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VALIDATION OF CORRELATIONS BETWEEN IN SITU INVESTIGATIONS AND SOIL PARAMETERS USING FEM MODELING

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Abstract: This paper aims to validate the geotechnical parameters obtained using new proposed correlations [1] with in situ test results by using the finite element method (FEM). To achieve this objective was performed a numerical modelling of a deep excavation built in the Northern area of Bucharest, by using several constitutive models such as Mohr-Coulomb, Hardening Soil and HS-Small. The geotechnical parameters used as input data for the numerical model were determined based on in situ soil investigations and using new proposed correlations described in [1]. The numerical model results in terms of horizontal displacements of the diaphragm walls supporting the deep excavation were compared to the measured displacements obtained from 2 inclinometers. Based on these results useful conclusions regarding the validity of the proposed correlations are drawn. This paper is part of wider doctoral research regarding correlations between in situ tests and geotechnical parameters.

Keywords: numerical modelling, CPT, FEM, in situ, geotechnical investigations

1. Introduction

This paper is part of broader research regarding correlations between in situ tests and geotechnical parameters for soils in Bucharest area. New specific correlations have been proposed for Bucharest soils, which are presented in [1], for usual in situ investigations as Cone Penetration Tests (CPT) and Dilatometer Marchetti Tests (DMT). These correlations can be used to determine geotechnical parameters for Bucharest's specific soils such as Bucharest Loam and Colentina Gravels directly from the in-situ tests. The correlations determined in [1] are presented in the following table.

Table 1

New correlations between in situ investigations and geotechnical parameters [1]

Parameter	Layer	Correlation	Correlation factor r^2
q_c – $E_{oed200\text{-}300}$	Bucharest Loam	$E_{oed200\text{-}300} = 3{,}1q_c + 5$ MPa	0.81
q_c - c	Bucharest Loam	$c_u = 0{,}021q_c + 0.001$ kPa	0.87
q_c - tanφ	Bucharest Loam	$\tan\varphi = 0{,}1q_c + 0{,}27$	0.71
q_c - M_{DMT}	Colentina Gravels	$M_{DMT} = 5.1$ qc + 34 MPa	0.83
q_c - tanφ	Colentina Gravels	$\tan\varphi = 0.003\ q_c + 0.72$	0.69

q_c- Cone resistance

$E_{oed200\text{-}300}$ – Oedometer modulus between 200 kPa and 300 kPa vertical load

c – Cohesion

tanφ – tangent of the friction angle

M_{DMT} – Dilatometer Marchetti constrained modulus

A first validation of the proposed correlations has been done based on existing correlations in the literature, which proved that they are estimating well the geotechnical parameters of those specific soils [1].

Further, the validation of the proposed correlations was done based on a back calculation involving numerical modelling and comparison with monitoring measurements. The aim of this paper is to present the results of the numerical modelling carried out using the FEM Plaxis 2D software for a deep excavation using different constitutive models used in current practice. Back calculation, the method that was used, is a common practice for calibration and verification of different types of results, as described in [2]. The geotechnical parameters used as input data for the numerical model were determined using the proposed correlations presented in [1] and presented in Table 1 here above. Differences obtained between in situ measurements as part of the geotechnical monitoring and those obtained from numerical calculations will be analyzed and discussed.

2. Geotechnical Investigations and results

Geological background

The research of the soils that compose the ground in the Bucharest area has been documented since the 19th century. The works of Murgoci [3] and Pache Protopopescu [4] are to be mentioned and are considered as reference works for the beginning of the last century. These works summarize and highlight for the first time the geological limits of the soil layers in Bucharest, using names similar to the current ones. The reference work for the geological description of the soil in the Bucharest area was published by Emil Liteanu in 1952 [5]. It completes and deepens the works mentioned before, describing in detail each geological layer. The State Committee of Geology of the Geological Institute of Bucharest published in 1966 the geological map of the Municipality of Bucharest under the coordination of E. Liteanu and G. Murgeanu.

Anthropogenic deposits and, where available, topsoil layer are followed by a layer of silty clayey soils, sometimes collapsible, of several meters thick and spread throughout the municipality and between the banks of the Dâmbovița and Colentina rivers. The layer described by these shallow deposits is also known as Bucharest Loams and is of Holocene age, part of the Quaternary period. This name will be used throughout this paper.

The shallow deposits are followed by a layer usually composed of gravel and sands, rarely of fine clayey sands. According to [5], its thickness varies between a few meters and up to 18 m - 20 m towards the bottom of the layer. The common name of this layer is the Colentina gravels and sands layer. According to the 1966 geological map of Romania, this layer is composed of both the Colentina Gravels and Sands layer and gravels belonging to the upper terraces of the Colentina and Dâmbovița rivers. It is geologically classified in the Upper Pleistocene, part of the Quaternary period. The first aquifer layer is also included in this layer, and it is unconfined.

The lithology in the Bucharest area continues with a clayey complex, sometimes with sandy lenses, of about 10 m to 25 m thick, which belongs from geological point of view to the Upper Pleistocene. It is known as the Intermediate Clay Complex.

According to the geological investigations cited above, as well as those carried out for the purpose of this paper, a layer of fine and medium sands, and occasionally fine gravels, underlies the Intermediate Clay Complex. This is known as the Mostistea Sands, a name retained throughout the paper. As geological age it is classified as Middle and Upper Pleistocene.

The numerical model consists of a deep excavation of a High-rise building. The site is located in the northern part of Bucharest, in the area of the intersection between Barbu Văcărescu Boulevard, Calea Floreasca and Pipera Road. The address of the site is Bd. Barbu Văcărescu, nr. 201. Access to the site can be made from all three traffic arteries (Bd. Barbu Văcărescu, Calea Floreasca, Șos. Pipera), which borders the site on the NE, SV and NW sides respectively. On the SE side, the site borders the Promenada Mall, with a height of 3S+P+2E, and on the south-west side with a private property with an exit in Calea Floreasca. It should be noted that two a metro lines are crossing under the central area of the planed excavation.

Geotechnical Investigations

Three geotechnical boreholes were drilled to depths of 20 m, 42.5 m and 60 m respectively to determine the soil parameters. From the geotechnical boreholes, disturbed and undisturbed samples were collected on which geotechnical laboratory tests were performed. In addition, 6 in-situ static cone penetration (CPT) tests were carried out at depths ranging from 24.8 m to 43.6 m and one Seismic Dilatometer Marchetti Test (SDMT) to a depth of 41.2 m. Groundwater was intercepted at 6 m below the upper elevation of the developed ground level.

Six representative soil layers were determined within the site boundaries. The synthetic lithology is shown below.

