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THE INFLUENCE OF EXCAVATION SUPPORT CONSTRUCTION ON THE SETTLEMENT OF ADJACENT STRUCTURES

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ABSTRACT

The paper presents an examination of the subsoil settlement in the neighbourhood of a deep excavation with particular reference to the excavation support construction method. Analysis was carried out for a selected deep excavation with a residential building in the vicinity. The most used technologies were considered, i.e., sheet pile walls, soldier pile walls, bored pile walls, and diaphragm walls. A total of 14 wall variants were examined to investigate the differences that certain characteristics have on the surrounding area. The impact of excavation support was assessed for two anchorage level schemes, taking into account three excavation phases. On this basis, the settlement profile next to the excavation wall was determined, and the settlement of the adjacent building was calculated. A key factor identified was the trench wall support scheme. Furthermore, a strong correlation was found between the stiffness of the wall and the maximum settlement observed.

Keywords: deep excavation, settlement, diaphragm wall, sheet pile wall, soldier pile wall, pile wall

INTRODUCTION

Due to the lack of available city space resulting from the continuous development of infrastructure, it is becoming increasingly common for newly designed buildings to have at least one underground storey and to be in the immediate vicinity of existing buildings. Each phase of the construction work is associated with the occurrence of different displacements (in magnitude and direction) and the time-varying displacements of the subsoil, and affect neighbouring buildings on a greater or smaller scale. One of the most relevant stages is a deep excavation phase with possible lowering of the groundwater table. The designer is required to prepare a project to support the excavation walls and to determine the influence of the construction execution on the adjacent area to ensure the safety of the excavation and the nearby facilities. As a result of the excavation, extensive wall displacements may occur and cause considerable settlement at the foundation level of the neighbouring structures. This situation may threaten the stability of all the facilities or "only" affect the utility conditions (Korff, 2009; Pająk, Sękowski, Kwiecień & Wieczorek, 2018). Settlement of the structure, depending on its location and the ground surface deformation, may be uniform or non-uniform, with the latter posing a greater negative influence. The issue of predicting ground deformation around an excavation and the influence on neighbouring buildings is an important issue in engineering practice and

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has been studied by many researchers in various aspects (Hsieh & Ou, 1998; Horodecki & Dembicki, 2007; Popielski, 2012; Mitew-Czajewska, 2014; Mei, Wang, Zhou & Fu, 2022).

The range and magnitude of settlement taking place are influenced by many factors. The most important are the depth of the excavation, the type of soil, the static scheme in which the retaining structure operates, and the support technology. The settlement profile in the vicinity of the excavation depends on the magnitude and nature of the deflection of the wall. It is not possible to completely avoid the shoring displacement; however, an excessive horizontal displacement of the excavation wall, identified as greater than 3% of the excavation depth (*H*), must not be allowed. According to Long (2001), the main factor (33%) for excessive displacements is the initial phase of the structure, i.e., when the wall is working in a cantilevered pattern. The second most commonly identified cause was the use of flexible support technology (22%).

The influence range of an excavation is usually defined as a multiple of the excavation depth (*H*). Various authors have defined, based on empirical data, the predicted influence range as follows: O'Rourke and Clough (1990) established it as 2–4*H* for cohesive soils (London clays), and for non-cohesive soils, Schweiger, Freiseder and Breymann (1999) found this range to be 1.5–2*H*. In Poland, the most widely used guidelines in practice are those published by Kotlicki, Łukasik, Godlewski and Bogusz (2020). The authors suggest assuming a range equal to 2*H* in sands, 2.5*H* in silts, and 3.0–4.0*H* in clays. Distance can be reduced by 20% if the groundwater level is not lowered. According to these guidelines, the area with the greatest influence is determined as 0.5*H* in sands, 0.75*H* in silts and 1*H* in clays. The maximum vertical displacements in the vicinity of the excavation are estimated to be between 0.002*H* in compacted sands and 0.02*H* in cohesive soils (Michalak & Przybysz, 2021).

The study focused on analyses of the static scheme influence and the type of excavation wall on the settlement of the adjacent area and the residential building in the vicinity. The assessment was based on a case study with two anchorage level schemes considering the three stages of excavation. The effect of the selection wall profile was also analysed with respect to the different types of walls: rigid (bored pile wall, diaphragm wall) and flexible (sheet pile wall, soldier pile wall). A total of 14 different variants were analysed for each scheme and stage. As a result, the influence of these elements on the range, magnitude, and profile of the ground surface settlement was assessed.

