# Innovative solutions for supporting excavations in slopes

## Solutions innovantes pour le soutien d'excavations situées dans des terrains en pente

Lüftenegger R.
*GDP ZT-OG, Austria*

Schweiger H.F., Marte R.
*Institute for Soil Mechanics and Foundation Engineering, Graz University of Technology, Austria*

ABSTRACT: The design of support measures for deep excavations is one of the key tasks in geotechnical engineering. The choice of the most appropriate support system depends on various obvious factors such as ground conditions and excavation depth but sometimes also on less obvious boundary conditions, for example when construction of ground anchors is not possible because permission of placing them in neighbouring property is not given. In these cases other options have to be pursued, resulting sometimes in non-conventional solutions. Examples for such innovative support systems are presented in this paper. In the first case the arching effect of the retaining structure was used to design an excavation pit without any anchors reaching on the neighbouring ground, because there was no permission for construction elements there. The 6 meter spanned arches consist of mixed in place columns (MIP), which rest on supporting walls (also mixed in place columns) oriented in the direction of the slope. In the second example the behaviour of a serrated sheet pile has been investigated. Comprehensive 3D finite element analyses have been performed on order to prove that the suggested retaining structures are feasible solutions.

RÉSUMÉ : La conception des mesures de soutènement pour les excavations profondes est une des tâches fondamentales dans la géotechnique. Le choix du système le plus approprié dépend de plusieurs facteurs évidents comme les conditions de sol ou la profondeur de l'excavation, mais parfois aussi de contraintes moins évidentes comme par exemple le cas où des ancrages ne sont pas possibles parce qu'il n'y a pas d'autorisation pour l'installation dans la propriété voisine. Dans ces cas, d'autres options doivent être envisagées, qui résultent parfois dans des solutions non-conventionelles. Quelques exemples de telles solutions sont présentés. Dans le premier cas, l'effet de voûte de la structure de soutènement a été utilisé pour la conception d'une excavation qui bordait une propriété pour laquelle il n'y avait pas d'autorisation pour l'installation des éléments d'ancrage. Les voûtes avec une portée de 6 mètres ont été réalisées avec des colonnes « Mixed in Place » (MIP) qui prenaient appui sur des parois orientées dans la direction de la pente. Dans le deuxième exemple, le comportement d'une palplanche dentelé a été examiné. Des analyses par la méthode des éléments finis 3D compréhensives ont été effectuées afin de prouver que la structure de soutien proposée était une solution réalisable.

KEYWORDS: deep excavation, finite element method, three-dimensional analysis.

## 1 INTRODUCTION

The design of support measures for deep excavations is one of the key tasks in geotechnical engineering and, depending on soil conditions and adjacent infrastructure, many different options exist. One of the most difficult situations to overcome is when space for support measures is limited and due to legal reasons support elements such as ground anchors cannot be built on neighbouring ground. The obvious solution in these cases, namely putting struts, is often not very convenient for the excavation process and sometimes even not possible, e.g. if the excavation is situated in a slope. These cases require special attention and two case histories where innovative solutions have been found are presented in this paper.

## 2 NUMERICAL ANALYSIS

In order to demonstrate the feasibility of the proposed design and to assess expected deformations a number of three-dimensional finite element analyses have been carried out. These analyses also served as basis for the design of the structural elements. The finite element code Plaxis 3D Foundation has been used for all analyses presented in this paper (Brinkgreve and Swolfs 2007).

It is well established that for this type of analysis simple linear elastic-perfectly plastic constitutive models are not very well suited and therefore a more advanced model, namely the Hardening Soil model, has been employed. This model is a so-called double hardening model and allows for plastic compaction (cap hardening) as well as plastic shearing due to deviatoric loading (friction hardening). The main features of this model, as implemented in Plaxis, can be summarized as following.

- Stress dependent stiffness according to a power law.
- Plastic straining due to primary deviatoric loading.
- Plastic straining due to primary compression.
- Elastic unloading / reloading.
- Failure according to the Mohr-Coulomb criterion.

A more detailed description of the Hardenings Soil model can be found e.g. in Schanz et al. 1999.

