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Numerical back analysis of a cantilevered sheet pile wall in sensitive, marine clay: Case study Campus Ullevål

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ABSTRACT: In Norwegian soft clays, deep excavations in urban environments are commonly supported by sheet pile walls (SPWs) combined with tieback anchors or struts. For medium excavation depths, cantilevered SPWs offer a cost-effective alternative by temporarily bracing the retaining structure against a bottom concrete slab or the surrounding soil, provided that strength and stiffness criteria are met. However, their use in Norway has been limited due to the risk of larger deflections compared to stiffer support systems, and the exponential increase in deformations and structural forces with excavation depth. At the Campus Ullevål construction site in Oslo, an up to 7-meter-deep excavation in sensitive, medium-stiff marine clay was supported using a 15-meter-deep cantilevered SPW. To reduce wall deformations and ensure stability, panels of deep dry soil mixing (DDSM) columns were installed within the pit. Deformation behavior and structural forces were monitored throughout the construction using a comprehensive instrumentation program. This paper introduces a novel design approach for cantilevered SPWs by accounting for drainage effects near the wall through a drained triangular zone. Numerical analyses were performed in Plaxis 2D along a representative cross section, testing different wedge geometries and soil properties. A bivariate grid search of shear modulus of clay and friction angle in the drained wedge was performed and compared against deflection measurements. A true minimum error value was not found, with optimal values trending towards the stiffer /stronger ends of ranges to match measurements, suggesting a conservative modelling approach. The findings indicate that the proposed design approach may capture wall deformations and could offer a practical alternative to fully coupled flow analyses, but further analysis is necessary to conclude. Moreover, the case study confirms that cantilevered SPWs can be successfully used in medium-stiff clays for excavation depths up to 7 meters.

KEYWORDS: clay, excavation, cantilever sheet pile wall, geotechnical design, case study.

1 INTRODUCTION

The continous 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 (e.g., quick clays) are usually supported by sheet pile walls (SPWs) and tieback anchors or internal struts to safeguard the excavation pit and its surroundings until sufficient support is provided by the permanent structure (Sandene et al 2024).

A cantilever sheet pile wall (SPW) is a type of retaining wall embedded into the ground without additional support from anchors or struts. Its stability relies solely on sufficient embedment depth below the excavation level, which provides passive resistance to counteract active earth pressures. Cantilever SPWs are typically used for shallow to moderate excavation depths, where ground conditions permit some wall deformation without compromising nearby structures or overall stability. Despite offering potential cost savings, their use in Norway has been limited due to concerns over larger deflections - which could negatively impact adjacent structures - and increased structural demands that may require more robust sheet pile profiles.

Traditional analytical approaches for designing cantilever walls are based on limit equilibrium concepts and assume that failure conditions are reached along the entire wall (Torrabadella, 2013). The design methods differ with respect to: (i) the shape of the earth pressure distribution, (ii) method of computing the equilibrium of the wall, (iii) theories used in derivation of earth pressure coefficients, (iv) distribution and direction of wall friction and (v) method for obtaining the internal friction angle (e.g., Krey, 1932; Blum, 1931; Rowe, 1951; Brinch Hansen, 1953; King, 1995). Bica & Clayton (1989) conducted a comprehensive review of 25 analytical methods for cantilever pile walls, comparing them with experimental data. They concluded that limit equilibrium methods can provide reasonable estimates of embedment depth and maximum bending moment, provided that the simplifying assumptions are balanced by appropriate modeling choices. Alternatively, finite element analysis is increasingly used to assess both the serviceability and ultimate limit states of retaining structures (e.g., Day, 1999).

The construction project Campus Ullevål (Oslo, Norway) offers the opportunity to investigate the performance of cantilevered SPWs for excavation support in marine, Norwegian clays. A comprehensive instrumentation program was installed to characterize the deformation behavior and structural forces during excavation.

This paper introduces a novel design approach for cantilevered SPWs by accounting for drainage effects near the wall through a drained triangular zone. Numerical analyses are performed in Plaxis 2D along a representative cross section, validating different wedge geometries and properties based on inclinometer measurements. The presented results intend to support the future design of cantilever SPW in clays.

2 CAMPUS ULLEVÅL

The construction project Campus Ullevål in Oslo (Norway) involved the development of a 10-storey building with two basement levels, requiring an excavation depth of 4 to 7 m. A retaining system was required to support the excavation, given the urban setting and proximity to existing structures.

