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Numerical back-analysis of a monitored deep excavation in Budapest considering time dependency of wall deformations

Prise en compte du temps dans le calcul des déformations d'un écran de soutènement: Retro-analyse d'une fouille instrumentée à Budapest

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ABSTRACT: A deep excavation project has been started in Budapest in 2017. The Hungarian subsidiary of Soletanche Bachy has completed the 0.6m thick diaphragm wall embedded in the Oligocene clay bedrock and the ground anchors drilled in the overlying granular stratum. The monitoring of the wall deformations by 4 inclinometers and traditional geodetic measurements provided a valuable data for research. The relatively long excavated stage provided an unique opportunity to examine time dependency of wall deformations depending mainly on the decompression and passive deformation of the Oligocene clay which solely consisted the passive support of the wall. The great layout dimensions of the site ensured that 3D effects are negligible and plain strain assumption is acceptable. Beside the simpler Winkler-type subgrade reaction wall models, plastic drained/undrained and fully couple flow analysis type 2D PLAXIS models were created to evaluate the results by calculations based on different principles, to examine the relevance of small strain stiffness of soils, to testify the consideration of time dependent behavior of Oligocene clay by coupled flow analysis.

RÉSUMÉ: Dans le cadre d'un projet d'excavation profonde démarré à Budapest en 2017, la filiale hongroise de Solétanche Bachy a réalisé un écran de soutènement constitué d'une paroi moulée d'épaisseur de 0.6m appuyée par un niveau de tirant d'ancrage et fichée dans le substratum argileux de l'Oligocène de Budapest. Les mesures de déformation inclinomètriques ainsi que les relevés géodésiques traditionnels ont rendu possible une retro-analyse du soutènement. La phase d'excavation en fond de fouille ayant duré un temps remarquablement long, une analyse des déformations en fonction du temps, principalement liée à la décompression et la déformation du massif de butée argileux, a pu être effectuée. Au vu des grandes dimensions du projet les effets tridimensionnels sont négligés et l'hypothèse des « plain strain » est retenue pour l'analyse. En plus du modèle d'interaction sol structure classique de type Winkler, divers calculs aux éléments finis sont établis avec logiciel PLAXIS 2D prenant ou non en compte la dimension temporelle du comportement des argiles de l'Oligocène. En effet, l'utilisation de modèles de sol « drained/undrained » incluant ou non les « small strain stiffnesses » suivant des calculs « plastic » ou couplant calcul des déformations et des écoulements permet justifier l'importance de la prise en compte du facteur temporel dans la modélisation des argiles de l'Oligocène.

Keywords: diaphragm wall, back-analysis, PLAXIS, small strain stiffness, stiff clay

1 INTRODUCTION

A 10 to 14m deep excavation was designed, executed and monitored by HBM Soletanche Bachy in Budapest. The excavation pit was open for about one and a half year before the base slab was finished to structurally support the temporary wall. The monitoring results by 4 inclinometers installed in the cage of the Dwall, provided an unique opportunity to analyse the wall deformations in time and therefore the time dependent behavoir of the clay bedrock providing the passive support of the wall. Thanks to the relatively uniform geotechnical and geometrical conditions, the 2D subgrade reaction and FEM plain strain back-analysis could be used to evaluate different clay model settings.

2 PROJECT INTRODUCTION

2.1 Geometry

Some schematic cross-sections are shown in Figure 1. The analysed part of the excavation pit is surrounded by roads at level -0,3..+2,2mRel.The excavation pit has a uniform final excavation level at -11,60mRel. The top of the 0,6m thick diaphragm wall varies between -3,15..+0,8mRel, while the bottom is uniformly at -14,90mRel. The 1,9-3,3m spaced ground anchors are uniformly at -3,33mRel.

2.2 Geotechnical conditions

Figure 1. presents the representative stratification in the zone of the IN1-IN2 and IN4, quite uniform along the site. The natural stratum are overlayed by a heterogeneous fill. Below that, the well-known stratification of the city centre was found:

- Relatively soft silt/fine sand layers down to -5,5..-8mRel
- Medium density gravelly sand/sand strata down to -9,9..-11,50mRel
- Medium plasticity clay bedrock

Several CPT tests, high quality core drillings with oedometric and UU triaxial laboratory tests were performed to characterize these layers. In addition, in a later stage, geophysical survey were done to measure the shear wave propagation velocity in the clay bedrock.

