Complex Geotechnical Engineering for Port of Gdansk Development – Gateway to Central-Eastern Europe

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Abstract. Port of Gdansk development strategy 2027 is to become leading European hub in the Baltic Sea and assumes i.a. improvement of inland road and rail accessibility, increase of the port capacity for container throughput and intermodal transport, increase port's throughput capacity for goods other than containerized cargo. The paper presents several complex geotechnical design & build tasks that have already been done for port's infrastructure and super-structure development and extension i.a. road tunnel under Vistula river linking port with city industrial area or Deep-water Container Terminal Gdansk with a new berth and container stacking yards. The paper discusses multiple geotechnical challenges and alternative soil treatment methods. The authors share conclusions and recommendations of successful application of various ground improvement technologies.

Keywords: diaphragm wall, vibro concrete column, vibro compaction, impulse compaction, stone column, CSC, column-supported embankment.

Conference topic: Case studies, transport infrastructure.

Introduction

The geographical location is the major determinant of development projects and growth opportunities of every port. Port of Gdansk (Fig. 1) is situated on the southern coastline of Baltic Sea enabling connection to the high seas through the Danish Straights. The economic location of the Port presents considerable advantages as the Baltic Region is the most rapidly growing part of Europe.

Fig. 1. General plan of Port of Gdansk
(source: ZMPG S.A., photo by Kacper Kowalski (2016))

The Port's development strategy 2027 is to become leading European hub in the Baltic Sea and assumes i.a. improvement of inland road and rail accessibility, increase of the port capacity for container throughput and intermodal transport, increase port's throughput capacity for goods other than containerized cargo. The overall consistent and coherent strategy is realised step-by-step what results in development and

increase of competitiveness of the whole region. This leads to new lands management and planning. The industrial part of Gdansk is known for its difficult ground water conditions with significant presence of marine and alluvial deposits developed in the form of sands and soft organic silts with very low strength and deformation parameters (see Table 1). Thus most of design and build geotechnical works are very challenging.

Table 1. Generalised stratigraphy, strength and deformation parameters

Layer	γ_{sat} [kN/m ³]	ϕ [°]	c [kPa]	E _{oed} [MPa]
Loose to medium dense SAND	19	30	–	42.0–95.0
Medium dense to dense SAND	20	35	–	80.0–110.0
Very soft SILT with organic content	16	7	9	1.4–4.0
Sandy to silty CLAY	20–22	15	17	20.0–29.0
Sandy GRAVEL	21	41	–	120.0–190.0

Inland road link

To provide a crossing beneath the Vistula river in Gdansk a new road tunnel was constructed linking sea port with city industrial zone, shortening thereby travel time to national road network. The project itself was divided into shield tunnel section (1,076 m) and two 40 m wide ramps, 340 m long on the eastern side and about 750 m long on the western side of the river. The

reinforced concrete ramps were designed as watertight U-shape and box type sections, consisting of a monolithic part constructed by connecting the foundation slabs and walls. The system was applied to the deeper excavations (above 6 m) and consisted of diaphragm walls embedded about 1.5 m into the low-permeable silty clay treated as natural clayey sealing. In case the low-permeable layer was not continuous or situated too shallow to the excavation level, which could potentially cause the loss of equilibrium of the excavation bottom (safety against heave), the slurry diaphragm walls and horizontal jet grouting (Soilcrete) slabs at larger depth were designed (see Fig. 2). High ground water uplift pressures were compensated by permanent micropiles acting in tension and capable of bearing compressive loads to support the ramp foundation slab (Topolnicki, Buca 2013, 2014).

For the excavations up to 20.5 m i.a. launching chamber the other sophisticated geotechnical solutions were adopted enabling the Tunnel Boring Machine (TBM) to commence the proper drilling (see Fig. 3). The following phases were implemented to begin construction of the launching chamber and protection of the excavation pit:

- Construction of 1.2 m thick slurry D-walls along the launching chamber, embedded 31.5 m below ground level (GL),
- Execution of anchored jet grouting slab in a single production process reaching depths up to 34 m below GL. After grouting process, the proper passive reinforcement was installed in fresh Soilcrete columns and disconnected 20 m below GL,
- Construction of TBM protection jet grouting block and slab in front of the launching chamber,
- Primary excavation to the depth of 4.5 m below GL and construction of a reinforced concrete top slab with wide openings enabling installation of TBM at chamber's bottom,
- Continuation of excavation to the depth of 12 m below GL and installation of temporary steel strutting structures followed by final excavation up to 20.5 m and construction of the foundation slab,
- Dismantling of steel strutting structures, construction of eyepiece at front wall and final preparation of the chamber for TBM assembly.

