Static analysis of restrained sheet-pile walls

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ABSTRACT:

The results of displacement and deformation measurements of the restrained sheet-pile walls designed as an earth-retention system for a railway embankment near the new motorway viaduct that was being under construction are described. The monitoring included inclinometer measurement of the wall deformation, geodetic observation of the horizontal displacement of the top of the wall crowns, and train roadbed settlement. The measurement results indicate interdependence between earth pressure and displacement of the retaining structure. Quantitative rates are presented referring to the Polish Standard Code (PN-83/B-03010), Eurocode 7 (PN-EN 1997-1: 2004) and selected literature (Weissenbach, 1975). The influence of pile driving on static behaviour of the sheet-pile wall system is taken into consideration in the final analysis.

SITE DESCRIPTION

- The WA81 viaduct near Grudziądz as a part of the motorway A1 traffic.
- Two parallel walls, 52 m long each, installed as shown in Fig. 2.
- The depth of excavation was 5.0 m adjacent to support P2 and 3.9 m adjacent to support P3.
- Sheet-pile heights 9.5 m and 7.0 m, respectively.
- Retaining structure restrained by one level of steel tie-rods running through the embankment (see Fig. 2).
- Pile driving for the new viaduct substructure. Two supports within the embankment slopes (see Fig. 3).
- Soils in- and under the railway embankment predominantly sands (see Fig. 3).

Fig. 1 View of the sheet-pile wall at the support P2.

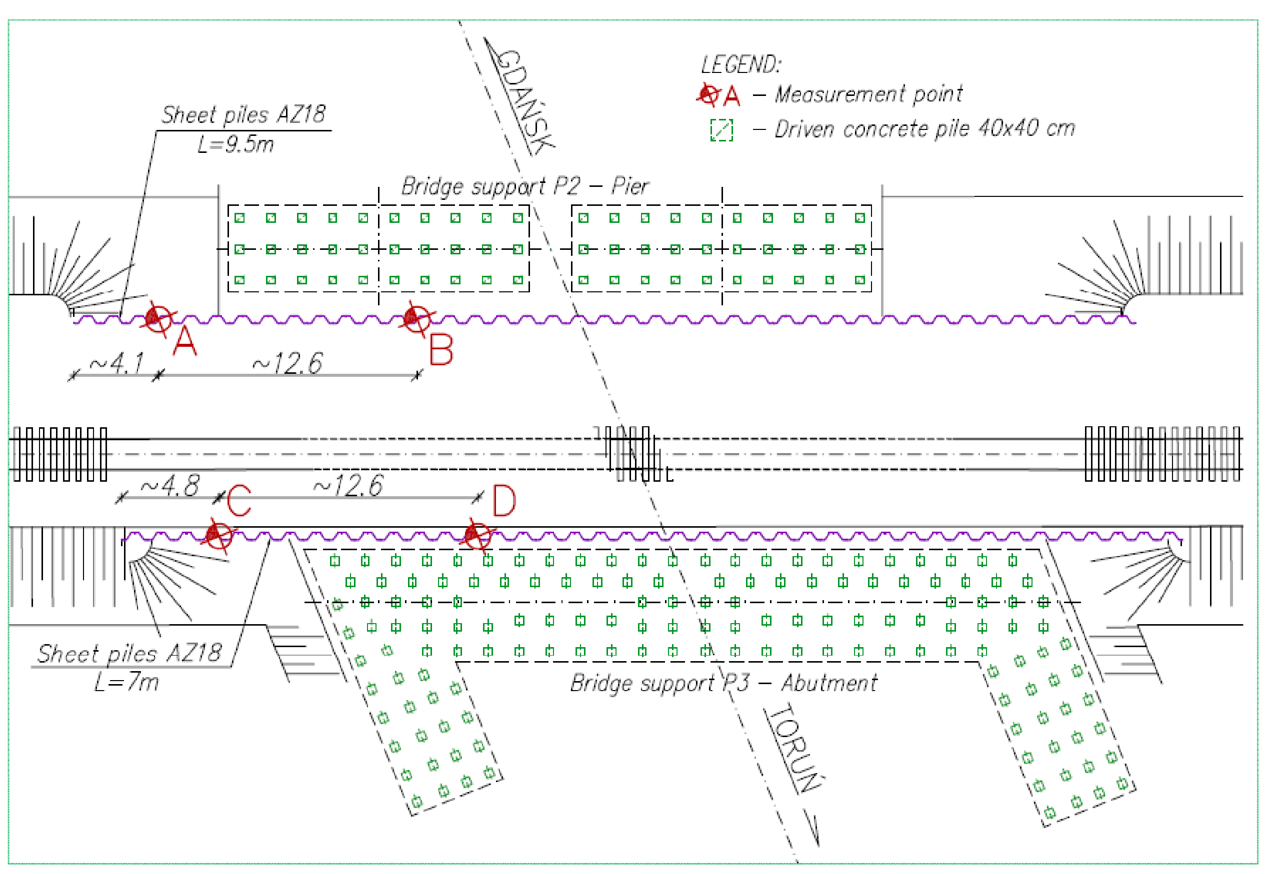

Fig. 2 Layout of the retained rail-trunk-line and the designed viaduct foundations

