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PRELIMINARY FIELD TESTS AND LONG-TERM MONITORING AS A METHOD OF DESIGN RISK MITIGATION: A CASE STUDY OF GDAŃSK DEEPWATER CONTAINER TERMINAL

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ABSTRACT

Appropriate risk assessment plays a fundamental role in the design. . The authors propose a possible method of design risk mitigation, which follows recommendations included in Eurocode 7. The so-called "Observational Method" (OM) can produce savings in costs and programmes on engineering projects without compromising safety. The case study presented is a complex design solution that deals with the heavy foundations of a gantry crane beam as one of the elements of a Deepwater Container Terminal extension. The paper presents a detailed process of the design of the rear crane beam being a part of the brand new berth, together with its static analysis, as well as the long-term results of observations, which have revealed the real performance of the marine structure. The case presented is based on excessive preliminary field tests and technical monitoring of the structure, and is an example of a successful OM implementation and design risk mitigation.

Keywords: monitoring,field test,Observational Method,CFA piles,micropiles,geotechnics

INTRODUCTION

Geotechnical Eurocode 7 (EC7) [6] recommends verification of limit states by one or a combination of four possible methods: **use of calculations**, adoption of prescriptive measures, experimental models and load tests or an **Observational Method (OM)**. In addition, on the basis of geotechnical design the European code introduces geotechnical categories of structures from 1 to 3, from relatively simple structures to structures involving abnormal risks, unusual or exceptionally difficult ground and loading conditions. The geotechnical categories are in line with local regulations [13].

More and more often in geoengineering the risk matter and its appropriate management are now being considered. Topolnicki [19] has classified available methods of ground improvement, paying particular attention to risk factor, and has introduced three categories of increasing hazard from A (low hazard) to C (high hazard).

In 2002, a European geotechnical forum was set up for the exchange of best practice ideas and innovations in geotechnical engineering, called GeoTechNet [15]. This forum published a document promoting modern design tools, including the application of the Finite Element Method (FEM) and the observational method, which can reduce costs and programmes on engineering projects without compromising safety. It also shows how the geotechnical community can benefit from developing scientific knowledge.

In current everyday design practice, however, most designs are based on engineering calculations only, with no or marginal use of the observational method. The aim of this study is to illustrate, taking recently completed extension of the Deepwater Container Terminal in Gdańsk as an example, how the design process can be improved by effective implementation of the observational method, leading to mitigation of risk and optimized engineering solution.

OBSERVATIONAL METHOD

The forerunner of the observational method in geotechnics was Peck [16]. In a recent work on OM, published by the Construction Industry Research and Information Association's Report 185 (CIRIA) [12], the definition of OM approach reads: "*The Observational Method in ground engineering is a continuous, managed, integrated, process of design, construction control, monitoring and review that enables previously defined modifications to be incorporated during or after construction as appropriate. All these aspects have to be demonstrably robust. The objective is to achieve greater overall economy without compromising safety.*"

Traditional ground engineering projects are usually based on a single, robust design and there is often no intention of varying the design during the construction phase [17]. Optional monitoring, if carried out, plays a very passive role to check only if the original predictions are still valid and provide confidence to all parties involved in the process (eg. client, contractor, designer). In comparison, in the OM monitoring plays an active role in both the design and during construction, allowing planned modifications to be carried out within an agreed contractual framework.

The enhancement of OM is also described in the Eurocode 7, but should only be considered whenever prediction of geotechnical behavior is "*difficult*" or the complexity of the interaction between the ground and the structure makes it "*difficult to design*". This code sets some general rules for OM that are required before construction is started. However, as stated by Patel et al. [15], EC7 is inconsistent and is lacking in any detailed instructions the geo-engineers shall follow. Moreover, the code doesn't concentrate on the advantages OM can bring to a typical construction processes, but recommends the method as one of the optional, alternative approaches to design. A totally different scientific approach is represented by the promoters of the observational method, who prove the effectiveness of appropriately implemented OM in major European projects [15]. The method requires full consciousness of the construction process and active participation and management by client, designer and contracting teams. Significantly more time is dedicated to designing and planning than constructing, but this leads to an efficient and effective organization of the engineering projects.

