Delft University of Technology

Document Version
Final published version

Licence
CC BY

Citation (APA)
Flessati, L. (2026). Towards displacement-based design of quay walls. In J. Grabe (Ed.), *Conference Proceedings Workshop Numerical Methods in Geotechnics 2026: AI in geotechnics, deformaton based design, numerical optimization* (pp. 69-79). Hamburg University of Technology.

Important note
To cite this publication, please use the final published version (if applicable).
Please check the document version above.

Copyright
In case the licence states “Dutch Copyright Act (Article 25fa)”, this publication was made available Green Open Access via the TU Delft Institutional Repository pursuant to Dutch Copyright Act (Article 25fa, the Taverne amendment). This provision does not affect copyright ownership.
Unless copyright is transferred by contract or statute, it remains with the copyright holder.

Sharing and reuse
Other than for strictly personal use, it is not permitted to download, forward or distribute the text or part of it, without the consent of the author(s) and/or copyright holder(s), unless the work is under an open content license such as Creative Commons.

Takedown policy
Please contact us and provide details if you believe this document breaches copyrights.
We will remove access to the work immediately and investigate your claim.

This work is downloaded from Delft University of Technology.

Towards displacement-based design of quay walls

Luca Flessati

Abstract: Maritime quay walls are critical components of ports, enabling the safe berthing of vessels and efficient cargo handling. Their structural performance is governed by soil-structure interaction, where wall movements and soil response mutually influence each other. Conventional design methods simplify this interaction by assuming predefined active and passive earth pressures, resulting in analytically solvable but highly conservative designs. This paper demonstrates the application of displacement-based design methodologies that explicitly account for soil-structure interaction, with a particular focus on some of the approaches currently adopted in the Dutch practice and by the Port of Rotterdam Authority. By including realistic deformation behaviour into the design process, these methodologies offer the potential for more efficient, economical, and reliable quay wall designs.

1 Introduction

Maritime ports serve as essential nodes within global supply chains, facilitating the transport of nearly 90% of international trade (International Chamber of Shipping, 2024). Quay walls (QWs), the structures that allow the berthing of ships and facilitate the loading and unloading of cargo, are essential components within port infrastructures. Quay walls typically rely on slender structural elements to resist lateral soil pressure. When soil pressure acts on the wall, it induces wall movements. These movements lead to soil displacements, which in turn change the soil pressure. Therefore, the soil structure interaction (SSI) plays a fundamental role in the mechanical response of QWs.

Traditional QW design methods rely on a set of simplifying assumptions that effectively eliminate SSI from the analysis. Lateral earth pressures are prescribed a priori by assuming the soil to be at failure under either active (land side) or passive (water side) conditions. Consequently, the stress distribution acting on the wall cannot evolve in response to wall deformations, and the resulting solution is independent of soil and wall stiffness. Additional simplifications concern the anchors response (usually assumed to be rigid) and the kinematic conditions at the wall toe. Depending on the method, the wall base may be assumed to prevent horizontal displacement while allowing rotation (free base), or to restrain both displacement and rotation (fixed base). The method proposed by Blum (1931), the basis of the traditional Dutch design approach, imposes even stronger constraints by requiring displacement, rotation, and curvature to vanish at the wall tip.

These assumptions reduce the problem into a statically determinate one that can be solved analytically. Moreover, since lateral earth pressure is treated as a predefined action, rather than a result of a SSI problem, these traditional design approaches can easily be used in the design codes partial safety factor framework. Despite their simplicity, in the Dutch context these approaches are not now used for the final design of quay walls, but are sometimes used to have a first indication of the dimensions of the wall. In practice, these traditional approaches are usually very conservative, providing technical solutions characterized by very long and very stiff walls. Such solutions may substantially increase construction costs and potentially create practical difficulties during installation.

The goal of this paper is demonstrating the application of displacement-based design methodologies for quay walls, with particular emphasis on the design solutions currently adopted for the Port of Rotterdam Authority (PoR). For the sake of brevity, the focus will only be on the wall response and the response in the soil in the proximity. Other mechanisms, such as piping and global QW loss of stability are not hereafter considered.

