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Evaluation of geotechnical monitoring results of deep excavation support and back analysis using FEM

Tomasz Topoliński
Keller Polska, Gdańsk, Poland, tomasz.topolinski@keller.com

ABSTRACT: This paper presents a deep excavation support design, the results of geotechnical monitoring, and back analysis using the finite element method (FEM). Because of its location on a slope, the construction of the residential building required an excavation with a maximum depth of 16 m in the temporary construction stage and a maximum depth of 10 m in the final construction stage. Due to the site topography and ownership requirements, a bored pile wall was proposed. CFA piles with diameters of 80 cm and 100 cm, and lengths of 15.5 to 20.0 m, were applied. In the temporary construction stage (during concrete works), it was supported by steel struts, and in the final construction stage it was a cantilever retaining wall. The design requirements and parameters of the bored pile wall exceeded typical applications. Therefore, geotechnical monitoring was implemented, consisting of geodetic, inclinometer and strain gauge measurements conducted from the time the piles were installed until the present moment. These monitoring results enabled a continuous assessment of the structure's safety and a back analysis performance using FEM.

KEYWORDS: Excavation support, geotechnical monitoring, observational method, FEM.

1 INTRODUCTION

The densification of urban development requires the implementation of increasingly daring excavation supports. This leads to the design of geotechnical structures for which there is insufficient comparable experience to confirm the safety of the typical assumptions and calculation methods. The observational method addresses this challenge. It is one of the design methods described in EN 1997-1:2004 (CEN, 2004) and is recommended for geotechnical structures whose behavior is difficult to predict. The observational method is based on geotechnical monitoring, the principles of which are outlined in ISO 18674-1:2015 (ISO, 2015). This method involves collecting and analyzing data from measurements and observations of structures. Recently, the importance of the observational method has grown in Poland thanks to large-scale infrastructure projects.

This paper describes a case study of deep excavation support, where the observational method was successfully applied. Such examples demonstrate the growing popularity of a comprehensive design approach based on geotechnical monitoring and numerical calculations. This concept is also included in the next generation of the Eurocode 7 - prEN 1997:202x (CEN/TC 250/SC 7 N 1189, 2018).

2 DESIGN REQUIREMENTS

The residential building was designed on the slope of a post-glacial hill. The elevation difference within the construction site reaches 27 meters. Two levels of underground garage that cut into the natural slopes was planned. The adjacent areas were protected state forest or a housing estate.

The task was to construct a structure that would enable a 16-meter-deep foundation excavation (temporary construction stage) and provide a 10-meter-high retaining wall (final construction stage) (Figure 1). The excavation support is located directly adjacent to the construction site boundary. Due to the proximity of the forest, permanent anchorage outside the construction site was not permitted.

3 GROUND CONDITIONS

The ground model included post-glacial soils classified as fine-grained (lightly clayey sand, silt) and coarse-grained (medium sand, fine sand, silty sand) (Figure 2) according to ISO 14688-1:2017 (ISO, 2017). There was no groundwater up to the foundation level.

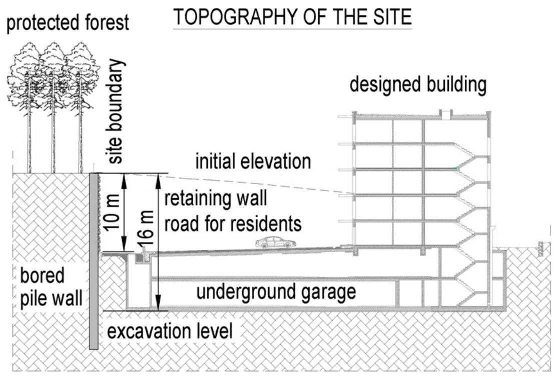

Figure 1. Topography of the site.