- 0.00 m - 1.50 m: Infill; consisting of building materials debris (brick, concrete) in clay matrix;
- 1.50 m - 4.50 m: Bucharest Loam; composed of stiff to hard, brownish silty clays, high to medium compressibility and medium to high plasticity;
- 4.50 m - 10.00 m: Colentina gravels; composed of coarse to medium coarse sands and gravels. In this layer the groundwater level was intercepted at -5.60 m to -6.50 m;
- 10.00 m - 32.00 m: Intermediate clay complex; composed of clays, silty clays and sandy silts, stiff, yellowish to brownish - black with high to medium compressibility and medium to very high plasticity;
- 32,00 m - 38,00 m: Mostistea sands; composed of fine to medium grained sands, brownish and greyish clayey sands and sands;

The determination of the geotechnical parameters required for the numerical model was carried out using the correlations determined in [1] and presented in Table 1. The values of the geotechnical parameters are presented in Table 2. Since it is not the subject of this paper and in order to eliminate possible errors, the geotechnical parameters of the backfill layer, Intermediate clay complex and Mostiștea sands were determined directly using geotechnical laboratory results.

Table 2

Soil parameters

Layer	Layer name	γ [kN/m^3]	φ’ [°]	c’ [kPa]	E_{50} [MPa]	E_{ur} [Mpa]	q_c [Mpa]	M_{DMT} [Mpa]
0	Backfill	17	16	5	8	24	1,6	-
I	Bucharest Loam	18,6	26	47	12	48	2,2	22,6
II	Colentina Gravels	19,5	37	-	85	255	14,5	26,6
III	Intermediate Clay Complex	19,4	22	30	21	105	2,7	30,9
IV	Mostistea Sands	19	36	-	77	230	12,2	24,2

γ - Unit weight for natural water content

φ' – effective friction angle

c' – effective cohesion

E_{50} – deformation modulus

E_{ur} – reloading soil modulus

q_c - Cone resistance

M_{DMT} - Dilatometer Marchetti constrained modulus

3. Geotechnical Monitoring

Due to the inhomogeneous nature of the soil, even in the most fortunate cases where sufficient geotechnical investigation points are available, the soil cannot be fully known and characterized. For this reason, monitoring equipment must be installed to monitor the behavior of the soil, so that the validity of the design assumptions can be checked during and after the construction. Also, when using the observational design method, as allowed by the Eurocode 7 [6], it is mandatory to install a monitoring system. The validation of the geotechnical model is important in order to detect possible failures caused by an improper investigation or design [7].

To verify our numerical model, monitoring equipment was installed as shown in Figure 1. In this case relevant are the results of the inclinometers 1 and 2. An inclinometer is used to monitor the horizontal deformations of a structural element. Measurements are made by lowering the equipment to the base of the installed inclinometer columns. The inclinometer columns are installed in the structural element before casting the element. Inclinometer measurements involve measuring displacements at successive intervals which may vary between 0.25 - 1 m. At each position the depth and inclination with respect to the vertical are recorded. According to the data sheets of the usual manufacturers of inclinometer equipment, measurements are made with an accuracy of approx. ±0.5 - 1 mm / 10 m depth. Typical components of the inclination measuring system are the inclinometer column, inclinometer probe, inclinometer cable and datalogger.

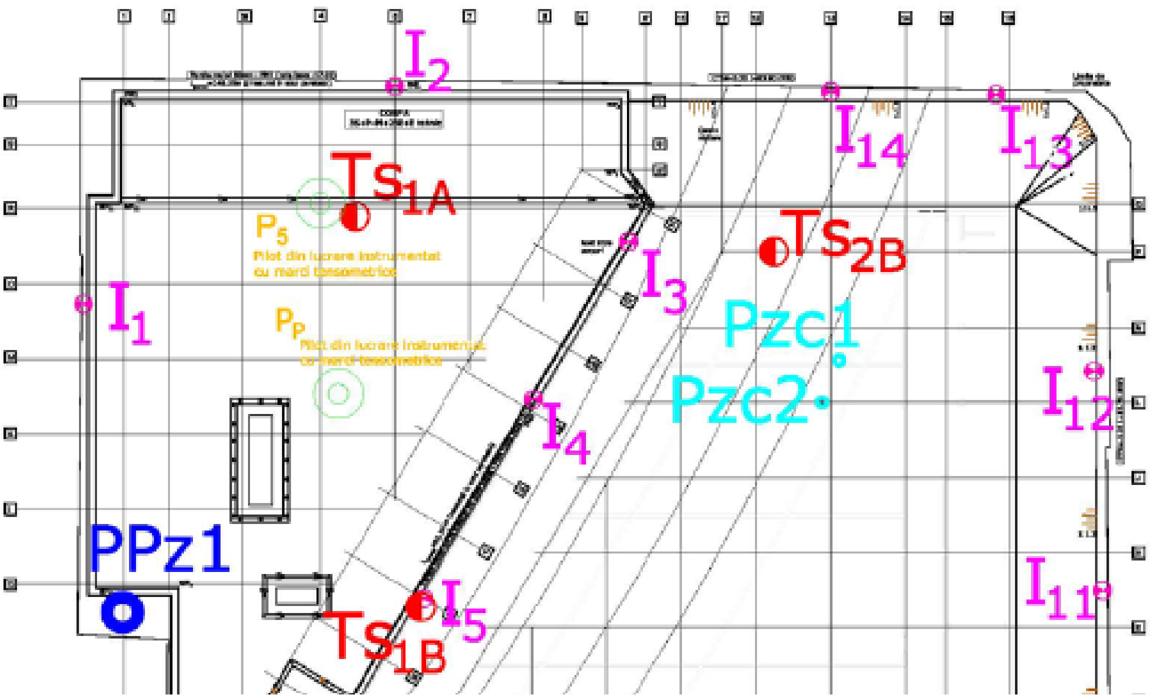

Fig. 1 – Geotechnical investigation points

For the assessment of the parameters used in the modelling of the structure, data obtained from geotechnical monitoring were used. The figure below shows the results of the monitoring for inclinometers I1 - I2.

