Uplift control of jet grouted bottom plug by friction mobilized along embedded anchoring elements

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ABSTRACT

Jet grouted horizontal plug was used to seal the bottom of 20.5m deep TBM launching shaft constructed for a new underwater road tunnel in Gdańsk. The plug was positioned right below the excavation level, and anchored with hollow bar steel micropiles 76 mm in diameter and 15m long, installed centrally in 1m diameter and 10m long jet grout (soilcrete) columns. The columns were arranged in a triangular grid with side length of 2.1m, and constructed as part of a single production cycle together with the plug columns that were 3.1m in diameter and 3.5m in length. The pull-out capacity of micropiles was determined by the frictional resistance mobilized at the tendon/soilcrete interface, and the measured average bond to soilcrete strength ratio was 0.25. A novel aspect was a combined use of the soilcrete plug as structural and watertight element. The adopted design solution contributed to reduction of bending moments and horizontal deformations of diaphragm walls, to more safer and controllable execution of works, and to cost and construction time savings.

INTRODUCTION

Unwanted inflow of ground water through the bottom of deep excavation pits may be restrained by horizontal plugs, often executed with jet grouting (Soilcrete) technology. In typical design situations the soilcrete plug is positioned below excavation bottom, at a depth ensuring that the total stress of reduced overburden outbalances hydrostatic uplift pressure with a sufficient margin of safety. The resulting depth is therefore usually significant, and often larger than the minimum embedment needed to satisfy stability of structural walls, protecting the excavation. Consequently, the excavation walls must be elongated to close the sealing trough. This triggers additional costs and may lead to uneconomical design. Beyond controlling seepage, the soilcrete plug may be also utilized as an efficient structural ‘strut’ below the excavation bottom, reducing wall bending and deflection. For this purpose the plug should be positioned as high as possible, preferably right below the excavation level. In this position, however, the plug is usually unstable and requires anchoring.

A combined function of an anchored soilcrete plug, acting as structural and watertight element, was successfully utilized during construction of two deep chambers built in difficult ground conditions prevailing at the Vistula River estuary in Gdańsk, and used for the assembly of the TBM and shield drilling of a new road tunnel, comprising twin bores about 12.5m in diameter and 1072.5m in length (Fig. 1).
THE SOLUTION ADOPTED FOR THE TBM LAUNCHING SHAFT

The design solution selected for the protection and bottom sealing of a 20.5m deep excavation pit for the TBM launching shaft was the result of geotechnical contractor’s proposal to move from working underwater to a dry operation to cut construction time and reduce costs (Fig. 2). The design changes also helped make the scheme much safer and more controllable during the execution of works. The ground conditions and the adopted protection system of the launching shaft are shown in Figure 3.
Figure 3. Ground conditions and the adopted protection system of the launching shaft.

The ground comprised alternating marine and alluvial deposits of medium-dense to dense sands and soft organic silts with a low strength to a depth of about 23m below GL, and underlying silty clay and sand layers. The low-permeable silty clay layer (Vla) could not be used as a natural cutoff barrier for the launching and receiving shafts because it would be unstable against uplift pressure developed during ground excavation works. The groundwater in the area of the tunnel is mostly confined, and its level is stabilized at a depth of about 0.5m above sea level (ASL), which was less than 1m below the working platform. This caused additional problems and required elevating the guide walls during the construction of the diaphragm walls. The tunnel structure is protected against maximum water level of +2.5m ASL during normal exploitation, representing extreme flood level (EFL). During construction stages two design water levels of +0.5m and +1.5m ASL have been also used in the analyses.

The adopted protection system of the launching shaft during temporary construction stages included 1.2m thick slurry diaphragm walls along the chamber boundaries and embedded to 31m below GL, concrete and steel strutting, and the soilcrete plug constructed at a depth of 24 to 20.5m below GL (Fig. 3). In order to balance the buoyancy force, the plug was anchored with hollow bar steel micropiles, 76mm in diameter and 15m long, installed centrally in 1m diameter and 10m long jet grout columns, constructed as part of a single production cycle together with the plug columns that were 3.1m in diameter and 3.5m in length. Along the plug section the tendon diameter was increased to 160mm with factory grouted corrugated sheathing. The anchoring elements were positioned in a grid of equilateral triangles with side length of 2.1m, adapted to the basic arrangement of the columns constituting the horizontal plug. Due to simultaneous execution of the anchoring elements and plug columns the monolithic connections between the two elements was
improved and thus the required rigidness and water tightness of the whole system protecting the excavation bottom was ensured.