4 EXCAVATION SUPPORT SYSTEM

The excavation support was designed as a bored pile wall using the continuous flight auger (CFA) technique (Figure 2). This paper presents and analyzes a section of the excavation support with the deepest excavation (Figure 4). A 20-meter-long piles with a diameter of 100 cm were spaced at intervals of 115 cm. They were made of C25/30 class concrete and were reinforced with cages of f_{yk}=500 MPa steel (Figure 3). A single pile design capacity for simple bending in ULS was M_{Rd}=1967 kNm, and for simple shear was V_{Rd}=285 kN. In the temporary construction stage, the bored pile wall stability was ensured by a temporary steel structure consisting of CHS508x12 mm pipes (struts) and 2xHEB400 built-up sections (walers) made of S355 steel (Figure 4). In the final construction stage, the bored pile wall was supported by reinforced concrete struts and beams with a cross-section of 50x50 cm, which were connected to the support structure of the designed building (Figure 2, Figure 5, Figure 6). The connections of the struts to the bored pile wall were considered as hinged.

The excavation support was designed in accordance with EN 1997-1:2004 (CEN, 2004). Static calculations were performed using analytical methods with GGU-Retain software (GGU Software) and the finite element method (FEM) with Plaxis 2D (Bentley Systems). Due to the significant height of the cantilever retaining wall, the designed bored pile exceeded typical applications. To ensure the required safety of the structure, the conventional calculation-based design method was supplemented with an observational method. Accordingly, advanced geotechnical monitoring was implemented. The acquired data was continuously analyzed and verified against the design assumptions. Values measured on the structure were compared with values predicted by numerical calculations. This analysis provided the basis for deciding when to proceed to the next stage of work, such as soil excavation and strut dismantling.

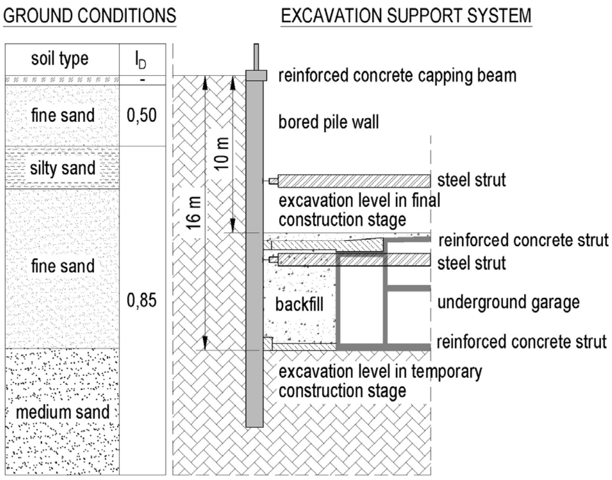

Figure 2. Excavation support system and ground conditions.

The design assumed the following sequence of works:

- Installation of piles.
- Construction of a reinforced concrete capping beam on the top of the bored pile wall.
- Step-by-step installation of the temporary steel struts and soil excavation to the foundation level of the building.
- Construction of the underground part of the building.
- Construction of permanent reinforced concrete struts.
- Backfill of the space between the bored pile wall and the building.
- Step-by-step dismantling of the temporary steel struts.
- Construction of the above-ground floors of the building.

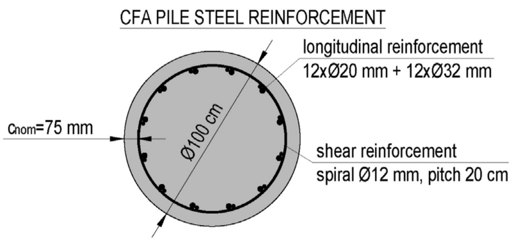

Figure 3. CFA pile steel reinforcement.

5 MONITORING SYSTEM AND PROGRAM

Geotechnical monitoring was an essential component of ensuring the safety and quality of the excavation support. This monitoring included the serviceability limit state (displacement) and the ultimate limit state of bearing capacity (pile cross-section).

The following elements of geotechnical monitoring were applied to the considered bored pile wall section:

- Geodetic measurement of displacement.
- Inclinometer measurement of deflection along the length of the pile.
- Strain gauge measurement.

Initial measurements were taken after the piles and reinforced concrete capping beams were completed. Subsequent readings were taken periodically from the start of the excavation process. Geodetic measurements were taken weekly, and inclinometer and strain gauge measurements were taken each time the static system of the bored pile wall changed (next step of excavation, steel struts dismantling, etc.).

