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THE RESULTS OF ANALYSES OF DEEP EXCAVATION WALLS USING TWO DIFFERENT METHODS OF CALCULATION

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Deep excavation walls can be analyzed and calculated by using classical methods (currently rarely in use due to their many simplifications) or numerical methods. Among the numerical methods we can distinguish a simplified approach, in which the interaction between soil and a wall structure is modelled by a system of elasto-plastic supports, and the finite-element method (FEM) in which the soil is modelled with mesh of elements. It is a common view that if we want to analyze only wall constructions, the first, simplified method of calculation is sufficient. The second method, FEM, is required if we want to further analyze the stress and strain states in the soil and the influence of the excavation on the surrounding area. However, as it is demonstrated in the paper, important differences may appear in the calculation results of both methods. Thus, the safety design of a deep excavation structure depends very much on the choice of calculating method.

Key words: deep excavation wall, numerical analysis, earth pressure.

1. Introduction

Excavation walls with several levels of struts or anchorages are geotechnical structures operating in complex conditions. They belong to the staged constructions, whose execution consists of several phases in which both the work scheme, as well as internal forces, deformation and displacement of the structure successively evolve. Calculation and design of deep excavation walls should therefore take into account the excavation phases. Moreover, the wall construction works interactively with the surrounding soil, which constitutes an additional complication in calculations, all the more so on account of the complexity of this physical phenomenon (Bolt, Dembicki, Horodecki [1, 2], Siemieńska-Lewandowska [3], Grzegorzewicz [4], Kłosiński [5], Ou, Lai [6]). Proper examination of the excavation structure behaviour requires numerical methods of calculation. Previously used conventional (analytical) methods are not suitable because of their large simplifications (Jarominiak [7]).

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In numerical methods, we can distinguish two basic ways of modelling the interaction between soil and structure. The first is a simplified method based on a modified Winkler model, in which the soil reaction is expressed by a number of elasto-plastic supports (Kosecki [8]) or by a continuous elasto-plastic substrate. The reaction of the soil occurs on both sides of the wall and can vary in range between boundary active pressure and boundary passive pressure, depending on the direction and value of wall displacement (Rymsza [9], Krasiński [10]). The active and passive boundary pressure values of the soil and the initial, static value of soil pressure are determined here by classical methods. This calculation method is generally believed sufficient to define the internal forces in an excavation wall, the forces in the struts or anchorages, as well as to predict structure displacements, and thus it is also considered sufficient to produce a safe construction design. Unfortunately, this method does not allow us to estimate the influence of deep excavation of the surrounding area. But we can obtain this by using the second method, in which the surrounding mass of soil is expressed by using a finite element mesh, and the soil is described by using a suitable material model, depending on the type of soil and its physical and mechanical properties. One of the advantages of the FEM computational analysis is that it provides a lot of information regarding the state of stress and strain, and other phenomena occurring in the soil (Kok Sien et. all [11]). However, this is not the only factor distinguishing the two calculation methods. Significant differences may also occur in results concerning internal forces in the wall structure, including its struts or anchorages. These differences and their possible causes will be discussed later in this article.

2. Description of calculation methods and computer applications

Two programs were used in the analyses. One was the author's original program, OGW (*Obudowy Głębokich Wykopów* – Deep Excavation Walls) (Krasiński [10]), using simplified methods, and the other was the PLAXIS program, using FEM (Plaxis Version 8 [12]). The OGW program describes soil reaction to wall displacements as a bilinear relationship (Fig. 1, Fig. 2). The calculation proceeds iteratively. After the wall displacements reach a certain value, the soil pressure behind the wall falls to the limit value, and here further on changes in the soil reaction may only occur if the wall is pushed back towards the soil (e.g., during pre-stressing of the anchors). The calculations are performed in stages corresponding to the subsequent phases in the excavation procedure. The calculation results produce the values of bending moments, forces in struts and anchors, as well as horizontal wall displacements for all the excavation stages. By using this method we can also obtain the distribution of soil reaction behind and in front of the wall.

