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Parametric study of deep excavation in clays

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Abstract. The most challenging issue when analyzing geotechnical structures by means of finite element method is the choice of appropriate constitutive soil model, especially with reference to serviceability limit states. The paper presents parametric study of a deep excavation in clays aiming to qualify the applicability of different soil constitutive models in such specific soil conditions. Three types of constitutive models are considered in the paper: linear elastic – perfectly plastic model (Mohr-Coulomb) as a simple and well recognized reference, hypoplastic model (Hypoplastic Clay) and nonlinear elasto-plastic cap models (Hardening Soil and Hardening Soil Small). Numerical analysis was performed using two finite element codes – Plaxis and GEO5 FEM both in 2D space and the results were compared to in-situ displacements measurements. The discussion on the suitability of chosen constitutive models for advanced modelling of deep excavation in preconsolidated clays is presented.

Key words: numerical analysis, constitutive models, hypoplasticity, hardening soil model.

1. Introduction

Nowadays, in the era of fast digitization development, spectacular progress in hardware and software development, the use of computer methods in geotechnical design and displacement prediction is gaining more and more popularity. Finite element method, is the most suitable for all geotechnical tasks giving information about stresses and strains in the soil body and modelling soil-structure interaction. On the other hand, it imposes higher demands on the engineer, requiring the profound knowledge of numerical methods, geotechnics and soil mechanics [1, 2].

One of the most challenging issues when analyzing geotechnical structures by means of finite element method is the choice of adequate constitutive soil model, especially in consideration of the serviceability limit states. The main issue is the choice of advanced, robust constitutive model describing complex soil behavior on the other hand being a convenient engineering tool, relatively simple to be defined by an engineer [3–7].

2. Choice of the constitutive model

In previous works, author made attempts to verify the suitability of the most common, conventional soil constitutive models for finite element modelling of displacements induced by deep excavations (horizontal wall displacements, uplift of the bottom of the excavation, as well as or rather most of all, the displacements of soil body and structures in the influence zone) in varied soil conditions [8–11].

Based on these works it was concluded that in the specific soil conditions of the area of Warsaw the excavations embedded in mixed Quaternary soils may be successfully analyzed using simple Mohr-Coulomb or modified Mohr-Coulomb elastic-perfectly plastic constitutive models [12] assuming proper calibration of its parameters [10, 13]. However, it should be noted that this models have several limitations, most of all, they tend to result in excessive settlements of the surrounding soil, which is not observed in practice. Common observations prove that the soil body around the excavation rises during excavation (unloading), while in numerical analysis settlements are usually obtained [8–11, 14–16].

On the contrary, when deep excavations are embedded in preconsolidated Pliocene clays the use of simple models is inappropriate [17]. In the paper the use of two advanced types of models for modelling excavations in preconsolidated clays is proposed. First is the Hypoplastic Clay (HC) model [18–20] that captures the actual nonlinear behavior of soils even at low loads that do not exceed their strength as well as during unloading. Unfortunately, extensive use of this model is limited due to the need of time consuming and demanding laboratory tests to determine its parameters [21]. The good applicability of Hypoplastic Clay model for Pliocene clays was proved by the author on several cases [9, 11], as well as by other authors [2, 22]. The second considered type of model is Hardening Soil model (HS) [23, 24], which introduces different function for yield and for failure enabling distinction between initial loading and unloading/reloading, as well as its modification – Hardening Soil Small model (HSS) [25], taking into account non-linear dependency of strain at small strain ranges.

Concepts of applying the Hardening Soil or Hardening Soil Small model in clayey soils are noted [26–33], but not necessarily in preconsolidated clays.

The paper presents a parametric study of a deep excavation in Pliocene clays concerning the selection of soil constitutive

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model. Four soil constitutive models were applied in the analysis: Mohr-Coulomb, Hypoplastic Clay, Hardening Soil and Hardening Soil Small and the results were compared to the results of in-situ displacements measurements. In conclusions section the discussion on the suitability of chosen constitutive models for advanced modelling of deep excavations in preconsolidated clays is presented.

3. Case example

The case example used for the analysis is a 20 m wide and 150 m long excavation of the metro station located in the downtown of Warsaw. The 14.6 m deep excavation was mostly executed in Pliocene clays. The excavation walls protected by diaphragm walls were supported by two levels of ground anchors and one level of steel struts. The typical cross section including construction stages and geotechnical conditions is shown in Fig. 1.

