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The combined use of geostructures and ground improvement to the rehabilitation of existing quay walls

L'utilisation combinée de géostructures et d'amélioration des sols pour la réhabilitation de murs de quai existants

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ABSTRACT: The rehabilitation of existing quay walls can be a challenging task, due to the construction constraints, short construction-time frame and environmental aspects. This paper describes the design solution proposed for the rehabilitation of a centenary quay structure comprising a wooden pile foundation and a sheet pile wall connected by tie-rods to anchor plates. Due to the very poor mechanical characteristics of the "in situ" soil deposits, consisting of very soft clay up to a depth of 30 m, as well as severe spatial restrictions for the implementation of new structures, a non-conventional rehabilitation solution combining new structures and ground improvement was proposed. After providing an overview of the field and laboratory testing programme performed to characterise the "in situ" soil deposits, the proposed solution is described. The results of the finite element analyses performed to evaluate the short- and long-term performance of the proposed solution are subsequently presented. The obtained results suggest that a solution more robust than that strictly necessary to satisfy ultimate limit state criteria is required, highlighting the importance of using a performance-based design in current geotechnical engineering practice.

RÉSUMÉ: La réhabilitation de murs de quai existants peut être une tâche compliquée, en raison des contraintes de construction, de la brièveté des temps de construction et des aspects environnementaux. Cet article décrit la solution de construction proposée pour la réhabilitation d'une structure de quai centenaire comprenant des fondations sur colonne de bois et un mur de palplanche connecté à des plaques d'ancrage par tiges d'ancrage. En raison des caractéristiques mécaniques très pauvres des dépôts sédimentaires "in situ", constitués d'argile très fines jusqu'à 30 mètres de profondeur, ainsi que des restrictions spatiales très strictes pour la mise en place de nouvelles structures, une solution de réhabilitation non conventionnelle combinant de nouvelles structures et une amélioration du sol est proposée. Après avoir fourni un aperçu du programme d'essai réalisé sur le terrain et en laboratoire pour identifier les dépôts sédimentaires "in situ", l'adoptée solution est décrit. Les résultats des analyses d'éléments finis effectués pour évaluer les performances de la solution proposée à court et long terme sont finalement présentés. Les résultats obtenus suggèrent qu'une solution plus robuste que celle strictement nécessaire pour satisfaire aux critères d'état limite est nécessaire, soulignant l'importance de l'utilisation d'une conception basée sur la performance dans les pratiques actuelles d'ingénierie géotechnique.

Keywords: rehabilitation of quay walls; soft soil; soil stabilisation; sheet piling; finite element analysis.

1 INTRODUCTION

As part of the rehabilitation of an old inner harbour into a modern waterfront residential and touristic area in Norrköping, Sweden, WSP developed a feasibility study and preliminary design of the rehabilitation of the existing quay structure. This structure, built in 1860 and rebuilt in 1930, consists of a concrete deck built on wooden piles and a steel sheet pile wall connected by tie-rods to anchor plates (*Figure 1*). Having a period of life already greater than 100 years, this quay structure is currently in an advanced degraded condition, predictably compromising its ability to accommodate the new construction loads and modern safety requirements.

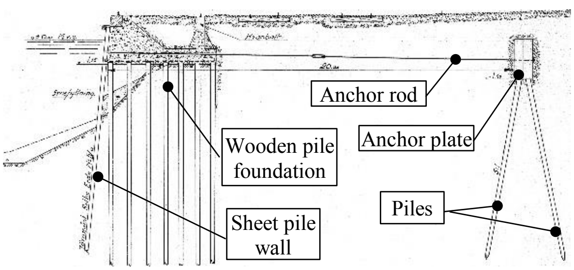

Figure 1. Existing quay structure.

