Stabilization of excavation bottom against uplift with Vibro Concrete Columns

Stabilisation du fond d'excavation contre le rénard à l'aide des Vibro Béton Colonnes

M.Topolnicki – *Technical University of Gdańsk, Poland*

ABSTRACT: Poor soils and high groundwater table were met during construction of modern urban carriageway with undercrossing pedestrian tunnel. Along the tunnel axis the ground conditions comprised up to 3.0 m of variable fill overlying highly compressible organic clay and peat deposits, 4.5 to 6.5 m thick. In order to overcome the problems of groundwater lowering for tunnel construction, excavation works were preceded with stabilization of the clay layer against uplift with Vibro Concrete Columns (VCCs), also used at this site to support the highway embankment. The VCCs were installed from the existing ground surface in a triangle grid of 1.6 m axial spacing, and were terminated in the subsoil at the level corresponding to the bottom of planned excavation. The effective length of VCCs was designed to prevent uplift of the pit bottom during excavation works, taking into account arching between the columns. With this solution tunnel excavation was successfully conducted without deep dewatering system. Furthermore, the VCCs installed below the tunnel slab yielded additional support for the tunnel structure.

RESUME: Pendant la construction d'un carrefour municipale moderne avec le tunnel profond pour les piétons on a rencontré des sols très faibles et le niveau de nappe phréatique très haute. Le sous-sol est formé des remblais incontrôlés de 3.0 m d'épaisseur et les dépôts alluvionnaires de vases et de tourbes de puissance de 4.5 au 6.5 m. Pour résoudre le problème du drainage profond et pour pouvoir construire le tunnel pour les piétons on a stabilise de la couche de vases à l'aide des Vibro Béton Colonnes (VBC) désignés aussi pour le support du remblais routier avant d'excavation de la fouille. Les VBC sont installées en réseau triangulaire de 1.6 m à partir du niveau du terrain jusqu'à la profondeur correspondant au niveau du fond d'excavation. Nous avons calculé la longueur efficace de colonnes en prenant en compte la possibilité de la rupture du fond d'excavation en train de travaux souterrains et l'effet de formation de voûte entre les colonnes voisines. La solution utilisé a permis la continuation des travaux d'excavation sans utiliser du systéme du drainage profond. Les colonnes installées au-dessous de la dalle de couverture du tunnel ont constitué un support additionnel pour la charpente du tunnel.

1 INTRODUCTION

A new carriageway in Gdańsk, completed in 1995, comprises a dual three lane urban highway and a tramway running within the central reserve. It provides a strategic east-west link of the city transportation system with the main road to Warsaw.

Along a distance of about 173 m this carriageway had to be constructed over deep deposits of highly compressible soils. A general philosophy of the design applied for this section was to employ a geogrid-reinforced embankment to transfer the carriageway loads via enhanced arching to a grid of Vibro Concrete Columns (VCCs), which further transfer the loads through the weak alluvial soils to the underlying sands (Topolnicki, 1996).

At the same building site a new pedestrian tunnel undercrossing the carriageway had to be constructed. In the first design, the 7.2 m wide and ca. 46 m long tunnel was supported on two rows of 11 to 13.5 m long cast-in-place concrete piles (of the Wolfsholtz type), located under the tunnel side walls. For construction of the tunnel an open pit excavation was foreseen, together with a suitable groundwater level lowering system, comprising of four ⌀500 mm deep wells, to provide a depression of 4 m at the central part of the tunnel. It has been estimated that the corresponding depression radius would be about 96 m, causing a potential danger to old neighbouring buildings, partly founded on wooden beams and piles. For this reason an expensive reinfiltration system was additionally considered in this design.