A Hardening Soil (HS) model of the PLAXIS program was applied for this particular analysis. This is an advanced model for soil behaviour simulation (Fig. 3). Soil stiffness is described much more precisely, using three different values (E_{50}, E_{ur}

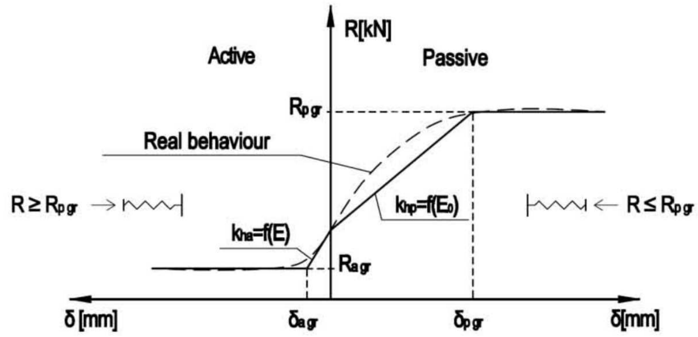

Fig. 1. Bilinear soil interaction dependence on the wall displacement.
Rys. 1. Zależność reakcji gruntu od przemieszczeń obudowy wykopu

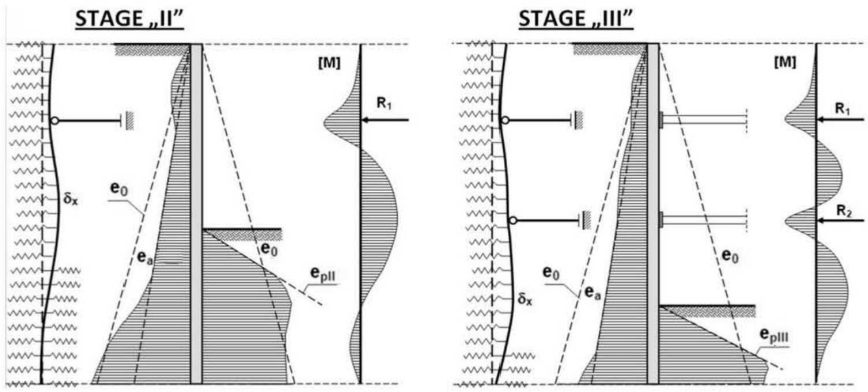

Fig. 2. The interaction between soil and wall modelled by elasto-plastic springs (OGW program).
Rys. 2. Współpraca gruntu z obudową wykopu, wymodelowana za pomocą więzi sprężysto-plastycznych (program OGW)

and E_{oed}). Here the increase of stiffness modulus value, resulting from stresses, was also taken into account. The HS model is based on a hyperbolic model which was modified to use the theory of plasticity, take into account the dilatancy of the soil and introduce a closed area of plasticity, CAP (Plaxis Version 8 [12], Duncan and Chang [13], Brinkgreve and Bakker [14]). The calculations of the excavation wall in the PLAXIS program were also carried out in stages (Fig. 4).

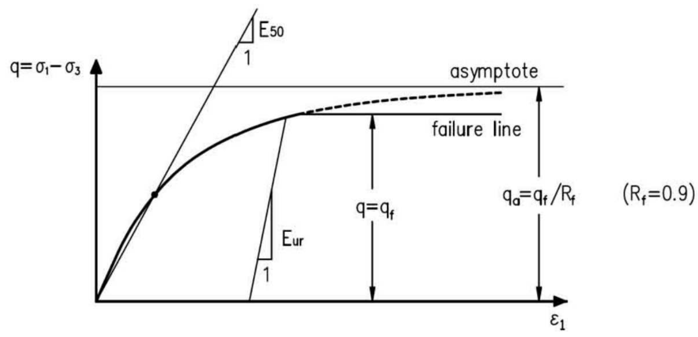

Fig. 3. Hyperbolic stress-strain relationship used in HS model (PLAXIS program).
Rys. 3. Hiperboliczna zależność naprężenie-odkształcenie w modelu HS gruntu (program PLAXIS)

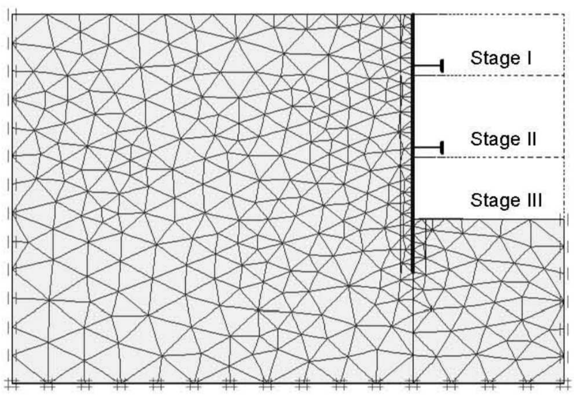

Fig. 4. Soil discretization by finite element mesh in PLAXIS program.
Rys. 4. Podział ośrodka gruntowego na siatkę elementów skończonych w programie PLAXIS

3. Examples of analysed deep excavation walls

3.1. Geometrical properties

For both calculation methods the same geometrical and material properties of the analysed excavation wall constructions were taken into account (Fig. 5). The excavation depth was achieved in three stages: 3 m, 7 m and 10 m. The excavation width was 12 m. A wall measuring 12.6 m in total height was analyzed as a diaphragm wall (option I) and as a sheet pile wall (option II). Two levels of struts or anchors at a depths of 2.5 m and 6.5 m were presumed. The inclination of the anchors was $\alpha = 20°$.

