# 3D Finite element analysis of a deep excavation: a case study from Budapest

## Analyse par éléments finis 3D d'une excavation profonde: une étude de cas à Budapest

**Zsolt Szilvágyi PhD** & Olivér Varga
*Geoplan Ltd., Hungary, 28 Pajkos str. H-1119, szilvagyi.zsolt@geoplan.hu*

ABSTRACT: Finite element method is widely used in geotechnical design due to its capability to predict soil deformations and structural forces even in complex cases. However, the number of parameters involved in modelling are very high, and their determination is not always straightforward. The subject of this case study is a deep excavation construction in the historical city center of Budapest. The construction process involved the teardown of an existing building and the renewal and extension of other connected buildings. A three level sub-basement had to be constructed in the close vicinity of existing buildings. Due to the complex layout of the basement, 3D modelling was necessary. The main objective is to present the wide range of application of FEM modelling throughout the project. Settlement prediction of connected buildings, design of the underpinning of existing foundations and a tied back diaphragm wall will be discussed in detail. Importance of proper modelling of small strain stiffness will be demonstrated. Aspects of structural FEM modelling will also be considered. Finally, comparison to monitoring results will be presented.

RÉSUMÉ : La méthode des éléments finis est largement utilisée dans la conception géotechnique en raison de sa capacité à prédire les déformations du sol et les forces structurelles, même dans des cas complexes. Cependant, le nombre de paramètres impliqués dans la modélisation est très élevé et leur détermination n'est pas toujours simple. Le sujet de cette étude de cas est une excavation profonde dans le centre-ville historique de Budapest. Le processus de construction impliquait la démolition d'un bâtiment existant et la rénovation et l'extension d'autres bâtiments connectés. Un sous-sol de trois niveaux a dû être construit juste à côté des bâtiments existants. En raison de l'aménagement complexe du sous-sol, une modélisation 3D était nécessaire. L'objectif principal est de présenter le large domaine d'applications de la modélisation FEM tout au long du projet. La prédiction de l'affaissement des bâtiments connectés, la conception de la reprise en sous-œuvre des fondations existantes et une paroi moulée ancrée seront discutées en détail. L'importance d'une modélisation appropriée de la rigidité aux petites déformations sera démontrée. Les aspects de la modélisation structurelle par éléments finis seront également pris en considération. Enfin, une comparaison avec les résultats de la surveillance sera présentée.

KEYWORDS: 3D finite element modeling, HSSmall, diaphragm wall, excavation support.

## 1 INTRODUCTION

Nowadays the increased need for construction in tightly built urban areas has gained prevalence which together with the limited available surface area in built districts for parking spaces, as well as the high value of underground spaces has resulted in the execution of deep excavations. This task is especially challenging if the built surrounding consists of protected national monument buildings as in our case in the 5th district of Budapest, Hungary. The site in question is located 140 m from the River Danube. Three existing monument buildings with a single level basement and 4-6 stories, will be rebuilt. In some areas only the main facade will be kept with total structural reconstruction, while in other areas only internal structural redesign was decided. The three buildings together frame a yard, where existing structures will be torn down and a 3-level basement will be constructed with reinforced concrete tied back diaphragm walls with an excavation depth of 15.5 meters, locally extending down to 17.0 meters. The buildings were built with shallow foundations and the reconstruction will result in a substantial increase in reaction forces under the walls, hence foundations will be underpinned by Jet-Grouting columns.

Preliminary analytical calculations have suggested significant settlement and tilt of the buildings in question and even surrounding buildings across the street, therefore a detailed 3D geotechnical finite element modeling was chosen to analyze the construction process. The modeling focused on the soil- and structural deformations arising from the construction of the diaphragm wall and excavation as well as structural design of these RC members.

## 2 INITIAL DATA, ANTECENDENTS

In the project permit phase, deep soil investigations inside the yard were unfeasible within the monument buildings; however, the preliminary site investigation report could be based on historical boreholes from the surrounding sites. According to these and geological literature the base rock layer here is upper Oligocene sandy clay which appears with interbedded silty sand and sandstone zones (Burghardt et al. 2020). The area was flooded also after Oligocene age, in the Miocene, while later it was dry land until Pleistocene. After the erosion of the Miocene layers the base rock layer was filled with terrace material of the Danube River, thick sandy gravel, gravelly sand layers were deposited. Further layers were deposited later the area became the flood zone of the river, these are mostly sandy silts, silty sands. Most recently, when the city has emerged, several meters of artificial fills have been constructed upon them.