3.1. Stages of construction. The following construction stages were considered in FE analysis, Fig. 1:
Stage 1: Greenfield
Stage 2: Construction of diaphragm wall
Stage 3: Excavation till –4.55 m bgs (below ground surface)
Stage 4: Installation of anchors at –3.73 m bgs, spacing 2.4 m
Stage 5: Excavation till –8.85 m bgs, stressing of anchors (F = 80% of FD, F = 400 kN)
Stage 6: Installation of anchors at –7.85 m bgs, spacing 1.3 m
Stage 7: Excavation till –11.85 m bgs, stressing of anchors (F = 80% of FD, F = 480 kN)
Stage 8: Installation of struts at –10.85 m bgs, spacing 2.0 m, ϕ508/14.2 mm
Stage 9: Final excavation till 14.6 m bgs.

3.2. Geotechnical conditions. In Warsaw, generally, the Quaternary Pleistocene and Holocene formations cover Tertiary clayey deposits. However, due to glacitectonic and erosion processes the clay layer is strongly deformed, uneven and irregular. Due to this irregularity, in the area of the excavation, the clay layer is elevated and lies directly under the small cover of anthropogenic soils. Geotechnical conditions considered for the analysis are presented in Fig. 1. Basic parameters of soil layers are compiled in Table 1.

Table 1
Basic geotechnical parameters of soil layers

	γ kN/m^3	φ’ °	c’ kPa	E MPa	Ko	υ
Fill	18	25	0	25	0.577	0.30
Clay 1	20.7	18	10	80	0.917	0.35
Clay 2	20.7	18	15	100	0.783	0.35

In the analyzed area, there is no general ground water table. The water table is discontinuous and carries low quantities of water. Water can be found only in sand lenses and pockets within the clay body. In effect, the ground water was not considered in the analysis.

More information about geotechnical conditions in Warsaw with particular emphasis on the Pliocene clay layer, its detailed description and parameters, can be found in [13, 34–39].

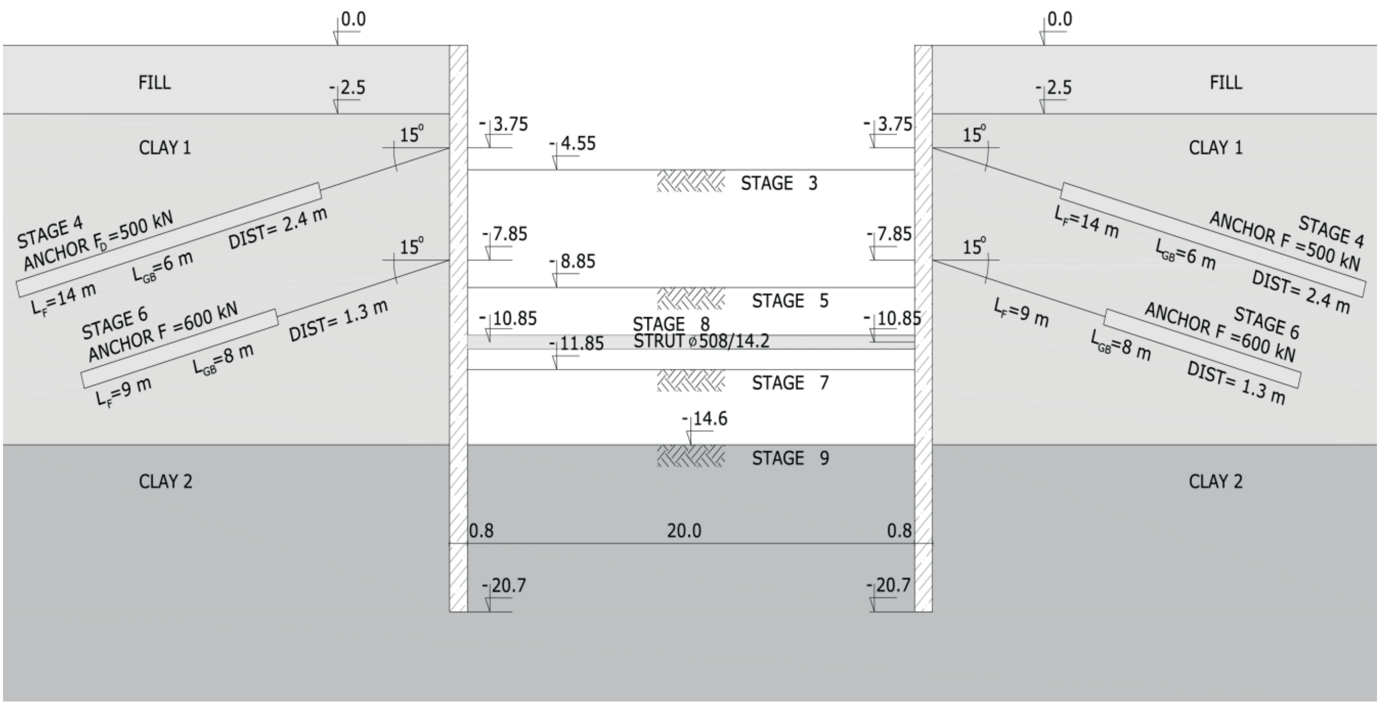

Fig. 1. Typical cross section

4. Numerical analysis

Numerical plane strain analysis was carried out by means of two geotechnical finite element programs – GEO5 FEM [40] and Plaxis [41] due to the limited availability of chosen constitutive models.

