameters (in kN and m) derived by correlations from the CPT results and used for the PLAXIS Hardening Soil model in the 3D geotechnical FEM calculations.

3 DESCRIPTION OF FOUNDATION SOLUTION

The load imposed on the foundation level and the compressibility of the ground led to the adoption of a pile foundation. Given the fact that bedrock would not be found at depth that would allow for a base-bearing solution, a floating pile solution has been considered from the start. The length of the piles should be sufficient to average variations of soil conditions over the site, and to reach the medium dense silty sand layer of the lake deposits.

In Andermatt large diameter (Ø1500mm) bored pile are generally applied. However, these are less appropriate for the intended floating pile foundation, as the ratio of pile excavation and concrete per square meter of frictional surface is economically unfavourable. Furtheron, pile load tests showed no advantage in unit skin friction for this type of piles compared to Displacement Screwed Cast in Place (DSCiP) piles of the Fundex type (see section 4).

Therefore, DSCiP piles were selected, with screw diameter of 590 mm and nominal shaft diameter of 550mm. 5 pile lengths were adopted, namely, 19,0 m, 23,0 m, 25,5 m, 26,0 m and 27,0 m. The total number of piles installed for the resort project is 790; 410 piles support the towers.

The piles are located under the load bearing structural elements. A monolithic raft without any dilation joint connects all the buildings. It aims to smoothen differential settlements and provides watertight floor below water level.

4 ESTIMATION OF SINGLE PILE BEARING CAPACITY

To estimate the single pile bearing capacity at each CPT profile, the calculation rules stated in the Belgian annex to EN 1997/1 are used (BBRI, 2008). The base resistance, R_b, is determined by :

$$R_b = a_b * \lambda * \beta * A_b * q_b \quad (1)$$

In Eq. 1, A_b the pile base section, q_b is the unit base resistance determined according to the De Beer method and α_b is an installation factor that translates the influence that the pile technology has on the ground stress. λ and β are reduction factor for piles with enlarged base and non-circular or square section (here both equal to 1,0).

The shaft resistance, R_s, is determined by:

$$R_s = \chi * \Sigma\,(a_{si} * h_{si} * \eta_{si} * q_{ci}) \quad (2)$$

In Eq. 2, χ is the shaft perimeter, α_s is the installation factor that translates the influence that the pile technology has on the ground (de)compression and shaft roughness and $\eta_{s,i}$ factor depending on the soil type. For full-displacement piles, the installation factor α_s is higher than for full excavation type of piles translating the favorable effect on shaft friction mobilization associated to the densification imposed by the lateral compression during driving or screwing of displacement piles.

For bored piles and DCSiP piles described further, the installation factors α_b are 0,5 and 0,8 and α_s 0,6 are and 1,0 respectively (BBRI 2008); for the gravel layer η_{si} = 1/100, for the lake deposits η_{si} = 1/60

The design value of single pile bearing capacity, $R_{a,d}$, was determined according to SIA 267:2003 through Eq.3

$$R_{a,d} = (\eta_a * R_{a,k}) / \gamma_{M,a} \quad (3)$$

In Eq. 3, η_a is a model factor to cover the uncertainties in determining the ultimate resistance analytically, equals 0,9 in absence of load test and to 1,0 when load tests are carried out; $\gamma_{M,a}$ is the partial safety factor on pile resistance, equals to 1.3.

The characteristic pile bearing capacity, $R_{a,k}$, is determined as a cautious estimate of the bearing capacities calculated at each CPT location according to the "model pile" procedures of EN 1997.

5 PILE TESTING

5 .1 *Load testing on bored piles*

A static load pile testing campaign was performed in 2012 at the worst CPT location on a 24 m and a 36 m long Ø1500mm bored pile according to SIA 267/1. Figure 2 shows the load-pile head settlement curves.

The average unit shaft friction measured using optical fiber extensometer technology and was shown to be approximately constant over the length of the pile. The α_s value considered in BBRI (2008) underestimates the shaft friction. The creep load deduced from the tests is approximately 70% of the failure load.

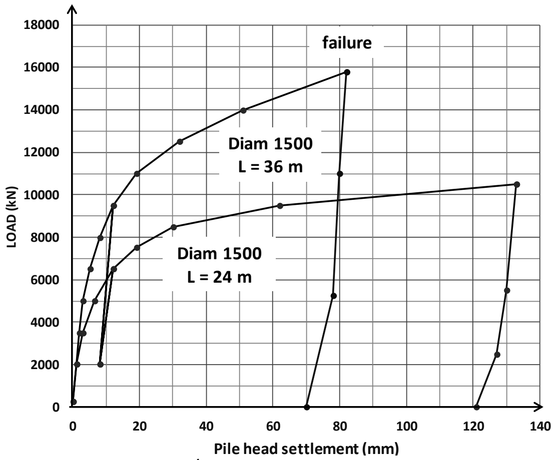

Figure 2. Static load test. Ø1500mm Bored piles 24 m and 36 m

5.2 *Load testing on screw full-displacement piles*

Static load testing according to SIA267/1 was carried out in 2015 on three DSCiP piles with a length of 22 m, 26 m and 27 m. Figure 3 shows the load-displacement curve for the 26 and 27 m pile. The result for the 22 m long pile is unlikely high and was be disregarded in further design.

