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Performance of cantilever sheet pile walls in sensitive clays: Case study Campus Ullevål

Performance des murs de palplanches en porte-à-faux dans les argiles sensibles: Étude de cas Campus Ullevål

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ABSTRACT: The continuous development of urban areas often requires underground constructions close to existing structures and in challenging ground conditions. In Norway, deep excavations in soft marine clays are usually supported by sheet pile walls (SPW) and tieback anchors or internal struts to safeguard the excavation pit and its surroundings until sufficient support is provided by the permanent structure. Cantilever SPWs are rarely used, as deflections are often quite significant. Furthermore, the acting earth pressure distribution, and consequently also the deflection profile, is often difficult to predict. Particularly, the earth pressure distribution and its development over time warrants further investigation.
The research project CURIOUS (Campus Ullevål: Research and Instrumentation Of Underground Structures) was initiated at NGI to study and improve the design of cantilever SPW in sensitive clays. A six-meter-deep excavation pit in Oslo was equipped with an elaborate monitoring system, including strain sensors on the SPW, inclinometers, load cells on temporary struts and geodetic points, to gain a better understanding of the earth pressure development along the retaining structure during the different construction stages. In this paper, results of the instrumentation are presented for different construction phases. The earth pressure distribution acting along the SPW was back calculated based on monitoring data and further compared with predicted distributions.

RÉSUMÉ: Le développement continu des zones urbaines nécessite souvent des constructions souterraines à proximité des structures existantes et dans des conditions de sol difficiles. En Norvège, les excavations profondes dans les argiles marines molles sont généralement soutenues par des murs de palplanches (SPW) et des ancrages de liaison ou des étançons internes pour protéger la fosse d'excavation et ses environs jusqu'à ce que la structure permanente fournisse un soutien suffisant. Les parois de palplanches en porte-à-faux sont rarement utilisées, car les déformations sont souvent très importantes. En outre, la distribution de la pression du sol, et par conséquent le profil de déflexion, sont souvent difficiles à prévoir. En particulier, la distribution de la pression du sol et son évolution dans le temps méritent d'être étudiées plus en détail.
Le projet de recherche CURIOUS (Campus Ullevål: Research and Instrumentation Of Underground Structures) a été lancé au NGI afin d'étudier et d'améliorer la conception des SPW en porte-à-faux dans les argiles sensibles. Une fosse d'excavation de six mètres de profondeur à Oslo a été équipée d'un système de surveillance élaboré, comprenant des capteurs de déformation sur le SPW, des inclinomètres, des cellules de charge sur des étançons temporaires et des points géodésiques, afin de mieux comprendre l'évolution de la pression de la terre le long de la structure de soutènement au cours des différentes étapes de la construction. Dans cet article, les résultats de l'instrumentation sont présentés pour différentes phases de la construction. La distribution de la pression de terre agissant le long du SPW a été calculée sur la base des données de surveillance et comparée aux distributions prédites.

Keywords: Case study; cantilever sheet pile wall; monitoring; sensitive clay.

1 INTRODUCTION

Many parts of Norway are characterized by challenging ground conditions, including soft marine clays (e.g., quick clays). The continuous development of urban areas increases the demand for construction measures in such ground conditions, often close to existing structures. Deep excavations in soft marine clays are usually supported by sheet pile walls (SPW) in combination with tieback anchors or internal struts to safeguard the excavation pit and its surrounding until sufficient support is provided by the permanent structure. Cantilever sheet pile walls have not been widely used in Norway for deep excavations in soft clay, due to the large displacements that occur when not supported by struts or tie-backs anchors.

The design of retaining structures in Norway is performed according to standards such as the Eurocode. The distribution of the acting earth pressure is often assumed based on literature recommendations (e.g., Terzaghi, 1943), as monitoring results are usually not available for similar boundary conditions. Today, design is most often supported with numerical analyses to predict the system behaviour. However, high-quality data of in-situ and laboratory tests are required to define the input parameter for constitutive soil models. The latter input parameters can further be updated during and after construction by back calculating monitoring results. However, different parameter sets can be used to reproduce the monitoring results.

The construction project Campus Ullevål (Oslo, Norway) offers the opportunity to investigate the performance of cantilever sheet pile walls for excavation support in soft marine clays. An extensive monitoring concept was installed to characterize the deformation behaviour and earth pressure distribution during construction. The results in combination with numerical analyses should further be used to (i) validate existing and (ii) elaborate new design procedures for cantilever sheet pile walls in soft clays.

