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Preliminary field tests as a method of geotechnical design risk mitigation

Essais preliminaires comme méthode de mitigation des risques pour la conception geotechnique

O. Mitrosz
Technical Office, Keller Polska Sp. z o.o., Gdansk, Poland

T. Brzozowski, R Buca
Technical Office, Keller Polska Sp. z o.o., Gdansk, Poland

ABSTRACT: Appropriate risk assessment plays a fundamental role in the geotechnical design. The authors describe recent design approaches recommended in Eurocode 7. The presented methods of design risk mitigation by means of preliminary field tests and interactive design produce savings in schedules and costs on engineering projects without compromising safety. The paper presents a case study of a complex geotechnical solution that deals with the heavy foundation of a gantry crane beam based on excessive preliminary field tests. The authors briefly describe a process of the design and construction, and conclude technical monitoring results of field trials tested prior to the design procedure. Finally, in the paper, the importance of appropriate design risk assessment and its mitigation is highlighted, and both preliminary field tests and interactive design (observational method) using latest developments and innovations in geotechnical engineering are recommended.

RÉSUMÉ: Une analyse des risques appropriée joue un rôle fondamental dans la conception géotechnique. Dans cet article, les auteurs décrivent les dernières approches de conception comme décrites dans l'Eurocode 7. Les méthodes de mitigation des risques liés à la conception par le biais d'essais préliminaires, offrent des économies du point de vue des couts et des délais tout en ne négligeant pas la sécurité. L'article présente l'étude de cas d'une solution complexe, proposée pour les fondations profondes d'une grue sur portique et basée sur une importante campagne d'essais préliminaires. Les auteurs expliquent brièvement le processus de conception et d'exécution avant de donner leurs conclusions sur les résultats des mesures effectuées lors des essais ayant précédés la phase de conception. Enfin, l'accent est mis sur l'importance d'une analyse appropriée des risques de conception et de leur mitigation et des recommandations sont faites sur les derniers développement et innovations en matière d'essais préliminaires et de méthodes de conception interactives (méthode observationnel).

Keywords: field test; technical monitoring; observational method; design risk; geotechnics;

1 INTRODUCTION

Eurocode 7 (*EC7*) 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*).

The EC7 introduces geotechnical categories of engineering structures from 1 to 3, from relatively simple structures to structures involving abnormal risks, unusual or exceptionally difficult ground and loading conditions. More often in geo-engineering the risk matter and its appropriate management are now being considered.

In 2002, a European geotechnical forum was set up for the exchange of best practice ideas and innovations in geotechnical engineering. They published a document promoting modern design tools, including the application of the finite element method (FEM) and the OM, which can reduce costs and programmes on engineering projects without compromising safety. This proves how the geotechnical community can benefit from developing scientific knowledge.

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

1.1 Observational method

In a recent work on the OM (Nicholson et al. 1999), published by the Construction Industry Research and Information Association's Report 185 (CIRIA), the definition of the 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, however, are usually based on a single, robust design and there is often no intention of varying the design during the construction phase. Technical 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, designer, contractor). In comparison, in the OM monitoring plays an active role in both design and build, allowing planned modifications to be carried out within a contractual framework.

The enhancement of the OM is also described in the EC7, 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*'. The code sets some general rules for the method that are required before starting construction. However, as stated by Patel et al. (2007), EC7 is inconsistent and is lacking in any detailed instructions the geo-engineers shall follow. Moreover, the code doesn't concentrate on the advantages the OM can bring to a typical construction processes, but recommends the method as one of the optional, alternative approaches to design. A different scientific approach is represented by the promoters of the OM, who prove on certain projects the effectiveness of appropriately implemented observational method. The method requires full consciousness of the construction process, 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 projects.

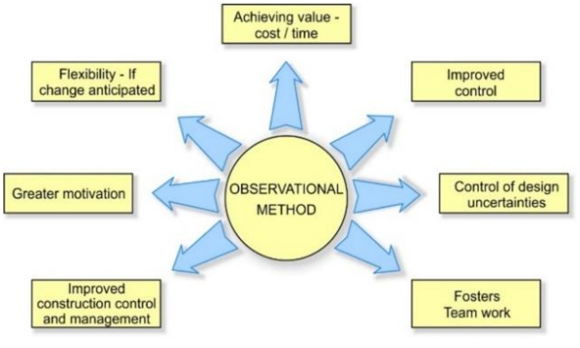

Figure 1. Benefits of the OM (Nicholson et al. 1999)

The latest and 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 (2016) describes the high accuracy of BIM application in geotechnics and advocates the use of Geo-BIM 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.