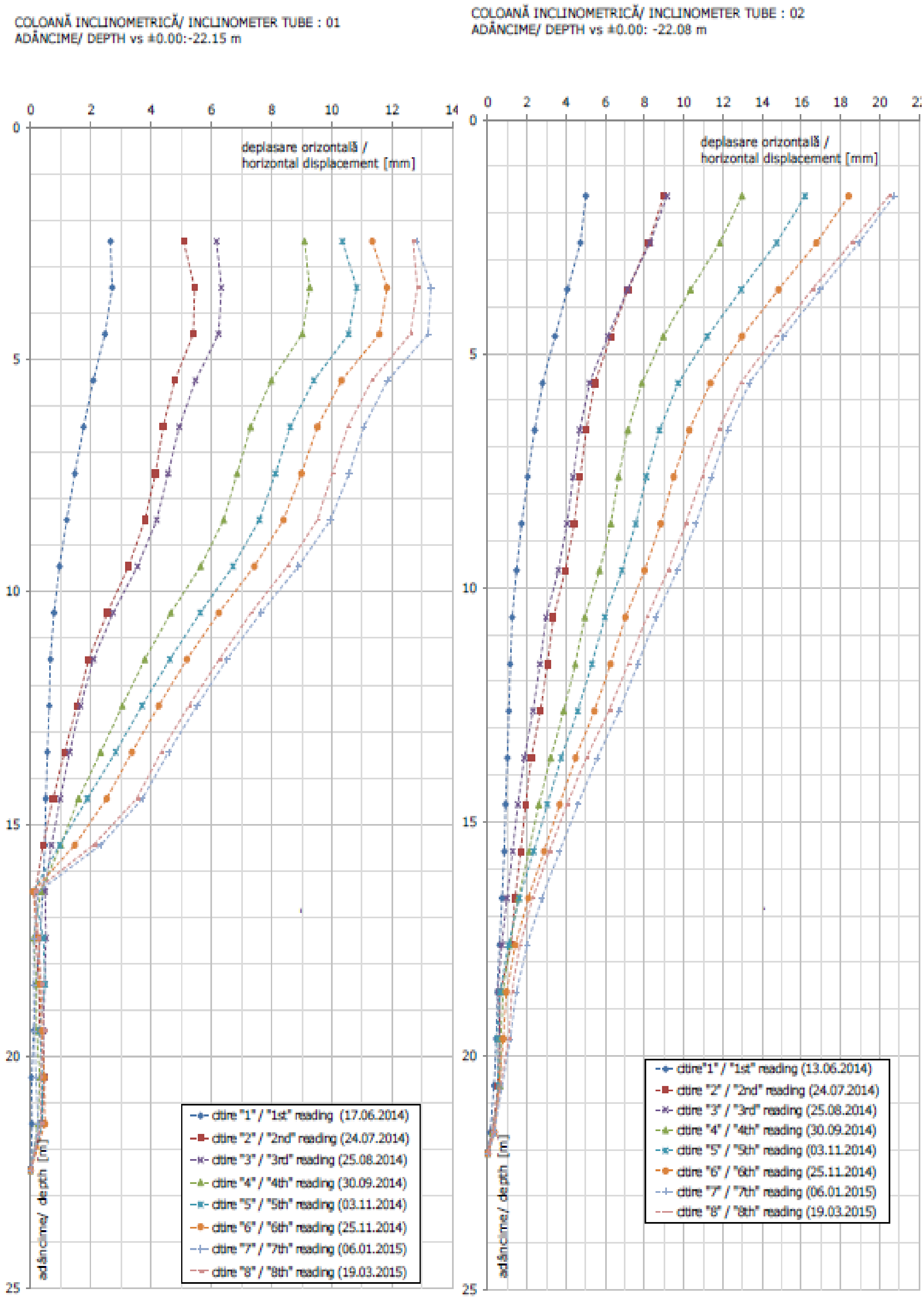

Fig. 2 - Horizontal displacement measured in I1 (max. 13.6 mm) - left and Horizontal displacement measured in I2 (max. 21.0 mm) - right

4. Numerical Analysis using Finite Element Method

In his paper Brinkgreve [8] details five aspects that are necessary to describe the behavior of a soil type. These are briefly presented below. The first aspect to be considered is the influence of water on the behavior of a soil, in terms of effective stresses and pore water pressures. The second aspect is related to the influence on the stiffness of a soil, the level of loading, the stress path (loading and unloading), the level of deformation, the density, the soil permeability, the level of consolidation and anisotropy. Irreversible deformations caused by loading must be taken into account, which is the third aspect. The fourth aspect focuses on factors that influence the soil strength, such as the loading rate, age and type of drainage. Other factors influencing the behavior of a soil are the level of compaction, the swelling and the pre-consolidation stress.

The modelling was carried out using the plane strain hypothesis, so that a relevant section was analyzed. According to Prakoso & Kulhawy [9] by modeling using the plan strain state can lead to an overestimation of the strains but provides reasonable information considering the low computation time. The problem of strains and stresses in deep excavation works is a three-dimensional problem. In order to be able to model it in two dimensions, simplifications were necessary. The analysis was carried out after the displacements arising from other works in the area were considered to have been consumed.

4.1. Construction stages

The deep excavation was modeled using several construction stages. The main modelled stages are the following. Figure 3 shows the initial site situation where the infrastructure of the neighboring building and its diaphragm wall can be seen. Then follows the stage when the foundation piles and the diaphragm wall are constructed.