In order to overcome the problems and high costs of installation and operation of a combined groundwater lowering and reinfiltration system an alternative solution was proposed and applied. In this design (Topolnicki, 1994) a novel application of VCCs was considered. The idea was to eliminate the use of deep dewatering and reinfiltration wells and to stabilize the bottom of excavation against uplift with VCCs acting as vertical pulled out anchors. The VCCs were formed in the subsoil prior to excavation works and were finished at the level corresponding to the bottom of future building pit. It has been assumed that the organic clay layer at pit bottom, of low permeability, will sufficiently reduce groundwater inflow to the excavation. The VCCs installed along pit side walls were finished ca. 2 m above the excavation bottom and acted as short retaining walls. This provided savings with respect to the volume of earth works. With these provisions excavation and tunnel construction works were successfully conducted without a combined deep dewatering and reinfiltration system. Moreover, the VCCs installed under the tunnel slab yielded additional support for the tunnel structure.

In the following further details concerning ground conditions are presented and stability analysis of a bottom layer anchored with VCCs is considered. Finally, the applied design and the construction technique are explained and experience gained during construction works is summarized.

2 SOIL AND GROUNDWATER CONDITIONS

Ground conditions were investigated by means of boreholes, dynamic soundings and laboratory tests (Geoprojekt 1989), trial embankment (Werno et al. 1993) and georadar investigations (NGI&Geostab, 1994). A generalized geological cross-section of the subsoil along the longitudinal tunnel axis is illustrated in Figure 1. The partially loose fill (I) of thickness between 1.6 and 3 m consists of sand, gravel, brick and concrete rubble and humus. It overlies a continuous layer of slightly decomposed wet peat (III), 0.7 to 1.2 m thick. Below peat there is a layer of plastic organic clay (IV) of thickness between 3.3 and 5 m, overlying a basal layer of fine and medium sand (V), containing organic matters when closed to peat. The organic clay is inhomogeneous, mixed with silt, sand and peat. Thus peat and organic clay build together a thick deposit of compressible soils. A trial embankment of base area ca. 40×54 m and 3 m tall, constructed above a 5.5 m thick peat/organic clay deposit covered by 2.5m thick fill, indicated high compressibility potential of peat and organic clay layers. The settlements recorded at the ground surface within one year after construction ranged from 28 to 44 cm, with no

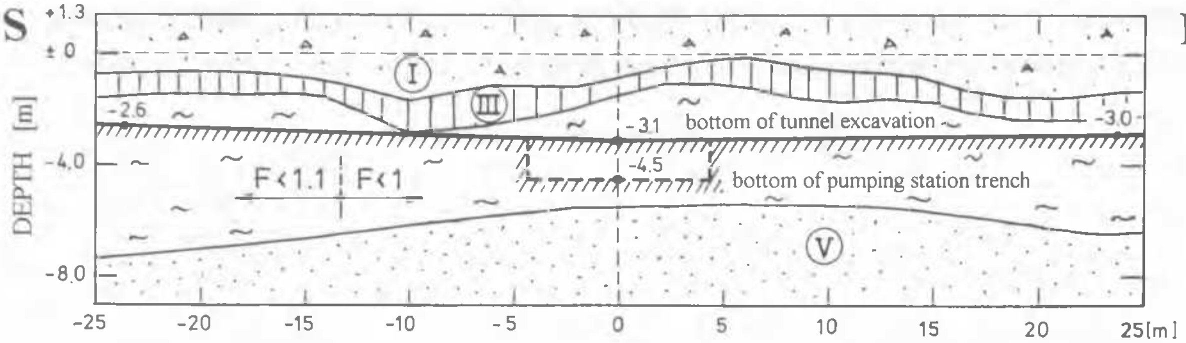

Figure 1. Generalized geological cross-section of the subsoil along the tunnel axis and the required excavation level of the building pit (soil conditions after Werno, 1994 and Geoprojekt, 1989); Legend: I - uncontrolled fill, III - peat, IV - organic clay, V - sand.

indication of stabilization (Werno et al. 1993). Geotechnical parameters of both compressible soils are listed in Table 1.

The groundwater level at the building site was observed to vary between +0.5 m above and -1.5 m below the sea level, depending on the season. The artesian water in basal sand layer, with a permeability of $k_{10} = 4.1*10^{-5}$ m/s, was found to stabilize between -0.54 and -1.17 m below the sea level.

Table 1. Main geotechnical parameters of compressible layers (Geoprojekt, 1989).