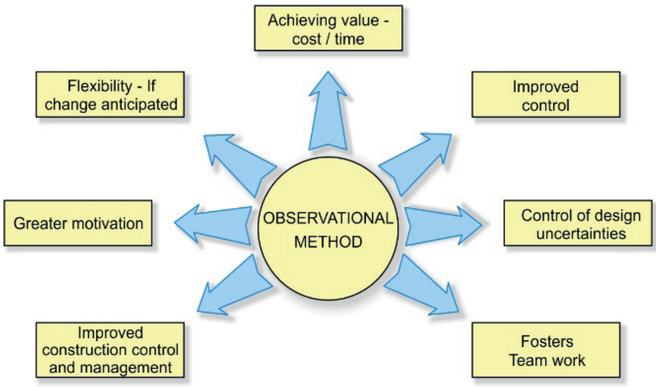

Fig. 1. Potential benefits of the OM [12]

The latest and more promising technologies like Building Information Modelling (BIM) can serve the observational method as a professional tool supporting the integrated process of planning, building and operating the investments.

Topolnicki [18] describes the high accuracy of BIM application in geotechnics and advocates the use of GeoBIM upgrade of the system that will take into account soil-structure interaction affecting the construction process. Due to its digital character and high management effectiveness, BIM can accelerate the preparation process of error-free design documentation and improve the execution process consequently optimizing the global cost of the project.

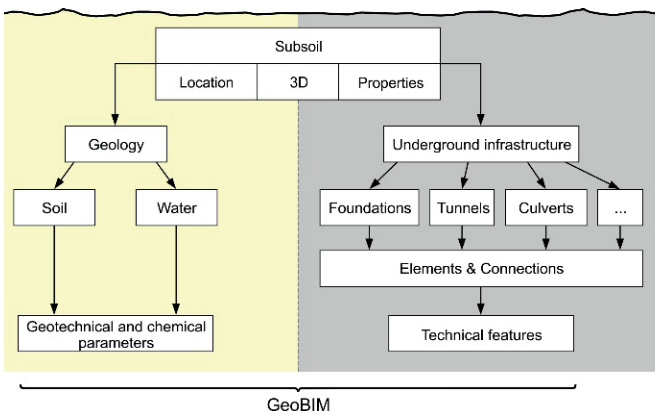

Fig. 2. GeoBIM model chart [18]

DESIGN METHODS

The Eurocode determines design methods for pile foundations and recommends the design should be based on one of the following approaches:

a) empirical or analytical **calculation methods** whose validity has been demonstrated by static load tests in comparable situations;
b) the results of **static load tests**, which have been demonstrated, by means of calculations or otherwise, to be consistent with other relevant experience;

c) the results of dynamic load tests whose validity has been demonstrated by static load tests in comparable situations;
d) the observed performance of a comparable pile foundation, provided that this approach is supported by the results of site investigation and ground testing.

Thus far, as engineering practice has always shown, the most common and traditional approach is a design based on calculations in which the load test is only verifying the final solution. Using empirical or analytical **calculation methods** requires its coherence with variety of static load tests performed in comparable situations and the wide experience of a designer in charge. In general, soil investigation is often insufficient and represents single points (eg. boreholes, soundings, etc.). This may lead to unsafe simplifications. Determining soil deformation and strength parameters may also be questionable. Part of the design risk may come from inaccurate calculation methods or the software used to estimate pile bearing capacity. Therefore, it should be noted that EC7 allows the use of the static load tests approach as a method of design risk mitigation.

There is no better and more reliable direct method of pile bearing capacity assessment than a **load test** performed on full-scale piles. Nonetheless, a tight construction schedule is often the main reason for not conducting the tests. It is generally believed, however, that the utilization of field test results in the design process will minimize risk and lead to safe and optimized foundation solutions. Consequently, well-planned field loading tests should be conducted in advance to production piles to allow early verification of the design performance in terms of pile stiffness and ultimate bearing capacity in soil conditions relevant for the specific site.