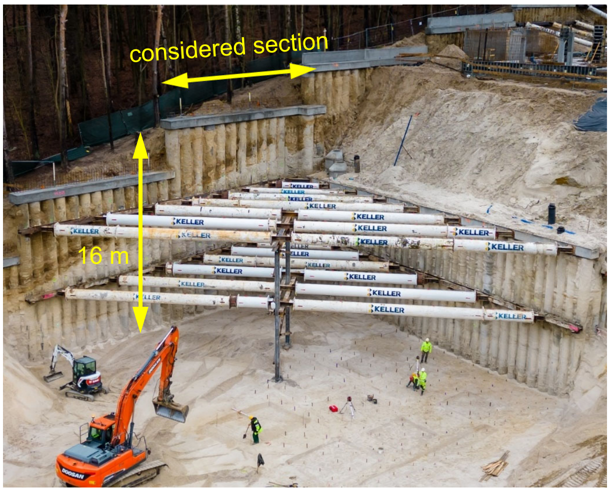

Figure 4. Temporary construction stage.

Geodetic measurements of displacement were carried out with typical instruments. The measurement points were attached to the piles and the reinforced concrete capping beam.

Deflection measurements were obtained along the length of the pile using a manual inclinometer, with readings taken every 50 cm. The inclinometer casing was installed in the fresh concrete of the pile together with the reinforcement cage. Directly measured angular deflections enabled calculation of the deflection curve and relative displacement.

Strain in the pile cross-section was measured using strain gauges embedded directly in concrete. Six measurement levels were established, with a pair of strain gauges aligned perpendicular to the bored pile wall at each level. The directly measured strain was then converted to stress in the concrete and steel reinforcing bars.

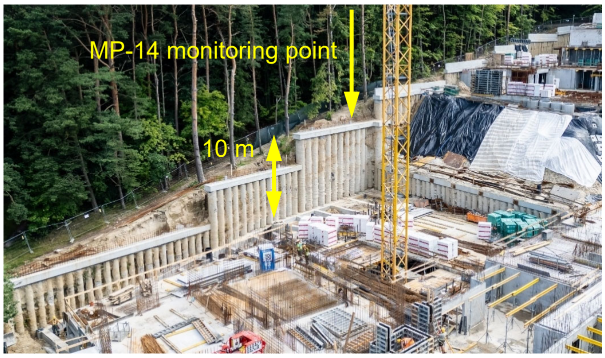

Figure 5. Final construction stage.

It was challenging to install the long and heavy reinforcement cages during pile construction. The measuring devices attached to the rebars, including strain gauges with cables and inclinometer casings, are susceptible to mechanical damage. Special attention was given to the process of lowering the cages into fresh concrete. This ensured the proper placement of the devices, which remained efficient after the bored pile wall was excavated (only about 10% of the 44 strain gauges malfunctioned). Inclinometers and strain gauges were installed in separate, adjacent piles to reduce the risk of damage and improve the installation process of the reinforcement cages.

Figure 6. Reinforced concrete struts.

6 MONITORING RESULTS AND VERIFICATION OF DESIGN ASSUMPTIONS

This part of the paper will present the basic results of the measurements taken at the MP-14 monitoring point, which experienced the greatest displacement (Figure 5). This monitoring point included strain gauge measurements on Pile No. 467, and geodetic and inclinometer measurements on Pile No. 468 (Figure 7).

6.1 *Displacement*

Geodetic measurements provide information about the position of specific points. These are absolute values in the context of displacement. Inclinometer measurements, on the other hand, provide information about changes in angular tilt. The horizontal displacement along the surveyed section is calculated from the angular tilt. If there is no fixed point, the inclinometer measurement should be treated as relative in the context of displacement.