When the final excavation stage was reached the maximum movement of the side wall and front wall of the launching chamber were observed 22 mm and 11 mm, respectively (see Fig. 4). The difference of movements is due the existence of jet grouting block which in effective way reduced active pressure on front wall. Calculated in 3D FEM model (see Fig. 5) maximum movement of the side wall was 32 mm and the front wall 12 mm which was close to the measured values.

Fig. 2. Typical box type section of a tunnel ramp with diaphragm walls, micropiles and jet grouting slab

Fig. 3. Longitudinal section through launching chamber and TBM Soilcrete block behind the front diaphragm wall

Fig. 4. Measured deformations of the front wall (on the left) and side wall (on the right) of the launching chamber (Topolnicki *et al.* 2014)

Fig. 5. Modelling of the launching chamber in 3D FEM

The structure was calculated to resist maximum water level of +2.5 m Above Sea Level (ASL) during normal exploitation, representing extreme flood level (EFL). However, during construction stages two design water levels of +0.5 m and +1.5 m ASL were also taken into account in the analyses.

Sufficient buoyancy safety of the excavation bottom along tunnel ramps had to be fulfilled at all times. When analysing buoyancy safety, the two limit cases investigated included the bearing capacity of individual anchoring element and the global stability of the tunnel structure assuming that the anchoring elements form a uniform enclosed soil monolith together with the surrounding ground within their zone of influence. Global stability of the structure subjected to uplift forces caused by hydrostatic pressures was verified for temporary and permanent construction stages with different ground water levels. Temporary stages 1a and 1b, corresponding to buoyancy force acting on the bottom surface of jet grouting slab and on the foundation slab, respectively, were calculated for ground water levels at +0.5 m (see Fig. 6) and +1.5 m ASL (see Fig. 7). In addition to the dead weight of the structure, enclosed soil mass and soilcrete slab also the effective weight of the ground adjacent to the soilcrete columns were taken into account. Frictional resistance that developed between ground and outer side of D-walls was conservatively neglected ($T_s = 0$).

The buoyancy safety in the uplift limit state (UPL after EN 1997) was checked using the modified general formulas, respectively for temporary stage 1a and 1b:

$$V_e \cdot \gamma_{dst} \leq (G_e + 2 \cdot T_e + G'_b + 2 \cdot T_b) \cdot \gamma_{stb} \quad (1)$$

where: V_e – uplift force at the underside of jet grouting slab (representing destabilising actions); G_e – dead weight of soilcrete slab; T_e – friction force between soilcrete slab and D-walls; G'_b – dead weight of soilcrete columns and enclosed soil mass under water; T_b – mobilised friction force between D-walls and enclosed soil; $\gamma_{dst/stb}$ – partial safety factors for destabilising and stabilising permanent actions in UPL (see Table A15 in App. A of EN 1997: $\gamma_{dst} = 1.0$, $\gamma_{stb} = 0.9$):

Fig. 6. Global stability diagram for launching chamber – temporary stage 1a with uplift force acting at the bottom of Soilcrete slab (water level +0.5 m ASL)

Fig. 7. Global stability diagram for launching chamber – temporary stage 1b with uplift force acting at the bottom of concrete slab (water level +1.5 m ASL)

$$V_p \cdot \gamma_{dst} \leq (G_p + G'_e + G'_b + 2 \cdot G'_s) \cdot \gamma_{stb} \quad (2)$$

where: V_p – uplift force at the underside of concrete slab (representing destabilising actions); G_p – dead weight of concrete slab; G'_e – dead weight of soilcrete slab under water; G'_b – dead weight of soilcrete columns and enclosed soil mass under water; G'_s – dead weight of D-walls under water.

It was found that global stability, with temporary stages governing the design, was decisive for the determination of final grid and length of anchoring elements preventing uplift of the excavation bottom. The complete design was cross-checked using several numerical analyses and the anticipated displacements were in line with the predictions. Special attention was paid to monitor the impact and influence of work sequencing with different water levels at each construction stage. Quality control was also a prime focus and most of techniques were tested on site prior to commencement of construction. The project involved construction of deep and large excavations for tunnel ramps and two TBM chambers as well as working with high ground water levels and challenging ground conditions. What is more, the adopted geotechnical design solution had to meet Contractor's requirements to move from working underwa-

ter to a dry operation that significantly shortened construction time and reduced costs.

Rail link

The quality of rail infrastructure in Poland is of key importance. Therefore, parts of the national railway network are being upgraded in recent years. As a part of improvement port's rail accessibility, opening the way to the major railway lines double-track railway no. 226 modernization project took place.