MATERIAL AND METHODS

The location on Kołłątaja in Wrocław (Poland) was used for analyses. A seven-metre-deep excavation was designed adjacent to the existing building, as shown in Figure 1a. The excavation segment (blue line) that has been examined is located close to the existing building and, therefore, will have the greatest influence on the neighbourhood. Figure 1b shows a side view of the actual five-storey townhouse in the vicinity of the planned excavation. The geotechnical profile and soil parameters were adopted based on local geological data and are summarised in Table 1. The soil profile is mainly composed of compacted medium sands, and the groundwater table has not been identified.

The total height of the wall was assumed to be 11 m. The location and loads from the adjacent property were set as follows. The first floor is elevated and intended for commercial premises. It has been assumed, in a simplified manner, that all storeys above the service units are of identical height and intended as residential units. Due to the lack of documentation of the foundation, a three-strip footing, one metre wide (pink lines in Fig. 1a), spaced 6 m apart was assumed. Subsequent footings, due to the large distance, were omitted. The axis of the closest footing was taken at 3 m from the retaining structure and at a depth of 0.8 m. The design loads (*q*) on the individual footings were assumed from the estimated construction load calculations as 417.75 kPa (for the footing F1) and 477.63 kPa (for F2 and F3). The calculation schemes are shown in Figure 2. To assess the influence of the support scheme and the phasing of the work, two schemes differing in the location of

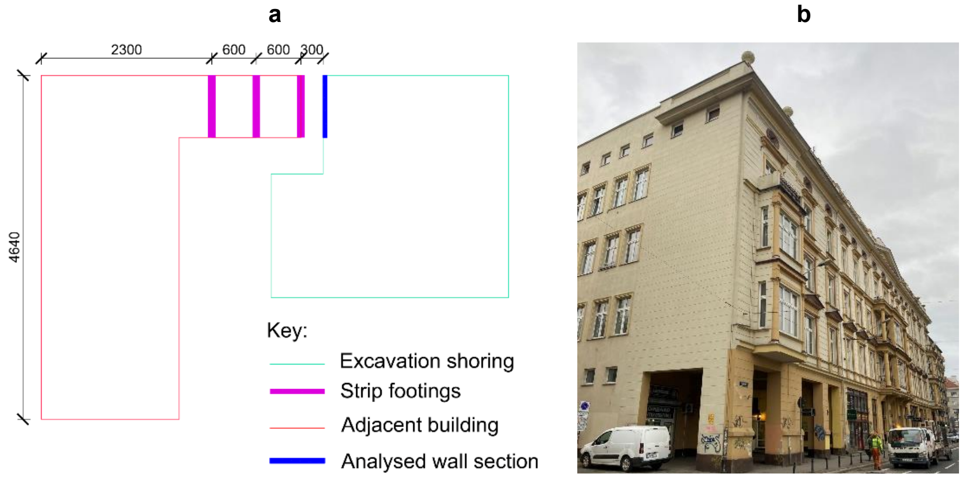

Fig. 1. Scheme of the deep excavation (a) and the townhouse in the vicinity (b)

Source: own work.

Table 1. Soil parameters

No	Soil	I_C [-]	I_D [-]	γ [kN·m^{-3}]	c [kPa]	φ [°]	υ [-]
1	sand with clay	0.60	–	20.60	10.65	11.6	0.32
2	medium sand	–	0.45	16.68	0	32.7	0.25
3	medium sand	–	0.70	17.66	0	34.2	0.25
4	medium sand/fine sand	–	0.80	17.66	0	34.9	0.25
5	clay with gravel	0.60	–	19.50	10	26.0	0.35

Source: own work.

the ground anchor heads were considered, and for each scheme, three stages of excavation were examined. Ground anchors with a 2 m spacing and with a pre-tension of 350 kN (upper) and 400 kN (lower) were used as support for the excavation wall in Scheme 1. In Scheme 2, it was 300 kN and 400 kN, respectively. The inclination, the total length, and the bond length in both variants were the same.

First, a static analysis was carried out to determine internal forces. Next, different excavation wall variants that met the ultimate limit state conditions were selected to analyse the wall and surface displacement. Two types of flexible wall (sheet and soldier pile wall) and two types of rigid wall (pile and diaphragm wall) were selected. Each technology was modelled in two or more common variants to test the differences that each parameter may have on the settlement of the subsoil next to the excavation. Due to the different positions of the ground anchor heads in Schemes 1 and 2, there was a difference in the internal forces. In both cases, the optimum profiles and reinforcement were selected in terms of the bending moment capacity. The exceptions are Variants A2 and A4 – due to the lack of profiles with smaller cross-sections – the same profiles are used in Schemes 1 and 2. The parameters of the retaining structures are shown in Table 2. The stiffness of the wall per metre is taken as EI, where E is Young's modulus of the material and I is the moment of inertia of the profile. In all cases, C30/37 concrete and S235 steel were used.

