Periodica Polytechnica Civil Engineering, 67(3), pp. 659–670, 2023

Two- and Three-dimensional Numerical Back-analysis of Deep Excavation Case Studies from Budapest, Hungary

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Received: 09 January 2022, Accepted: 07 March 2023, Published online: 14 March 2023

Abstract
In this paper, the numerical back analyses of four, typical, monitored deep excavations completed in Budapest are presented. The typical excavation solution in Budapest city center, down to 15–18 m excavation depth, is a diaphragm wall embedded in the clay bedrock and supported by prestressed anchors embedded in the sedimentary soils above the clay. In these case studies this solution is analyzed with traditional Winkler type and more complex PLAXIS 2D and 3D finite element models. The focus of the study was to compare the measured wall deformations with the calculated ones derived by the listed methods. As the clay bedrock is a deterministic layer for the wall behavior, several different FEM models were prepared to analyze the appropriateness of the potential constitutive models for its proper characterization. As a conclusion, practical proposals were made for practitioners for future excavations.

Keywords
deep excavations, back-analysis, small strain stiffness, FEM, 3D modelling

1 Introduction

Since the middle of the 1990s, many deep excavation projects were finished in Budapest. Except for the very deep excavations for metro stations, conventional bottom-up method is applied down to ~15–18 m excavation depth, typically with ground anchor supported diaphragm walls. Generally, the 50–80 cm thick diaphragm walls are embedded into the low permeability clay bedrock underlying most part of the city. The 14–23 m long, injected, prestressed, temporary ground anchors are tied into the sedimentary gravelly-sandy layers overlying the clay. These can safely support the wall and limit its deformation to an acceptable level with regards to the adjacent buildings. Though many projects have been successfully finished during the last 25 years in these similar conditions, comprehensive evaluation of design methods by comparison with monitoring results hasn't been done so far.

One of the focus areas of our research project is the back analysis of a few, well monitored deep excavation works with these typical characteristics. In this paper, the numerical back-analysis of 4 deep excavation with 2D subgrade reaction, "Winkler-type" models and 2D finite element models are presented, as these tools are more often used by practitioners. Furthermore, 3D finite element models have been prepared for 2 of these projects to evaluate its applicability and to compare its results with more common 2D calculation results. The motivation of the latter subject is that many studies [1–4] have shown the potential of 3D modelling but, except in the case of special projects, it has not been spread in practice so far.

Introduction of the back analyzed deep excavation projects is given first. Then the concept of the back analysis series is presented including the relevant engineering and modelling characteristics. The results of the projects are mainly focusing on comparison of measured wall deformations to the calculated ones.

2 Back analyzed case studies

2.1 General notes

Previous papers by the authors introducing the preceding studies led to present comprehensive research:

- Detailed analysis of 9 case studies' monitoring database from Budapest [5] to find patterns in the wall movements and surface settlements for the anchors and propped excavations.

Cite this article as: Szepesházi, A., Móczár, B. "Two- and Three-dimensional Numerical Back-analysis of Deep Excavation Case Studies from Budapest, Hungary", Periodica Polytechnica Civil Engineering, 67(3), pp. 659–670, 2023. https://doi.org/10.3311/PPci.21829

- 2D and 3D back analysis of single projects in Budapest [6–11] to answer principal questions on soil models and 3D model settings.
- Parametric comparison of 2D and 3D calculation results for the typical deep excavation solutions in Budapest [12] following the concept of studies of [13] and [14].

Finally, 4 projects were chosen for back-analysis due to the following:

- The ground conditions and the geometry are quite typical for the Budapest center. Therefore, the conclusions are applicable for many future projects.
- The quality of the soil data and the monitoring results are adequately detailed and precise for research objectives.

2.2 General project characteristics

The case study projects main characteristics:

- Geometry:
    - 2–5 underground story of a new office/hotel buildings in the center of Budapest with the need of approximately 9–18 m deep excavation below ground surface.
    - Typically, 30–150 m wide excavation pits with negligible 3D effects at the location of the monitored sections out of the corners' zones.
- Ground and groundwater conditions:
    - 2–5 m thick layer of man-made fill at the surface.
    - Fine/coarse river sediments to 10–15 m depth.
    - Miocene/Oligocene impermeable clay bedrock.
    - Average ground water level at 4–8 m depth.
- Deep excavation solutions:
    - Retaining wall by 50 and 60 cm thick diaphragm wall embedded in the clay bedrock to provide a watertight base for the excavation.
    - Drilled, injected, prestressed, temporary ground anchors embedded in the sediments of the Danube to provide wall stability and limit deformations until the construction of the internal structure.
    - Monitoring by inclinometers installed in the diaphragm wall and the traditional geodetic measurements to control inclinometer results and to monitor adjacent buildings.

For simplicity, the projects are referred as Project "A", "B", "C" and "D" hereinafter as detailed in Table 1. Fig. 1 shows a photo of Project "D" under excavation.

Table 1 General project characteristics for back analysis

Project	Excavation depth [m]	Clay bedrock top level's depth [m]	Diaphragm wall thickness [m]	No. of anchor levels [no.]	No. of inclino-meters [no.]
"A"	9	13	0.6	1	2
"B"	11–13	11–13	0.6	1	4
"C"	16	13	0.6	2	1
"D"	18	15	0.6	3	3

Fig. 1 View of Project "D"

2.3 Design and geotechnical project characteristics

Schematic cross-sectional drawings of the analyzed deep excavations are shown in Figs. 2–5 indicating its geometry, some structural parameters, and the stratification, as well. Their design in practice was based on Winkler-type models to calculate deformations and internal forces and on supplementary calculations to check overall stability, Kranz-stability, etc. The deterministic ground parameters are the subgrade modulus and the strength parameters which were empirically predicted from CPT test results and from triaxial and oedometric tests when undisturbed sampling of clay bedrock was feasible. In case of Project "D" with regards to its great depth, 2D finite

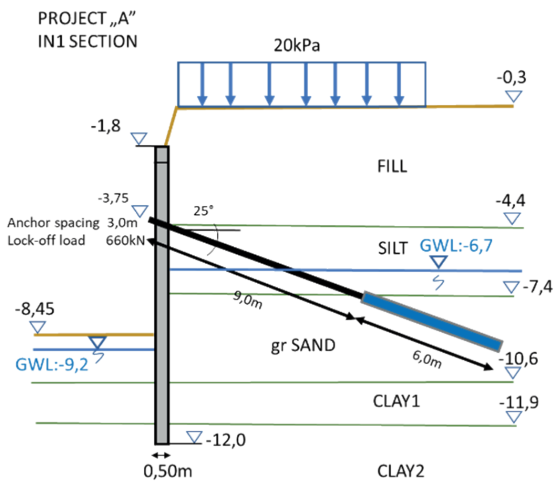

Fig. 2 IN1 section and soil characteristics of Project "A"