Soil	γ [kN/m^3]	w_{mean} [%]	N [%]	M_0 [MPa]
Peat	10.1-12.4	258	29-77	0.3 - 0.98 (0.58)
Organic clay	13.2 -17.8	63	5-15	0.5 - 3.8 (1.8)

Soil	τ_{max} [kPa]	ϕ_u [deg.]	c_u [kPa]	k_{10} [m/s]
Peat	33-49	10.5	5.7	$3.8*10^{-9}$ to $2.3*10^{-7}$ ($5*10^{-8}$)
Organic clay	33-63	9.2	9.2	$1.6*10^{-8}$ to $2.7*10^{-8}$ ($2.5*10^{-8}$)

where: γ = bulk density, w = water content, N = loss on ignition, M_0 = oedometric modulus in the stress range between 0.1 and 0.2 MPa, τ = maximum vane shear strength, ϕ_u = undrained friction angle, c_u = undrained cohesion, k_{10} = coefficient of permeability at 10^0C, ( ) = mean value, respectively.

As shown in Figure 1, the bottom of the building pit is entirely positioned within the organic clay layer, which has a very low permeability in relation to the underlying sand. Estimations of stability of an unprotected pit bottom against uplift, conducted for a design artesian water head corresponding to the level of -0.6 m below the sea, indicated that for a major part of the excavation bottom the safety factor F, defined as:

$$F = \frac{h_1 \gamma_1}{H \gamma_w} \geq 1.1 \tag{1}$$

where: h_1, H, γ_1 are explained in Fig.2, γ_w = unit weight of water,

would be less than 1.0 (see Figure 1), and less than 1.1 for the whole bottom area. For instance, in the central part of the excavation F was in the order of 0.8, and within a small excavation trench for the pumping station 0.3. Moreover, at both ends of the building pit, where the tunnel dead weight is much lower than the weight of the excavated soil, additional anchorage bearing capacity for a long term stability of the tunnel slab was required. Consequently, open excavation pit to the required depth could not be accepted without deep dewatering or bottom stabilization.

3 STABILITY OF PIT BOTTOM ANCHORED WITH VCCs

The general philosophy of the considered design of bottom stabilization with VCCs is based on the evaluation of ultimate pull out bearing capacity of VCCs with account for arching interaction between the columns.

A general situation at the excavation bottom stabilized with VCCs is depicted in Figure 2. The soil layers 1 and 2 represent, presumably, clay and sand, while $k_2 > 100 k_1$. Artesian water head in layer 2 equals H. The columns, having a diameter d and length $h_1 + h_2$, are arranged in a grid of equilateral triangles with a side length a, while their spacing is less than $3d$ to mobilize enhanced arching. Strong compaction impact in layer 2, occurring during formation of VCCs at close spacing, allows to set arching angle of 45^0, starting from the column base.

For an impervious bottom layer stabilized with VCCs the safety factor F_S against uplift, evaluated for a bottom area secured with a single column, can be written as:

$$F_S = \frac{G_1 + G_{k1} + G'_{k2} + G'_2}{\left(\sqrt{3}/2\right) a^2 H \gamma_w} \geq 1.1 \tag{2}$$

where:

$$G_1 = \tfrac{1}{4} h_1 \gamma_1 \left(2\sqrt{3}\, a^2 - \pi\, d^2\right) \tag{3}$$

$$G_{k1} = \tfrac{1}{4} \pi\, d^2 h_1 \gamma_b \tag{4}$$

$$G'_{k2} = \tfrac{1}{4} \pi\, d^2 h_2 \left(\gamma_b - \gamma_w\right) \tag{5}$$

$$G'_2 = \frac{\gamma'_2}{4\sqrt{3}} \left[\left(h_2 - \frac{a-d}{2}\right)\left(6\, a^2 - \pi\, \sqrt{3}\, d^2\right) + (a + 2d)(a - d)^2\right] \tag{6}$$

and γ'_2 = submerged unit weight of layer 2, γ_b = unit weight of concrete.