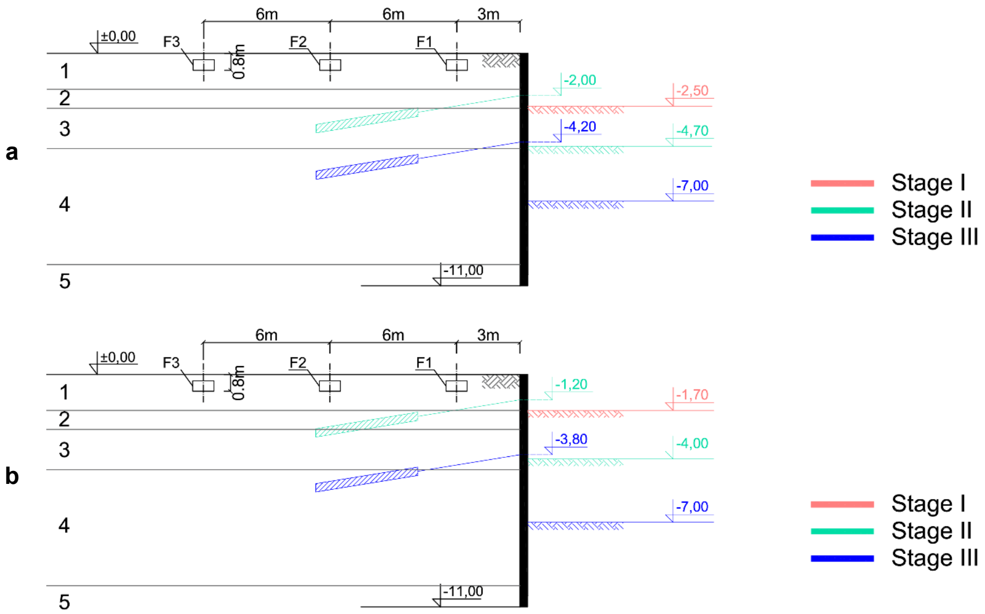

Fig. 2. Task Scheme 1 (a) and Scheme 2 (b)

Source: own work.

Table 2. The list of types and technical parameters of excavation walls

No	Type	Scheme 1		Scheme 2	
		description	*EI* [kNm^2]	description	*EI* [kNm^2]
A1	sheet pile wall[c]	“Z” profile MMZ11-1	30 030	“Z” profile SCZ 14	20 601
A2	sheet pile wall[h]	“Z” profile AZ 12-700	39 690	“Z” profile AZ 12-700	39 690
A3	sheet pile wall[c]	“U” profile MMU11-1	46 620	“U” profile MMU7-1	21 630
A4	sheet pile wall[h]	“U” profile PU 400×125	35 280	“U” profile PU 400×125	35 280
B1	soldier pile wall	HD260×68.2 1 m spacing	22 050	HE240 AA 1 m spacing	12 264
B2	soldier pile wall	HP305×79 1 m spacing	34 440	HD260×68.2 1 m spacing	22 050
C1a	bored pile wall[s]	0.6 m diameter 0.5 m spacing IPE180 per pile	210 870	0.4 m diameter 0.3 m spacing IPE A 120 per pile	61 200
C1b	bored pile wall[s]	0.6 m diameter 0.5 m spacing 7 Ø14 bars per pile	412 500	0.4 m diameter 0.3 m spacing 6 Ø10 bars per pile	119 100
C2a	bored pile wall[t]	0.6 m diameter 0.6 m spacing IPE200 per pile	181 170	0.4 m diameter 0.4 m spacing IPE140 per pile	49 800

Table 2 (cont.)

No	Type	Scheme 1		Scheme 2	
		description	*EI* [kNm^2]	description	*EI* [kNm^2]
C2b	bored pile wall[t]	0.6 m diameter 0.6 m spacing 6 Ø16 bars per pile	349 800	0.4 m diameter 0.4 m spacing 6 Ø12 bars per pile	94 200
C3a	bored pile wall[cn]	0.6 m diameter 0.8 m spacing IPE240 per pile	140 580	0.4 m diameter 0.6 m spacing IPE200 per pile	37 800
C3b	bored pile wall[cn]	0.6 m diameter 0.8 m spacing 8 Ø16 per pile	262 350	0.4 m diameter 0.6 m spacing 12 Ø10 per pile	62 700
D1	diaphragm wall	0.4 m thickness 5 Ø20 per metre	175 890	0.4 m thickness 10 Ø10 per metre	159 900
D2	diaphragm wall	0.6 m thickness 5 Ø16 per metre	594 000	0.6 m thickness 10 Ø10 per metre	540 000

[c]cold rolled; [h]hot rolled, [s]secant, [t]tangent, [cn]contiguous.