Figure 8 shows the results of geodetic and inclinometer measurements. These graphs allow us to track pile displacement increments over time as the work progressed and the static system of the bored pile wall changed. The deformations indicate pile bending, tilt and horizontal movement. In the final construction stage, the displacement from geodetic measurement at the top of the bored pile wall was 73 mm at the time of the last inclinometer measurements (06.12.2024). This value increased slightly in subsequent measurements, reaching a maximum of 76 mm. It remained unchanged for the following three months of geodetic measurements, indicating stabilization. FEM design calculations estimated the maximum displacement to be 38 mm, assuming the nominal stiffness of the concrete cross-section. However, the more unfavorable value of 0.01H for cantilevers, as reported in the literature (Smoltczyk, 2002) - about 100 mm in this case - was used in the design stage to reliably predict maximum displacement.

As shown in Figure 8, the results indicate a probable displacement of the pile base in the direction of the excavation. This is evident from the larger values from geodetic measurements than from inclinometer measurements. The point geodetic and continuous inclinometer measurements were synthesized. The graph obtained from the inclinometer measurement was translated to roughly match the geodetic measurement. The translation distance was determined by minimizing the sum of deviations using the least square method. Thus, the hypothetical shape of the displacement along the entire length of the pile was obtained, including sections inaccessible to geodetic measurements. The interpreted value of the horizontal movement of the pile base increases with the progress of the work and reaches a maximum of 20 mm. In the design stage calculations, the maximum horizontal movement of the pile base was predicted to be 1 mm.

The significant increase in displacement after the removal of the lower steel strut (24.05.2024) is noteworthy. The preceding and subsequent phases were associated with much more minor changes in displacement.

Taking into account the complex characteristics of the design, the accuracy of the displacement predicted at the design stage was considered satisfactory. Nevertheless, the maximum value and distribution of the measured displacement differ from the design calculations. The following part of the paper attempts to explain the resulting discrepancies through back analysis.

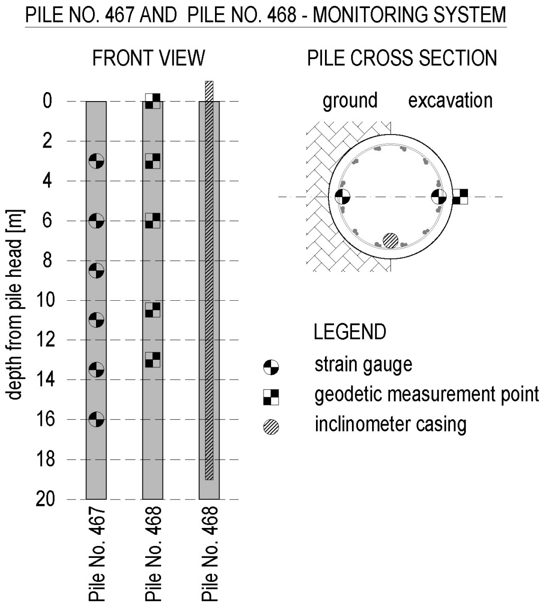

Figure 7. Monitoring system at MP-14 monitoring point.

6.2 *Stress*

Strain gauges measure the point strain within the pile cross-section. Given the stiffness of the material, the corresponding stress can be calculated:

$$\sigma = \varepsilon \times E \tag{1}$$

Where:
σ - stress,
ε – strain,
E - modulus of deformation.

Therefore, the concrete modulus of deformation is an essential parameter in interpreting a pile strain gauge measurement. Concrete samples were taken during pile construction and tested for compressive strength at 28 days. Based on these results and the relationships provided in EN 1992-1-1 2008 (CEN, 2008), the mechanical properties of the concrete used for Pile No. 467 were determined:

$$E_{cm} = 22 \times (0{,}1 \times f_{cm})^{0{,}3} = 33{,}37\ GPa \tag{2}$$

$$f_{ctm,fl} = \max\left[\left(1{,}6 - \frac{h}{1000}\right) \times f_{ctm}; f_{ctm}\right] = = 3{,}03\ MPa \tag{3}$$

Where:
f_{cm} - mean value of concrete compressive strength,
f_{ctm}- mean value of axial tensile strength of concrete,