2 Quay wall design solutions

At present, QWs in PoR are predominantly constructed using combi-walls, consisting of steel tubular piles combined with sheet piles. The tubular piles have both a retaining function and a bearing function for vertical loads. In contrast, the sheet piles serve only to transmit earth and water pressures to the primary elements. Combi-walls are used due to their large stiffness, strength, bearing capacity and for their economic convenience (de Gijt & Broeken, 2013). The most commonly adopted structural configurations are: (i) anchored walls (AW, Figure 1a) and (ii) walls with relieving platforms (RPW, Figure 1b).

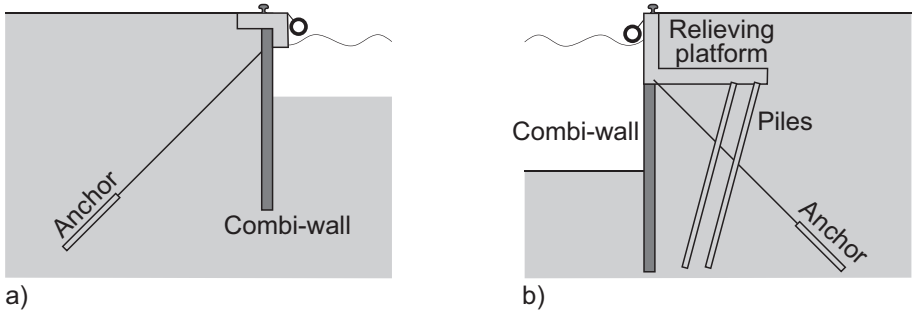

Figure 1: Type of walls currently adopted in the PoR: a) anchored walls and b) walls with relieving platform

Currently, AWs are typically designed with a single row of anchors, as the installation of underwater anchors remains technically challenging. However, new technologies aimed at the safe, reliable, and cost-effective installation of underwater anchors are currently under development (Roubos, 2026). To ensure adequate pull-out resistance, design guidelines (CUR 166) require the anchor bodies to be embedded in the deeper sand layers. Consequently, the

anchors are generally installed at relatively large inclinations. AWs are commonly adopted for quay walls with moderate retaining heights or where the operational loads from cranes, moored vessels, and stored cargo are relatively limited.

In contrast, RPWs are generally adopted for large retaining heights and the high operational loads associated with deep-sea ports. The relieving platform redistributes the vertical loads, transferring a significant portion of the backfill and crane loads directly to the wall and its supporting piles. As a result, the vertical stresses in the soil immediately behind the wall are substantially reduced, resulting to lower lateral earth pressures acting on the wall. This, in turn, limits the horizontal deflection of the wall and improves its overall structural performance. Despite these advantages, RPWs are associated with high construction costs and a substantial environmental impact due to their intensive use of concrete and steel. According to Roubos (2026), the embodied carbon footprint of an RPW is approximately 50 tonnes of CO_2 per metre of quay wall.

The optimisation of these structures therefore offers considerable potential for both economic and environmental benefits. However, achieving this remains challenging due to the complex interaction between the soil and the structure that cannot be accounted for with the traditional approaches. In the following, displacement based design approaches for quay walls with progressively increasing levels of detail and computational cost are presented.

3 Subgrade reaction based approaches

According to this approach, the retaining wall is schematized as a beam, while the soil reactions and anchor forces are concentrated at the beam nodes and represented by means of independent linear or non-linear springs (Figure 2a). The main challenge of this approach lies in defining the constitutive relationship of the non-linear springs modelling the lateral soil reaction. In the simplest applications, elastic-perfectly plastic laws are adopted, where the limiting resistances are typically derived from active and passive earth pressure theories, while the initial elastic stiffness depends on both the soil stiffness and the geometry of the wall.