The soilcrete plug was installed utilising the Super-Jet monitors and air-shrouded jetting technology. The jet grouting parameters were determined after in-situ tests. Because of significant drilling depths it was also necessary to check the vertical deviation of each drilling hole with the use of a special inclinometer. The verticality test allowed detection of displacement of columns and a shorter response time to fill in possible leaks in the plug. The average bore deviation was only 0.4% throughout construction in the launching shaft, revealing a high level of precision.

Special attention was paid to the impact of the phasing of the work with different design water levels at each stage of the excavation. The following phases were implemented to begin construction of the launching shaft and protect the excavation pit:

- **Phase I** – Execution from GL of the middle section of the anchored jet grouted plug, leaving a free stripe along the planned slurry diaphragm walls. The anchoring elements and jet grouted plug were constructed in a single production cycle reaching depths up to 34m below GL. After grouting, the proper reinforcement was installed in fresh soilcrete columns and disconnected about 19m below GL (Fig. 4a),
- **Phase II** – Construction of 1.2m thick slurry diaphragm walls along the chamber boundaries and embedded to 31m below GL (Fig. 4b),
- **Phase III** – Construction of a jet grouted plug along diaphragm walls to provide proper water tightness of the system (Fig. 4c),
- **Phase IV** – Construction of a jet grouted block and slab in front of the launching shaft, stabilising TBM at the beginning of shield drilling operation (Fig. 4d),
- **Phase V** – Primary excavation to the depth of 4.5m below GL and construction of a reinforced concrete top slab with wide openings, enabling lowering and installation of the TBM at the chamber bottom,

![Figure 4. Main construction phases adopted for the TBM launching shaft.](image-url)
Phase VI – Continuation of excavation to the depth of 12m below GL and installation of a temporary steel strutting structure (Fig. 4e),

Phase VII – Excavation to the depth of 20.5m below GL and construction of the bottom slab, anchored to steel micropiles (Fig. 4f),

Phase VIII – Dismantling of the steel strutting structure installed at the depth of 11.5m below GL and construction of the eyepiece at the headwall; the launching shaft is ready for TBM assembly.

In order to monitor the diaphragm walls, the soilcrete plug with anchoring, installation of the strutting and groundworks in the excavation carefully, as well as to analyse large amounts of production data and control measurements, a virtual model of all structural elements of the launching chamber was prepared, corresponding to the design solution (Fig. 5a). On particular stages of groundworks it was possible to check, among others, displacement of the diaphragm walls or stresses in the steel struts, and to compare the measurements results with permissible values. Example records of wall displacements, illustrating the beneficial impact of the soilcrete plug acting as a structural support below the excavation level, are shown in Figure 5b.

![Figure 5. (a) Virtual model of the launching chamber; (b) Displacements measured perpendicular to the wall surface for two excavation depths (inclinometers 18 and 20).](image)

FIELD VERIFICATION OF THE BOND RESISTANCE OF ANCHORING ELEMENTS

In the adopted design solution the bond strength of the micropile to soilcrete interface plays a key role, and is especially important in temporary construction stages when the hydrostatic uplift pressure acting on the plug needs to be counterbalanced by a sufficient anchoring capacity of the micropiles embedded in soilcrete columns. To verify the bond resistance in the field, a series of four pull-out test was conducted on the construction site using the arrangement shown in Figure 6. The tests A1 and A2 comprised tendons with factory grouted corrugated sheathing diameter 160mm and active bond lengths of 1 and 2m, respectively. In tests B1 and B2 a row hollow stem bar type DSI T76-1600 was used, and the active bond sections were 3 and 4m long. All tendons
Figure 6. (a) Arrangement of four pull-out tests: 1 - soilcrete column with a diameter of about 1m, 2 - corrugated sheathing, 3 – p.v.c. sheath, 4 - hollow stem bar DSI T76-1600; (b) Factory grouted corrugated sheathing diameter 160mm; (c) Tendons A1 and A2.

were installed in ‘fresh’ soilcrete columns that were about 1m in diameter and 8.5m in length. A standard testing set-up with a hydraulic jack and a reaction beam was used (Figs. 7a,b). After completion of test A2 the tendon was extracted from the column for a visual inspection, confirming that the slip occurred at the corrugated sheathing/soilcrete interface (Fig. 7c).