The fill was hardly characterizable due to its heterogenity therefore it was conservatively described during design. The silt and gravelly sand layers were characterized based on the CPT sounding results and the great amount of experience on these Danube sediments.

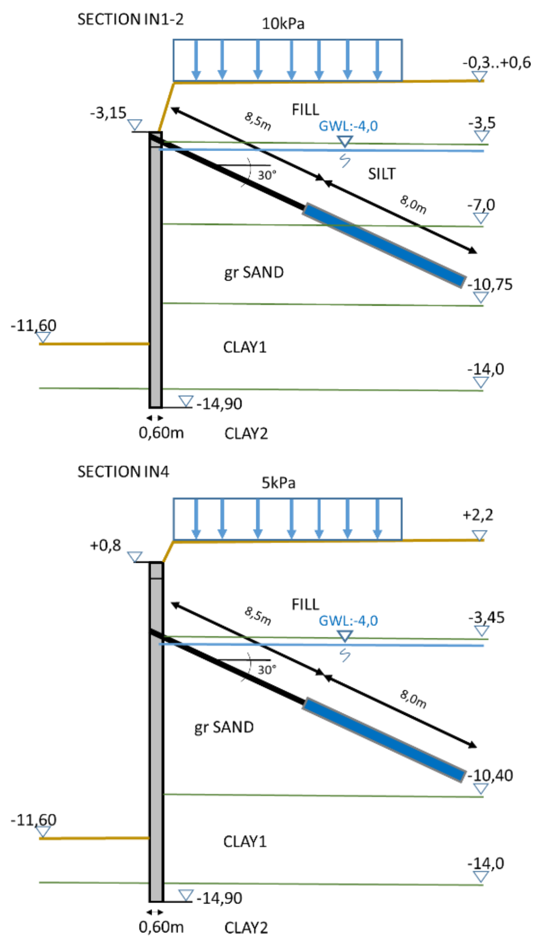

Figure 1. Cross sections of the wall in zone of IN1-IN2 (above) and IN4 (below)

The explored clay bedrock is also well-studied in the past years (Görög 2008 , Kálmán-Horváth 2012) and known as an alteration between stiff clay and weather rock with high, but manifold level of overconsolidation. In case of excavation pits projects, its shallow region is generally characterized as a drained material with high cohesion (60..200kPa) and low friction angle (25..13deg) and with reasonably high oedometric modulus (10..30MPa). In the last few years, thanks to their improving capacity, CPT explorations could penetrate this formation several times, providing results in range of qc=5-80MPa and Rf=2,5-5%. Table 1. lists the measured properties of the clay bedrock.

Table 1. Measured properties of the clay bedrock

Clay BEDROCK		**Top 3m**	**Below 3m**
CPT qc [MPa]	MIN	5	5
	AVG	10+3/m	20+3/m
	MAX	25	35
$E_{oed.200-400}$ [MPa]	MIN	13.7	14.7
	AVG	23.5	22.4
	MAX	35.1	35.7
φ [deg]	MIN	28	22
	AVG	30	32
	MAX	43	51
c [kPa]	MIN	40	60
	AVG	160	170
	MAX	340	480
v_s [m/s]	AVG	300	400

A few notes shall be made for wholity:

- The preparation of the undisturbed samples are quite difficult, even with triple cased core drilling and wtih profound assistance in the laboratory due to the high consistency (Ic=1,3..1,6) and the brittle behavoir.
- Few minutes after opening the cores, crackings could be observed along the sample indicating its high overconsolidation ratio therefore the measured strength and stiffness are assumably lower than in-situ state.
- By evaluation of the oedometric tests a preoverburden pressure of 350-500kPa can be observed, therefore the listed oedometric modulis are representative for the unloading-reloading stage.

2.3 Inclinometric results

As part of the monitoring system, 4 inclinometers were operated by Sixense Soldata along the perimeter of the excavation, as shown in Figure 2. The inclinometers pipes were installed on the steel reinforcement. Therefore the monitored vertical section of the wall extends from the top of the capping beam down to the bottom of the steel cage, about 50cm above the bottom of the wall.

The inclinometric results were controlled by traditional geomatic measurements of the capping beam. Based on those results and the inspection of the rotation at the bottom part of the inclinometers, necessary corrections of 2...7mm were approximated. The measured deflection curves were shifted with these values towards the excavation side to consider the rigid-body movement of the whole wall itself.

Weekly/fortnightly measurements were done for about 17-18 months. Maximal displacements of 20...30mm were measured in all the inclinometers, conform to the expectations of 0,2% of the total depth of the excavation.