Conventional solutions for quay walls, such as mono-anchored sheet pile walls or gravity block quay walls, are difficult to implement, due to severe spatial restrictions. In particular, in some critical areas, the land owned by the client is limited to a distance of about 12 m from the waterfront, with a solution within this boundary being required by the client to avoid possible conflicts with the foundations of the planned residential buildings. This restriction has effectively prevented the use of anchoring blocks (with or without piles), since those would find themselves both within the active earth pressure zone and within the global stability slip surface. In addition, due to the very poor mechanical characteristics of the "in situ" soil deposits, including very soft clay deposits up to a depth of about 30 m in some areas, gravity walls are also not recommended, at least without appropriate soil reinforcement. Moreover, since bedrock is located at large depths (varying between 30 and 42 m), anchoring new sheet pile wall into the bedrock would imply steeply inclined tie-back anchors and drilling through the existing structure and, therefore, this solution was also disregarded. Reinforcement of the seabed through jet-grouting, although technically feasible, was also discarded due to environmental concerns and high cost in Sweden. Finally, the proposed solution needed to take into account the following three aspects: (1) the existing quay structure needs to remain operational during construction; (2) the new quay structure cannot transmit any loads onto the existing structure; (3) a minimum waterway depth for navigation needed to be considered in the design process.

In this paper, an overview of the "in situ" geotechnical conditions is firstly provided, giving particular emphasis to the characterisation of the soft clay deposits. Subsequently, the proposed solution is described and the results of the finite element (FE) analysis performed to evaluate the short- and long-term performance of the proposed solution are presented.

2 GEOTECHNICAL CONDITIONS

An extensive field and laboratory testing programme was performed to characterise the "in situ" soil deposits. It was concluded that, in general, the geotechnical profile consists of a thin deposit of filling material, overlying a thick deposit of very soft to soft clay. At deeper levels, a deposit of loose to relatively loose silt overlays a deposit of moraine, resting on the bedrock, which is located, in some areas, as deep as about 42.0 m below the ground level. An overview of the characteristics of each deposit is provided below, giving particular emphasis to the clay deposit, due to its importance on the overall performance of the quay. The presentation is, however, limited to the materials occurring in the area close to the critical cross-section, which combines the largest thickness of clay with the

12 m-wide spatial restriction for the new structures. Note also that the presentation is done in terms of levels (rather than depths), with the surface being at level $z \approx +1.3$ m for the cross-section under analysis.

2.1 *Very soft to soft clay deposit occurring along the active side of the quay structure*

Intact samples were collected in the area of interest and subjected to identification (ID) tests, constant rate of strain (CRS) tests, undrained monotonic triaxial compression (UMTC) tests with mean stress increasing and undrained direct simple shear (UDSS) tests. Complementary, cone penetration tests with pore pressure measurement (CPT-u) were also carried out in the field.

Briefly, the ID tests revealed that, up to a level of $z \approx$ -21.0 m, an *gyttja* clay with an unit weight, γ, of about 16 kN/m^3 and natural water content close to the liquid limit, $w_L \approx 80-90\%$, occurs in the area. At deeper levels, the clay material is intercalated by very thin sandy sequences, having $\gamma \approx 18$ kN/m^3 and $w_L \approx 40\%$.

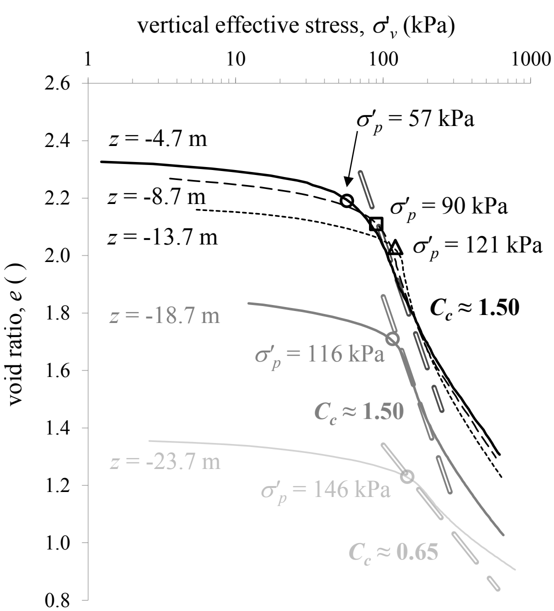

Figure 2. CRS test results.

The CRS test results obtained for samples collected at various levels (from -4.7 to -23.7 m) are depicted in *Figure 2*. Note that a very low strain rate of 0.7%/h was used to minimise undesirable strain rate effects (e.g. Leroueil *et al.*, 1983). Note also that, at this preliminary stage of the project, neither destructuration (i.e. breakage of interparticle bounding) nor creep behaviour were not taken into account and, therefore, both the pre-consolidation pressure, σ'_p, and the compressibility index, C_c, were directly estimated from the registered CRS curves, as shown in the figure. Nevertheless, as shown later, the values of σ'_p estimated from these tests do not differ much from those inferred from CPT-u test results.