For the detailed design of excavation support systems, after the teardown of building structures within the yard, additional soil investigations were carried out which refined the engineering knowledge on the physical properties of the layers. These included a 25 m deep borehole with continuous coring, a 20 m deep cone penetration test (CPT) and a 16.5 m deep seismic CPT. A comprehensive laboratory testing program has been performed on the core samples, consisting of oedometer, triaxial, uniaxial and direct shear tests in order to quantify the stiffness and shear strength parameters of the baserock layer. Seismic CPT results were used to derive small strain stiffness parameters.

The excavation support structures has been designed in the tender phase based on 2D Winkler beam and FEM calculations and showed relatively large displacements and the need for three levels of anchors. During the preparation of the construction design stage serious doubts have emerged in the project team about the effects of the excavation on the surrounding buildings. To clear these doubts, 2D analytical methods (Dulácska 2020) were used to set up displacement limits under the surrounding high value buildings approx. 25-30 m away from the diaphragm wall, to avoid cracks in them. These methods, such as (Hamza 1993), (Chang-Yu 2006), (Wang et al. 2010), usually estimate a ground surface settlement profile based on the deformations of the diaphragm wall in a 2D model and can provide a crude first estimation of expected surface deformations. (Dulácska 1992) has published his similar method based on experience in local tunneling projects. In this project the accepted horizontal displacement at the top of the diaphragm wall was set at 15 mm, and according to the analytical calculations (Dulácska 2020) this would avoid any cracks in the structures. This strict deformation limit, the proximity of the foundations, as well as the complex layout of the basement area and the 15.5 m depth of the excavation all demanded state-of-the art 3D numerical modeling.

A general overview of the excavation contour and the buildings in question can be seen in Figure 1.

Diaphragm wall layout is shown with red lines; the closest and most valuable building, the Court of Auditors Headquarters across Apáczai Csere Street is to the left. Structural state assessment deemed this three-story building the most critical to differential settlement due to its wood slabs and lack of ring beams.

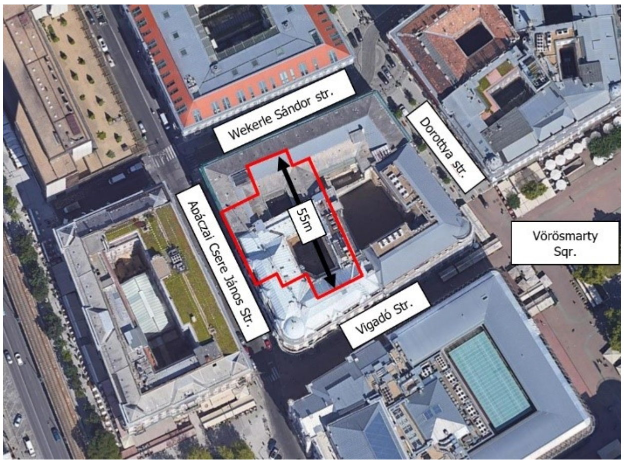


Figure 1. Excavation contour (red line) and existing buildings from Google Maps

## 3 FINITE ELEMENT MODELLING

The excavation, diaphragm wall and the surrounding area has been modeled with the Plaxis 3D Connect Edition V20 software package (Plaxis BV 2020). Soil layers have been modeled with the Hardening Soil Small material model which can consider higher stiffness of soils at very small strain levels. This is crucial in its ability to "automatically" determine depth of influence for settlement calculations and to precisely calculate deformation zone behind a diaphragm wall. The two most important parameters governing small strain stiffness behavior in this model are the small strain stiffness $G_{0,ref}$ and the threshold shear strain $\gamma_{0.7}$. (Benz 2007). These parameters were obtained by comparison of shear wave velocity ($v_s$) measurements by the seismic CPT with $v_s$ correlations for local soils based on regular CPT data (Wolf and Ray 2017a) and (Wolf and Ray 2017b) and (Szilvágyi 2018). Input parameters are listed in Table 1.