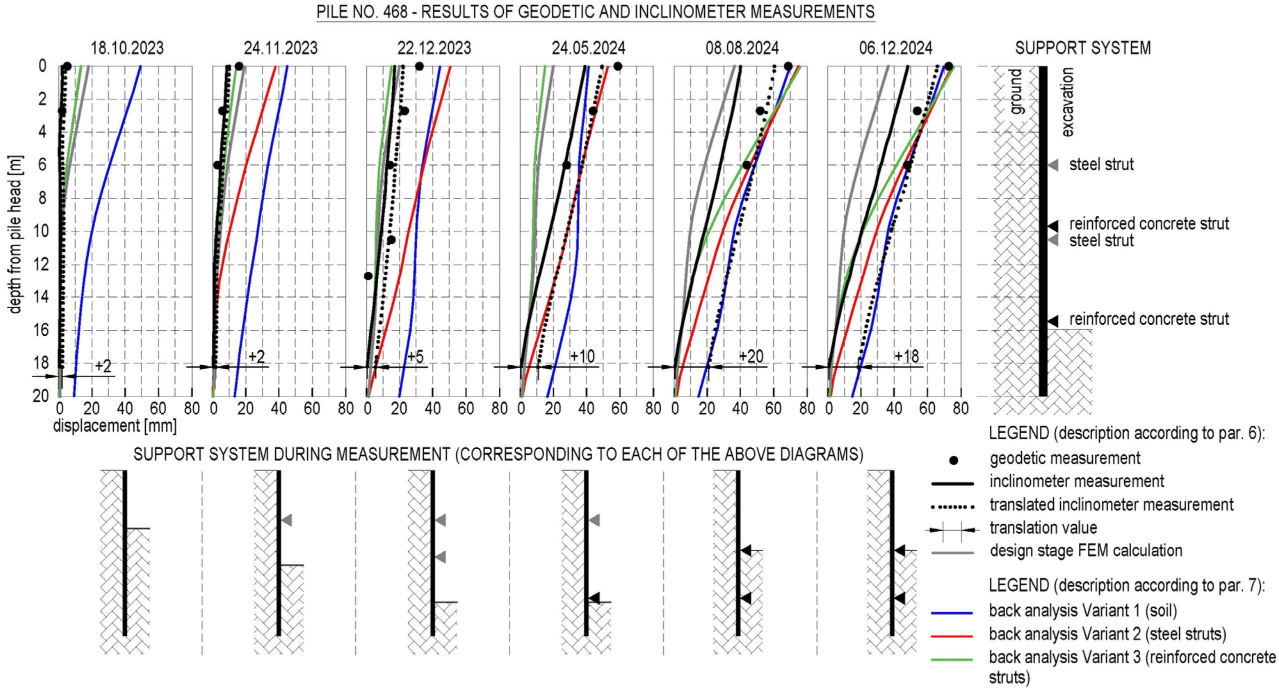

Figure 8. Measured and calculated displacement.

E_{cm}- secant modulus of elasticity of concrete,
$f_{ctm,fl}$- mean flexural tensile strength of reinforced concrete,
h - overall depth of a cross-section of reinforced concrete.

Figure 9 shows the measured values and distribution of stress in concrete in the final construction stage. Since strain gauges were attached to the reinforcing bars, the results correspond to their locations in the cross-section of the pile. The extreme values at the edge of the cross-section were determined analytically under the assumption of engineer's beam theory (Euler-Bernoulli). Figure 9 shows the interpretation of the results for the deepest level of strain gauges. Extreme stress occurred in this cross-section of the pile, but it did not exceed the compressive and tensile strengths of the concrete and steel bars. In particular, the condition was met:

$$\sigma_{t,max} < f_{ctm,fl} \tag{4}$$

Where:
$\sigma_{t,max}$ – maximum tensile stress.

Accordingly, the pile cross-section was in the elastic, uncracked stage of concrete. Thus, the initial assumption for the analysis of a linear stress distribution in the cross-section was confirmed. Additionally, the internal forces in the pile could be determined from the stress diagram using the superposition method shown in Figure 9.

$$M_{meas,16m} = W_{x,el,red} \times \sigma_{bend} = 472{,}59\ kNm \tag{5}$$

$$N_{meas,16m} = A_{red} \times \sigma_{ax} = 1223{,}51\ kN \tag{6}$$

Where:
$M_{meas,16m}$ - bending moment,
$W_{x,el,red}$ – reinforced concrete equivalent elastic section modulus,
σ_{bend} - edge stress from bending,
$N_{meas,16m}$ - normal force,
A_{red} - reinforced concrete equivalent cross – sectional area,
σ_{ax}- edge stress from axial force.