The first feature in the load-displacement curve is the significant plastic displacement exhibited after the unloading procedure. The load-displacement curve shows that for two of the last three load steps, there were some difficulties in load stabilization. The creep coefficient is approximately 2mm for the penultimate load step, which defines the ultimate bearing resistance according to SIA 267/1. In fact, this creep behavior, common to all three tests performed for loads over 70% of the proof load corroborates the behavior shown in the bored pile tests, and confirms that a non-negligible fraction of settlement may occur during time due to creep if the piles are loaded above a certain threshold. The development of the normal force along the shaft deduced from load test indicates that a very small end bearing resistance develops and a more or less constant friction with depth should be considered when modelling the pile.

The load tests on the DSCiP piles have been performed at the second worse CPT location instead of the lowest CPT location. A spatial variability correction factor of 1,08 based on the CPT result at the test location is introduced to correct for that spatial variability, and to compare with the load test on the bored piles.

Comparison of the load test results and the bearing capacities calculated according to section 2 is given in table 1. The adopted calculation rule provides safe estimation of single pile bearing capacity. For the soil-structure interaction (performed with non-factored loads, i.e. at SLS), the “allowable load” will be used as acceptance criterion. It accounts for the spatial variability through the coefficient 1,08. This value fits well with the characteristic value of load that can be applied to the piles according to the calculations based on the CPT results and accounting for their spatial variability.

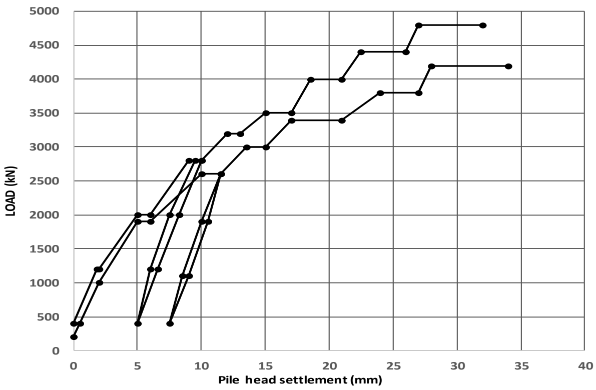

Figure 3. Static load test. Ø590/550mm DSCiP (Fundex) pile length 26 m and 27 m.

Table 1. Measured and predicted single pile bearing capacity for different test pile length (DSCiP 590/550). All loads in kN.

	22 m	26 m	27 m
Measured Ultimate load according SIA 267/1 (k > 2mm)	(3850)	3800	4000
Allowable working load at test pile location (50% ultimate load or 90% creep load)	(1925)	1900	2000
Allowable working load from load test reduced for spatial variation by 1,08	(1778)	1759	1850
Applied load for rare Service Limit State	1450	1650	1750
Predicted ultimate capacity at test location according to BBRI (2008)	3250	3550	3650
Predicted R_d accounting for spatial variability ξ = 1,15; (η = 1,0)	2100	2400	2450
Characteristic value of load that can be applied: $F_k = R_k/1,4$	1510	1700	1750

Comparison of the ultimate bearing capacities of bored and DSCiP piles shows, however, that the bearing capacity per m³ of pile of the DSCiP piles is higher than the bored piles’ one. Despite its larger number of piles, the foundation with DSCiP piles is more economic than using large diameter bored piles for the specific ground conditions of the Project requiring floating piles.

6 SOIL-STRUCTURE INTERACTION

6.1 *Introduction*

A large group of floating piles supporting a raft in rather compressible deposits implies a complex interaction of soil, piles and structure. The behavior of a single pile as observed in the pile load test does not depict the foundation behavior. Classical spring models based on single pile behavior are insufficient to assess settlements and forces in the structure as they do not account for several effects that are fundamental in this type of solution, such as pile group and raft interaction as well as redistribution of the loads and negative skin friction and bending of periphery piles induced by raft loading. These effects are even enhanced in the Project due to the proximity, at one point, of the two towers (see figure 4), and some structural particularities of the project such as the monolithic raft common to all high and low structures and a high concentrated load of 25 MN applied locally where the highest tower building crosses the road underneath at its corner near the other tower building.

All these reasons made evident that a refined soil-structure interaction was needed for the geotechnical as well as for the structural design of the buildings. The aim of such analysis is to assess the pile loads, the global settlements and the corresponding solicitations in all the structural members to allow accurate design of their sections and reinforcement.