As expected, the samples exhibit very high compressibility, with values of C_c of about 1.50 being estimated for the shallowest clay deposits. Regarding σ'_p, values in the range of 57 to 146 kPa were estimated from the CRS tests, as also depicted in *Figure 2*. In addition, the Wissa's Linear Theory (Wissa *et al.*, 1971) was employed to estimate the vertical hydraulic permeability, k_v, of the samples, with values in the range of 1.0×10^{-10} to 1.0×10^{-9} m/s being obtained.

With respect to the UMTC tests, samples collected at different levels were anisotropically consolidated to similar effective stresses to those existing in the field and subjected to undrained triaxial shearing. The obtained results are shown in *Figure 3*. As expected, samples submitted to greater effective stresses tend to exhibit stiffer stress-strain responses and reach higher peak shear stresses. Moreover, typical features of normally consolidated (NC) to lightly overconsolidated (OC) clay samples can be observed (e.g. Potts and Zdravković, 1999): (1) generation of positive excess pore pressure throughout loading, resulting in an effective stress path bending to the left; (2) significant drop of shear resistance after reaching the peak shear stress (in some cases, a decrease of about 50 %); (3) significant reduction of stiffness with strain (by more than an order of magnitude).

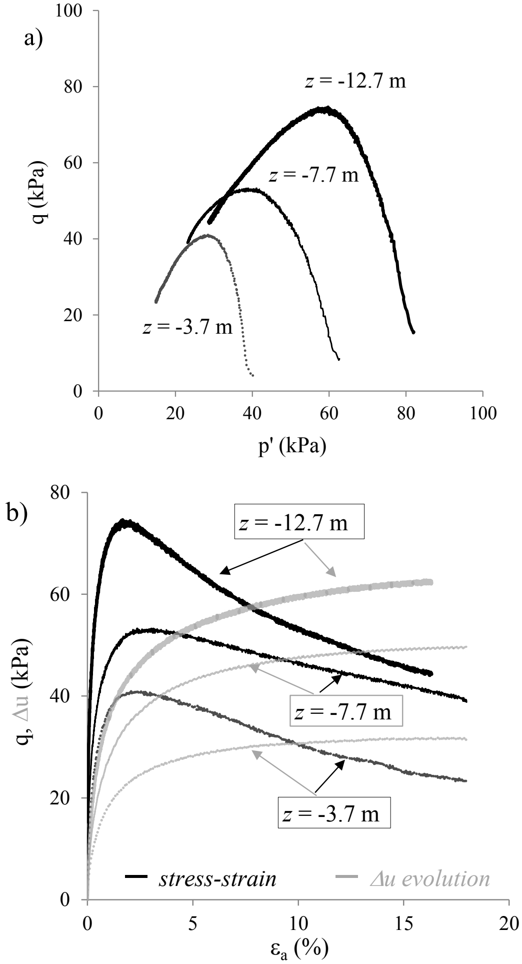

Figure 3. UMTC test results: (a) stress path; (b) stress-strain response and excess pore water pressure build-up with axial strain.

The peak and residual (i.e. at the end of test) values of undrained shear strength, c_u, measured in the UMTC tests are compared with the peak values determined in the UDSS tests, as well as the values inferred from two CPT-u tests performed in the area of interest (termed as W205 and W206) in *Figure 4*. A very good agreement between the field and laboratory test data is apparent, with the c_u values inferred from CPT-u test results plotting close to those measured in UDSS tests and in between the residual and peak c_u values obtained from the UMTC tests.

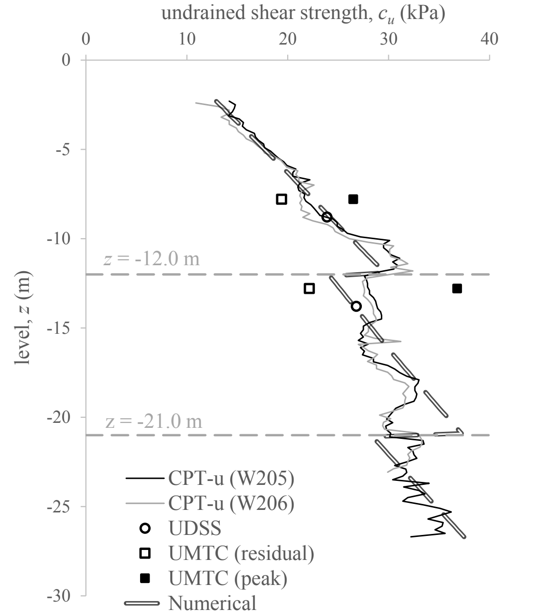

Figure 4. Variation of undrained shear strength with level for the clay deposits occurring along the active side of the quay structure.