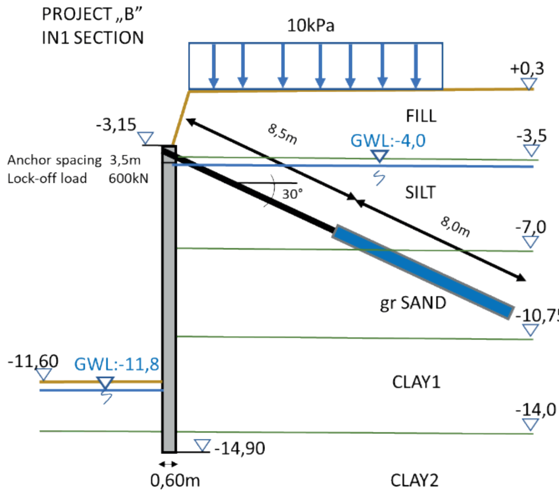

Fig. 3 IN1 section and soil characteristics Project "B"

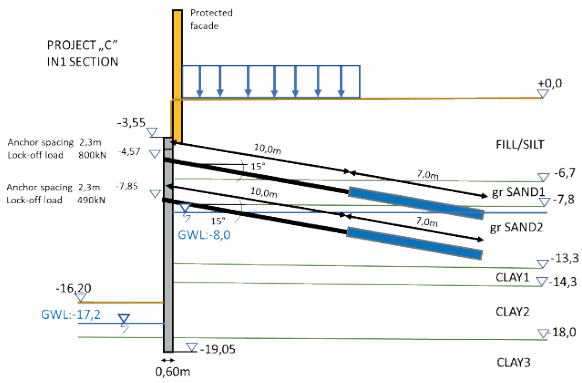

Fig. 4 IN1 section and soil characteristics of Project "C"

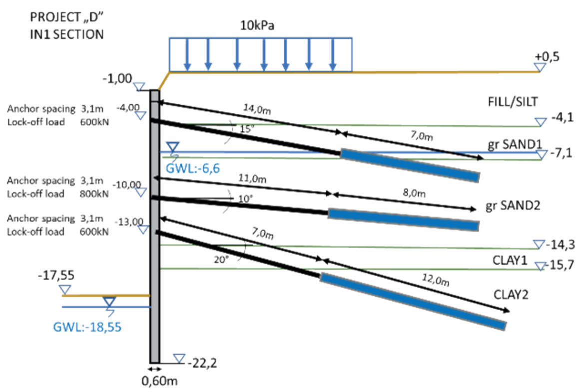

Fig. 5 IN1 section and soil characteristics of Project "D"

element PLAXIS models were prepared to have independent calculations without the limitations of Winkler type models and therefore increase calculation reliability.

Original design of these projects was done by 3 different design offices with different considerations; therefore, their calculated results are not shown here. Nevertheless, it can be noted, that their applied characteristic soil properties for design are in general slightly more conservative compared to the below presented parameters which provided the best fit of calculated and measured wall deformations.

The clay bedrock formation, as deterministic strata for wall behavior, is variably overconsolidated or cemented with relatively high strength (c_u = 60–500 kPa) and stiffness (CPT q_c = 3-80 MPa, E_{oed} = 10–80 MPa) and with a weathered or softened top zone of 1–5 m. Previous comprehensive studies [15–16] provide a wide range of mechanical properties but time dependent mechanical behavior was not investigated in detail. In design, generally drained behavior is considered with cautious strength and stiffness parameters. However, in some cases deformation generation lasted for several months [6] and the inherent uncertainty in determining the permeability of this formation is at least one order of magnitude (k value is in range of $10^{-10}\ldots10^{-12}$ m/s). Considering that the full excavation phase of such excavations generally lasts for 1–4 months until foundation is completed, precise choice of drained or undrained modelling are hardly done in practice.

3 Objectives and concept of the back analysis series

Some of the main questions in local design practice of deep excavations are the following:

- Can Winkler-type models be adequately precise for excavations deeper than 12–15 meters?
- In which cases it is necessary or profitable to build 2D or 3D finite element models with complex constitutive models for the design?
- Are CPT tests appropriate tools to estimate clay bedrock parameters for excavation design?
- How subgrade reaction modulus could be approximated for the clay bedrock of Budapest?
- What kind of clay soil model is recommended?

Looking for the answers the following calculation models were built to back analyze the introduced case studies: 2D subgrade reaction (Winkler-type) models by PARIS software of Soletanche Bachy, 2D FEM models by PLAXIS 2D and 3D FEM models by PLAXIS 3D for project "C" and "D". All the finite element models were built up with "hardening soil model with small strain stiffness" (HSS) soil model considering hardening behavior and small strain stiffness as previous studies ([6–12]) proved its general appropriateness. "Plastic" type calculations were done excluding direct consideration of time dependency. However, as drainage of clay bedrock is relevant, all FEM models were calculated with drained, undrained "A" and undrained "B" type clay models using the same strength and stiffness parameters. The analysis of the results was done for the final excavation phase. Table 2 shows the list of the 46 models with the nomination used hereinafter.

4 Details of numerical models

4.1 Soil model

CPT tests are used for soil characterization in Budapest for nearly a decade since penetration into the clay bedrock has become possible by appearance of modern CPT trucks. Even if this clay bedrock was well studied previously [15, 16], no scientific study was done to find empirical correlation between CPT results and stiffness parameters. Therefore, the proper choice of stiffness parameters in practice was the designer's responsibility in general.

Table 3–6 indicate the average CPT q_c results and the applied soil parameters in FEM models of Project "A", "B", "C" and "D" after 2–3 iterative corrective steps to better correalte calculated wall deformations with measured ones. The iteration continued until finding an overall good fit for all monitored sections using the same parameters within the same project. However, achieving a perfect fit was not possible for the different monitored sections of the same project using the same soil parameters. This implies that natural heterogeneity of the soil results in a limit of achievable accuracy by calculations.