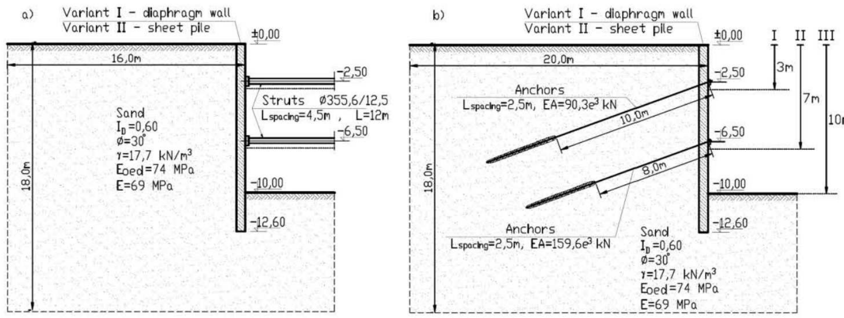

Fig. 5. Geometrical parameters of the deep excavation walls.
Rys. 5. Parametry geometryczne analizowanych konstrukcji obudów głębokich wykopów

3.2. GEOTECHNICAL PARAMETERS

The soil was taken as a homogenous, normally consolidated, fine sand with a density index of I_D = 0.6. The geotechnical parameters of the sand are given in Table 1. To avoid additional influences, the subsoil was assumed to be unsaturated.

Table 1

Geotechnical parameters of the soil used in analyses.
Geotechniczne parametry gruntu przyjęte w analizach

OGW	Sand I_D=0,6	c=0 kPa	γ = 17,7 kN/m^3	E_0=55 MPa	ν = 0,2	interfaces	Diaphragm wall
		ϕ = 30?	γ'=9,1 kN/m^3	E=69 MPa	Φ_L=1		
					Z_c=5m	δ_a	0,5
					η = 0,85	δ_p	-0,67
Plaxis		c_{ref}=0,5 kPa	γ_{unsat}=17,7 kN/m^3	E_{50}^{ref}=65 MPa	ν_{ur}=0,1	interfaces	Sheet pile
		ϕ = 30°	γ_{sat}=19,3 kN/m^3	E_{ur}^{ref}=178,8 MPa	K_o^{NC}=0,5		
		ψ = 0°	k_x=1	E_{oed}^{ref}=74,37 MPa	R_f=0,9	δ_a	0,5
			k_y=1	m=0,5; p_{ref}=100 kPa		δ_p	-0,67

4. THE RESULTS OF CALCULATIONS

4.1. DIAPHRAGM WALL WITH STRUTS

Figure 6 shows the bending moments in the strutted diaphragm wall (option I) during subsequent stages of excavation. Greater values were obtained from the PLAXIS program calculations, especially in stages II and III. Significant differences exist primarily in the force values inside the struts. Additionally, we should note that in the PLAXIS program resulted in the first level strut forces increased significantly in excavation stage III, whereas in the OGW program it resulted that the same forces at this stage were virtually unchanged. The differences in the wall displacements obtained from the two calculation methods were mainly due to global movements of the entire structure together with soil mass displacements, which were caused by the soil relaxation during the excavation. Diagrams in Fig. 7 clearly show almost parallel displacement shifts obtained from the two methods. Naturally, it was not possible to model global soil mass displacement and relaxation using the OGW program method.

