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FEM in der Geotechnik

Eléments finis et géotechnique

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Large scale 3D numerical simulations of deep excavations in urban areas – constitutive aspects and optimization

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1 Introduction

The recent introduction of advanced constitutive models in commercial finite element (FE) codes has made the prediction of settlements and deformations of geotechnical structures considerably more reliable than in the past. For many years simple constitutive models designed to compute ultimate limit loads were applied to problems that intrinsically require applying models that reliably represent prefailure non-linearities. The inability of simple models to compute realistic displacements has lead a generation of engineers to distrust FE results altogether, even in situations where no other methods are available to correctly account for the complex behavior of nonlinear soil-structure interaction. With the introduction of advanced constitutive models the finite element method has finally matured to a point where it can be used for the entire range of geotechnical problems, including deformation analyses.

In this paper we present a case study of an excavation for a large building being a part of the Pont-Rouge property development project by Swiss Federal Rail Road (SBB) in Lancy-Geneva. In the context of this project, finite element models have allowed us to verify and optimize the design of the support system. The commercial nonlinear finite element program ZSoil v.2013 [1] was used in the study.

The excavation, illustrated in Figure 1, is 160 meters long, about 60 meters wide and 14 meters deep. Compressible layers of silty-clayey deposits [2], in which most of the excavation takes place, represent the biggest challenge for the design of a support system capable of limiting deformations on surrounding buildings.

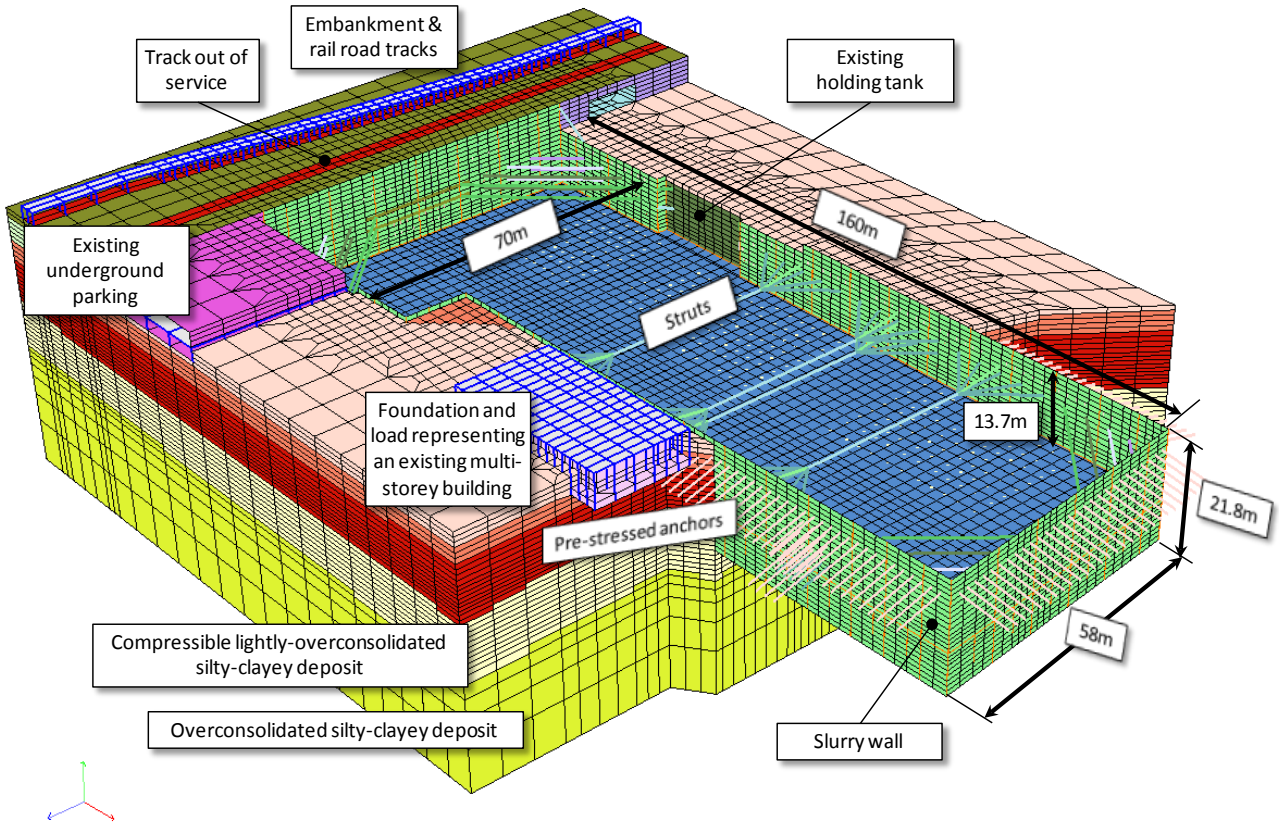


Figure 1: Schematic view of the excavation.