Source: own work.

On this basis, the influence range of the excavation on settlement and their magnitude was assessed for all the cases examined. All analyses were carried out according to Eurocode 7 (European Committee for Standardization [CEN], 2004) using the method of dependent pressures with GEO5 software – Sheeting Check. In the ultimate limit state, the DA2 was used, and in the serviceability limit state of the neighbouring townhouse, four parameters were assessed: settlement, rotation, relative deflection, and tilt of the building.

RESULTS AND DISCUSSION

When the ultimate limit state of the excavation support is analysed, the bending moment assessment is crucial. By carefully selecting the static support scheme, its magnitude can be minimised. It should be emphasised that not only the final scheme of the structure's operation, but also all intermediate stages must be considered. The values of the maximum bending moments for the different excavation stages in Schemes 1 and 2 (see Fig. 2) are summarised in Table 3. In Scheme 1, the highest values of the bending moment are observed in the first stage, i.e., in the cantilever pattern. In Scheme 2, the highest bending moment was reached at the final stage, but in all stages of the structure's operation, the values were quite similar.

The values of vertical deformation of the ground surface near the excavation wall calculated for all schemes, stages, and variants of the excavation wall are shown in Figures 3 and 4. The predicted theoretical influence range, according to Kotlicki et al. (2020), is 11.2 m (due to the lack of lowering of the water table), and the expected zone of greatest influence is 3.5 m. The total calculated influence range of the excavation in both schemes is 10.56 m (about 1.5*H*), which is slightly lower. However, in this respect, the results are in line with the dataset for sands. Regarding the zone of greater influence, in the case of Scheme 1, the maximum settlement occurs up to approximately 4 m from the excavation wall, but beyond this distance, the excavation influence and the ground surface settlement continue to be significant. In Scheme 2, the maximum settlement is found in the distance up to 5 m from the excavation wall. As a result, it should be noted that in the zone considered to be less affected by the excavation, this influence may still be high.

Table 3. The values of maximum bending moments

Scheme	Moment [kNm]		
	Stage I	Stage II	Stage III
1	174.75	111.72	81.85
2	63.91	69.50	73.20

Source: own work.

