Performance of a full-scale embankment test on soft organic silt improved with hybrid concrete-stone columns in the reclaimed area of the existing basin

Performance d'un essai de grande échelle de remblai sur un limon mou organique avec colonnes mixtes béton-gravier dans une zone de réhabilitation d'un bassin existant

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ABSTRACT: The paper describes a case study detailing the use of hybrid concrete-stone columns to support the pavement structure in the quay wall area at the Port Gdansk. The design solution was optimized based on the soil profiles, the geologic origin, the loading conditions and the project serviceability requirements. A full-scale instrumented test section was constructed during a field trial program. The test section simulated the weight of the engineered fill, pavement and surcharge load. The settlements and pore water pressures were monitored during the construction of the test section. The monitoring program measurements were compared with calculated and allowable values from the preliminary design. The results of the embankment test show that improving the existing upper granular layer can work as a load transfer platform by utilizing the arching effect to spread the applied load to the columns over the soft organic layer.

RÉSUMÉ: L'objet de cette contribution est l'étude de cas d'une solution géotechnique de soutènement d'un quai. Le calcul a été effectué en considérant les différents profils de sol, leur origine, les charges et les critères aux états limites de service. Les auteurs présentent la technique utilisée de colonnes mixtes béton-gravier et le programme d'essais sur site intégrant l'exécution de grande échelle d'un remblai, la simulation du poids du remblai apporté, de la chaussée et de la charge apportée ainsi qu'une observation des tassements et des surpressions de l'eau interstitielle. Les tassements mesurés à partir de l'essai sont discutés et comparés avec les valeurs calculées et les valeurs autorisées. Cette contribution présente l'idée d'améliorer la couche supérieure de sol granulaire en tant que matelas de transfert de charge et présente le mécanisme de voûte permettant de distribuer les charges dans les colonnes au niveau de la couche molle organique.

KEYWORDS: ground improvement, concrete-stone column, hybrid column, vibro replacement, rigid inclusion, embankment.

1 INTRODUCTION

Geographical location is the major determinant of the development prospects and growth opportunities of every port. The Port Gdansk (Poland) 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. The port's development strategy until 2027 is to become the leading hub on the Baltic Sea. The extension of the Deepwater Container Terminal (*DCT*) – Fig. 1, a new 656 m quay with adjacent 25 ha container stacking yards, allows the terminal to meet the growing demand for deep-sea services in Central-Eastern Europe and enables the handling of ultra large container vessels (Buca and Mitrosz 2016).

Figure 1. New berth and container stacking yards situated on the left part of the visualization (source: DCT Gdansk S.A.).

The geotechnical part of the design and build process concerning the new berth and adjacent container stacking yards was divided into two major parts: the heavy foundation of the ship-to-shore gantry crane beam and deep soil improvement of the platform, quay wall (45 m landwards from the seaside crane rail) and transition zone connecting both areas (Fig. 2). A significant part was the reclaimed area of the existing basin with a backfill depth from 3 m to 14 m; therefore, the geotechnical engineering faced many challenges.

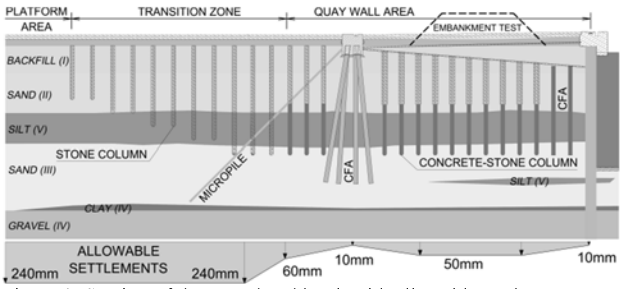

Figure 2. Section of the completed berth with allowable settlements.

In the **platform area** the aim of the soil improvement was to compact loose fill to even the settlements and offer sufficient stiffness for the pavement structure. The improved upper layer function was to uniform the loads and distribute them to the deeper silty layer that controlled the global settlements. In the **transition zone**, soil improvement elements were adopted in variable grids and lengths to ensure the smooth transition of the allowable settlements, so the length of the zone was estimated to be at least 33 m. In all cases, the geotechnical solution was adjusted to the soil profiles, the geologic origin, the loading conditions and the project serviceability requirements (Fig. 2). 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 to reduce the earth pressure on the quay wall structure.