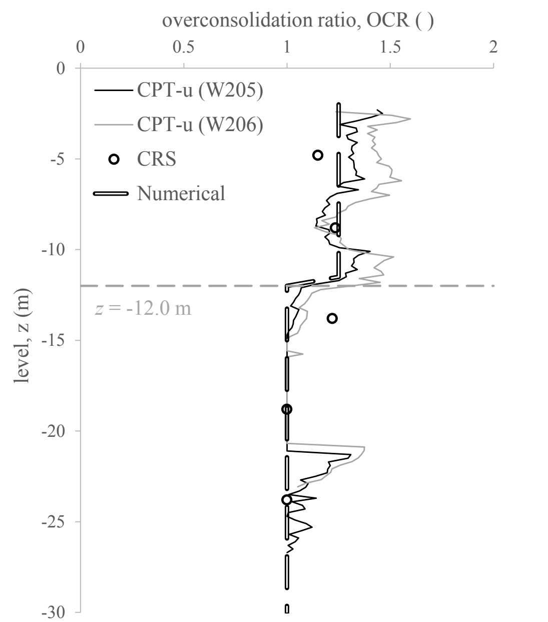

Figure 5. Variation of overconsolidation ratio with level for the clay deposits occurring along the active side of the quay structure.

Additionally, *Figure 5* compares the values of the overconsolidation ratio (OCR) estimated from CRS and CPT-u tests. Once more, field and laboratory test data seem to agree well, with the clay deposit being lightly overconsolidated (OCR ≈ 1.25) for levels above z = -12.0 m, while normally consolidated at deeper levels. As detailed later, the c_u and OCR profiles adopted in the numerical analysis are also plotted in *Figure 4* and *Figure 5*, respectively.

2.2 Very soft to soft clay deposit occurring along the passive side of the quay structure

Two CPT-u tests (termed as W303 and W304) were performed to characterise the clay deposit occurring along the passive side of the marine quay. The inferred variation of c_u with z is depicted in *Figure 6*. As expected, due to the smaller overburden pressure at which the deposits occurring on the passive side of the wall are submitted, for a given level, the values of c_u of these deposits are smaller than those estimated for the deposits on the active side of the quay.

In relation to OCR, its variation with level is shown in *Figure 7*. It can be observed that very large *OCR* values were obtained for the shallower materials, probably due to the frequent variations of the water level. Also depicted in *Figure 7* is the variation of the pre-overburden pressure (i.e. POP ≈ $\sigma'_p - \sigma'_{v,0}$) with level, which was, in this case, used to model these deposits, as indicated in Table 1.

2.3 Silt and moraine deposits

Ram sounding tests were performed to estimate the strength and stiffness characteristics of the silty-sand deposit, with about 5.0 m of thickness, and the moraine deposit with approximately 7.0 m of thickness occurring at deeper levels. The inferred values are shown in Table 2. In addition, a large amount of rock probing tests were performed to define the location of the bedrock.

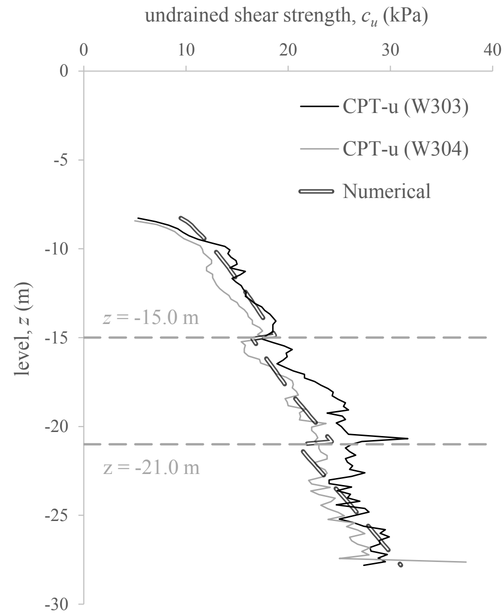

Figure 6. Variation of undrained shear strength with level for the clay deposits occurring along the passive side of the quay structure.