Groundwater table was taken as expected average level at the time of excavation, and in the final stage at the top of the diaphragm wall level which was an assumption on the safe side. The model contained the foundations of the existing buildings on the site and their Jet Grouting underpinning. Existing and remaining building structures were only taken as loads without their stiffness to save modeling and computational time.

Table 1. Applied material model parameters. (For abbreviations see (Plaxis BV 2020a).)

| Layers | *1* | *2* | *3* | *4* | *5* | *6* |
|---|---|---|---|---|---|---|
| Soil type | siFSa | saMGr | MSa | grMSa | ClH | ClM |
| Drainage | D | D | D | D | UD-B | UD-B |
| $\gamma_{unsat}$ (kN/m$^3$) | 18 | 18 | 18 | 18 | 19 | 19 |
| $\gamma_{sat}$ (kN/m$^3$) | 19 | 19 | 19 | 19 | 20 | 20 |
| $e_{init}$ (-) | 0.65 | 0.35 | 0.40 | 0.50 | 0.43 | 0.41 |
| $E_{oed,ref}$ (MPa) | 13.2 | 22.3 | 37.6 | 17.7 | 24.0 | 26.0 |
| $E_{50,ref}$ (MPa) | 13.2 | 22.3 | 37.6 | 17.7 | 48.0 | 52.0 |
| $E_{ur,ref}$ (MPa) | 46.2 | 66.8 | 112.9 | 53.0 | 240.0 | 260.0 |
| m (-) | 0.75 | 0.5 | 0.5 | 0.5 | 1.0 | 1.0 |
| $\gamma_{0.7}$ (-) | 3.4E-4 | 7.3E-5 | 6.9E-5 | 7.2E-5 | 9.8E-5 | 1.0E-4 |
| $G_{0,ref}$ (MPa) | 68.1 | 290.8 | 350.1 | 375.2 | 370.0 | 450.0 |
| $c_{u,ref}$ (kPa) | - | - | - | - | 500 | 550 |
| $c'_{ref}$ (kPa) | 18 | 2 | 2 | 2 | - | - |
| $\phi'_{ref}$ (deg) | 25 | 34 | 34 | 35 | - | - |
| $\psi$ (deg) | - | 4 | 4 | 5 | - | - |
| k (m/s) | 1.0E-6 | 1.0E-4 | 1.0E-4 | 1.0E-5 | 1.0E-9 | 1.0E-8 |
| OCR (-) | 1 | 1 | 1 | 1 | 2 | 2 |

Diaphragm walls were modeled with anisotropic, elastoplastic plate elements based on previous modeling experience. The horizontal stiffness of the wall was reduced to 20% of the vertical' and in the corners hinged connection was modeled. Beam elements were used to model struts and anchor elements combined with embedded beam elements to model prestressed soil anchors. The model considered a 100 m x 120 m surrounding of the site, containing the basement level of the most crucial neighboring building as well, shown on the right in Figure 2.

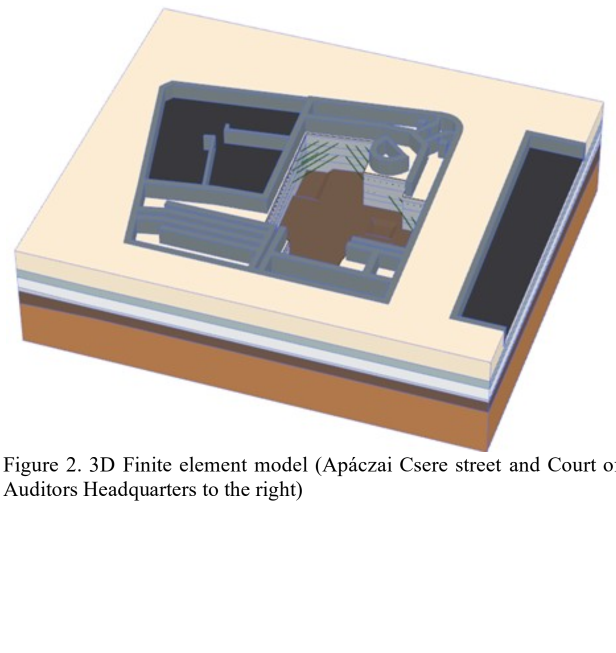

Figure 2. 3D Finite element model (Apáczai Csere street and Court of Auditors Headquarters to the right)

The large area and quite detailed soil stratification resulted in approx. 220 000 elements causing a significant calculation time of around 24 hours on a high-performance PC.

