CALCULATIONS VERSUS MEASUREMENTS OF THE DIAPHRAGM WALL TRENCH STABILITY AND OF THE DEFORMATIONS

COMPARAISON DES CALCULS ET DES MESURES DE LA STABILITÉ ET DES DÉFORMATIONS D’UNE TRANCHEE DE PAROI MOULÉE

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ABSTRACT – The paper reviews different analytical and numerical practical verification methods of trench stability, focusing on the models which take into account tridimensional effects and arching phenomena appearing in the ground. The analytical methods reviewed in this paper, being the basis of the diaphragm wall trench stability comparative calculations, are: plain strain state, arching effect in vertical plane (Schneebeli Method), arching effect in horizontal plane (Plaskowski-Kowalewski and Huder Methods). The numerical calculations presented in this paper were done using the FEM software PLAXIS 3D, considering two soil behavior models: Mohr-Coulomb and Hardening Soil. The paper compares the results of the trench stability calculations for two diaphragm walls with different lengths and the same depth, performed in alluvial soils in Bucharest: 2.7 m and 6.2 m length and a depth of approx. 30 m. For comparison, the results of the inclinometer measurements regarding the displacement of the soil close to the trench are presented.

1. The impact of the diaphragm walls performance on adjacent constructions

During the last ten years, numerous buildings with several underground levels below ground water level were designed and constructed. These require excavations supported by diaphragm walls (d-walls) even deeper than 30 m. Frequently, the d-walls are constructed in the immediate vicinity of existing constructions, some of them being very sensitive to additional settlements. Thus, the verification of the trench stability during the excavation for the d-wall panels becomes very important. Also, the deformations outside the excavations that might affect the adjacent constructions have to be minimized.

From the construction point of view, d-wall panels having large lengths of 5 m to 8 m are desired in order to reduce the time and the cost of the construction as well as the risk of infiltrations through the construction joints.

Consequently, the general excavation shall be sustained by d-walls with a perimeter of approx. 1,000 m.

On the excavation area close to an already existing building, the d-walls have a thickness of 1.00 m and a depth of about 30 m. The toe of the d-walls is embedded in a clayey layer.

2. Case study: deep excavation for Dâmbovița Center Complex, Bucharest, Romania

The designed complex is a set of commercial, socio-cultural and housing constructions, to accommodate commercial spaces, conference rooms, hotels, offices, recreation areas etc., totaling over 650,000 m². It also includes the building remained unfinished "National Museum", later renamed "Radio House".

A large infrastructure construction is provided, having a foot print of approx. 5 ha, with 6 underground levels with approx. 6,500 underground parking spaces.

In order to speed up the construction process, the contractor proposed a larger value of the d-wall panel length i.e. 6.20 m instead of 2.70 m.

Detailed calculations were performed in order to ensure the trench stability and minimal deformations in the adjacent foundation ground.

Below are summarized the results of the verifications regarding the trench stability.

In order to confirm the trench stability verifications, an in situ monitoring program during construction of the panels was adopted. Thus, five inclinometer columns were installed adjacent to the...
d-wall, outside the excavation, down to 50 m depth (20 m below d-wall toe level).

3. Overview of the used methods

3.1. Simplified method. Plane strain state analytical method

Trench stability is verified based on the pressures which act on its face and their resultants:
- \( P_a \) – active earth pressure resultant;
- \( P_w \) – pore water pressure resultant;
- \( P_s \) – slurry pressure resultant.

\[
\sigma_z = \frac{\gamma l}{\sin 2\phi} \left( 1 - \frac{n}{2} \sin 2\phi \right) \quad (3)
\]

\[
n = \frac{z}{L} \quad (4)
\]

According to Schneebeli, the value of the ground vertical efforts on the trench face is (Fig. 3):

\[ p_a = \tan^2 \left( 45^\circ - \frac{\phi}{2} \right) \cdot \sigma_z - 2c \cdot \tan \left( 45^\circ - \frac{\phi}{2} \right) \quad (5) \]

According to Schneebeli’s theory, each point on the trench face is in equilibrium, if the slurry pressure is higher than the interior earth pressure (in case of an active earth pressure caused by a complete mobilization). Schneebeli considers this theory is also valid for cohesive soils.

3.2. Arching effect in vertical plane analytical method (Schneebeli Method)

The method, which considers the “silo effect”, presumes that, in the area of the trench, the soil volume displaces vertically. Thus, the geological stress is reduced adjacent to the trench and, as a result, the lateral earth pressure decreases.

3.3. Arching effect in horizontal plane analytical method (Piaskowski - Kowalewski Method)

Piaskowski – Kowalewski theory assumes that, in a d-wall trench, the arching effect is similar to the behaviour of a tunnel arch.