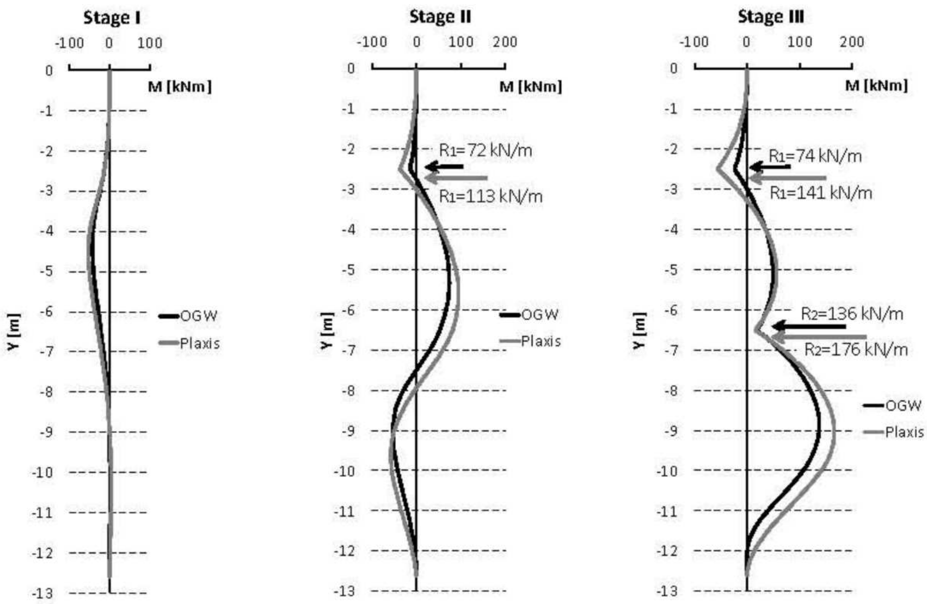

Fig. 6. Bending moments and strut forces in the diaphragm wall.
Rys. 6. Momenty zginające i wartości sił w rozporach dla obudowy ze ściany szczelinowej

Plots of normal soil reaction distribution on both sides of the wall (Fig. 8) show that in the PLAXIS program the maximum passive soil reaction was mobilized at greater depths than in the OGW program. Very interesting and significant differences can be noticed in the distributions of active soil pressure behind the wall. These differences appear mainly in stages II and III, and concern the upper parts of the wall. In stage I the plots of active soil pressure from both methods are fairly close together. In the OGW calculations elastic supports in the upper parts of the wall were successively “turned off”, which usually meant that no changes in the soil pressure value occurred at subsequent excavation stages. The PLAXIS program results show a visible pressure value increase in the upper parts of the soil. This is the direct reason for the above-mentioned increased force values in the struts and bending moments.

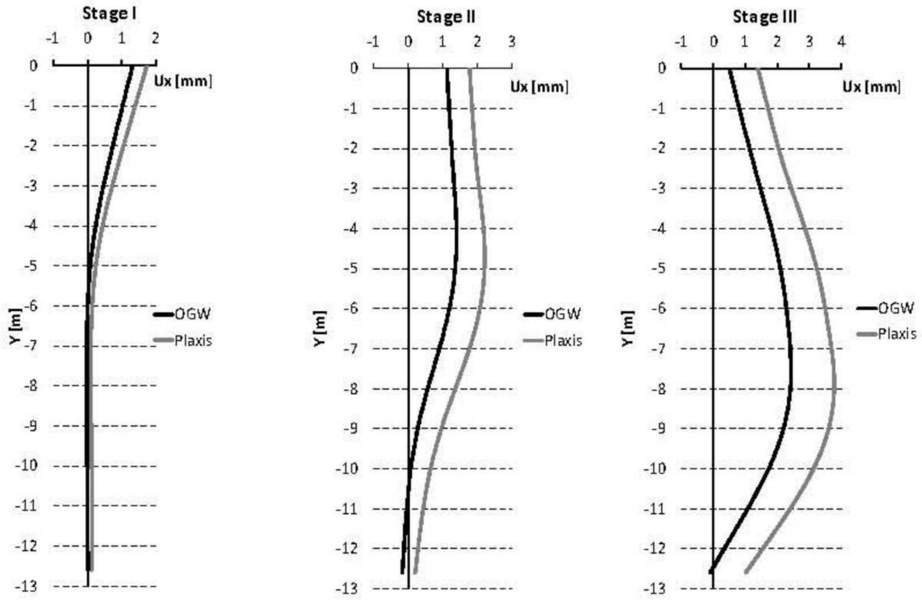

Fig. 7. Horizontal displacements of the diaphragm wall with struts.
Rys. 7. Przemieszczenia poziome rozpieranej ściany szczelinowej