### 3.1 *Modelling difficulties*

To benefit from the many advantages of 3D modelling several obstacles need to be tackled. These include high modelling time demand, which may be reduced in case of complex layouts by geometrical simplifications. If the modelling task emerges during the construction phase of a project, our experience is, that there is usually no time to gather information and model the superstructure of surrounding buildings. Structural stiffness however usually reduces settlement differences, therefore, if it was not modelled, some reserves may be presumed when evaluating the results. In order to model foundations and structural loads of surrounding buildings, structural state evaluation is necessary. This is also required by the Chamber of Engineer's specifications and preferably it is done in the preparation phase of the project.

Regarding pore pressure distribution due to dewatering, we found that the most reliable method is to perform permanent groundwater flow calculations in each excavation stage. However, this has significant calculation demand in 3D. A reasonable result can be achieved with the interpolation method suggested by (Plaxis 2018). This requires to set the excavated soil volume to dry, the soil volume directly below to interpolate and the soil volume around the excavation to global groundwater level. We have encountered some difficulties with the interpolation method, most commonly a local shear failure has arisen inside the excavation, even if based on the shear strength parameters of the clay layer here we would not expect such a failure in reality. We found, that at big groundwater level differences, high gradients, a single volume with interpolation is not sufficient to achieve a plausible result. In this case more clusters are suggested to be set to interpolate.

A peculiar result of undrained behaviour was found when analysing horizontal earth pressures on the diaphragm wall. Excavation level was already in the clay. When performing undrained analysis, above the excavation level a reduction was observed in the horizontal earth pressures in the total excavation stage, as shown on the left side of Figure 3, moreover, the calculation showed pressure on the side of the excavation. We found this to be a result of horizontal unloading of the clay layer due to the excavation, and depending on geometry and groundwater levels, in the undrained analysis even suction may occur in the clay cluster. This has a beneficial effect on shear strength and earth pressures, which we did not wanted to rely on as designers. We found that the dismissal of suction was not possible even if we used the 'ignore suction' option (Plaxis 2020 version), only a drained analysis solved this issue. To analyse this problem in detail, it is beneficial to assess differences of drained and undrained material models, as the effect on the deformations and structural forces of the diaphragm wall may be significant. See the publications of (Wehnert 2006), (Galavi 2016), (Plaxis BV 2020b) for more details.

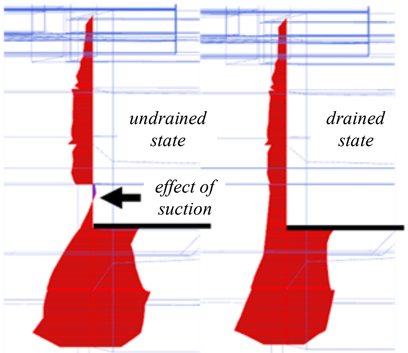


Figure 3. Effect of suction on earth pressures in undrained calculation

### 3.2 *Main results*

Results show, that as expected, structural deformations increase continually as the excavation progresses, and the heave of excavation bottom also appears. Among the surrounding buildings, the largest deformations appear at the Mahart building's staircase, where in ground layout there is a positive edge. According to structural data supply here the vertical loads of the building are significant, this also explains the large displacements. Total displacements until the total excavation phase are shown in Figure 4. Foundations next to the diaphragm wall have a displacement of around 1 cm, except the staircase area, where the magnitude of deformations is around 3 cm, with a largest value of 3.8 cm adding up from 2.8 cm vertical and 2.6 cm horizontal deformation.

The largest displacements of the staircase area appear at the positive edge, and displacements gradually decrease with distance. In order to limit the shift of the staircase into the excavation pit, further anchors were added to the corner area in two levels. These have reduced the displacements efficiently compared to earlier calculations.

