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## EXCAVATION SUPPORTED BY DIAPHRAGM WALLS – INCLINOMETRIC MONITORING AND NUMERICAL SIMULATIONS

### ZABEZPIECZENIE WYKOPU ŚCIANĄ SZCZELINOWĄ – MONITORING INKLINOMETRYCZNY I SYMULACJE NUMERYCZNE

#### Abstract

The results of the numerical simulation of deep excavation supported by a diaphragm walls are presented. Mohr-Coulomb and Hardening Soil models were used. The obtained horizontal displacements are compared with the results of the inclinometric monitoring and the observed discrepancies are discussed. Movement of the bottom of the wall – which is often neglected – is clearly observed both in the numerical simulations and in-situ measurements.

**Keywords:** diaphragm wall, FEM, inclinometric monitoring

#### Streszczenie

W artykule przedstawiono rezultaty symulacji numerycznych wykonywania wykopu zabezpieczonego ścianą szczelinową. Wykorzystano modele konstytutywne Mohra–Coulomba i Hardening Soil. Obliczone przemieszczenia poziome porównano z wynikami pomiarów inklinometrycznych. Omówiono zaobserwowane różnice. Zaobserwowano przemieszczenia dolnego punktu ścianki.

**Słowa kluczowe:** ściana szczelinowa, mur oporowy, MES, monitoring inklinometryczny

## Symbols

$\phi$	– angle of internal friction [deg]
$\gamma$	– soil bulk density [kN/m <sup>3</sup> ]
$c$	– cohesion [kPa]
$E$	– Young's modulus [MPa]
$E_{ur}$	– unloading/reloading modulus in standard drained triaxial test [MPa]
$E_{50}$	– secant modulus corresponding to 50% of the deviatoric stress at failure in standard drained triaxial test [MPa]
$E_{0ref}$	– initial stiffness [MPa]
$E_{oed}$	– oedometer loading modulus [MPa]
$k$	– Darcy's coefficient [m/d]
$\Delta X$	– total horizontal displacement [cm]
$\Delta X_r$	– relative horizontal displacement [cm]

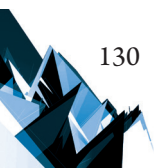
## 1. Introduction

Diaphragm walls are very often used to provide support during excavation. Due to the complex nature of soil-wall interactions (caused by the nonlinear behaviour of the soil, changes in the water level and the complicated phasing of the excavation execution) numerical simulations are often used in the design process of such structures. During the execution of excavations, inclinometric monitoring is used and the measured values of the horizontal displacements are compared with the calculated values [9]. Over recent years, the Mohr-Coulomb model has commonly been used to model the soil behaviour. Recently, more sophisticated constitutive models are often used (e.g. the Hardening Soil model, examples are provided in [8, 9] and [10]). According to [8], use of the Hardening Soil model results in a much better correlation between measured and calculated deflections of the wall than use of the Mohr-Coulomb model. This is due to descriptions of the nonlinear behaviour of soil (e.g. soil stiffness in different strain range) being closer to reality in the Hardening Soil model.

In this work, the results of the inclinometric monitoring of diaphragm walls are presented. Measured horizontal displacements are compared with results of numerical simulations (performed after excavation) – the possible reasons for discrepancies are discussed. Mohr-Coulomb and Hardening Soil models are used to model soil and differences in the obtained results are outlined.

## 2. Description of the analysed object

The analysis concerned the Alma Tower office and retail building located at Rondo Młyńskie in Kraków. The building has fourteen storeys plus a three-level underground carpark and is located in the immediate vicinity of Pilotów and Meissnera streets. The distance from the street is around 30 m. The nearest buildings are within 20 m (Fig. 1).