2 GEOLOGICAL CONDITIONS

The area of the study lies in the industrial part of city of Gdansk located in the Gulf of Gdansk and is recognised as the Vistula Spit, forming a natural barrier against sea intrusion. Soil sedimentation due to transport by the Vistula River was the main phenomenon creating the geological formations. The region is known for its difficult ground and water conditions with a significant presence of marine and alluvial deposits, aged from Holocene, developed in the form of sands and soft organic silts with very low strength and deformation parameters.

Distinguished layers presented in Table 1 fit the soil classification adopted to local conditions after Robertson et al. (1986) and Schmertmann (1978), respectively for sand deposits and sensitive silts. Lunne et al. (1997) defined the applicability and usefulness of *in situ* static and dynamic soundings as rather moderate to poor in terms of soil type identification and recommended additional tests. Therefore, to determine the appropriate stratigraphy, a dense mesh of boreholes were drilled on site and analysed. The coherence of the values obtained from Dynamic Probing Heavy (*DPH*) tests with the CPT results was checked using a correlation presented in the Eurocode (2007) (see Fig. G.1) and the results were in good agreement.

The sensitive silty layer (stratum *V*) with an organic content of ca 6 m thickness was present all over the site and contained inclusions of fine sand, sandy dust and local peat. According to the BAGEO soil investigation campaign (2014), the organic content (I_{om} from 4.3 to 31.7%), cone resistance (q_c from 0.3 to 2.1 MPa), friction ratio (R_f from 3.6 to 18.8%) and undrained shear strength (s_u from 6.5 to 71.9 kPa) made the weak layer highly diversified in its parameters and thus the silty layer needed some special geotechnical treatment adjusted to the structure.

Table 1. Generalized stratigraphy, strength and deformation parameters.

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

The parameters for sandy soils were defined using the relation between the number of blows in the DPH test to achieve 10 cm of penetration (N_{10H}) and the relative density (I_D) after Eurocode (2007) (see Eq. 1). The relative density was rechecked with the results available from the CPT tests using the recommended regional formula after Borowczyk set in the Polish Standard (2002) applicable for non-cohesive soils with a uniformity coefficient, c_u>3 (see Eq. 2). The results from both formulas were in line. After determining the relative density, it was possible to correlate its value with the internal friction angle (□). One of the most popular Meyerhof (1956) correlations was used (see Eq. 3) and the coherence was checked using the more direct formula recommended in the German Standard (1990) (see Eq. 4).

$$I_D = 0.23 + 0.38 \cdot \log(N_{10H}) \tag{1}$$

$$I_D = 0.709 \cdot \log(q_c) - 0.165 \tag{2}$$

$$\phi' = 28° + 0.15 \cdot I_D \tag{3}$$

$$\phi' = 23° + 13.5 \cdot \log(q_c) \tag{4}$$

$$M = E_{oed} = \alpha_m \cdot q_c \tag{5}$$

The constrained modulus (*M*) for organic silts was derived using a coefficient (α_m) adapted from Sanglerat (1972) after Mitchell and Gardner (1975) (see Eq. 5). Taking into account the above approach, an effective overburden pressure of 100 kPa, a mean α_m of 5 and q_c of 1 MPa, the estimation of the constrained modulus would be 5 MPa, which showed good agreement with the oedometer tests made on samples of the deep silty layer proving at least 4 MPa. However, for shallow silty layers, up to 3 and 6 meters below ground level, a more conservative estimation of the modulus was defined – 1.4 MPa and 2.8 MPa respectively. The constrained modulus for clays and sands was derived using a coefficient (α_i) after Senneset et al. (1989) adapted to regional conditions, with a coefficient equal 15 for clays and 8 for sands.

