ON SAFETY OF SLURRY-WALL TRENCHES

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Abstract: The paper focuses on a numerical simulation of ground movements and a stability evaluation during the trench excavation process (slurry-walls technology). A special attention is paid to the modelling of failure and to the defining of safety measures. Numerical calculations are based on the FLAC 3D code. It is found that the soil kinematics at failure coincides with the literature data. Three kinds of safety factor are investigated – all results converge if trench is deep enough. In addition, the applied probabilistic safety analysis takes into account the random level of the ground water, as a most important factor, and the soil strength. The acceptable level of safety is found.

1. INTRODUCTION

An increasing interest towards applications of slurry walls can be observed since the late sixties. It is so, because geotechnical engineers cope with more and more complicated ground situations, including water and soil conditions, as well as construction sites situated in urban regions. Bearing in mind a very limited space in the city infrastructure, deep excavations are required by needs of underground parking places, protection of existing objects in the excavation vicinities, large loadings transmitted from the ground and from the structure, etc., the concept of vertical rein-forced-concrete panels is both technically effective and economically justified.

The paper focuses on the first phase of the technology when a vertical finite--length trench element is excavated under a hydrostatic support from inside of a bentonite slurry. It seems that a little interest is paid to the static analysis and safety margins of the trenches (as well as a surrounding ground), except for – but a few – recent papers. During the trench excavation some dangerous consequences can result from the variations of the groundwater table, the pore pressure, the soil strength, the slurry level, weak soil lenses, suction forces, etc. All of them are random, sometimes correlated, and significantly influence the required level of safety.

In the stability evaluation, a half-elliptic sliding block method introduced by KOWALEWSKI and PIASKOWSKI [1] is often considered (figure 1a) or a 3D method of columns which corresponds to the method of 2D slices (figure 1b). Methods resulting from the Janssen silo-pressure analysis, adopted by Terzaghi and Schneebeli, become less popular. Three parameters are very important for the local stability of the trench: the isotropic supporting stress p_z caused by the bentonite slurry, the hydrostatic pressure p_w due to the groundwater table and the effective horizontal soil pressure σ' in

soils near the face of the trench. Note that standard engineering measures of safety, like the local ratios $FS = p_z/(\sigma' + p_w)$ or $FS = (p_z - p_w)/\sigma'$, are not useful in the context of advanced numerical calculations. Indeed, the forces acting on all faces of the trench have to be always in equilibrium (boundary conditions formulated in stresses).
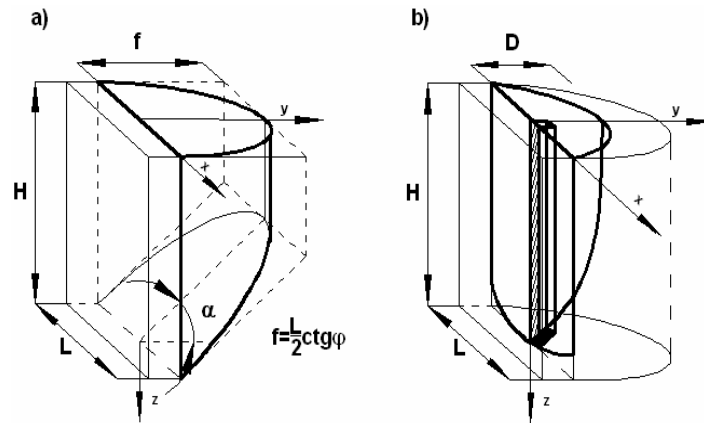

Fig. 1. Shape of the sliding wedge by: a) Piaskowski and Kowalewski; b) Tsai and Chang

A full elastoplastic stability evaluation using the 3D finite difference method for a continuum is the objective of the paper. The shape of the sliding wedge is to be verified and margins of safety are to be estimated.

2. ASSUMPTIONS FOR THE NUMERICAL STUDY

A vertical trench, 1 m thick and 10 m deep ($H = 10$ m), with the typical length of 6 m ($L = 6$ m), is considered. Due to two planes of symmetry, only one quarter of the 3D boundary problem is of interest, figure 2a. The soil model corresponds to a macrohomogeneous sand, the elastoplastic material with the Coulomb–Mohr yield criterion and the nonassociated rule of plastic flow. The geotechnical parameters are as follows: the unit weight $\gamma = 18.5$ kN/m^3, the earth pressure coefficient at rest $K_0 = 0.47$, the friction angle $\varphi = 32°$, the cohesion $c = 0$ kPa, the dilation angle $\psi = 0°$, Young's modulus $E = 70$ MPa and Poisson's ratio $\nu = 0.25$. The groundwater table varies from 0.5 m to 3.0 m below the ground surface. It is assumed that there is no hydraulic contact between the bentonite suspension and the groundwater. Neither flow nor changes in the pore pressure are taken into consideration during the trench excavation.