The adopted testing procedures, particularly with respect to the number of piles tested, loading steps and the sequence and duration of loading/unloading cycles, shall be such that conclusions can be drawn about the deformation behaviour, creep and rebound of a piled foundation from the measurements recorded. Moreover, the loading shall be such that the ultimate pile bearing capacity should be readily assessed [6]. This is often difficult to achieve for compression piles when the load versus settlement plots show a continuous curvature. In these cases, according to EC7, a limit settlement of pile head equals to 10% of pile base diameter can be adopted as an "ultimate" settlement.

OVERALL CONCEPT OF A CASE STUDY

The presented case is an example of the implementation of a field test programme as a method of design risk mitigation. It represents a design solution that deals with a heavy foundation of a gantry crane beam as one of the elements of the Deepwater Container Terminal (DCT) extension (Fig. 3). A new 656 m quay, with adjacent 25 ha container storage yards, allows for the terminal to meet the growing demand for deep-sea services in Central-Eastern Europe and enables the handling of ultra large container vessels [1]. The DCT is located in the industrial part of the city of Gdańsk, on the Vistula Spit which forms a natural barrier against sea intrusion. Soil sedimentation transported by the Vistula River was the main phenomenon in creating this geological formation. The region is known for its difficult ground and water conditions, with a significant presence of marine and alluvial deposits represented by sands and soft organic silts with very low strength and deformation parameters [2].

Fig. 3. Bird view of the new quay wall and container storage yards (acc. DCT Gdansk S.A.)

The geotechnical part of the design concerning the new berth and the adjacent container storage yards was divided into two major parts: the foundation of the Ship-To-Shore (STS) gantry crane beam, and deep ground improvement of the platform area, quay wall area (45 m landwards from the seaside crane rail) and of the transition zone between both areas (Fig. 4). A significant portion of the works comprised a reclaimed area of an existing basin, with a backfill depth of 3 to 14 m. and therefore represented a challenging geotechnical task.

In the **platform area** the aim of the soil improvement was to compact loose fill to even the settlements and ensure sufficient stiffness to the pavement structure. The function of the improved upper layer was to distribute the loads and transmit them uniformly to a deeper layer of silt which governed total settlements. In the **transition zone**, soil improvement elements were adopted in variable grids and lengths to ensure a smooth transition within the range of allowable settlements. In all cases the adopted geotechnical solutions were tailored to local soil profiles, loading conditions and functional requirements. In the most crucial zone, the **quay wall area**, ground improvement aimed not only to reduce the settlements of the pavement under the surcharge load, but also reduced the earth pressure acting on the quay wall structure.

This paper focuses on the detailed design process of the gantry crane beam foundation.

