A simplified modelling approach for the analysis of port quay-walls with relieving platform

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1 Introduction

Port infrastructures are vital for maintaining global supply chains, as maritime transport accounts for roughly 80% of international trade by volume [1]. The increase in vessel dimensions over recent decades has placed growing structural and operational demands on berthing facilities, forcing port authorities to develop increasingly robust retaining structures. Among these, quay walls with relieving platforms represent one of the most sophisticated configurations, wherein the wall, piles, anchor systems, and concrete superstructures work together to resist the lateral earth pressure, the loads from the cranes and stored goods. The complex mechanical interaction of these structures with the soil has driven widespread adoption of numerical modelling in quay wall engineering practice.

Finite element models considering the nonlinear soil behaviour and the complex geometry of the structure became, in recent years, part of common design practice [2,3]. In fact, due to the presence of the relieving platform and the piles supporting it, in principle, simplified design approaches, such as those only considering the wall and lumping the lateral soil reactions at the wall nodes as nonlinear springs [4,5] cannot be adopted. Consequently, time-consuming numerical analyses are required in the design process, limiting the number of configurations (e.g., pile diameter, length, and spacing) that can be explored, as well as the application of probabilistic approaches for reliability assessment.

The goal of this work is to demonstrate how, with an appropriate yet simple modelling approach, the response of a quay wall with a relieving platform can be reproduced as that of an anchored flexible retaining wall.

2 Site description and numerical model

The structural system analysed in this study is a deep-water quay wall constructed at the Port of Rotterdam (Figure 1a). The retaining system utilizes a combi-wall configuration (Figure 1b). The primary structural elements consist of large-diameter, open-ended tubular steel piles with an outer diameter of 1.42 meters and a wall thickness of 24 millimetres, installed with a spacing of 3.3 meters (Figure 1b). These piles resist most of the lateral earth pressure on the wall, with load redistribution between adjacent elements governed by arching mechanisms within the retained soil. The closure between two adjacent piles is achieved using hot-rolled steel sheet piles (type PU28), positioned between the primary piles and embedded to a depth that ensures hydraulic stability and prevents preferential flow or soil migration toward the excavated face.

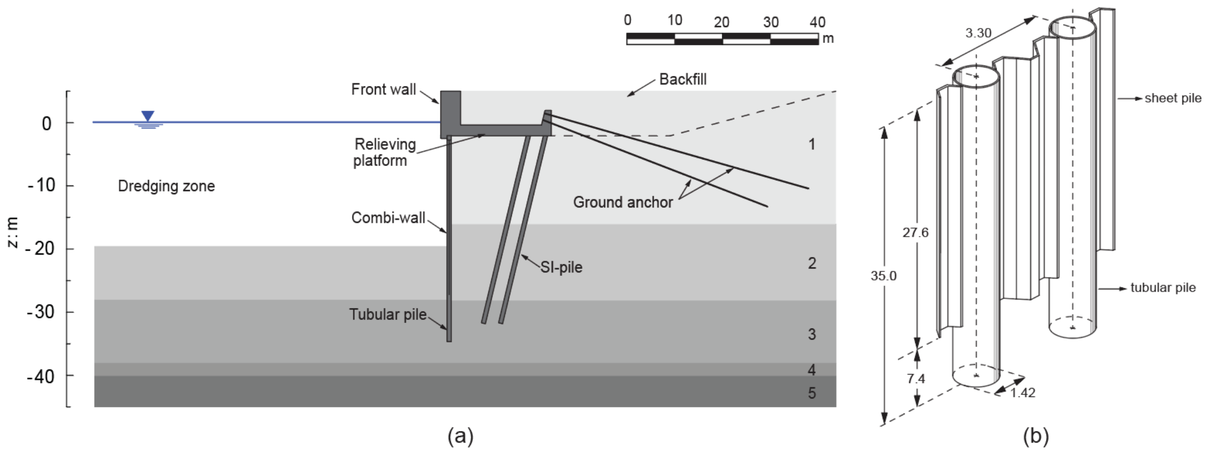

Figure 1. Reference problem: (a) quay wall with relieving platform geometry and (b) combi-wall sectional properties.

Before the wall construction, site characterization through cone penetration test (CPT) and boreholes was performed across the entire quay wall. The CPT results in terms of cone resistance (q_c), sleeve friction (f_s) and consistency index (I_c, [6]) are shown in Figure 2. Based on the measurements, the site is schematized in five layers (1 to 5 in Figure 1 and

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Figure 2), predominantly composed of sand with varying relative densities.