Figure 7. (a) Installation of test tendon; (b) Testing set-up; (c) Corrugated tendon, retrieved from the soilcrete column after completion of test A2.
The recorded relationships between the applied pull-out force and the ‘net’ tendon displacement (after subtracting elastic bar elongation along the free lengths) are shown in Figure 8a, noting that the maximum applied load had to be terminated at about 1600 kN, corresponding to the ultimate bearing capacity of the DSI T76-1600 bar (Technical Approval, 2011). Assuming a uniform shear strain distribution along all active bond lengths the corresponding bond strength vs. average shear strain relationships were determined, as presented in Figure 8b. The plotted data indicate that the ultimate bond strengths can be related to about 0.6 to 0.8% shear strain mobilisation. In case of test A1, with the shortest bond length of only 1m, it is likely that multiple ‘slips’ occurred with a progressing displacement, and that a larger force is needed to effectively fully withdraw the tendon from the column as compared to pull-out force compatible with 0.8% shear strain.

Figure 8. (a) Pull-out force vs. tendon ‘net’ displacement (after subtracting elastic bar elongation along the free length); (b) Bond strength vs. average shear strain along bond length.

When analysing the ultimate bond strength of the tendon to soilcrete interface one should also take into account the soilcrete strength, shown in Table 1. The data represent soilcrete UCSs at 28 days of curing, measured on cube wet grab samples with a side length of 15 cm. The ratio of the ultimate bond stress to the mean UCS of soilcrete was found to be in the range of 0.2 to 0.33, with a mean value of 0.25. No distinct difference between the bonding efficiency of the corrugated sheathing and the row deformed bar embedded in soilcrete column was noticed.

Table 1. Summary of pull-out tests and evaluation of the ultimate bond strength.

<table>
<thead>
<tr>
<th>Test</th>
<th>Soilcrete UCS</th>
<th>Active bonding surface [m²]</th>
<th>Ultimate bond strength [MPa]</th>
<th>Ratio of the ultimate bond strength to mean UCS, [-]</th>
<th>Remarks</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Cubes (15 cm), at 28 days [MPa]</td>
<td>Mean [MPa]</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>A1</td>
<td>7.2, 8.2, 9.8, 10.2</td>
<td>8.85</td>
<td>0.503</td>
<td>2.0 (1)</td>
<td>0.23</td>
</tr>
<tr>
<td>A2</td>
<td>5.4, 5.8, 6.0, 6.2</td>
<td>5.85</td>
<td>1.005</td>
<td>1.5 (1)</td>
<td>0.26</td>
</tr>
<tr>
<td>B1</td>
<td>5.2, 6.4, 7.4, 7.6</td>
<td>6.65</td>
<td>0.716</td>
<td>2.2 (1)</td>
<td>0.33</td>
</tr>
<tr>
<td>B2</td>
<td>8.2, 8.4, 9.2, 10.2</td>
<td>9.00</td>
<td>0.955</td>
<td>1.8 (2)</td>
<td>0.20</td>
</tr>
</tbody>
</table>
SELECTED DESIGN ASPECTS

The complete design was checked using several numerical analyses and the displacements anticipated were in line with the predictions. Special attention was paid to the impact and influence of the phasing of the work with different water levels at each stage of the excavation. In relation to this study, however, the following design aspects require further discussion.