One typical cross-section was modelled considering geotechnical conditions, geometry and construction stages as described in chapter 3 and shown on Fig. 1. Both numerical models were calibrated and validated on the basic constitutive linear elastic – perfectly plastic soil model (Mohr-Coulomb). The parameters of soil layers considered in calibration analysis are given in Table 1. The results obtained in both programs in terms of vertical displacements of the ground surface behind the excavation wall, horizontal displacements of the wall and vertical displacements of the bottom of excavation in all construction stages were compared. When the differences obtained in two models (programs) were negligible models were set for further parametric analysis.

4.1. Model 1. The first model was made in GEO5 FEM program [40], because another parametric study was made on this case before [9]. Model dimensions are 40×100 m. Finite element mesh generated automatically, with local refinement around the excavation, consisted of 7048 nodes and 4189 elements (2641 15-nodes triangle surface elements, 387 beam elements, and 1161 contact elements).

The first model and the mesh are shown in Fig. 2. In this program, anchors and struts are elements added in construction stages, after mesh generation.

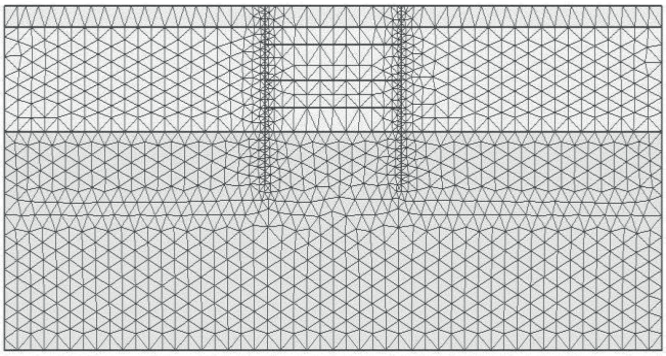

Fig. 2. Finite element mesh – Model 1 (GEO5 FEM)

After calibration of the model (as described before) the parametric analysis was made changing the soil constitutive model to Hypoplastic Clay model for both clay layers. Parameters of this model are compiled in Table 2. Basic geotechnical parameters, as c’, φ’, K_0, υ, if required, were taken as stated in Table 1.

All parameters except e_{max}, φ_{cv} and r for the Hypoplastic Clay model where evaluated basing on laboratory and field tests carried out by the employees of the Department of Geotechnics and Underground Structures of the Warsaw University of Technology [35–37]. Parameters e_{max}, φ_{cv} and r were taken as average for similar soil types [40].

Table 2
Parameters of Hypoplastic Clay model

	κ	λ	e_0	e_{max}	φ_{cv} °	r
Clay 1	0.019	0.071	0.57	2.5	27	0.3
Clay 2	0.019	0.071	0.57	2.5	27	0.3

Determination of all parameters of the Hypoplastic Clay model for Pliocene clays is a part of separate study, which is in the process.

4.2. Model 2. The second model was made in Plaxis program [41]. Model dimensions are identical, 40×100 m. Finite element mesh generated automatically, with local refinement in the area of the excavation, consisted of 1235 triangle 15-nodes elements and 10445 nodes. The second model and the mesh are shown in Fig. 3.

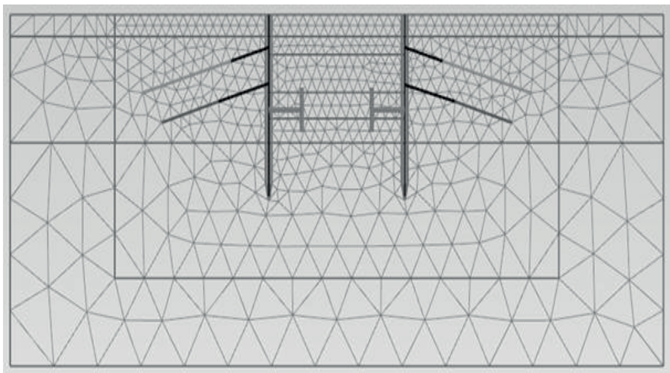

Fig. 3. Finite element mesh – Model 2 (Plaxis)

After calibration of the model the parametric analysis was made changing twice the soil constitutive model to Hardening Soil and then Hardening Soil Small (HSS) model. Parameters of these models are compiled in Table 3. Last two columns present two specific, additional parameters of HSS model. Basic geotechnical parameters, as γ, c’, φ’, K_0, υ, were taken as stated in Table 1.