## 3 CASE HISTORY 1 – MIXED IN PLACE COLUMNS

The first example is concerned with an excavation situated in a slope, just below existing buildings. The owner of one of the buildings was particularly alerted because he experienced significant damage to his building in the past due to nearby construction activities. He did not allow ground anchors to reach his property. Thus the task was to stabilize the excavation without ground anchors and at the same time provide sufficient support to keep deformations, which could lead to damage of the building located above the excavation, to an absolute minimum. This could be achieved by arches of 6 meter span constructed by mixed in place columns (MIP), which rest on supporting walls (also mixed in place columns) oriented in the direction of the slope. The earth pressure exerted from the slope was transferred to 5 meter deep mixed in place walls underneath the planed building at the base of the slope. Figure 1 shows the slope with the supporting structure and Figure 2 a detail of the MIP columns.

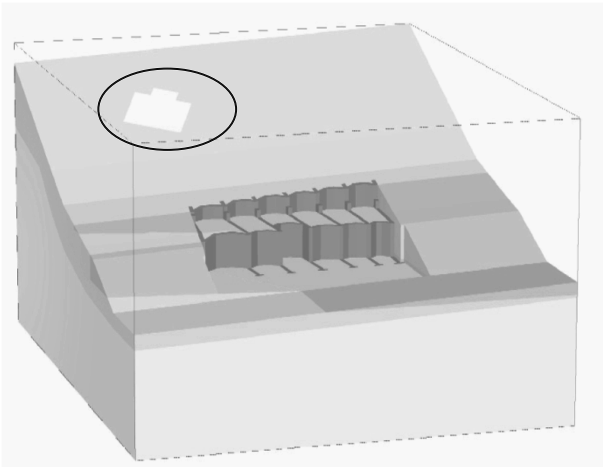

Figure 1. Overview of slope and support structure including critical building.

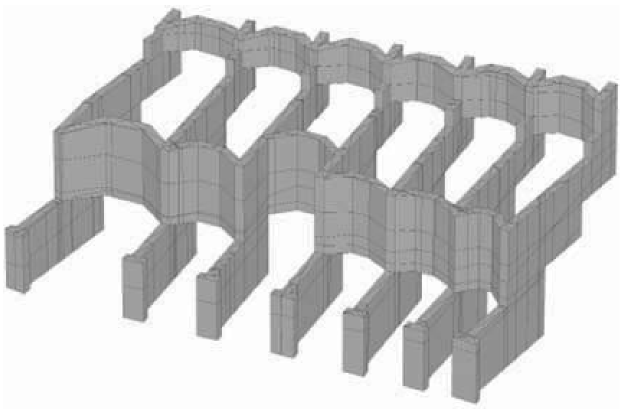

Figure 2. Layout of support structure (MIP columns).

Based on the results from site investigations a representative underground model consisting of three layers was established for the 3D finite element analysis, namely soft sandy silt (0-4 m below surface), stiff laminated sand-silt (4-8 m below surface) and semi-solid sand-silt (below 8 m from surface). The most important parameters for these layers are summarized in Table 1. $E_{oed}^{ref}$ is the stiffness from an oedometer test for the reference vertical effective stress of 100 kPa, $E_{50}^{ref}$ is a secant stiffness at 50% of maximum deviatoric stress in a triaxial compression test at a reference cell pressure $\sigma_3'$ = 100 kPa, $E_{ur}^{ref}$ is the unloading/reloading stiffness, again at a reference cell pressure of 100 kPa from a triaxial test, and m is a parameter determining the stress dependency of above stiffness parameters. φ', c' and ψ are the conventional Mohr-Coulomb strength parameters which define ultimate strength in the Hardening Soil model.

The MIP-method improves the mechanical properties of a soil by mechanically mixing and adding binder slurry. The result is a “soil-concrete-mixture” in which the soil is used as aggregate. For the MIP-columns a constitutive model based on the Mohr-Coulomb failure criterion was applied. Based on an unconfined compressive strength of 5 MN/m², whereas this value includes a partial factor of safety on material strength, the material parameters listed in Table 2 have been adopted.

Table 1. Material parameters for Hardening Soil model for soil layers.

| Parameter | *Layer 1* | *Layer 2* | *Layer 3* |
|---|---|---|---|
| Friction angle, φ' (°) | 25 | 27.5 | 30 |
| Cohesion, c' (kPa) | 0 | 1 | 5 |
| Dilatancy angle, ψ (°) | 0 | 0 | 0 |
| Unit weight, γ (kN/m³) | 20 | 20.5 | 21 |
| $E_{oed}^{ref} = E_{50}^{ref}$ (kPa) | 10000 | 25000 | 45000 |
| $E_{ur}^{ref}$ (kPa) | 30000 | 75000 | 135000 |
| m (-) | 0.5 | 0.5 | 0.5 |