**Structural (internal) pull-out capacity of the micropile.** The design pull-out capacity of the hollow bar type T76-1600 had to evaluated for temporary (up to 2 years) and permanent situations, allowing for design service life of the tunnel structure of 100 years. The assumed double protection against corrosion consisted of a thick soilcrete cover, and a sacrificial thickness of steel (acc. to EN 1993-5, Tab. 4.1). The quoted value of loss of steel thickness was 3.25mm for aggressive natural soils and 100 years’ service life. Taking into account nominal and reduced cross-sections of the hollow bar due to a long-term corrosion, reading $A_{\text{nom}}=25.48\,\text{cm}^2$ and $A_{\text{cor}}=17.57\,\text{cm}^2$, as well as a partial safety factor for the resistance of cross-section in tension $\gamma_m=1.25$ (cf. EN 1993-1, 6.1) the design pull-out resistance of the adopted micropile is:

$$f_{\text{yd}} = f_{\text{s}} / \gamma_m = 490 / 1.25 = 392 \, \text{MPa} = 39.2 \, \text{kN/cm}^2,$$

$$R_{d,\text{temp}} = f_{\text{yd}} \cdot A_{\text{nom}} = 999 \, \text{kN} \text{ for all temporary stages, and}$$

$$R_{d,\text{perm}} = f_{\text{yd}} \cdot A_{\text{cor}} = 689 \, \text{kN} \text{ for the permanent situation.}$$

**Tendon to soilcrete bond capacity.** The characteristic compressive strength of soilcrete for the tunnel project was set to $f_{\text{ck,cube}}(28\text{-days})= 5 \, \text{MPa}$. Based on the test data presented above, the ultimate bond strength can be evaluated as $f_{\text{bu}} = 0.25 \cdot f_{\text{ck,cube}}$. With a partial safety factor $\gamma_u=2$, the design bond strength of the tendon to soilcrete interface equals $f_{\text{bd}} = f_{\text{bu}} / 2 = 0.625 \, \text{MPa}$. Consequently, the design pull-out resistance of the adopted micropile equals:

- for the row bar embedded in 10m long soilcrete column: $R_{\text{bd}} = \pi \cdot 0.076 \cdot 10 \cdot 625 = 1,491 \, \text{kN}$, and
- for the corrugated part embedded in 3.5m thick soilcrete plug: $R_{bd} = \pi \cdot 0.16 \cdot 3.5 \cdot 625 = 1,099 \, \text{kN}$.

**External pull-out capacity of soilcrete column.** The ultimate baring capacity of the soilcrete column pulled out from the ground reads:

$$R_{sk} = \sum (q_{cl} / k_s \cdot t_i) \cdot \pi D$$

where: $q_{cl}$ – static cone tip resistance in $i$-th soil layer [MPa], $k_s$ – coefficient depending on soil type, $q_{cl}$ value and piling technology [-], $t_i$ – thickness of $i$-th soil layer [m], $D$ – column diameter [m]. Taking into account a design soil profile, showing average cone tip resistance of about 2.5 MPa in clay and about 20 MPa in sand, and a soilcrete column of one meter diameter the ultimate pull-out capacity is:

$$R_{sk} = (2500/40 \cdot 4.0 + 20000/150 \cdot 6.0) \cdot \pi 1.0 = 3,297 \, \text{kN}.$$  

The design pull-out resistance for a 10m long soilcrete column pulled out from the ground reads:

$$R_{sd} = R_{sk} / \gamma_u = 3297 / 2 = 1,648 \, \text{kN}.$$  

Comparing the above design situations it can be concluded that the minimum pull-out resistance of the considered anchoring system is governed by the design tensile capacity of the adopted micropile type DSI T76-1600.

**Global stability against uplift.** Global stability of the launching shaft 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 forces acting on the undersides of jet grouted plug and of the foundation slab, respectively, were calculated for ground water levels at +0.5m (Fig. 9a) and +1.5m ASL (Fig. 9b). In addition to the dead weight of the structure, enclosed soil and soilcrete plug also the effective weight of the ground adjacent to the soilcrete columns was taken into account. Frictional resistance that may develop between the ground and the sidewalls of the shaft structure was conservatively neglected \((T_s=0)\). The permanent stage 2, with buoyancy force acting on the bottom surface of foundation slab, was analysed for EFL at +2.5m ASL taking into account additional stabilising component resulting from the dead weight of structures to be constructed inside the chamber (e.g. technological building).