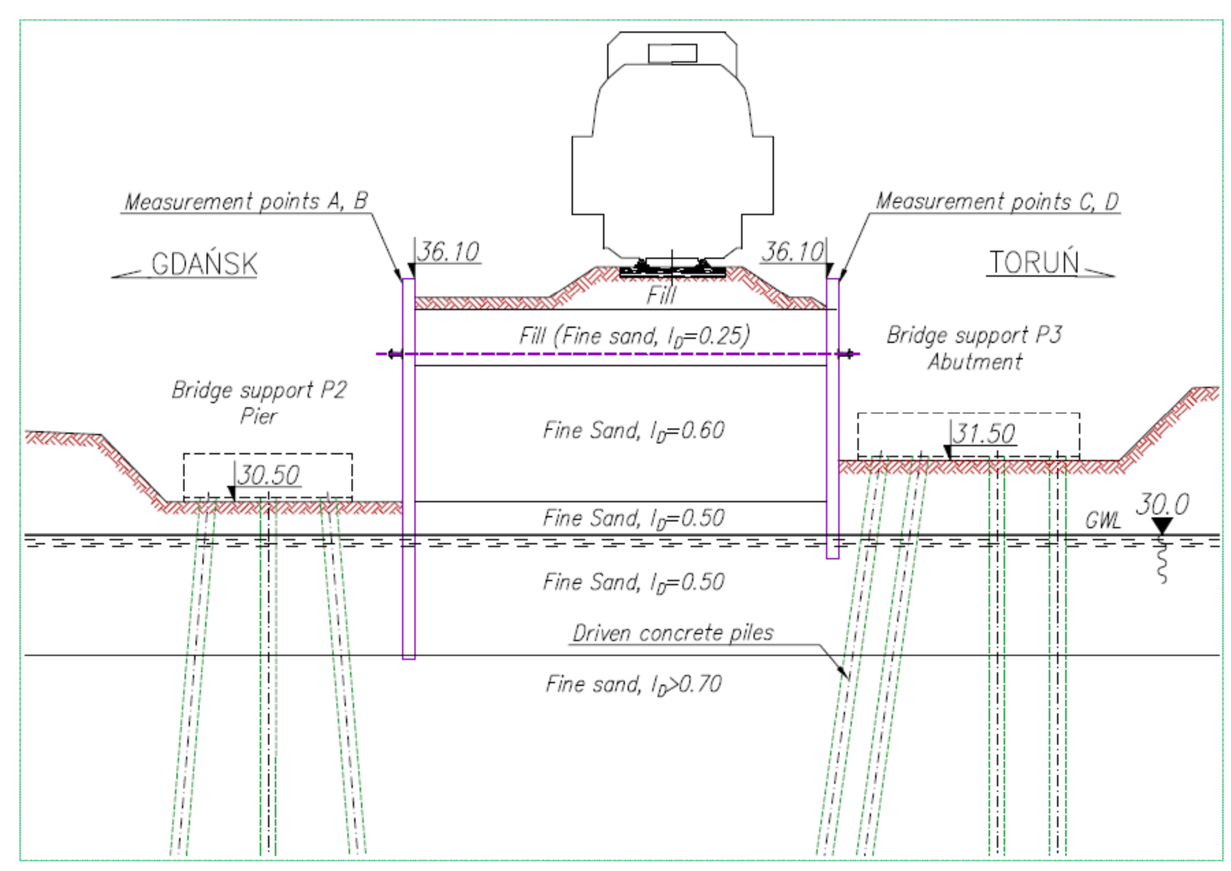

Fig. 3 Embankment retained by sheet-pile walls (cross section after piling completion)

MEASUREMENTS

Table 1. Description of measurement phases

Measure phase	Date	Work state in 2007
0-g	05-09	No excavation
1-g,i	05-28	Working platform for tie-rods
2-g,i	06-06	Post-tensioning of steel bars
3-g,i	06-18	Full excavation
4-g,i	08-16	After piling for support P3
5-g,i	09-05	After piling for support P2
Type of measurement: g-geodetic, i-inclinometer		

ANALYSIS OF THE RESULTS

- Displacement of the retaining wall needed to develop limit states of earth pressure.
- Determining loading states for sheet-pile walls in individual phases of foundation and piling installation.
- Determination of forces in the post-tensioned tie-rods based on wall crown displacements and structure deformation.