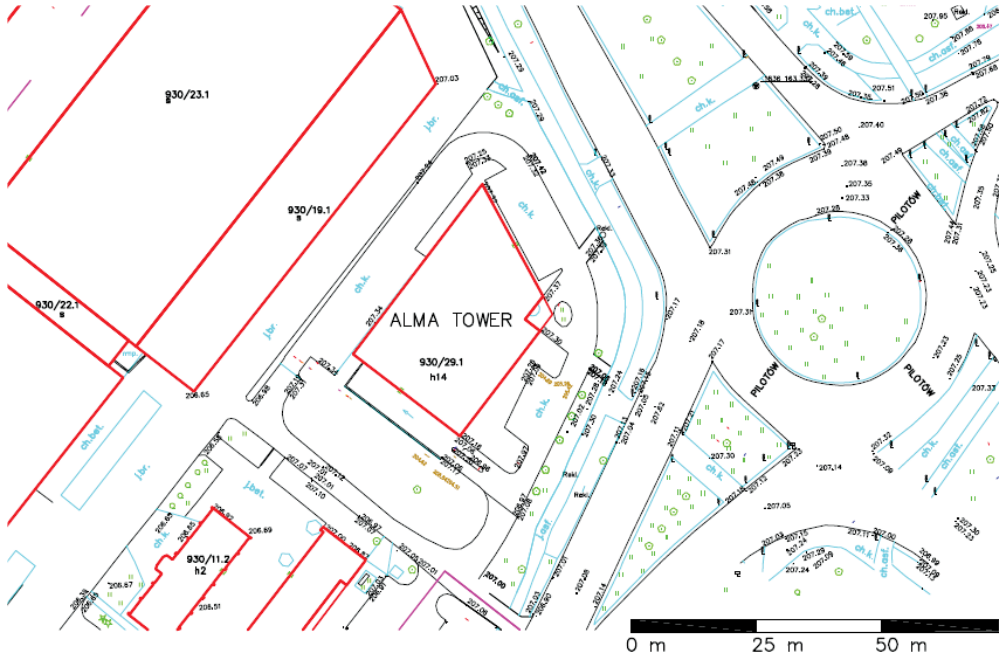


Fig. 1. Location map of the Alma Tower

Due to high groundwater level and the vicinity of property boundaries, Keller Polska proposed that the excavation be protected using the company's proprietary method. One of elements of this method is the construction of 60-cm-thick diaphragm walls. During the execution of the project, the walls constituted the retaining structure of the excavation, and were used as the foundation walls of the underground levels of the building.

Based on the static calculations performed using the GGU-Retain software during the preparation of the design documents, it was assumed that the rotational stability of the diaphragm walls at the excavation stage would be ensured by the floor slabs, and at the location of the planned ramp to the car park, by temporary steel bracing structures. The floor slabs bracing the diaphragm walls would be supported on temporary columns made of HEB 300 sections anchored in concrete piles (Figs. 2, 3).

Geotechnical investigation shows that in the place of the excavation top layer of the soil (about 1 m) consists of made soil. The next layer is organic silt (about 5.1 m, divided into two sub-layers (3.3 and 1.8 m thick), with slightly different strength and stiffness parameters), the third layer is sandy gravel (about 7.3 m). Clay layers are present under the bottom of the wall. The geotechnical conditions are presented in Fig. 3.

The excavations were performed as a staged construction and were started upon completion of the diaphragm walls and the reinforced-concrete capping beam. In the first stage of construction, the excavations were performed inside the diaphragm walls, which were acting as a retaining structure down to the first floor of the carpark. After the completion of the floor slab with a technical opening and the erection of the temporary bracing structure, the excavations were then performed using the top-down method.

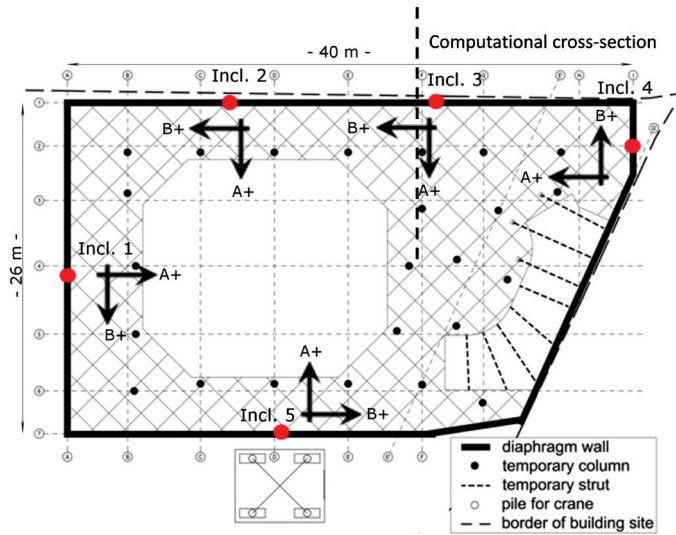


Fig. 2. Diagram of the excavation retaining structure with arrangement of inclinometer casings