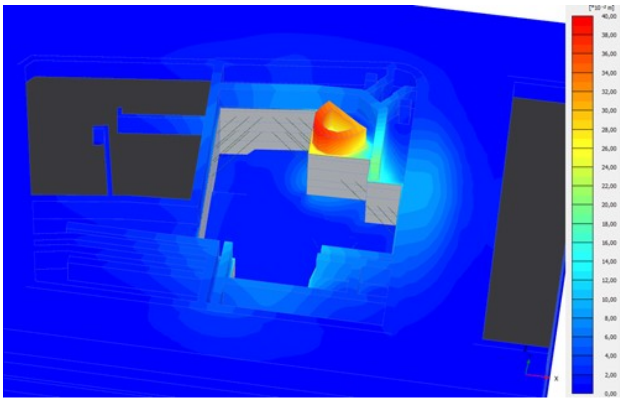

Figure 4. Total displacements at deepest excavation stage. Red color is 4 cm.

Settlements around the pit can also be observed from Figure 4. Observing the displacement field, it can be stated, that surrounding buildings sustain only insignificant settlements, foundation of the Court of Auditors Headquarters sustain 1 mm settlement. At the edge of the model, deformations are below 1 mm, except the Wekerle street side, where it is 1 mm. Figure 5. shows a section parallel to Wekerle street, in the middle of the excavation pit with the total displacement results. Effect on the Court of Auditors Headquarters can be clearly seen as marginal.

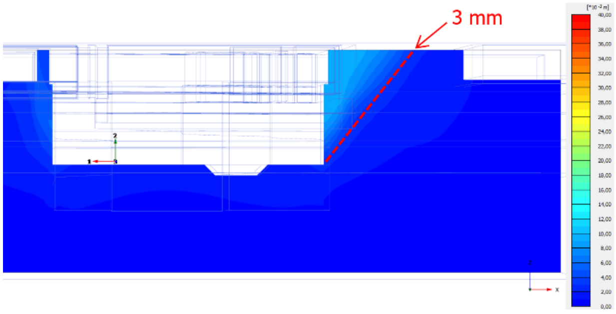


Figure 5. Total displacements at deepest excavation stage in section view.

Analysing the displacements in later construction stages, it can be found, that after the deepest excavation stage, after the construction of two levels of basement slabs, further deformations appear after releasing the anchors. Also, the final stage which corresponds to the construction of the designed building brings further displacements.

Diaphragm wall displacements (Figure 6.) and bending moments correspond well with surface deformations, their largest value are at the staircase.

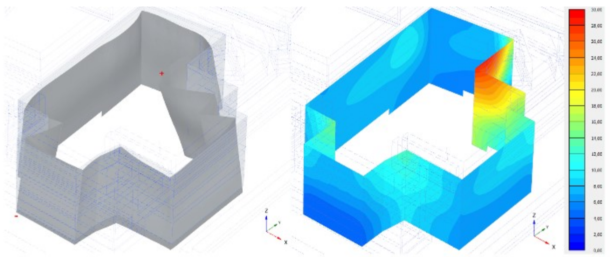

Figure 6. Diaphragm wall displacements. Red color is 3cm.

As the presented results have demonstrated, 3D geotechnical finite element modelling can be used in construction projects even for complex geometries. Closely lying surrounding buildings and difficult ground layouts of the diaphragm wall certainly require 3D modelling.

## 3.3 *Material model sensitivity study*

Base layer of the design area is medium to high plasticity clay. A key aspect in the mechanical modelling of the clay requires to assess drained and undrained behaviour. Undrained shear strength of cohesive soils is most commonly determined by well-established local correlations from CPT test results, however, it is important to assess, how long the undrained state is valid throughout the construction process. It is usually assumed, that unloading processes tend to result in quicker drainage and drained state usually is on the safe side in terms of displacement magnitudes. The drainage state can be assessed by using the consolidation time factor (Vermeer and Meier 1998), (Kempfert and Gebreselassie 2002), (Lächler and Vermeer 2008):

$$T_v = \frac{k \cdot E_{oed}}{\gamma_w \cdot D^2} \cdot t = \frac{c_v}{D^2} \cdot t \qquad (1)$$

where $T_v$ is time factor, k is coefficient of permeability, $E_{oed}$ is constrained modulus, $\gamma_w$ is unit weight of water, D is seepage length, t is construction time.