The aim of Column-Supported Embankment (CSE) was to increase global stability and reduce settlement of 11 m high railway embankment on soft soils (see Fig. 8). There were two principal reasons to use CSE (Sloan 2011): accelerated construction compared to more conventional construction methods (i.e. prefabricated vertical drains) and protection of adjacent running track from excessive settlements due to double-track widening. On top of the columns a Load Transfer Platform (LTP) was constructed to help transfer the load from the embankment to the columns and also prevent a "bearing capacity" type of failure above the columns.

Oil terminal

The brand new Oil Terminal in Gdańsk is the first complete marine hub on the Polish coast which is strategic for energy safety in terms of oil deliveries. The terminal is provided with six large reservoirs of 62.5 thousand cubic metres each (total capacity of 375,000 m³), external and internal pipelines for oil and complete infrastructure (see Fig. 9).

Foundations of tanks were analysed on Controlled Stiffness Columns (CSC) based on Rigid Inclusion philosophy (ASIRI 2012). To control the improved soil stiffness and properly assess settlements concrete columns with LTP were designed under foundation slabs, whereas for rigid ring foundations the columns were reinforced and fixed to the capping beam. The aim of implemented soil improvement solution was to reduce the settlements up to 100 mm and limit differential settlements up to 50 mm. The analysis run in Plaxis for different construction phases proved the calculated values were below the allowable (see Fig. 10).

Fig. 8. Cross section of Column-Supported Embankment

Fig. 9. Crude oil terminal in Gdansk under construction (source: Ministerstwo Skarbu Państwa)

Fig. 10. Results of settlement calculations

Development of the PERN Oil Terminal opens up huge potential for Polish refineries and operators of oil and makes Gdansk important component of the crude oil grid connecting the Baltic Sea with the Mediterranean Sea.

Deepwater container terminal

The increase of Port's capacity for container throughput is planned to be realised by expansion of Deepwater Container Terminal Gdansk that will allow the terminal to meet growing demand for deep-sea services in Central-Eastern Europe (see Fig. 11). The new 650 m long quay will increase annual handling capacity to up to 3 million TEU (Twenty-foot Equivalent Units) in the first stage of construction. New berth at DCT will be able to handle Ultra Large Container Vessels of capacity exceeding 18,000 TEU.

Geotechnical part of design and build process concerning new berth and adjacent container stacking yards, the 25 ha contract area was divided into two major parts: heavy foundation of STS (Ship-to-shore) gantry crane beam and deep soil improvement of certain areas i.e. platform (container stacking yards), quay wall (45 m lanwards from the seaside crane rail) and transition zone connecting both areas. The quay wall area (combi-wall structure: tubular steel piles 1676/25 mm, etc 3.145 m and steel profiles AZ 24-700) consisted of a temporary bund the-so-called the "onshore" part and

part containing significant reclamation of the existing basin called the “offshore” part.

In the platform area the aim of soil improvement was to improve loose hydraulic fill, to even the settlements, to offer stiffness to the engineered fill for compaction and to distribute the uniform loads on the platform to the bottom silty layer, that governed the global settlements. Whereas in transition zone, soil improvement elements were adopted in variable grids and lengths to ensure smooth transition from the area where settlements up to 240 mm were allowed to the quay wall area where settlements were restricted to 60 mm (after 8 years). Thus the length of the zone was estimated to be at least 33 m (see Fig. 12).

Fig. 11. Visualisation of the new berth and container stacking yards (source: DCT Gdansk S.A. 2016)

Fig. 12. Vibrocats producing columns on DCT site

In all cases the geotechnical solution was adjusted to the soil profiles and its origin, loads (surcharge 40 kPa, stack height 5 containers 56 kPa) and required serviceability limits (allowable settlements 10 mm for crane foundations and from 50 mm to 240 mm for pavement structure). In the most crucial zone, quay wall area, ground improvement aimed not only to reduce the settlements of the pavement structure under the surcharge load of 40 kPa but also to reduce the earth pressure on the quay wall structure.