Figure 11 shows the calculated values of the bending moment for the other levels of Pile No. 467 in the final construction stage.

The latest strain gauge measurements in the final construction stage showed slight variation, indicating stabilization. Analysis of stress distribution in all measurement levels and construction stages of the bored pile wall confirmed a high realized factor of safety. The design assumptions and the partial factors method applied by the standards provided the required safety. However, the measured stress and the resulting bending moment distribution differed significantly from the design stage calculations. The maximum value of bending moment was measured at the level of the lower reinforced concrete strut in the final construction stage. In contrast, in the design stage calculations it occurred at the upper reinforced concrete strut. The measured stress will be subject to back analysis later in the paper.

7 BACK ANALYSIS

7.1 *Assumptions*

The back analysis consisted of an FEM calculations using geotechnical monitoring data. The goal was to represent the structure's actual behavior in the numerical model. The purpose of the back analysis was to clarify the discrepancies between the design stage calculations and measurement results.

Back analysis calculations were performed in a plane strain model using Plaxis 2D software. The 'hardening soil' model was adopted for the soil. The bored pile wall was modeled as a linear elastic 'plate' element. The steel and reinforced concrete struts were modeled as 'fixed end anchor' flexible supports. The stiffness of the plate representing the bored pile wall was equivalent to the stiffness of piles in the uncracked stage of concrete. Soil stiffness was characterized by the oedometer modulus E_{oed}, which was obtained from the geological investigation report. The stiffness of the flexible supports, which represent the steel and reinforced concrete struts, was assumed to be theoretical average value:

$$S = \frac{E \times A \times L}{s} \tag{7}$$

Where:
S – theoretical strut stiffness,
E – modulus of elasticity of steel or concrete,
A – cross – sectional area of single strut,
L – design length of strut,
s – average strut spacing.

Figure 8 and Figure 11 show that there are discrepancies in displacement and stress between the measured and calculated values at the design stage. A qualitative assessment was performed to determine the influence of individual factors on the structure's behavior. However, exact values were not compared. In general, the measured displacement of the excavation support was greater than predicted by FEM calculations at the design stage. Therefore, for further analysis, it was assumed that the cause of the discrepancies was an overestimation of the stiffness of the geotechnical structure's elements.

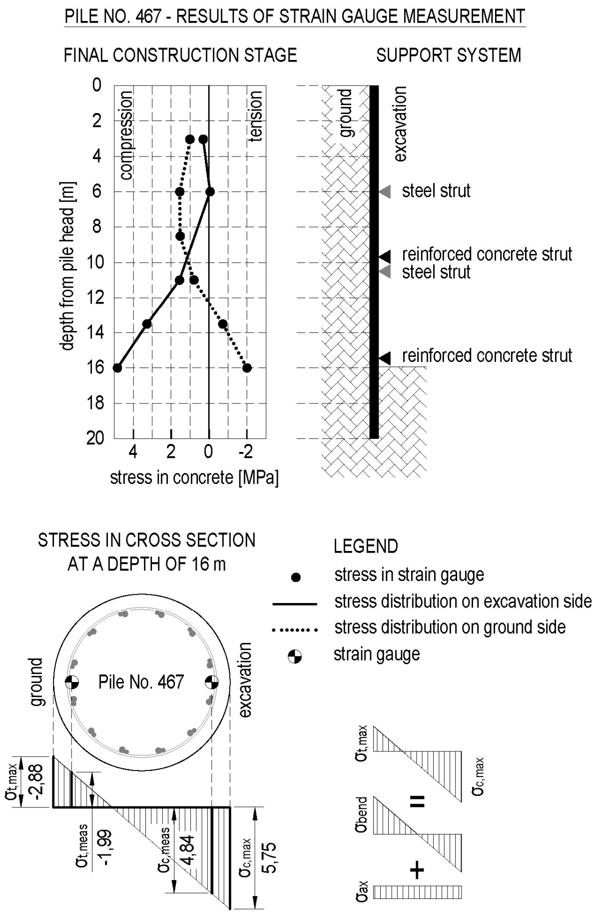

Figure 9. Results and analysis of strain gauge measurement.