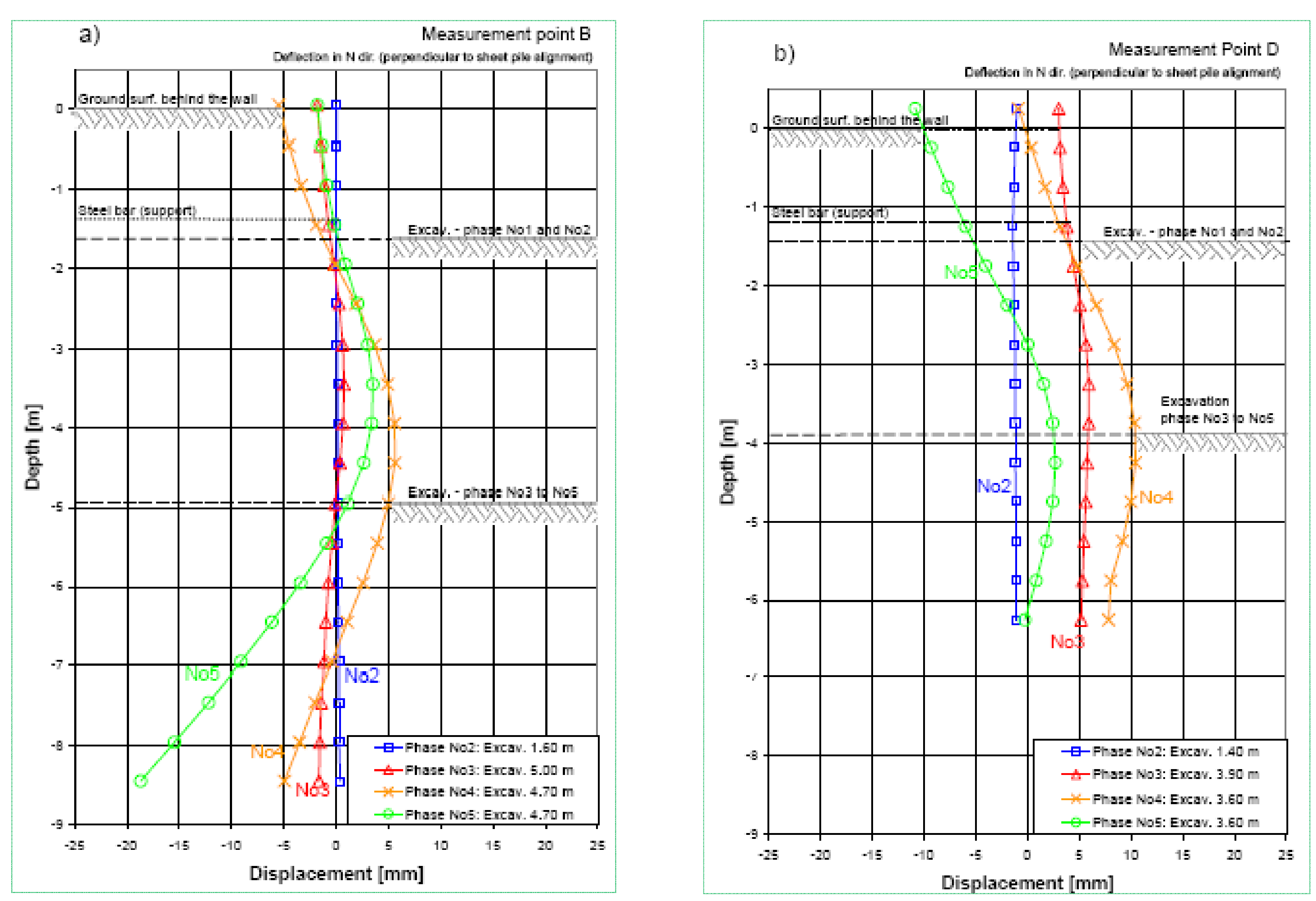

Fig. 4 Wall displacement: a) - in point B, b) - in point D

CONCLUSIONS

- The designed sheet-pile walls with tieback system enabled the contractor to carry out earth and foundation work safely, while still allowing normal operations on the railway.
- Comparison of the measured displacements with indications given in standards (e.g. Eurocode 7) shows that wall displacements did not reach values needed to mobilise the limit active earth pressure .
- The results suggest also that wall displacements did not reach values needed to mobilise half of the full passive pressure.
- It means that, the retaining structure was exposed to less load than had been assumed. Also, the actual tie-rod forces were lower than computed by design.
- Initial pile driving (in area of support P3) caused wall movements on both sides of embankment, with each wall moving toward its respective excavation, and with certain deflection of every wall.
- It shows that pile driving can increase both the active earth pressure and the passive pressure mobilised below the excavation.
- The second phase of pile driving (in the area of support P2) caused expected movement of the wall B toward the embankment, as a result of pile-induced horizontal stresses,
- Additionally in the second phase of piling an incomprehensible (unexpected) displacement of the wall D, also toward the railway line was induced.

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ABSTRACT: This paper describes the results of displacement and deformation measurements of the restrained sheet-pile walls designed as an earth-retention system for a railway embankment near the new motorway viaduct that was being under construction. The monitoring included inclinometer measurement of the wall deformation, geodetic observation of the horizontal displacement of the top of the wall crowns, and train roadbed settlement. The measurement results indicate interdependence between earth pressure and displacement of the retaining structure. Quantitative rates are presented referring to the Polish Standard Code (PN-83/B-03010), Eurocode 7 (PN-EN 1997-1: 2004) and selected literature (Weissenbach, 1975). The influence of pile driving on static behaviour of the sheet-pile wall system is taken into consideration in the final analysis.

1. INTRODUCTION

Before construction of the foundation for the new motorway A1 viaduct WA81 over the existing and active railway track has begun, the pile contractor, which was also responsible for external design, proposed his own solution for retaining the railway embankment in the foundation area.

The submitted solution recognised the necessity of maintaining the railway track in service all the time. Simultaneously, the rules governing monitoring including type and extent of measurements have been established with the General Contractor. The pile contractor chose that the restrained sheet-pile walls would serve as a railway embankment retention system. The bracing of the sheet-pile walls was achieved by tying both walls with tie-rods. Tie-rods were installed below the track ballast and sub-ballast. In all construction phases, i.e. during excavation and during the installation of piles, inclinometer measurements of sheet-pile wall deformation, geodetic observation of the horizontal displacement of the top of walls and measurements of railway track settlement were carried out. Although the above-mentioned monitoring measurements were made because of engineering requirements, the resulting data have been found to be so interesting that the authors decided to publish them in this paper. Because of limitations of the paper, only three problems will be presented:

- Displacement of the retaining wall needed to develop limit states of earth pressure, taking into consideration the kinematic scheme of structure deformation (comparative analysis according to PN-83/B-03010 and Eurocode 7);
- Determining loading states for sheet-pile walls in individual phases of foundation and piling installation (analysis of interdependence between earth pressure and wall displacement taking into account the influence of pile driving);
- Determination of forces in the post-tensioned tie-rods based on wall crown displacements and structure deformation (verification of the assumptions regarding earth pressure variability).