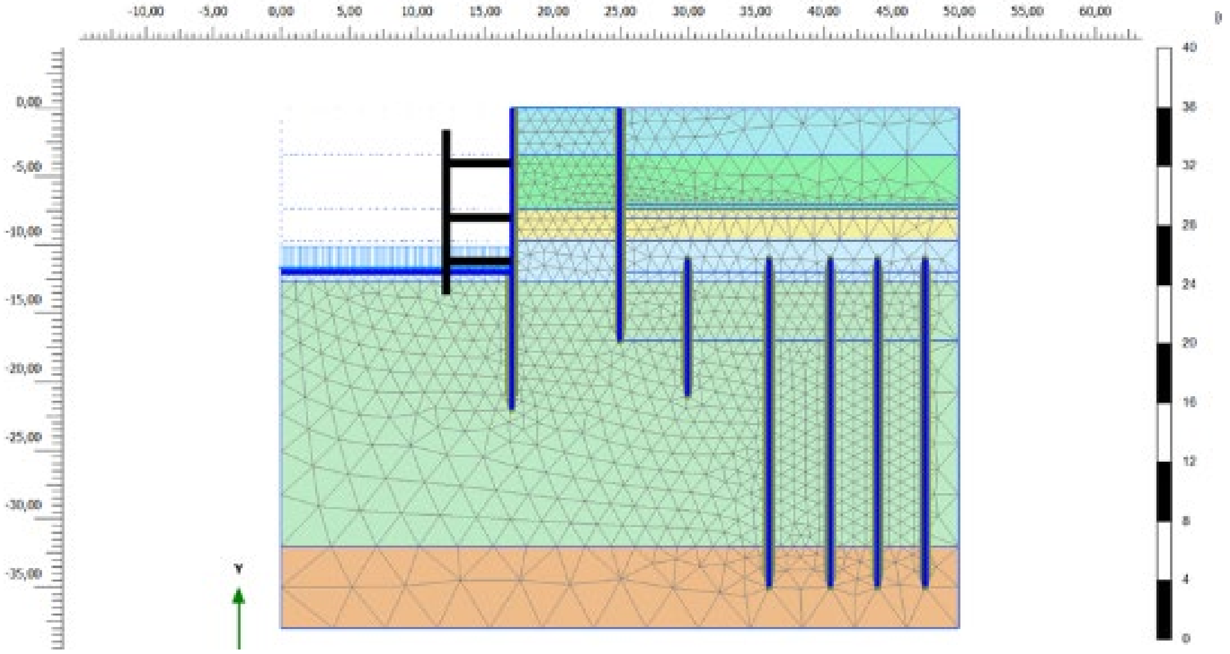

Fig. 3 – Stage 1 – Construction of diaphragm walls and foundation piles

In the next stage, the excavation is carried out down to -7.05 m as the groundwater level is lowered to 50 cm below the excavation level. This execution stage is shown in Figure 4.

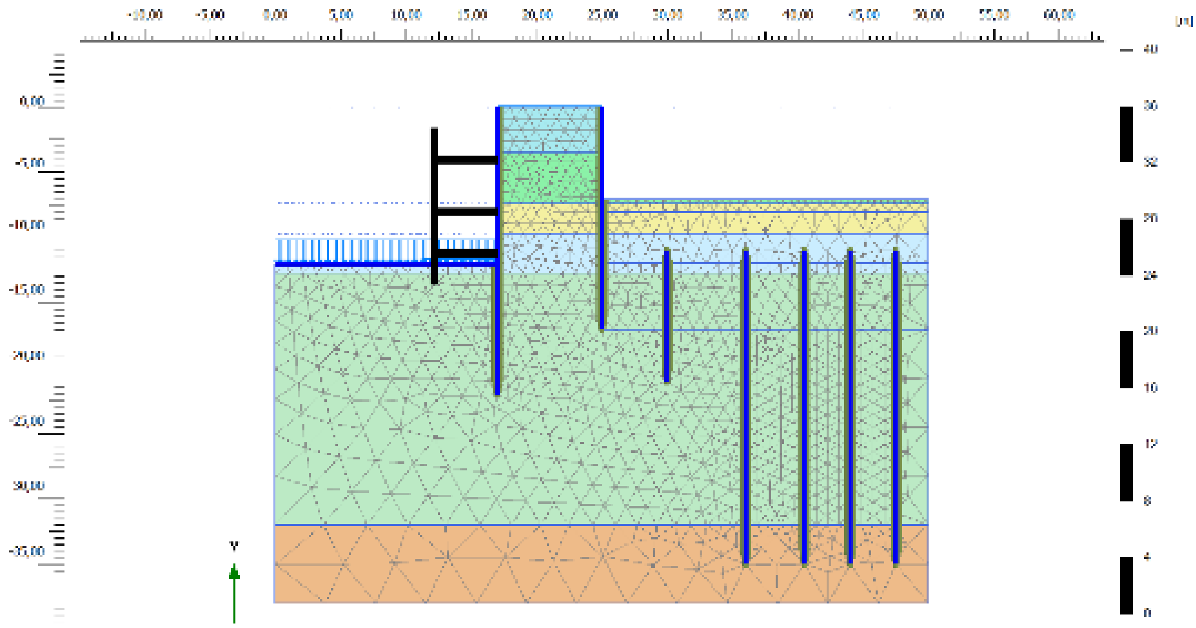

Fig. 4 – Stage 2 – Excavation to -7.05 m with groundwater level lowering to -7.55 m and casting the slab over the 3rd underground level

The last excavation stage down to an elevation of -12.00 m is shown in Figure 5. Before excavation the groundwater level is lowered 50 cm below the final excavation level.

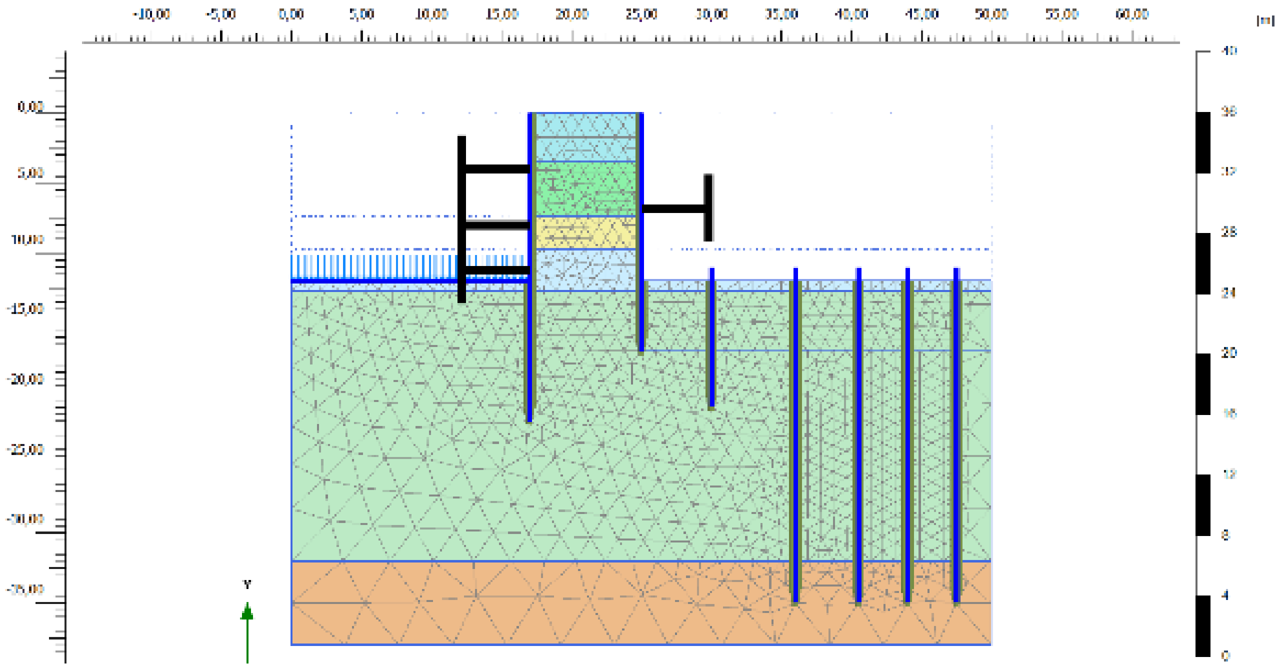

Fig. 5 – Stage 3 – Excavation to -12.00 m with groundwater level lowering to -12.50 m

The last stage of the modelling consisted in pouring the raft and the slab from to -9.55 m. This stage is shown in Figure 6.