During the iteration, the following principles were kept:

- Strength parameters were estimated based on CPT test results mainly in line with the proposals of the soil reports.
- Stiffness parameters were revised based on CPT test results following the given empirical correlations in EN 1997 standard. These correlations are providing a potential interval of correlation factors and iteration was done to find a better fit by changing parameters within this interval. As shown in below table, the clay bedrock was characterized using formulae of E_{ur} = 2.5÷3 × $q_{c.avg}$ and by accepting E_{oed} = E_{50} = 3 × E_{ur}.
- In Winkler-type models same strength parameters were applied as in FEM models. The subgrade reaction modulus was defined to be equal to the oedometric modulus of the soils derived from CPT results.

Table 2 List of models – IN1/2/… indicates the analyzed sections monitored by inclinometers

Software	Clay bedrock soil model		Project "A" IN1/IN2	Project "B" IN1/IN2/IN3/IN4	Project "C" IN1	Project "D" IN1/IN2/IN3
PARIS	Drained"		A_IN1/2_WIN	B_IN1/2/3/4_WIN	C_IN1_WIN	D_IN1/2/3_WIN
PLAXIS 2D	HSS	Drained	A_IN1/2_2D_DRA	B_IN1/2/3/4_2D_DRA	C_IN1_2D_DRA	D_IN1/2/3_2D_DRA
		Undrained A	A_IN1/2_2D_UNDRA	B_IN1/2/3/4_2D_UNDRA	C_IN1_2D_UNDRA	D_IN1/2/3_2D_UNDRA
		Undrained B	A_IN1/2_2D_UNDRB	B_IN1/2/3/4_2D_UNDRB	C_IN1_2D_UNDRB	D_IN1/2/3_2D_UNDRB
PLAXIS 3D	HSS	Drained	-	-	C_IN1_3D_DRA	D_IN1/2/3_3D_DRA
		Undrained A	-	-	C_IN1_3D_UNDRA	D_IN1/2/3_3D_UNDRA
		Undrained B	-	-	C_IN1_3D_UNDRB	D_IN1/2/3_3D_UNDRB

Table 3 Final soil model of Project "A"

Layer			Silt	grSand	Clay1	Clay2
CPT results	q_c [MPa]	MIN	1	15	3	5
		AVG	2	25	4	15
		MAX	2	50	6	40
Friction angle	φ	°	26	35	15	28
Cohesion	c	kN/m^2	5	0	60	110
Oedometric modulus	E_{oed}	MN/m^2	6	40	4	15
Secant modulus	E_{50}	MN/m^2	6	40	4	15
Unload.-reload. mod.	E_{ur}	MN/m^2	18	120	12	45
Index of hardening	m	-	0.7	0.5	0.9	0.9
Undrained shear strength	s_u	kPa	-	-	100	350
Inittial shear stiffness	$G_{0,ref}$	MN/m^2	75	130	100	180
Threshold shear strain	$\gamma_{o,7}$	-	0.0001	0.0002	0.0002	0.0004
Overcons. ratio	OCR	-	1	1	1	1
Pre-overburden pres.	POP	-	0	0	0	300
Subgrade react. mod.	k	MPa/m	6	40	4	45

Table 4 Final soil model of Project "B"

Layer			Silt	grSand	Clay1	Clay2
CPT results	q_c [MPa]	MIN	1	5	5	10
		AVG	3	16	11	20
		MAX	10	25	30	35
Friction angle	φ	°	24	35	28	28
Cohesion	c	kN/m²	10	0	110	130
Oedometric modulus	E_{oed}	MN/m²	6	30	10	20
Secant modulus	E_{50}	MN/m²	6	30	10	20
Unload.-reload. mod.	E_{ur}	MN/m²	18	90	30	60
Index of hardening	m	-	0.7	0.5	0.9	0.9
Undrained shear strength	s_u	kPa	-	-	330	400
Initial shear stiffness	$G_{0,ref}$	MN/m²	75	130	200	300
Threshold strain	$\gamma_{o,7}$	-	0.0001	0.0002	0.0001	0.0002
Overcons. ratio	OCR	-	1	1	1	1
Pre-overburden pres.	POP	-	0	0	450	450
Subgrade react. mod.	k	MPa/m	6	30	30	60

Table 5 Final soil model of Project "C"

Layer			Fill/Silt	grSand1/ grSand2	Clay1/Clay2/Clay3
CPT results	q_c [MPa]	MIN	-	20	4/5/10
		AVG	1	40/25	5/8/13
		MAX	-	55/40	6/10/16
Friction angle	φ	°	30	38/36	28
Cohesion	c	kN/m²	10	1	60/80/110
Oedometric mod.	E_{oed}	MN/m²	10	80/50	5/8/12
Secant modulus	E_{50}	MN/m²	10	80/50	5/8/12
Unload.-reload. mod.	E_{ur}	MN/m²	30	240/150	15/24/36
Index of hardening	m	-	0.6	0.5	0.9
Undrained shear strength	s_u	kPa	-	-	120/190/330
Initial shear stiffness	$G_{0,ref}$	MN/m²	80	200/130	120/150/180
Threshold strain	$\gamma_{o,7}$	-	0.0002	0.0002	0.0001/0.0005/0.0005
Overcons. ratio	OCR	-	1	1	1
Pre-overburden pres.	POP	-	-	-	0/150/300
Subgrade react. mod.	k	MPa/m	30	80/50	5/24/36

Table 6 Final soil model of Project "D"

Layer			grSand1	grSand2	Clay1	Clay2
CPT results	q_c [MPa]	MIN	40	15	6	15
		AVG	50	30	8	18
		MAX	75	50	11	22
Friction angle	φ	°	38	34	15	28
Cohesion	c	kN/m²	1	1	60	200
Oedometric mod.	E_{oed}	MN/m²	80	50	5	16
Secant modulus	E_{50}	MN/m²	80	50	5	16
Unload.-reload. mod.	E_{ur}	MN/m²	240	150	15	48
Index of hardening	m	-	0.5	0.5	0.9	0.9
Undrained shear strength	s_u	kPa	-	-	100	400
Initial shear stiffness	$G_{0,ref}$	MN/m²	130	90	120	180
Threshold strain	$\gamma_{o,7}$	-	0.0001	0.00015	0.0005	0.0005
Overcons. ratio	OCR	-	1	1	1	1
Pre-overburden pres.	POP	-	-	-	0	1000
Subgrade react. mod.	k	MPa/m	80	50	5	48

The only exception is the intact, overconsolidated clay bedrock which had a subgrade reaction modulus equal to the unload-reloading modulus of the soil considered.
- Degree of overconsolidation of the clay bedrock is very varying as previous scientific studies showed [16]. Based on geological information and a few oedometric tests pre-overburden pressure value were estimated for the intact clay bedrock.
- G0 and $\gamma_{0.7.\mathrm{ref}}$ values were estimated to have a good average value of different empirical formulas listed in [17] and in [18]. The precise value has quite a small relevance on the results, the deterministic question is if the clay bedrock is kept in small strain's range or degradation of small strain stiffness is occurring.