The current Dutch design practice (CUR 166) adopts a more sophisticated constitutive model that aims to capture the non-linear soil behaviour more realistically. This constitutive law is piecewise linear as it is shown in Figure 2b. For nil displacements, the lateral earth pressure p_0 is calculated from the at rest lateral earth pressure coefficients. On the active side, the response is elastic perfectly plastic and the stiffness is defined k_{h1}, while the ultimate active pressure value p_a is calculated through the limit active pressure. On the passive side, the response is subdivided into 4 linear branches. The first one, from p_0 to $0.5p_p$, being p_p the ultimate passive pressure value, is characterized by stiffness k_{h1}. The second branch, from $0.5p_p$ to $0.8p_p$, is characterized by stiffness k_{h2}, while the third one, from $0.8p_p$ to p_p, is characterized by stiffness k_{h3}. Finally, the last branch for lateral pressure equal to p_p defines the failure condition (i.e. zero stiffness). The characteristic secant stiffness values k_{h1}, k_{h2}, k_{h3} (Figure 2b). These values were derived from the Dutch practice and put in relation with the type of soil and the cone tip resistance value q_c or the undrained strength S_u (Table 1). According to De Gijt & Broeken (2013), the values on

the table was only validated for retaining heights up to 8-10m. For larger structures the CUR 166 suggests to correct the spring stiffness values to account for wall dimensions.

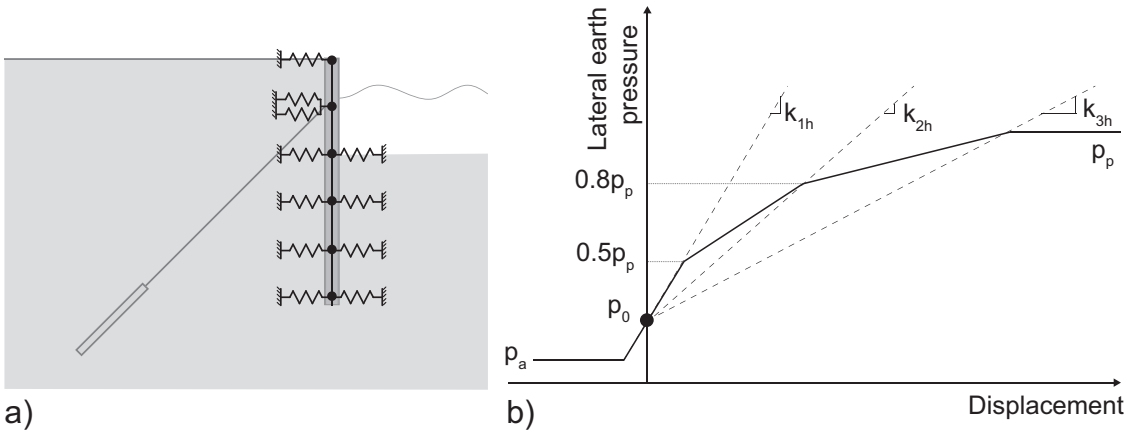

Figure 2: a) Schematization of wall as beam interacting with non-linear springs and b) Piecewise linear spring constitutive law

Table 1: Low and high secant characteristic subgrade reaction values

		k_{h1} (MN/m^3)	k_{h2} (MN/m^3)	k_{h3} (MN/m^3)
Sand	q_c (MPa)			
loose	5	12 - 27	6 - 13.5	3 - 6.75
medium	15	20 - 45	10 - 22.5	5 - 11.25
dense	25	40 - 90	20 - 45	10 - 22.5
Clay	S_u (kPa)			
soft	25	2 - 4.5	0.8 - 1.8	0.5 - 1.125
stiff	50	4 - 9	2 - 4.5	0.8 - 1.8
very stiff	200	6 - 13.5	4 - 9	2 - 4.5

From a theoretical point of view, this approach can also accommodate inclined walls, sloping ground surfaces and soil-wall interface friction, by properly calculating the active and passive earth pressure. In the practice, the influence of these factors is disregarded on the spring stiffness, and therefore the results have to be interpreted with particular attention.