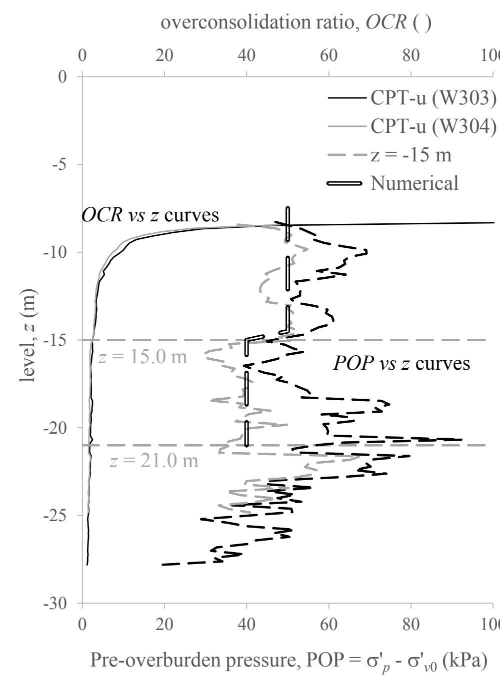

Figure 7. Variation of the overconsolidation ratio with level for the clay deposits occurring along the passive side of the quay structure.

3 FINITE ELEMENT ANALYSIS OF THE PROPOSED SOLUTION

Taking into account all imposed constraints (particularly the one limiting any intervention within a 12 m-wide area from the waterfront), a non-conventional, though cost-efficient solution combining new structures, soil improvement and rock filling was proposed. Specifically, within the critical area (i.e. where geotechnical conditions are more adverse), the solution consists of the installation of a new 12.5 m-long sheet-pile wall of AZ26-700N type in front of the existing structure, connected to a 0.6 m-thick and 12.0 m-wide reinforced concrete slab. In order to transmit the loads directly into the bedrock, end-bearing piles are installed in two different rows: (1) immediately behind the new sheet-pile wall, consisting of bored steel pipe piles with a diameter of $D = 406$ mm, a thickness of $t = 14.2$ mm and a length of $L \approx 42.0$ m, spaced every $s = 4.2$ m and (2) at the back end of the concrete slab, consisting of bored steel pipe piles with $D = 220$ mm, $t = 12.5$ mm, $L = 44.0$ m and $s = 4.2$ m. By having a stiff structure built on piles, only few loads are expected to be taken by the existing structure, as required by the client. Complementary, soil stabilisation with lime-cement columns is used to improve the mechanical characteristics of the clay deposits adjacent to the existing structure and, therefore, to reduce the active earth pressures. In addition, a rock fill is to be placed under water to increase the passive earth-pressure on the new sheet-pile wall. These two measures were deemed necessary to increase the overall stability of the quay. Finally, the gap between the new and the existing structure is to be backfilled with lightweight materials.

It is important to note that the existing tie-backs are to be detached during the final stages of the construction, leaving the area behind the quay structure available for the construction of new residential buildings. A new set of tie-backs spaced every 4.2 m is used to prevent large displacements of the sheet pile wall before the construction of the concrete slab. Note also that a new marina is planned to be built in a different area and, therefore, there was no need to consider mooring forces in the present analysis.

In order to evaluate the short- and long-term performance of the proposed solution, a 2D plane strain FE analysis was performed using PLAXIS 2D 2018. The estimated time required for each construction phase was introduced in the analysis, allowing for the partial dissipation of the excess pore water pressures generated during construction.