The model domain is divided into four parts of different zone concentrations: fine ones near the excavation (minimal mesh distance of 0.5 m) and coarse ones near the corners (maximal mesh distance of 2.0 m), figure 2b.
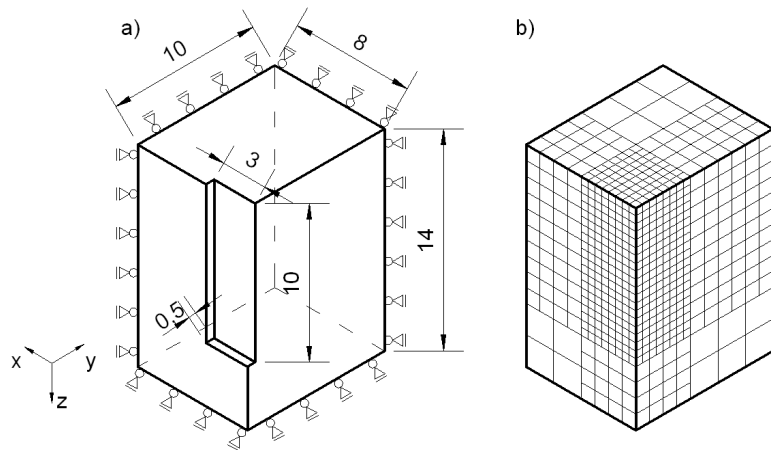
Fig. 2. The geometric assumptions:
a) dimensions of the domain of calculation and boundary fixities; b) a finite difference mesh

The excavation process is modelled by a successive removing of 1 m-thick ele-ments from the mesh. The hydrostatic slurry pressure is taken as an external loading, linearly increasing with depth, applied to all faces of the trench, including the toe. Calculations were performed in terms of the effective stress (a fully drained case). Dynamical forces which are produced during working movements of the bucket of a trenching machine are not taken into account. They do not have much influence, if the length of the bucket is much smaller than the length of the trench (sometimes, contractors do not confirm this opinion). For simplification, the groundwater pressure is subtracted from the slurry pressure. Thus only the reduced boundary loading from both fluids is applied to all faces and to the toe of the trench. The bentonite slurry with the unit weight of 10.5 kN/m^3 is used as the stabilizing fluid and the value of 10.0 kN/m^3 is accepted for the unit weight of water. The slurry level is kept un-changed on the ground surface, i.e., above the groundwater table. The code FLAC 3D is used for numerical calculation.

3. DETERMINISTIC MEASURES OF SAFETY

The stability of the trench is basically investigated by the reduction of $\tan\varphi$ (FS_1) as well as by the reduction of the unit weight of the bentonite slurry (FS_2) and by the increase of the unit weight of the surrounding sand (FS_3). Three different factors of safety are pro-posed, all comparing real parameters with the reduced (increased) ones at failure:

$$FS_1 = \frac{\tan\varphi}{\tan\varphi_{\text{red}}} \quad \text{– reduction of the ground strength parameter (if cohesion } c = 0\text{),}$$

$$FS_2 = \frac{\gamma_z}{\gamma_{red}}$$ – reduction of unit weight of the bentonite slurry,

$$FS_3 = \frac{\gamma_{gzw}}{\gamma_g}$$ – increase of unit weight of the surrounding sand.

The terms "reduction" and "increase" mean successive 5% changes of the parameters in each stage during calculations. The calculations provide the evidence that the 5% level can be accepted as a compromise between time of calculations and required accuracy.