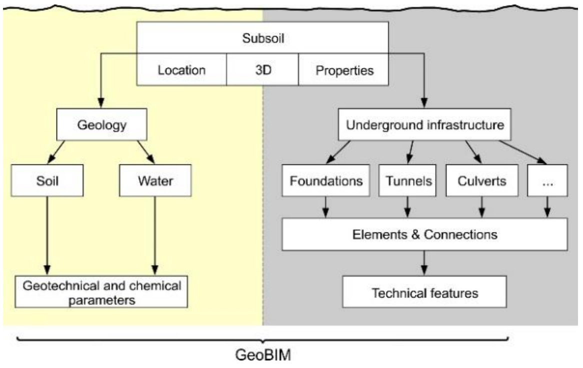

Figure 2. Geo-BIM model chart (Topolnicki 2016)

1.2 Design methods

The EC7 determines the following design approaches for pile foundations:

a) empirical or analytical **calculation methods**;
b) the results of **static load tests (SLT)**;
c) the results of dynamic load tests;
d) the observed performance of a comparable pile foundation.

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

Nonetheless, a tight construction schedule is often the main reason for not conducting the tests. There is no better and more reliable direct method of pile bearing capacity assessment than a load test performed on full-scale pile within the *in situ* conditions. Proper utilization of field test results in the design process minimizes risk and leads to safe and optimized foundation solutions. Consequently, well-planned field tests should be conducted in advance to production piles to allow early verification of the design in terms of real pile stiffness and ultimate bearing capacity. Therefore, the loading shall be such that the ultimate pile bearing capacity should be trustably assessed. This is often difficult to achieve for piles in compression when the load settlement plot show a flat, continuous curvature. In these cases, according to EC7, a limit settlement of the pile top equal to 10% of the pile base diameter should be adopted as the ‘failure‘ criterion.

2 A CASE STUDY CONCEPT

The presented case is an example of the implementation of a field test procedure 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 DCT extension. 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 Baltic Sea (Buca and Mitrosz 2016).

The DCT is located in the industrial part of the city of Gdansk, on the Vistula Spit which forms a natural barrier against sea intrusion. Soil sedimentation transported by the Vistula River created the 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 (Table 1).

Table 1. Generalized stratigraphy, strength and deformation parameters

Layer *(stratum)*	γ_{sat} (kN/m^3)	ϕ (°)	c (kPa)	E_{oed} (MPa)
FILL (*I*)	20	32	–	60
SAND (*II*)	19-20	30	–	42-74
Medium dense **SAND** (*III*)	20	35	–	80-110
Sandy **GRAVEL** (IV)	21	41	–	120-190
Soft organic **SILT** (*V*)	16	7	9	1.4-4.0
Sandy to silty **CLAY** (*VI*)	20-22	15	17	20-29

The geotechnical part of the design concerning the terminal extension was divided into two major parts: the foundation of the Ship-To-Shore (STS) gantry crane beam, and deep ground improvement of the platform, quay wall area (45 m landwards from the seaside crane rail) and of the transition zone between both areas (Figure 3). A significant part of the works comprised a reclaimed area of an existing basin, with a backfill depth of up to 14 m and therefore represented a challenging geotechnical task. Soil improvement solution in the platform area has been widely discussed by Mitrosz et al. (2017). Therefore, this paper focuses on the detailed design process of the crane beam foundation.

3 DESIGN PROCESS

The design process commenced with analysing the client's requirements and soil investigation. The quay wall was designed as a combi-wall steel pile structure, with a front capping beam anchored by means of paired tie-rods in the rear crane beam (*RCB*), which was founded on a system of raked continuous flight auger (*CFA*) piles and micropiles (Figure 3 and 6). The 656 m long beam was divided into 27 sections and loaded with STS cranes with the following characteristics: rail ctc spacing 35 m, 4 corners, 8 wheels per corner (spacing 1000 mm).