Table 3
Parameters of Hardening Soil and Hardening Soil Small models

	Eur MPa	E_{50} MPa	υ_{ur}	m	OCR	$\gamma_{0.7}$	E_0 MPa
Fill	25	8.3	0.2	0.6	1	0.0001	190
Clay 1	80	26.6	0.2	0.6	3	0.0001	430
Clay 2	100	33.3	0.2	0.6	3	0.0001	430

4.3. Results of numerical analysis. As a basis to verification of the results of the parametric analysis, three leading displacement parameters were chosen: the horizontal displacement of the top of the diaphragm wall (u_{xw}), the heave of the bottom of excavation described by the value of vertical displacement of the bottom of excavation (y_{uplift}) and vertical displacements of the ground surface in the influence zone (y_{ground}). In the parametric study formulated as described above, the results of numerical analysis were compared to the results of real displacements measurements taken during construction.

First parameter of the comparative analysis – the horizontal displacement of the top of the diaphragm wall (u_{xw}) – is considered during the design of the wall of excavation in the serviceability limit state. Based on the results of all calculation series it may be observed that the displacements of the top of the excavation wall (u_{xw}) obtained applying different constitutive models in subsequent construction stages differed up to 60% comparing to the measured value (from 1.5 to 11.3 mm), but at the end resulted in similar maximum values in the last construction stage, comparable to those measured during construction – the difference obtained was up to 19% (up to 5.5 mm), except for the Hardening Soil Small model. For this model the analysis resulted in underestimation of displacements of the wall, by 26% (7.7 mm), which is rather unfavorable and unsafe in the construction design.

The graphs of the horizontal displacements of the top of the excavation wall (u_{xw}) in construction stages in all calculation series (Model 1 – Hypoplastic clay, Model 2 – Hardening Soil and Hardening Soil Small with reference to basic Mohr-Coulomb model) compared to geodesic in-situ measurements are presented in detail in Fig. 4.

Similar observations were found in relation to the second analyzed parameter – the uplift of the bottom of the excavation (y_{uplift}). The maximum values of the heave of the bottom of the excavation, occurring in final excavation stage, obtained applying different constitutive models were similar and comparable to maximum value of the excavation uplift measured

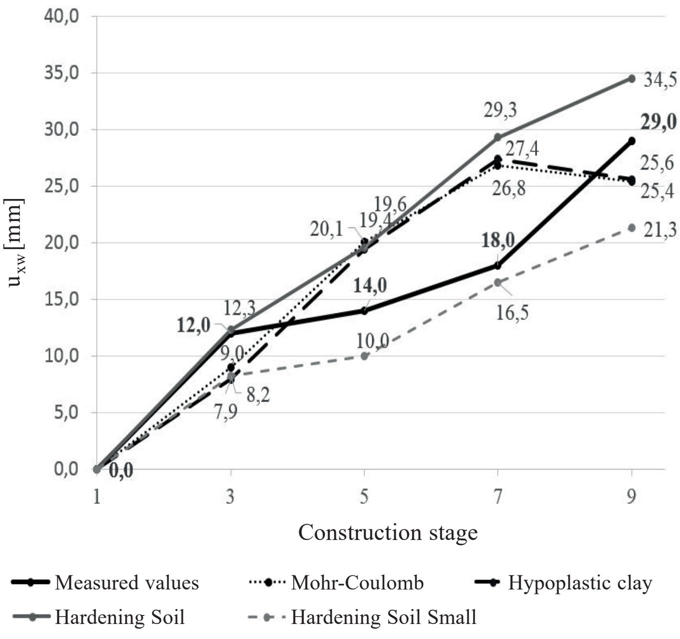

Fig. 4. Horizontal displacements of the top of the wall in construction stages obtained using different constitutive soil models

during construction. The differences were up to 25% (15 mm) comparing to the measured value. However, again, the use of Hardening Soil Small model resulted in underestimation of those displacements by 51% (30,6 mm).

The maximum values of the uplift of the excavation (y_{uplift}) in all calculation series compared to the value measured during construction are presented in Fig. 5.