It should be mentioned that the measurements results of one of the monitored phase turned out to be problematic, so the authors would appreciate any assistance in properly interpreting the obtained data.

2. SITE DESCRIPTION

2.1. Building site and measurement program

The subject site is located near Grudziądz and the WA81 viaduct that was being constructed will carry the motorway A1 traffic. Pile driving for the new viaduct substructure, namely the south abutment and pier was carried out. These two supports happened to fall within the railway embankment slopes. The contractor of pile foundations designed the restrained (braced) sheet-pile walls to retain the embankment (Sahajda, 2006).

Two parallel walls, 52 m long each, were installed as shown in Fig. 1. The depth of excavation was 5.0 m adjacent to support P2 and 3.9 m adjacent to support P3. Sheet-pile heights were 9.5 m and 7.0 m, respectively. Retaining structure was restrained by one level of steel tie-rods running through the embankment below the track (see Fig. 2).

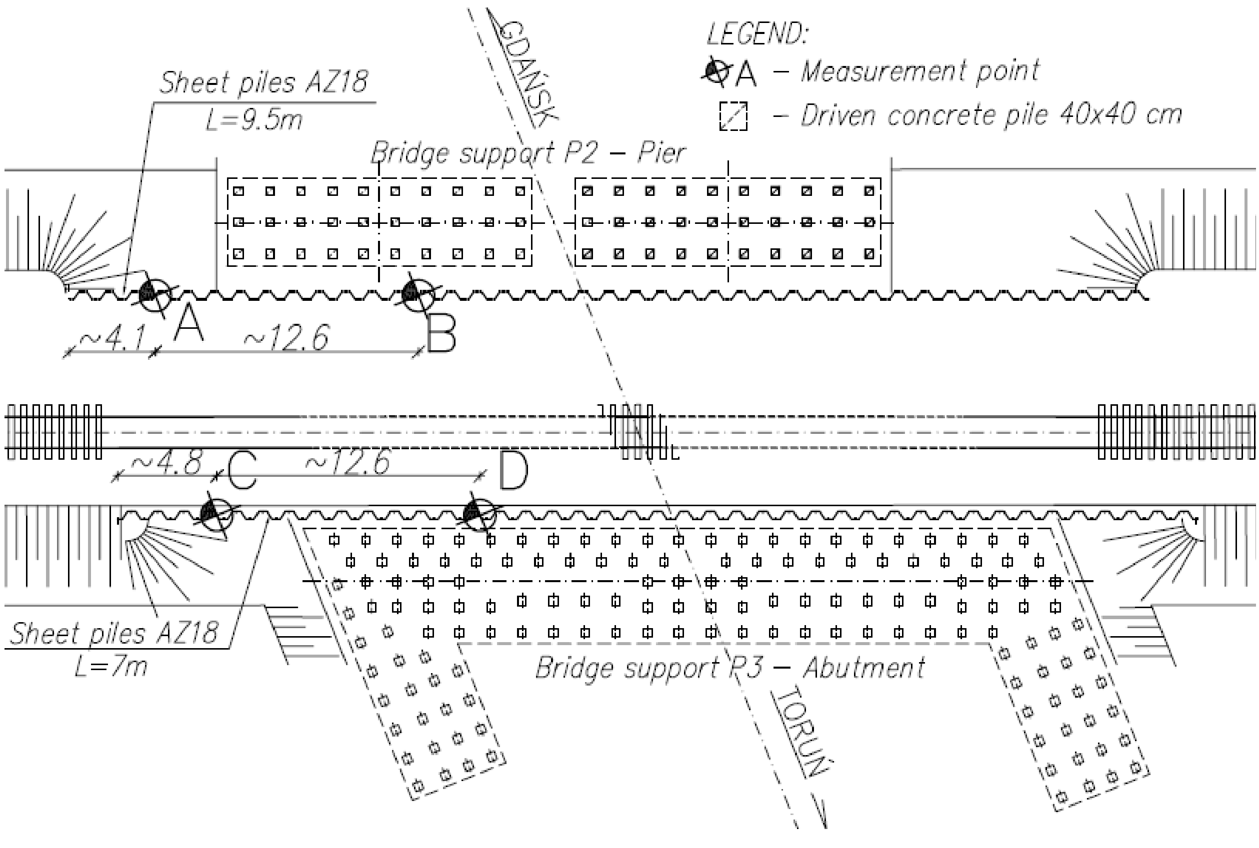

Fig. 1 Layout of the retained rail-trunk-line and the designed viaduct foundations

The sheet-pile work started in May 2007. As soon as driving of the sheeting was completed, a shallow excavation about 0.20 m below the restraint line was made, and then the steel bars were installed. Next, the bars were tensioned by means of a torque spanner, and then the bearing devices were fastened to the wall. Once the steel bars were in place, excavation down to the final level could be completed and installation of the viaduct piling could begin. Piling for support P3 was driven first, followed by piling for support P2. The location of measurement points A, B, C and D are shown on Fig. 1, while the investigative phases are presented in Table 1.