3 SOIL IMPROVEMENT

As is common practice for the Rigid Inclusion (*RI*) concept when the upper granular layer is non-existent or is thin and the element is just embedded in the Load Transfer Platform (*LTP*), a pile cap increasing the support area is recommended. The structure of the LTP – i.e. minimum thickness and composition – has to allow for appropriate load transfer between the rigid elements and the soil, as well as to limit internal forces in the supported structure that leads to optimal design. To offer sufficient stiffness for the foundations, in most cases the mattress is composed of soil reinforced by geosynthetic sheets or steel meshes. The design and execution details of the overall RI concept is described in the ASIRI recommendations (2012). However, the design process of an embankment reinforced at its base with geosynthetic layers, LTP recommendations and the possible mechanisms of vertical load distribution are also widely discussed in literature – i.e. Van Eekelen et al. (2016), EBGEO (2010) or BS 8006 (2010).

3.1 *Implemented method*

The presented case study concentrates on a concept where the upper granular layer is present and interacts with the applied columns (Fig. 3). To streamline the production process, alternatively combined concrete-stone columns were used, known as Vibro Replacement (*VR*). The 2.5x2.5 m hybrid columns implemented in a square grid consisted of a stone head and rigid concrete shaft at the bottom. The idea of 0.8 m diameter stone heads was to homogenize and compact the upper granular platform, leading to an increase in its angle of internal friction and stiffness. As the method is a full displacement technique, is volumetric in character and leads to a relative improvement in the strength parameters of the ground surrounding the columns, the behaviour of ground improved with stone columns is usually described as a lumped mass with improved parameters. Priebe (1995 and 2005) provided a well-known method for the assessment of the lumped parameters of the layer improved with stone columns. In the studied case an 8% replacement ratio was sufficient enough to offer a stiff load transfer platform. The idea of 0.6 m diameter concrete shafts was to "bridge" the soft soil (stratum *V*) and applied load after the installation of columns. The load was shared between the columns and the ground, which led to reduced overall settlements and earth pressure in the silty layer.

3.2 *Load distribution mechanism*

The mobilization of shear strength in the granular material occurs and concentrates at the edges of the concrete column head (Chevalier et al. 2011), resulting in the transference of the load from the peripheral zones towards the rigid column. This model is called the diffusion cone method. In this mechanism, load transfer takes place via a shear along the cone whose geometry depends on the peak friction angle of the material being used for the platform (Fig. 3).

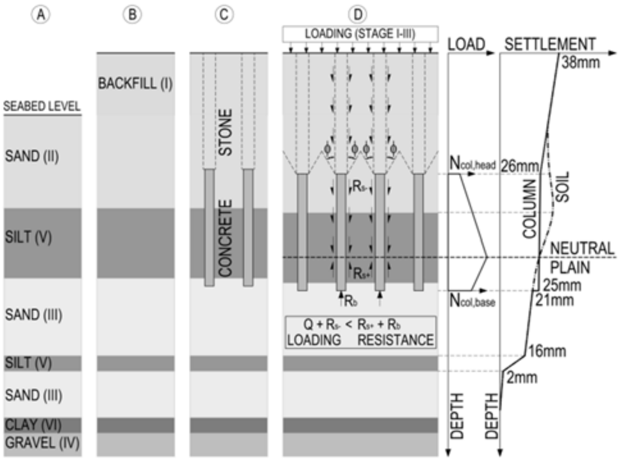

Figure 3. Soil-column interaction diagram with phases: A – initial, B – backfilling, C – soil improvement, D – embankment test.

The working of the system requires the load to be transferred to the column that is achieved by an arching effect due to the stone columns improving the granular platform (stratum *I* and partially *II*). The basic principle of the model is that the load concentrates on the concrete column head, so to mobilize the effect, the relative displacement (penetration) of the head into the upper sand is required. As a result of the stiffness, the contrast between the rigid material and the surrounding soft soil, the columns attract most of the load. This mechanism also needs an end bearing capacity at the column toe that transfers the load to the sand (stratum *III*). In the presented case, the Bustamante and Gianeselli (1982 and 1998) design method based on *in situ* test results was chosen to derive the ultimate bearing capacity:

$$R_b = K \cdot q_{ca} \cdot A_c \tag{6}$$

where K is the coefficient dependent on the soil type and piling technology (for the displacement piles for sand/gravel K=0.5-0.75), q_{ca} is the arithmetic mean value of the cone resistance over a length of ±1.5D and A_c is the base area of a pile.