Fig. 13. Typical cross section in the offshore part with allowable settlements (after 8 years)

Thus in typical cross section deep soil improvement consisted of (see Fig. 13):

- Two rows of reinforced 0.6m CFA piles. Bored piles were introduced instead of full displacement technics to avoid additional horizontal stresses acting on front wall structure during installation process.
- Pre-treatment of hydraulic fill with Vibro Compaction (VC) and/or Impulse Compaction (IC) with an overlapping 10 m safety zone depending on the depth of the basin to assure required stiffness of this layer acting as transmission layer in applied soil improvement system. Such system requires load transfer platform to transfer the load to the column (ASIRI 2012). This needs to develop the “bearing capacity” of a column and displacement of soil surrounding the column. The end bearing capacity is achieved by proper penetration of the column into the bearing soil. As the upper part of the soft layer is subject to settlement, downdrag forces (negative skin friction) develop (see Fig. 14).
- Post-treatment with deep Stone Columns (SC) and/or Vibro Concrete Columns (VCC) in a shifted grid to VC. The geometry (diameter, grid) of stone columns was designed to provide replacement ratio above 12% in the silty layer (Priebe 1995, 2005). These elements caused vertical early drainage of silty layer underlying the hydraulic fill and speeded up the consolidation process, what was observed in significant early settlements of the platform. As the method is fully displacement technique, has a volumetric character and leads to a relative improvement of the strength parameters of the ground surrounding the columns, the behaviour of ground improved with stone columns was taken into account in Plaxis calculations as a lumped mass with improved parameters. The idea of using VCC was to “bridge” the silty layer and load applied at the ground surface after installation of columns. In fact, concrete columns did not improve the strength and deformation parameters of the soft layer, but they created an arching effect. Thus a large part of loading was shared between the columns and the load transfer layer “over” the silty layer which was then less sheared and less compressed. A safe estimation was 70% carried by the columns and 30% passing on soft layer, which limited stress increase in the weak layer. This phenomena was essential for the combi-wall structure to reduce the internal

forces. However, for this mechanism it was necessary to develop negative skin friction in the upper part of the columns just down to the neutral point (see Fig. 14). The applied calculations in Plaxis software accounted for this method allowing accurate soil-structure interaction.

- Leveling of the surface and heavy Roller Compaction (RC).

Prior to commencement of proper soil improvement works, according to contract requirements, full-scale trial test (see Fig. 15) had to be done to validate all designed technologies, contractors capabilities and technical solutions. Field trial programme assumed the execution of overload embankment as substitute weight for fill, pavement and surcharge load and long term observation of settlements on 6 settlement gauges.

The embankment was built in three phases layer by layer up to the required level (+5.7 m). The proper sequence of execution made it possible to assess and distinguish the real values of immediate settlements that will occur meanwhile the installation of engineered fill up to +2.0 m or after pavement and surcharge load. Taking into account quick stabilization of settlements it may be concluded that part of immediate settlements will occur in the construction phase. The measured total values of settlements were within range 30÷33 mm (see Fig. 16) and were below as well as the allowable settlements and the estimated values in 2D Plaxis calculations (38÷42 mm). The results of field trial were reported. After analysis, the effectiveness of implemented methods validated the early design assumptions.

The other part of geotechnical challenge was the heavy foundation of STS rear crane beam. To fulfill the very restrict Engineer's requirements concerning displacements i.a. tolerance of gantry crane rails movement $\Delta h = 10$ mm, capping and rear crane beams needed to be joint to assure convergence of displacement between the rails. Thus the quay wall structure was joint to the

rear crane beam with two rows of tie-rods (bottom M145, top M120). As the rear beam had to offer sufficient horizontal and vertical stiffness for the whole system it was designed (see Fig. 13):

- Four rows of raked CFA piles (dia. 0.65 m, up to 29 m below GL),
- Single row of 45° inclined T103S hollow bar system micropile (dia. 0.3 m, up to 36 m).

Fig. 14. Principle of soil-column interaction and load transfer

Fig. 15. Cross section of large scale embankment test – field trial executed on DCT site

Fig. 16. Results of full-scale embankment test on soft organic silt improved with VCC in the reclaimed area

These elements were also successfully tested on site prior to piling works. Finally, the executive design was prepared and issued for construction. However, the complex geotechnical solution was constantly, at all stages controlled and coordinated to keep all adopted ideas in line with hydrotechnical and structural engineering.

Conclusions

Further expansion of port facilities and access infrastructure brings outstanding development opportunities for Poland and landlocked countries such as Czech Republic, Slovakia, Hungary, Belarus and Ukraine being located within the port hinterland less than 800 km away. The engineering has to follow that challenging demand providing the Port of Gdansk sophisticated but safe foundations. It is highly recommended to perform field trials prior to commencement of works, to set proper QA/QC procedures and to monitor the real life of the structure in order to verify the implemented solution, keep the high quality of work and mitigate any risk. Presented in the paper successful applications of complex geotechnical solutions in difficult ground water conditions at the Vistula River estuary, provided by latest design software and advanced technologies combined with high quality procedures made it possible to perform large infrastructure projects opening the gateway to Central-Eastern Europe widely.

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