The construction site is characterized by a relatively flat terrain, with ground surface elevations ranging from +97.1 m to +98.3 m above sea level. The uppermost soil layer consists of fill and dry crust clay, with a typical thickness of 1-2 m. Beneath this, a 6-7 m thick layer of medium stiff to stiff, low-sensitive clay is present, which overlies a deposit of soft to medium stiff quick clay extending down to bedrock. In the northern part of the site, the clay strata are interbedded with discontinuous silt and sand layers found between approximately 15 m depth and the bedrock. These interbedded layers vary in thickness from a few centimeters up to 2 m. During the site investigations, the groundwater table was observed at depths between 1.0 and 1.5 m below the original ground surface. The clay has a typical unit weight of approx. 19.3 kN/m^3, and an average water content of about 30 %. The undrained shear strength in compression (s_{uC}) is highest in the dry crust, reaching up to 70 kPa, then decreases to a plateau of around 40 kPa within the low-sensitive clay. Below this, from roughly 10 m depth, it increases linearly at a rate of approximately 2.5 kPa/m (Monsås et al. 2024).

The construction process began with driving sheet pile profiles to a depth of 14.5 m. After this, a shallow excavation of top soil and installation of deep dry soil mixing (DDSM) on the passive side of SPW to provide support during excavation. DDSM columns in a ribbed pattern perpendicular to the wall was installed at approx. 3.5 m spacing down to a depth of 20 m below terrain (+77 masl.). The layout of the SPW and DDSM panels is shown in Figure 1.

The excavation working sequence is shown in Figure 2. Excavation continued to the final depth of 4 to 7 m, with temporary support of SPW provided by a soil berm along the perimeter of the pit. Inclined struts were then installed and braced against a base slab of concrete. Once the berms were removed, the concrete slab was extended and cast against the SPW in small, symmetrical sections to maintain stability. Finally, the struts and walers were removed, allowing the sheet pile wall (SPW) to act as a cantilever from the concrete base, with a maximum excavation depth of 7 m.

The instrumentation program included fiber optic strain gauges (FOSG) attached to the SPW, bi-axial inclinometers (INC) installed on the soil-side of the SPW and within the construction pit, geodetic points (GEO) along the top of the SPW as well as load cells on internal struts (not shown here). The location of INC 5 and the cross-section selected for numerical analyses are shown in Figure 1.

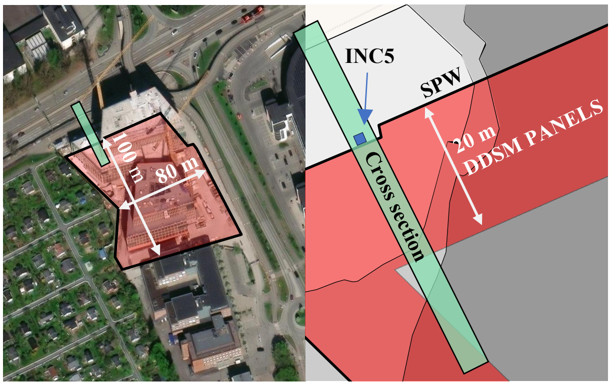

Figure 1. Overview of the Campus Ullevål excavation pit, showing the location of inclinometer INC 5 and the cross-section selected for numerical analysis.

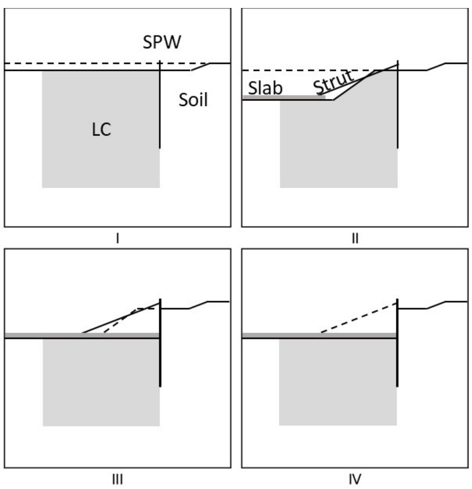

Figure 2. Working sequence of the excavation (Monsås et al. 2024). Dashed lines indicate the excavation or removal of struts prior to a given stage.

3 NUMERICAL DESIGN APPROACH

Numerical calculations were performed in *Plaxis 2D 2024.3*, following the geometry presented in Figure 3. Three clay units, namely (i) Crust, (ii) Clay 1 and (iii) Clay 2, were considered in the model (see Figure 3). The NGI-ADP model, enabling consideration of anisotropic shear strength for undrained conditions, was selected for the units (i), (ii) and (iii). Soil parameters (undrained shear strength, unit weight, water content etc.) have been calibrated based on high-quality in-situ and laboratory data. Please refer to 0 for an overview of the NGI-ADP input parameters.

Fill material (iv) was modeled using the hardening soil model and the stabilized soil volume (vi) using the Mohr-Coulomb soil model.