4.2 Structural elements
Regarding the structural elements a few remarks needs to be taken:
- Diaphragm walls were modelled with linear elastic plate elements using E = 19.5 GPa and Poisson ratio ν = 0.2. In the 3D models, the horizontal stiffness of the plate element was taken as 20% of the vertical stiffness by adjusting $E_{horizontal}$ = 3.9 GPa.
- Capping beam of the diaphragm wall is modelled as a linear elastic beam element at the top of the plate element of the diaphragm wall.
- The ground anchors were modelled as advised in [19]: free length is modelled as a node-to-node anchor while the injected, fixed part is modelled as a "free grout body" embedded beam element. In 3D the latter element is an embedded pile element (Fig. 6). Their structural parameters are adjusted according to their real parameters while skin resistance of the fixed part was taken to provide an elastic behavior. Pre-stress of anchors were set by adjusting the lock-off load used on site.

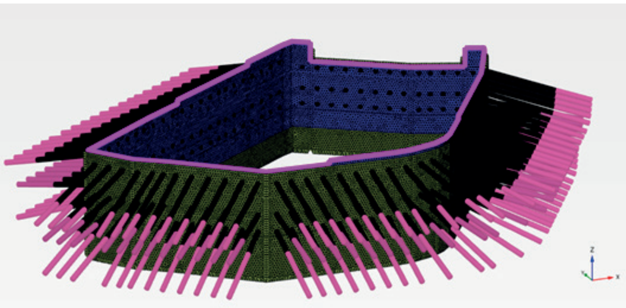

Fig. 6 One of the 3D PLAXIS model of Project "D"

4.3 Other modelling considerations
Construction stages were defined to have all relevant phases in the model until reaching the bottom of the pit by excavation of the soil:
- preparation of working platform,
- installation of diaphragm wall,
- excavation and dewatering down to anchoring platform,
- anchor installation and stressing,
- previous 2 steps repeated as per project conditions,
- reaching final excavation level.

Internal groundwater level was defined 1 m below the given excavation level except were dewatering monitoring data showed greater lowering of groundwater in advance. Interpolation of pore pressures were set in clay bedrock to simply model influence of groundwater head difference due to groundwater lowering inside the excavation pit.

Plastic type calculations were set assuming total consolidation of soil in all the steps. As above the clay bedrock non-cohesive soil lays, this setting is only questionable at final excavation steps were the clay bedrocks presence starts to be relevant. However, consideration of strains time dependency was analyzed by using different drainage conditions as explained in previous chapters.

5 Results
As listed in Table 2, the 10 Winkler-types, 30 finite element 2D and 6 finite element 3D numerical models were prepared to analyze 10 different wall sections. The measured and the numerical results are qualitatively compared section by section. Figs. 7–16. shows some representative result diagrams. The primary focus is on the deformations. Since the bending moments are governing structural design of the excavations, their comparison can be seen below for most of the models, as well. In case of Project "D", some load cells were installed to measure real anchor force. Even if these are not adequate for scientific conclusions, some remarks are included below (see Table 5).

5.1 Winkler-type models
Looking at the results by the Winkler-type models, the following can be observed:
- In case of Project "A" (Figs. 7–8) Winkler-type models provided a nearly perfect fit to measured wall deformations indicating the leaning behavior of the wall. The finite element models provided a different