2 Constitutive behavior

The Hardening Soil small strain constitutive model (HSS) [3][4][5] was used to represent the nonlinear behavior of the natural subsoil. The main features of the HSS model are:

- Stress-dependent stiffness: variable stiffness with increasing depth or decreasing stress level due to unloading are essential for simulating the behavior of high retaining walls undergoing horizontal displacements.
- Shear strain-dependent stiffness, valid also in the small strain region (Figure 2): In the case of serviceability limit state analyses; this is important for computing realistic settlement distributions around excavations. It generally leads to narrower settlement troughs, which are in better agreement with observations.
- Unloading/reloading stiffness (Figure 2): The introduction of a distinct unloading/reloading stiffness allows the model to represent heave in and nearby excavations more accurately than models with a single modulus.
- Two hardening mechanisms: Hardening due to plastic shear strain, as well as plastic volumetric straining are both included in the HSS model, implemented in ZSoil. This is an important feature as the shear hardening handles pre-failure plastic straining, i.e. plastic strains may occur before reaching the Mohr-Coulomb envelope, (Figure 2) even in overconsolidated material, whereas the volumetric mechanism accounts for preconsolidation effects.This is necessary as both types of mechanisms can appear at different locations in one model.

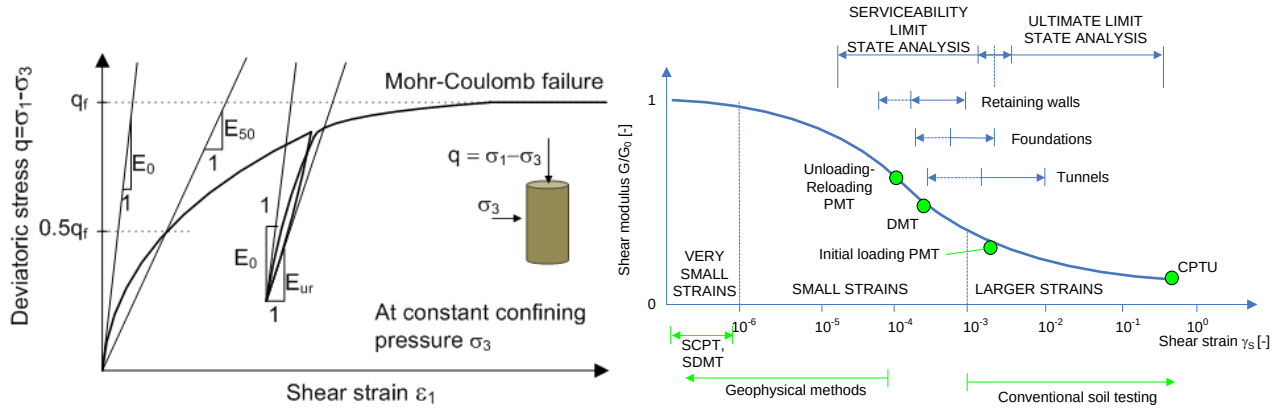


Figure 2: Definition of the stiffness moduli describing the behavior of the HSS model in triaxial conditions. Pre-failure nonlinearities and strain-dependent variation of shear stiffness in small and larger strain amplitudes (left); comparison with the ranges for typical geotechnical problems and different tests (right).

3 Selection of a relevant support system solution

Search for a relevant support system for the excavation was carried out in the classical manner by starting from traditionally applied solutions and iteratively including appropriate countermeasures which would satisfy retaining system requirements.

In the first stage, a traditional support system was considered. This solution consists of an excavation, supported by a system of slurry walls (1.0 m wide nearby the railroad tracks and 0.6 m in the remaining part). Then the slurry wall is supported by two rows of non-prestressed struts (Figure 3) and a blocking slab at the bottom of the excavation. In this case, the numerical model revealed lack of sufficient bearing resistance of soil at the passive side of the retaining wall resulting in considerable displacements (about 19 cm), not only in the mid-height but also at the bottom of the diaphragm wall. As a consequence, large settlements (approx. 17 cm) are computed on the mobilized active side below the existing multi-storey building. The same kinematics were obtained with an auxiliary 2D model as illustrated in (Figure 3).