In this paper, the monitoring setup is presented, and preliminary results of strain sensors on the SPW, inclinometers as well as load cells on temporary struts are shown. Furthermore, the monitoring data is used in the second step to estimate the acting total earth pressure along the SPW at different stages of excavation.

2 CANTILIVER SHEET PILE WALL

Cantilever sheet pile walls are used to support soils for low to moderate depth of excavation. The earth pressures generated at both sides of the retaining structure distribution fulfils the force and moment equilibrium (Gopal Madabhushi and Chandrasekaran, 2005) and were studied by various researchers (e.g., Terzaghi, 1934; Bransby and Milligan, 1975).

The earth pressure theories according to Rankine or Coulomb are usually applied in practical engineering to predict the acting earth pressure along the SPW. However, previous investigations showed that certain deformations are required to mobilize the active and passive earth pressure (Terzaghi, 1934).

For the stability of the excavation, the installation depth of the SPW represents a key design parameter. Various design principles for cantilever sheet pile walls were elaborated by researchers (e.g., Terzaghi, 1934; Padfield and Mair, 1984; Broms, 1995) and were summarized to design guidelines (e.g., Bolton, 1991a, 1991b).

3 CAMPUS ULLEVÅL

3.1 Project description

This case involves construction of a 10-storey building, with two basement storey in Oslo, Norway. For this, an excavation pit up to 7 m deep was required. Due to the location in urban environment close to existing structures, a retaining structure was designed and installed to support the excavation. The retaining structure was comprised by an internally supported 15 meter deep floating SPW. At a final stage, the SPW wall cantilevered up to 7 m from a concrete base slab. During excavation, the SPW was supported internally with ground reinforcement.

3.2 Ground conditions

The ground surface is nearly flat at the construction site, ranging between +97.1 and +98.3 m above the sea level. The top layer, composed of fill material and dry crust clay, shows a thickness of 1 to 2 m. A medium stiff to stiff, low-sensitive clay of 6 to 7 m thickness is situated below the top layer and overlays a soft to medium-stiff quick clay down to bedrock. The clay is interbedded by silt and sand layers (between +83 m down to bedrock) in the northern part of the site. These sand and silt layers can range in thickness from a few cm to 2 m. The ground water table was situated 1 to 1.5 m below the original terrain during the investigations.

The typical unit weight for the clay is 19.3 kN/m^3, and a water content around 30 %. Undrained shear strength c_u^A varies from 75 kPa in the dry crust, before falling to a bottom plateau around 40 kPa, before rising linearly 2.5 kPa/m from approx. 10 m depth.

3.3 Excavation work

The final excavation depth is situated at +91.4 m, approx. 7 m below the original terrain. The working steps for excavation is illustrated in Figure 1 and can be summarized as follows:

I. The sheet pile profiles (type AZ17-700) were driven 14.5 m into the soil and approx. 1 m top soil was excavated. Soil improvement with *lime-cement (*LC*)* was installed to support the excavation. The LC was installed in a ribbed pattern perpendicular to the SPW (lengths and depths of individual ribs amount to 20 m). The LC ribs are spaced approx. 3.5 m apart (see Figure 2) and are composed of overlapping columns (Ø600 mm).

II. The soil in the central part of the pit was excavated down to the final level (7 m below original ground surface). The SPW was supported during excavation by a soil berm (4 m wide plateau, 1:1.5 inclination). Struts and walers were installed approx. 0.5 m below original terrain. The strut is disposed approx. 23° and further connected to a concrete slab at the excavation floor.

III. The berms supporting the SPW were excavated, and the slab was extended and cast into contact with the SPW. This work was carried out in small sections of 6 m width and symmetrically in both sides of the construction site, ensuring global stability of the concrete slab.