This approach can account for very simple steady state seepage conditions (one-dimensional upward and downward seepage, typical when thin low permeability horizontal layers are present), but it cannot consider neither more complex seepage nor hydro-mechanical coupling. As a result, the method should not be used under partially drained conditions, where the rate of excess pore pressure dissipation is comparable to the rate of excavation.

Moreover, this approach cannot predict the vertical ground settlements associated with horizontal wall movements. Although some simplified procedures to estimate settlements from wall deflections exist, the results should be interpreted with particular caution. This limitation is

particularly relevant because, in many cases, the control of vertical ground movements is one of the governing design criterion.

Finally, the applicability of the approach is restricted to relatively simple two-dimensional geometries. It cannot adequately represent complex structural configurations, such as RPWs, irregular excavation geometries, complex loading conditions, strongly layered soil profiles, or all the situations where the stress distribution within the soil differs significantly from the assumptions underlying active and passive earth pressure theories.

Despite these limitations, the method remains extremely attractive in engineering practice. In fact, it provides results (in terms of wall deformations, bending moments, shear forces, and anchor loads) in a very short time while still accounting for SSI. For this reason, it is well suited to preliminary and conceptual design stages, where several alternative solutions must be sieved.

4 Finite element approaches

Finite element (FE) analyses are considerably more computationally demanding than the approaches discussed previously; however, they overcome the inherent limitations of the subgrade reaction method. In particular, FE models can realistically represent complex structural configurations, heterogeneous soil stratigraphies, hydraulic conditions (also hydro-mechanical coupling), and staged excavation processes. Also three-dimensional geometries and loads (e.g. moored vessels and cranes) can be reproduced, but, at present three dimensional models are not commonly adopted in the design.

In recent years, numerous studies have demonstrated the capability of FE analyses to accurately reproduce the mechanical response of QWs. Within the context of PoR, experience gained from several case studies (Lai et al., 2025; Hao et al., 2026; Roubos et al., 2026) has enabled the development of a site-specific framework for calibrating the parameters of the Hardening Soil constitutive model (Schanz et al., 1999). The calibration is based on literature q_c based correlations, providing a practical methodology for site characterization.

As an example, Figure 3 (adapted from Roubos et al., 2026) compares the measured displacements of the RPW following excavation (dotted line) with the preliminary FE predictions (solid line) developed within the CLARION project (CLARION, 2024). The dashed line represents an alternative prediction approach, which is discussed in the following section. The numerical results are in reasonably good agreement with the field measurements, particularly considering that the soil parameters were not calibrated against the in situ observations.

Despite the capability of FE analyses to reproduce the response of QWs, several aspects require careful consideration. In particular, the following topics are discussed: (i) the use of characteristic/design soil properties; (ii) the choice of an appropriate soil constitutive model and (iii) the use of FE analyses to develop and calibrate simplified wall calculation models.