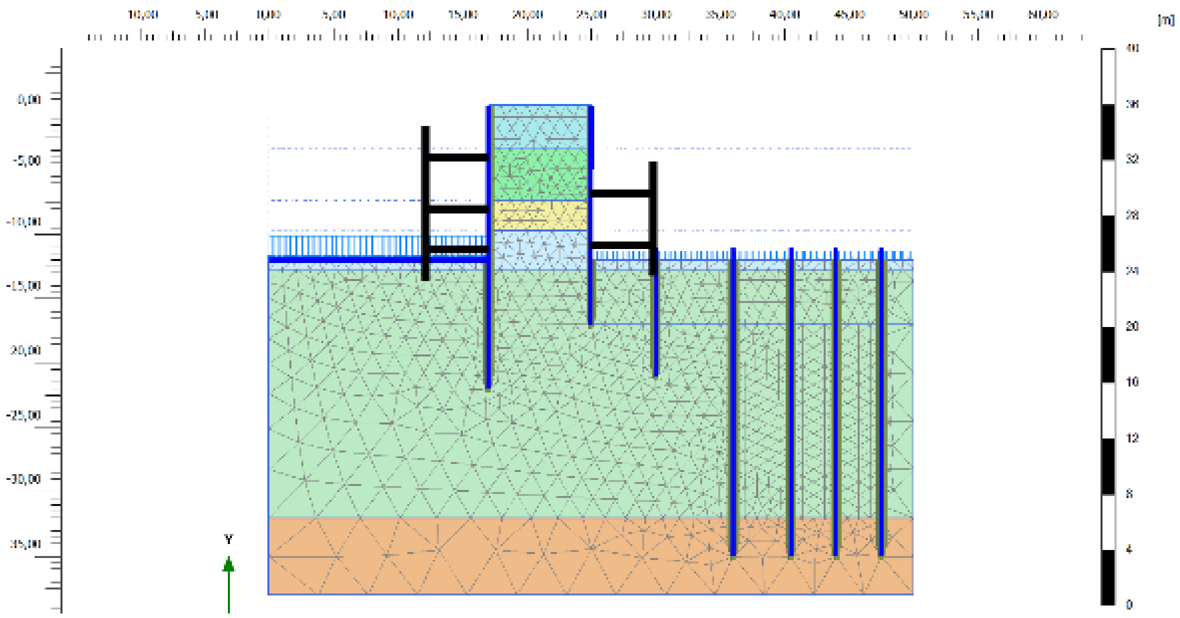

Fig. 6 – Stage 4 – Casting the raft and the slab at -9.55 m

4.2. Results of FEM Modelling

The numerical modelling was performed using the FEM software PLAXIS 2D, using 3 different constitutive laws: Mohr-Coulomb (MC), Hardening Soil (HS), HS-Small.

4.2.1. Results with Mohr-Coulomb constitutive law

Figure 7 shows the displacements of the model in the stage after casting the foundation raft.

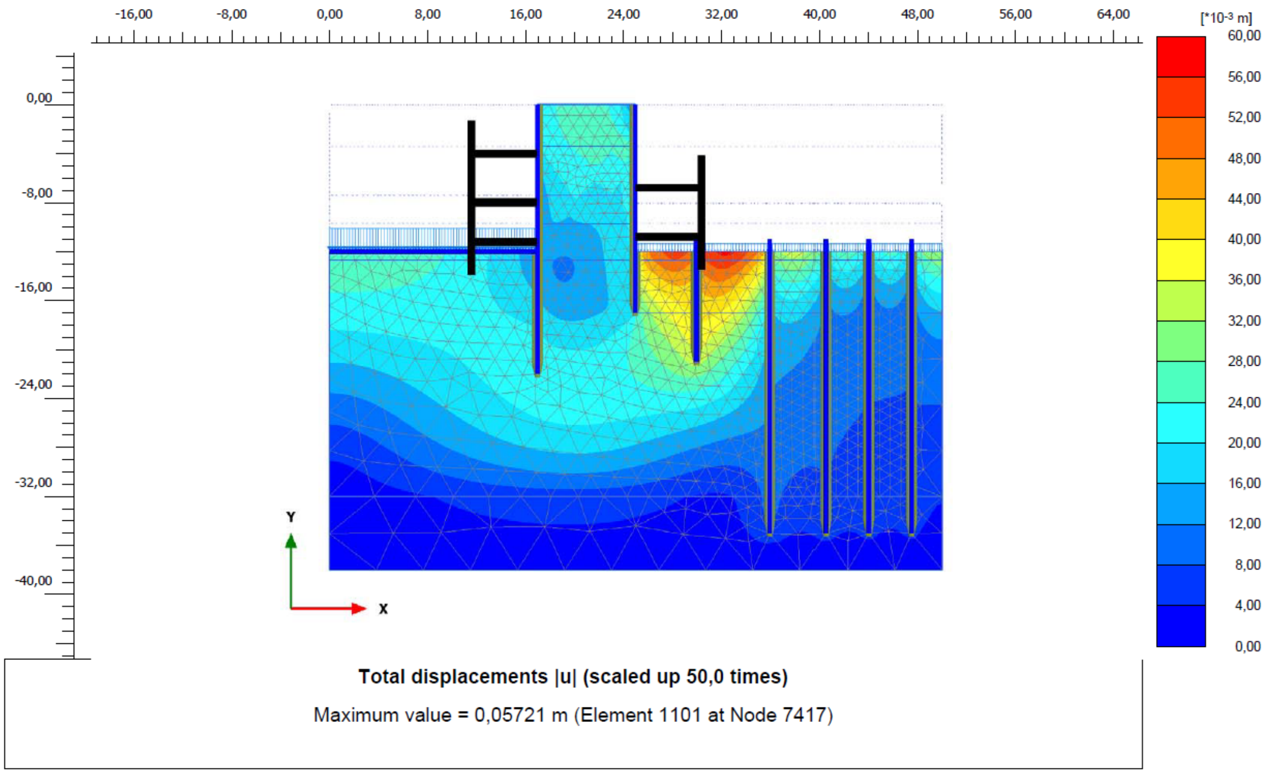

Fig. 7 – Total displacement during Stage 4 (Mohr-Coulomb)

After the execution of the foundation raft the displacements in the x-direction range from 3.5 cm on the top of the diaphragm wall to 1.8 cm at the base of the diaphragm wall. The displacements are shown in Figure 8.