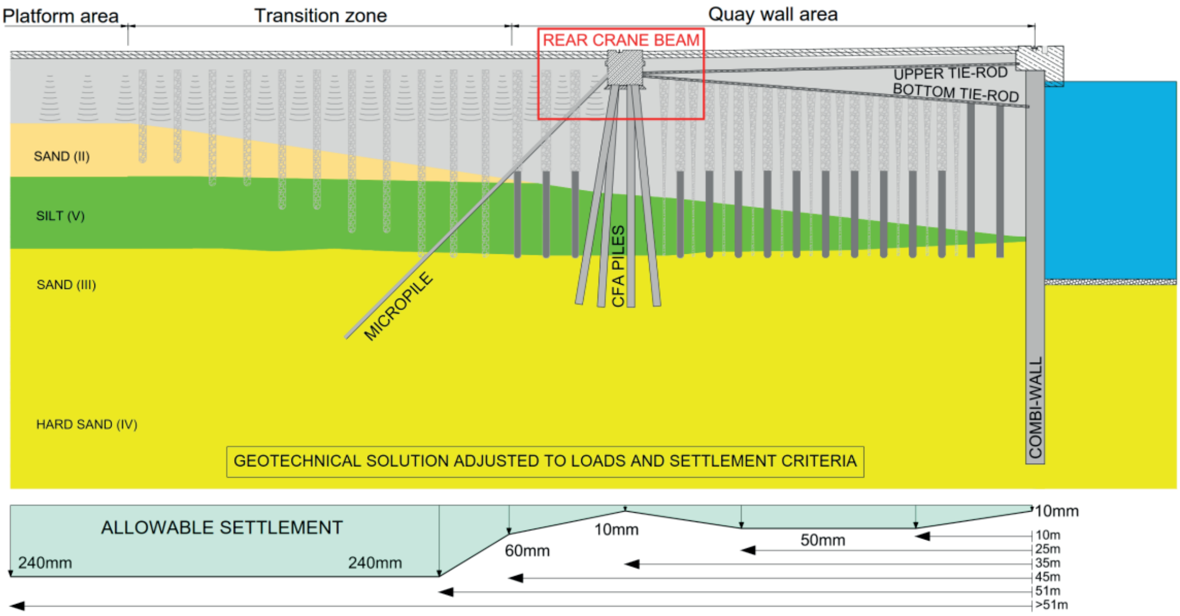

Fig. 4. A typical cross-section of the quay wall in offshore part of the project

DESIGN PROCESS

The design process started with analysing the employer's requirements [7] and soil investigation. The quay wall was designed as a combi-wall steel pile structure, with front capping beam anchored by means of pair of tie-rods in the Rear Crane Beam (RCB), which was founded on a system of raked Continuous Flight Auger (CFA) piles and micropiles (Fig. 4 and 6). The 656 m long beam was divided into 27 sections and loaded with STS cranes with the following characteristics: rail centre-to-centre 35 m, corners 4, wheels per corner 8 (spacing 1000 mm) [7]. In addition, basic crane loads defined in the employer's requirements had be factored by 1.5 (g_e) to allow for possible future increase in equipment specification. Moreover, this increase did not include partial safety factors which should be used in the design. Consequently, for the final design crane loads had to be increased appropriately, as indicated in Tab. 1.

Tab. 1. Crane loads

	Vertical load [kN/m]		Horizontal load [kN/m]
load	**dead**	**live**	**live**
basic	564	376	141
characteristic[1]	846	564	212
design[2, 3]	1396	931	318

Notes: ¹ Incl. employer's factor for future load increase (g_e=1.5); ² Incl. partial safety factors for actions: γ_G=1.35 for permanent unfavorable actions, and g_Q=1.50 for variable unfavorable actions; ³ Incl. the reliability class factor g_n=1.1, applicable for vertical loads according to [14] only.

For geotechnical analyses two most representative soil profiles were selected, and used to design the rear crane beam and its supporting elements. The analysis of the beam was done considering six positions of the crane, identified as the most critical (Fig. 5).

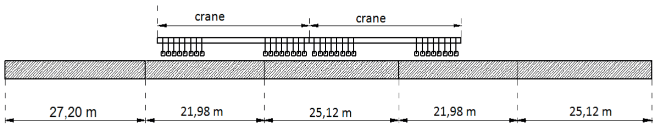

Fig. 5. Example of tandem crane loads acting on the RCB consisting of five sections

The behavior of the structure was analyzed in a linear-elastic as well as a non-linear range using FEM [3], and inspecting the convergence of both analysis. Two independent FEM models were used to investigate the performance of gantry crane beam and foundation elements. The first model, created with Plaxis 3D software, aimed to represent the behavior of a complete quay wall structure taking into account sea actions. The second one, created with Robot Structural Analysis software, focused on a proper modelling of the isolated RCB and its elements (Fig. 6). In this case the forces acting on the front capping beam due to dredging works, waves, mooring forces, surcharge loads, crane operations, temperature fluctuations, tensioning forces, etc. had to be transferred to the rear crane beam through the upper and bottom tie-rods (Fig. 7). In the course of a multi-stage analysis possible failures modes of the anchoring system were also analyzed considering various accidental combinations. Finally, the maximum anchoring forces were determined and used in the analytical RCB model.