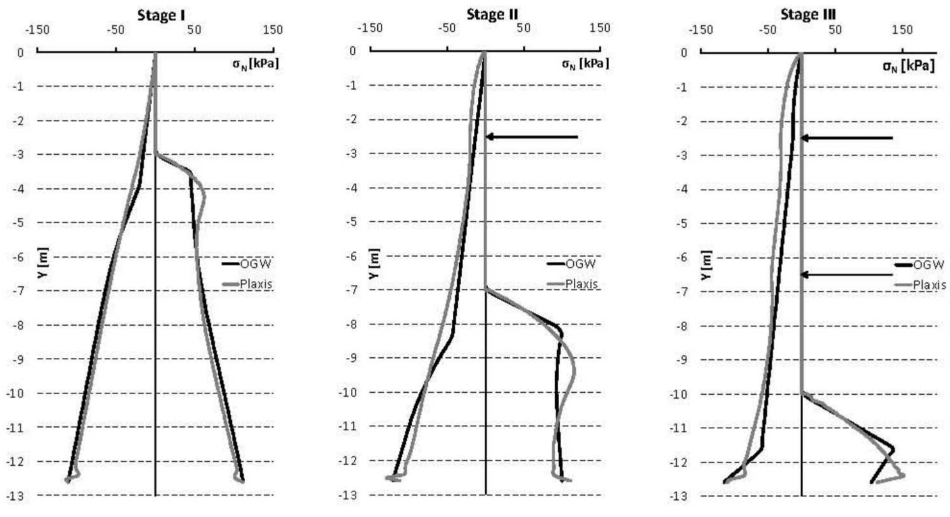

Fig. 8. Soil pressure acting on the diaphragm wall with struts.
Rys. 8. Rozkłady parcia gruntu działającego na rozpieraną ścianę szczelinową

4.2. Sheet pile wall with struts

In the case of the sheet pile wall, higher values of bending moments were obtained from the OGW calculations (Fig. 9). Significant differences were also observed with regard to the forces in the struts, but this time, only in first level, where a greater strut force value was obtained from the PLAXIS program, while in the second level both methods produced similar strut force values. Moreover, the PLAXIS results showed an increased force in the first strut of stage III, like in the case of the diaphragm wall, whilst the OGW results showed even a small force value decrease in this stage.

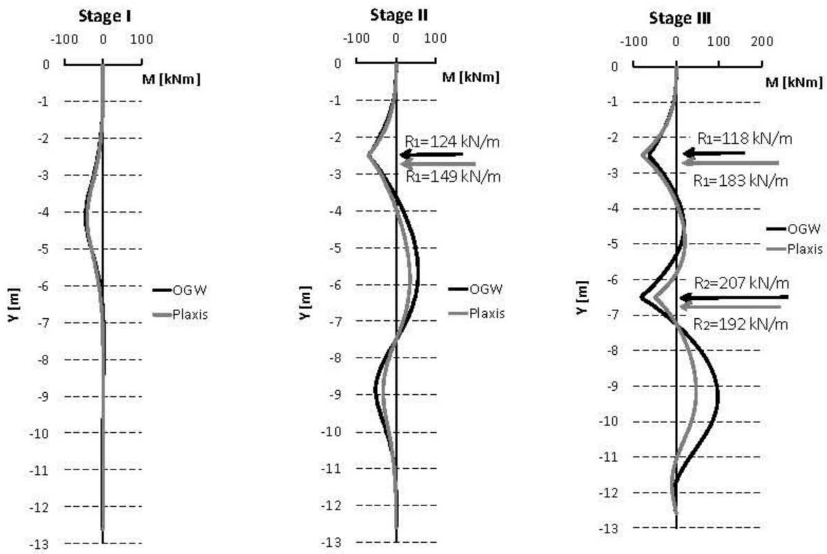

Fig. 9. Bending moments and strut forces in the sheet pile wall.
Rys. 9. Momenty zginające i wartości sił w rozporach dla obudowy ze ścianki szczelnej

The maximum value of wall displacement was obtained from the OGW program (Fig.10), although it occurred in the lower part of the wall (near the bottom of the excavation), whereas in the PLAXIS program maximum displacement occurred at the very top of the wall. The soil reaction plots in Fig.11 again showed that the PLAXIS program produced an increase of soil pressure in the upper excavation sections during stages II and III. Moreover, there were marked soil pressure concentration differences behind the sheet pile wall. In the OGW results the concentrations occurred in areas between the struts, whilst the PLAXIS results put these concentrations in the struts axes. The variation in soil pressure concentrations explains the differences in wall bending moment values and strut force values.