The upper part of the soft layer is subject to settlement compared to the column, developing downdrag forces (negative skin friction) till a neutral plain is achieved. Then, the settlement of the column gets larger than that of the surrounding soil, which develops positive skin friction. The soil-column interaction determines the share of the load and settlement of the system. Also, the structural capacity of the columns has to be checked.

4 FULL-SCALE EMBANKMENT TEST

To confirm the validity of the calculations, a full-scale instrumented embankment test was performed (Fig. 4). The field trial program assumed the overload of the embankment as a substitute weight for the engineered fill (stage I), pavement (stage II, 16 kPa) and surcharge load (stage III, 40 kPa) and observation of settlements on 6 settlement gauges (S1-S6) and piezoelectric sensor (P1).

After the execution of VR columns from a working level of +1.0 m, gauge S1 and sensor P1 were installed in soil stratum *V*. To determine the granular platform compaction after treatment, 7 m deep CPT tests were performed, proving that the cone resistance was high above the required 10 MPa (mean q_c>20 MPa). Five days later, stage I of embankment was finished reaching level +2.0 m and gauges S2-S6 were installed. On the sixth day stage II of the embankment was achieved reaching level +3.0 m (+16 kPa). After the next 6-7 days of monitoring, stage III of the embankment was built reaching a final level of +5.7 m (+40 kPa) and was monitored for the next 51 days.

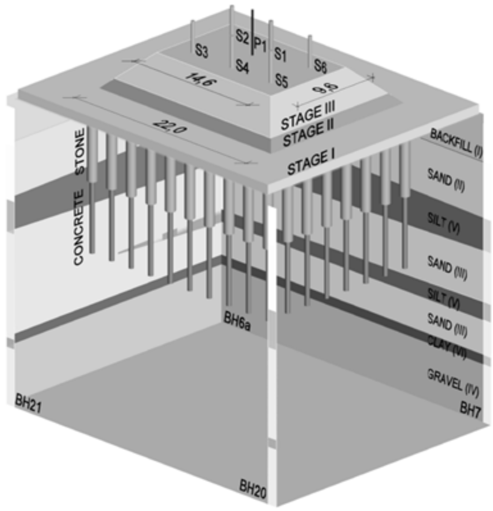

Figure 4. Full-scale instrumented embankment test.

4.1 *Field measurements*

The appropriate sequence of embankment execution made it possible to assess and distinguish the real values of immediate settlements that would occur during the pavement construction and surcharge load (Fig. 5). One can observe the development of the settlements on all the gauges showing a similar type and pace of deformation. Local fluctuations in the deformations and/or heaves may be observed due to the dynamic influence of heavy equipment (piling rigs, cranes) present in the surrounding of the embankment. The mean pace of the settlements observed was 0.212 mm/day (from day 26 to 34) with the curves starting to flatten. However, due to the tight schedule, the dewatering works started during the monitoring of the embankment. It can be observed that turning on the pumps (day 34) close to the field test caused disturbances resulting in a mean pace of 0.383 mm/day (from day 34 to 40). The decision was made to postpone dewatering till day 54 and full stabilization was achieved once more with a mean pace 0.150 mm/day (from day 40 to 54). Without dewatering, the consolidation time would have been longer and the overall increase of settlements would have been at least 2-3 mm smaller. In order to predict further results appropriately, trend lines for each gauge were performed with an extrapolation for 100 days. These curves were characterized by a high coefficient of determination (R≥0.99), proving high convergence with the measurements. As far as sensor P1 is concerned, the increase in pressure caused by the execution of the embankment to its final level was approximately 12 kPa in relation to the surcharge load (40 kPa), which proved the partial transfer of pressure in the upper layers. The speed of pressure dissipation was about 2.5 kPa/week; therefore a return to the initial pressure was observed after 5 weeks.