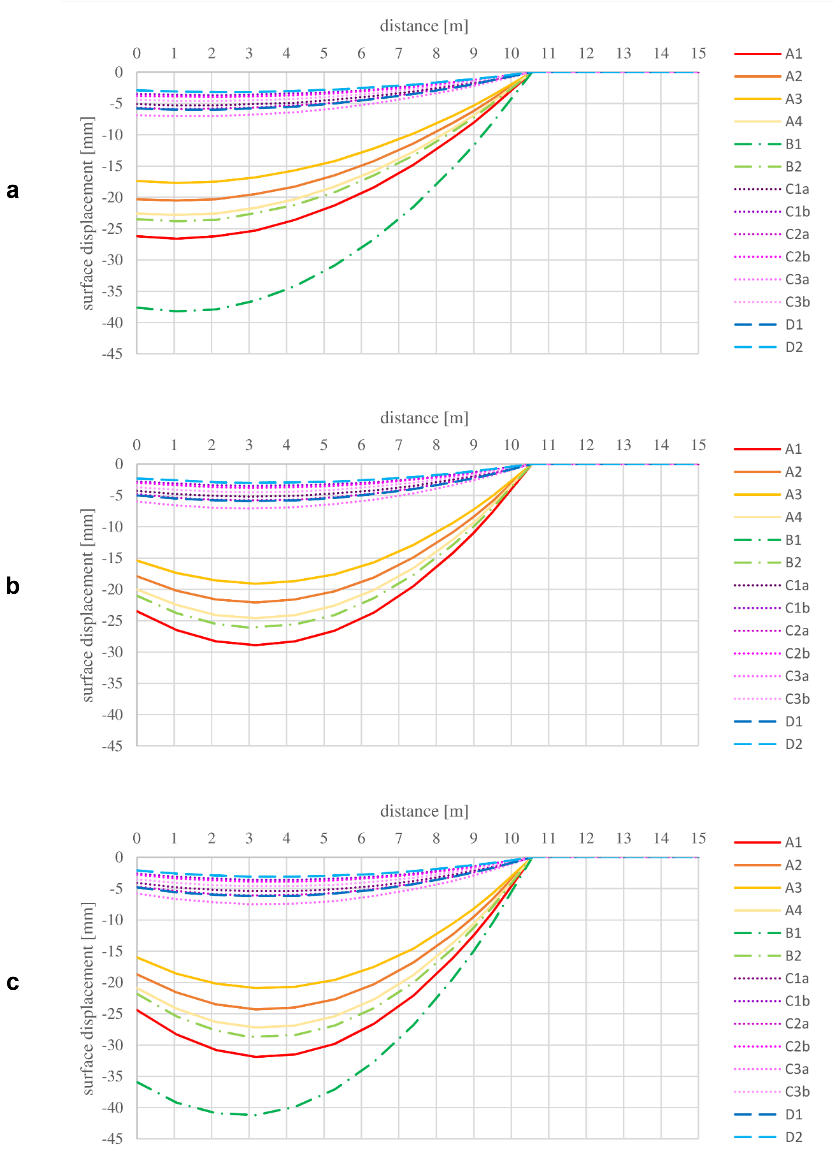

Fig. 3. The summary of vertical deformation of the ground surface depending on the distance from the excavation in Stage I (a), Stage II (b) and Stage III (c) for Scheme 1

Source: own work.

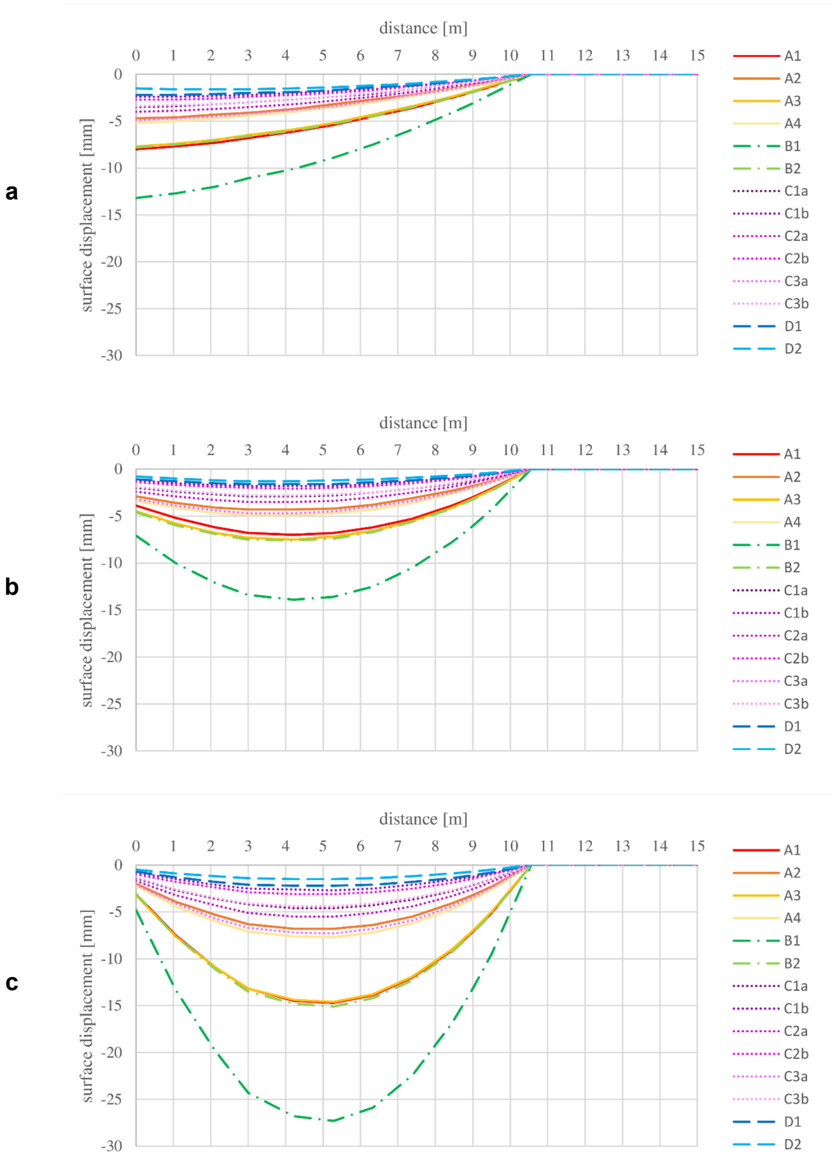

Fig. 4. The summary of vertical deformation of the ground surface depending on the distance from the excavation in Stage I (a), Stage II (b) and Stage III (c) for Scheme 2

Source: own work.

