Studia Geotechnica et Mechanica, Vol. XXVII, No. 1–2, 2005

THE DISPLACEMENTS OF ANCHORED DIAPHRAGM WALLS

ANNA SIEMIŃSKA-LEWANDOWSKA

Warsaw University of Technology, Institute of Roads and Bridges,
00-637 Warszawa, ul. Armii Ludowej 16, Poland.

Abstract: Due to the influence of the engineering works on buildings in the vicinity of deep excavations, two large Warsaw underground stations were constantly monitored. The results of measurements of the displacements of diaphragm walls are presented. Assessment of displacements was carried out on the basis of high-precision land surveying of fixed points positioned on diaphragm walls. The conclusions concerning maximum horizontal displacement of diaphragm walls as well as evaluation of real value of soils parameters based on the results of measurements have been presented.

1. INTRODUCTION

It was assumed that the cut-and-cover methods were to be used for construction of all of the stations and a part of the route tunnels of the Warsaw underground. For this reason the works were carried out in deep excavations. Two underground stations: A 13 (Centrum) and A 14 (Świętokrzyska) were built in the center of the city, in complex geotechnical conditions, in the area of compact settlement. The depths of excavations for both stations were 17.5 m and 14.5 m, respectively. In the course of the construction of both stations, the displacements of deep excavation walls and the adjacent areas were measured.

2. DESCRIPTION OF THE EXCAVATION OF STATION A13

There are the Quaternary soils in the area of the station. According to the report on geotechnical investigations, the following geotechnical layers were distinguished:

- layer I – uncontrolled fills, 1–3 m thick, in some places up to 4 m;
- layer II – moraine deposits reaching the depth of 8–11 m below ground level (bgl), consisting of medium and stiff sandy clays and clayey sands of the Warta glaciation and deeper stiff sandy clays of the Odra glaciation;
- layer III – continuous layer of sand deposits, 15 m thick, represented by fine and medium sands;
- layer IV – cohesive soils, 1–5 m thick;
- layer V – sand deposits (fine, medium and coarse sands as well as gravels and sandy gravels of 10–15 m thickness).

Two levels of ground water table were observed. Due to the way of carrying out the works and considering a temporary stability of excavation bottom, the water table was lowered during the works.

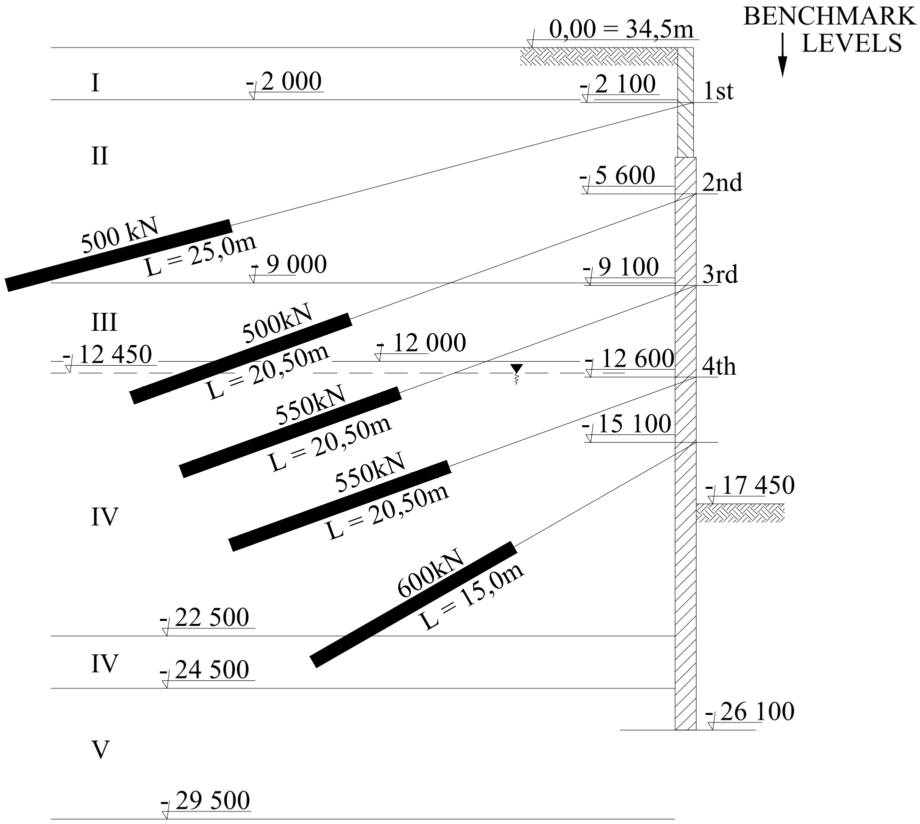

Fig. 1.The cross-section of the station A13

The lining of the station A13 excavation was made up of an anchored diaphragm wall joined in its upper part to a soldier pile wall [3]. The diaphragm wall is 0.80 m thick and 22 m high. The stability of the excavation walls was ensured by four or five levels of ground anchors. Schematic drawing of the structure as well as geotechnical conditions and location of the benchmarks is shown in figure 1.