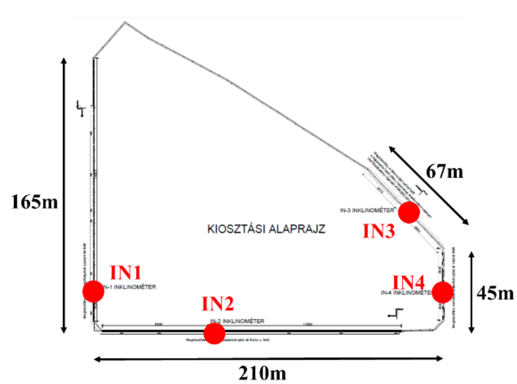

Figure 2. Layout of the pit and the inclinometers

3 BACK-ANALYSIS

3.1 Objectives

The objective of the back-analysis was to evaluate design analysis methods and different clay material model settings by comparison of the monitored wall movements with calculated ones obtained by Winkler-type subgrade reaction models and FEM models. The analysis of clay model settings focused on the following subjects:

- drained/undrained behavoir of clay bedrock as proper choice can hardly be made based on recommendations of Vermeer (1998) due to the high uncertainty of the permeability of the clay,
- time dependency of stress-strain relationship,
- relevance of small strain stiffness.

Table 2 summarizes the calculations performed for all the 4 sections equipped with inclinometers.

Table 2. List of models of the analysis

Method & Soil model	Clay bedrock	Model reference
PARIS Elasto-Plastic	drained	PAR_DRA
	undrained	PAR_UND
PLAXIS Plastic HS	drained	PLX_HS_DRA
	undrained	PLX_HS_UND
PLAXIS Plastic HSS	drained	PLX_HSS_PLA_DRA
	undrained	PLX_HSS_PLA_UND
PLAXIS Fully coupled flow HSS	drained	PLX_HSS_FUL_DRA
	undrained	PLX_HSS_FUL_UND

3.2 Soil model

The parameters of the soil model for FEM and Winkler type models are shown in Table 3. The derivation of those values were shortly introduced in chapter 2.2 but a few additional comments must be made:

- After a first comparison monitored movements and calculation results by original design soil parameters, the parameters of the fill were significatnly improved to have a nice fit of measured and calculated deformation curves in IN3 and IN4 for cantilever stage of wall before anchoring.
- The stiffness parameters of the clay bedrock were considered as the measured average oedometric modulus of 23,5MPa are representative as unloading-reloading modulus. Its value was increased to 30MPa to consider reduction effect of the disturbation of drilling.

Table 3. Soil parameters in the back analysis

Soil model	Fill	Silt	gr Sand	Clay top 3m	Clay
γ_{unsat} kN/m^3	18	20	19	21	21
γ_{sat} kN/m^3	20	21	22	22	22
e_{init}	0.8	0.8	0.5	0.4	0.4
E_{50} MPa	15	6	30	10	20
E_{oed} MPa	15	6	30	10	20
E_{ur} MPa	45	18	90	30	60
c kPa	30	10	0	110	130
Φ ∘	24	24	35	28	28
Ψ ∘	0	0	5	0	0
su kPa	-	-	-	330	400
$K_{0,x}$	0.59	0.59	0.43	0.53	0.53
$G_{0,ref}$ MN/m^2	75	75	130	200	300
$\gamma_{o,7}$	10^{-4}	10^{-4}	2.10^{-4}	10^{-4}	2.10^{-4}
R_{inter}	0.67	0.67	0.67	0.67	0.67
POP kPa	0	0	0	450	450
k_{avg} m/s	10^{-6}	10^{-7}	10^{-5}	10^{-10}	10^{-11}
k_H MPa/m	15	6	30	30	60

3.3 Construction stage

The modelled construction stages are:

- Preparation of platform and diaphragm wall
- Excavation with dewatering 0,8m below anchor level
- Anchoring and pre-stressing to 550-600kN
- Excavation and dewatering to -11,6mRel

The time-dependent analysis included the separation of last stage into 18-21 substages to considered the real progress in time.

3.4 PARIS models

Winkler-type models were made with PARIS, the internally developed software of Soletanche Bachy. The subgrade reaction modulus was taken equal to the oedometric modulis of the sedimentary layers and to the unloading-reloading modulus of the clay. The earth pressure theory of Kerisel-Absi were used with 2/3*Φ friction along the wall surface. The undrained models of the clay bedrock was prepared using Caquot earth pressure theory with zero friction angle and cohesion equal to undrained shear strength.