Table 1. Description of measurement phases

Measure phase	Date	Work state in 2007
0-g	05-09	No excavation
1-g,i	05-28	Working platform for tie-rods
2-g,i	06-06	Post-tensioning of steel bars
3-g,i	06-18	Full excavation
4-g,i	08-16	After piling for support P3
5-g,i	09-05	After piling for support P2
Type of measurement: g-geodetic, i-inclinometr		

The installation torque moment for the steel bars was 0.1 kNm. For the 32 mm dia. bars used, this moment corresponds to the force of about 15 kN. In further analysis, it is assumed for simplification that the posttension effect can only reduce some bar looses. It is worth mentioning that the bars were placed in PCV pipes. Such bar installation allowed determination of the internal forces in the tie-rods by measuring elongation of the steel bars and assuming their pure axial tension.

Unfortunately, inclinometer measurements prior to excavation were not recorded; therefore, the first measurement (when platforms for tie-rod installation had been built – refer to phase 1) had to be calculated using strain analysis. However, complete results of the geodetic measurements of the top wall displacements were available. The calculated displacements for the tops of the wall matched the field measurements to within ± 1 mm (which is equal to the accuracy of geodetic measurement). It should be mentioned that the measured displacements were also verified and confirmed by FEM analysis.

2.2. Ground and water conditions

According to the field investigation (Troć & Wojtasik, 2006), sandy soils dominate the subgrade (Fig. 2). The top layers of embankment

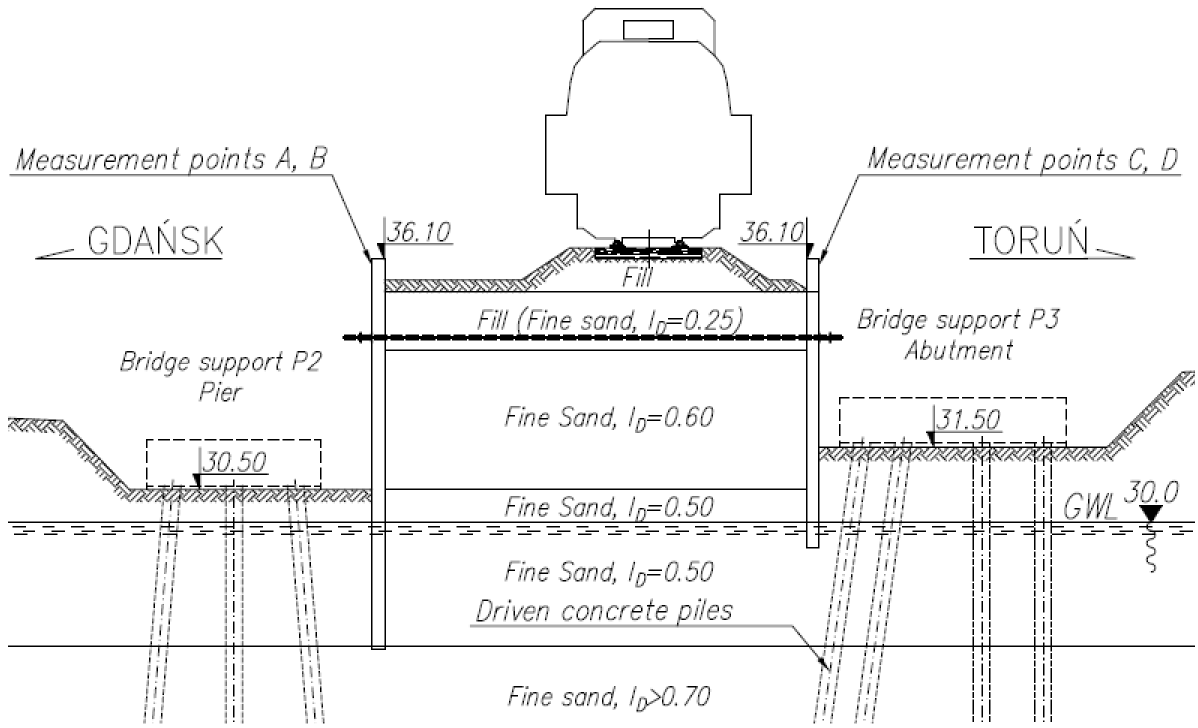

Fig. 2 Embankment retained by sheet-pile walls (cross section after piling completion)

are made of crushed stones and fine sands with humus with density index $I_D = 0.25 \div 0.30$. Below these layers medium dense to dense sands with $I_D = 0.50 \div 0.80$ were found. The ground water table stabilized 0.50 m below the lowest excavation level. The available geotechnical documentation included an interpretation of the static sounding CPT which was carried out according to literature indications (Sikora, 2005).

3. ANALYSIS OF THE RESULTS

3.1. Measurement results

Measurements of deformation of the restrained sheet-pile walls were carried out at four points. The two primary measurement sites (B and D) were located near the piling area, and the two secondary measurement points (A and C) were situated out of the excavation (Fig.1). Because of limitations of this paper, only the results obtained from the primary points are presented (B - Fig. 3a, D – Fig. 3b). The phases 2 through 5 have been taken into consideration regarding comments in p. 2.1.

- After the initial excavation (phase 1), the horizontal displacement of the wall B in the direction of the excavation was 2 mm, and was nearly zero for the wall D. In phase 2, after post-tensioning of the bars, the wall B practically did not displace while the wall D moved 1 mm towards the embankment. The difference in the observed displacement of the walls B and D is most likely occurred due to heterogeneous ground conditions and the small small accuracy of the geodetic measurements (±1 mm).
- After completion of the excavation (phase 3), a small displacement of about 1 mm with slight bending in the middle of the sheet pile was observed on the wall B (Fig. 3a – line 3). The opposite wall D displaced about 6 mm towards the excavation (Fig. 3b – line 3). To explain the larger displacement of the wall D, note the different lengths of embedment of these walls. The wall D has a much smaller length in the passive pressure zone than does the wall B.