3. DESCRIPTION OF THE EXCAVATION OF STATION A14

In vicinity of the station, there occur the Tertiary–Pliocene deposits disturbed by glacial tectonics.They are mainly cohesive deposits (clays, silty clays) including layers of saturated silty sands and fine sands.These layers do not have a horizontal position. The diaphragm walls are located in the clays. There are two levels of ground water:

the first, free water table, is 4 m bgl, the second, artesian water table, below the bottom of the station and it stabilizes about 7 m higher. The table of ground water was lowered by pumping. Its level was maintained about 0.50 m below the bottom of the excavation.

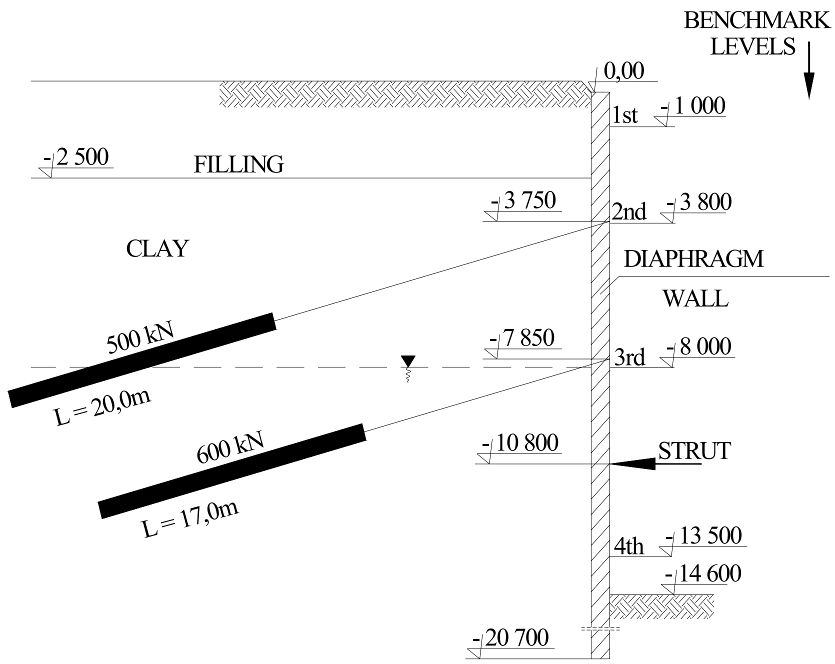

Fig. 2. The cross-section of the station A14

In order to protect the excavation, diaphragm walls (0.80 m thick and 20.7 m deep) were designed [4]. Two levels of ground anchors and one row of steel struts ensured the stability of the walls. The cross-section of the excavation wall and geotechnical conditions are presented in figure 2.

4. MEASUREMENTS

At the station A13 monitoring of the diaphragm wall was carried out on all levels of anchoring in 25 vertical lines spaced about 20 m along the excavation. The measuring points identified by special benchmarks were located on supports made of angle sections and fixed near the anchors heads. Figure 3 presents location of vertical measurement lines on a plan view of the station, and figure 1 shows location of the measuring points in a cross-section of the excavation. A time schedule of the monitoring matched the assumed sequence of works.