Equation (2) has been derived with the assumption that the average unit shaft resistance τ of VCC fulfils two ultimate conditions. In layer 1 the friction bearing capacity along corresponding column shaft must prevent uplift of the upper soil plague under water pressure, i.e.:

$$\tau_1 = \frac{\frac{1}{4}\left(2\sqrt{3}\, a^2 - \pi\, d^2\right) H \gamma_w - G_1}{\pi\, d\, h_1} \leq \tau_1^{ult} \tag{7}$$

In layer 2 there must be a sufficient friction bearing capacity to carry the weight of the interacting soil, i.e.:

$$\tau_2 = \frac{G'_2}{\pi\, d\, h_2} \leq \tau_2^{ult} \tag{8}$$

In Equations (7) and (8) τ_1 and τ_2 represent ultimate values of τ in layer 1 and 2, respectively, as for tension cast-in-place vibro piles. If the condition (8) is not fulfilled, the VCC acts as a tension pile and G'_2 in Equation (2) has to be replaced with a conventional pull out bearing capacity. Furthermore, since VCCs do not have

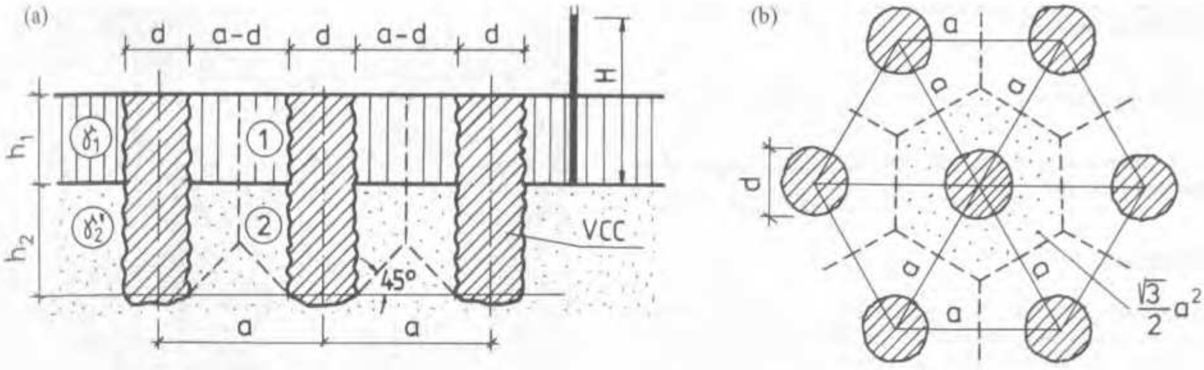

Figure 2. Definition sketch for evaluation of stability of excavation bottom stabilized with VCCs.
(a) cross-section, (b) arrangement of VCCs in plan.