The NGI-ADP has shown in experience to be unable to accurately predict forces acting on SPW, especially as deflections increase, with a significant underestimation of the earth and water pressures. For this reason, an approach with a drained field behind the SPW was proposed in design to better predict earth pressure on SPW. A wedge with a drained material (v) (hardening soil model) was modeled for the final cantilevered stage. Initial trials showed wedge geometry had significant influence on calculation results. In the parameter study, three different wedge geometries were investigated with base assumed at a fixed point slightly below excavation bottom, shown in Figure 3. A: Constant width of 1 m behind SPW, B: Variable width (1 m near bottom, 2 m near top) C: Triangular with apex angle eq. to 30° (approx. 45° - φ/2)

The SPW was modeled using an elastoplastic plate material with mechanical properties equivalent to AZ17-700, with bending stiffness EI of $76.10 \cdot 10^3$ kN/m. The concrete slab was modeled with linear elastic material with E of $15 \cdot 10^6$ kN/m^2. The modulus for concrete was chosen to be lower than reference values to account for early loading (after few days) and model effects.

The complete construction sequence was modeled in Plaxis, see Figure 3. The groundwater level was assumed 1 m below ground surface. No effect on groundwater table was assumed during construction, as the SPW was installed watertight, and dissipation up through the clay was estimated to be negligible.

A total of 9x8x3=216 simulations were run. $G_{ur}/\ s_u^A$ for clays and φ for the drained wedge were varied with a simple grid search.

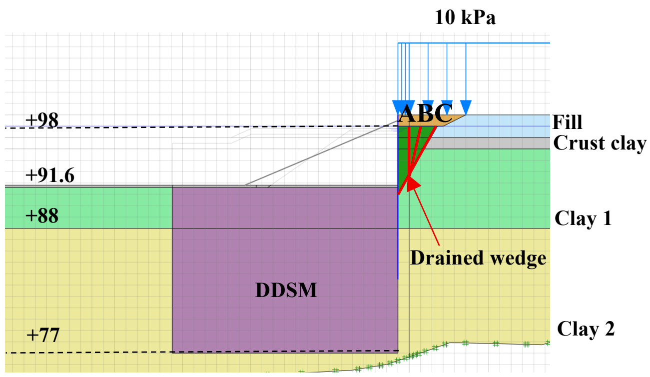

Figure 3. Geometry of the calculation model, including drained wedge geometries A, B C.

Table 1. Overview of input parameters for NGI-ADP soil units.

	(i) Crust clay	(ii) Clay 1	(iii) Clay 2
Unit weight		19.3 kN/m^3	
s_u^A [kPa]	70	42	42 + 2.5 / m
s_u^P / s_u^A		0.35	
s_u^{DSS} / s_u^A		0.63	
τ_0 / s_u^A		0.70	
G_{ur} / s_u^A		Var. 200 – 1250	

Table 2. Overview of input parameters for other soil units.

	(iv) Fill	(v) Drained wedge	(vi) DDSM
Unit weight	19.3 kN/m^3		19.5 kN/m^3
s_u^A [kPa]	-	-	76.00
φ [°]	35	Var. 20-36	
$E_{u,ref}$ [kPa]			$26.9 \cdot 10^3$
$E_{50} = E_{oed}$ [kPa]	$15 \cdot 10^3$		
E_{ur} [kPa]	$40 \cdot 10^3$		

4 RESULTS AND DISCUSSION

4.1 *Validation of the design approach based on inclinometer measurements*

Data extraction from FEM calculations was conducted by vertical line cross section was taken immediately behind SPW, at x = 0.1 m, as is roughly the case for IN5. Figure 4 shows the computed horizontal phase displacement at stages II, III and IV as illustrated in figure 2. As the drained behaviour is only modeled for the cantilevered phase, a single common path for the three model variants is shown for stages II and III. The gray field represents the range of results from the grid search calculation. For the relatively wide range of values for the chosen parameters, rather narrow bands of outcome are observed from FEM calculations. The sensitivity towards the selected parameters (G_{ur}/ s_u^A and φ) with respect to phase displacement from cantilevering do not appear to be very high in this case.

For construction stages II and III, a poor fit with deflection profile with depth was achieved, while deflection near terrain was a fair fit.

For stage IV, the following can be observed: The actual cantilever deflection profile was a reasonably good fit for all three models. All models are unable to predict the deflection at slab level and below. This suggests model inaccuracies with respect to modelling of the slab. The deflection profile above slab level seems to fit reasonably well with measurements, so some confidence towards the wedge approach is gained. Larger wedges seems to increase deflections, but the difference between model B and C is rather small. A possibly spurious deflection profile is observed for model A, suggesting possible numerical inaccuracies which could be addressed by increasing mesh density in the wedge.