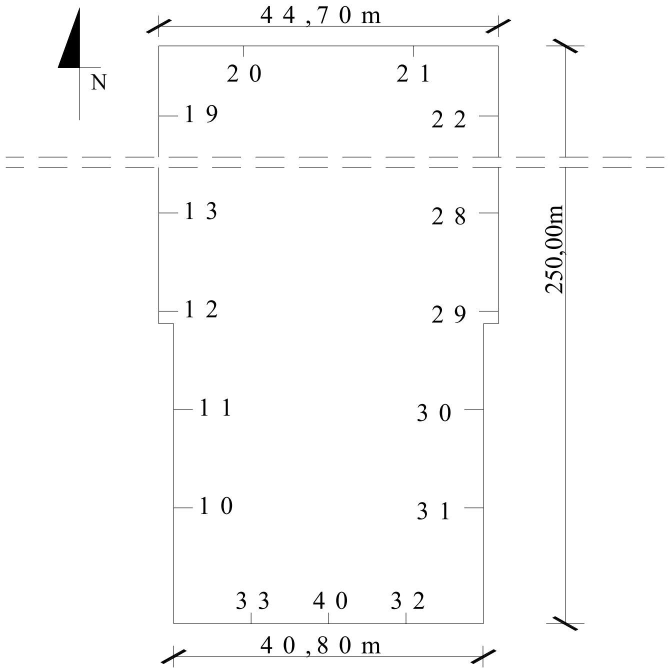

Fig. 3. Sketch of vertical lines and measuring points of the station A13

The method of measurements was designed in such a way as to obtain a full picture of the displacements of the excavation walls in a period of observation (9 months). A great depth of excavation (in the deepest 17.45 m zone) implicated occurrence of the considerable values of horizontal displacements of the diaphragm wall. Monitoring and continuous analysis of the results of the measurements allowed an investor and a contractor to control permanently the stability of the walls as well as to supervise the stress and reliability tests of ground anchors in exploitation ensuring that their capacity is in agreement with that of the design.

At the station A14 the program of the monitoring of the wall and the surface of the ground included:

• Measurements of the horizontal displacements of diaphragm walls in 9 (I–IX) vertical cross-sections at the depths of 1.0 m, 3.8 m, 8.0 m and 13.5 m bgl (below ground level). The benchmarks, like at the station A13, were installed on angle sections mounted in the wall.

• Measurements of the displacements of the surface of the ground – benchmarks (P1–P6) installed on eastern and western sides of excavation in two cross-sections investigated at the distance of 1 m, 7 m and 14 m from the excavation wall.

• Measurements of heaving of the excavation bottom on the basis of measurements carried out on the deep benchmarks (P_{d1}, P_{d2}).

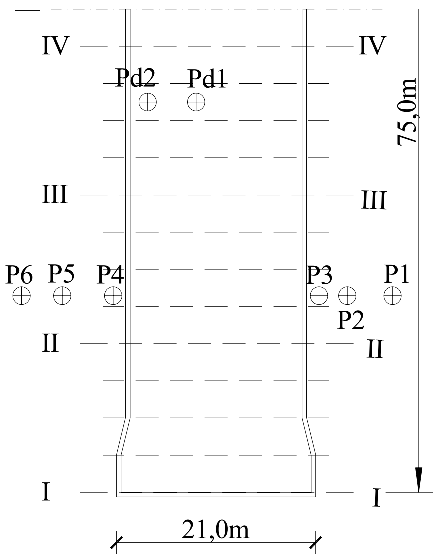

Fig. 4. Sketch of vertical lines and measuring points of the station A14

Figure 4 presents a sketch of vertical lines on the plan of station and location of the measuring points in the cross-section of excavation in figure 2. The time schedule of measurements matched the respective stages of the excavation making.

5. RESULTS OF MEASUREMENTS

At the station A13 the maximum values of horizontal displacements were measured on the first measuring level and they were + 0.014 m at the depth of excavation of 13.6 m and + 0.015 m at the depth of 17.5 m, which constituted 0.103% and 0.09% of its height.

At the station A14 the maximum vertical displacements of the anchored diaphragm wall appeared on the first measuring level reaching the value of + 0.016 m, which was 0.164% of excavation depth, that is 14.6 m (figure 5). A total value of the vertical displacements of excavation bottom measured in the middle of its width and caused by relaxation is + 0.060 m (figure 6). In the geotechnical conditions of the Pliocene clays, the maximum value of the relaxation of the ground surface was + 0.0075 mm at the distance of 14 m from the excavation edge [3].