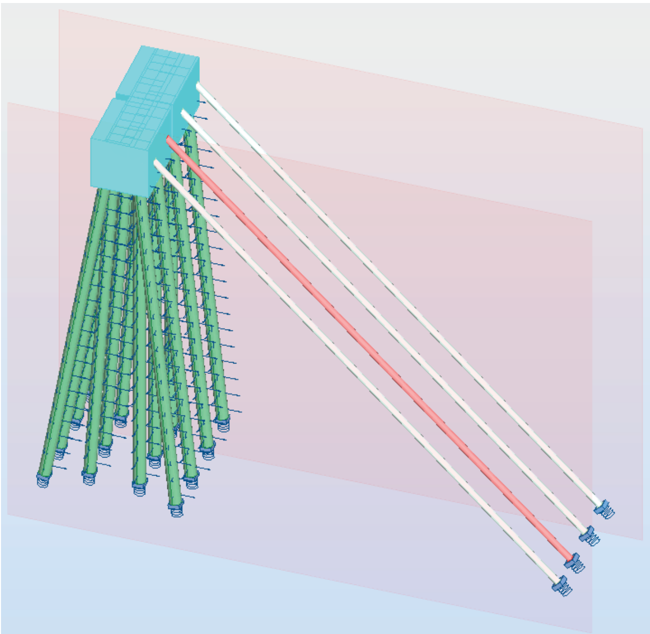

Fig. 6. FEM model of the RCB

For FEM modelling of CFA piles elastic beam elements were used together with interface elements at the pile to soil boundaries. Because of a special hinged connection between micropiles and the RCB, the micropiles were modelled by means of string elements capable of transferring tension forces only (Fig. 7).

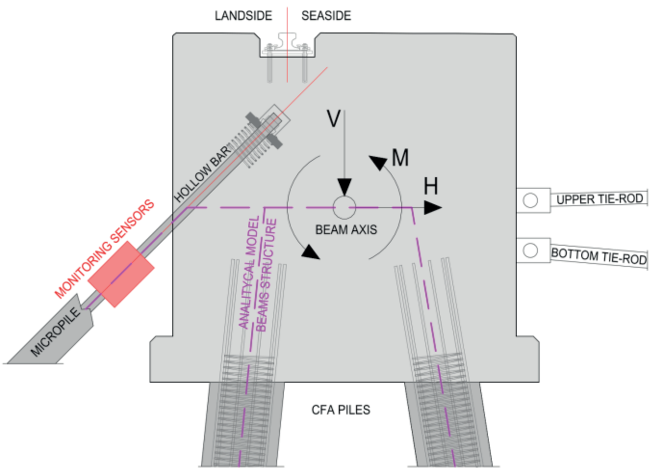

Fig. 7. Cross-section of the RCB showing CFA piles and the tie-rod (V – vertical force, H – horizontal force, M – bending moment). The red section indicates the location of monitoring sensors.

Also sensitivity analyses were conducted to check possible system failure caused by micropile defect, assuming a missing element in most unfavorable locations. The results of the sensitivity analysis for accidental combinations did not govern the design of micropiles as normal forces were 30-35% lower than in the original model. The RCB analytical models were also checked for reduced horizontal stiffness of springs representing soft soil layers (organic silt) or even without soil improvement elements in the quay wall area. All these results showed insignificant increase of internal forces in the micropiles in comparison to Ultimate Limit State (ULS) results. This led to a conclusion that the resistance of RCB to lateral deflection is governed by the stiffness of the upper sand layers, which was also proved by FEM analysis using Plaxis 3D.

The simulations performed taking into account the most unfavorable sections of the RCB showed uniform reaction of the designed foundation elements. In the ULS condition, the predicted axial forces in CFA piles and micropiles due to the action of design loads were about 2180 to 2275 kN and 1750 to 1950 kN, respectively. For the micropiles this range of forces can be compared with the internal bearing capacity of 2670 kN of the hollow bar element type T103S.

As for the observational method and EC7 recommendations, the geotechnical design should be verified on real-scale elements on site and prior to the construction works to validate the effectiveness of the solution adopted. Consequently, a detailed plan of preliminary field loading tests was elaborated and executed to reduce the design risk to a minimum.

PRELIMINARY FIELD TESTS

The rear crane beam was to be supported on racked CFA piles up to 29 m long, with a diameter of 650 mm and the inclination angle of 9.5°. Field tests were performed with two representative CFA piles 20.5 and 29.0 m long (C1 and C2, Fig. 8). The aim of tests was to verify the preliminary design predictions and to determine acceptance criteria for the production piles. The load test set-up and the loading procedure adopted aimed at reaching the ultimate load corresponding to pile head settlement of 10% of the pile base diameter (i.e. $s_{min} > 65$ mm), in line with EC7 [6].