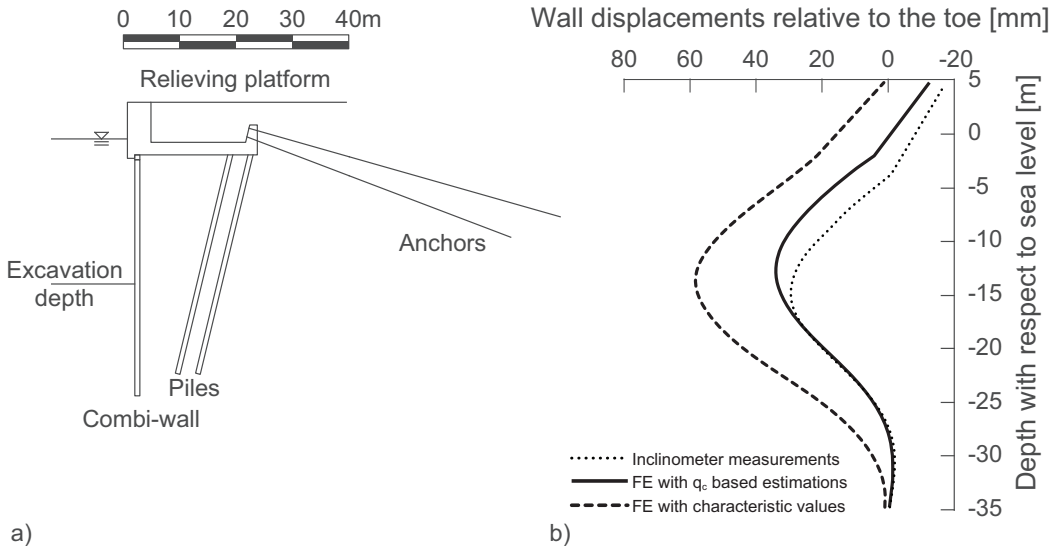

Figure 3: a) considered geometry and b) comparison of field measurements and numerical results, adapted from Roubos et al., (2026)

4.1 The use of characteristic/design values

According to the current partial factor design framework adopted in Eurocode 7 (EN 1997-1), the design should be performed by using characteristic or design values of the soil properties rather than best-estimate values. Consequently, the most representative estimates of the ground parameters are conservatively reduced to account for uncertainty and to satisfy the prescribed level of safety.

The implications of this approach are illustrated with reference to the case study of Roubos et al. (2026). Since the comparison is performed considering only permanent loads, the analysis is carried out using characteristic soil properties in accordance with the EC7 framework (serviceability limit state). The FE prediction obtained under these assumptions is shown by the dashed line in Figure 3b. Compared with both the field measurements and the prediction based on best-estimate parameters, the use of characteristic values leads to a significant underestimation of the system performance, resulting in predicted wall displacements that are, in this case, almost twice those observed in the field.

Moreover, it is worth noting that in non-linear SSI problems, the (arbitrary) reduction of soil strength parameters (it theory also stiffness should be reduced but this is not accounted for, Flessati & di Prisco, 2023) may lead to the development of non-realistic failure mechanisms. As a result, the design may be governed by mechanisms that are not representative of the actual system response. For this reason, in non-linear analyses (not only geotechnical but also structural), partial factor design approaches based on direct reduction of material properties (such as the M2 set of partial factors in Eurocode 7 terminology) should be applied with particular caution.

4.2 Soil constitutive law

The reliability of FE predictions is affected by uncertainties associated with both the constitutive model formulation and the calibration of its parameters. In the following, for the sake of brevity only the most diffused (and simple) classes of models will be presented and the limitation of these models will be discussed with the reference of a simplified case of an excavation of an AW (Figure 4). In particular it is considered a combi-wall constituted by tubular piles of diameter 1.42 m, thickness 0.02 m, length 33.5 m and spacing 3.29 m, connected to 3 x PU28 steel sheet-piles. The depth of excavation is equal to 18 m, the water table is at 4 m from the ground surface. The anchor is schematized as a rigid constraint at 2 m depth. The wall is assumed to be installed in a homogeneous medium-dense sand.