Cross-section II
Horizontal displacements of walls

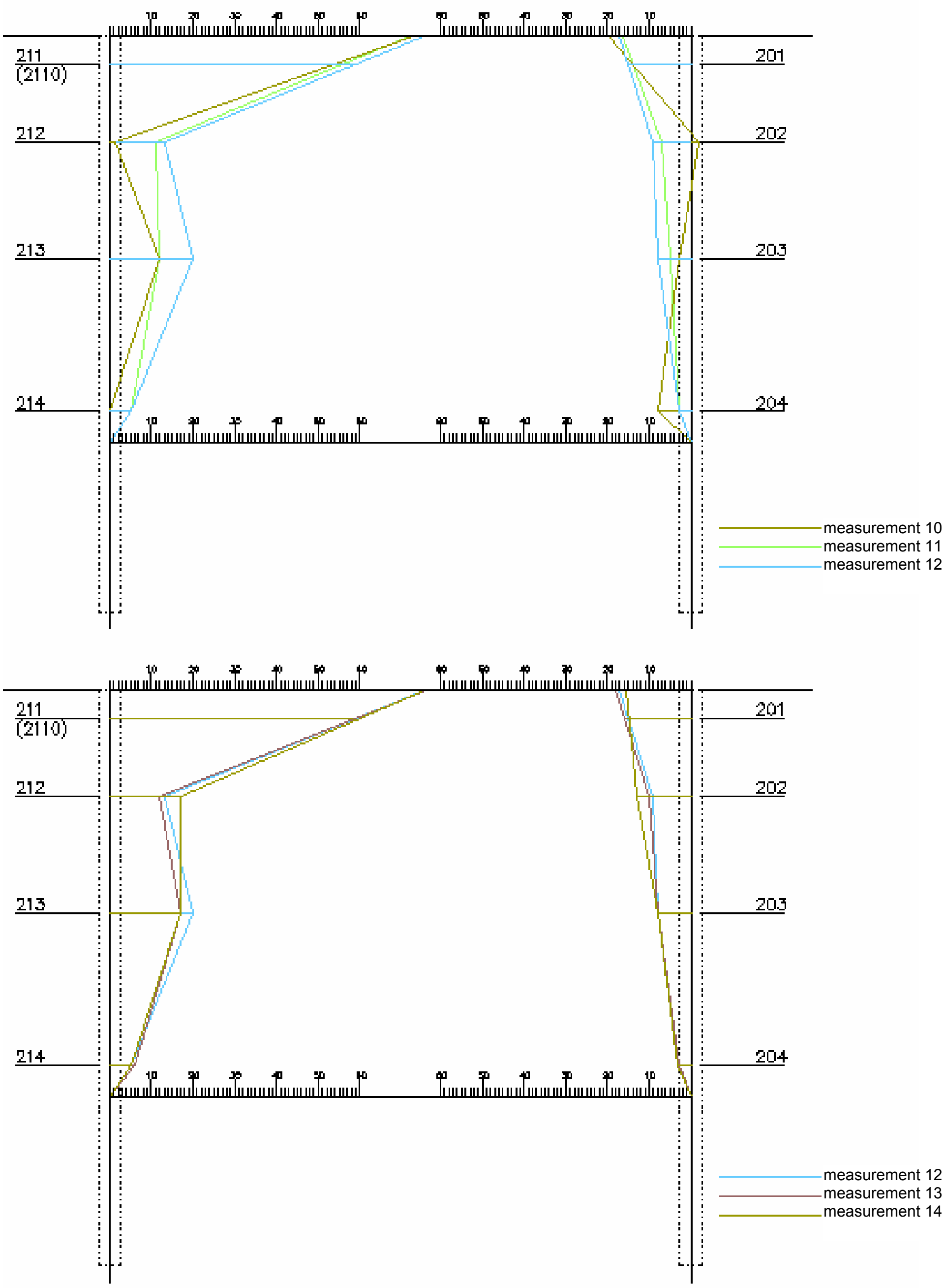

Fig. 5. Horizontal displacements of diaphragm wall at the station A14

benchmarks P1–P6, vertical displacements of surcharge
benchmarks Pd1, Pd2, vertical displacements of bottom excavation
cross-section II, vertical displacements of walls