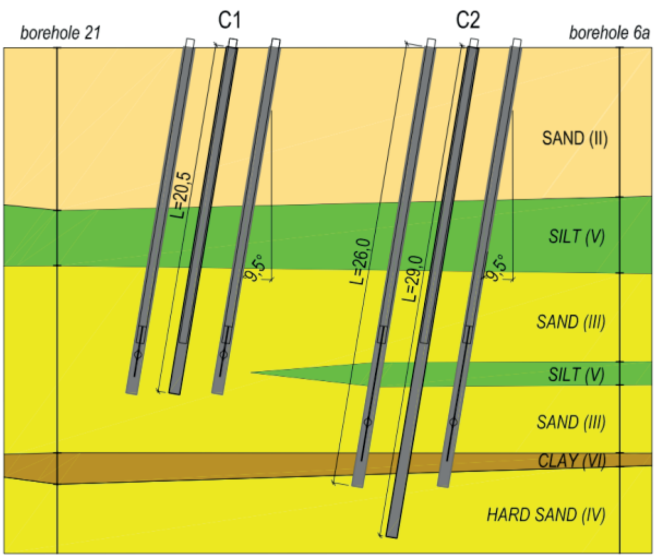

Fig. 8. Scheme of CFA piles used for two loading tests

The load-settlement curves obtained from field tests enabled determination of the bearing capacity by means of the bisector method [8] and evaluation of pile stiffness in the working load range (Fig. 9). The particular shape of these curves made it possible to model the piles precisely in the calculation simulations.

Results of the load tests confirmed the assumed bearing capacity of CFA piles. For the shorter pile C1 the ultimate bearing capacity was 4500 kN, leading to allowable design load of 3000 kN per pile with account for the negative skin friction effect. The longer pile C2 achieved 7000 kN bearing capacity during the test, resulting in analogous design load of 4500 kN per pile. It should be noted that both tested piles remained stable up to the last loading step.

Test piles proved to be safe enough to support the design load of 2275 kN, estimated from the most conservative FEM model. The corresponding pile stiffness under the design load was 238 to 325 MN/m, which was in line with the prediction of the preliminary design. Based on the test results the RCB model was updated, and revised design analyses were conducted. Then, it was decided to commence the production CFA piles. For quality assurance it was planned to execute 9 additional post-production static loading tests on selected piles. In case of unsatisfactory results of control tests a contingency plan to install additional piles was prepared. However, all control tests reached the required axial pile stiffness, being in average 424 MN/m. It has been estimated that the adopted program of preliminary field tests and proper quality control procedures enabled saving of approximately 4000 lm of CFA piles without compromising safety.

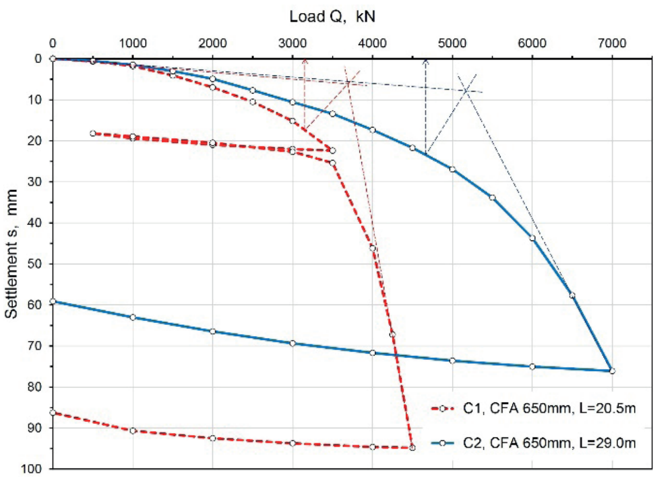

Fig. 9. Static load tests of C1 and C2 testing piles

The rear crane beam was also designed to be anchored by means of self-drilling T103S hollow bar system micropiles with a diameter of 300 mm, inclined at 45° degrees and up to 36 m long. Field tests were performed on five micropiles (M1 to M5). The aim of the tests was to verify the preliminary design assumptions, determine unit skin friction in isolated sand layers and check the ultimate bearing capacity of full-length micropiles (Fig. 10), limited by tensile strength of the hollow bar system of 3550 kN, according to product specification.