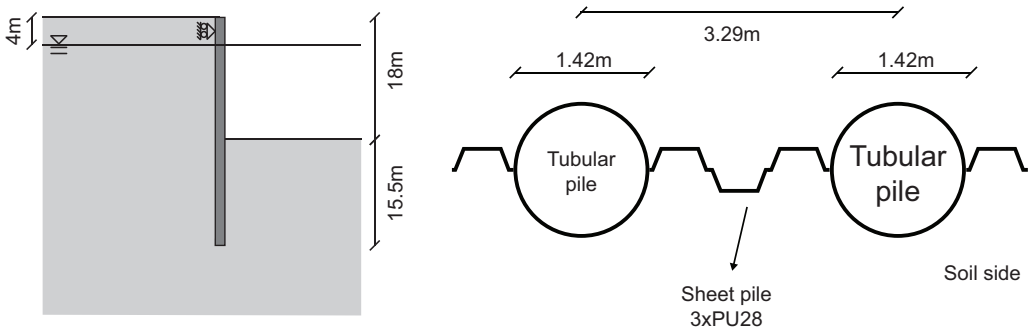

Figure 4: Geometry of the considered anchored QW

As a comparison, the soil behaviour is reproduced by means of the hardening soil model (HS) and an elastic-perfectly plastic constitutive law. In this second case, a Mohr-Coulomb failure surface with a non-associated flow rule is considered. The HS model parameters (Table 2) were chosen on the basis of the values adopted for a medium dense sand. The elastic perfectly plastic constitutive law properties were chosen by considering: (i) the same friction and dilatancy angle with respect to those of Table 2, (ii) the same Poisson ratio and (iii) an elastic stiffness that correspond to the HS elastic-plastic one (E_{50}) at the wall mid point.

Table 2: Hardening soil parameters

E_{50}^{ref} (MPa)	E_{oed}^{ref} (MPa)	E_{ur}^{ref} (MPa)	ν (-)	m (-)	ϕ' (°)	ψ (°)	R_f (-)
45	45	135	0.2	0.5	34	3	0.9

The results in terms of wall bending moment at the end of the excavation for the two cases (HS, solid line, whereas the elastic-perfectly plastic MC, dashed line) reported in Figure 5a are in good agreement. The results in terms of wall horizontal displacement are still in reasonable agreement, even if the MC results clearly cannot reproduce the response of the passive side at the wall toe Figure 5b. This is because with elastic perfectly plastic models exhibit the same stiffness under loading and reloading/unloading. However, the real problem concerning the MC results is related with the displacements at the ground surface in the land side (Figure 5c). In fact in this

case the numerical results clearly show an unrealistic heave. This unrealistic behaviour, that can also be observed for different geometries/soil mechanical properties (and also other excavation problems, such as tunnel excavations), highlight that for QWs the use of simple elastic-perfectly plastic constitutive laws should be avoided.

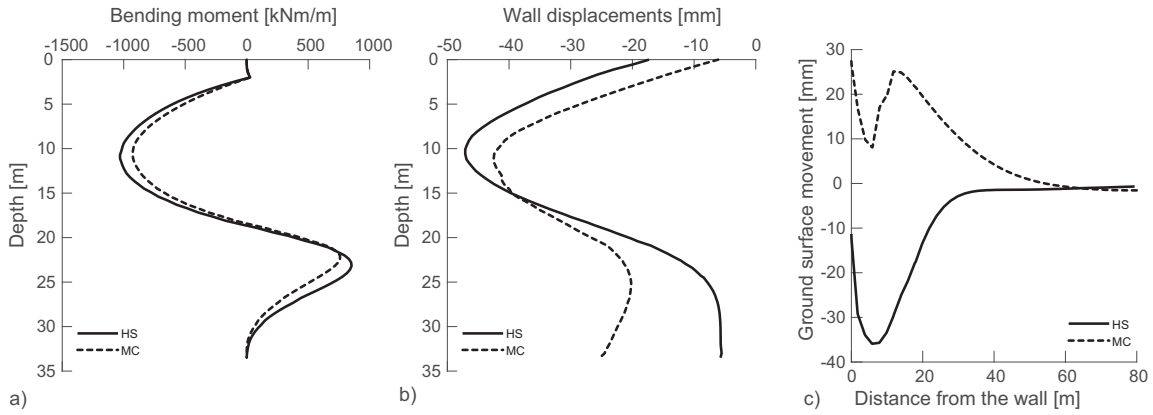

Figure 5: Influence of soil constitutive law on numerical results: a) wall bending moments, b) wall displacements and c) landside settlements

The results reported in this section highlight that, even when the same basic mechanical parameters (strength and stiffness) are adopted, different constitutive models may provide different predictions because they represent stiffness, stress dependency, unloading/reloading behaviour, and dilatancy in different ways. On the one hand, more sophisticated constitutive models can better reproduce some aspects of soil behaviour; on the other hand, they generally require the calibration of a larger number of parameters. In practical applications, however, the available in-situ and laboratory data may not always be sufficient to reliably determine all the required parameters. In such cases, the selection and calibration of the constitutive model necessarily rely, to a certain extent, on the engineering judgement and experience of the designer. Increasing the complexity of the constitutive model does not necessarily result in more reliable predictions if the available data do not support an adequate calibration.