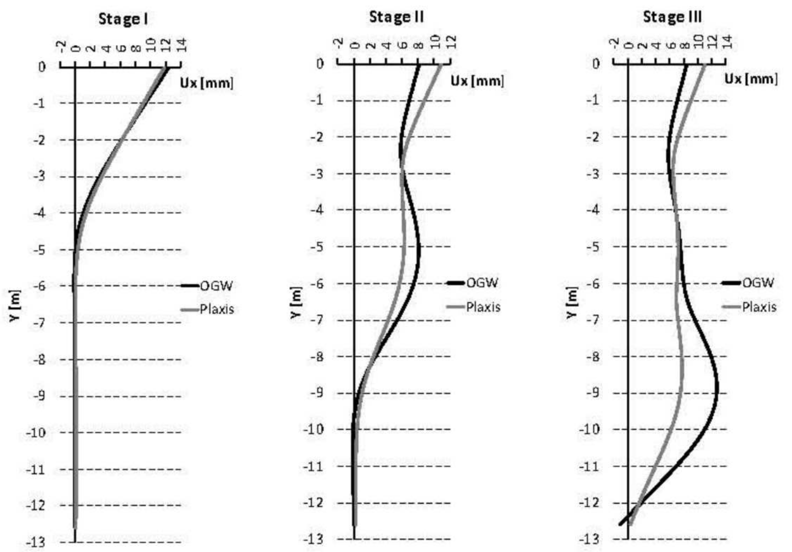

Fig. 10. Horizontal displacements of the sheet pile wall with struts.
Rys. 10. Przemieszczenia poziome rozpieranej ścianki szczelnej

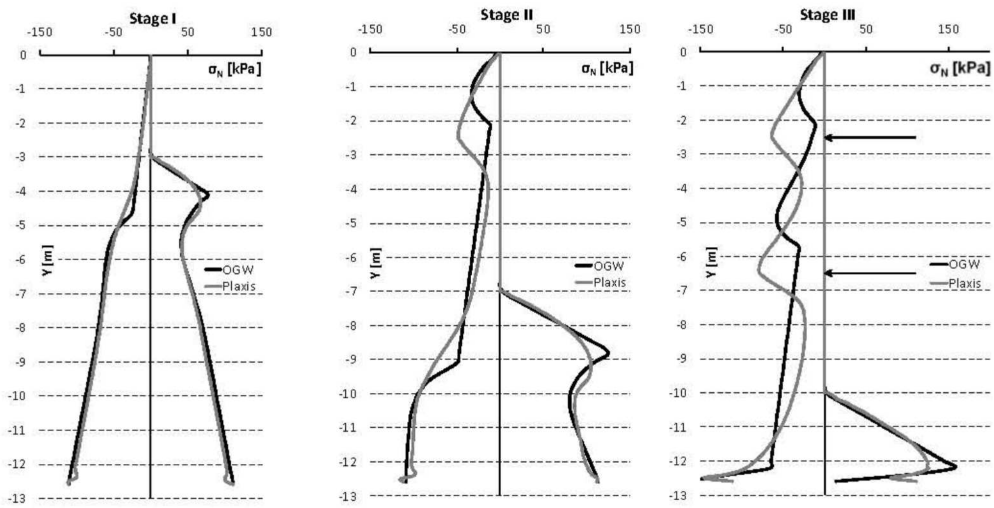

Fig. 11. Soil pressure acting on the sheet pile wall with struts.
Rys. 11. Rozkłady parcia gruntu działającego na rozpieraną ściankę szczelną

4.3. Anchored diaphragm and sheet pile wall

The analysis of anchored walls generally showed a similar pattern of result differences between the two methods as in the case of the strutted walls. Likewise, the distribution and bending moment values in anchored walls was similar to those in strutted walls (compare Fig. 6 with Fig. 12 and Fig. 9 with Fig. 13). Slightly larger differences existed between forces in struts, and the corresponding forces in anchorages. These may result from the choice of anchor pre-stressing forces. The displacement plots for anchored walls, not presented here, showed very slight maximum value differences in relation to the walls with struts. These small differences, about 1mm for a diaphragm wall and 3mm for a sheet pile wall, resulted mainly from the pre-stressing of the anchors.

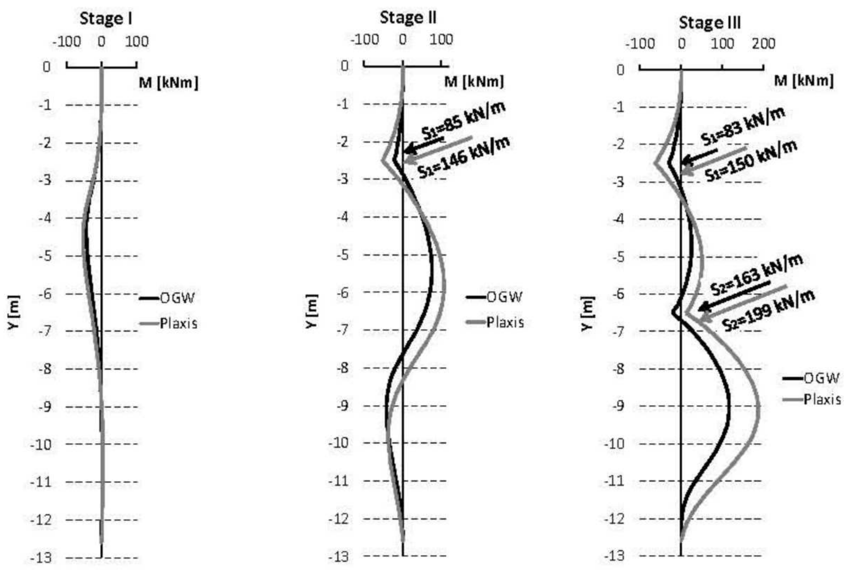

Fig. 12. Bending moments and anchor forces in the diaphragm wall.
Rys. 12. Momenty zginające i wartości sił w kotwach dla obudowy ze ściany szczelinowej