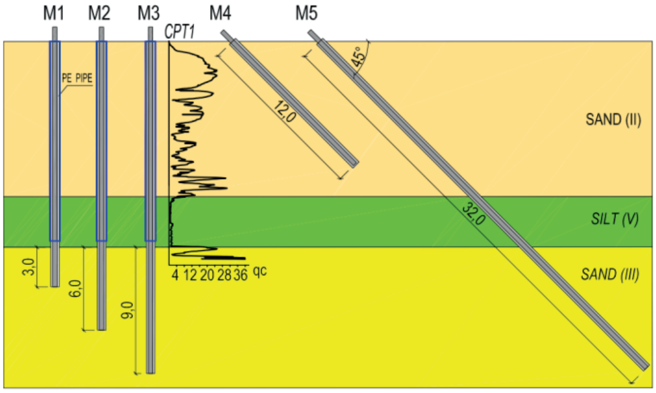

Fig. 10. Schematic diagram of tested micropiles with soil profile

Field tests performed on vertical micropiles (M1-M3) enabled assessment of the ultimate skin friction (τ_{ult}) in the bottom sands (stratum III). The hollow bars were isolated with PE pipes to create a free length, resulting in zero friction in the upper layers.

Load test done on inclined micropile (M4) enabled assessment of the ultimate bearing capacity in the upper sands (stratum II). The full-length micropile (M5) was used to determine the axial stiffness that could be used in the RCB analytical model. This test was carried out above the yield pull-out force of the bar of 2680 kN, and reached the load of about 3400 kN.

Tab. 2. Results of micropile tests

Micropile	$R_{t;k}$ [kN]	L_r [m]	Q_{test} [kN]	τ_{ult} [kPa]
M1	890	3,0[a]	1260	446
M2	1780	6,0[a]	2340	414
M3	2670	9,0[a]	2900	342
M4	1800	12,0[b]	1700	150
M5	>4000	32,0	~3400	-

where: $R_{t;k}$ – calculated ultimate load; L_r – length of grouted body ([a] in the bottom sands; [b] in the upper sands); Q_{test} – maximum load during test; τ_{ult} – ultimate skin friction.

For micropiles M1-M3 it can be noticed that the ultimate skin friction for 3.0 m of grouted body was significantly higher than for 6.0 m and 9.0 m, in particular (Tab. 2). Most likely, the difference was caused by a high variability of sand compaction within sands. For the final design conservative values of 342 kPa and 150 kPa were used for the bottom and upper sands, respectively.

The load-uplift curves obtained from the field tests for M4 and M5 micropiles are presented in Figure 11. During the

M5 test, yield strength of the bar was reached. Taking into account creep criteria (k_s < 2,0 mm) determined in codes [4], [5], next-to-last load step represent the bearing capacity of tested micropiles. The resulting axial stiffness of the full-length micropile was approximately 160 MN/m, and was used in the final design calculations.

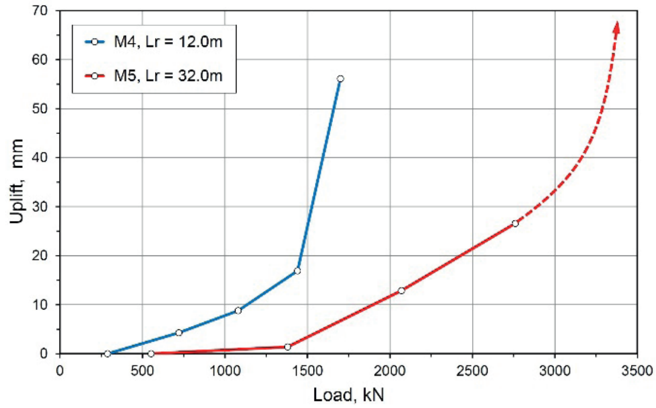

Fig. 11. The results of static load tests of M4 and M5 micropiles, corrected for bar elongation

A computational back analysis of the most unfavourable sections of the RCB revealed a uniform loading of the micropiles. Test micropiles proved to be safe enough to carry on the design load of 1950 kN, estimated from the most conservative FEM model. In addition, the field tests help to specify the QA criteria.