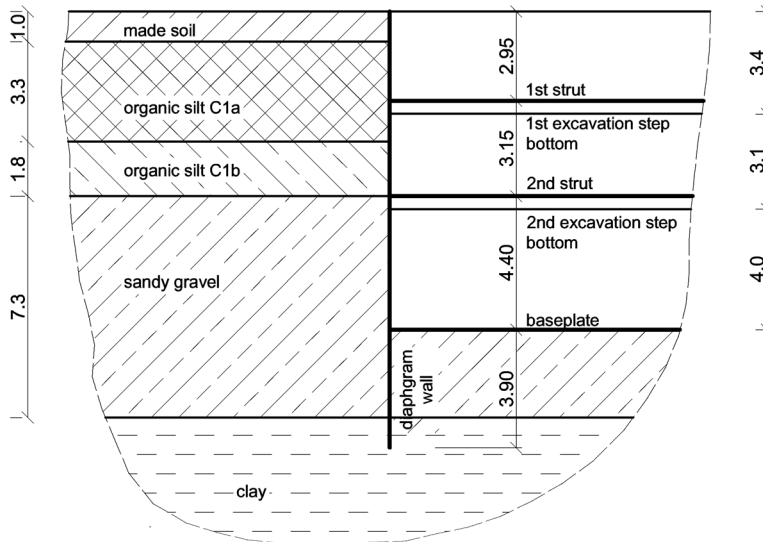


Fig. 3. Analysed cross section – geotechnical conditions and construction elements

During the excavations, the diaphragm wall was subject to detailed surveys and its deformations were monitored through measurements of inclinometer casings installed inside the walls. After the works were completed, it was found that the measured displacements of the diaphragm wall were lower than estimated at the design stage. This indicates that the assumptions adopted for the entire process related to the deep excavation were correct and that the planned works were correctly performed.

As part of that project, Keller Polska Sp. z o.o., based on its own detailed design, constructed a diaphragm walls with a thickness of 60 cm and a depth of up to 16 m. In addition, temporary bracing structures were also installed and temporary columns were erected for deep foundations for the tower crane using CFA piles. The works enabled the construction of further parts of the building inside a dry and safe excavation in accordance with the established work schedule. The final excavation depth was about 10.5 m, the groundwater level was observed at about 3 m below the terrain level.

### 3. Inclino-metric monitoring

The purpose of the inclinometric measurement was to determine the extent of the deformation of the diaphragm walls during the construction works involving the excavation inside the diaphragm walls for the three underground levels. As the earthworks progress, the diaphragm walls gradually take over the load caused by ground pressure, which, in turn, results in the displacement and deformation of these walls. These changes are monitored through the observation of inclinometer casings installed inside the walls. The measurements were made using an inclinometer manufactured by GLÖTZL for two positions of the probe in each casing. This enabled calculations of the inclinometer bias and measuring error, which did not exceed 0.8 mm. Measuring direction A+ for each casing was the direction towards the excavation, and direction B+ was the direction along the monitored wall. The inclinometer casings were installed by KELLER inside the diaphragm wall panels at depths of up to 15 m below ground level in the locations where the estimated deformation was the greatest. The arrangement and numbers of the casings are depicted in Fig. 2.

The inclinometer measurement started on 2012-12-09 with the '0' series, which was the recording of the initial geometry for each casing. The observations of that series were conducted two days before the commencement of excavation inside the diaphragm walls (2012-12-11). Changes to the geometric features of the diaphragm walls could be determined through the comparison of subsequent measurement series with the '0' series. Inclino-metric monitoring measurements were scheduled to take place at the end of each stage of construction. The first series was made after excavation for the first underground level (date of completion of the stage: 2012-12-14), the second series was performed after excavation for second level (2013-01-13) and the third, after excavation to the final depth (2012-02-13).