Taking into account the quick stabilization of settlements it may be concluded that part of the immediate settlements will occur during the large scale pavement construction as the process continues. The measured total values for relevant gauges S2-S6 were within the range 30-33 mm and were below the allowable settlements in the quay wall area (50 mm) and the estimated values in the Plaxis 2D FEM calculations using the Mohr-Coulomb soil model (38-42 mm). It should also be noted that the favourable effect of the concrete stiff pavement (0.7 m thickness) in flattening the settlements was not accounted for in the calculations and during the test, which errs on the side of caution.

The construction of embankment layer by layer and installation of monitoring equipment was successful. Measured values of deformations for all 6 gauges were in agreement with design assumptions and proved quick stabilization of settlements, thus validated the effectiveness of implemented method.

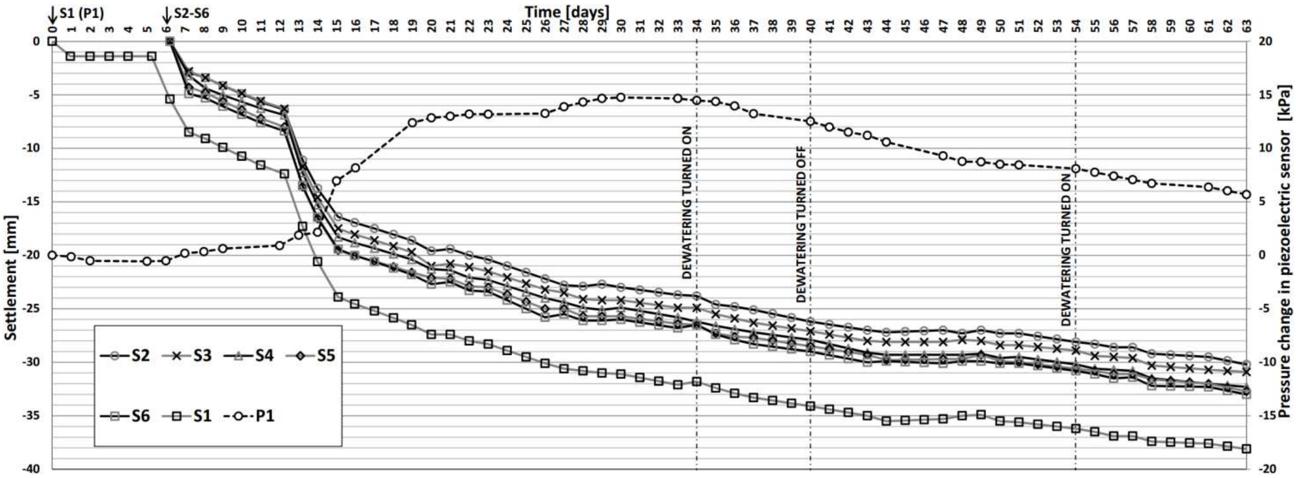

Figure 5. The results of the full-scale embankment test on soft organic silt improved with VR columns (63 days of monitoring).

5 CONCLUSIONS

The presented case was just a part of the complex geotechnical engineering that was implemented at the DCT site. The applied system was successful and the full-scale embankment test validated the calculations.

The presented rigid inclusions concept is widely used for soil improvement under embankments or other types of foundations, but has its limits and restrictions in terms of design and execution, so an appropriate risk has to be accounted for and taken into consideration (Katzenbach et al. 2013 and Topolnicki 2013). Several lessons learned from failed applications of RI used to improve weak subsoils were lately observed in infrastructure projects and reported by Świniański and Marchwicki (2014). One should not forget that what is very important for the final effectiveness of the geotechnical system is the high quality of the working platform preventing concrete elements from damage caused by ill-conceived further construction, especially when the columns are designed as unreinforced concrete elements.

This is why, before choosing any geotechnical solution, the geo-engineer has to consider a variety of components: the applicability of certain technology and its limits, type of structure, type of applied loads, structure sensitivity to settlements and type of foundation soil. It is also highly recommended to perform field tests prior to the commencement of works, to set appropriate QA/QC procedures and monitor the real life of the structure in order to verify the implemented solution, maintain the high quality of work and mitigate any potential risk. The applied solution also needs to fit the construction timetable and finally has to be economically viable. Thus, geotechnical engineering has to face many challenging demands.

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