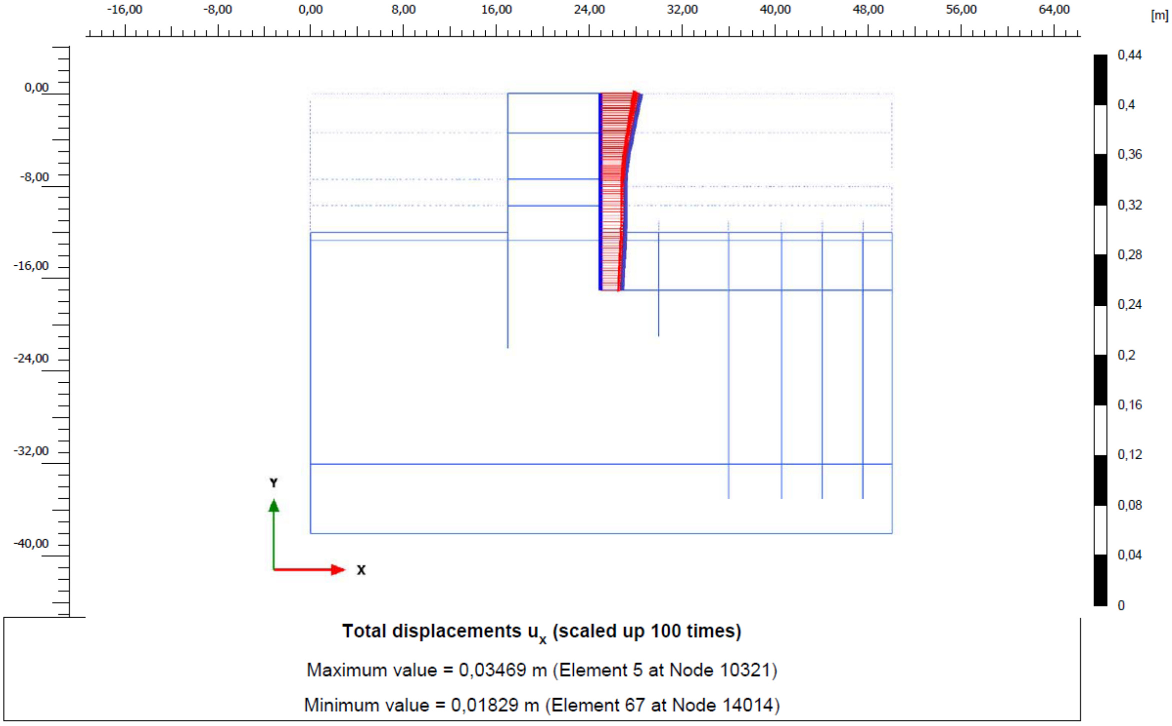

Fig. 8 – Displacement on the x axis during Stage 4 (Mohr-Coulomb)

4.2.2. Results when using Hardening Soil Model

Figure 9 shows the displacements of the model in the stage after the raft. This shows a total deformation of approx. 20 mm a the top of the diaphragm walls and a slight heaving of the excavation.

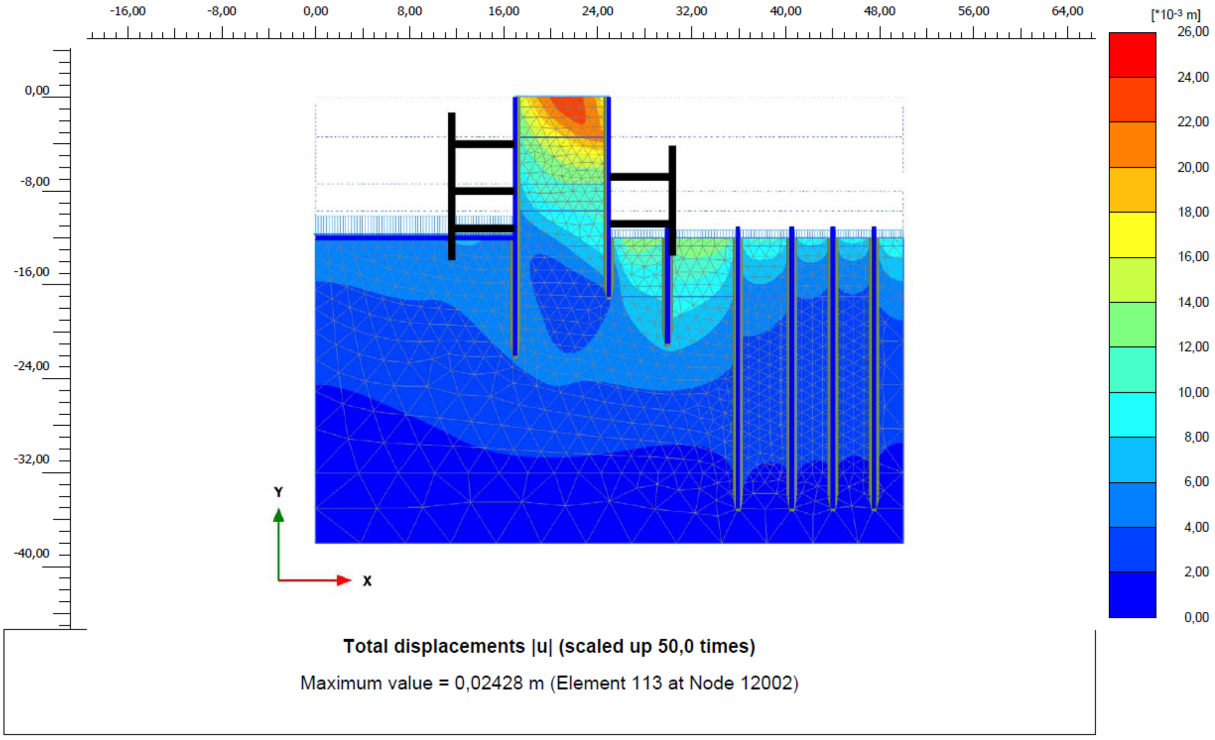

Fig. 9 – Total displacement during Stage 4 (Hardening Soil)

After casting the foundation slab the displacements in the x-direction range from 2.7 cm on top of the diaphragm wall to 0.26 cm at the base of the excavation. This significantly varies from the results of the model using Mohr-Coulomb.