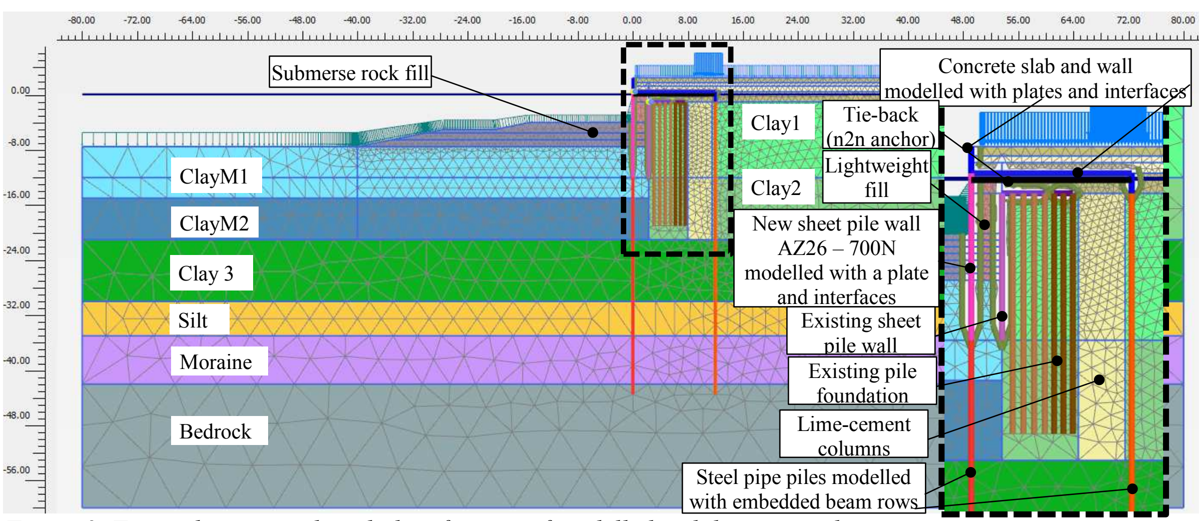

Figure 8. Finite element mesh and identification of modelled soil deposits and structures.

A long-term consolidation period, corresponding to the lifetime of the quay structure (120 years) was introduced after the simulation of the construction. The FE mesh is shown in *Figure 8*.

3.1 *Material models*

The soft soil model was selected to describe the short- and long-term responses of the clay deposits, with the lab and field test results being used to calibrate the model constants listed in Table 1. Three different layers had to be considered to accurately model the variation of c_u and *OCR* (or *POP*) with z, as shown in *Figure 4* to *Figure 7*. For all remaining materials, due to the limited experimental characterisation and/or little influence on the response of the quay structure, a simple elastic linear model coupled with the Mohr-Coulomb failure criterion was employed, with the model constants being listed in Table 2. Note that the guidelines provided in Trafikverket (2014) were taken into account to estimate the strength, stiffness and hydraulic conductivity of the soil stabilised with lime-cement columns (termed as Stab soil in Table 2). In relation to the rock fill and lightweight sandy materials, due to the lack of experimental information, typical characteristics from the literature were adopted (e.g. Trafikverket, 2014).

Table 1. Model constants adopted for the clay deposits

	Active side		Passive side		Both
Layer	**Clay1**	**Clay2**	**ClayM1**	**ClayM2**	**Clay3**
z_{top} (m)	-1.0	-12.0	-7.5	-15.0	-21.0
z_{bot} (m)	-12.0	-21.0	-15.0	-21.0	-30.0
γ_{sat} (kN/m^3)	16.0	16.0	16.0	16.0	16.0
e_{init} ( )	2.30	1.85	2.30	1.85	1.35
λ^* ( )	0.20	0.23	0.20	0.23	0.12
κ^* ( )	0.009	0.011	0.009	0.011	0.008
ν ( )	0.20	0.20	0.20	0.20	0.20
ϕ' (°)	28.0	28.0	24.0	24.0	20.0
ψ (°)	0.0	0.0	0.0	0.0	0.0
OCR ( )	1.25	1.00	–	–	1.00
POP (kPa)	–	–	50.0	40.0	–
k_x=k_y (m/s)			5.0x10^{-10}		

Table 2. Model constants adopted for the remaining soil and rock deposits.