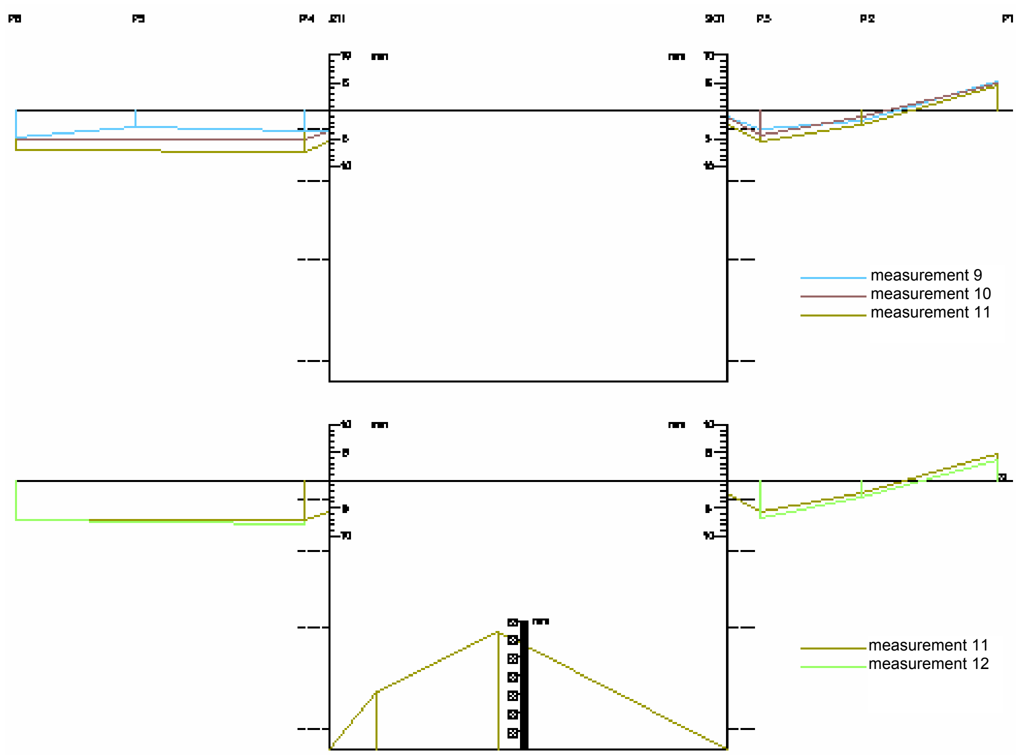

Fig. 6. Vertical displacements of bottom of excavation

6. BACK ANALYSIS

The objective of theoretical considerations presented below was to calibrate an elastic–perfectly plastic soil model with Coulomb–Mohr's yield criterion by means of back analysis based on the test measurements of the displacements of anchored diaphragm wall. The model parameters for a given soil are never constant but rather depend on the tension or the deformation history, whose mutual dependence is random by nature. They can be optimally estimated by an attempt to match the results of theoretical predictions with those of experimental test using various criteria. A specific form of the criterion depends on the calibration procedure and the definition of its state variables. As reported by GRYCZMAŃSKI [1] in local calibration, the best solution seems to assume the expected sum J of square of variance of the values of soil load response measured in laboratory based on those calculated from the calibrated constitutive model equations. This criterion was given by Gryczmański in the form of equation:

$$J_s = \sum_{i=1}^{N} \{ C_{vi} [\hat{\varepsilon}_{vi} - \varepsilon(p'_i, q_i, \theta_i; A_1, ..., A_n)]^2 + C_{si} [\hat{\varepsilon}_{si} - \varepsilon(p'_i, q_i, \theta_i; A_1, ..., A_n]^2$$
$$+ C_{\theta i} [\hat{\varepsilon}_{\vartheta i} - \varepsilon_{\vartheta}(p'_i, q_i, \theta_i; A_1, ..., A_n)]^2 \} = \min, \tag{1}$$

where:

N – the number of measurements,

$C_{vi}, C_{si}, C_{\vartheta i}$, C_{vi} – the weight coefficients,

$\hat{\varepsilon}_{vi}, \hat{\varepsilon}_{si}, \hat{\varepsilon}_{\vartheta i}$ – the measurement results for $p' = p'_i$, $q = q_i$, $\theta = \theta_i$,

$A_1, ..., A_n$ – the set of estimated model parameters.

In the case of constitutive models that relate current effective tension states to the deformation, the regression functions $\varepsilon_v, \varepsilon_s, \varepsilon_\vartheta$ are given in analytical form. The values of ε_{vi}, ε_{si}, $\varepsilon_{\theta i}$ are obtained by gradual solving the calibrated model equation given by a specific tension path. Condition (1) is therefore modified by GRYCZMAŃSKI [1] to the form:

$$J_\varepsilon = \sum_{i=1}^{N} [C_{vi}(\hat{\varepsilon}_{vi} - \varepsilon_{vi})^2 + C_{si}(\hat{\varepsilon}_{si} - \varepsilon_{si})^2 + C_{\theta i}(\hat{\varepsilon}_{\vartheta i} - \varepsilon_{\vartheta i})^2 = \min . \tag{2}$$

In such a criterion, the values of ε_{vi}, ε_{si}, $\varepsilon_{\theta i}$ are obtained numerically at the measuring points *i*. At each step of the optimization, the values of model parameters are determined and introduced to the numerical analysis (constitutive equations). Then ε_{vi}, ε_{si}, $\varepsilon_{\theta i}$ are calculated and J_ε is determined in order to minimize its value. In practice, this procedure in geotechnical engineering employs “simplex” method. Because of the specificity of the test results and the importance of the representative measurements of the horizontal and vertical diaphragm wall displacements in each phase of excavation deepening, the method of simplified back analysis has been used. The reality is described by the values of horizontal displacement of the wall in a chosen measuring section as a response not to a load path but rather to one of the tension states in the soil and the construction structure. Equation (2) is simplified to the form:

$$J_\varepsilon = \sum_{i=1}^{N} [y_i - \hat{y}_i(E, \nu, \phi, c)]^2 = \min, \tag{3}$$

where:

y_i – the measuring results of horizontal displacements of anchored diaphragm wall in the point *i*,

$\hat{y}_i$ – theoretical horizontal wall displacements in the point *i* obtained from the calibrated model.