The actual stiffness of the struts may be lower than the theoretical value. This could be due to clearance in connections, defects, imperfections, temperature changes, rheological effects, or damage. The behavior of the corner struts may also affect the outcome. When there are different surcharges on the ground, the entire retaining structure can shift toward the wall with less load. Soil stiffness is a complex issue involving the spatial variation of parameters shaped by natural processes. Although advanced laboratory and in situ tests are used to determine soil stiffness, the results are limited to the specific location of the investigation. Therefore, the values of soil stiffness must be interpreted with caution.

As part of the back analysis, a numerical simulation with three variants was performed (Table 1). In each variant, the stiffness of one of the geotechnical structure's elements was iteratively reduced until the calculated displacement of the bored pile wall top was close to the latest geodetic measurement (76 mm). The properties of the other model elements remained unchanged from the design stage assumptions.

Table 1. Back analysis calculation variants.

Name	Description of stiffness reduction.
Variant 1	Soil: 15-times reduction of E_{oed}
Variant 2	Steel struts: 13-times reduction of E_s
Variant 3	Reinforced concrete struts: 45-times reduction of E_{cm}

Geodetic measurements were used for model calibration because they are the most reliable data available. Verification of the variant reducing pile stiffness was not performed since the strain gauge measurements confirmed that the piles were in concrete uncracked stage.

The displacements and internal forces obtained from the back analysis calculations were compared with the geotechnical monitoring data (Figure 8, Figure 11). Then, conclusions were drawn about how the stiffness of each element affects the system.

7.2 *Displacement*

Variant 1 displacement (Figure 8, Figure 10) is horizontal movement and tilt. It includes the top and base of the bored pile wall. The calculated values are a good approximation of the geodetic and inclinometer measurement results in the final construction stage. In earlier phases, however, the calculated values are higher than the measured. Given the presence of dense sand in the ground, the obtained value of the reduction in the soil modulus of deformation seems high, unless the CFA technique significantly loosened the sand. Nevertheless, the relationship between the increase in displacement of the bored pile wall base and the soil stiffness reduction identified in this variant suggests that E_{oed} was overestimated in the design stage calculations.

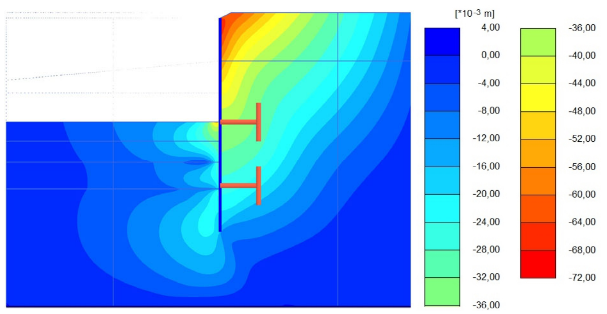

Figure 10. Horizontal displacement in Variant 1 in the final construction stage.

In variant 2, the displacement has tilted shape. It includes the top of the bored pile wall and is negligible at the base (Figure 8). The numerical simulation captures a moderate displacement increase after the upper steel strut is removed. However, there is no convergence of displacement in the initial phases, because the calculated values are larger than the measured ones. Excessive deformation was observed in the elements connecting the steel waling with the piles during the dismantling of the upper level of the steel strut. This was probably the cause of the unexpected and significant increase in displacement of the bored pile wall after the lower steel strut was removed (24.05.2024 on Figure 8).

In variant 3, the displacement has tilted shape (Figure 8). In the numerical simulation, there is a significant increase in displacement after the removal of the upper steel strut that did not occur. Before the upper steel strut is removed, the calculated displacements are smaller than the measured ones. Reduced stiffness in reinforced concrete struts seems to be less probable than in steel struts and soil. This is because these elements are massive, have simple and rigid connections, and structural robustness.