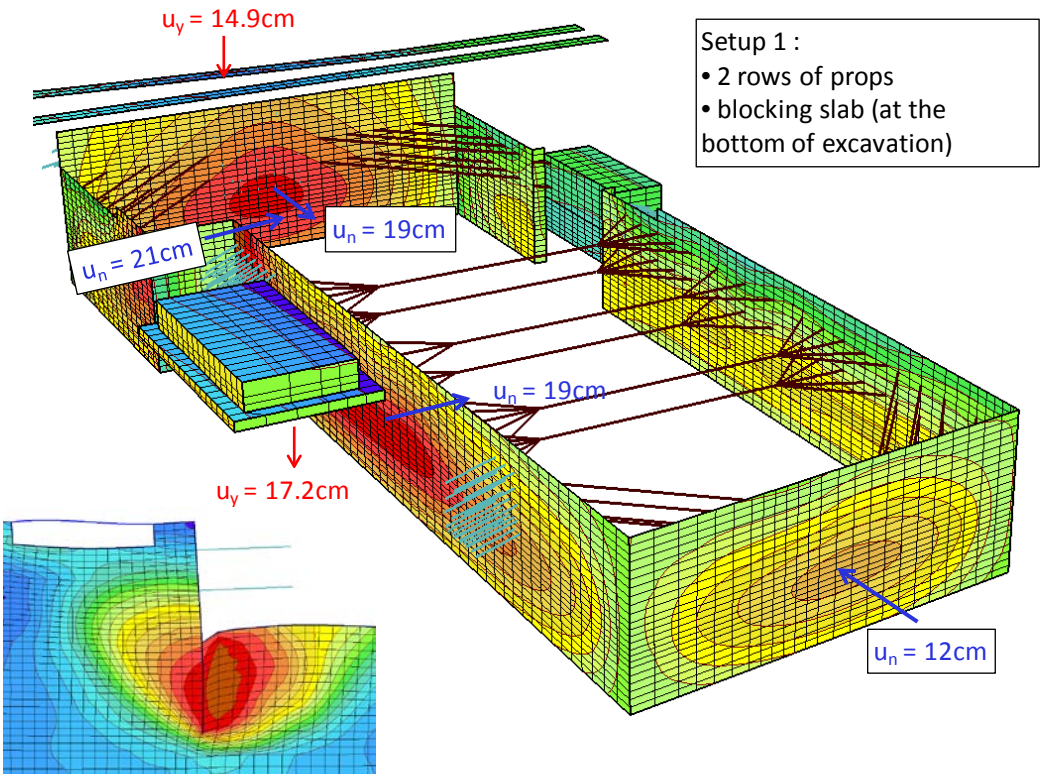
Considering the important compressibility of the soils nearby the excavation, a lattice of jet-grouting columns was introduced in order to reduce wall deflections (Figure 4). Initially, these jet-grouting column walls were modeled by means of shell elements with an equivalent width of 0.6 m every 10.0 m in both directions.

In the preliminary stage of support optimization, a few simplifications have been assumed for the sake of rapidity of computing:

- Paneling in the retaining wall was neglected, i.e. shells representing the diaphragm wall were continuous without explicitly defined hinges.
- Jet-grouting column walls were modeled with elastic shells. The assumption of the elastic jet-grouting wall required the introduction of interface elements between soil and wall elements in order to account for interaction between two materials with different stiffness.
- Jet-grouting column walls were perfectly connected with the retaining wall.

The obtained results showed that wall deflections were reduced considerably to approximately 4-5 cm as illustrated in Figure 5. However, the maximum allowable settlements for the rail road tracks and the neighboring office building were still exceeded.

Therefore, in the next step, pre-stressing of struts was considered in the bottom row, whereas the upper row of props was replaced with pre-stressed anchors (Figure 5). These modifications have led to a slight reduction of both horizontal and vertical displacements.



Auxiliary 2D model. Excavated zone nearby the existing building

Figure 3: Setup of solution 1 for the support system (blocking slab is not shown), and resulting displacements at the end of excavation.

The maximal horizontal wall displacements on two of the side walls and the settlement under the rail road tracks along the short side of the excavation, as well as the differential settlement along the short side of the tower are summarized in Table 1.

Table 1: Comparison of wall displacements and settlements for the 3 support systems.

Solution n°	Wall displacement		Settlement RR tracks	Differential settlement tower
	Near RR tracks	Long side		
	[cm]	[cm]	[cm]	[cm]
1	19.0	19.0	14.9	8.6
2	4.5	5.6	4.1	3.9
3	2.9	2.9	3.0	0.9

The simulations showed that only solution 3 was able to keep displacements and settlements within acceptable limits. Solution 3 was therefore selected for further refinement and optimization.