The readings of successive measurement series formed the basis for the determination of incremental displacement and cumulative displacements of inclinometer casings in the A+ A- and B+ B- planes. The depth of the probe was established using a local reference system from the top edge of the diaphragm wall.

The measurement results were processed under the assumption that the lowest point of the casing was stable [1] (Fig. 4a). The displacement of this point cannot be ruled out on the basis of inclinometer measurements alone. This movement can only be detected through independent measurements, e.g. surveys of the control points located on the capping



beam near the casing [2–4]. The inclinometer measurements suggest the occurrence of a displacement of the lower sections of the diaphragm walls. The measurements indicate a large displacement of the upper sections of the wall away from the excavation. Considering the distribution of the soil pressure on the wall, this is impossible. Therefore, these are only apparent displacements related to the displacement of the reference system of the inclinometer.

The stages of construction were accompanied by geodetic monitoring. This also involved observations relating to the total displacements of points located on the cap beam of the slurry wall. The measurements were conducted with a Topcon GPT-9003M total station using the project control network. In the land survey report, the accuracy of the measurements was specified as 2–3 mm. The measurement results transformed from the national coordinate system (2000') to A+ and B+ directions of the point installed near inclinometer 3 are given in Table 1.

Table 1. Survey observations

Date	X [m]	Y [m]	$\Delta X$ [m]	$\Delta Y$ [m]	$\Delta A+$ [m]	$\Delta B+$ [m]
2012-11-30	5549926.482	7426454.560	Survey initial measurement			
2012-12-11	5549926.489	7426454.562	0.007	0.002	-0.003	0.003
2012-12-14	5549926.480	7426454.566	-0.002	0.006	0.005	0.004
2013-01-13	5549926.477	7426454.569	-0.005	0.009	0.009	0.004
2013-03-03	5549926.474	7426454.568	-0.008	0.008	0.011	0.002

Unfortunately, the surveys were not carried out at the same time as inclinometer measurements. Therefore, in order to make use of these measurements, it was assumed that the absolute displacements during each period (between successive construction stages) changed steadily over time. The authors are aware that this assumption is flawed; however, it is considered that the error caused by this assumption does not exceed the surveying error (2–3 mm). With this assumption, the total displacement of the top point of inclinometer casing on a given day ( $D_c$ ) can be calculated as:

$$D_c = D_p + \Delta D \quad \Delta D = \frac{D_n - D_p}{l_{p-n}} \cdot l_{p-n} \quad (1)$$

where:

- $\Delta D$  – displacement correction resulting from the time difference between geodetic and inclinometric measurement;
- $D_p, D_n$  – absolute displacements in previous and next geodetic measurements, respectively;
- $l_{p-c}, l_{p-n}$  – the time period from the previous and next geodetic measurement with regard to a given day, respectively.

The displacements of the top and bottom point of the inclinometer casing have been determined (Table 2, Fig. 4b). The maximum correction  $\Delta D$  was 1.5 mm for the second series (2012-12-23).

Table 2. Displacement of inclinometer casing (A+, A- plane)

Date	Displacement of upper point [mm]	Displacement of lower point [mm]
2012-12-09	Inclinometer initial measurement	
2012-12-23	8.0	4.5
2013-01-18	11.0	8.7
2013-03-07	13.0	16.8