If $T_v < 0.01$, undrained, if $T_v > 0.4$, drained state can be assumed. If the time factor lies between the two limit values, both states are suggested to be assessed. In our case, we analysed both approaches. The deformation field was similar in both cases after total excavation, and as expected, drained state resulted in larger settlements. In terms of surface settlements, the typical difference was around 3-5 mm, largest difference was smaller than 1 cm. Drainage in our case did not affect the settlement through significantly.

Diaphragm wall deformations were also showing 3-5 mm differences depending on drainage conditions, and largest difference was 1 cm. Drained state resulted in 15-20% larger bending moments. Hence, although in our case displacements were only slightly different, structural forces were considerably higher in the drained state, which has a direct effect on construction cost.

Next step is choosing material model and determining mechanical parameters. The HS Small model is capable of following the mechanical processes in the soil occurring during construction of an excavation pit. A separate unload-reload stiffness can be considered, depending on stress path and load history, elastic and plastic deformations can be calculated, time effects can be assessed, and stress dependent stiffnesses can be used. A special feature is the consideration of higher soil stiffnesses at small strains (Benz 2007), which results in more precise settlement calculations, and in a realistic settlement through. These features alone however do not warrant true and precise results, high quality laboratory and in-situ testing is essential to obtain realistic model parameters.

To assess the effect of small strain stiffness, we have performed calculations with both the HS and HS Small material models. Significant difference was found. In the state of total excavation, the largest difference was 3.5 cm and typically 1.5-2.5 cm larger displacements were obtained with the HS model. The difference may stem from the fact, that a larger area behind the diaphragm wall was mobilized by the excavation with the HS model. Surface settlements were also larger, the Court of Auditors Headquarters building sustained 1.0-1.5 cm higher settlements with HS. In the final construction stage, surface displacements were typically 2.5-3.5 higher with HS and the highest difference was locally 4.5 cm. Diaphragm wall displacements were similarly higher in the final construction stage with HS, highest difference was around 3 cm. Comparing bending moments, HS model shows 80-90% higher values and locally even much higher differences appeared. Figure 7 shows a comparison of bending moments. In the HS model, a lack of underpinning in the passive zone can be observed and a different displacement mechanism results in significantly higher bending moment in the central area.

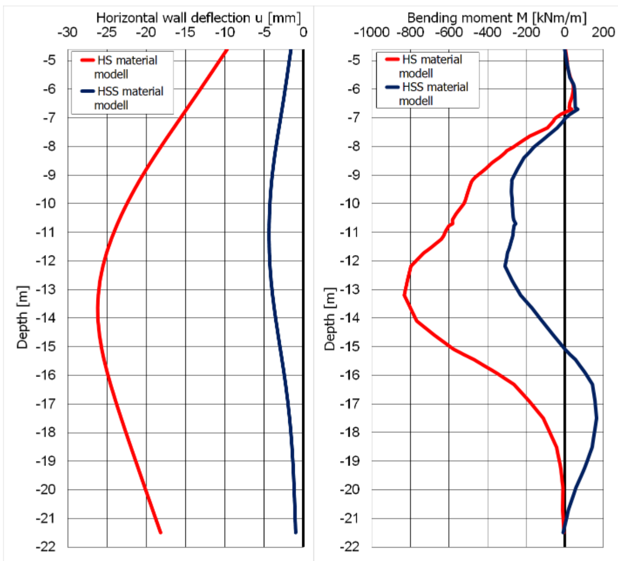


Figure 7. Comparison of Hardening Soil and Hardening Soil Small model results.