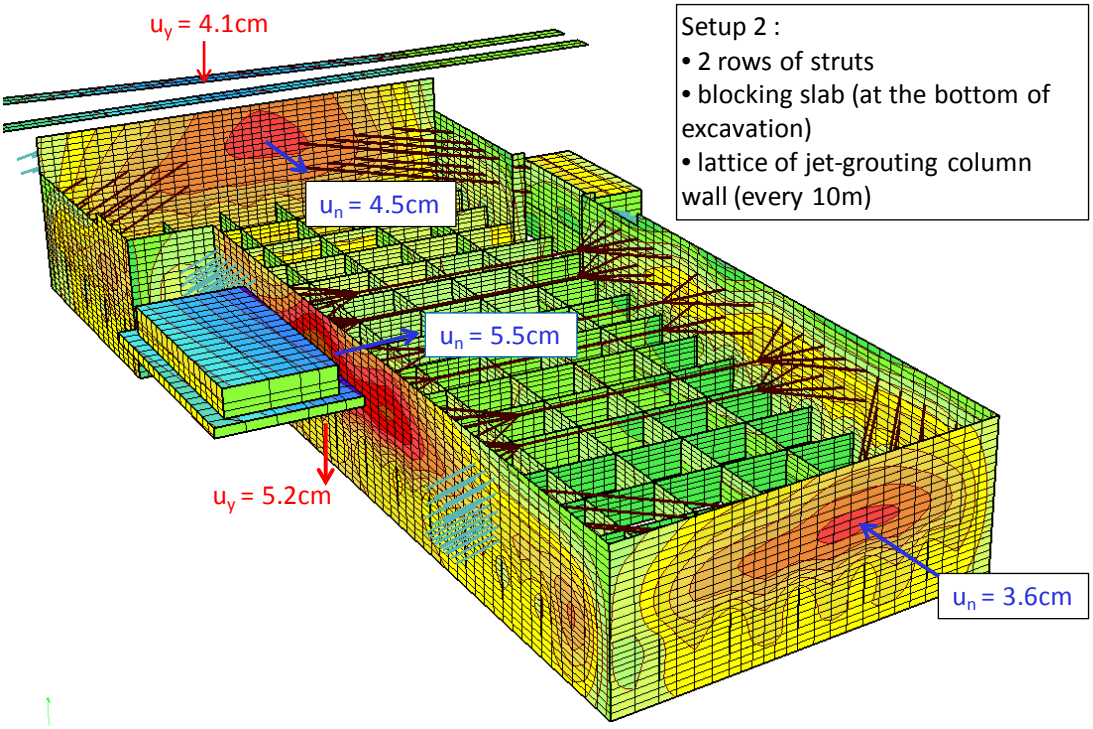


Figure 4: Setup of solution 2 for the support system including a lattice of jet-grouting column walls (blocking slab is not shown), and resulting displacements at the end of excavation.

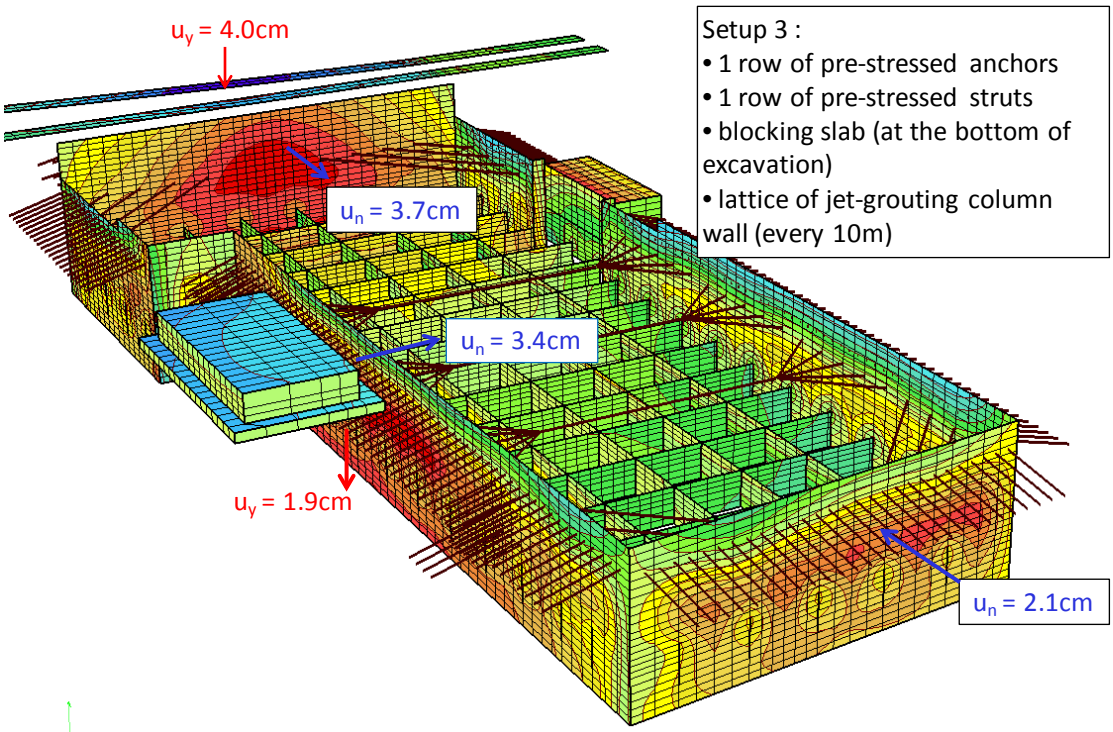


Figure 5: Setup of solution 3 for the support system including a lattice of jet-grouting column walls and pre-stressed props and anchors (blocking slab is not shown), and resulting displacements at the end of excavation.

4 Optimization of selected solution

Besides allowing the selection of the appropriate support system, simulation of the 3 solutions helped identifying a few problems that were corrected in the final model of the selected solution. These adjustments are listed below:

- Introduction of plastic hinges allowing the release of rotational constraints between diaphragm wall segments because of non-zero moments along construction joints (Figure 6).

- Elastic-plastic material behavior specified for jet-grouting column walls with tensile ultimate strength equal to 250 kPa. This removed the need for having to add contact elements on both sides of the walls assuming realistic adherence of jet columns and soil and possible plasticity occurring in jet-grouting columns. The use of elastic-plastic material for grouted soil is also justified by the tensile forces occurring in the upper part of jet-grouting walls, which were the results of soil heave during excavation (Figure 7). Moreover, the high compressive forces required a widening of the jet-grouting walls to an equivalent width of 2 m (2 rows of alternated jet-grouting columns Ø1.2m).

- In order to avoid transmission of tensile forces between the jet-grouting walls and the slurry walls, elastic-plastic joints were introduced (Figure 6 and Figure 7).