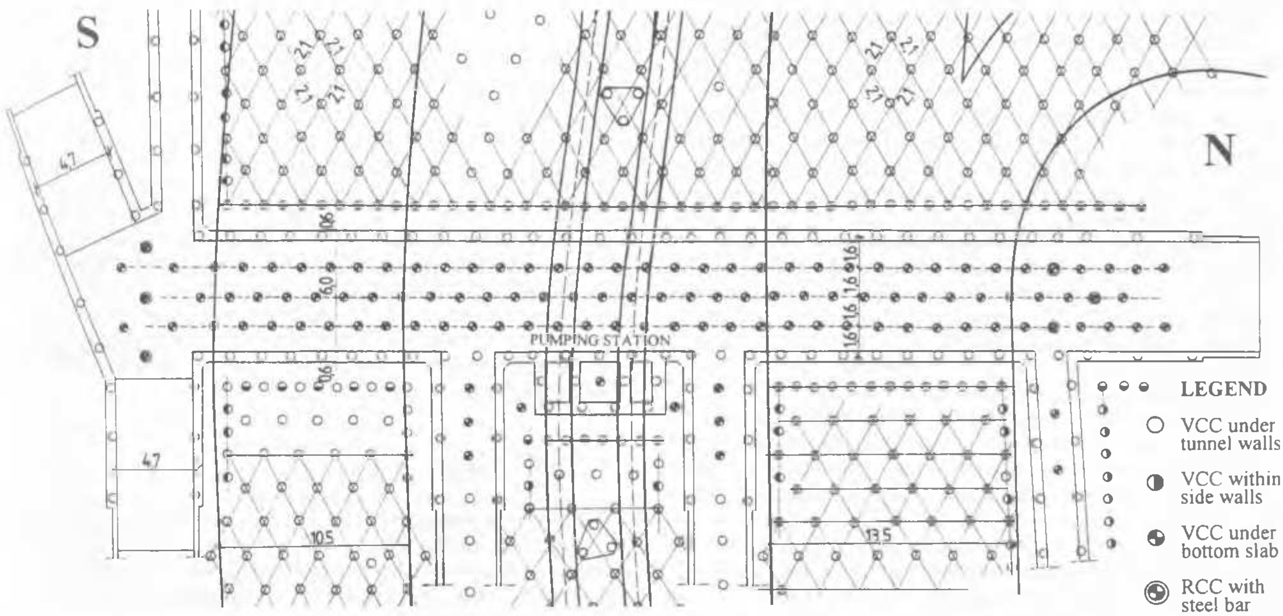

Figure 3. View of the tunnel layout and arrangement of VCCs.

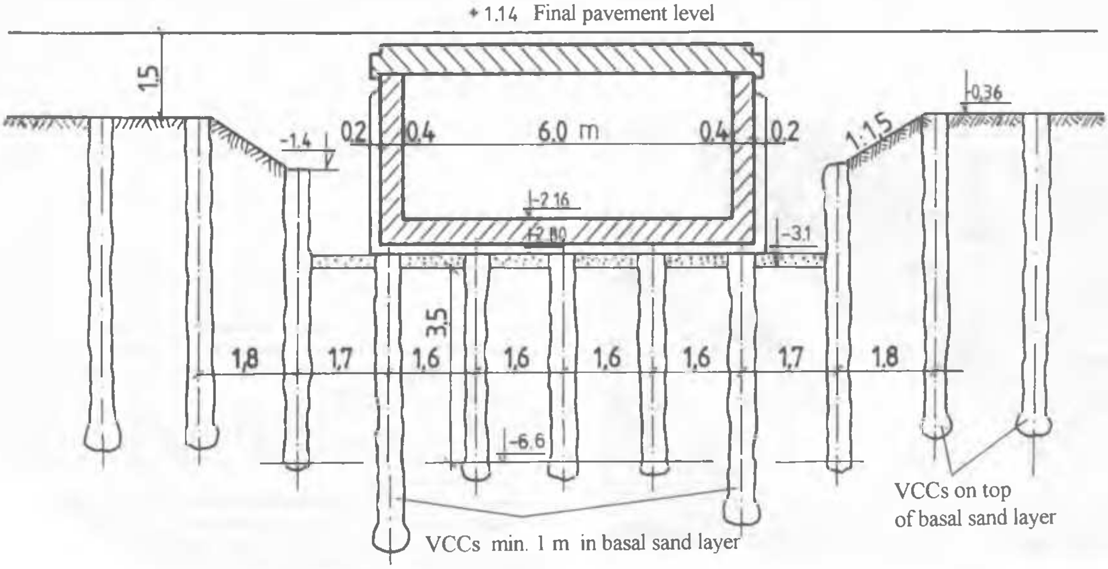

Figure 4. Cross-section of the tunnel and foundation support.

any reinforcement, the tensile stress σ_R at the interlayer level should fulfil the condition:

$$\sigma_R = \frac{G'_{k2} + G'_2}{\frac{1}{4}\pi\, d^2} \le R \qquad (9)$$

where R represents a design value of the tensile strength of an unreinforced concrete.

4 CASE HISTORY

Within the area of excavation pit bottom the VCCs were installed from the existing ground surface at the level of about +1.3 m above the sea. The VCC construction technique (cf. Keller, 1990) employed an electric bottom feed vibrator which penetrated into the bearing sand layer to the required depth. The poker was filled with a low slump concrete and gradually raised and partly pushed down to form in the subsoil a concrete column under the presence of air pressure and vibro-compacting action of the poker.