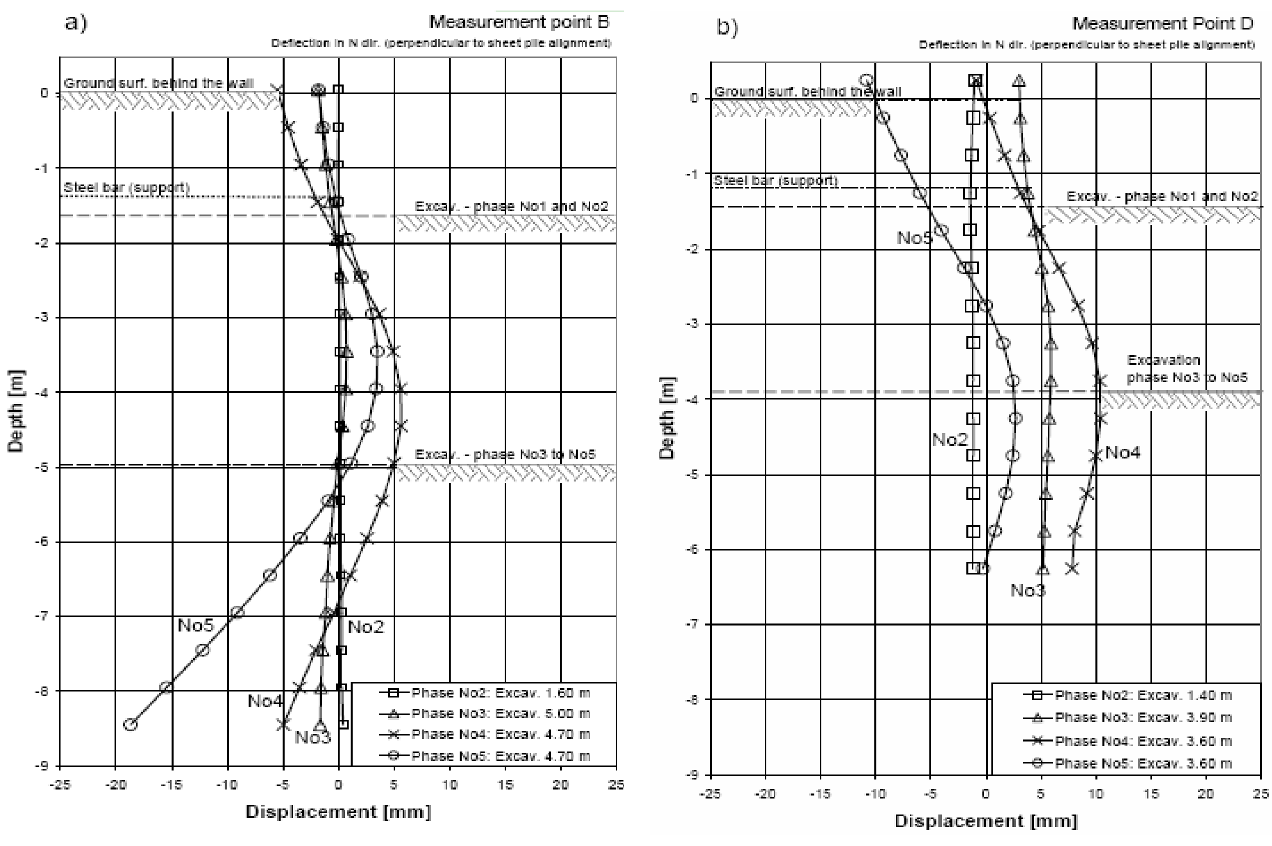

Fig. 3 Wall displacement: a - in point B, b - in point D

- After installation of piles for support P3 (phase 4), the wall B experienced an increase in displacement from 1 mm to 5 mm, where the level of maximal wall deflection depressed approximately 1 m. Displacement of the wall D (close to the pilling area) has increased from 6 mm to 10 mm (Fig. 3b-line 4). This increase can be explained by the effects of pile driving, similar to the compaction-induced stresses (Rymsza, 1997).
- In phase 5 (after piling in the area of support P2), the bottom (toe) of the wall B moved 13 mm towards the embankment, while the deflection of the middle of this sheet-pile wall was reduced to 3 mm. The wall D moved parallely about 8 mm towards the embankment (Fig. 3b - line 5).

The authors would appreciate any feedback which would help to explain the „mechanism" for wall displacements in this phase.

3.2. Interpretation analysis

3.2.1. Displacements for limit pressures

During monitoring, there was no direct measurement of earth pressure. Hence, to determine the value and/or distribution of earth pressure, an indirect method – based on the observed displacement of the retaining structure – has to be applied. For this analysis, values of wall displacements needed to generate limit active and passive earth pressure should be estimated (Rymsza, 1997). The previously mentioned displacements depend on the kinematic scheme of wall yielding (Fig. 4). Quantitative indications given in literature and in standards are often divergent because of many additional factors affecting test results (Weissenbach, 1975).

Comparison of the different guidelines concerning relative displacements of a retaining wall $s/h = \bar{\rho} = v/h$ is given in Table 2, where yielding values needed to develop:

- limit active earth pressure (Table 2a),
- limit passive earth pressure (Table 2b),
- 50% of limit passive pressure (Table 2c).