- Adjustments and introductions of several details aimed at better representing the reality (e.g. cross-ties to distribute strut forces, local increase of anchor lengths).

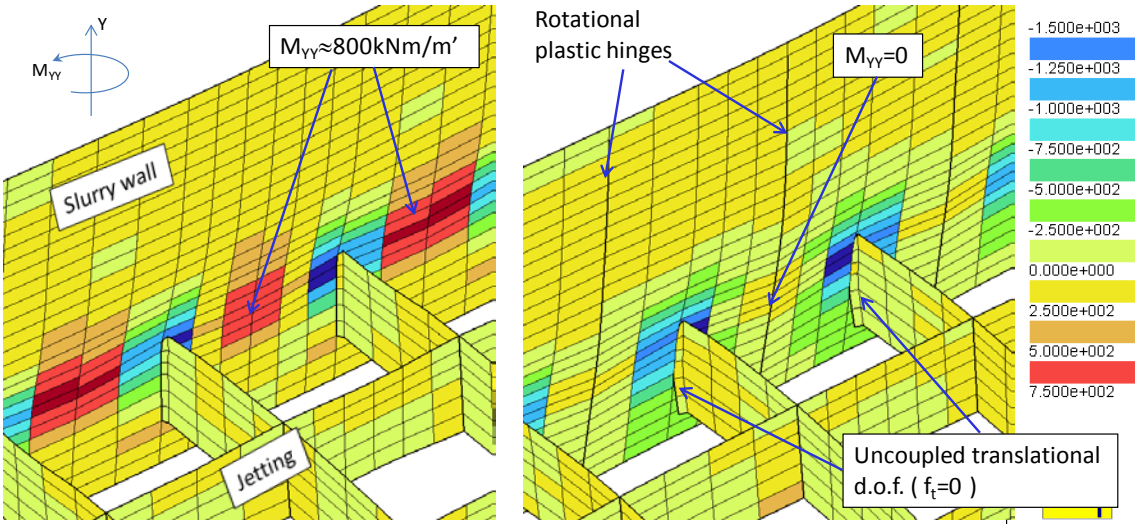


Figure 6: Effect of accounting for hinges between diaphragm wall segments and uncoupling of translational degrees of freedom: excessive bending moments at construction joints between wall segments in the preliminary model (left), eliminated moments at joints between wall segments and effect of uncoupling of translational degrees of freedom with null tensile resistance introduced in the final model (right).

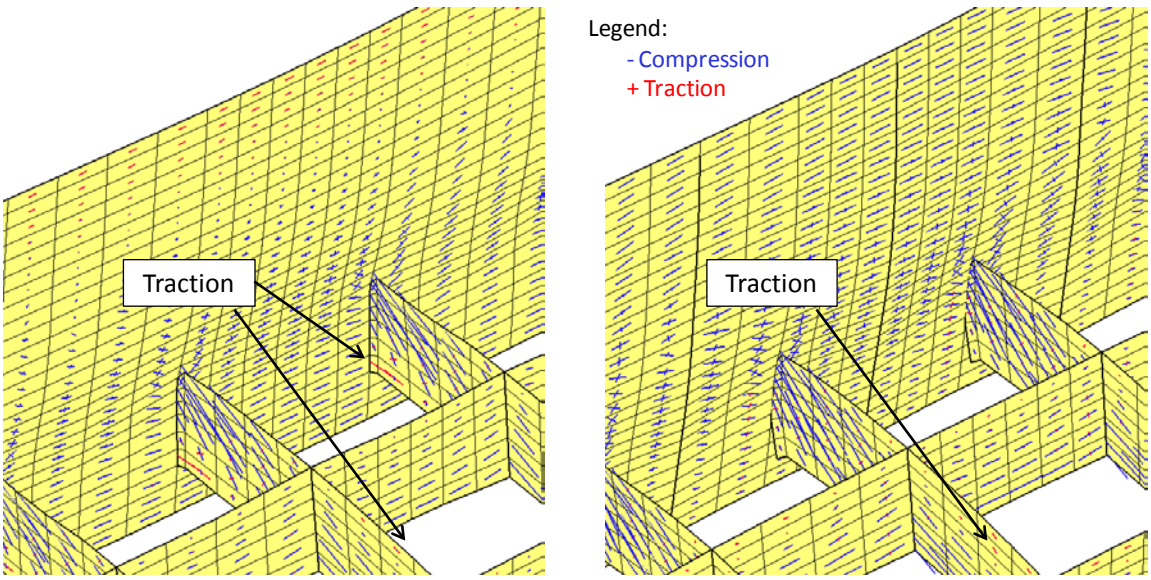


Figure 7: Diagrams of principal forces: tensile zones between diaphragm wall and jet-grouting column wall were successfully removed by the introduction of translational uncoupling.