Summarizing the results, in our case there is a significant difference between HS and HS Small model results. HS model seems to overpredict deformations, hence for economical design, the use of HS Small is suggested. However, to achieve realistic results, specific laboratory and in-situ testing program is needed, which require further costs, and high quality testing, but this effort is returned in construction cost, especially if surrounding buildings need to be braced due to overcalculated deformations in the design phase. We would like to emphasize, that the measurement of HSS model parameters was until recently not common practice in Hungary, and although recently for some local soil types correlations based on regular soil investigation methods have been published, such as (Wolf and Ray 2017a), (Wolf and Ray 2017b), (Szilvágyi 2018), without a comprehensive testing program, these parameters can not be estimated safely. Reliable design can only be performed based on a thorough soil investigation and testing program.

### 3.4 *Comparison of 2D and 3D FEM calculations*

Deep foundation structure and excavation pit structure design is significantly simpler in 2D, than in 3D, as model building time, calculation time and result evaluation time are all shorter. 2D design is routinely used for excavations with simple geometry layouts. However, surrounding buildings loads are very difficult to take into account in 2D, as the structural layout rarely corresponds to plain strain conditions. A main structural wall not parallel to the section, or single columns already complicate load analysis. 2D models are also unable to predict deformations and true load bearing close to corners, which means negative corners will most probably be overdesigned and uneconomical, positive corners might apply too bold structural solutions if design is based on 2D models.
For our case study, eleven 2D sections were analysed and compared to the 3D calculation. The 2D models showed consistently higher surface settlements, than the 3D model, in most areas, the settlement was two-three times higher in 2D. Diaphragm wall deformations were also higher, although the difference was not as significant as with the surface settlements. This was also a result of some calibration of the structural loads between the 2D and 3D model based on the comparison of diaphragm wall displacements. The results of a calibrated 2D model are shown in Figure 8.

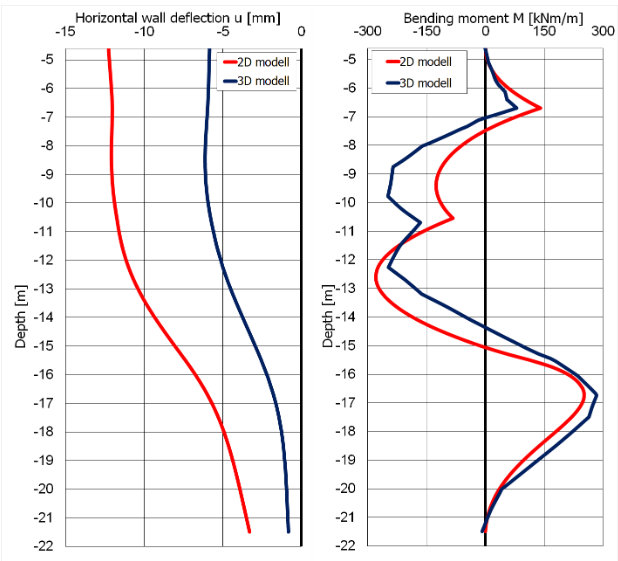


Figure 8. Comparison of 2D and 3D model results.

In the regular corners, transverse struts were used. 2D sections taken in the vertical plane of the struts showed admissible results compared to the 3D model, deformations were almost identical, and structural forces showed a 20-25% difference. Sections taken in the positive corner showed the significant underestimation of deformations.

### 3.5 *Comparison of calculation and measurements*

As designers we have stressed the need for and importance of validating the calculations with monitoring measurement results. Therefore, two inclinometer tubes have been placed in the diaphragm wall, where the largest deformations were calculated to measure horizontal deflections. Unfortunately, one of the tubes has been damaged by an excavation machine and rendered useless, while the other location was affected by the connection of a very stiff reinforced concrete working platform needed to perform the excavation works. Although altered by these issues, the inclinometer measurements showed maximum horizontal displacements of 6 mm. Neither of the surrounding structures suffered from cracks. These results show that the predictive modelling was successful. A photo from the construction is shown in Figure 9.

