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Numerical Assessment of the Performance of a Strengthening Technique for Historical Masonry Quay Walls Subjected to Traffic Loading

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Abstract. Historical quay walls, constructed in unreinforced masonry, play a crucial role in the infrastructure of many Dutch cities. Designed originally as gravity retaining walls, these structures are increasingly subjected to traffic loads due to vehicles operating on roads built on their backfill. This study conducts a preliminary numerical evaluation of a strengthening technique aimed at prolonging the service life of such quay walls, focusing on a specific case in Amsterdam. The strengthening method involves drilling tubular steel piles through the existing masonry to anchor into a stable soil layer, with the piles bonded to the masonry using low-shrinkage casting concrete. The assessment models the interaction between the strengthening technique and the existing quay structures, including a detailed simulation of the installation process, identified as critical for proper simulation of the structural behaviour. While the technique significantly enhances the quay's force capacity, an improvement in displacement capacity was not evident, highlighting the need for further investigation.

Keywords: Quay walls · Retaining structures · Traffic loading · Unreinforced masonry · Timber piles · Strengthening

1 Introduction

Quay walls, also known as waterfront retaining walls, are structural elements built along bodies of water to provide support and stability to the adjacent land, while facilitating maritime activities such as docking, loading, and unloading of vessels. Globally, extensive networks of quay walls span across urban cores, rivers, ports, and flood prevention zones [1]. Among these, quay walls constructed from unreinforced masonry that edge the canals are both crucial and iconic elements within the infrastructural landscape of numerous Dutch cities, such as Amsterdam. Here, the historic canal ring district also enjoys recognition as a UNESCO World Heritage site [2]. These structures, often surpassing a century in age, have experienced various levels of damage due to several factors, including overloading, foundational failures or settlements [3, 4], material ageing and degradation, and insufficient maintenance, among others. The current state of

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reliability of these quays, especially in their capacity to support the demands of urban traffic, remains unknown [5]. This concern is amplified by instances of collapses, as for a portion of the historic quay Grimburgwal [6]. At the same time, renewal of long sections of quays is economically unfeasible and environmentally unsustainable. Addressing this, the necessity for the development and application of strengthening techniques to retrofit these structures is evident. This paper aims to provide a preliminary numerical assessment of a strengthening technique tailored for masonry quay walls. The organisation of the paper is as follows: Sect. 2 provides a general description of the structural configuration of Dutch historic masonry quay walls. Section 3 of the paper details the strengthening technique considered in this paper. Section 4 presents the numerical model of the proposed strengthening technique applied to a case study of a quay wall in Amsterdam, the Netherlands. A comparison in terms of structural behaviour and capacity of the quay in its unreinforced conditions is provided against the strengthened configuration under the action of monotonically increasing vehicular traffic loads. Concluding remarks highlighting current limitations and potential avenues for improvement of the performed study are ultimately presented in Sect. 5.

2 Historical Quay Structures in the Netherlands

Historical quay structures, situated alongside rivers and canals, are a common sight in Dutch urban landscapes. When constructed until the 1920s, these structures typically feature a masonry wall resting on a timber base, supported by a foundation of timber piles (Fig. 1). Timber beams, known as *kespen*, are positioned between the foundation piles and the base. Originally, the purpose of these quays was to provide docking for ships and to protect the land from high water levels. With this in mind, they were constructed to function as gravity retaining walls, ensuring stability against the soil pressure from behind. Nowadays, they face additional pressures unforeseen in their original design, attributed to vehicular traffic on roads built atop their backfill, as depicted in Fig. 1. These additional pressures are not only unforeseen but also larger and different in nature compared to the soil pressure they were designed to withstand. The performance of the strengthening measure described in Sect. 3 will be assessed under the application of loads due to vehicular traffic on their backfill.

3 Considered Strengthening Technique

The strengthening technique whose structural performance is examined in this study is loosely based on the *GrachtCompactPaa*l system, conceived by the Royal BAM Group as an innovative strengthening measure to extend the lifespan of quay walls in Amsterdam [7]. Exact details on the implementation of the *GrachtCompactPaa*l system in the field are not yet known to the authors. Moreover, such considerations are beyond the scope of this paper which aims at exploring the appeal of the considered strengthening technique at a conceptual level. To facilitate a qualitative assessment of the potential enhancement in the structural response of the quay to traffic loads, a few assumptions have been made, assuming these hold true in real-world applications. Such assumptions are described in the following paragraphs.

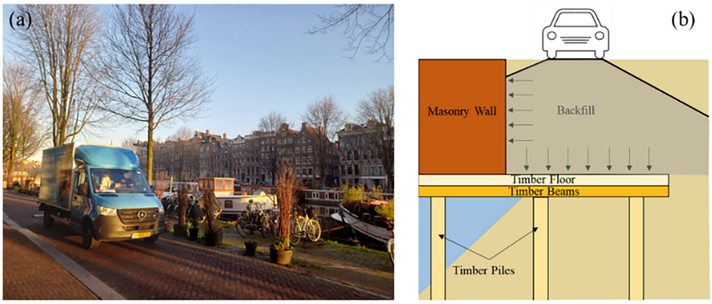

Fig. 1. (a) Vehicular traffic on carriageways constructed on the backfill of masonry quay walls in Amsterdam, the Netherlands and (b) schematic showing how this vehicular traffic creates pressure distributed by the soil on quay wall structures.