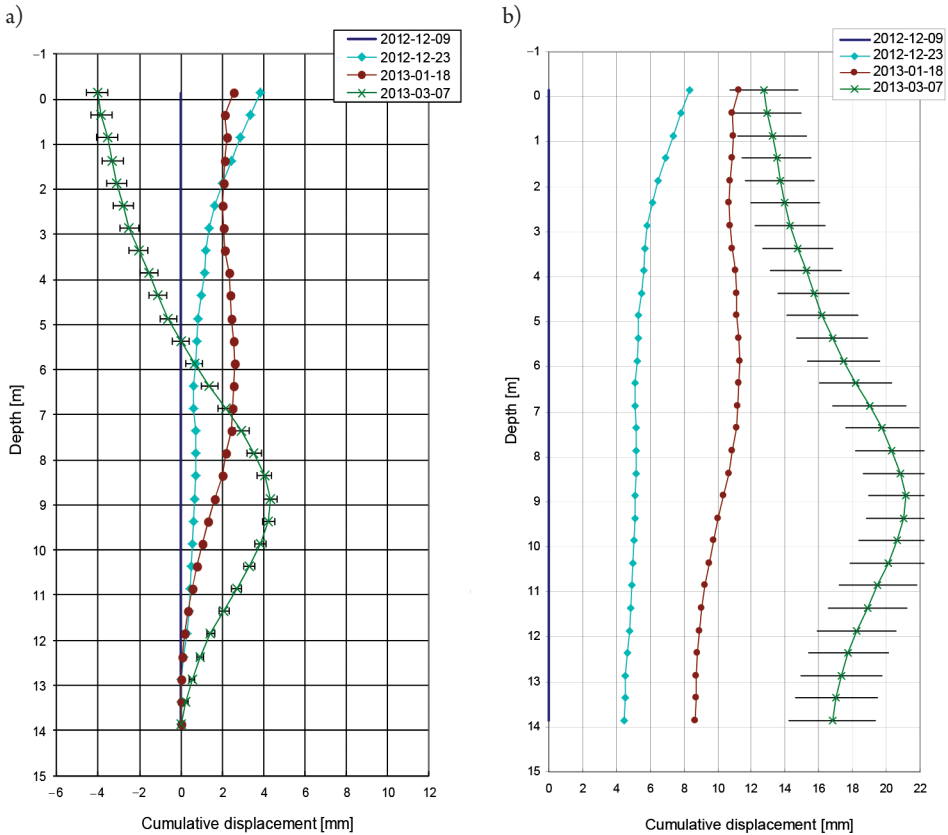


Fig. 4. Cumulative displacement of diaphragm wall with measurement errors (Inclinometer 3 A+A- plane): a) relative – with use of a local inclinometer reference system; b) total – based on geodetic observations

#### 4. Numerical simulations

Numerical simulations of the excavation were performed. The following initial assumptions were taken:

- ▶ plane strain conditions,
- ▶ Mohr-Coulomb elastic-plastic model for soil, with tensile ‘cut-off’ condition (no tension) or Hardening Soil Small Strain model,





- ▶ elastic model for the wall (beam elements), struts (truss elements) and baseplate (beam elements),
- ▶ contact elements between the wall and soil (with Coulomb friction law),
- ▶ stage construction algorithm with partial unloading (alternatively, final displacements (steady state of the seepage) or consolidation model),
- ▶ 2-phase model (soil + water).

All numerical simulations were performed with use of FEM system ZSoil v 12 (which is described in detail in [5] and [12]). A full description of methodology used can be found in [6]. Half of the F-F cross section was analysed; results from inclinometer 3 were compared with numerical simulations results.

The following construction stages were modelled:

- 1) initial state (no excavation),
- 2) installation of the wall,
- 3) excavation to a depth of 3.40 m (compared with measurement from 2012.12.23),
- 4) installation of the first strut at a depth of 2.95 m,
- 5) excavation to a depth of 6.50 m (compared with measurement from 2013.01.18),
- 6) installation of the second strut at a depth of 6.10 m,
- 7) excavation to the final depth of 10.5 m (compared with measurement from 2013.03.07),
- 8) installation of the bottom slab,
- 9) turning off of drainage.

In order to reproduce the changes in ground water level which are observed in reality during the simulation, the water level on the excavated side of the wall was gradually lowered to keep the free surface to around 0.5 m below the bottom of the excavation. On the retained side, the water level was constant at around 3 m below the surcharge.