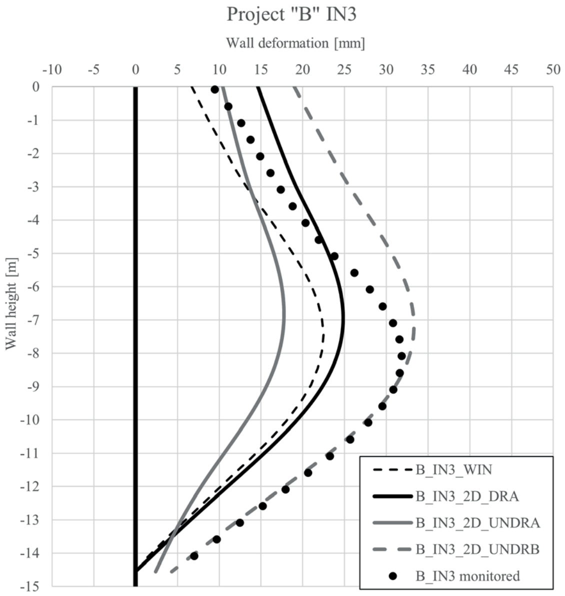

Fig. 7 Wall deformations of Project "A" IN1 section

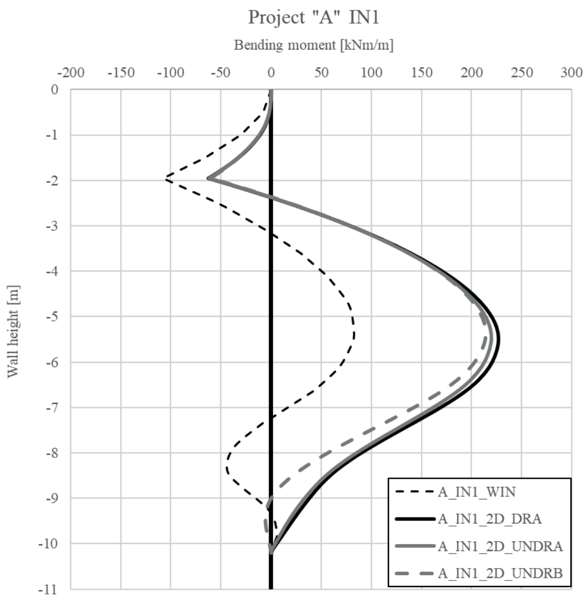

Fig. 8 Bending moments of Project "A" IN1 section

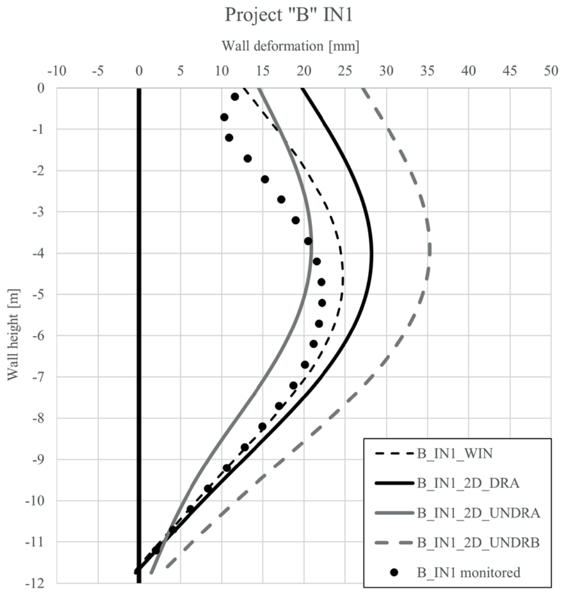

Fig. 9 Wall deformations of Project "B" IN1 section

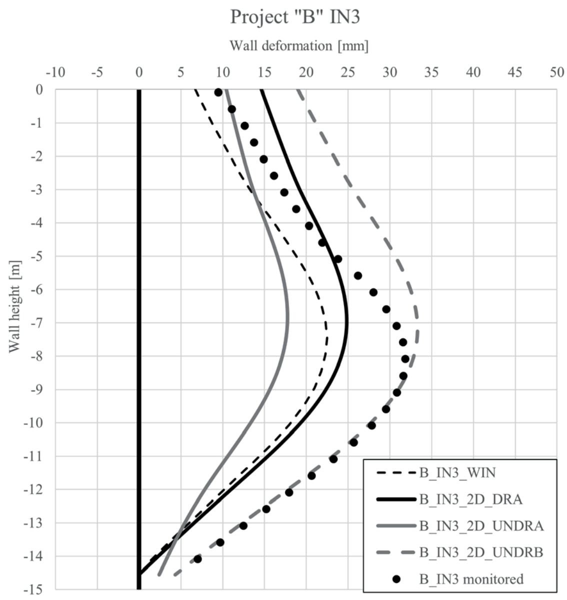

Fig 10 Wall deformations of Project "B" IN3 section

wall behavior with stiff embedment in the clay bedrock at the base of the wall. Regarding the bending moment, the great difference between the Winkler-type and the finite element models needs to be noticed indicating the necessary prudence for the design of such shallow excavation.

- Regarding Project "B" it can be observed that a nearly perfect fit could be found with exception of IN3 where movements were underestimated by about ~5–10 mm (Figs. 9–10). In this section a rigid body movement of the wall was observed on site, however considerable difference in soil characteristics was not explored within the same project. Nevertheless, this implies that soil heterogeneity can result in greater differences than the difference between the results derived by slightly different models applied in this study.
- For the Winkler model for Project "C" (Figs. 11–12) a good fit with measured deformations could be found even if the calculated curvature of wall is higher than the measured one. However, calculated and monitored movements are within a 5 mm range from each other which must be considered as a good correlation.

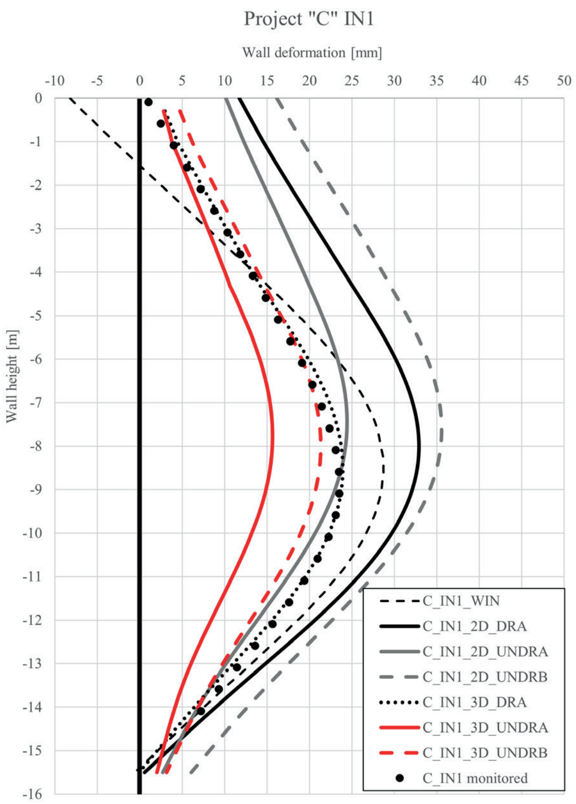

Fig. 11 Wall deformations of Project "C" IN1 section

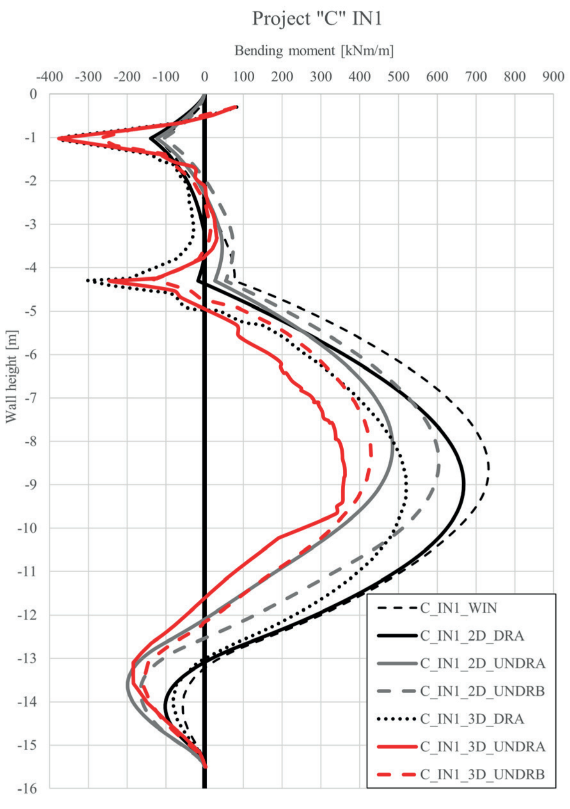

Fig. 12 Bending moments of Project "C" IN1 section

- In case of Project "D" (Figs. 13–16) a quite good fit for Winkler-type models and monitored wall movements were found except in section IN2. The latter section showed somewhat higher movements than any other section showing a creep-like deformation until finishing the base slab. Therefore, similarly to Project "B" IN3 section, soil heterogeneity must be mentioned as a potential cause.
- In general, it can be stated that Winkler-type models can be a reliable tool for the design of excavations even in case of deeper excavations. However, the deeper the excavation the lower the correlation between the calculated and measured deformations.
- The subgrade reaction modulus of the Budapest clay bedrock can be estimated from CPT results for anchored diaphragm walls using the equation k [MPa/m] = 2.5÷3 × $q_{c.avg}$ [MPa]
- Looking at the Table 7, it can be noticed that the anchor forces could be reliably estimated by Winkler-type models in Project "D" IN1 section. As increasing anchor forces are related to wall deformations, it is likely that the other sections would show similarly good fit between calculated and measured forces if latter would exist.