6.2 *Soil-structure calculation procedure*

The interaction has been analyzed using two codes used in sequence: a structural Finite Element Model SCIA simulates the entire buildings allowing for the consideration of their stiffness above the foundation raft, while 3D FE Plaxis code simulates the raft, the piles and the soil. The loads from the structures were transmitted to the elastic spring node supports in the SCIA model simulating the piles. The obtained reactions are applied as loads on the piles in Plaxis. Both programs are run in sequence until the pile loads and the pile settlements in both models converge to the same values. In order to assess the effect of raft and its stiffness, the raft is modelled in PLAXIS as a massless plate. The contact pressure obtained using PLAXIS was introduced in the structural model as well. The piles were modelled in PLAXIS using the embedded pile formulation. Mobilization of shaft friction was defined according to a linear distribution of shaft resistance mobilization. This linear function was defined in a way that the full shaft resistance would correspond to the one determined in the calculation rule and validated in the pile load tests. The Hardening Soil model was used. The main parameters are shown in figure 1.

The structure is assumed to have a linear behavior and is considered as being built in one step, ignoring the stepwise increase of stiffness of the building during its erection.

6.3 *Numerical results*

Figure 4 shows the calculated settlements (without weight of the raft) for entire fully terminated Project at the end of the iteration procedure. One observes the interaction effect between the two towers, leading to an overlap of the settlement troughs of both buildings. Differential settlements remain well within limiting values and are accounted for explicitly in the structural and architectural design. The pile loads do not exceed the allowable values in table 1.

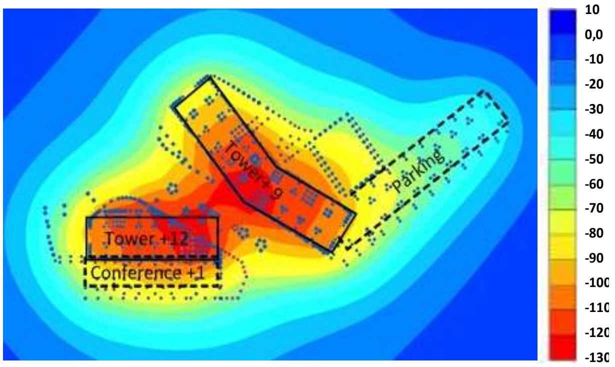

Figure 4. Calculated surface settlements (mm) at final load (maximum value 124,7 mm); the 9 and 12 level towers, adjacent low rise buildings and pile locations (dots) are indicated.

7 MEASURED SETTLEMENTS AND DISCUSSION

The settlements during construction were monitored, after the execution of the raft and basement walls, at different locations in the building and in the surroundings. The figure 5 shows the settlement and the applied footprint load during construction as function of time. Settlements due to the weight of the raft and basement is considered to be very low.

An increase of the rate of settlement is observed at a mean footprint load of 50 à 60 kPa corresponding probably to some preconsolidation threshold.

The applied loading at the end of civil construction works does not include the weight of the shell and finishing works as they were not yet installed. The load applied to the foundation represents 70% of the total load considered for the settlement calculations presented in section 6.3. The final settlements are thus expected to be about 40% larger than the values measured at the end of the civil works.

The maximum settlements values correspond quite well with the calculated ones. The observed and predicted general shape of the trough (see figure 6) also correspond well.

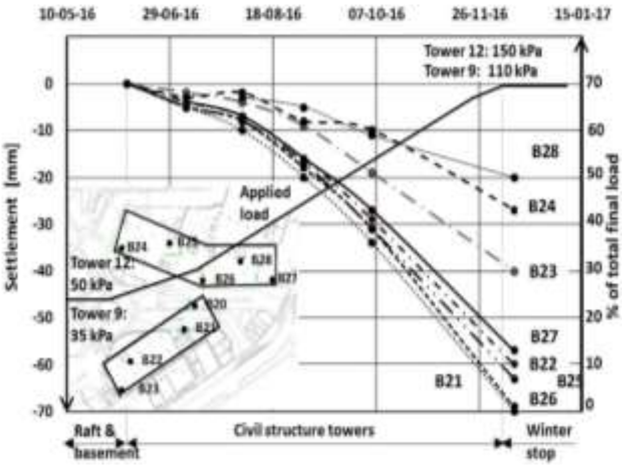

Figure 5. Settlement curves (after raft) as function of applied loading. At the winterstop, 70% of the final load applies to the foundation as finishing and shell are not yet constructed.

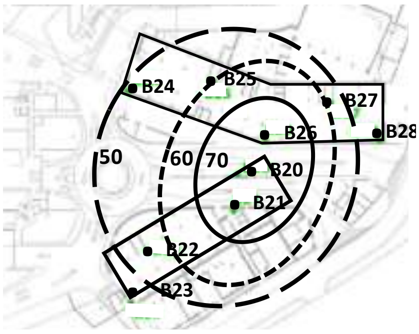

Figure 6. Measured settlement trough (mm) by structural load (after raft construction). Values to be multiplied by 1,4 to extrapolate to settlement at final load.

8 CONCLUSION

The 3D analysis of the piled raft foundation for Andermatt Gotthard Residences has shown the importance of a soil/structure interaction to assess several effects with importance for both geotechnical and structural design. The static pile load tests were fundamental in providing the adequate insight on pile behavior. Improvement to further design may consist in accounting for some preconsolidation and stepwise increasing stiffness of the building during its construction.

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