The material parameters used in the simulation were taken from the existing geotechnical evidence (for Mohr-Coulomb model). The parameters for the Hardening Soil model were calibrated using simplified, empirical correlations, according to [11]:

$$E_{50} = E \quad (2)$$

$$E_{ur} = 3 \cdot E_{50} \text{ (for clay and organic silt)} \quad (3)$$

$$E_{ur} = 2.6 \cdot E_{50} \text{ (for sandy gravel)} \quad (4)$$

$$E_{oed} = 1.4 \cdot E_{50} \text{ (for organic silt)} \quad (5)$$

$$E_{oed} = 1.8 \cdot E_{50} \text{ (for clay)} \quad (6)$$

$$E_{oed} = 0.8 \cdot E_{50} \text{ (for sandy gravel)} \quad (7)$$

The values of  $E_{0ref}$  were estimated according to [7] and [11], with the use of the approximative relationship between 'static' and 'dynamic' soil stiffness.



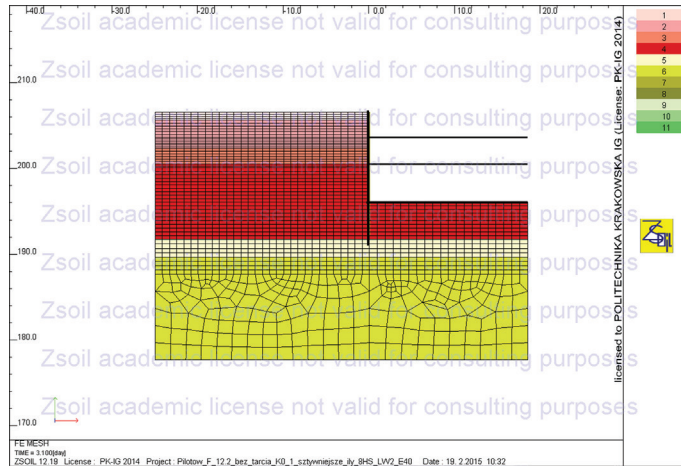


Fig. 5. Numerical model of the analysed object, step 8

Table 3. Material parameters used in analysis  
Mohr-Coulomb and elastic model (from existing geotechnical evidence)

Material	$E$ [MPa]	$k$ [m/d]	$g$ [kN/m <sup>3</sup> ]	$c$ [kPa]	$f$ [°]
uncontrolled embankment (made soil)	100	10	20.0	10	20
C1a (organic silt)	7.35	0.0003	18.2	18	9
C1b (organic silt)	5.2	0.0003	16.9	14	6
IIIb (sandy gravel)	250	25	20.5	0	41.5
clay	65	0.0001	20	60	13
concrete	31000	–	24	–	–

Hardening Soil model (strength parameters are the same as for the Coulomb-Mohr model)

Soil	$E_{ur}$ [MPa]	$E_{s0}$ [MPa]	$E_{0ref}$ [MPa]	$E_{oed}$ [MPa]
C1a (organic silt)	22	7.35	175	10.5
C1b (organic silt)	15.6	5.2	120	7.4
IIIb (sandy gravel)	645	250	1935	193
clay	66	22	462	40

## 5. Comparison of measured and calculated displacements

Numerical simulations show that initial assumptions of there being no horizontal displacements of the bottom of the wall (typically used in inclinometric monitoring) is not correct – the obtained values at the final stage of the excavation are about 2.5 cm for the Mohr-Coulomb model and about 0.2 cm for the HS model (Figs. 6, 7). Therefore, in the latter part of this work, the displacement of the bottom of the wall is subtracted from

the displacement of the every other point and such relative displacement is compared with the measured displacement.

Usage of the Mohr-Coulomb model leads to overestimation of the horizontal displacements of the wall. Usage of the HS model allows the obtaining of a strong correlation of calculated and measured displacements of the wall, up to the fifth construction step (however, displacements of the medium part of the wall are somewhat underestimated). The degree of correlation for further construction is worse in upper part of the wall – the numerical model underestimates the tendency of this part of the wall to move backwards (Figs. 6, 7). Use of a consolidation model (with a real-time schedule of excavation) does not lead to the obtaining of noticeable differences in horizontal displacements (vs. final displacements model with a steady seepage state). Both models have a tendency to overestimate the horizontal movement of the bottom part of the wall.