Figure 9. Excavation in progress

## 4 CONCLUSIONS

Advanced geotechnical numerical methods are now available to design companies. Case studies can be used to point out special aspects, needs of numerical modelling in projects. Benefits of appropriate site investigation techniques and proper modelling are demonstrated in the paper.
We have showed the capabilities of 3D modeling compared to 2D design and emphasized, that surrounding building foundations, complex layouts with many corners require 3D modelling. Sadly, in many inner city projects we find, that the effect of the construction on surrounding buildings is not analyzed in advance, as this design task is underestimated and sometimes totally disregarded in project planning. This leaves the Client and the construction company with a high amount of unknown risk, not to mention the safety of people working and living in surrounding buildings.
3D geotechnical finite element modeling is a valuable tool for risk assessment, and compared to simple 2D analytical methods, much more reliable, as these can not analyze true spatial load transfer. These 2D analytical methods are usually valid for plain strain, homogeneous soil conditions, and can only provide a rough settlement prediction.
We have demonstrated, that 3D modeling, based on high quality testing program can result in an economical design even in complex layouts. The parameters of HS Small model require special soil investigations, but with these, geotechnical finite element modeling is now a valuable tool for practice and design, not only research.

## 5 ACKNOWLEDGEMENTS

This project has been commissioned by Bohn Mélyépítő Kft. Their work and valuable feedback is greatly acknowledged.

## 6 REFERENCES

Benz T. 2007. *Small-Strain Stiffness of Soils and its Numerical Consequences*, PhD Thesis, Universität Stuttgart, 1-165.

Burghardt B., Shell P., Szilvágyi L. 2020. Dorottya Hotel Budapest (DHB), Budapest V. kerület, Wekerle Sándor utca - Apáczai Csere János utca - Vigadó utca - Dorottya utca, *Soil Investigation Report*, 1-72.

Chang-Yu O. 2006. *Deep Excavation – Theory and Practice*. London CRC Press, 1-552.

Dulácska E. 2020. *Repedésmentességi Statikai szakvélemény a Dorottya Projekt, mélygarázs építése körüli épületekről*, 1-14.

Galavi V. 2016. *Groundwater flow, fully coupled flow deformation and undrained analyses in Plaxis 2D and 3D*, 247-273.

Hamza Associates. 1993. *Procedure for Settlement Prediction: Diaphragm Walls, Cairo Metro Line No. 2.* Report Prepared for Campenon Bernard SGE.

Kempfert H.G. and Gebreselassie B. 2002. Zur Diskussion von dränierten oder undränierten Randbedingungen bei Baugruben in weichen Böden, *Bautechnik* 79-9, 603-611.

Lächler A. and Vermeer P.A. 2008. *Tiefe Baugruben in weichen Böden*, Von der Forschung zur Praxis Symposium anlässlich des 80. Geburtstags von Prof. U. Smoltczyk am 14. November 2008 in Stuttgart, 117-136.

Plaxis BV. 2018. *Shaft and Dewatering Online Course* www.plaxis.nl. visited on 2019. May 1st.

Plaxis BV. 2020a. Plaxis *3D Reference Manual*. p. 262, 350, 442

Plaxis BV. 2020b. *Geotechnical Analysis – Optimizing Undrained Parameters for Undrained Soft Soils*. Online Course

Vermeer P.A. and Meier C.P. 1998. *Standsicherheit und Verformungen bei tiefen Baugruben in bindigen Böden*. Baugrundtagung, Stuttgart, 133-148.

Szilvágyi Zs. 2018. *Dynamic soil properties of Danube sands, PhD thesis*, Széchenyi István Egyetem, MMTDI, 1-131.

Wang J. H., Xu Z. H., Wang W.D. 2010. *Wall and Ground Movements due to Deep Excavations in Shanghai Soft Soils*. Journal of Geotechnical and Geoenvironmental Engineering Vol. 136, Issue 7.

Wehnert M. 2006. *Ein Beitrag zur drainierten und undrainierten Analyse in der Geotechnik*, Institut für Geotechnik der Universität Stuttgart, 1-128.

Wolf Á. and Ray R.P. 2017a. *Comparison and Improvement of the Existing Cone Penetration Test Results - Shear Wave Velocity Correlations for Hungarian Soils*, Int. Journal of Env., Chem., Eco., Geo., and Geo. Eng. 11: 4, 1-10.

Wolf Á. and Ray R.P. 2017b. *Nyíróhullám terjedési sebesség becslése CPT adatokból hazai talajviszonyokra*, Geotechnika 2017 Budapest, Magyarország: Konferencia Iroda Bt., 26-41.