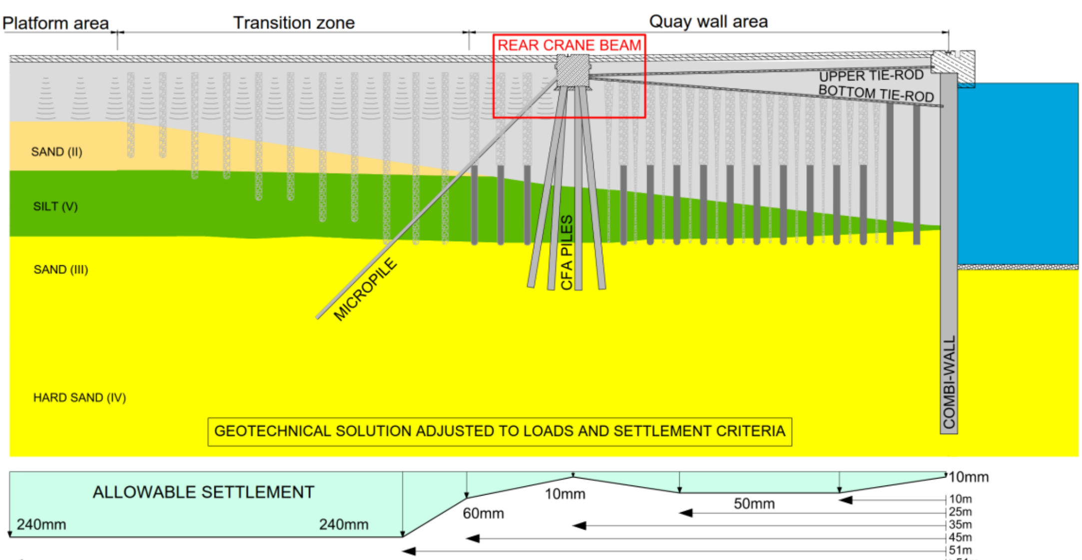

Figure 3. A typical cross-section of the quay wall in offshore part of the project

A defined crane loads included partial safety factors and had be factored to allow for future increase in equipment specification (Table 2).

Table 2. STS crane loads

	Vertical load (kN/m)		Horizontal loads (kN/m)
Load	dead	live	live
basic	564	376	141
unfactored [1]	846	564	212
factored [2, 3]	1396	931	318

Where [1] includes client's factor for future load increase ($\gamma_c = 1{,}50$), [2] includes partial safety factors for unfavorable actions: $\gamma_G = 1{,}35$ for permanent and $\gamma_Q = 1{,}50$ for variable actions, [3] includes additional reliability class factor $\gamma_o = 1{,}10$, applicable for vertical loads only after National Ordinance.

For the purpose of geotechnical analysis two most representative soil profiles were selected, and used for the RCB foundation. The calculation of the beam was done considering 6 most critical positions of the cranes in tandem. The behavior of the structure was analyzed in a linear-elastic as well as a non-linear range using FEM, and inspecting the convergence of both analysis. Two independent FEM models were investigating the performance of crane beam and foundation elements. The first model, created with Plaxis 3D environment (Figure 5), aimed to represent the global behavior of a complete quay structure taking into account sea actions. Generally, a Hardening Soil model was used apart from organic silts where Soft Soil model was assumed to be more appropriate. The second model, created in Autodesk Robot Structural Analysis software, focused on the isolated RCB. In this case the loads acting on the front capping beam due to dredging, wave actions, mooring, etc. had to be transferred to the RCB through the upper and bottom tie-rods (Figure 6). Finally, the maximum anchoring forces were determined and used in the further analytical RCB model.

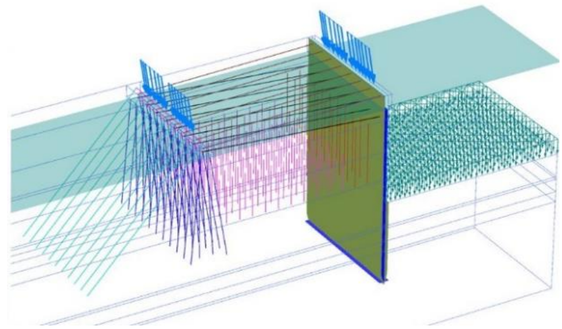

Figure 5. FEM Plaxis model of the quay wall structure with the rear crane beam and foundation elements