The main source of the obtained discrepancies is the quality of the geotechnical investigation. It is very common in Polish design practice to find values of the geotechnical parameters with use of correlation between the degree of compaction  $I_D$  or the liquidity index  $I_L$  and other parameters ( $c$ ,  $\phi$ ,  $E$ ). This method leads to the obtaining of reasonable results for shallow layers of soil; thus, it can be used in direct foundation problems. However, for deep layers of soil, it results in underestimating the soil stiffness and strength. Therefore, for complicated problems (such as deep excavation), such an approach could be used only as

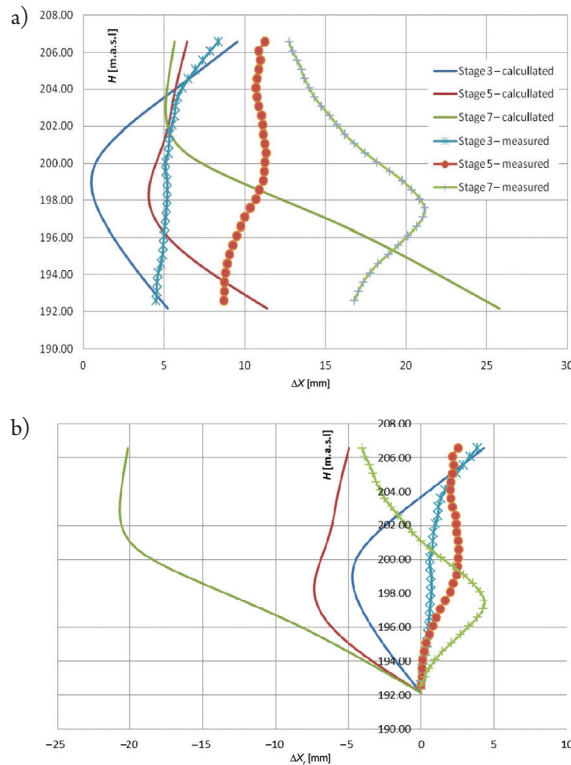


Fig. 6. Horizontal displacements of the wall – calculated (Mohr-Coulomb model) and measured a) total b) relative

an initial estimation of soil parameters. More sophisticated approaches (for example, triaxial tests) are recommended in order to obtain a better estimation of soil parameters and give the chance to improve the correlation between the calculated and measured displacements of the wall. Thus, usage of the Mohr–Coulomb model for estimation of the displacements of the wall is not recommended. Usage of the Hardening Soil model leads to the obtaining of results which are much more compatible with monitoring results, even if the quality of the geotechnical evidence is not so high.

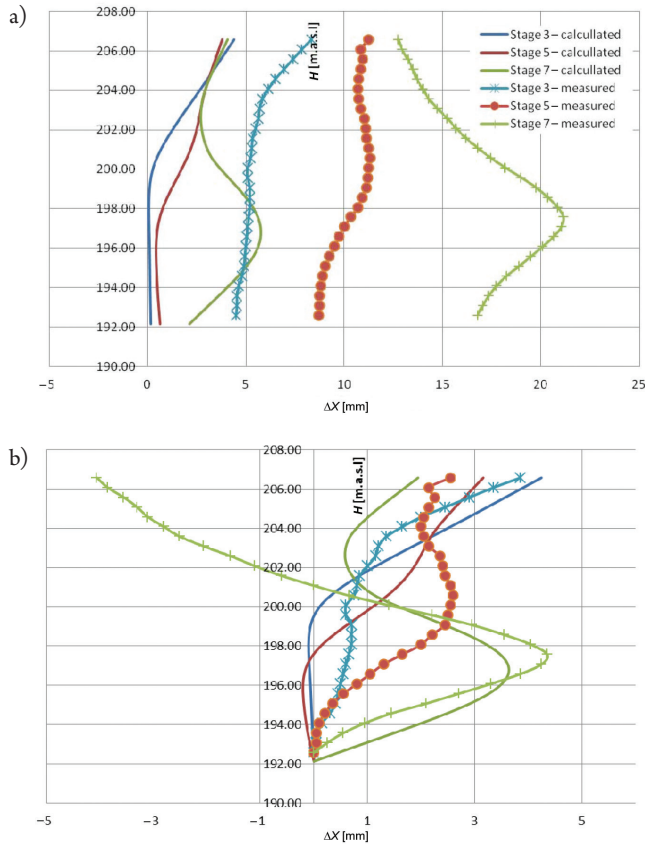


Fig. 7. Horizontal displacements of the wall – calculated (Hardening Soil model) and measured a) total b) relative