3.5 PLAXIS models

PLAXIS 2D was used for the FEM type analysis. Hardening soil (HS) and hardening soil small (HSS) models were applied for all the layers in the different computations. Eur=3*Eoed was applied for all layers. The G0 values were estimated using formulas listed by Benz (2007) and from the shear wave velocity measurements. The models with undrained clay behavoir, the undrained B option of PLAXIS was used with undrained shear strength. Plastic type analysis were used to estimate the strain values at theoretically infinite duration of construction and fully couple flow computation was down to consider time effect. In latter case construction stages were defined conform to site activities with 7-122 day long durations.

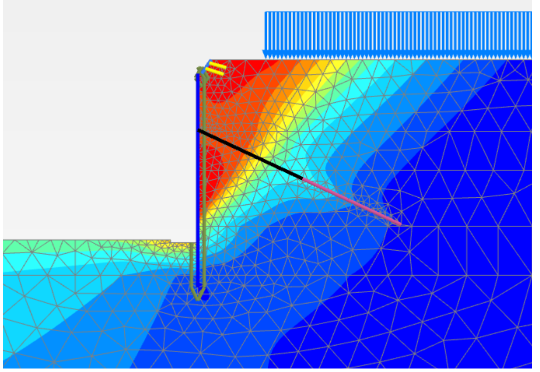

Figure 3. Soil strains in PLX_HSS_FUL_DR

4 RESULTS

The monitored deflections in final stage can be seen on Figure 4 (IN2) and 5 (IN3) in comparison with calculated values using drained models. Very similar trends were found for section IN1, IN2 and IN4: the calculated deformations by model PAR_DRA and PLX_HSS_PLA_DRA fits very well on monitored ones, while wall deformation by PLX_HSS_FUL_DRA underestimates them with about 30-50%. The neglection of small strain stiffness results in strong overestimation of wall deformation as shown by the results of PLX_HS_PLA_DRA.

In case of section IN3, the trends of the calculated results are the same as presented for IN1, IN2 and IN4. On the other hand, the deformations are underestimated by PAR_DRA and PLX_HSS_PLA_DRA models, as well. A pedestrian by-pass structure lays slightly behind the active soil wedge in the zone of IN3 resulting in some modeling uncertainty, especially for the cantilever stage before the realization of the anchors.

Figure 6 presents the comparison of monitored and calculated wall deflections by undrained clay models for IN1. The 7-9cm high wall deformations by PLX_HS_PLA_UND model are not presented for better visibility of others. Section IN1, IN2 and IN4 showed again very

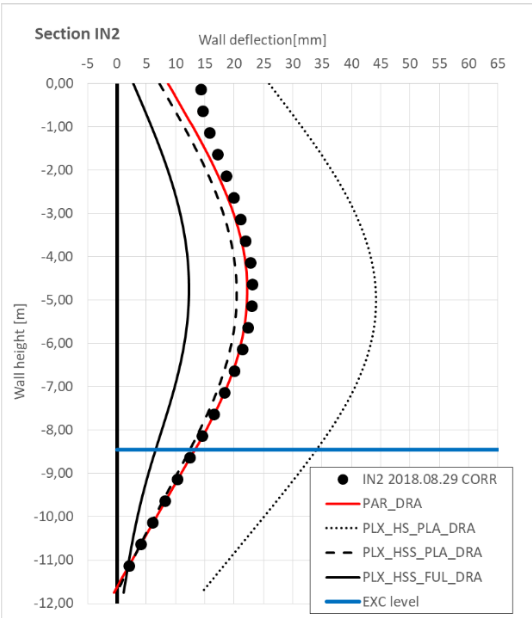

Figure 4. Measured and computed wall deflection for IN2 – DRAINED models

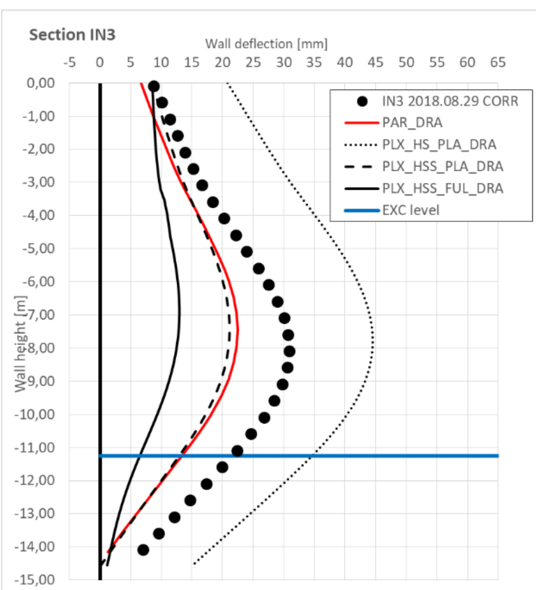

Figure 5. Measured and computed wall deflection for IN3 – DRAINED models

similar trends with each other: curves by PAR_UND and PLX_HSS_FUL_UND models fit well with monitored values, however the latter