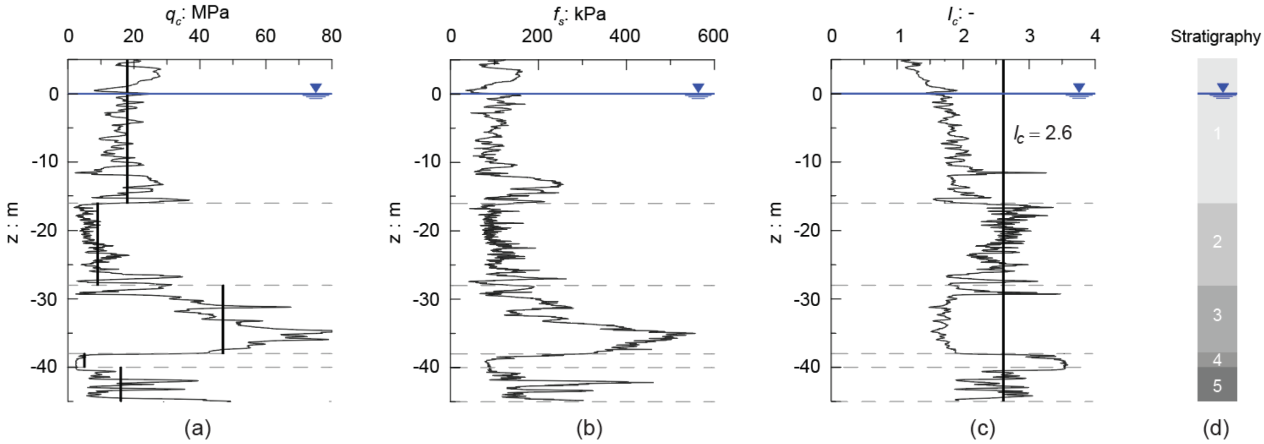
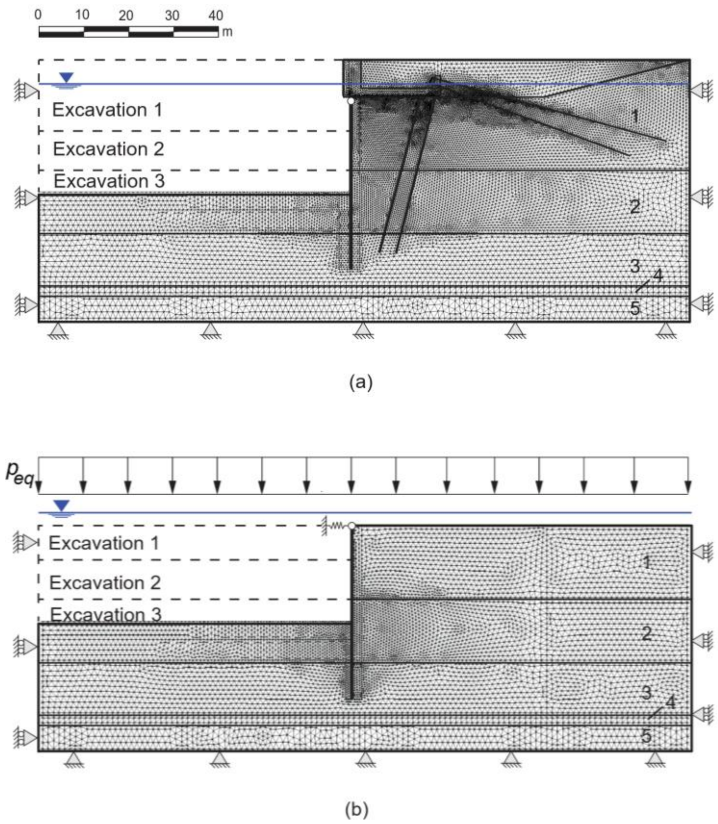


Figure 2. Site investigation results: (a) CPT cone resistance, (b) CPT sleeve friction, (c) consistency index and (d) derived soil stratigraphy.

After the construction of the structure the excavation took place in three subsequent dredging phases: (*i*) the first one up to a depth of −8.50 m from the water table, (*ii*) the second one up to a depth of −16.00 and (*iii*) the last one up to the design depth of −20.50 m (see Figure 3). The construction and the excavation process was numerically simulated (Figure 3) by means of two-dimensional finite element drained analyses performed by using Plaxis 2D.



Layer	E_{50}^{ref} (MPa)	E_{oed}^{ref} (MPa)	E_{ur}^{ref} (MPa)	ϕ' (∘)	ψ (∘)
1	71	57	213	39	9
2	34	30	103	33	1
3	84	67	252	38	7
4	16	15	48	31	0
5	40	36	121	33	2

(c)

Figure 3. Finite elements models: (a) quay wall with relieving platform, (b) simplified model and (c) material parameters from CPT results.