Finally, it was decided to commence production micropiles. For quality assurance it was planned to conduct 6 additional post-production static load tests on micropiles. Similarly to CFA piles, a contingency plan to install additional micropiles was prepared. However, all tests reached the expected axial stiffness of production micropiles of 165 MN/m, in average.

TECHNICAL MONITORING

Because of a complex character of the design and works carried out to construct the quay wall it was decided to verify the implemented design solution of rear crane beam foundations by applying an innovative monitoring system, installed on the hollow bars of 7 micropiles spreaded along the beam. The location of monitoring sensors installed on a micropile is highlighted in red in Figure 7. The development and set-up of the monitoring system, calibration of the sensors in laboratory and the assembly of the equipment on site has been described in detail by Miśkiewicz et al. [10].

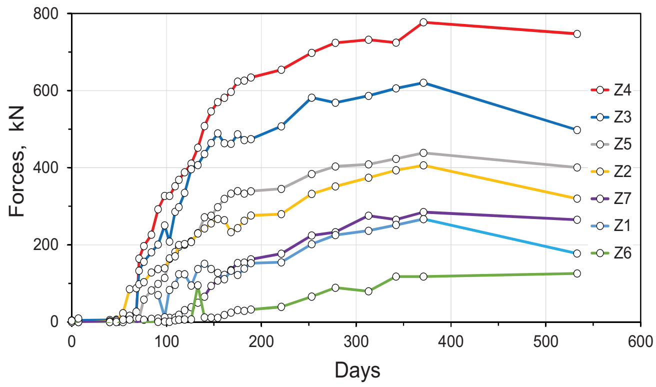

Fig. 12. Tensile forces in the micropiles

The observations on site continued in varying weather conditions for over 500-days (Fig. 12). The results obtained confirmed that the actual tensile forces in the micropiles are on the safe side. In the most loaded bar, Z4, the maximum measured force was F_{meas}=777 kN. For comparison, the calculated unfactored tensile force in a representative cross-section and for the corresponding stage of quay wall construction was $F_{calc,k}$=807 kN, indicating a ratio of $F_{meas}/F_{calc,k}$=0.96. Consequently, the monitoring results confirmed high accuracy of design calculations.

The grand opening ceremony of terminal T2 took place on the 24th of October 2016, corresponding to 250 day in Figure 12. Since then a small relaxation of tensile forces has been noticed. At present, the monitoring system is still operational, and the long-term data are collected for control and further analyses.

CONCLUSIONS

The presented case study illustrates the design practice based on Eurocodes, with special emphasis on the observational method. Nonetheless, in the authors' opinion, EC7 is an extensive general document, which is lacking detailed implementation rules. Currently, the next version of more "easy-to-use" Eurocode 7 is in preparation, and should be introduced in 2020.

In the end a designer is responsible for the accuracy of the applied solution and has to account for potential risk. This is why, before choosing any geotechnical solution, a geo-engineer has to consider a variety of components: applicability of certain technology and its limits, type of structure, type of applied loads, structure sensitivity to settlements and ground conditions. It is also highly recommended that field tests should be performed prior to commencement of works, to set appropriate QA/QC procedures and monitor 'real life of structure' in order to verify implemented solutions and maintain a high quality of work and reduce potential risk. The applied solution also needs to fit into the construction schedule, and should be economically attractive. As a result, geotechnical engineering has to face many challenging demands.

The reported case deals only with a part of comprehensive ground engineering works that were implemented at the DCT site. The focus is on the geotechnical design, testing and monitoring of the rear crane beam foundation system and its vital elements. It has been shown that well-planned full-scale preliminary tests and observations allow not only to optimise construction costs, but also significantly help to mitigate design and executional risks. Furthermore, the demonstrated case is another perfect example of a successful co-operation [9], [11] between academia and practitioners to deliver a high quality engineering product that produces savings in costs and programme without compromising safety.

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