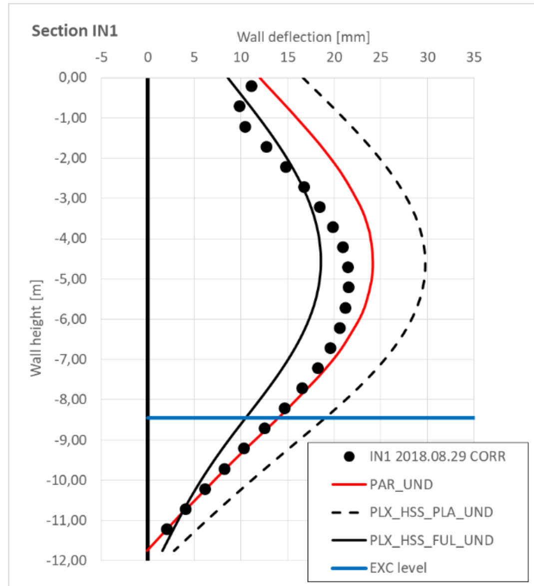

Figure 6. Measured and computed wall deflection for IN1 – UNDRAINED models

are generally underestimates the measurements with 5-20%. The PLX_HSS_PLA_UND results are generally overestimates the movements with about 10-25%. The evalaution of the computations showed that section IN3 is an exception again: the wall deflection by PLX_HSS_PLA_UND are in perfect match with monitored values, while PAR_UND and PLX_HSS_FUL_UND underestimates the deformation with 20-30%.

Figure 7 shows the measured and computed maximum wall deflection vs time curves for IN1. Very similar results were found for all the cases: the displacements are overestimated in the first stages; after the excavations of the final 1-3m of soil, the computed maximum wall deflections are lower than the measured ones. Drained models shows always lower deformations and therefore greater difference from measurements than results by undrained models.

The bending moment curves by the different models for IN2 are shown in Figure 8. PLAXIS HSS plastic type (black curves) and PARIS