IV. Struts and walers were removed, leaving the SPW cantilevered from the concrete slab.

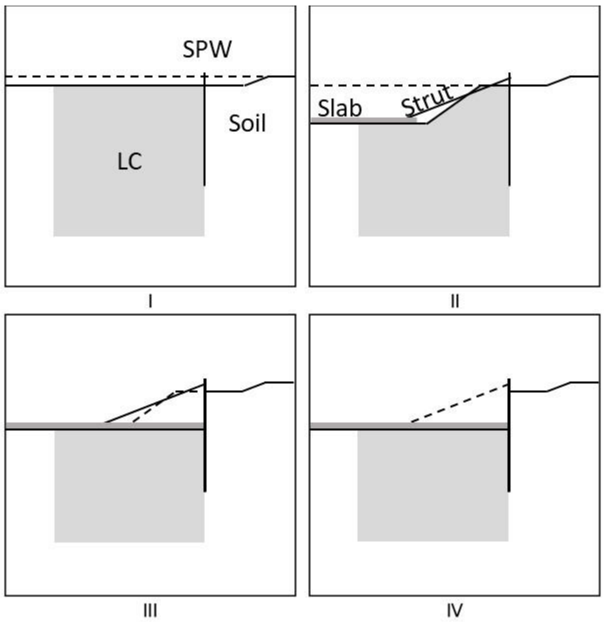

Figure 1. Working sequence of the excavation. Dashed lines indicate the excavation or removal of struts prior to a given stage.

3.4 Instrumentation

The monitoring concept is composed of *fiber optic strain gauges* (FOSG) attached to the SPW; three *inclinometers* (INC) installed on the soil side of the SPW; and tracking of three *geodetic points* (GEO) along the top of the SPW. For an overview of the installed instrumentation, reference is made to Figure 2. Further instrumentation was installed at the site but not further discussed in this paper.

The deformation behaviour of the SPW and the adjacent soil was measured by three inclinometers and geodetic points tracked by automatic surveying equipment. Two inclinometers, INC1 and INC2, were installed immediately behind the SPW. INC3 was installed 5 m behind the SPW (see Figure 2).

The deflection curve of the SPW was measured with fiber-optic sensors. Two sensors, one on each flange of a single Z-profile, measured the strains along the SPW at an interval of 0.5 m (see Figure 2).

The instrumentation shown in Figure 2 can be summarized as follows:

① Geodetic point, GEO1 GEO2 and GEO3 are placed at equivalent locations further along the SPW.
② Fiber optic strain gauges on the SPW.
③ Inclinometer, INC1 – 0.5 m resolution, biaxial
④ Inclinometer, INC2 – 2.0 m resolution, uniaxial
⑤ Inclinometer, INC3 – 2.0 m resolution, uniaxial

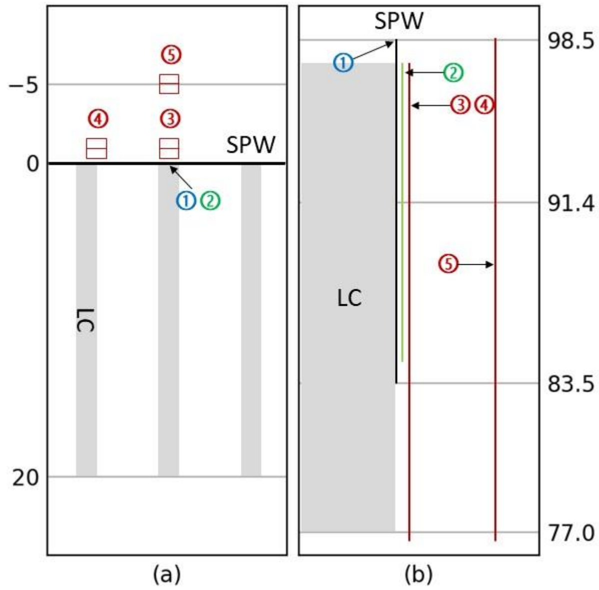

Figure 2. Overview of the instrumentation setup. (a): Plan view, (b): Section view. Annotative colors: blue = geodetic points, green = fiber optics strain gauges, red = inclinometers.

4 RESULTS

4.1 Deformations

Figure 3 shows the measured displacements of the inclinometers (INC1, INC2, INC3) and geodetic points (GEO1, GEO2, GEO3) for three construction stages, namely excavation in the central part of the building pit (see II in Figure 1), full excavation with struts installed (see III in Figure 1), and the cantilever stage (see IV in Figure 1). It should be mentioned for stage IV that the measurements were taken immediately after removing the struts and installing the foundaton slab. It can be seen that the deformations increase with the excavation process. While INC1 and INC2 (both situated behind the SPW) are in good agreement, INC3 shows smaller displacements in all three excavation stages (see Figure 3). While the maximum displacements of INC1 and INC2 amounts to approx. 150 mm in stage IV, only 65 mm head displacements was measured at INC3. The results of the geodetic points show a wide scatter (stage II: 26 – 38 mm, stage III: 50 – 130mm and stage IV: 135 – 210 mm), leading to a poor agreement with INC1 and INC2, both installed directly behind the SPW. While GEO2 and GEO3 show similar trends in phases III and IV, the displacements of GEO1 are significantly larger in the latter phases.