![Figure 9. Global stability scheme for the launching shaft; (a) Temporary stage 1a with uplift force acting on the underside of the soilcrete plug and with GWL at +0.5m ASL; (b) Temporary stage 1b with uplift force acting on the underside of the foundation slab and with GWL at +1.5m ASL.](image)

The buoyancy safety in the uplift limit state (UPL) was checked using the following general formula:

\[
V_k \cdot \gamma_{dst} \leq (G_k + T_k) \cdot \gamma_{stb}
\]  

where: \(V_k\) - characteristic value (index \(k\)) of uplift force at the respective level, representing destabilising action, \(G_k\) - characteristic value of effective dead weight component (lower estimate), \(T_k\) - characteristic value of effective friction component (lower estimate), \(\gamma_{dst/stb}\) - partial safety factors for destabilising and stabilising permanent actions in the UPL, adopted from EC7, table A15, Appendix A \((\gamma_{dst} = 1.0, \gamma_{stb} = 0.9)\).

Particular form of Eq.1 depends on the analysed construction stage. In case of stage 1a (cf. Fig. 9a) it reads:

\[
V_e \cdot \gamma_{dst} \leq (G_e + 2T_e + G'_b + 2T'_b) \cdot \gamma_{stb}
\]  

while for stage 1b (cf. Fig. 9b) it changes to:

\[
V_p \cdot \gamma_{dst} \leq (G_p + G'_e + G'_b + 2G'_s) \cdot \gamma_{stb}
\]
where: $V_e$ - uplift force at the underside of the jet grouting plug (index $e$), $G_e$ - dead weight of soilcrete plug ($G'_e$ - under water), $T_e$ - friction force between soilcrete plug and tunnel wall, $G'_b$ - dead weight of soilcrete columns and adjacent ground under water, $T_b$ - friction force between tunnel wall and the ground, $V_p$ - uplift force at the underside of concrete slab (index $p$), $G_p$ - dead weight of concrete slab, $G'_s$ - dead weight of tunnel walls under water.

It was found that the global stability condition was decisive for the determination of the number and length of anchoring elements preventing uplift of the excavation bottom, with temporary stages being more important than the permanent situation.

**CONCLUSIONS**

The project involved construction of large excavations for tunnel ramps and two deep TBM shafts as well as working with high groundwater levels and some challenging ground conditions. The adopted design was the result of geotechnical contractor’s proposal to move from working underwater to a dry operation to cut construction time and reduce costs. A novel aspect was a combined use of anchored soilcrete plugs in the TBM shafts acting as structural and watertight elements. In case of the launching chamber the location of soilcrete plug just below the excavation bottom contributed to significant reduction of bending moments and horizontal deformations of diaphragm walls.

Quality control was also a prime focus, and most of techniques were trialled before work on the main construction started. Special attention was paid to the adopted anchoring system, comprising hollow stem steel micropiles installed centrally in soilcrete columns forming anchoring piles and the sealing plug in a single production cycle. Dedicated pull-out tests revealed that the ultimate bond strength $f_{bu}$ of the tendon to soilcrete interface can be evaluated in relation to the characteristic unconfined compressive strength of soilcrete $f_{ck}$, taking $f_{bu} = 0.25 \cdot f_{ck,cube}$. This finding is in a good agreement with the recommended value of $f_{bu} = f_{ck}/3$, proposed by Burke and Yoshida (2013), noting the usually adopted relation between the compressive strength measured on cylindrical ($h/d=2:1$) and cube samples, i.e. $f_{ck} = 0.8 \cdot f_{ck,cube}$, leading to $0.25/0.8 = 0.31$.

When analysing buoyancy safety for temporary and permanent construction stages, the limit cases investigated included the uplift capacity of the individual anchoring elements, and the global stability of the shaft structure assuming that the anchoring elements form a uniform soil monolith together with the surrounding ground within their zone of influence. Special attention was paid to the impact and influence of the phasing of the work with different water levels at each stage of the construction.

**REFERENCES**