Significant differences can be observed when comparing the settlement profile in Schemes 1 and 2. In Scheme 1, the largest increase in maximum deformation occurs in Stage I (where the largest bending moments were observed). The increases in subsequent stages are small, and for some of the schemes, i.e., rigid walls (excluding the C3a pile wall) and Variant B1 (soldier pile wall), even a reduction of settlement was observed in Stage II. The observed pattern was the result of the horizontal deformation of the wall itself, for which the highest values occurred at the top of the wall in Stage I. In contrast, in Scheme 2, in Stage II, a reduction of settlement was observed too (except for Variant B1), but in Stage III, a large increase of vertical displacements in relation to Stage I was recorded. In addition, generally, the settlement found for Scheme 2 is considerably less. The maximum horizontal deformation of the wall in this case also occurred mostly at the top of the wall in Stage I (but was much lower) or, for walls with the lowest stiffness, below the lower anchor head in Stage III. Thus, the significant influence of the applied anchorage level scheme and the staging of the works on the settlement of the subsoil can be seen. The shallow support location of the excavation wall (by ground anchors or bracing) has a beneficial effect not only on the bending moments but, above all, on the settlement of the surface next to the trench. This is because the depth of the excavation during the cantilevered stage, described as the most dangerous operation pattern, is reduced. The position of the anchorage also determines the nature of the wall deflection and settlement profile. In the case of Scheme 1, where the anchors are installed lower, the greater displacements are located closer to the wall than in Scheme 2.

Special attention should be paid to Variant B1 in Scheme 1. As mentioned above, a reduction in settlement was observed in Stage II. For Variant B1, they have not only decreased, but also ground uplift was observed near the excavation. This was omitted in Figure 3 for readability reasons and is shown separately in Figure 5. This is the wall with the least stiffness in this scheme, and the observed effect is due to the pre-tension of the ground anchor used. However, Scheme 2 uses a wall with lower stiffness, and this effect did not occur. It should be noted that with ground anchors and a wall of very low stiffness, uplift of the ground next to the wall may theoretically occur. This should be monitored through the choice of excavation support scheme and pre-tension of the anchors.

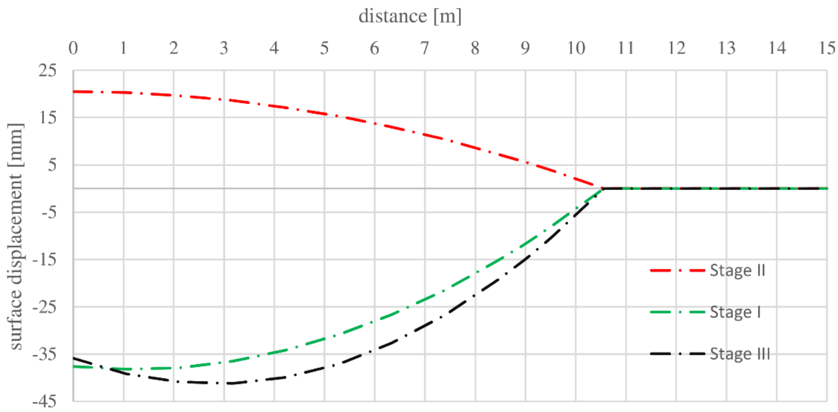

Fig. 5. The summary of vertical deformation of the ground surface for the B1 type of excavation wall for Scheme 1

Source: own work.

Analysing the effect of the selection of excavation wall technology, as expected, it was confirmed that the wall identified as being rigid (Variants C and D) generally gives less ground surface settlement next to the excavation than flexible ones (Variants A and B). The stiffness of the excavation wall expressed as *EI* was identified as the main factor. Overall, it can be considered that the most important parameter affecting stiffness is the choice of steel profile or reinforcement. Even within a single technology, it is possible to choose a variant with significantly different stiffnesses. The maximum surface displacements after complete excavation (Stage III) as a function of the excavation wall stiffness is shown in Figure 6. As can be seen in both schemes, maximum settlement is a power function of the wall stiffness (*EI*). Stiffness in the range up to 100 kNm^2 has the greatest influence on settlement, and for higher values, the influence of stiffness is low. Therefore, it can be concluded that it is ineffective to increase the stiffness of the wall to achieve less settlement in a certain stiffness range. Of greater importance is the use of an accurate static scheme and wall support levels. Considering the influence of the selection of the technology variant, it can be observed that the greatest differences in settlement occur for flexible walls (Variants A and B), which is obviously related to the influence of their stiffness. However, when a particular static scheme is selected, even technologies considered to be rigid (Variants C and D) can produce significantly different settlement results. The reason for this is that walls with relatively low stiffness (less than 100 kNm^2) are sufficient to transmit the bending moments.