$\hat{y}_i$ is a function of the above-mentioned model parameters (E, ν, ϕ, c), provided that Coulomb–Mohr's yield criterion is used. Such an approach to a global calibration of the soil model assumed and the rules of the back analysis is due to the shortcomings and errors of an available measuring database that has been created from experimental analysis of anchored diaphragm wall. This allows a holistic description of the reality for all load paths described.

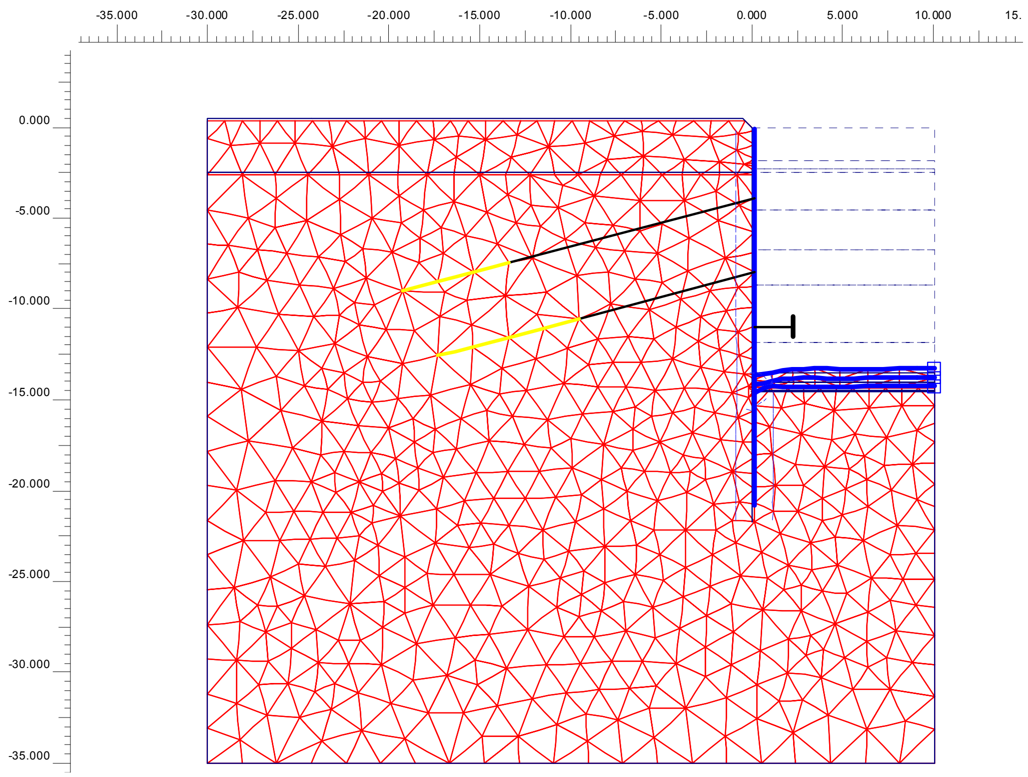

Fig. 7. FEM mesh of the excavation A14

The calculations have been performed using finite element method, applying a computer program that takes into account an elastic–plastic analysis of bi-phases medium, assuming plane strain. The elastic–perfectly plastic model for ground soil with Coulomb–Mohr's yield criterion is used for soil deprived of isotropic or kinematic reinforcement. The assumption of unassociated rule of flow is accepted. The isotropic linear elastic medium model for the reinforced concrete structure (diaphragm wall, bottom plate) of ground anchors and struts is used. Axial symmetry over excavation center is assumed. The mesh has been created from isoparametric six- and fifteen-node triangle elements of the "interface" type. The soil, the diaphragm wall and the reinforced concrete construction have been modelled using 2D isoparametric massive elements. The anchors and the struts have been modelled by 2D isoparametric linear elements assuming their proper stiffness. Contact elements (interfaces) of zero thickness are provided to model a contact of the construction structure with the soil. This allows us to model sticking and slipping phenomena in accordance with Coulomb–Mohr's yield criterion. The parameters chosen for the contact elements are in agreement with the parameters of the adjacent soils.