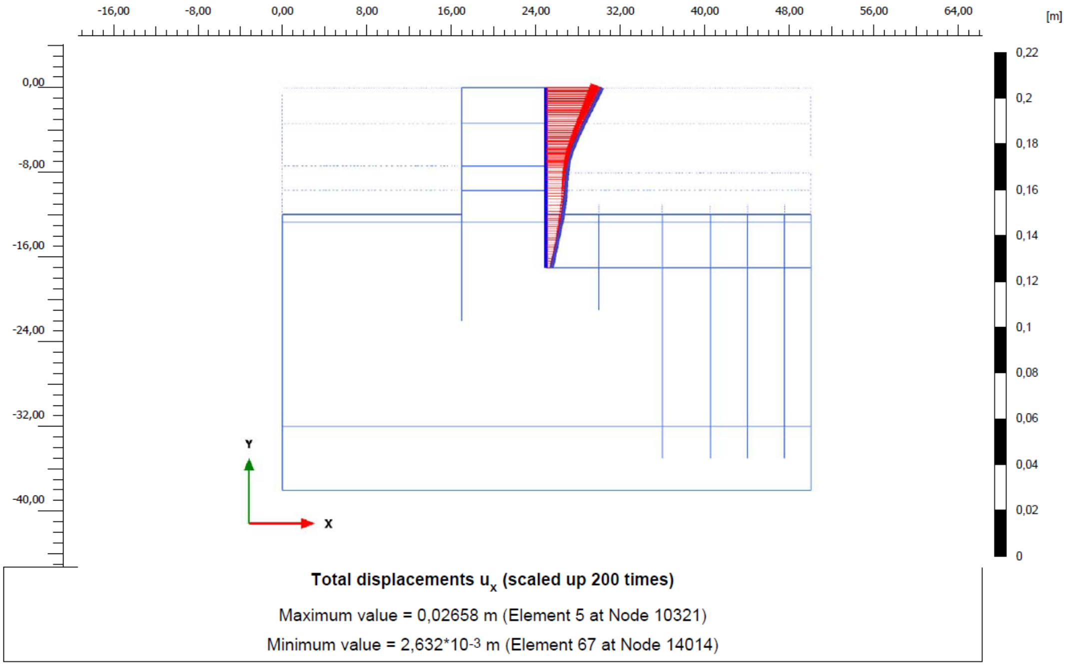

Fig. 10 – Displacement on the x axis during Stage 4 (Hardening Soil)

4.2.3. Results when using HS-Small Model

Figure 11 presents the displacements of the model in the stage after the foundation raft is constructed. As it can be observed a total displacement of 14 – 15 mm is displayed at the top of the diaphragm wall and a slight heaving of the foundation ground of about 5-6 mm.

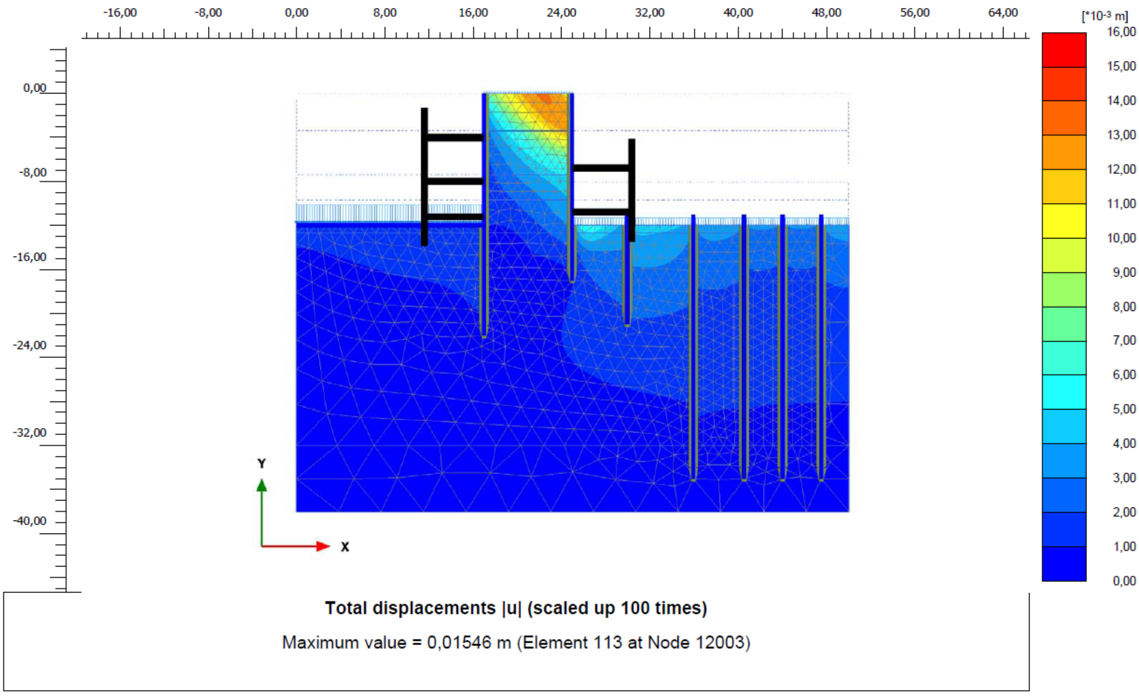

Fig. 11 – Total displacement during Stage 4 (HS-Small)

The displacement on the x axis is displayed in figure 12. As can be observed they vary from ca. 1,8 cm on top and around 0,5 cm on the bottom of the excavations. This significantly varies from the results of the model using Mohr-Coulomb but are similar to those using the Hardening Soil.