4.3 Simplyfing the modelling approach through FE

As previously discussed, RPWs are highly complex structures that cannot be analysed using simplified methods. Consequently, their design relies computationally demanding FE analyses. This limits both the number of configurations (e.g. pile diameter, length, and spacing) that can be investigated and the application of probabilistic approaches for reliability assessment (Montali et al., 2026). The objective of this section is to demonstrate that, by adopting an appropriate yet simple modelling approach, the response of an RPW can be reproduced by an equivalent AW, allowing the analysis of the system by means of simplified subgrade reaction methods.

The reference case considered is the RPW presented in Section 4.1. The numerical results (Figure 6a, adapted from Montali et al., 2026) show that, at the end of the excavation process, the active

soil wedge on the landside (i) is only marginally influenced by the piles and (ii) detaches from the relieving platform. As a result, the vertical stresses are transferred entirely to the wall and the piles.

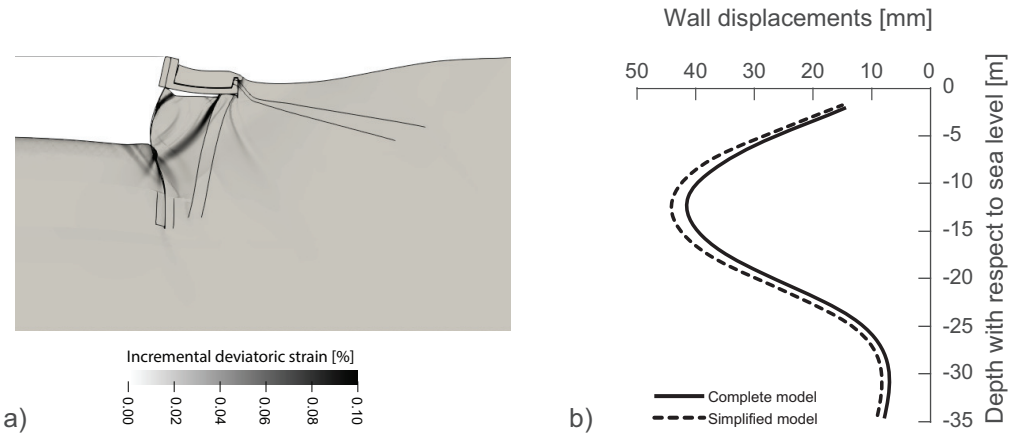

Figure 6: a) deviatoric plastic strain increments relative to the case of section 4.1 and b) comparison between the complete and the simplified model

Based on these observations, Montali et al. (2026) proposed that, when only the wall response is of interest, the RPW can be represented by a simplified structural model in which the relieving platform and the piles are omitted, while the anchors are modelled as springs. In this simplified model, (i) the combi-wall extends only to the foundation level of the relieving platform, (ii) an initial surcharge equivalent to the self-weight of the soil prior to construction is applied at this level, and (iii) this surcharge is removed before the first excavation stage. The application and subsequent removal of this surcharge leave the soil on the active side of the wall in an overconsolidated state. The comparison of wall horizontal displacements at the end of the excavation shown in Figure 6b demonstrates excellent agreement between the simplified (dashed line) and the complete FE models (solid lines), with a maximum discrepancy lower than 5%.

The range of applicability of this simplified approach, particularly with respect to soil properties, wall geometry, and stiffness of the relieving platform, is currently under investigation. In addition, ongoing research is focused on how the induced overconsolidation can be reliably incorporated into the constitutive formulations adopted in subgrade reaction models.

5 Conclusions

The design of modern quay walls increasingly requires methodologies that explicitly account for soil-structure interaction to achieve safe, economical, and sustainable solutions. Traditional analytical approaches remain valuable for preliminary design, but their simplifying assumptions often lead to (very) conservative designs.