The *FS*-calculations are done for every increased trench depth (from 1 m to 10 m, 1 m step). The response of the trench during every stage of the excavation, i.e., for any fixed excavation depth, is monitored when the reduction procedure is continued until the trench collapses. For a ductile elastoplastic behaviour, it is not evident when such a failure takes place. Therefore, it is assumed that the failure means a "rapid increase" of displacements of one or more points selected on the face of the trench. For simplification, only the ground-level point on the plane of symmetry can be selected, where a maximal displacement is expected. During the successive reduction of $\tan\varphi$, the evaluated displacement speeds up starting from a certain value denoted by u_{lim}. This finding yields from the observations of the displacement curve $u = u(\tan\varphi)$ and corresponds to a point of its largest curvature (second derivative) which has to be found. Alternatively, two following tangent lines intersect near this point: the almost horizontal one – fitting the first part of the displacement curve, and the almost vertical one (close to the state of plastic yielding).
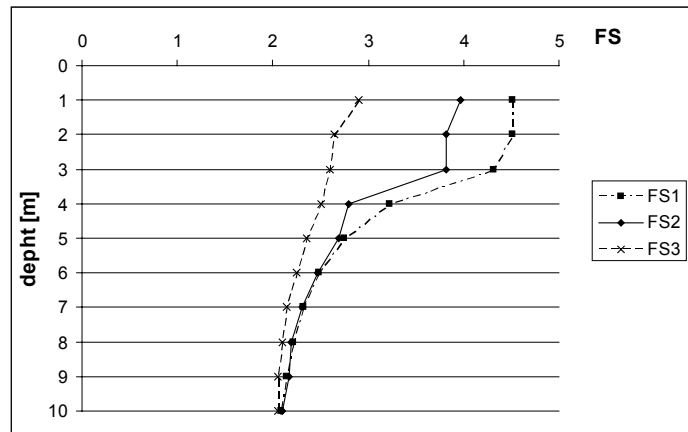

Fig. 3. The dependence of the safety factors FS_i on the depth of excavation [m]

Figure 3 presents the dependence of the safety factors *FS* on the depth of excavation. For shallow trenches, the differences between the safety factors FS_i are quite

significant, but the differences vanish, if the values of FS_i become small. The calculations in this section and the results in figure 3 correspond to the fixed groundwater table which is situated 2.0 m below the ground surface.

Although based on the same concept and leading to similar results, the coefficients FS_2 and FS_3 seem to be more controversial than FS_1. Indeed, even highly reduced values of $\tan\varphi$ are physically acceptable, in contrast to small values of the unit weight of the bentonite slurry (say, less than 7 kN/m^3) or large values of the unit weight of the soil (say, more than 30 kN/m^3). Note a kind of (almost) stabilisation of FS_i values with depth which can reflect some silo-effects.

A spatial distribution of displacements within the sliding wedge, just before the failure due to the reduction of $\tan\varphi$, is presented in figure 4. The results confirm kinematics proposed by TSAI and CHANG [3] (figure 1b) more than the one used earlier by PIASKOWSKI and KOWALEWSKI [1] (figure 1a).
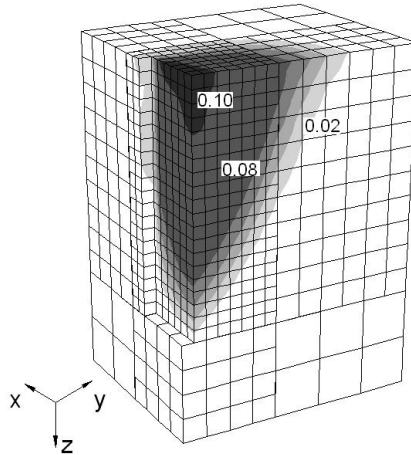

Fig. 4. The displacements of the continuum within the sliding wedge near the trench [m]

4. RELIABILITY INDEX. CALIBRATION

Assume that the coefficient of friction $f = \tan\varphi$ is random, has an expected value μ_f and a standard deviation σ_f. Define the following standardized random variable $Z_1 = (f - \mu_f)/\sigma_f$. Similarly, $Z_2 = (t - \mu_t)/\sigma_t$ stands for the depth of the random groundwater table, $t > 0$. If so, the limit state condition, expressed in terms of the displacements of a representative point, takes the form $U(Z_1, Z_2) = u_{\lim}$. Numerical calculations revealed that the dependence on the value of $u_{\lim}$ is not very important, if $u_{\lim}$ is large enough (steady plastic-flow zones). The mechanical assumptions justify a probabilistic simplification that Z_1 and Z_2 are statistically independent. Let $\mu_f = 0.55$, $\sigma_f = 0.055$ (the

coefficient of variation denoted by $v_f = \sigma_f/\mu_f = 0.055/0.50 = 10\%$) and $\mu_t = 2.0$ m, $\sigma_t = 0.5$ (i.e., $v_t = \sigma_t/\mu_t = 0.5/2.0 = 25\%$). The data can reflect much greater variability of the water conditions due to ineffective drainage system, floods and so on. The limit state condition for $u_{\lim} = 0.015$ m is plotted in figure 5, making use of selected results that yield from the numerical analysis ($H = 10$ m).
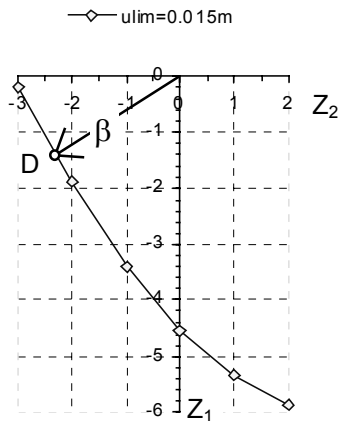