## 6. Final remarks

The obtained results indicate that the correct prediction of the diaphragm wall deformation might be a serious problem. Even the use of the sophisticated Hardening Soil Small Strain model for the soil does not provide results that are consistent with field monitoring deformation results. The uncertainty of soil parameters, the variable nature of the groundwater level and the estimation of external loads – which in reality may differ in



relation to the assumed in calculations – is the main source of the differences between the calculated and the measured displacements at the diaphragm wall. However, it should be noted that these differences are limited to a few millimetres, which in engineering practice is a good approximation of the actual work of the construction. The use of inclinometers to measure the movements of the walls of deep excavations provides the opportunity to review their deformation and allows comparison with the design goals at every stage of excavation work – this can guarantee the safety of conducting deep excavations.

## References

- [1] Wolski B, Toś C., *Geodezyjno-geotechniczna weryfikacja procedur interpretacji wyników pomiarów inklinometrycznych*, Zeszyty Naukowe Politechniki Rzeszowskiej nr 195, Budownictwo i inżynieria Środowiska z. 34, 2002, 54–63.
- [2] Wolski B., *Monitoring metrologiczny obiektów geotechnicznych*, Wydawnictwo Politechniki Krakowskiej, Kraków 2006.
- [3] Horodecki G.A., Bolt A.F., Dembicki E., *Przemieszczenia ścian szczelinowych stanowiących obudowy wykopów głębokich*, 49. Konferencja Naukowa KILiW PAN KN PZITB Krynica 2003, materiały konferencyjne, Warszawa–Krynica 2003, 33–40, [http://pg.gda.pl/~ghor/HBD\\_Krynica.pdf](http://pg.gda.pl/~ghor/HBD_Krynica.pdf) (access: 01.10.2015)
- [4] Woźniak M., Zaczek-Peplinska J., *Pomiary przemieszczeń budowli w warunkach głębokich wykopów*, [in:] *Teoretyczne podstawy budownictwa*, t. VI *Geodezyjne systemy pomiarowe*, Kulesza J., Wyczałek I. (ed.), Oficyna Wydawnicza Politechniki Warszawskiej, 2014, 9–18.
- [5] Podleś K., Truty A., Urbański A., *Analiza zagadnień geotechnicznych w systemie Z\_SOIL*, X Jubileuszowa Konferencja Naukowa, „Metody Numeryczne do Projektowania i Analizy Konstrukcji Hydrotechnicznych”, Korbielów 1998, 100–108.
- [6] Grodecki M., Truty A., Urbański A., *Modelowanie numeryczne ścian szczelnych*, Górnictwo i Geoinżynieria, Kwartalnik AGH, Rok 27, z. 3–4. 2003.
- [7] Alpan I., *The geotechnical properties of soils*, Earth-Science Reviews, 6:5-49, 1970, 36, 48.
- [8] Căpraru C., Chirică A., Numerical modelling and constitutive soil models for deep excavations analysis, SGEM2012 Conference Proceedings, June 17–23, Vol. 2, 2012, 215–222.
- [9] Likitlersuang S., Surarak C., Wanatowski D., Erwinoh E., *Finite element analysis of a deep excavation: A case study from the Bangkok MRT*, Soils and Foundations, 53(5), 2013, 756–773.
- [10] Obrzud R., *On the use of the Hardening Soil Small Strain model in geotechnical practice*, Proceedings of Numerics in Geotechnics and Structures 2010, Elsevier International 2010.
- [11] Obrzud R., *The Hardening Soil model. A practical guidebook*, ZACE Services Ltd., Lozanna 2014.
- [12] Z\_Soil.PC, *Theoretical Manual*, ZACE Services Ltd., Lozanna 2000.