Referring to the indications in Table 2, some comments have to be summarized for clarification.

- According to Weissenbach (1975), displacements (s, s/h) needed to mobilise limit active earth pressure depend on kinematic schemes of wall yielding (Fig. 4). Three schemes (Fig. 4a, b, d) are taken into account in the case of passive pressure. The influence of wall-ground interface friction ($\delta = k\phi \le \phi$) and mobilised cohesion ($c_m \le c$) on limit earth pressures is discussed in the mentioned book.
- According to the Polish Standard Code (PN-83/B-03010, 1983), the relative displacements ($\bar{\rho}_a$, $\bar{\rho}_p$) needed to mobilise limit active/passive earth pressure should be determined as a function of the angle of internal friction of soil (ϕ) and height of a wall (h). This standard code does not give any guidelines for deflection scheme (Fig. 4c). However, only this Standard (PN, 1983) determines displacements ρ_a, ρ_p for all kinds of soil.

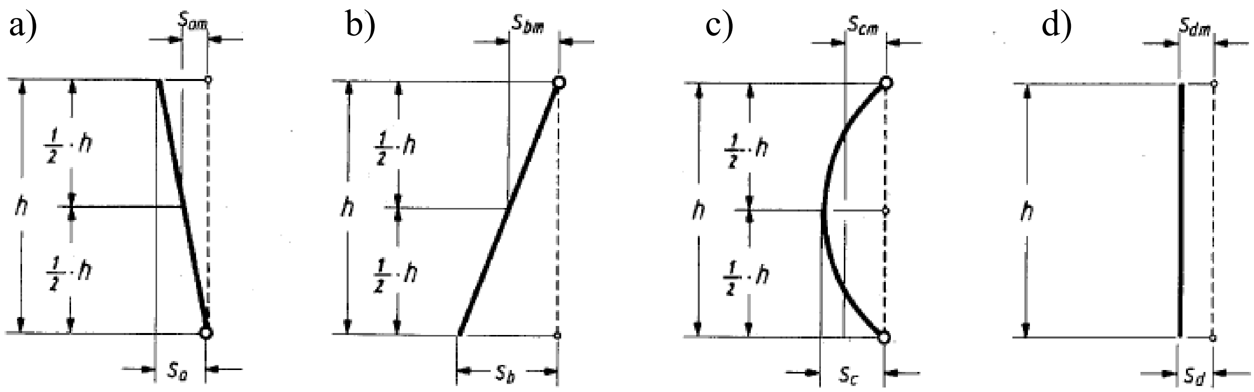

Fig. 4 Schemes of displacement of a retaining wall: a – rotation at the toe, b – rotation at the top, c – curvilinear deflection, d – parallel translation

Table 2. Relative displacements of retaining wall needed for a given earth pressure state

Type of wall movement	Relative displacement in middle dense uncohesive soil		
	A. Weissenbach "Baugruben" Teil 2 s/h [%]	PN-83/B-03010 "Retaining walls" $\bar{\rho}$ [%]*	PN-EN 1997-1 "Eurocode 7" v/h [%]**
a) Limit active earth pressure			
Rotation around the toe	0.20 ÷ 0.40	0.30	0.25 ÷ 0.35
Rotation around the top	0.40 ÷ 0.80	0.30	0.50 ÷ 0.75
Central bending	0.20 ÷ 0.40	Not provided	0.25 ÷ 0.35
Parallel movement	0.10 ÷ 0.20	0.15	0.15 *
b) Limit passive earth pressure			
Rotation around the toe	15 ÷ 22	3.5	6 ÷ 13
Rotation around the top	7 ÷ 15	3.5	6 ÷ 10
Parallel movement	4 ÷ 25	1.7	4 ÷ 8
c) 50% of limit passive earth pressure			
Rotation around the toe	3.2	0.5	1.3 ÷ 3.0
Rotation around the top	1.2	0.5	0.8 ÷ 1.4
Parallel movement	1.5	0.3	0.7 ÷ 1.3

* Values for $\phi = 34^o$, $h_p = 2 \div 4$ m (passive zone), h_a = 6 to 8 m (active zone)
** Average values based on indications for loose and dense soils

- The Eurocode 7 (PN-EN, 2004) determines relative displacements:

v_a/h – concerning limit active earth pressure (4 schemes – Fig. 4);
v_p/h – needed for development full passive pressure (3 schemes – Fig. 4a, b, d);
(v_p/h)– needed to mobilise 50% of limit passive pressure (schemes– as above).

Indications are given only for uncohesive soils in loose and/or dense state.

3.2.2 Analysis of wall loading state

No direct measurement of earth pressure was carried out during monitoring. In order to estimate the actual loads on the restrained sheet-pile walls, an indirect method based on interdependence between earth pressure and wall displacement has to be applied. In such analysis, the wall yielding measured *in situ* (s_{ma}, s_{mp} – in active and passive zone, respectively) should be related to the following comparative values:

s_{la} - needed for development of limit active earth pressure,

s_{lp}, s_{ip}, - needed to mobilise limit or intermediate (namely, half of the limit value) passive pressure, respectively.

These values are given in Table 3, where the corresponding displacements have been determined according to Eurocode 7 (2004). It should be noted that the wall heights studied (h_a, h_p – in active and passive zones, respectively) are different from the as-designed values. The comparative heights (h_a, h_p) were determined for two walls (B and D) in three phases (2, 3 and 4), taking into account the scheme of wall deformation and the safety factor (SF = 1.5).