Table 2. Material parameters for MIP-columns.

| Parameter | *MIP* |
|---|---|
| Friction angle, φ' (°) | 30 |
| Cohesion, c' (kPa) | 250 |
| Unit weight, γ (kN/m³) | 22 |
| Elasticity modulus (kPa) | 300000 |
| Tension cut off,* $\sigma_t$ (kPa) | 125 |

* based on reinforcement by steel rods and nails

The results of the calculation show the expected stiff behaviour of the chosen support system. The maximum calculated horizontal deformation of about 15 mm occurs at the front upper corner of the lower excavation step (Figure 3). At the back of the wall (near the border of the neighbouring property) deformations are in the order of millimetres and thus the expected settlements in this area can be considered to be not significant and will not cause any damage to the building (Figure 4). However, the finite element analysis could not model the construction process of the MIP-columns, i.e. the columns were assumed "wished-in-place" before excavation starts and therefore displacements due to the construction process have to be added to these values.

The 3D-model was also used to check the tension zones in the MIP-body. The main tension stresses were located at the connections of the arches and the wall elements. In this area the MIP-wall was reinforced with steel beams (HE-B profiles).

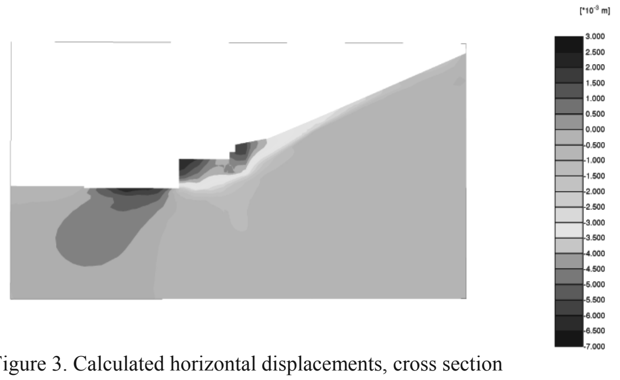


Figure 3. Calculated horizontal displacements, cross section

The measurements during construction on one hand confirmed the results for the numerical analysis but on the other hand showed that significant deformations occurred during construction of the MIP-columns itself (Figure 5). After construction of the columns (panels) deformation measured were less than 15 mm, comparing well with the finite element predictions. Figure 5 shows the deformations of different points on the top of the MIP-wall. At the neighbouring buildings no movements were recorded. Figure 6 presents a view of excavated MIP-walls.

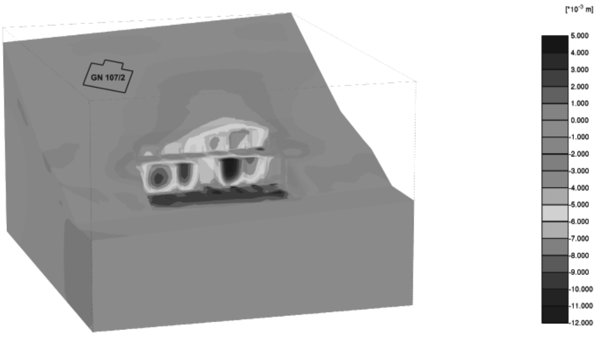


Figure 4. Calculated horizontal displacements with critical building

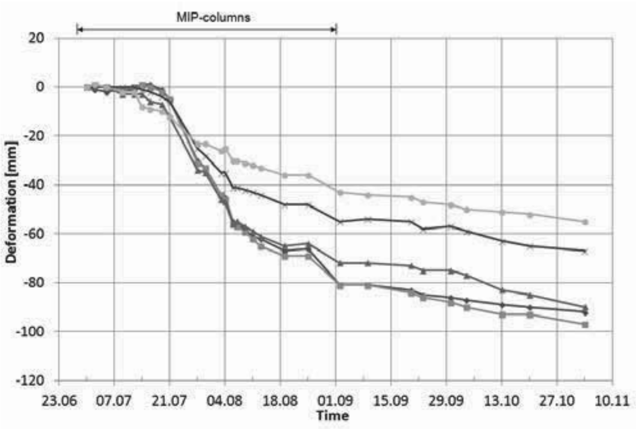


Figure 5. Measured horizontal displacements at several points at the top of the MIP-wall

Figure 6. View of excavated MIP-walls

The large deformations during the production of the columns had two main reasons. In the first part of the production too many MIP columns were produced within a small area. The MIP columns, which take some time to gain strength, weakened the slope during construction, which was already close to critical state. Furthermore, to reduce the length of the MIP columns (in order to save money), deeper working planes than planned were excavated.