Table 1

Horizontal displacements of diaphragm wall at station A14
Zn – damaged benchmark; zd – disassembled benchmark; * – measurements of additional benchmark; sign + means displacements towards the excavation

	Point No.	Mrmt 1	Mrmt 2	Mrmt 3	Mrmt 4	Mrmt 5	Mrmt 6	Mrmt 7	Mrmt 8	Mrmt 9	Mrmt 10	Mrmt 11	Mrmt 12	Mrmt 13	Mrmt 14
Data		23.06	06.07	13.07	17.07	20.07	30.07	06.08	14.08	27.08	03.09	10.09	17.09	24.09	02.10
II/W	201	0	1	12	5	5	4	10	12	14	14	14	15	16	15
	202	0	0	0	1	–1	0	0	0	–2	–2	7	9	10	13
	203	0	0	0	0	0	1	1	–1	–5	3	5	8	Zd	Zd
	204							0	0	0	8	3	Zn	Zd	Zd

The reality (y_i – in formula (3)) is here represented by horizontal wall displacements in the point i. The measuring data from section II (shown in figure 5) were taken as the basis for the back analysis. The results of measurements in section II in the consecutive phases of excavation deepening and construction building are given in table 1. The data from the 11th displacement measurement of points 201, 202, 203 have been taken as the basis for the back analysis, which corresponds to the strut disassembly phase after completion of the bottom plate. A full picture of the excavation wall displacements has been responsible for this choice. Discrete presentation of the territory in this phase is shown in figure 7. The theoretical ($\hat{y}_i$ – in formula (3)) horizontal wall displacements of points 201, 202, 203 in phase X, determined from the calibrated model, have been calculated for the variable values of the soil parameters. The back analysis has been performed by step-by-step method. Formula (3) is written in the following equivalent form:

$$J_\varepsilon = \sum_{i=1}^{N} | y_i - \hat{y}_i(E, \nu, \phi, c) | = \varepsilon = \min . \tag{4}$$

The values for the soil parameters are taken from the documentation of BP Metroprojekt [4]. For the surface layer, constant parameters that do not change in the subsequent phases of calculations are assumed. For classified layer II – tight clay soil –

Pliocene clay (Pl), the values for ϕ and c are assumed to be in the range between the generalized values (ϕ_u, c_u) determined without outflow of water and the effective values (ϕ', c'). For the back analysis an assumption has been made that determination of the minimum value of J_ε will be made for the increasing values of elasticity modulus E_0 of clay (from 24 MPa) and a combination of ϕ with c. The value of E_0 = 24 MPa has been determined from the geotechnical documentation [5]. The value of the coefficient K_0 has been determined from the Jaky formula. It has been assumed that it is constant for the entire wall height. The summary of the parameters assumed for the calculations is given in table 2.

Table 2

The values of soil parameters assumed for the calculations of the excavation wall at station A14

Soil type	Γ	Φ_u	Φ'	C_u	C'	E_0
	kN/m^3	°	°	kPa	kPa	Mpa
NN	17.00	25	–	0	–	10
I		13	19	37	20	24

In the calculations, the following combinations of ϕ with c have been taken:

ϕ = 13°, c = 37 kPa; ϕ = 16°, c = 37 kPa; ϕ = 19°, c = 37 kPa;

ϕ = 13°, c = 20 kPa; ϕ = 16°, c = 20 kPa; ϕ = 19°, c = 20 kPa.

For each combination of ϕ with c, the calculation of $\hat{y}_i$ in points 201, 202, 203 has been performed in X phase of construction with the clay elasticity modulus E_0 taking the following values: 24, 50, 65, 75, 80, 85, 90, 95, 100, 105, 110,115, 120, 125, 130, 140,150,160,170,180, 200,220, 240, 260, 280 MPa.

In order to determine the minimum values of sum of moduli of the difference between the actual and the theoretical wall displacements in the three points under consideration, for the above described combinations of E_0, ϕ and c, more than 1000 calculations have been performed [2]. The evaluation of the results has been done in two steps: independently for each single measuring point ($|y_i - \hat{y}_i|$) and globally for the three points simultaneously

$$\sum_{i=1}^{3} |y_i - \hat{y}_i| .$$

In the case of the back analysis performed independently for each single measuring point, identical values y_i and $\hat{y}_i$ have been obtained for 9 combinations of ϕ, c and E.