For each layer a uniform average q_c value is used to derive the Hardening Soil model [7] constitutive parameters. In particular, the oedometric reference stiffness (E_{oed}^{ref}) and the soil friction angle (ϕ') are directly related to the normalized cone tip resistance (Q_{tn}) as it follows [8,9]:

$$E_{oed}^{ref} = 5 Q_{tn}\, p_{atm} \qquad \phi' = 17.6^\circ + 11.1^\circ \log_{10}(Q_{tn}) \tag{1}$$

where p_{atm} is taken as 101 kPa. The dilatancy was calculated as $\phi' = 30^\circ + 0.8\,\psi$ [10], whereas the secant reference stiffness (E_{50}^{ref}) assumed equal to E_{oed}^{ref} and the unloading reloading reference stiffness $E_{ur}^{ref} = 3E_{oed}^{ref}$ as in [11].

The Poisson's ratio, the failure ratio and the exponent for stiffness stress dependency are assumed equal to 0.2, 0.9 and 0.5, respectively.

Two different quay wall geometries were considered, a full geometry accounting for the presence of the relieving platform (Figure 3a) and a simplified model in which the effect of the platform is accounted for in the stress initialization of the model (Figure 3b). In both cases, the steel combi-wall was reproduced by means of elastic plate elements with the sectional properties of the steel cross-section represented in Figure 1b. The relieving platform was modelled by means of solid elastic elements with concrete properties. Along the normal direction, the soil-wall and the soil-platform interface cannot sustain any tensile force, whereas in the tangential one is frictional. The tangent of interface friction is equal to the 80% of the soil one. Both pile and anchors were modelled through embedded beams with steel properties. The pile interface properties were calibrated as in [12], whereas the anchor ones from the load test performed on the anchors before the excavation.

The soil domain was discretized by approximately 5000 15-nodes triangular finite elements (Figure 3a). On the lateral boundaries of the domain horizontal displacements are not allowed whereas in lower one all displacements are prevented (Figure 2a). Drained analyses are performed and the water table is kept constant at $z = 0$. The analysis phases follow the construction phases.

The simplified problem is schematized in Figure 3b. In this case, the relieving platform and the piles are absent. The anchors are schematized as a linear spring characterized by a stiffness equal to the one of the anchors free lengths. The "combi-wall" only extend up to relieving platform plane of foundation. Initially, at this depth, a surcharge (p_{eq}) equivalent to the self-weight of the soil before construction is present. This surcharge is then removed before the first excavation. Due to this loading and subsequent unloading, the soil in the "active" part of the wall is overconsolidated, i.e. in the Hardening Soil model perspective, the state of stress at the beginning of the excavation lies inside the yield surface and not on it.

3 Results and discussion

The horizontal displacements of the retaining wall following the three excavation phases illustrated in Figure 3 are presented for both models in Figure 4. The results obtained from the simplified model (dotted lines) demonstrate very good agreement with those of the complete model (solid lines), with a maximum discrepancy of approximately of 4%.

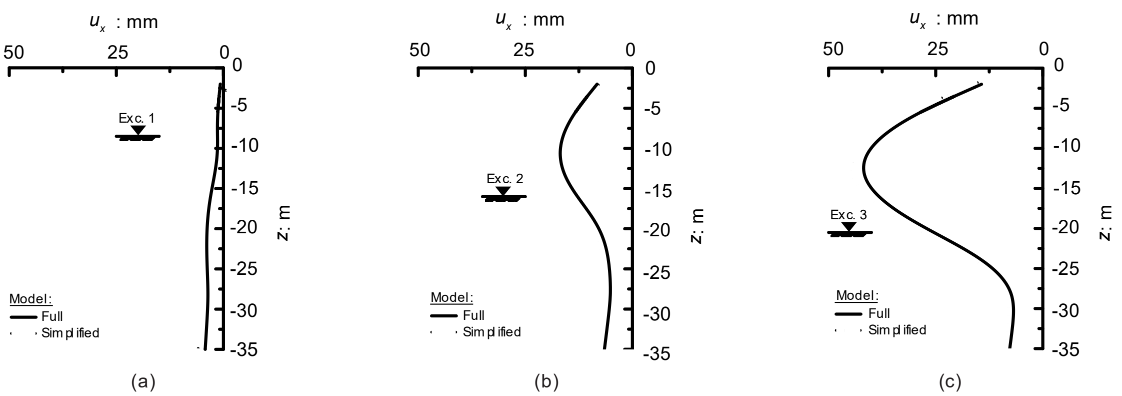


Figure 4. Horizontal displacement profiles comparison: (a) excavation 1, (b) excavation 2 and (c) excavation 3.