Deposit	Silt	Moraine	Stab soil	Rock Fill	LW sand
z_{top} (m)	-30.0	-35.0	-7.5	-4.0	-0,5
z_{bot} (m)	-35.0	-42.0	-15.0	-7.5	-7,5
γ_{sat} (kN/m^3)	19.0	21.0	16.0	18.0	11.0
E (MPa)	10.0	20.0	4.5	50.0	40. 0
ν ( )	0.30	0.30	0.30	0.30	0.30
ϕ' (°)	30.0	36.0	30.0	40.0	40.0
c' (kPa)	0.0	0.0	5.0	0.0	0.0
ψ (°)	0.0	6.0	0.0	10.0	10.0
k_x=k_y(m/s)	10^{-7}	10^{-6}	10^{-7}	10^{-2}	10^{-4}

3.2 *Obtained results*

3.2.1 *Short-term performance*

Figure 9 illustrates possible failure mechanisms after the end of the construction of the quay, identified from a strength reduction analysis (Brinkgreve *et al.*, 2018). It is interesting to observe the stabilising effect of the rock fill. Note that the capacities of the structural elements were not exceeded. Overall, a factor of safety of FS ≈ 2.0 was obtained in the safety analysis, suggesting that the proposed solution is more robust than that strictly required to verify the Ultimate Limit State (ULS) stability requirement (in this case, characterised by FS ≥ 1.65).

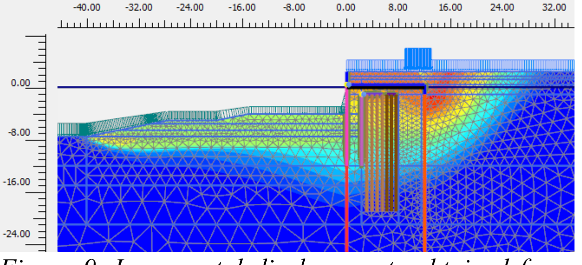

Figure 9. Incremental displacements obtained for a safety analysis after the end of the quay construction.

In addition, the loads transmitted to the sheet pile wall and reinforced concrete slab (in terms of bending moment, *M*, envelope), as well as to the piles (in terms of axial force, *N*, envelope) at the end of the construction for the ULS – Design Approach 3 are schematically illustrated in *Figure 10*. Note that, in this project, an independent ULS calculation was performed after each serviceability limit state (SLS) calculation.

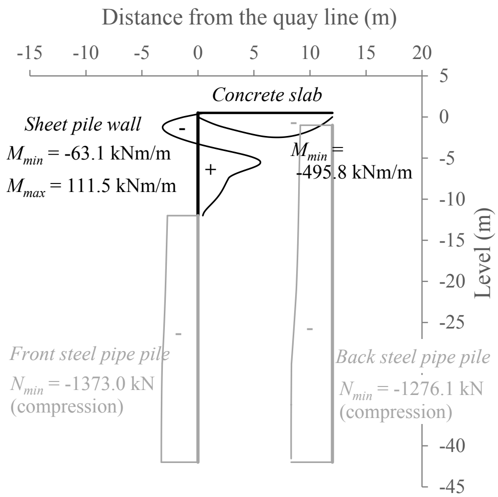

Figure 10. Bending moment or axial force envelope of the new structures after construction for the ULS.

3.2.2 Long-term performance

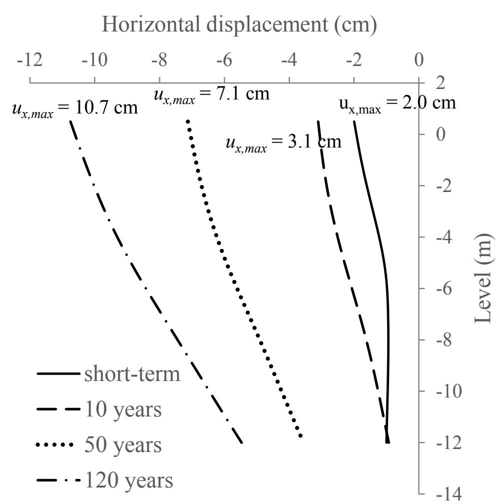

Figure 11. Estimated horizontal displacements due to consolidation for several periods of life of the new sheet pile wall.

The design of the structure was basically dictated by the long-term displacements (i.e. due to consolidation). As shown in *Figure 11*, the proposed solution is expected to limit the movement of the sheet pile wall to about 10.7 cm.

4 CONCLUSIONS

This paper provides an overview of the challenges encountered during the design of the rehabilitation of an existing quay structure. Due to the spatial restrictions and consequent impossibility of anchoring the new structures beyond the active earth pressure zone, the proposed solution relies on its own stiffness and embedment into the rock strata to achieve stability and limit deformations. Indeed, in this case, the design of the quay structure was dictated by the allowable deformations, highlighting the importance of the performance-based design in current engineering practice.

5 ACKNOWLEDGEMENTS

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