The bottom area of the tunnel pit, 9.2 m wide and about 55 m long, was secured with 7 rows of VCCs (Figures 3 and 4). Three central rows of 3.5 m long VCCs, installed with a regular spacing of 1.6 m, were provided to stabilize the bottom area between two rows of longer VCCs, located under tunnel side walls, which were primarily designed to carry major part of tunnel load. The length of these columns was adjusted to varying level of the bearing sand layer, while a minimum penetration of 1 m in sand was required. The remaining two rows of VCCs, positions along pit walls, were extended above the bottom of excavation to the level of -1.4 m and formed short retaining walls. For the whole bottom area stabilized with VCCs the most critical values of the design conditions, defined by Equations (2), (7), (8) and (9), were as follows: $F_S = 1.10$, $\tau_1 = 4$ kPa, $\tau_2 = 9$ kPa and $\sigma_R = 93$ kPa.

At the location of a small pumping station in the central part of the tunnel, where a 9 m long and 4 m wide trench to the depth of -4.5 m was required, sixteen VCCs were installed with a length of 4 m below the trench bottom. In this case the condition described by Equation (7) governed the design, since the clay layer below the trench bottom was only 0.8 m thick. In addition to the stabilizing action of VCCs, local dewatering system comprising of several ∅50 mm wellpoints was applied at the trench periphery. With a corresponding reduction of the design water head of 0.7 m the following design parameters were evaluated for the trench bottom: $F = 1.20$, $\tau_1 = 33$ kPa, $\tau_2 = 10$ kPa and σ_R=260 kPa. Laboratory tests yielded concrete class between B15 and B25, consequently R_{min} =590/2=295 kPa.
Calculations of tunnel slab stability against uplift indicated that additional anchorage bearing capacity was required at the north and south entrance to the tunnel. This pull out capacity was provided with 3 reinforced concrete columns (RCCs) at each end,

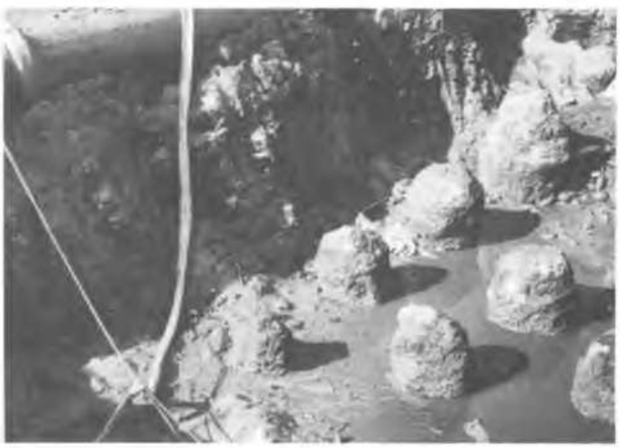

Figure 5. Excavation works in progress.

Figure 6. Building pit after excavation to the final depth.

installed with the same equipment. In this case ordinary concrete mixture B25 was used to fill the poker and a reinforcing bar I120 was inserted into the fresh column.

Excavation works for the tunnel pit started 4 weeks after execution of the described layout of VCCs, and after completion of VCCs used to support the carriageway embankment on both sides of the tunnel. A view of the pit bottom in an intermediate and final phase of excavation is shown in Figure 5 and 6, respectively. The entire bottom was stable, while most of the VCCs had to be trimmed to the required level. At two locations on the bottom water inflow was larger than expected and surface pumps had to installed. The reasons were not exactly known, however in one case an old wooden pile was probably pulled out from the subsoil.

5 CONCLUSIONS

The most important criteria for the outlined design are the column spacing and length, thickness and continuity of the impervious bottom layer and the effective hydraulic head. If too long columns are applied also the tensile stress may appear higher than the reduced concrete tensile strength.

For particular applications an optimalization study should be performed with respect to the total cost of bottom stabilization with VCCs in relation to conventional dewatering and other possible solutions. It seems that the proposed stabilization method can be competitive in situations where initial stability is not lower than F=0.5.

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