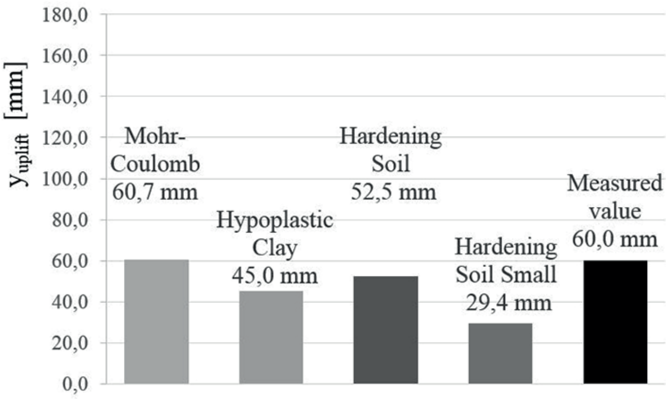

Fig. 5. Maximum value of the uplift of the bottom of the excavation obtained using different constitutive soil models

The third analyzed parameter concerned the displacements of the ground (y_{ground}) induced by the excavation in the influence zone. The parametric study showed significant differences in the nature of displacements occurring around the excavation in all stages of excavation between models 1 and 2 (HC versus HS and HSS). The range of the influence zone is similar, but the direction of displacement differs. Fig. 6 and Fig. 7 present the general layout of the displacements around the excavation in final excavation stage in model 1 (Hypoplastic Clay) and Model 2 (Hardening Soil) respectively, illustrating those differences. Fig. 8 shows deformed mesh in final excavation stage for the Hardening Soil Small model.

In the Model 1, directly behind the wall small settlements occur, up to 2.6 mm, which complies with the settlement observed during construction (2.5 mm). Then, moving away from the wall, the elevation of ground surface is obtained, up to 6.5 mm in the final excavation stage, which is only 13% less then measured on site.

In the Model 2 (for both constitutive soil models – HS and HSS) significant settlement directly behind the wall is obtained, up to 13.6 mm, which is 7.5 times more than the real settlement observed during construction (2.5 mm). The settlement is then decreasing towards the end of the influence zone. No heave or elevation of ground surface occurred in that case.

The results of all calculation series (Model 1 – Hypoplastic Clay, Model 2 – Hardening Soil and Hardening Soil Small with reference to basic Mohr-Coulomb model) are further presented in detail in form of graphs of vertical displacements of the ground surface (y_{ground}) behind the excavation compared to geodesic in-situ measurements in four construction stages (representing successive excavation):

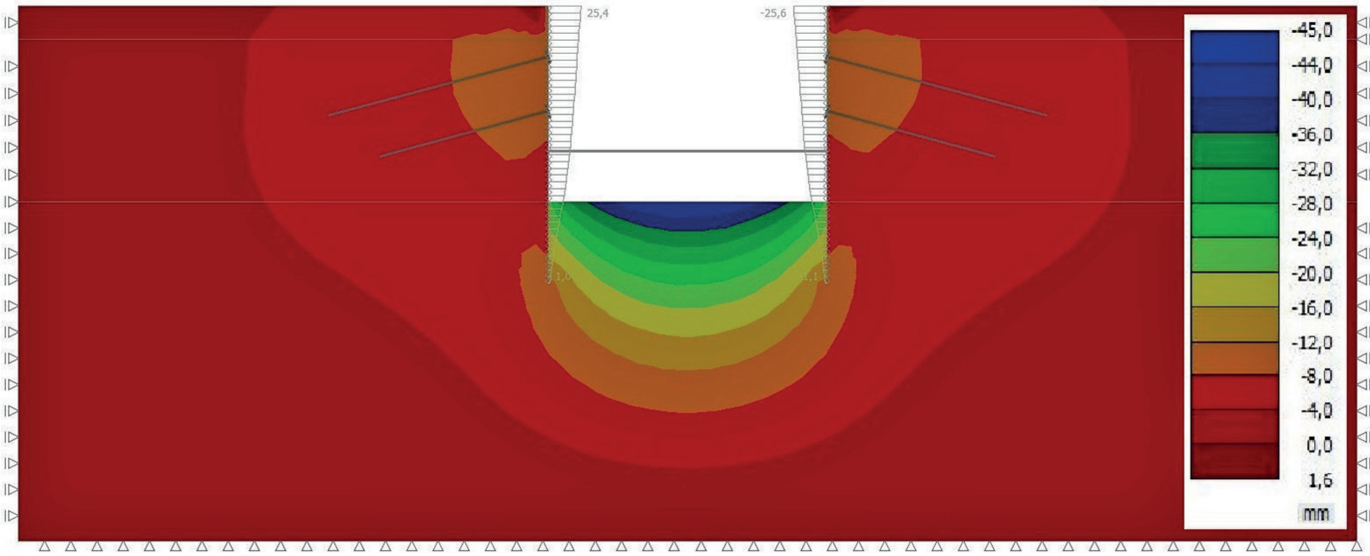

Fig. 6. Vertical displacements in the final excavation stage – Model 1 (GEO5)