3.4. Arching effect for short trenches analytical method (Huder Method)

Huder (1972) proposed a reduction factor of the active earth pressure for short trenches (Xanthakos, 1994).
\[ A = \frac{1 - \exp(-2nK_a \tan \phi)}{2nK_a \tan \phi} \]  
\[ \text{where,} \]
\[ n = \frac{h_r}{l} \]  
\[ p_a = [\gamma h_u + \gamma' (h_c - h_u)] A K_a \]

3.5. Numerical verification method

For these verifications, Mohr-Coulomb and Hardening Soil constitutive models of PLAXIS 3D software were used. The d-wall trench was excavated completely in one step in order to compare the results of numerical verifications with the analytical ones, in terms of safety factors.

4. Comparative verifications

The results of the verifications carried out using the above mentioned methods for the d-wall trench stability with in plane lengths of 2.70 m, respectively 6.20 m are concluded below. The geotechnical parameters are presented in Figure 1. The unit weight of the bentonite slurry was considered \( \gamma_s = 11 \text{ kN/m}^3 \).

4.1. Plain strain state verification

It should be mentioned again that this method do not consider the effect of the panel in plane length. Equation (2) is not fulfilled for all panel depths. According to equation (1), it results a global safety factor \( F_s = 0.32 \).

4.2. Analytical verification considering arching effect

Two in plane panels lengths of 2.70 m and 6.20 m were considered (Figure 5 and 6).

Table 1 and 2 present the values of the forces acting on the trench face and of the global safety factor according to equation (1). The values of the resultant forces \( P_w, P_c, P_a \), were calculated for 1 m length of the d-wall panels.

Table 1. The obtained safety factors

<table>
<thead>
<tr>
<th>Panel length (m)</th>
<th>Schneebeli Method</th>
<th>Piaskowski-Kowalewski Method</th>
<th>Huder Method</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>( P_a ) (kN/m)</td>
<td>( F_s )</td>
<td>( P_a ) (kN/m)</td>
</tr>
<tr>
<td>2.70</td>
<td>298</td>
<td>5.04</td>
<td>747</td>
</tr>
<tr>
<td>6.20</td>
<td>580</td>
<td>2.59</td>
<td>2,454</td>
</tr>
</tbody>
</table>

Table 2. Water and slurry resultant forces

<table>
<thead>
<tr>
<th>Panel length (m)</th>
<th>( F_w ) (kN/m)</th>
<th>( P_s ) (kN/m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>2.70</td>
<td>3,125</td>
<td>4,625</td>
</tr>
<tr>
<td>6.20</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

4.3. Numerical verification

Based on the geotechnical parameters (Saidel et al., 2010) for Hardening Soil and Mohr-Coulomb constitutive models, a minimum safety factor of 1.5 resulted. The safety factor was selected for minimal
deformations, below 5 mm (comparable with in situ measurements) on a linear part of the curve safety factor-deformation (Figure 8).

Figure 7. PLAXIS 3D model 2.70 m d-wall trench and failure mechanism on a 6.20 m d-wall trench

Figure 8. Curve safety factor-deformation considering 6.2 m trench length and HS model

5. In situ measurement. Conclusions

Figure 9 (Râileanu et al., 2012) presents the evolution of the displacements adjacent to a 6.20 m trench for the following stages: "zero" measurement, trench excavation, introducing the steel cage and after concreting.

Maximum displacement of $\delta_{\text{max}}=6$ mm was measured resulting the ratio between displacement ($\delta_{\text{max}}$) and d-wall depth (H) of $2 \times 10^{-3}$.

Comparing above mentioned measurement results with the displacement values of a d-wall required to mobilize the active earth pressure

$$\delta=(0.5...2) \cdot 10^{-3} \cdot H,$$  \hspace{1cm} (9)

it results that in the ground close to the d-wall trench, the stress state corresponds to a linear behaviour of the soil and the deformations are reduced (inducing insignificant settlements of the adjacent constructions).

As a conclusion, considering for the analytical methods a global safety factor $F_s = 1$ and for the numerical models a safety factor of $F_s = 1.5$, both the trench stability and the limiting deformations for the adjacent usual buildings are fulfilled even for 6.20 m panel lengths.

The practical verification methods of the trench stability which take into account the arching effects were validated. Also, the importance of the in situ measurements, which reveal the real deformations, was demonstrated.

Figure 9. Horizontal deformations measured near the open trench

6. References


Saidel T., Căpraru C., Marcu A. (2010). Influence of constitutive laws and geotechnical parameters on deep excavations design, examples from recent projects in Bucharest Proceedings of the XIVth Danube-European Conference on Geotechnical Engineering