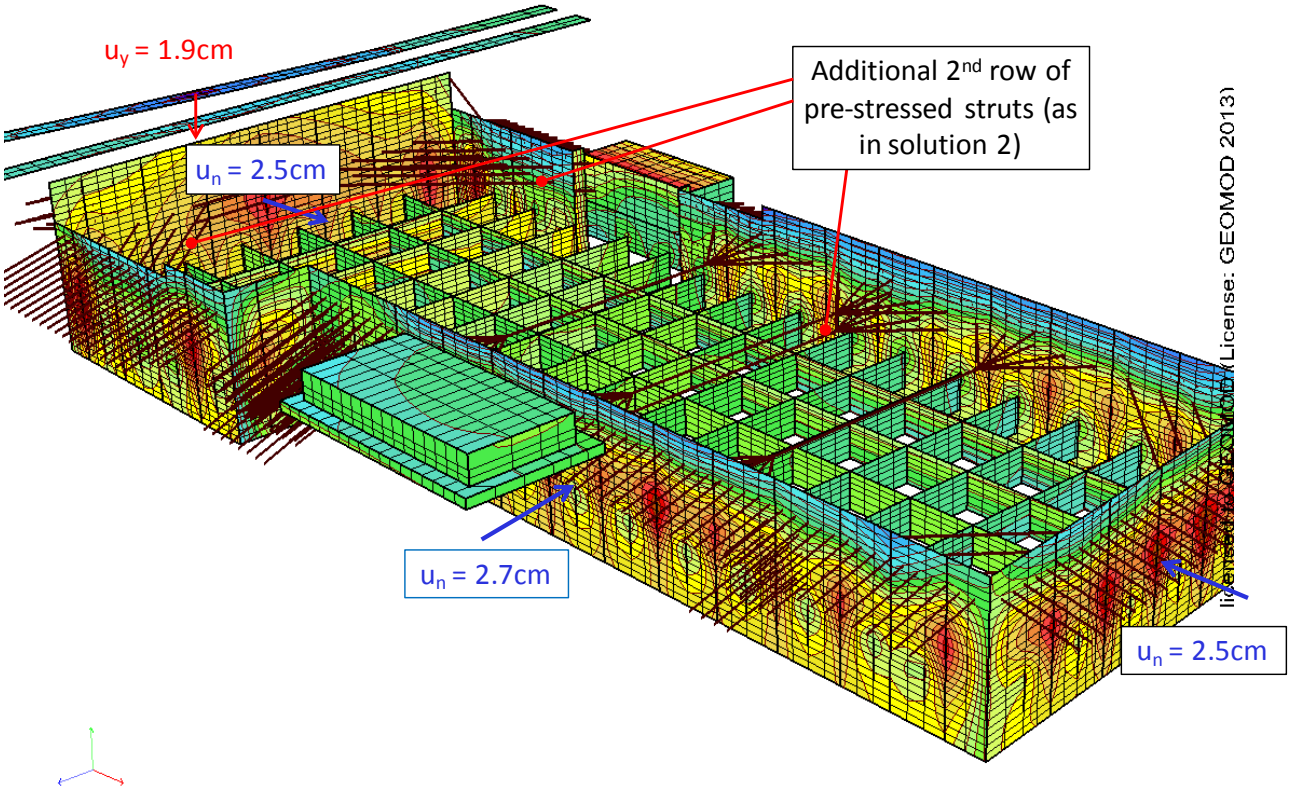


Figure 8: Wall displacements obtained with the refined model of solution 3.

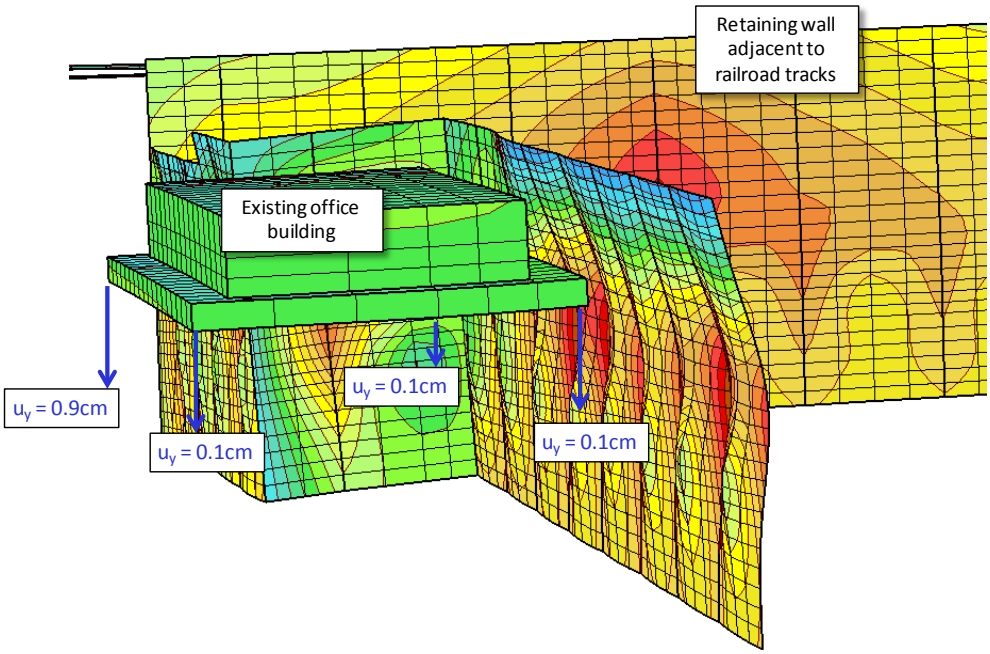


Figure 9: Deformed slurry wall segments and resulting settlements of the existing office building.