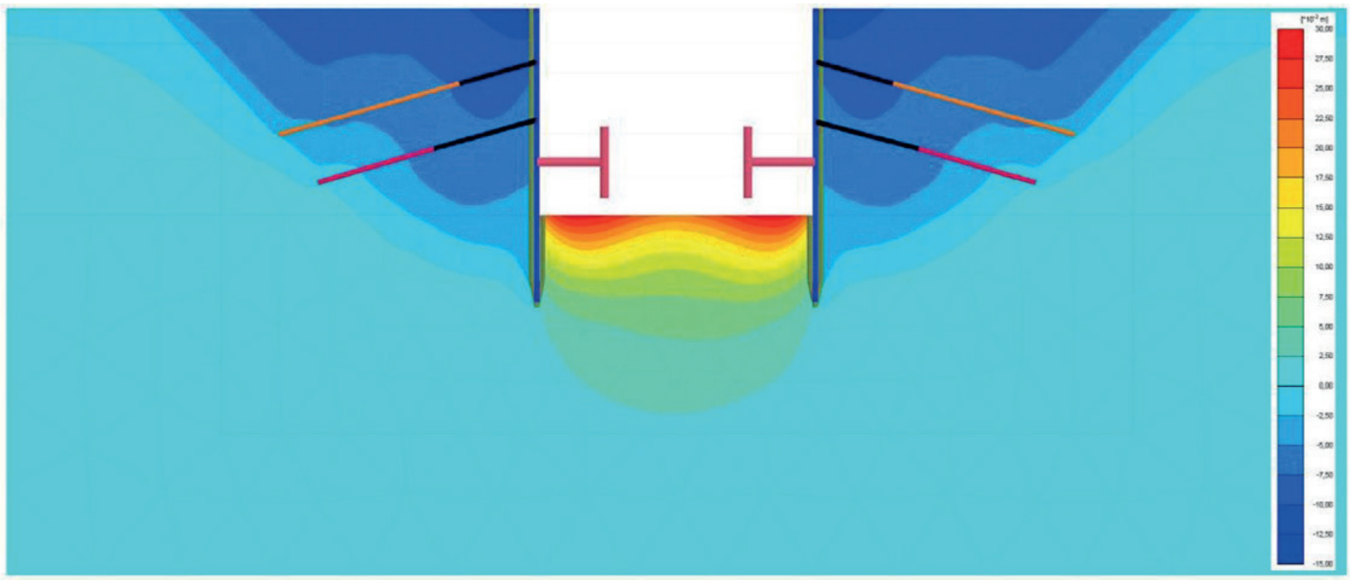

Fig. 7. Vertical displacements in the final excavation stage – Model 2 (Plaxis)

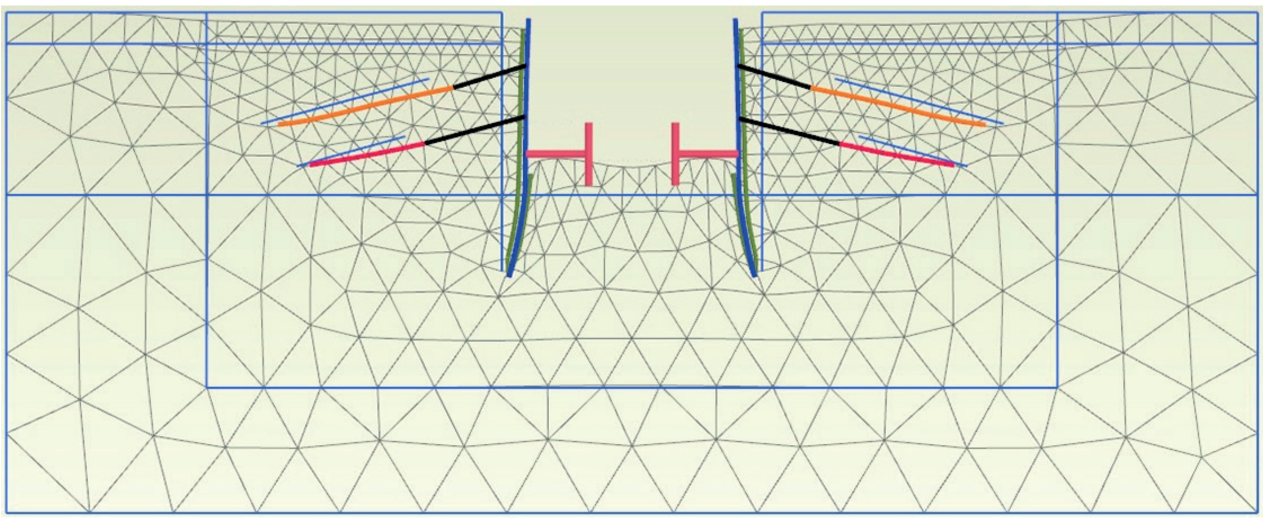

Fig. 8. Deformations int the final excavation stage – Model 2 (HSS)