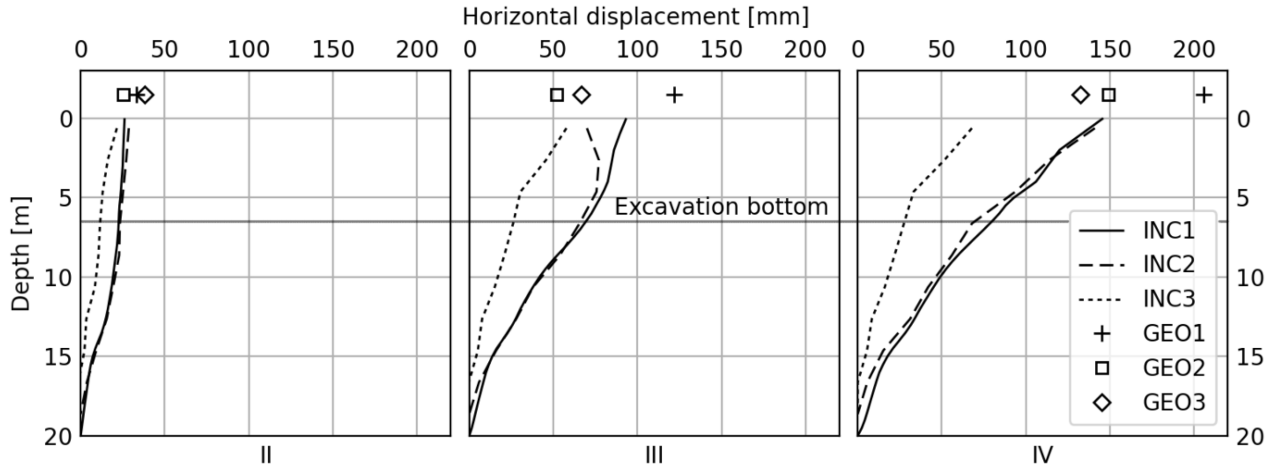

Figure 3. Measured deflection of the soil and SPW at different stages of excavation. (II) partial excavation, (III) full excavation with struts installed, (IV) cantilevered stage.

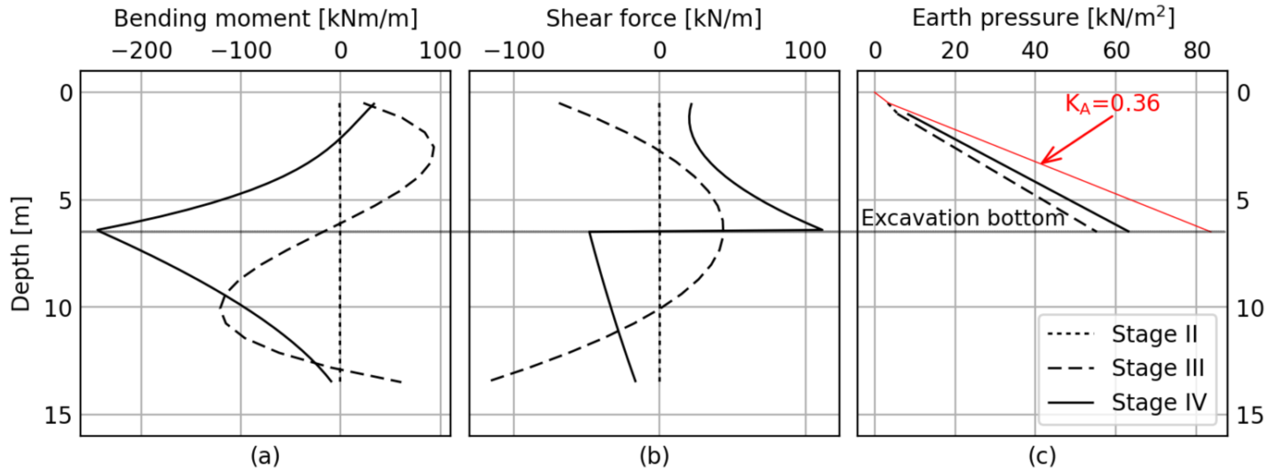

Figure 4. Structural forces along the SPW derived from FOSG monitoring data. (a): Bending moment (b): Shear force (c): Earth pressure