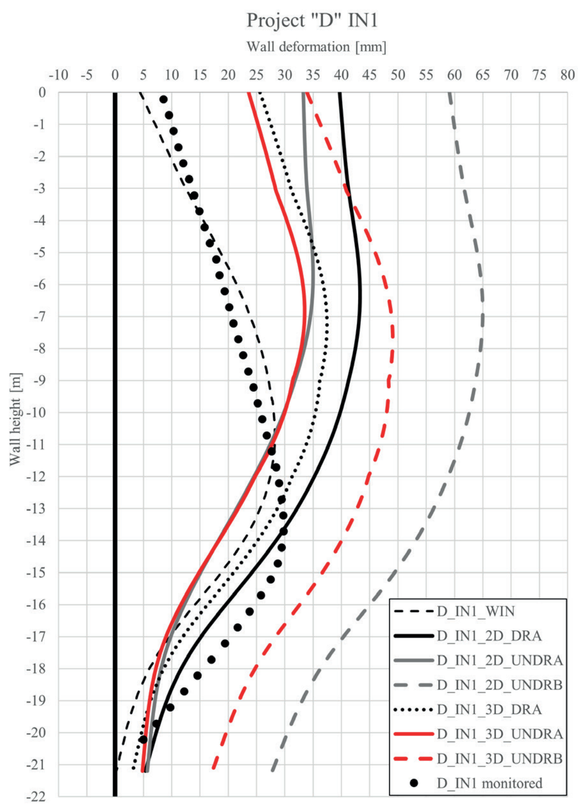

Fig. 13 Wall deformations of Project "D" IN1 section

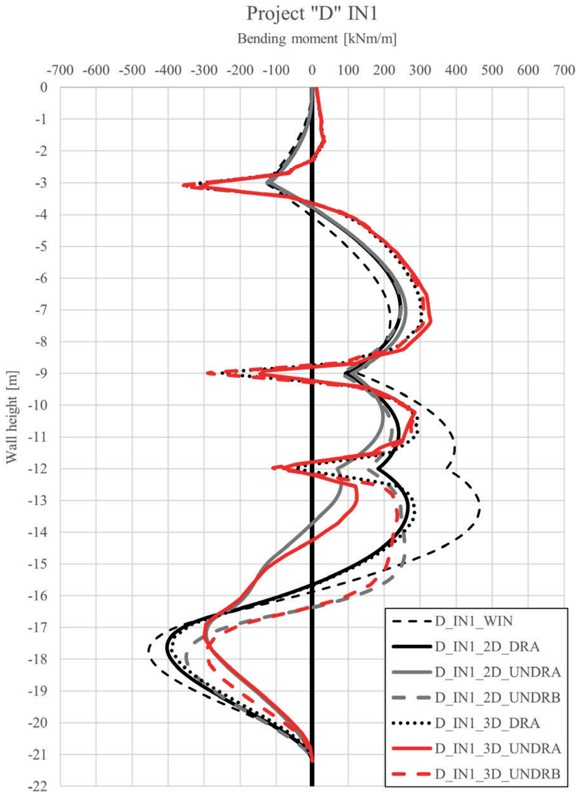

Fig 14 Bending moments of Project "D" IN1section

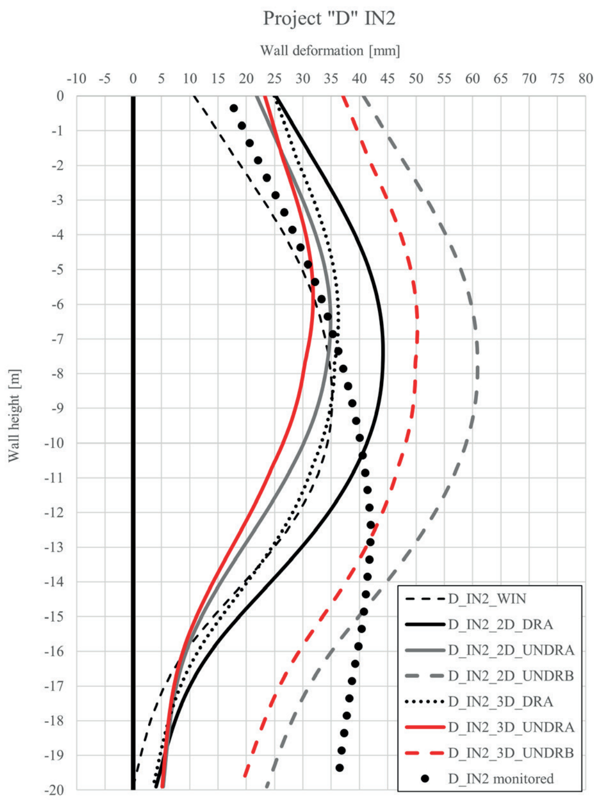

Fig 15 Wall deformations of Project "D" IN2 section

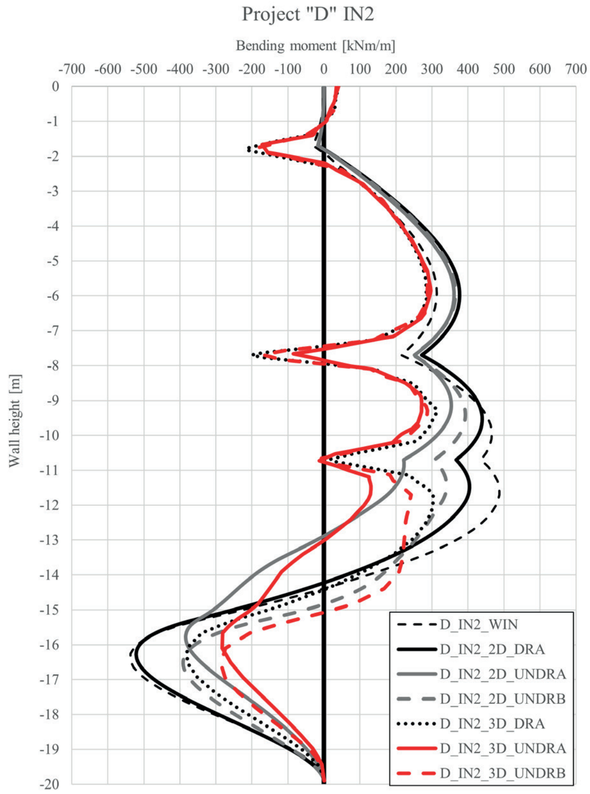

Fig 16 Bending moments of Project "D" IN2 section

5.2 2D finite element models

In comparison with the Winkler-type calculation results, the following can be noticed about the 2D FEM results:

- In case of Project "A", even if the wall is only embedded into the softer clay layer, the wall has too stiff embedment compared to measured deformations. Due to this, bending moments in PLAXIS models are much higher. At the same time, the anchors provide a softer support and therefore show higher movements compared to monitored results and Winkler-type calculations.
- Looking at the curves of Project "B", similarly softer anchor behavior and higher wall deformation at its top part can be observed in most of the sections. The only exception is IN3 section again, where the excessive deformation of the passive soil regime balances the excessive movement of the top of the wall by softer anchors. Excluding IN3 section, the drained type models show the best fit to monitored wall deformations while undrained "A" type shows slight underestimation of movements. Undrained "B" type clay model always gives the biggest wall deformations overestimating the movements.

Table 7 Anchor force results of Project "D" IN1 and IN3 section

Model name	MAX Wall def.	MAX Bending moment per l m of wall		MAX Anchor force		
		Exc. side	Earth side	Row no. 1	Row no. 2	Row no. 3
	[mm]	[kNm]	[kNm]	[kN]	[kN]	[kN]
"D" IN1						
D_IN1_WIN	28,2	465	454	725	1 038	729
D_IN1_2D_DRA	43,3	266	403	873	1 003	793
D_IN1_2D_UNDRA	35,0	260	294	827	913	699
D_IN1_2D_UDRB	65,0	257	350	895	1 061	855
D_IN1_3D_DRA	37,4	304	390	786	999	738
D_IN1_3D_UNDRA	33,5	328	293	763	922	675
D_IN1_3D_UNDRB	49,0	312	291	789	1035	770
D IN1 monitored	30	-	-	810	1030	760
"D" IN3						
D_IN3_WIN	18,7	479	323	611	855	688
D_IN3_2D_DRA	28,6	322	332	709	845	738
D_IN3_2D_UNDRA	21,5	230	238	687	782	672
D_IN3_2D_UDRB	47,0	386	306	723	884	785
D_IN3_3D_DRA	22,8	318	266	670	820	703
D_IN3_3D_UNDRA	19,4	234	204	658	769	654
D_IN3_3D_UNDRB	34,5	345	222	684	855	738
D IN3 monitored	10,0	-	-	-	-	660

- Very similar observations can be made by the results of Project "C" and Project "D" where a reasonably good fit of the embedded part by drained and undrained "A" type clay was found but anchors show soft behavior and ~10 mm more deformation at the top part of the wall can be observed again. Undrained "B" type model looks conservative by overestimating the deformations. The only exception is IN2 section of Project "D", but its potential reason was explained at summary of Winkler-type models' results above.
- In general, looking at the deeper parts of the walls, the use of soil constitutive models with hardening behavior and small strain stiffness provides realistic calculations with plain strain conditions. Application of formulas $E_{ur} = 2{,}5 \div 3 \times q_{c.avg}$ and $E_{oed} = E_{50} = 3 \times E_{ur}$ are adequate and rough estimation of small strain parameters based on literature can be appropriate.
- The clay bedrock can be modelled as a drained material to get realistic wall deformations. Even if this assumption led to underestimation of wall deformations in 2 sections. However, this is most probably due to soil heterogeneity and therefore drained clay model with an adequately detailed soil exploration program can be a recommended way. Nevertheless, undrained "B" type clay model can be used as a conservative tool providing overestimation of deformations in most of the cases. Application of Undrained "A" type models are not recommended for Budapest clay bedrock, underestimation of deformations and bending moments can occur.
- Anchors are always providing a softer support in 2D finite element models compared to measured wall deformations and Winkler-type models. The finite element grout body of anchors showed 5–10 mm excessive movements towards the pit. This phenomenon is excluded from the Winkler-type models where a notional fix point of the anchors must be defined.
- More varying tendencies of bending moments can be observed, and it is hard to draw up clear proposal. However, drained clay model shows curves quite similarly to Winkler-type ones used by practitioners for a long time.

5.3 3D finite element models

3D models were prepared for Project "C" and "D" only as their application for shallow and wide excavations like Project "A" and "B" was considered not profitable in practice. Looking at Figs. 11–16, the following can be noticed:

- 3D models show slightly smaller deformations and bending moments compared to the results by 2D models with the same clay model. This is most probably due to the 3D effects like plate model of diaphragm wall, stiffening influence by the presence of corners of excavation wall and reduced earth pressure due to arching.
- In present study, we analyze cross-sections which are located out of the corner of the excavations. It is important to notice, that in these conditions, the difference between 2D and 3D models' results are lower than the difference between models with different drainage settings. Nevertheless, using 3D numerical models, application of drained clay model can be recommended similarly to 2D observations.
- The anchor behavior and the deformation of the top of the wall is closer to the measured results. However, in case of Project "D" softer anchor support can be observed again.
- In general, it can be stated that 3D models can result in slightly more realistic models compared to 2D finite element models, but deviant anchor behavior could not be eliminated.
- Reduction of bending moments can be quite large in 3D models compared to other ones, especially to Winkler-type models indicating that further studies on 3D modelling can potentially be profitable to find savings on structural works.

6 Conclusions

The research study showed that Winkler-type models can be adequately precise for the design of 8–18 m deep excavations by anchor supported diaphragm wall in the introduced conditions. However, the deeper the excavation the greater the potential inaccuracy of the results and the need for control calculations by more sophisticated numerical models. The subgrade reaction modulus for the intact Budapest clay bedrock can be estimated from the CPT results using the equation k [MPa/m] = 2.5÷3 × $q_{c.avg}$ [MPa].

By 2D finite element calculations, with soil models considering hardening and small strain stiffness behavior, realistic models can be prepared supplemented by their inherent advantages compared to Winkler-type models. However wider active soil wedges and greater anchor grout body movements can result in excessive deformations along the anchor supported part of the diaphragm wall in comparison with monitored results. Time dependent deformations of clay could be relevant for this kind of deep excavations, but detailed consolidation study is too complex for design purposes.