5. Analysis

The most important and interesting objective of this analysis is to explain the significant differences between the two calculation methods concerning the force and distribution of soil pressure acting on deep excavation walls (see Fig. 8 and Fig . 11). These differences are in turn the main reason for disparities in values concerning strut or anchor forces and bending moments in the wall.

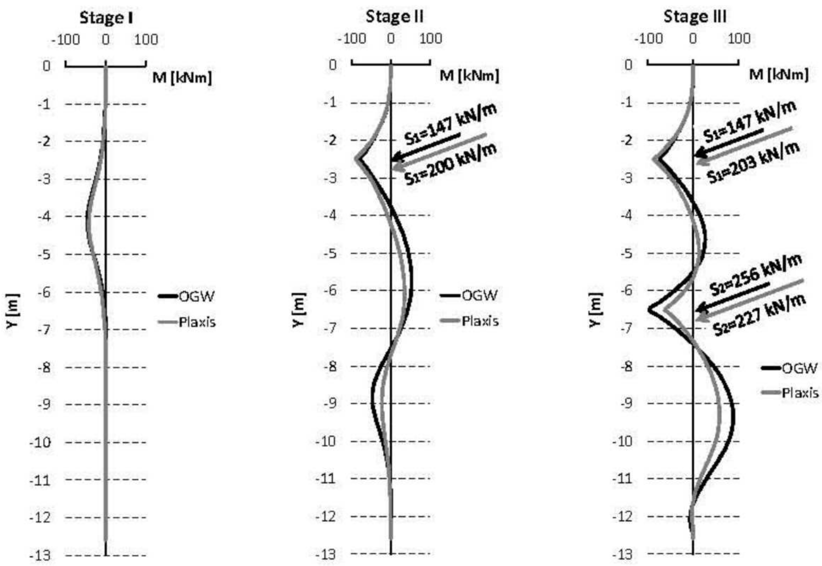

Fig. 13. Bending moments and anchor forces in the sheet pile wall.
Rys. 13. Momenty zginające i wartości sił w kotwach dla obudowy ze ścianki szczelnej

These differences in the result of calculation are caused by the soil movement that occurs behind the wall in the subsequent stages of deep excavation. The OGW calculation method does not take this soil movement into consideration. Fig. 14 provides a simplified illustration of this physical phenomenon.

In phase I of excavation, a deflection of the upper part of the wall occurs, causing the soil directly behind the wall (block 1) to relax and slip down. Thus, friction forces are generated on the shear surface, which reduce the soil pressure to the limit value or to a value somewhere between active and static soil pressure. Next, there is the installation of struts on the first level and the further deepening of the excavation. Due to the deformation of the wall below the struts, another shear surface is created in the soil and the next section of soil (block 2) slips down. Thus friction is again generated, but this time on two surfaces – on the top surface and on the bottom surface of block 2. Block 1 now is stopped from slipping further down by the strut. Thus the horizontal component (T_2^h) of the upper frictional force acts on block 1 and in consequence increases soil pressure on the adjacent part of the wall. In this way the mounted strut not only bears the pressure from the lower part of the wall (q^{II}), but also the above-mentioned friction forces horizontal component acting on block 1. Hence, the soil pressure increases at the top part of the wall during the second step of excavation. The FEM (PLAXIS) calculation results show this soil pressure increase, whereas the OGW result does not. In stage III the situation is repeated, except of this time the friction force (T_3) acts on blocks 1 and 2, and is thus distributed on two struts.