4.2 Structural forces and earth pressure

The bending moments M, the shear forces V and the acting earth pressure along the SPW, all derived from the measured deflection curve κ from FOSG data, are shown in Figure 4. The construction stages II, III and IV, defined in Figure 1, are indicated by different line strokes. The deflection curve κ was derived by fitting a 3rd degree polynomial to the strain measurements. For the cantilevered system in stage IV, the deflection curve was composed by two 3rd degree polynomials joined at the excavation bottom, to fit better with the expected profile. The bending moments *M* were estimated according to Equation (1) by assuming linear elastic beam theory:

$$M = -EI \cdot \kappa \quad (1)$$

where *EI* represents the bending stiffness (76.08 MNm^2/m for AZ17-700). The shear forces *V* and the acting earth pressures *p* represent the first and second derivative of the moment curve, respectively. The earth pressure is only shown in Figure 4 (c) for full excavation stages (III and IV), and only down to the excavation floor. The active earth pressure, calculated according to Rankine (1856) assuming a friction angle φ = 28°**,** is additionally given in red.

M and *V* approximate zero for stage II, as no defection of the SPW was measured. An S-shaped bending moment distribution was observed after excavating the berms (stage III). The derived earth pressure shows an increase over depth, albeit slightly smaller compared to the calculated active earth pressure according to Rankine theory (1856). The maximum moment is observed at the excavation bottom after removing the struts and extending the foundation slab (see solid, black line in Figure 4 (a). The kink in the moment curve leads to a corresponding discontinuous shear force profile. The earth pressure profile (by definition, a straight line with the approach used in this paper) increases slightly in stage IV compared to stage III but remains smaller than the estimated active earth pressure.

5 DISCUSSION

The displacements of the geodetic survey points (GEO1, GEO2, GEO3, installed at the SPW) and the head displacements of inclinometers (INC2, INC3, installed close to the SPW) show a certain scatter. Different observations are discussed in the following:

- The measured displacements at the upper part of INC1 and INC2 show a difference at stage III, as the SPW was only partly shimmed to the waler at this stage. While the single sheet pile was not supported by the waler and strut close to INC1, a support is given at INC2 (see Figure 3,III). The deflection curve for INC1 resembles a cantilevered system, as opposed to INC2, which clearly shows the effect of the strut and waler. At stage IV, where the gap between SPW and waler was closed, both inclinometer measurements are in good agreement.
- The bottom of the inclinometers is not fully fixed as they are situated on top of bedrock, enabling horizontal displacements without measuring deflections. As GEO2 and GEO3 show a good agreement with results of INC1 and INC2 it is assumed that movements at the inclinometer foot can be neglected.
- It is assumed that the large deviation of GEO1 from INC1 and INC2 cannot be related to measurement inaccuracy only (see Figure 3,III and IV). It must be further investigated whether human influence or other reasons cause this deviation.

Contrary to expectation, the bending moment M and shear force V do not converge to 0 at the top of the SWP for the cantilevered system (see stage IV in Figure 4). It is assumed that the fiber optic strain gauges were influenced by thermal or mechanical effects. Exposing the sensors to high heat while removing the walers with blowtorches could affect the measurements. Mechanical effects such as bending of the SPW can be caused during the construction work. It is feasible that other causes affecting the readings can also be present.

A spurious earth pressure profile is derived when including the topmost strain measurement datapoint. Therefore, the topmost datapoint were neglected when deriving the profiles shown in Figure 4 (c).

The derived acting earth pressure profile is increasing from stage III to stage IV. Typically, as deflections outwards increase, the earth pressure is expected to decrease. One possible explanation might be related to the consolidation process, as the clay behind the SPW progresses into a partially drained state. This explanation must be further investigated.

6 CONCLUSION

The performance of cantilever sheet pile walls in soft Norwegian clays is investigated within the research project CURIOS. An extensive monitoring concept, including inclinometers, geodetic points and fiber optic strain gauges, was installed to study the displacement behaviour and the acting earth pressure along the SPW.

This paper presents results of the instrumentation for different stages of excavation. Furthermore, results of fiber optic strain gauges are used to calculate structural forces acting on the sheet pile wall as well as the acting earth pressure. The results indicate that the derived earth pressure is smaller compared to the active earth pressure, derived according to Rankine (1856).

Further analyses of the measured data are required to characterize the deformation behaviour, the structural forces, and the acting earth pressure for different stages, considering consolidation processes. In a second step existing design procedures for cantilever sheet pile walls will be validated and/or revised for Norwegian, soft clays.

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