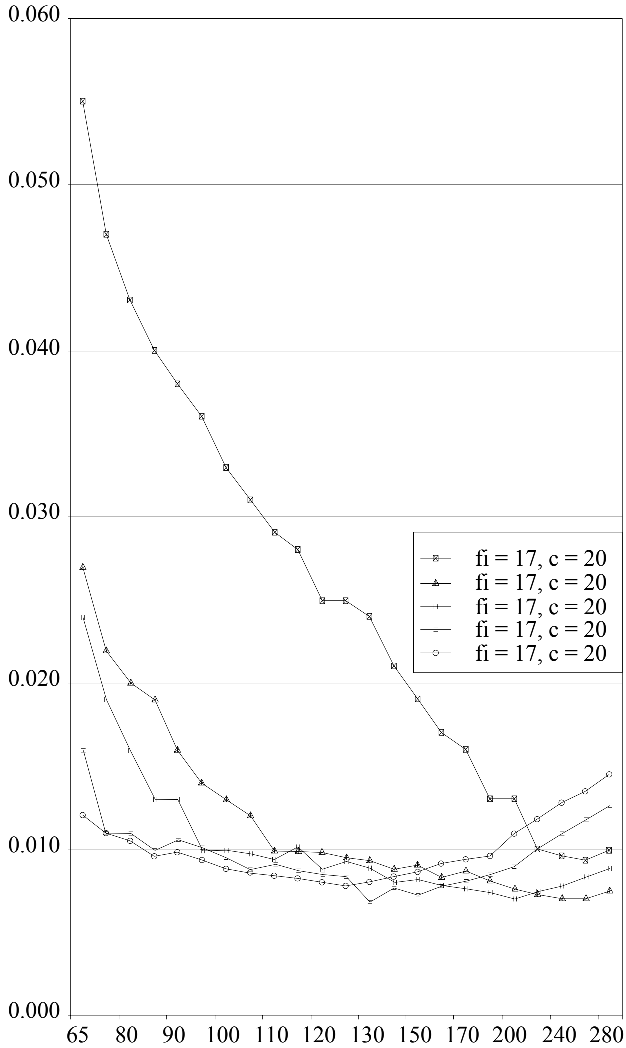

Fig. 8. The results of calculations done based on back analysis

Evaluating the results of the back analysis of the three point displacements it can be stated that such a combination of ϕ, c and E for which $\varepsilon = 0$ has not been obtained. The minimum value of

$$\sum_{i=1}^{3} | y_i - \hat{y}_i |$$

has been obtained for the above-described six combinations of ϕ with c and the elasticity modulus value ranging from 125 MPa to 260 MPa. For the further analysis of the results we assume that for a model parameter estimation such a combination of ϕ, c and E is reliable at which $\varepsilon \leq 0.01$. The combination of $\phi = 13°$ with $c = 20$ kPa has been disregarded since it does not fulfil the criteria assumed. The analysis results of the remaining five combinations are presented in figure 8. It can be stated that the value of the elasticity modulus, determined from the results of the back analysis, for which the criterion $\varepsilon \leq 0.01$ is fulfilled, ranges from 85 to 110 MPa. For the value of elasticity modulus determined from the geotechnical documentation ($E = 24$ Mpa), the value of $\varepsilon \in \langle 0{,}056{,}0{,}318 \rangle$ and it does not fall below the assumed value of 0.01. This means that the theoretical horizontal wall displacements for this value of the modulus significantly differ from the actual.

7. CONCLUSIONS

The maximum actual horizontal displacements of an anchored diaphragm wall appear on the top of the wall, in the deepest excavation section and their value does not exceed 0.2% of H_W (the greatest depth of excavation). The shape of the displacements has a cantilever-like character, which means that they decrease with depth. This conclusion was based on the analysis of the measurement results of horizontal displacements of anchored diaphragm walls. The measurements were carried out in 34 vertical lines for four measurement levels during 19 months. These walls were properly designed and the anchors ensured their stability achieving the capacity required.

The elastic-perfectly plastic soil model with Coulomb–Mohr's yield criterion assumed in the back analysis of the diaphragm wall of station A14 allows an appropriate evaluation of the anchored construction displacements. The strongest agreement between theoretical and actual results of the wall displacements ($\varepsilon \leq 0.01$) has been obtained for the Pliocene clay elasticity modulus $E_0 \in \langle 85{,}110 \rangle$ MPa [2].

REFERENCES

[1] GRYCZMAŃSKI M., *On calibration of constitutive models* (in Polish), Zeszyty Naukowe Politechniki Śląskiej, Budownictwo, 1995, Nr 80, pp. 37–52.
[2] SIEMIŃSKA-LEWANDOWSKA A., *Displacements of anchored diaphragm walls* (in Polish), Oficyna Wydawnicza Politechniki Warszawskiej, Prace Naukowe, Budownictwo, 2001, Vol. 139.
[3] SIEMIŃSKA-LEWANDOWSKA A., GRZEGORZEWICZ K., *Analysis of displacements of anchored diaphragm walls of the Centrum and Świętokrzyska stations* (in Polish), Proc. Conference on Underground Constructions, AGH, Kraków, 2000, pp. 456–464.
[4] B.P. METROPROJEKT, *I linia metra w Warszawie. Projekt techniczny korpusu stacji A13*.
[5] B.P. METROPROJEKT, *I linia metra w Warszawie. Projekt techniczny stacji A14, Konstrukcja i metody budowy korpusu.*