Using E_{ur}[MPa] = 2.5÷3 × $q_{c.avg}$[MPa] and $E_{oed} = E_{50} = 3 \times \mathrm{E}_{ur}$ equations and applying of drained behavior can lead to realistic wall behavior even if underestimation of wall deformations can happen. Using the same formula to derive stiffness parameters and adjusting Undrained "B" type clay behavior is a conservative method to predict wall deformation.

Bending moments calculated by plain strain Winkler type and finite element models are in the same range. However, in case of greater excavation depth, difference can increase, and Winkler-type models can give higher values than finite element models. Anchor forces are in the same range for all models and about 5–15% difference of calculated results could be observed between the different models.

The preparation of 3D models still needs much greater efforts compared to any 2D model. However, a single 3D model can replace several 2D models on the same project. The introduced settings can lead to realistic wall behavior where positive influence of the corners on deformations and bending moments can be exploited during design. The difference between the results of 3D models with different drainage settings is slightly smaller compared to the 2D ones but similar tendencies could be observed between the different models. Wall deformations and bending moments are somewhat smaller compared to the same 2D models' results but the difference is not exceeding the differences due to different drainage settings. In case of 3D models, similarly to 2D versions, the application of drained clay can be recommended. The unrealistically soft anchor behavior is present in the 3D models too, but the magnitude of the anchor body movement is mitigated compared to 2D results. To conclude, even if present design tools are adequate for routine design works, the 3D models should become the tool to design deep excavations without engineering simplifications. Nevertheless, further improvement of the 3D tools is necessary to minimize required efforts to build complex geometry and to run excessive calculations.

Acknowledgements

Special thanks to all colleagues of HBM Soletanche Bachy Hungary and its partners who has participated in the back analyzed projects and made it possible to use as-built data for research objectives.

References

[1] Zdravkovic, L. "Modelling deep excavations in 3D analysis", In: Mahler, A., Nagy, L. (eds.) Proceedings of Workshop on Deep Excavations by ISSMGE Hungarian National Committee, Budapest, Hungary, 2009, pp. 139–165. ISBN: 978-963-06-6665-7

[2] Moormann, C., Klein, L. "Design of deep excavations under consideration of spatial earth pressure", In: Proceedings of the XVI. European Conference on Soil Mechanics and Geotechnical Engineering: Geotechnical Engineering for Infrastructure and Development, Edinburgh, Scotland, 2015. pp. 613–618. ISBN: 978-0-7277-6067-8

[3] Hong, Y., Ng, C. W. W., Liu, G. B., Liu, T. "Three-dimensional deformation behaviour of a multi-propped excavation at a "greenfield" site at Shanghai soft clay", Tunnelling and Underground Space Technology, 45, pp. 249–259, 2015.
https://doi.org/10.1016/j.tust.2014.09.012

[4] Dong, Y. P., Burd, H. J., Houlsby, G. T. "Finite-element analysis of a deep excavation case history", Géotechnique, 66, pp. 1–15, 2016.
https://doi.org/10.1680/jgeot.14.P.234

[5] Szepesházi, A., Móczár, B. "Deep excavation works in Budapest - Evaluation of deformation monitoring results", Concrete Structures, 24(1), pp. 22–32, 2022. (in Hungarian)
https://doi.org/10.32969/VB.2022.1.4

[6] Szepesházi, A., Móczár, B. "Numerical back-analysis of a monitored deep excavation in Budapest considering time dependency of wall deformations", In: Proceedings of the XVII ECSMGE-2019, Reykjavik, Iceland, 2019, Paper 136. ISBN 978-9935-9436-1-3
https://doi.org/10.32075/17ECSMGE-2019-0136

[7] Faragó, T., Szendefy, J., Szepesházi, A., Karner, B. "Back analysis of shoring system at the excavation site of M4 subway, Budapest", In: Proceedings of the 16th Danube-European Conference on Geotechnical Engineering, Skopje, Macedonia, 2018, pp. 627–632.
https://doi.org/10.1002/cepa.740

[8] Zsiros, N., Móczár, B., Szepesházi, A. "Back analysis of Budapest Rákóczi square metro4 station deep excavation", In: Proceedings of the 16th Danube-European Conference on Geotechnical Engineering, Skopje, Macedonia, 2018, pp. 839–844.
https://doi.org/10.1002/cepa.775

[9] Szepesházi, A. "3D finite element modelling of deep excavations", In: Proceedings of the XXIII. Széchy Károly Memorial Conference, Budapest, Hungary, 2017, pp. 54–79. (in Hungarian)

[10] Szepesházi, A., Móczár, B., Csapody, G. "2D and 3D numerical back analysis of a deep excavation in Budapest", Concrete Structures, 2, pp. 34–41, 2016. (in Hungarian)

[11] Szepesházi, A. "Back-analysis of a deep excavation in Budapest", In: Proceedings of the 25th European Young Engineers Conference, Bucharest, Romania, 2016, pp. 177–186.

[12] Szepesházi, A. "Comparison of 2D and 3D finite element modelling results of deep excavations", In: Proceedings of 5th Kézdi Memorial Conference, Budapest, Hungary, 2019, Paper 1. (in Hungarian)

[13] Ou, C.-Y., Chiou, D.-C., Wu, T.-S. "Three-dimensional finite element analysis of deep excavations", Journal of Geotechnical Engineering, 122(5), pp. 337–345, 1996.
https://doi.org/10.1061/(ASCE)0733-9410(1996)122:5(337)

[14] Benson Hsiung, B.-C., Yang, K.-H., Aila, W., Hung, C. "Three-dimensional effects of a deep excavation on wall deflections in loose to medium dense send", Computers and Geotechnics, 80, pp. 138–151, 2016.
https://doi.org/10.1016/j.compgeo.2016.07.001

[15] Görög, P. "Engineering geological assesment of eocene and oligocene clayey rocks of Buda", PhD thesis, Budapest University of Technology and Economics, 2008.

[16] Kálmán-Horváth, E. "Determination of the coefficient of earth pressure at rest in overconsolidated Kiscelli clay", PhD thesis, Budapest University of Technology and Economics, 2012.

[17] Kramer, S. L. "Geotechnical Earthquake Engineering", Prentice Hall, 1996. ISBN: 0133749436

[18] Benz, T. "Small-Strain Stiffness of Soils and its Numerical Consequences", PhD thesis, University of Stuttgart, 2007.

[19] Bentley Communities "PLAXIS 3D Reference Manual", [online] Available at: https://communities.bentley.com/products/geotech-analysis/w/wiki/46137/manuals---plaxis (Accessed: 4 March 2021)