5 Prediction of settlements of the above-ground structure

Once the support system had been optimized, the refined numerical model was used to evaluate settlements of the structure members. Construction elements, a bottom slab and the above-ground structure, were introduced and external vertical loads were applied (Figure 1). Such a complete model can account for:

- Realistic stress-strain history related to the excavation stages (increasing soil overconsolidation due to overburden removal) and secondary loading (weight of the structure members)
- Global rigidity of the structure providing possible stress redistribution during loading

Since the building consists of two main parts, namely a three-level underground parking (a light part of the building) and three multi-storey “towers”, a soil improvement by means of pile inclusions has been designed in order to avoid excessive differential settlements of the structure, and deformations of the adjacent railroad and road infrastructure. Screw cast-in-place piles, ∅600 mm in diameter and 13.5 to 16 m long, formed pile group cores with 3x3 m interval between piles (Figure 10). The cores are located below construction walls. Figure 11 underscores the need for using pile groups in order to limit differential settlements. The computed settlements of the structure (Figure 12) are expected to vary between 2 and 4 cm in the parking zone and 4 to 6 cm below the multi-storey buildings.

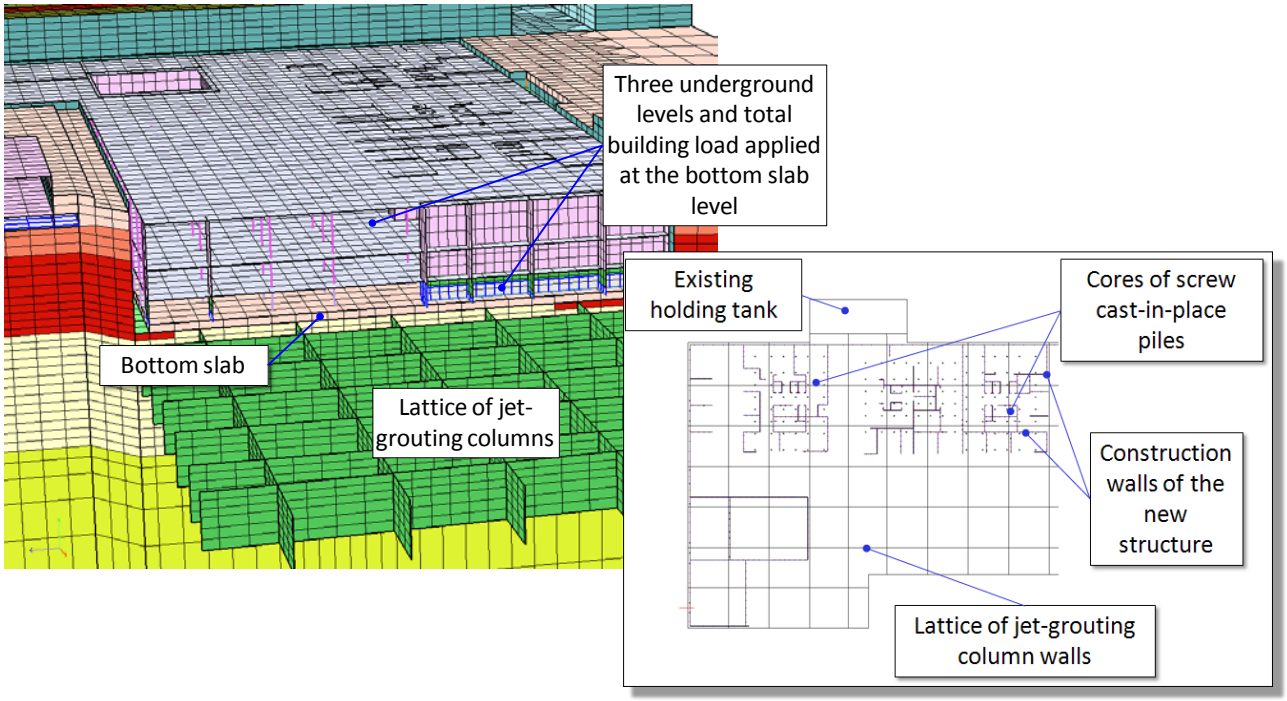


Figure 10: Modeling of the above-ground structure and piles for supporting vertical loads.