This presented case study clearly shows that it is possible to support the earth pressure exerted from a slope by arches constructed by means of soil improvement techniques without any anchors reaching on neighbouring ground. The numerical analysis was able to prove that the design concept is feasible, however, it is important to observe the deformation during the construction stages because not all aspects of the construction process, in this case of the MIP-columns, can be taken into account in the numerical model.

## 4 EXAMPLE 2 - SERRATED SHEET PILE WALL

The second example is concerned with the same problem, namely limited space for support measures, but this time it is in an urban environment, namely in the city of Salzburg, Austria. Again the excavation was very close to the adjacent property and it was not allowed to put any construction elements, such as ground anchors, there. In this case the solution chosen was a serrated sheet pile wall. Generally, the subsoil conditions in Salzburg consist of a top layer with backfill and gravel, and soft silty sand and clayey silt layers underneath. The layout of the sheet pile wall follows from Figure 7 (3D finite element model). Every 6 to 8 m there is a 3 m deep indentation in the sheet pile wall. The construction of diagonal compression and tension bars at the top transfers the earth pressure to the right-angled parts of the sheet pile walls. A steel construction, similar to a whaler beam, on top prevents non-homogeneous deformations of the wall. After excavation a drainage layer and a concrete slab is installed to prevent long term movements of the wall and to reduce the influence of the soft layers below excavation level.

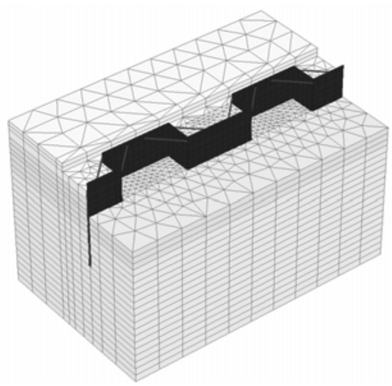

Figure 7. 3D finite element model

The key material parameters for the soil layers considered in the analysis are listed in Table 3. Again the Hardening Soil model has been employed.

Table 3. Material parameters for Hardening Soil model for soil layers.

| Parameter | *backfill* | *silty sand* | *clayey silt* |
|---|---|---|---|
| Friction angle, $\varphi'$ (°) | 35 | 27.5 | 25 |
| Cohesion, c' (kPa) | 0,1 | 3 | 5 |
| Dilatancy angle, $\psi$ (°) | 0 | 0 | 0 |
| Unit weight, (kN/m$^3$) | 19/21 | 20/21 | 18/20 |
| $E_{oed}^{ref} = E_{50}^{ref}$ (kPa) | 52000 | 30000 | 15000 |
| $E_{ur}^{ref}$ (kPa) | 208000 | 120000 | 60000 |
| m (-) | 0 | 0.5 | 0.5 |

The 3D model showed that deformations can be kept to a minimum with maximum values below 10 mm (Figure 8), which was also confirmed by observations during construction. Deformations due to driving and removing of the sheet pile wall are not considered in the analysis. Experience has shown that in this type of soils settlements can reach 20 to 30 mm, and in this particular case observed values were within the lower range.

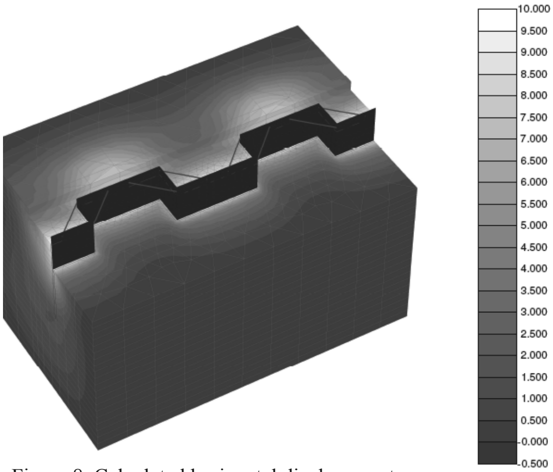


Figure 8. Calculated horizontal displacements

The bending moments of a sheet pile wall with this particular shape and the strengthening construction on top of the wall is not the same as for a cantilever wall, which one would obtain from a 2D analysis and therefore the 3D analysis was essential and helped to estimate the influence of the special support measures. However, 3D finite element analyses are quite time consuming if many different scenarios have to be investigated.