7.3 *Bending moment*

Modifying the stiffness of the various elements of the geotechnical structure affects the bending moment distribution in piles. Calibrating the model based on the displacement of the top of the bored pile wall did not converge clearly on the bending moment (Figure 11). Variant 1 and Variant 2 show corrected bending moment values compared to the design stage calculations, but the distribution is similar. However, in Variant 3, a different bending moment distribution was obtained than in the other cases. There was no positive bending moment value, nor did the value increase over the reinforced concrete strut.

The distribution of bending moment calculated from strain gauge measurements differs significantly from the design stage and back analysis calculations. The measured distribution is similar to cantilever excavation support static system. However, it is intriguing that there is a positive bending moment in the upper part of the bored pile wall. Despite the different distribution, the maximum absolute value of the bending moment in all design stage and back analysis calculations is similar.

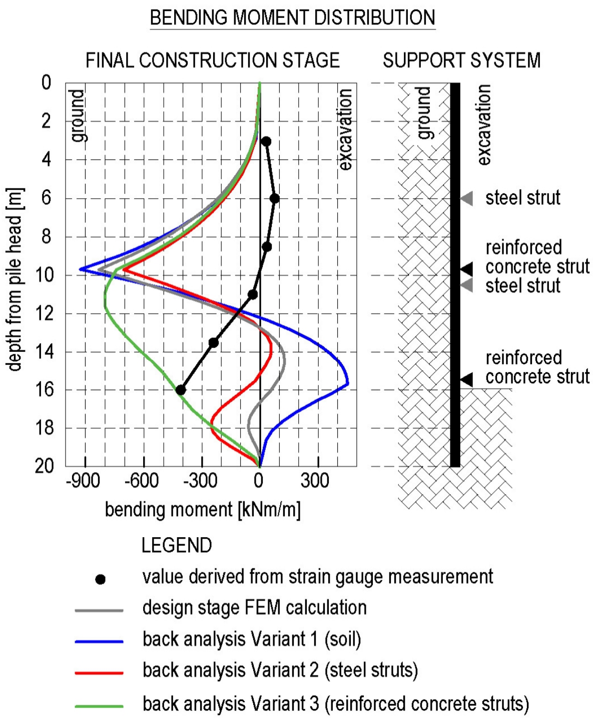

Figure 11. Measured and calculated bending moment in pile No. 467.

7.4 *Conclusions*

Based on the back analysis, it was found that reducing the stiffness of the elements in the geotechnical structure brings the calculation results closer to the measured values. Reduction of soil and steel strut stiffness provides a good match for displacement. In the case of internal forces, reducing the reinforced concrete strut stiffness results in a different bending moment distribution than the other variants. However, it cannot be concluded that the calculated internal forces are similar to the measured values.

The horizontal movement of the bored pile wall base determined by inclinometer and geodetic measurements, only occurs with a significant reduction in soil stiffness (Variant 1). Expanded geodetic measurements near the bottom of the excavation could confirm this phenomenon and validate the applied interpretation of the results. As an alternative, deeper inclinometers could be installed in the soil below the base of the bored pile wall and the zone of influence of the geotechnical structure.

8 SUMMARY

The typical design assumptions provided a high margin of safety for the reinforced concrete cross-section of the bored pile wall. However, the estimation of displacement is subject to uncertainty due to the difficulty of determining the actual stiffness of the geotechnical structure elements. For this reason, it is worthwhile to supplement analytical and numerical calculations at the design stage with recommendations from the literature and comparable experience.

A sensitivity analysis of the FEM model should be performed for complex geotechnical structures to account for possible deviations in the properties of the structural elements. Changes in stiffness can significantly impact the predicted deformation and internal forces.

In the case of the considered bored pile wall, the stiffness of the steel struts and soil was found to be crucial. These parameters should be subjected to detailed analysis in future similar projects. Additionally, it would be advisable to expand monitoring of strut displacement and internal force measurements. This would provide data for the back analysis and enable more accurate safety assessment of the excavation support.

9 ACKNOWLEDGEMENTS

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