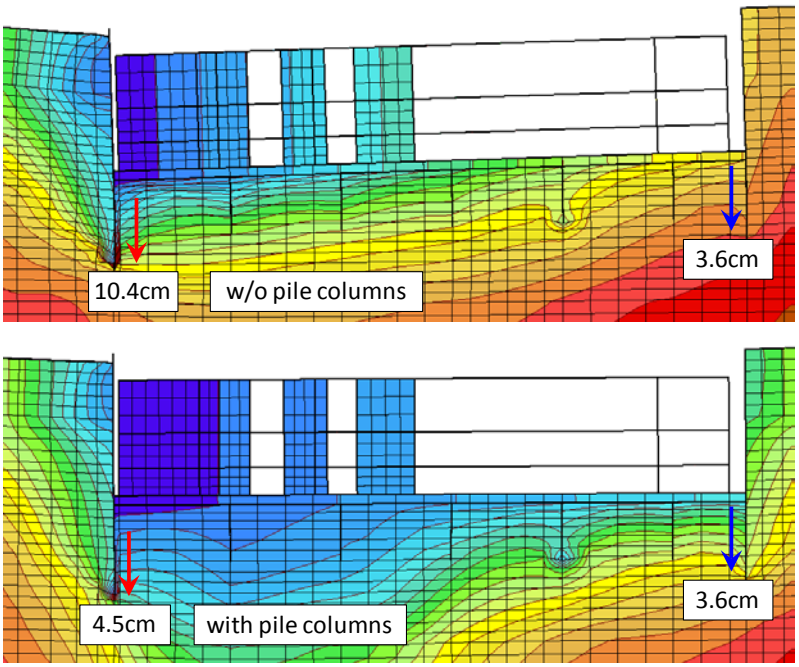


Figure 11: Comparison of structure settlements with pile inclusions and without (results obtained with an auxiliary 3D slice model).

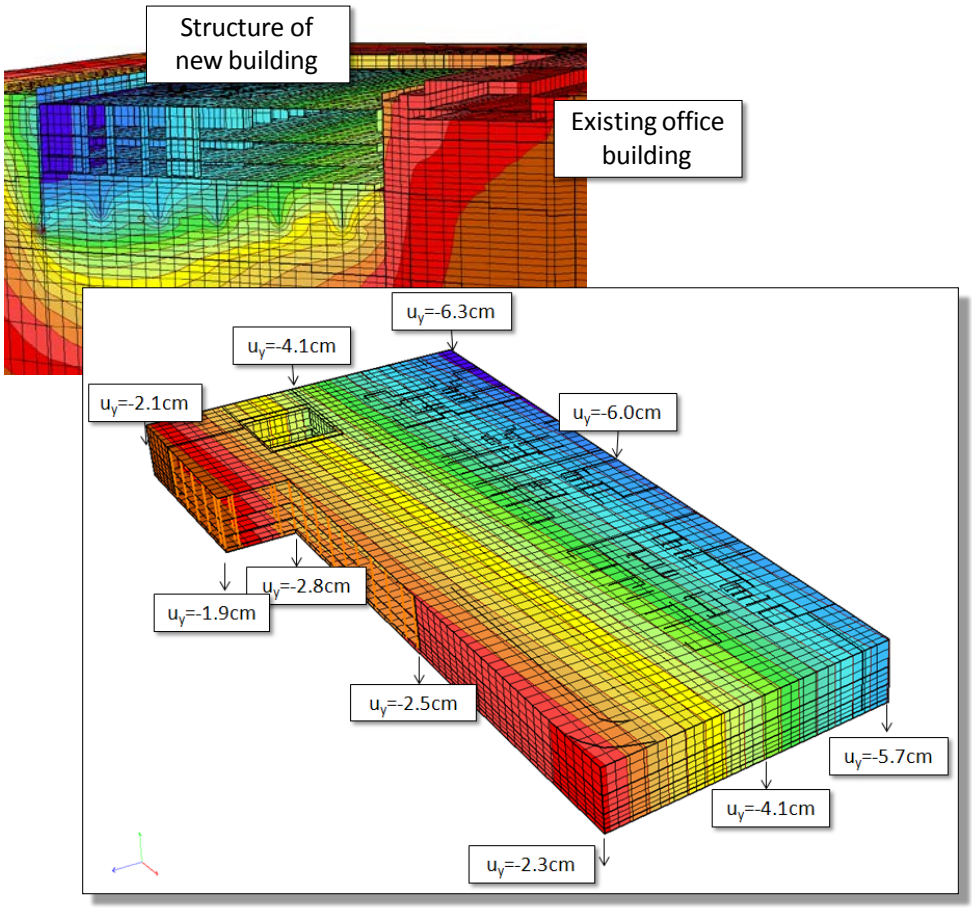


Figure 12: Prediction of settlements for the designed complex of office buildings.

6 Conclusions

Using the example of this case study we were able to show the valuable information that a detailed 3D finite element model can provide for the design and optimization of a geotechnical structure. In particular for situations in soft soils, where deformations of retaining walls can become important, the design is frequently dictated more by serviceability than by ultimate limit strength considerations. Detailed 3D models with advanced constitutive models for representing the soil behavior are capable of providing answers for both types of questions.

The model has allowed making the following design choices:

- Jet-grout column walls are necessary to limit horizontal displacements in the bottom parts of the retaining wall.
- Introduction of pre-stressed struts is required to prevent mobilization of soil deformation behind the retaining wall which would lead to excessive wall deflection in its middle part and, in consequence, to settlements of the adjacent buildings.
- Need for applying pile-based inclusions that limit settlements of the new structure was underscored.
- Local lengthening of anchors for the members initially fixed in the observed mobilized active zones of the subsoil.

In addition, several problems, that modelers can be confronted with during the elaboration of a finite element model, have been identified and appropriate solutions have been presented.

7 References

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