Fig. 5. The Hasofer and Lind reliability index
$\beta \cong 2.7$ and the design point D,
$D(-1.4; -2.3) = D(-\beta \cdot \cos\alpha, -\beta \cdot \sin\alpha)$, where $\alpha = 58.6°$

The shortest distance β from the origin $(0,0)$ to the limit state line in the space of the variables $(Z_1; Z_2)$ is a standard measure of safety, THOFT-CHRISTENSEN and BAKER [2]. Since the limit state function U is almost linear around the design point D, the probability of failure can be approximated simply by:

$$p_f = \Phi(-\beta), \quad \text{so} \quad p_f \cong 0.0035.$$

The obtained values of β and p_f are relatively small in the context of the standards of structural reliability, but they seem to be acceptable for short-time construction with ductile behaviour and plastic reserves of strength. In addition, the limit condition in displacements $u_{\lim}$ is more conservative than the one expressed in terms of bearing capacity forces.

The design point $D(z_1^d, z_2^d)$ determines two design values of the physical parameters:

$f^d = (\tan\varphi)^d = \mu_f + z_1^d \cdot \sigma_f = \mu_f \cdot \gamma_f$ where $\gamma_f = 1 + z_1^d \cdot v_f = 1 - \beta \cdot v_f \cdot \cos\alpha = 1 - 1.4 \cdot 10\% = 0.86 = 1/1.16$,

$t^d = \mu_t + z_2^d \cdot \sigma_t = \mu_t \cdot \gamma_t$ where $\gamma_t = 1 + z_2^d \cdot v_t = 1 - \beta \cdot v_t \cdot \sin\alpha = 1 - 2.3 \cdot 25\% = 0.43 = 1/2.35$.

The results in terms of the partial safety factors γ_f and γ_t need a comment. The dominating role of the groundwater table localization is evident: $\gamma_t \ll 1$. This happens due to the greater sensitivity of the model subject to changes of the groundwater table than to random fluctuations of $f = \tan\varphi$.

Formally speaking, this conclusion results from the design-point coordinates for which $|z_2^d| = 2.3 > |z_1^d| = 1.4$. Furthermore, it is the large variability of the groundwater table depth t that leads to such a conclusion ($v_t = 25\% > v_f = 10\%$). Apparently, the partial safety coefficient $\gamma_t = 0.43$ is very low in the considered example. However, this coefficient is to correct the expected value μ_t which is greater than the characteristic value accepted sometimes as a safe probabilistic 95%-quantile. Bearing this in mind, the comparison with the EC7 guidelines becomes not straightforward, unless a full information on probability distributions is available.

Note that such elements as foundations situated near the trench, random localisation of soft-soil lenses, a pore pressure generation, etc., can change the conclusions.

5. CONCLUSIONS

a) The numerical calculations indicate that the safety of the trench supported by the bentonite slurry decreases with its depth. However, the changes are not very significant below the depth of 7 m and a kind of convergence is observed. Such a finding can correspond to a silo-effect.

b) Relatively large values of the factors of safety are observed for deterministic situations, like $FS = 2 \div 3$ or more. This conclusion can be misleading to a certain degree, because some important factors are not taken into account: a fluid flow and changes in the pore pressure, technological excitation by movements of the bucket of trenching machine, soil inhomogeneities, foundations, etc.

c) There is no doubt that the localisation of the groundwater table – related to the level of the slurry in the trench – is the pivotal parameter of the model.

d) The standard safety factor FS_1 (the relative reduction of soil-strength parameters) seems to be the most coherent and representative for safety estimation within the deterministic approach.

e) Standard probabilistic analysis of safety, expressed in terms of the reliability index β, reveals that the large values of FS in the deterministic approach can be overestimated.

f) The observed kinematics of sliding wedges confirms engineering assumptions within the limit state approach, especially the ones presented by TSAI and CHANG [3].

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