According to the data quoted in Table 3, neither the sheet-pile wall B nor the wall D was loaded highly enough to reach the limit active earth pressure. The loading state – which is defined as the nearest limit active pressure – was in the phase 3 with reference to the wall D. In this case according to the ratio $s_{ma}/s_{la} \approx 0.60$, the intermediate active pressure ($E_{la} = E_a < E_{ia} < E_0$) was estimated as $E_{ia} \approx SF \cdot E_a \approx 0.8\ E_{ia,d}$. Because of the significant discrepancy in the data concerning the comparative values s_{lp}, s_{ip} due to curvilinear flexible deformation of the sheet-pile wall and curvilinear distribution of the unit earth pressure, estimation of passive zone (h_p) and real intermediate passive pressure ($E_0 < E_{ip} < E_p = E_{lp}$) became more difficult.

Table 3. Measured displacements of the retaining walls vs. limit values according to EC 7 (2004)

Geometrical indications: height of the sheet-pile wall: h relative displ.: s/h = ρ = v/h wall movement: s = ρ = v	Measurement point B			Measurement point D		
	Cantilever ca 1.6 m	Exc. 5m before piling	After piling for supp. P3	Cantilever ca 1.4 m	Exc. 3.9m before piling	After piling for supp. P3
Computational active zone height h_a [m]	3.0	8.0	8.0	3	6.5	6.5
s_{la}/h needed to mobilise limit active earth pressure [%]	0.15	0.25	0.25	0.15	0.15	0.5
s_{la} wall motion for limit active earth pressure [mm]	4.5	20	20	4.5	9.8	32.5
s_{ma} measured value in the active zone [mm]	2	1	7	0	6	11
Computational passive zone height h_p [m]	1.4	3	3	1.7	2.6	2.6
s_{lp}/h needed to mobilise limit passive earth pressure [%]	4	6	6	4	4	5
s_{lp} wall motion for limit passive earth pressure [mm]	56	180	180	68	104	130
s_{ip}/h to mobilise 50% of limit passive earth pressure [%]	0.7	1.3	1.3	0.7	0.7	1
s_{ip} wall motion for 50% of limit passive earth pressure [mm]	9.8	39	39	11.9	18.2	26
s_{mp} measured value in the passive zone [mm]	2	0	6	-1	6	11

Nevertheless, because of the very small measured displacements of the embedded parts of the sheet-pile walls (where the ratios: $s_{mp}/s_{lp} \approx 0.05$ to 0.08, $s_{mp}/s_{ip} = 0.25$ to 0.35), one could note that the real intermediate passive pressure *in situ* had not reached even half of the limit value ($E_0 < E_{ip} < 0,5\ E_{lp}$) in either static phase. The restrained sheet-pile structure was designed with a limitation of the possible passive earth pressure, where max $E_{pd} = 2/3\ E_{lp}$. Hence, the results of monitoring show that the actual loading of the retaining structure was less than had been assumed in the design calculations.

3.2.3 Determination of forces in the tie-rods

The sheet-pile walls were anchored together near their tops by means of tie-rods led through the embankment (Fig. 2). In designing of the retaining system, static forces in the tie-rods were calculated using the classical method (designed values N_d – Table 4). In the verification analysis, based on measurements of the wall displacements at the level of the tie-rods (s_{mB}, s_{mD}) and on elongation of the steel bars $\Delta L_m = s_{mB} + s_{mD}$, the actual forces $N_m = (\Delta L_m/L)AE$ were estimated indirectly, where the values ΔL_m, N_m are given in Table 4.

Table 4 Comparison of tie-rod forces

Analysed value	Excavation phase 3	Pilling P3 phase 4	Pilling P2 phase 5
Measured ΔL_m [mm]	4,0	3,0	-3,0
Monitored N_m [kN]	72,6	54,4	(0,0)
Designed N_d [kN]	92,9	82,7	82,7

The actual forces in tie-rods in phases 3 and 4 were lower than had been assumed in design. According to the measurement results, in phase 5, the steel bars shortened approximately 3 mm. Because of the high slenderness of the bars (d=32 mm, L ≈ 11.5 m), the tie-rod forces in phase 5 were interpreted as $N_m = 0$.

4. SUMMARY

The analysis presented in p. 3.2 leads to the following conclusions:

- The designed sheet-pile walls with tieback system enabled the contractor to carry out earth and foundation work safely, while still allowing normal operations on the railway.
- Comparison of the measured displacements with indications given in standards (e.g. Eurocode 7) shows that in any static phase, wall displacements did not reach values needed to mobilise the limit active earth pressure or half of the full passive pressure. It means that, in reality, the retaining structure was exposed to less load than had been assumed. Also, the actual tie-rod forces were lower than computed by design.
- Initial pile driving (in area of support P3) caused wall movements on both sides of embankment, with each wall moving toward its respective excavation, and with certain deflection of every wall. It shows that pile driving can increase both the active earth pressure (due to compaction–induced horizontal stresses within an embankment) and the passive pressure mobilised below the excavation (as the responding reaction in lower part of a wall).
- The second phase of pile driving (in the area of support P2) caused expected movement of the wall B toward the embankment, as a result of pile-induced horizontal stresses, and an incomprehensible (unexpected) displacement of the wall D, also toward the railway line. However, this scheme of wall displacements and structural deformation provide an explanation for the gradual decrease of tie-rod forces.
- Monitoring on construction sites not only enables control over the safety of the structure under construction, but also field measurements – carried out to fullfill even a typical engineering requirements – can provide interesting information that may be useful in practice.

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