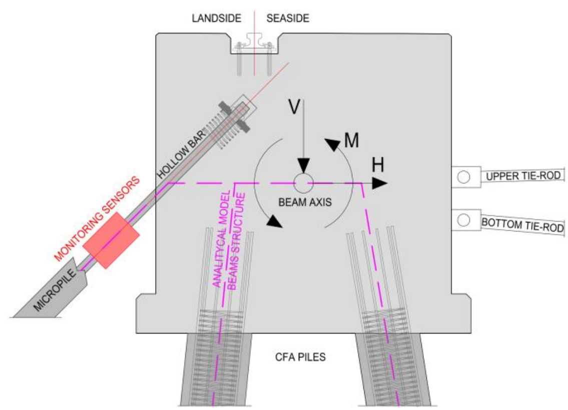

Figure 6. Cross-section of the RCB with monitoring sensors highlighted in red

For FEM modelling of CFA piles elastic beam elements were used with interface elements simulating soil-pile interaction. 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 (Figure 6). Due to a multi-stage analysis, possible failure modes of the anchoring system (micropile defect) were also analyzed considering various accidental combinations. However, the results of that sensitivity analysis for did not govern the design of micropiles as axial 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). All these results showed insignificant increase of internal forces in the system in comparison to ultimate limit state (ULS) results. This led to a conclusion that the resistance of RCB to lateral deflection is mostly governed by the stiffness of the upper sand layer, which was also proved by FEM analysis using Plaxis. In the ULS, the predicted axial forces in CFA piles and micropiles due to the action of design (factored) loads were about 2180 to **2275 kN** and 1750 to **1950 kN**, respectively. For the micropiles the estimated value shall be compared with the structural capacity of the bar, 2670 kN.

As for the OM 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. Consequently, a detailed plan of preliminary field tests was put through to reduce the design risk to a minimum.

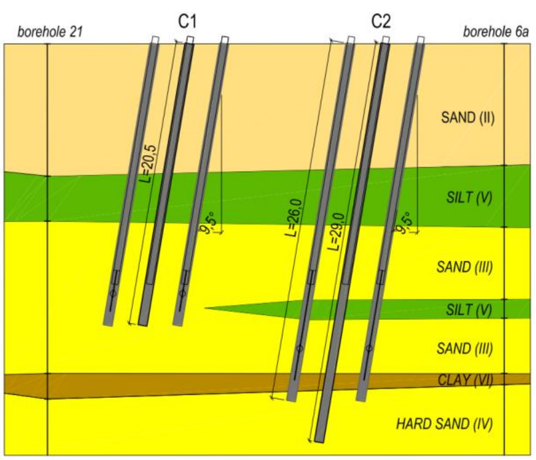

Figure 7. Scheme of CFA piles used for load tests

4 PRELIMINARY FIELD TESTS

The RCB was to be supported on raked CFA piles 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, Figure 7). 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 aimed to reach the 'failure' criterion (eg. $s_{min} > 65mm$).

The load-settlement curves obtained from field tests enabled determination of the bearing capacity by means of the bisector method and evaluation of real pile stiffness in the working load range (Figure 8). The particular shape of these curves made it possible to model the piles precisely in the calculation simulations. The ultimate bearing capacity was 4500 kN and 7000 kN, leading to allowable design load of 3150 kN and 4650 kN, including the negative skin friction effect, respectively for pile C1 and C2. It should be noted that both tested piles remained stable up to the last loading step.

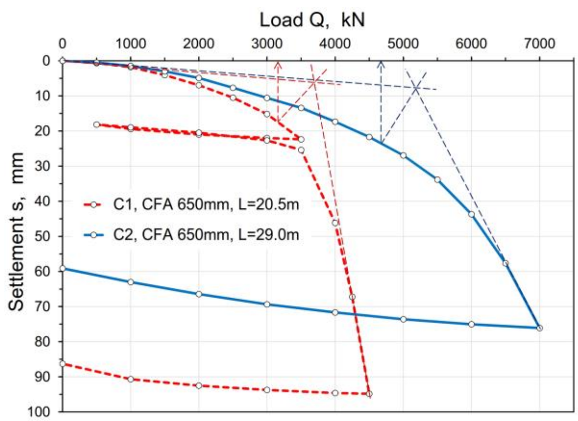

Figure 8. Static load tests of C1 and C2 testing piles

Thus, test piles proved to be safe enough to carry on the extreme design load of 2275 kN. The corresponding pile stiffness under the design load was from 238 to 325 MN/m, which was in line with the preliminary prediction. Based on these results, the RCB model was updated. Then, it was decided to commence the production CFA piles. As a quality assurance it was planned to execute 9 additional post-production SLT. In case of unsatisfactory results of control tests, a contingency plan to install additional piles was prepared. However, all control tests reached the required pile stiffness, average 424 MN/m. Finally, it has been estimated that the adopted program of preliminary tests and proper quality control procedures enabled saving of 4000 lm of CFA piles without compromising safety.