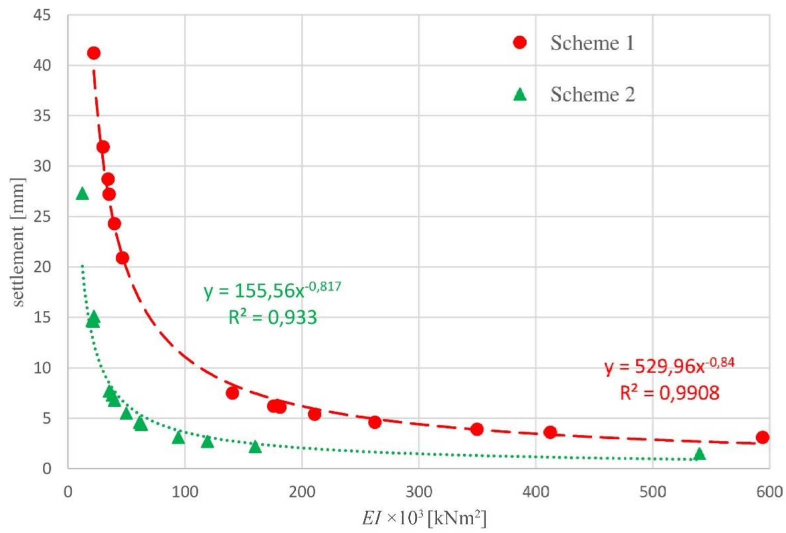

Fig. 6. Relation between the stiffness of the type of excavation wall (*EI*) and maximum ground settlement behind the excavation wall for Scheme 1 (red) and Scheme 2 (green)

Source: own work.

For all variants analysed, the maximum ground surface settlement (from Stage III) obtained for rigid technologies was between 3.1 mm and 7.5 mm in Scheme 1 and between 1.5 mm and 7.3 mm in Scheme 2. It is about from 0.0002*H* to 0.001*H*. For flexible technologies, it was between 20.9 mm and 41.2 mm in Scheme 1 (0.003*H*–0.006*H*) and between 14.6 mm and 27.3 mm in Scheme 2 (0.002*H*–0.004*H*). Thus, the use of an alternative variant and an adequate support scheme can reduce the predicted displacements several times. The values for rigid and some flexible solutions are in line with the data in the literature, but, in general, the use of flexible walls can lead to higher settlement than initially assumed. Therefore, special care must be taken when using this type of technology. As can be seen, the selection of the optimum protection method to limit settlement should take several aspects.

Analysing the effect of subsoil displacement on the adjacent building, similar conclusions can be reached. Here, however, the key element is the settlement of the foundations and its influence on the construction. Table 4 presents the final values of maximum settlement s_{max}, relative deflection $\Delta_{max,}$ rotation θ_{max}, and tilt ω_{max} calculated based on F1, F2 and F3 footing vertical displacements. The limits have been adopted in accordance with the National Annex of Eurocode 7 as 50 mm, 10 mm, 0.002, and 0.003, respectively. Bold indicates where the limit state for the parameter is exceeded. With this location of the footings, there is a clear tilt of the building towards the excavation.

Table 4. The summary of settlement in Stage III

No	Scheme 1				Scheme 2			
	s_{max} [mm]	Δ_{max} [mm]	θ_{max} [-]	ω_{max} [-]	s_{max} [mm]	Δ_{max} [mm]	θ_{max} [-]	ω_{max} [-]
A1	31.9	3.5	**0.0032**	0.0027	13.2	0.5	0.0012	0.0011
A2	24.3	2.7	**0.0025**	0.0020	6.3	0.0	0.0005	0.0005
A3	20.9	2.3	**0.0021**	0.0017	13.2	0.4	0.0012	0.0011
A4	27.2	3.0	**0.0028**	0.0023	7.1	0.1	0.0006	0.0006
B1	41.2	5.7	**0.0044**	**0.0034**	24.3	1.1	**0.0022**	0.0020
B2	28.7	3.1	**0.0029**	0.0024	13.5	0.5	0.0012	0.0011
C1a	5.4	0.6	0.0005	0.0005	4.2	0.0	0.0004	0.0004
C1b	3.6	0.4	0.0004	0.0003	2.5	0.0	0.0002	0.0002
C2a	6.1	0.7	0.0006	0.0005	5.1	0.0	0.0004	0.0004
C2b	3.9	0.4	0.0004	0.0003	2.9	0.0	0.0002	0.0006
C3a	7.5	0.9	0.0008	0.0006	6.7	0.1	0.0006	0.0006
C3b	4.6	0.5	0.0005	0.0004	4.1	0.1	0.0004	0.0003
D1	6.2	0.7	0.0006	0.0005	2.1	0.1	0.0002	0.0002
D2	3.1	0.5	0.0003	0.0003	1.4	0.0	0.0001	0.0001

Source: own work.