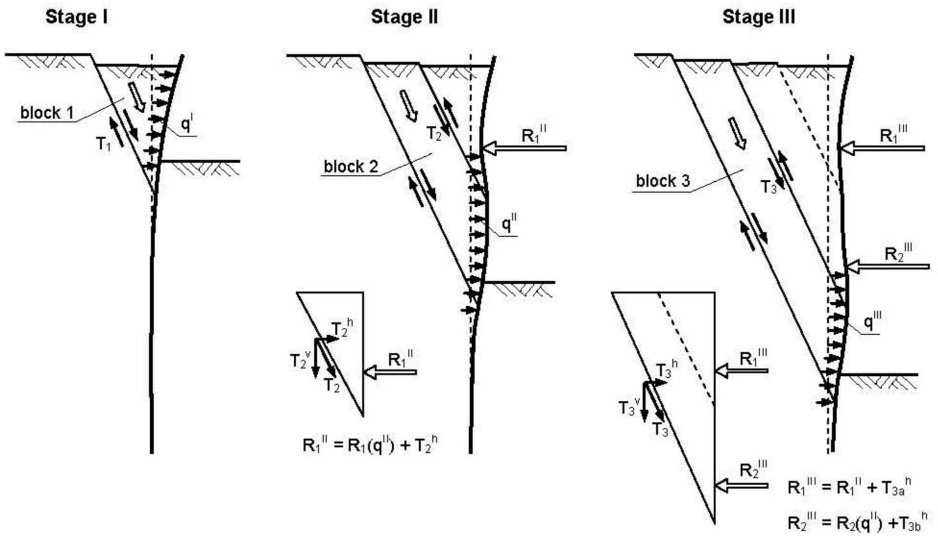

Fig. 14. A scheme of soil deformations in the area behind the wall during subsequent excavation stages.
Rys. 14. Schemat deformacji ośrodka gruntowego w rejonie za obudową w kolejnych etapach głębienia wykopu

The above explanation contradicts the commonly held opinion that when a wall and soil interact, and soil pressure decreases to limit value, then this value will not increase unless the wall is shifted back against the soil. As this analysis shows, such an increase may also occur due to the movement of soil in a region behind the wall.

The above-described phenomenon also explains the differences between OGW and PLAXIS calculation results for soil pressure distribution behind the wall. OGW calculations locate soil pressure concentrations in areas between struts, because they assume that pressure increases only in place where the wall moves towards the soil. The PLAXIS calculations, on the other hand, show the distribution of soil pressure concentration near the supports (struts or anchors) caused by the friction forces resulting from soil movement blockage after support installation. The phenomenon of soil pressure concentrations is more visible in less rigid walls, such as ones made of steel sheet piling (Fig. 11).

6. Conclusions

The results of two different calculation methods show significant differences regarding the forces in struts (or anchorages), bending moments and the distribution of soil pressure behind a deep excavation wall (see Fig. 6 to Fig. 11). These differences clearly demonstrate the serious inadequacies of simplified methods that use the OGW program or similar applications. Using the elasto-plastic supports to model the interaction between soil and a wall does not take into account complex physical phenomena occurring in the soil. These phenomena, as the analysis of simple wall examples has shown, can significantly and adversely affect the work conditions of the wall.

It is difficult to determine which method gives more realistic results, nevertheless, one has to recognise that the method based on elasto-plastic supports is connected with the risk of underestimating forces in struts and anchors, and in some cases also the values of bending moments in excavation walls. Unfortunately, this method is more popular among the engineers and designers. Therefore there should be greater awareness that by relying solely on this more popular method one can design struts or anchors of inadequate bearing capacity, which could in turn lead to construction failure.

The arguments presented in this article can only be fully verified on the basis of measurements made on real excavation structures.

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WYNIKI ANALIZ OBUDÓW GŁĘBOKICH WYKOPÓW UZYSKANE Z DWÓCH RÓŻNYCH METOD OBLICZENIOWYCH

Streszczenie

Obudowy głębokich wykopów można analizować i obliczać przy użyciu metod standardowych (analitycznych – obecnie nie stosowanych ze względu na znaczne uproszczenia) lub numerycznych. Wśród metod numerycznych możemy wyróżnić rozwiązanie uproszczone, w którym współpraca pomiędzy gruntem i ścianą obudowy wyrażona jest za pomocą systemu podpór sprężysto-plastycznych oraz metodę elementów skończonych (MES), w której grunt modelowany jest za pomocą siatki elementów. Powszechnie uważa się, że do analizy samej konstrukcji obudowy wystarczająca jest metoda pierwsza, uproszczona. Druga metoda, MES, jest wymagana wówczas gdy dodatkowo chcemy przeanalizować stany naprężenia i odkształcenia w ośrodku gruntowym oraz określić oddziaływanie wykopu na otoczenie. Jednak jak pokazano w artykule, w wynikach obliczeń z obu metod mogą pojawić się znaczne rozbieżności. Bezpieczeństwo projektowania konstrukcji obudów głębokich wykopów w dużym stopniu zależy więc od wyboru metody obliczeniowej.

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