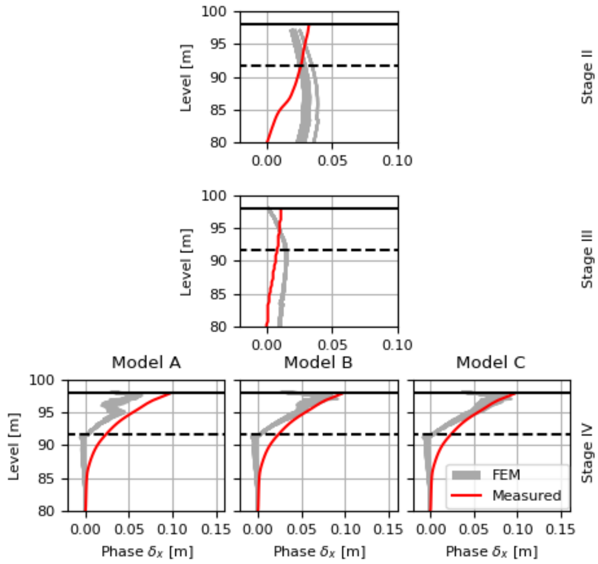

Figure 4. Model and measurement results, phase deflection with depth at certain stages in construction sequence. Horizontal lines: solid = terrain, dashed = slab level.

4.2 *Back calculation of optimal parameter combinations*

A numerical back calculation was performed as a parameter optimization through r^2-error minimization of the FEM results and measurement data, results shown in Figure 5. As the wedge and the models' ability to capture deflections in cantilevered stage is the topic of interest in this, only the deflection profile above slab level was considered. Measurement data was zeroed from baseplate level to be more relevant for the model optimization. For all models, the minimal "best fit" parameter set, indicated with a red star, is at or near extreme values in the ranges. This is a clear indication that a true minimum was not found within the range. Both parameters are in the stiff/strong end of the defined range, indicating that some aspect of the modeling approach is on the conservative end with regards to deflection/stiffness parameters.

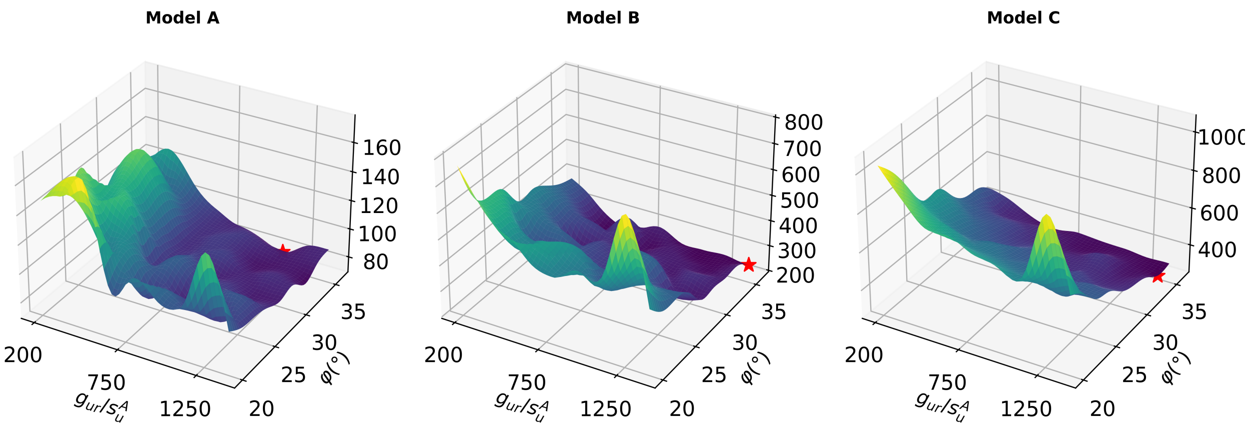

Figure 5. Grid search r[2]-error surfaces for the bivariate analysis. Lower is better. Surface minima within the analysis bound are indicated with a red star.

5 CONCLUSION

A new procedure for designing cantilevered SPW in clay was presented in this paper. Instead of using a fully coupled analysis, undrained material behaviour with the NGI-ADP model together with a drained wedge behind was assumed. This paper evaluated the new approach by comparing numerical analyses with measurements from a single inclinometer at the SPW. A simple grid search of two parameters was performed. Results, although inconclusive, indicate it is possible to use such an approach to achieve reasonable and plausible conservative estimates for deflection of SPW in the cantilevered phase.

Further work should include more refined multivariate analyses including sensitivity studies and include more of the measurement data. Moreover, a complete fully coupled flow analysis should also be conducted for comparison.

6 STATEMENT OF FUNDING

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