– stage 3 (excavation –4.55 m bgs) – Fig. 9,
– stage 5 (excavation –8.85 m bgs) – Fig. 10,
– stage 7 (excavation –11.85 m bgs) – Fig. 11,
– stage 9 (excavation –14.60 m bgs) – Fig. 12.

5. Summary and conclusions

The main goal of the paper was to analyze the possibility of application of two types of advanced constitutive models – hypoplastic and hardening – for modelling of excavations in preconsolidated clays (Pliocene clays in Warsaw). The constitutive models were – Hypoplastic Clay [18–20], Hardening Soil [23, 24] and Hardening Soil Small [25].

Three basic displacement parameters were chosen for the analysis: the horizontal displacement of the top of the diaphragm wall (u_{xw}), the heave of the bottom of excavation described by the value of vertical displacement of the bottom of excavation (y_{uplift}) and vertical displacements of the ground surface in the influence zone (y_{ground}). These are the parameters describing the safety of the analyzed structure and surrounding buildings. Though, the results of the presented analysis may be used in the estimation of hazards or risk analysis associated with the construction of deep excavations.

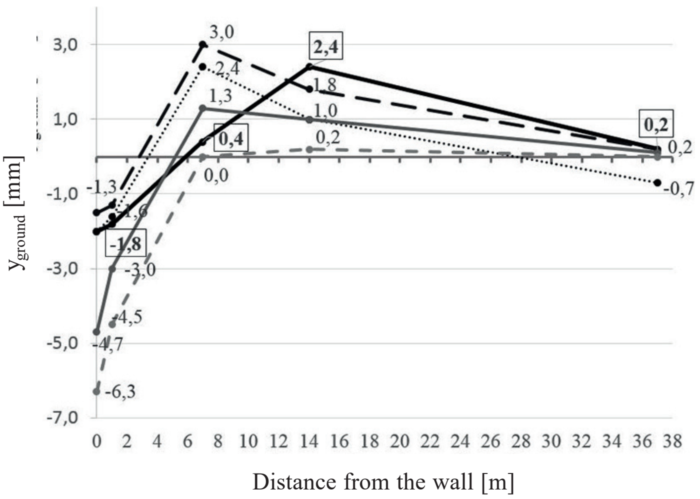

Fig. 9. Vertical displacements of ground surface behind the excavation wall in Stage 3 (excavation below the first row of anchors)

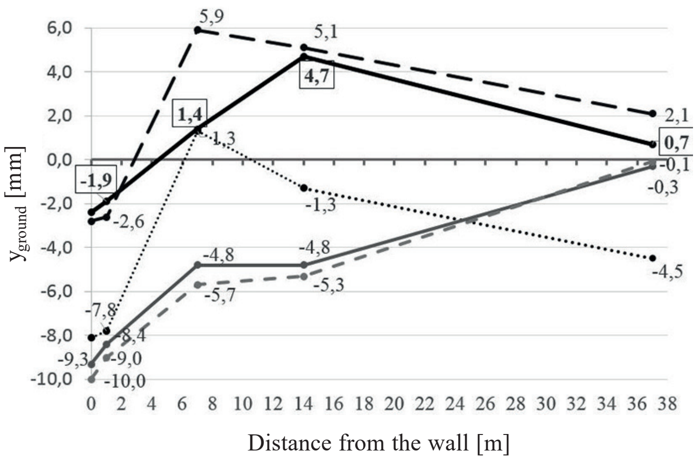

Fig. 11. Vertical displacements of ground surface behind the excavation wall in Stage 7 (excavation below the level of struts)