This agreement can be attributed to three principal factors: (i) the piles are situated sufficiently far from the wall to limit the interaction with the active wedge that develops in the soil adjacent to the wall (Figure 5); (ii) the active wedge detaches from the relieving platform (Figure 5); and (iii) the relieving platform transfers all vertical stresses to both the wall and the piles. Consequently, the vertical stress distributions, not only within the passive zone, but also within the simplified model are almost coincident both in the active and passive zones. A minor discrepancy is nonetheless observable towards the base of the wall, which may be attributed to the presence of piles near the wall toe, where local transfer of vertical stresses occurs. These findings confirm the validity of the proposed simplified approach.

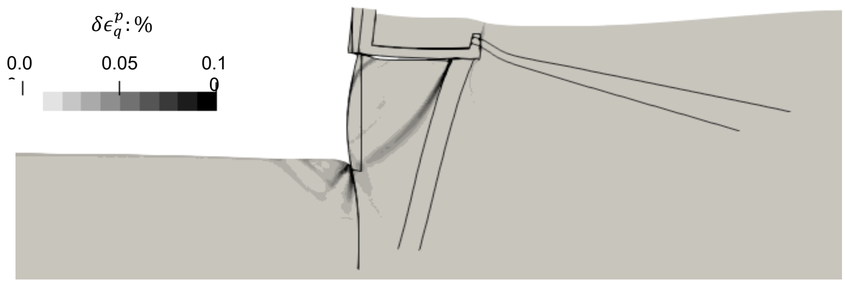


Figure 5. Incremental plastic strain at the end of excavation 3 of the full model (deformation upscaled 50 times).

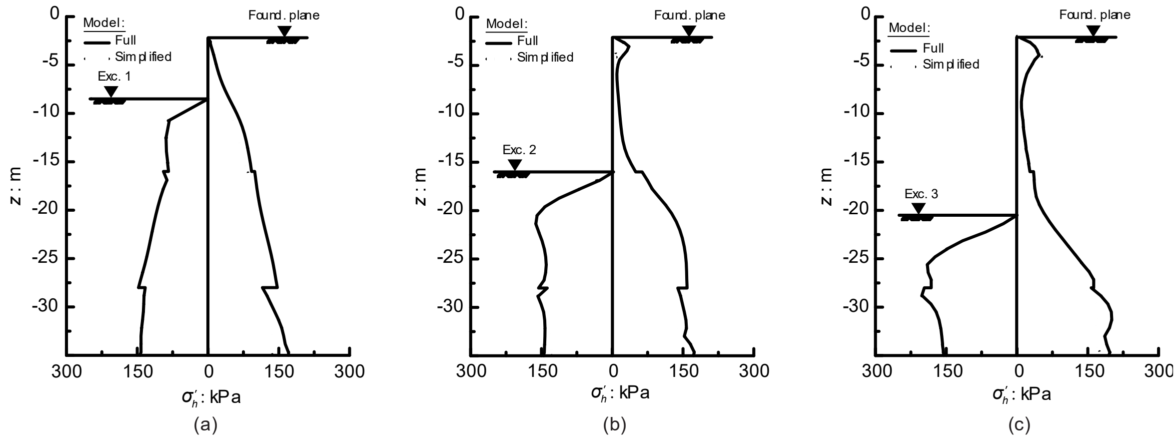


Figure 6. Effective horizontal stress comparison: (a) excavation 1, (b) excavation 2 and (c) excavation 3.

4 Conclusions

In this contribution, a simplified modelling technique is proposed for the analysis of quay walls with relieving platform, with the primary objectives of reducing computational effort and modelling time. Particularly, it is shown that the response of this complex system can be approximated to the one of a simply anchored flexible retaining structure.

The proposed simplified procedure can successfully be used under the following conditions: (*i*) the piles supporting the relieving platform do not significantly interfere with the active failure wedge developing behind the wall; (*ii*) the relieving platform width is larger than that of the active wedge (*iii*) the relieving platform, the wall and the piles are sufficiently rigid to allow a full detachment between the platform base and the underlying soil, resulting on a free stress condition on the upper side of the active wedge. A fundamental aspect of the proposed modelling procedure is the careful initialisation of the stress state within the reduced model. This step ensures that the initial response in the active side is elastic, as in the complete model where an unloading process is observed.

The agreement between the complete model and the simplified one also implies that the wall response can also be reproduced by using models considering beams interacting with nonlinear springs modelling the lateral soil reactions. These models are computationally very efficient, enabling systematic parametric studies and design optimisation with reduced calculation time. Furthermore, their simplicity and efficiency make them well suited to probabilistic assessments for reliability-based design.

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