The RCB was also designed to be anchored by means of self-drilling T103S hollow bar system micropiles with a grouted body diameter of 300 mm and the inclination angle of 45°. Field tests were performed on five micropiles (M1 to M5, Figure 9). The aim of these tests was to verify the pre-design assumptions and check the ultimate bearing capacities of micropiles limited by tensile strength of the bar, 3550 kN. Field tests performed on micropiles M1-M3 enabled assessment of the ultimate skin friction in the bottom sands. The hollow bars were isolated with PE pipes to create a free length, resulting in zero friction in the upper layers. Test on micropile M4 enabled assessment of the ultimate bearing capacity in the upper sands, whereas, the full-length micropile M5 was used to determine the global axial stiffness that could be used in the RCB analytical model.

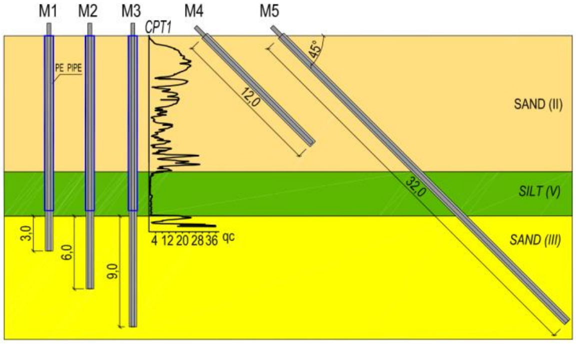

Figure 9. Scheme of tested micropiles

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 (Table 3). Most likely, the difference was caused by a high variability of relative density of sands. For the final design conservative values of 342 kPa and 150 kPa were adopted for the bottom and upper sands, respectively. The resulting axial stiffness of the micropile M5 was ~160 MN/m, and was used in the final design modelling. Thus, test micropiles proved to be safe enough to carry on the extreme design load of 1950 kN. Finally, it was decided to commence production micropiles. As a quality assurance it was planned to conduct 6 additional post-production SLT on micropiles. Again a contingency plan to install additional micropiles was prepared. However, all tests reached the expected axial stiffness of 165 MN/m, in average.

Table 3. 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 $R_{t,k}$ (kN) is a calculted ultimate capacity, L_r (m) is a length of grouted body (^{a} in the bottom sands, ^{b} in the upper sands), Q_{test} (kN) is a maximum load during the test, τ_{ult} (kPa) is an ultimate skin friction.

5 TECHNICAL MONITORING

Because of a complex character of the design and build process carried out to construct the quay wall it was decided to verify the implemented geotechnical solution of the RCB foundations by an innovative monitoring sensors, installed on the hollow bars of 7 micropiles (Figure 6). The whole monitoring system has been described in detail by Miśkiewicz et al. (2017). The observations on site continued in varying weather conditions for over 500 days (Figure 10). The results obtained confirmed that the actual tensile forces are on the safe side. In the most loaded bar (Z4) the maximum measured force was $F_{meas} = 777\ kN$, whereas, the calculated unfactored tensile force for the corresponding stage of quay construction was $F_{meas} = 807\ kN$. Consequently, the monitoring results confirmed high accuracy of the design calculations. At present, the monitoring is still operational, and the long-term data is collected for control and further analysis.

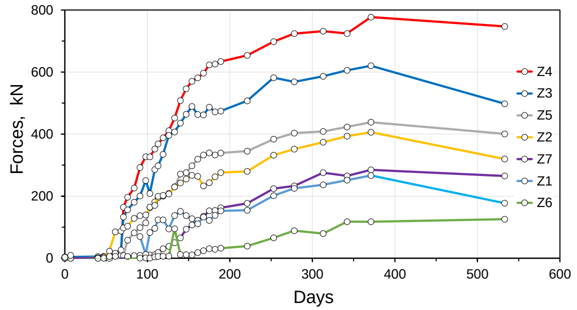

Figure 10. Tensile forces in the monitored micropiles

6 CONCLUSIONS

The presented case study illustrates the design practice based on EC7, with special emphasis on the observational method or interactive design. Nonetheless, in the authors' opinion, EC7 is an extensive general document, which is lacking precise implementation rules. Currently, the next generation 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 QC/QA 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 the construction timetable, and should be economically viable. As a result, geotechnical engineering has to face many challenging demands.

The reported case deals only with a part of complex ground engineering project that was 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 field 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 a perfect example of a successful co-operation 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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