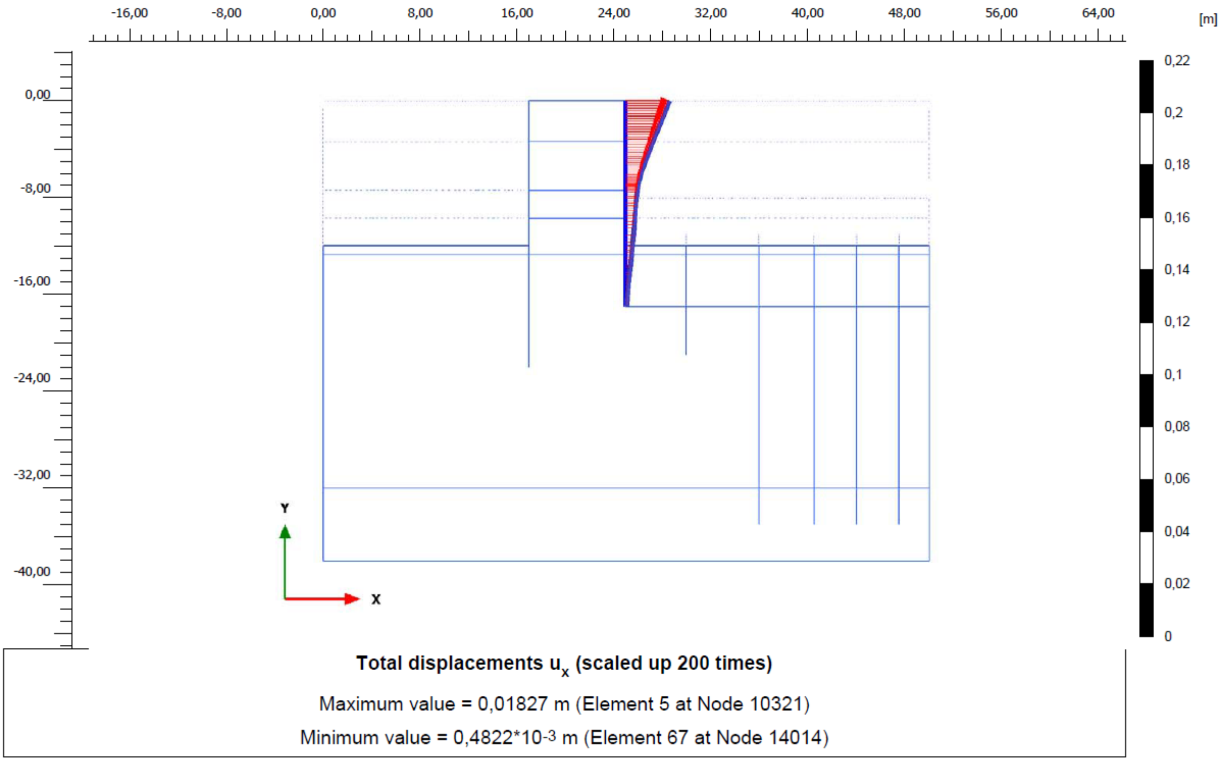

Fig. 12 – Displacement on the x axis during Stage 4 (HS-Small)

4.3 Discussion

In this paragraph comments and observations are provided on the base of the numerical modelling results in comparison with the geotechnical monitoring results. All numerical modelling results show higher maximum displacement values in the x-direction than the geotechnical monitoring results. This may indicate, among other things, a conservative estimation of geotechnical parameters. In the numerical modelling presented above the primary correlations proposed in [1] were used to determine the geotechnical parameters and not their safe values. Safe values are the values determined by the translation of the regression line with the values of the standard deviation. Using the safe value of the correlations [1] is supposed to lead to an even more conservative estimate of the displacements. The shape of the displacement diagram of inclinometers I1 and I2 shown in Fig. 3 is reflected in the numerical modelling results using the Hardening Soil and HS-Small constitutive models. When using the Mohr-Coulomb model, the horizontal displacement diagram shows a significant displacement of the diaphragm wall along its entire length, caused - according to the modelling - by a significant displacement of the excavation base in the x-direction. The displacement of the excavation base is also captured when using Hardening Soil and HS-Small models, but it has a much lower value, and it is close to the monitoring results.

In absolute terms of horizontal displacement, it can be seen that the modelling the excavation using advanced constitutive models, such as HS-Small and Hardening Soil, provides results that almost match the results from the geotechnical monitoring, which is to be expected.

The monitoring results indicate an increase of the horizontal displacement ranging from 0.6 cm to 0.8 cm in Stage 1 and from 1.2 cm to 1.8 cm in Stage 3. Following the displacements resulting from the numerical modelling, it can be seen that the maximum displacement was obtained in Stage 1 of the excavation, before the first slab was poured as a supporting element of the excavation. This phenomenon can be explained by the fact that the software simulates the final phase of an excavation stage with displacements and consolidation processes being consumed. In reality, as the execution is carried out quite quickly, the ground has not yet consumed all its displacements. In the modelling of Stage 2 and Stage 3 of the excavation, a slight return of the horizontal displacement at the top of the diaphragm wall is observed, with an increase of displacement at the bottom of the wall. The displacements at the bottom of the diaphragm wall show values between 0.5 and 2 mm for the Hardening Soil and HS-Small models and 1.8 cm for the Mohr-Coulomb model. The heaving of the excavation base from numerical modelling displays values between 0.5 mm and 1.2 cm for the HS-Small and Hardening Soil models and about 6 cm for the Mohr-Coulomb model. The heaving of the excavation base observed during the geotechnical monitoring is similar to the mathematical modelling using the HS-Small constitutive model. Vertical displacements near the diaphragm wall were not monitored. Instead, the displacement of the excavation base, measured in the center of the excavation, indicates a heaving of about 2 - 3 cm, which is closer to the results of the HS-Small model and not to the Mohr-Coulomb modeling.

5. Conclusions

This paper provides a comparison of the strains of a deep excavation supported by diaphragm walls resulting from finite element modelling with those obtained from monitoring with the aim to validate the geotechnical parameters used as input data. The geotechnical parameters for the layers Bucharest Loams and Colentina Gravels were determined using the new correlations proposed in [1]. Several constitutive models such as Mohr-Coulomb, Hardening Soil and HS-Small were used in the validation process. Based on the obtained results, useful conclusions can be drawn on the validity of the proposed correlations. All numerical modelling results show higher horizontal displacement values than those obtained from geotechnical monitoring. This may indicate among other things a conservative estimation of geotechnical parameters. But there is a good fit in the shape of the displacement diagram compared to I1 and I2 inclinometer measurements when using Hardening Soil and HS-Small constitutive models. For Mohr-Coulomb model the horizontal displacement diagram shows a significant displacement of the diaphragm wall along its entire length, due to the displacement of the excavation base. Excavation base displacement is also captured in the Hardening Soil and HS-Small models, but it is much lower. In contrast, the measured horizontal displacement of the excavation base has lower values than those modelled with the more advanced Hardening Soil and HS-Small models.

The geotechnical parameters used in the numerical model were determined using the new proposed correlations presented in [1], wishing to validate the correlations through a back-calculation method.

It should be noted that even a detailed numerical modelling cannot capture all the complex phenomena that occur within the soil mass. In addition, the external forces acting on the earth mass as well as on the structural elements within the modelling can only be estimated and their exact value varies more or less constantly. Also, some dynamic loads from traffic on the surrounding street network have been included in the calculation as estimated static cover loads.

In view of the above it can be concluded that mathematical modelling using FEM models can provide good and reliable results for geotechnical design. Also, the results of the modeling using parameters determined with the new correlations [1] reveal similar displacements to the one measured during the geotechnical monitoring of the excavations. Nevertheless, due to the fact that many variables and assumptions are needed to create a FEM model, a calibration of the determined new correlations is not the most suitable by using back calculations.

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