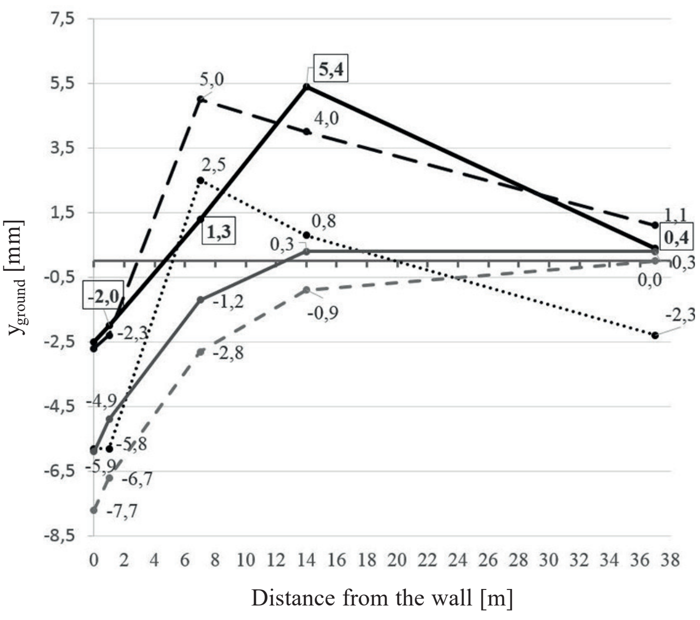

Fig. 10. Vertical displacements of ground surface behind the excavation wall in Stage 5 (excavation below the second row of anchors)

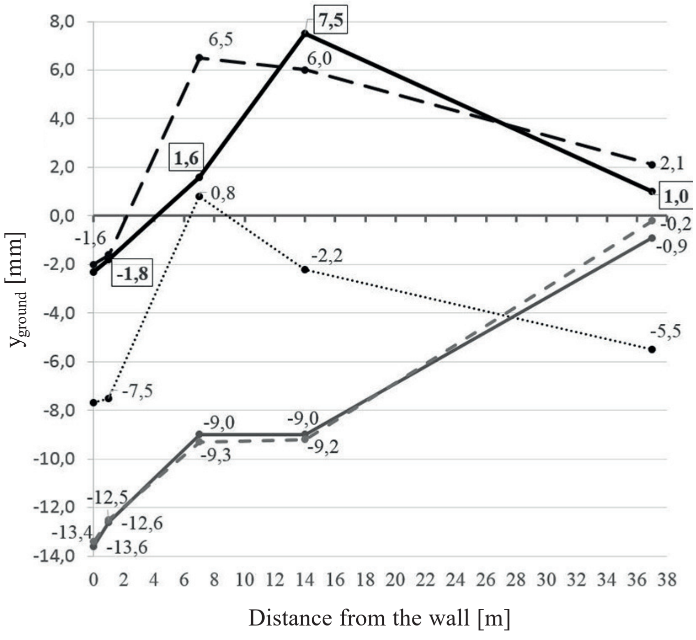

Fig. 12. Vertical displacements of ground surface behind the excavation wall in Stage 9 (final excavation)

The results of numerical, parametric analysis in terms of those three parameters were compared to the results of in-situ displacements measurements taken during construction.

The observations in relation to theoretical (calculated) values of horizontal displacements of the wall (u_{xw}), as well as vertical uplift of the bottom of excavation (y_{uplift}) are similar for all analyzed models. Obtained values were of the same range, differing up to 19%–25% in the final excavation stage, reflecting the real displacement in an acceptable manner from an engineering point of view. Except the Hardening Soil Small model, which tends to underestimate both – displacement of the wall (u_{xw}), up to 26%, and the heave of the bottom of the excavation (y_{uplift}), up to 51%, both in the final excavation stage.

Significant differences were obtained in terms of the vertical displacement of the ground surface behind the wall (y_{ground}). The use of the Hypoplastic Clay model resulted in a very good mapping (only up to 13% difference comparing to the real displacement) of the slight elevation of the ground surface observed in reality in all construction stages, whereas for both Hardening Soil and Hardening Soil Small models no heave or elevation of the ground surface was obtained in the influence zone behind the wall. The values of settlements directly behind the excavation wall were also significantly overestimated by both latter models (being up to 7,5 times higher than measured settlement values).

Hypoplastic Clay model proved to be suitable for modelling excavations made in preconsolidated clays specific for the area of Warsaw, both in terms of the design of excavation walls (Fig. 4) and the prediction of vertical displacements of ground surface behind the excavation wall (Fig. 9–12), only underestimating the uplift of the bottom of the excavation (25%), the parameter, which is usually not critical for the structure and the vicinity.

On the contrary, Hardening Soil models didn't provide proper estimation of displacements of the ground surface behind the excavation wall in the influence zone, also underestimating significantly displacements of the wall and the heave of the bottom of the excavation.

Though it may be concluded that the Hypoplastic Clay constitutive model is best suitable for modelling deep excavations executed in preconsolidated Pliocene clays in Warsaw.

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