However, the most important factor in this case is the rotation. In Scheme 1, in all flexible wall variants (A and B), the serviceability limit state conditions were exceeded. For rigid ones (C and D), all conditions are met. On the contrary, for Scheme 2, the conditions were met for almost all wall types (except for soldier pile wall B1). This indicates that, with the right choice of support levels, even technology considered flexible can be sufficient to ensure serviceability conditions for neighbouring structures. Obviously, the settlement values would be different with a different footing location.

CONCLUSIONS

The selection of the most effective protection of a deep excavation wall for settlement limitation should consider several aspects, and each case should be analysed individually. The extensive case study presented confirmed the well-known dependence of settlement on the type of trench wall construction. However, some additional findings can be drawn from the results presented. The outcomes can be summarised as follows:

– The position of the head of the ground anchors and the staging of the excavation, i.e., the working pattern of the structure, are key factors affecting the maximum bending moments and the value of settlement and the subsoil settlement profile; positioning the upper anchorage level higher reduces the height of the wall in the cantilever stage and limits surface settlement, but moves the location of the point of maximum settlement further away from the excavation.
– The total influence range of the excavation for the analysed schemes is consistent with the literature data for sands, but the range of the greatest excavation influence on ground surface settlement was larger than expected regardless of the support scheme used; therefore, significant settlement is to be expected even away from the zone of direct influence of the excavation.

– The settlement occurring when using flexible walls (such as soldier pile walls or sheet pile walls) is generally greater than those for bored pile or diaphragm walls (rigid solutions), but application of adequate technology variants can reduce the settlement even for profiles with relatively low stiffness; however, special care must be taken when using low stiffness solutions, as settlement may be greater than indicated in the literature.
– The major parameter influencing settlement around an excavation is the stiffness of the excavation support wall used, with this influence being greatest at a stiffness (*EI*) of up to 100 kNm^2 per metre – at higher stiffnesses, the reduction in settlement is insignificant; thus, for flexible walls (and to some extent rigid walls), the main factor to consider should be the stiffness of the section and not other parameters.
– The serviceability limit state analysis of the structure adjacent to the excavation indicated that limit values were exceeded almost only for the rotation parameter and the flexible walls; however, even walls with low stiffness can be sufficient with an appropriate excavation support scheme.

Authors' contributions

Conceptualisation: M.M.; methodology: M.M. and M.T.; validation: M.M. and M.T.; formal analysis: M.M. and M.T.; investigation: M.M.; resources: M.M.; data curation: M.M.; writing – original draft preparation: M.T.; writing – review and editing: M.M. and M.T.; visualisation: M.M. and M.T.; supervision: M.T.; project administration: M.T.; funding acquisition: M.T.

All authors have read and agreed to the published version of the manuscript.

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WPŁYW KONSTRUKCJI OBUDOWY WYKOPU NA OSIADANIA ZABUDOWY SĄSIADUJĄCEJ

STRESZCZENIE

W pracy przedstawiono badanie osiadania podłoża gruntowego w sąsiedztwie głębokiego wykopu ze szczególnym uwzględnieniem metody zabezpieczenia ścian wykopu. Analizę przeprowadzono dla wybranego głębokiego wykopu, w sąsiedztwie którego znajduje się budynek mieszkalny. Rozważono najczęściej stosowane technologie, tj. ściankę szczelną, ściankę berlińską, palisadę oraz ścianę szczelinową. Łącznie przeanalizowano 14 wariantów obudowy, aby zbadać, jakie właściwości mają wpływ na otaczający ją obszar. Wpływ podparcia wykopu oceniono dla dwóch schematów poziomów zakotwienia z uwzględnieniem trzech faz wykopu. Na tej podstawie określono profil osiadania obok wykopu i obliczono osiadania sąsiedniego budynku. Zidentyfikowanym kluczowym czynnikiem był schemat podparcia ścian wykopu. Ponadto stwierdzono silną korelację między sztywnością obudowy a zaobserwowanymi maksymalnymi wartościami osiadania.

Słowa kluczowe: głęboki wykop, osiadanie, ściana szczelinowa, ścianka szczelna, ścianka berlińska, palisada