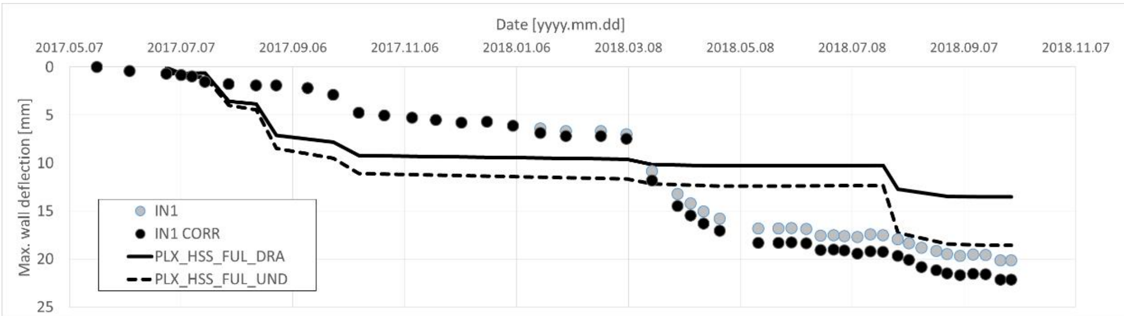

Figure 7. Measured and computed wall deflection vs time in section IN1

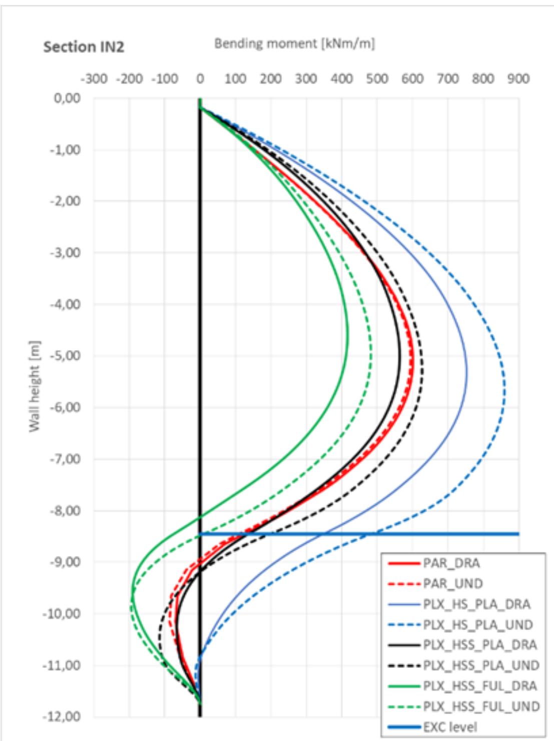

Figure 8. Bending moments IN2

results (red curves) are practically the same, regardless of drainage type of clay. The maximum bending moments by PLAXIS fully coupled flow analysis (green curves) are 15-30% lower as a consquence of stiffer behavoir clay. The simple PLAXIS HS soil model results (blue curves) imples softer clay behavoir, resulting in 20-50% higher maximum bending moments compared to black and red curves.

The anchor forces for all the section are compared in Table 4. Anchor forces are significantly lower in PARIS models compared to any PLAXIS models. While the PARIS and PLX_HSS_PLA_DRA models showed very similar wall deformations and bending moments, the resulting anchor forces are 23-30% higher in all PLAXIS models.

Table 4. Comparison of calculated anchor forces

Max. anchor force [kN]	IN1	IN2	IN3	IN4
PAR_DRA	800	757	672	726
PAR_UND	793	750	674	722
PLX_HS_PLA_DRA	1126	1076	898	948
PLX_HS_PLA_UND	1207	1155	941	1047
PLX_HSS_PLA_DRA	1033	984	831	897
PLX_HSS_PLA_UND	1082	1033	838	968
PLX_HSS_FUL_DRA	873	834	744	787
PLX_HSS_FUL_UND	925	877	757	863

5 CONCLUSIONS

As a summary, it can be stated, that Winkler-type subgrade reaction models, as the most widely used method for design of deep excavations, showed very well correalation with monitored wall deflections. However appropriately precise analysis could be made with more complex FEM models by consideration of small strain stiffness of soils.

Against the presented difficulties, the theoretical conrtadiction of using strength values from UU type triaxial test as effective parameters for drained models, the characterization of the shallow region of the bedrock could realistically be done. Due to the shallow embedment into the clay, the drainage type, the choice on total or effective stresses has lower relevance but in other cases it can be more important.

If undrained strength parameters can be more reliably derived, the wall has a limited duration of relying on the clay‘s passive resistance, it can be advisably to use an undrained model for clarity. However stiffness parameters shall be defined with care to predict accurately wall deformations an avoid significant over- or underestimation. As in Winkler-type elasto-plastic models very minor plastification can occur due to the high strength values of the clay, the drainage type of clay has negligible inluence.

It is clear, that by FEM analysis it necessary to consider small strain stiffness of soils to find the proper range of displacements, even if just cautios estimation of required parameters can be done based on literature. However geophysical in-situ test can be a quick and economic way to confirm results of such estimations.

The subsequent inclinometric results showed the evolution of wall displacement in time, highlighting the time dependent stress-strain relationship of the clay bedrock. Great efforts were made to prepare fully coupled flow analysis models with PLAXIS considering the real duration of excavation phases. The trend of the maximal displacement-time diagrams correlates quite well with the measured values, but the deformations are overestimated in the period of shallow excavation levels and generally underestimated in the final stages. Such analysis are hardly feasable in design stage, but potentially still useful to estimate relevance of construction time on evolution of strains in soil, especially for deeper excavation in softer clays where higher stains in cohesive soils can be expected.

It was shown that bending moments are less sensitive for the analysis method and the settings of the clay model but small strain stiffness shall be considered in FEM models to obtain similar results with subgrade reaction models. On the other hand, consideration of time by a fully coupled flow analysis, can lead to much lower bending moments, therefore it shall not be applied directly for reinforcement design.

Comparison of anchor forces showed that any FEM model results are significantly higher than values obtained by the subgrade reacion models. In PARIS the anchor stiffness is proportional to the apparent free length, which is adjusted manually. In PLAXIS the designed anchor structure and geometry is modelled with some assumptions on skin friction along the bonded length. Therefore evaluation is quite impossible without in-situ measurements of anchor forces.

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