Subgrade reaction methods provide an effective intermediate approach by incorporating soil-

structure interaction with limited computational effort, making them well suited for conceptual design and evaluating alternative solutions. Nevertheless, their applicability is restricted to relatively simple geometries and loading conditions.

Finite element analyses offer the most comprehensive framework for modelling quay walls. However, the direct application of characteristic/design soil parameters within non-linear finite element analyses may produce overly conservative predictions and unrealistic failure mechanisms, indicating the need for further developments in partial factor based design frameworks. The reliability of the results strictly depends on the adopted constitutive model, that have to be chosen with particular attention. Finally, recent developments in simplified finite element based modelling approaches show that even complex relieving platform quay walls can be represented by equivalent simplified models with excellent accuracy.

These developments provide a promising pathway towards displacement-based design methodologies that combine the accuracy of advanced numerical analyses with the efficiency required for engineering practice. Continued research on simplified modelling techniques, constitutive formulations, and probabilistic design approaches will further support the development of more economical, sustainable, and reliable quay wall infrastructure.

References

[1] Blum, H. (1931): Einspannungsverhältnisse bei Bohlwerken und deren vereinfachte Berechnung mit Hilfe von “ideelle” und “stellvertretender” Belastung, PhD thesis, Technische Hochschule Carolo Wilhelmina zu Braunschweig

[2] CLARION (2024). Climate resilient port infrastructures https://projectclarion.eu/

[3] CUR 166 Damwandconstructies, 6e herziene druk, CUR Bouw & Infra, Gouda 2012.

[4] de Gijt, J. G., & Broeken, M. L. (2013). Quay walls. CRC Press.

[5] EN 1997-1: 2004, Eurocode 7 - Geotechnical design, Part 1: General rules, European Committee for Standardization, Brussels.

[6] Flessati, L., & di Prisco, C. (2023). Deep tunnel faces in cohesive soils under undrained conditions: application of a new design approach. *European Journal of Environmental and Civil Engineering*, 27(8), 2630-2644.

[7] Hao, N., Duffy, K., Reale, C., and Gavin, K. (2026): Performance-based reliability assessment of quay walls. *Computers and Geotechnics*, 191, 107794.

[8] International Chamber of Shipping. (2024). Shipping and world trade: Global supply and demand for seafarers. http://www.ics-shipping. org/shipping-fact/

[9] Lai, F., Duffy, K., Gavin, K., Lu, D., & Roubos, A. (2025): Integration of Field Monitoring and Numerical Modelling to Evaluate the Construction Performance of a Deep-Sea Quay Wall. *Journal of Geotechnical and Geoenvironmental Engineering*, 151(12), 04025156

[10] Montali, F., Alver, O., Gavin, K., Jommi, C., Flessati, L. (2026): A simplified modelling approach for the analysis of port quay-walls with relieving platform International Conference on Advances and Innovations in Soft Soil Engineering 24 - 26 August, Delft

[11] Roubos, A. (2026): Impact Infra Innovation Soil - Structure - Materials, 2nd Ports and Waterways Conference, 4-6 February 2026, Delft

[12] Roubos, A., Flessati, L., Alver, O., Gavin, G. (2026): CLARION: Smart & sustainable climate resilient Quay Walls Proceedings of the 21st ICSMGE, Vienna, Austria, 14-19 June 2026.

[13] Schanz, T., Vermeer, P. A., and Bonnier, P. G. (1999): The hardening soil model: Formulation and verification. *Beyond 2000 in computational geotechnics*, Routledge, 281-296

6 Acknowledgements

The authors gratefully acknowledge the contributions of A. Roubos (Port of Rotterdam and TU Delft), O. Alver (TU Delft), F. Montali (TU Delft), and K. Gavin (TU Delft). This paper is supported by the Clarion project, which has received funding from the European Union's Horizon Europe research and innovation program under Grant Agreement 101147041.

Author

Dr. Luca Flessati
Technische Universiteit Delft
Faculteit Civiele Techniek en Geowetenschappen
Stevinweg 1
2628 CN Delft
e-mail: `l.flessati@tudelft.nl`