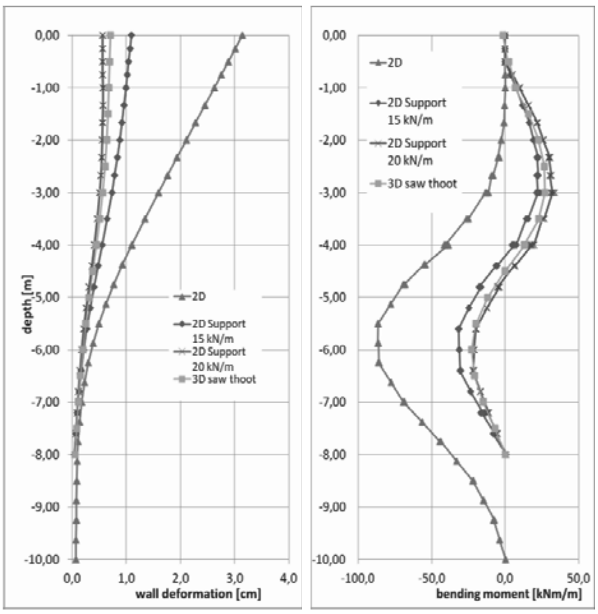


Figure 9. Comparison of 2D and 3D analysis of wall deformation and bending moments

An attempt was therefore made to develop an equivalent 2D analysis for performing parametric studies for a preliminary design. For that reason a 2D model of the sheet pile wall with a supporting force on top of the wall was created. It turned out that for the case of a 8 m deep sheet pile wall and a 4 m deep excavation (groundwater is also at 4 m depth) a supporting force between 15 kN/m und 20 kN/m lead to similar wall deformations und bending moments (Figure 9). This supporting force has to be carried from the additional wall elements spanning across the edges of the two lines of the serrated sheet pile wall (see also Figure 8). The calculations revealed that the earth pressure distribution of the rectangular part of the serrated sheet pile wall is between the active and the at-rest earth pressure (see Figure 10).

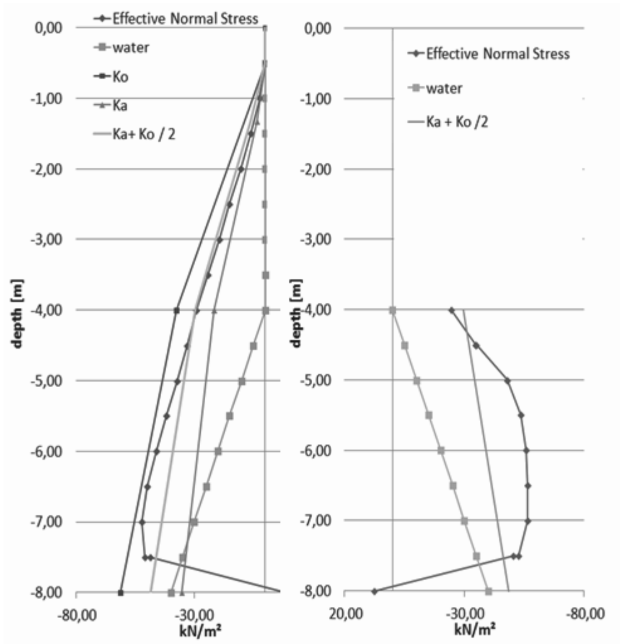


Figure 10. Earth pressure distribution on active and passive side

For the given geometry (distance of 8 m between the rectangular walls) this earth preasure distribution leads to a maximum resistance by wall friction of about about 20 kN/m. This shows a good correlation with the presented calculations.

## 5 CONCLUSIONS

3D finite element modelling allows complex geotechnical structures to be analysed. In the two presented case histories the calculations helped to estimate the arching effect of a curved retaining structure in order to design an excavation pit without any anchors reaching on the neighbouring ground. In the second example the behaviour of a serrated sheet pile has been investigated. In both case the numerical analyses proved the feasibility of the chosen design and improved the understanding how these complex structures behave.

However, even with 3D models it is usually not possible to include all excavation stages in great detail and, more importantly, installation effects are beyond the capabilities of standard numerical tools and this has to be kept in mind when assessing numerical results. Therefore it is essential to monitor the behaviour of the structure during construction and have appropriate counter measures in mind when deformations due to installation effects or unforeseen ground conditions reach critical limits.

## 6 REFERENCES

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