This reinforcement technique entails boring a 250 mm diameter core through both the masonry and the timber floor/foundation of the quay wall. No decrease in strength of the masonry is assumed to occur as a result of this drilling. A grout injection pile, composed of a tubular steel pile, is then driven to a depth of 22 m below *Normaal Amsterdams Peil (NAP)*, to reach the second strong layer of sand below the city. Here, NAP refers to the reference plane for water height in the Netherlands: a NAP height of 0 m is approximately equal to the average sea level of the North Sea [8]. The upper 7 m portion of this tubular steel pile has a thickness of 22 mm, with the thickness diminishing for the subsequent 15 m. The gap between the tubular steel pile and the boundary of the bored core within the thickness of the existing masonry wall is sealed with a low-shrinkage casting mortar/concrete, establishing a structural bond between the injection pile and the masonry. The escape of this grout from the bottom of the quay is prevented via the presence of a 10 mm thick steel disc (Fig. 2).

The steel piles of the *GrachtCompactPaal* system are suitable for installation in both straight and angled profiles throughout their depth along the quay's length, thereby enhancing horizontal load-bearing capabilities and resistance to bending stiffness. Cement grout may also be utilised to augment the resistance of the load-bearing sand layer that the steel tubular piles reach, providing additional strength and stiffness to the tubular piles beneath the depth of the timber floor. However, for the analysis presented in this paper, as outlined in Sect. 4, the focus is exclusively on straight profiles located at the position of each timber beam/*kespen*. The potential increase in strength and stiffness of the sand layer, or the reinforcement of the piles below the timber floor of the quay, is also not taken into account.

4 Assessment of the Strengthening Measure

The case study selected to evaluate the improvement in structural response through the application of the strengthening technique described in Sect. 3 is the *Marnixkade* quay situated along the *Singelgracht* canal on the northwest side of Amsterdam. Archival records indicate that this quay was built towards the end of the 19th century and is now approximately 130 years old. The quay features a clay brick masonry gravity retaining

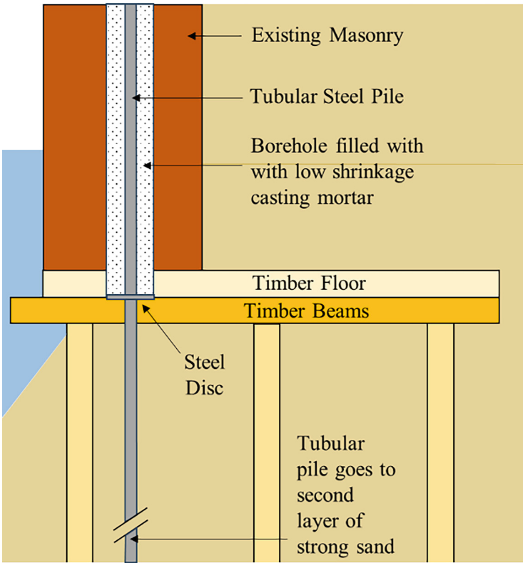

Fig. 2. Schematic of the assessed strengthening technique.

wall topped with a capstone, resting on a timber floor supported by timber piles. Three rows of timber piles run parallel to the waterway, with timber beams (*kespen*) positioned on top of them, perpendicular to the waterway. This quay was selected as a case study due to a comprehensive inspection [9] of both its superstructure and foundations carried out in 2016, aimed at assessing the feasibility of constructing an underground parking garage in its proximity.

From this inspection, the masonry gravity wall was found to have a height of 1.4 m and a thickness of 0.65 m. The timber floor was 70 mm thick. Each timber beam above the piles (*kesp*) measures 2.4 m in length and was originally 200x200 mm in cross-section. The piles feature a circular cross-section and are tapered, meaning their diameter decreases with depth along the length of the pile at a rate of approximately 9.75 mm per metre. Initially, their cap diameters ranged from 200 to 260 mm, with an average diameter of 235 mm. The centre-to-centre distances of the piles in the longitudinal direction vary between 900 mm and 1200 mm, while in the transverse direction a regular distance of 1100 mm was measured. For this paper, it is assumed that strengthening piles are installed at a centre-to-centre spacing of 1.1 m, staggered in terms of plan location with respect to the original timber piles. These strengthening piles are positioned in the centre of the thickness of the masonry wall. Significant discrepancies have been identified between the original design dimensions recorded in archival documents and the findings from the field inspection. These differences are predominantly observed in the timber elements that are in contact with water. Environmental exposure over time has resulted in the reduction of their cross-sectional dimensions. Nevertheless, these damages have not been taken into account for the preliminary assessment of the proposed strengthening measure discussed here.

The ground level at *Marnixkade* is situated at a height of 0.58 m above *Normaal Amsterdams Peil* (NAP), while the water level is at -0.40 m NAP. The soil conditions beneath the quay wall have been explored through cone penetration tests, revealing poor soil conditions until the first sand layer is encountered. It is assumed that the tips of the foundation piles reach this layer at approximately -13 m NAP, resulting in each pile being around 12 m in length. Drawing on the information collated above, the geometry of the *Marnixkade* quay has been determined for the numerical model described in Sect. 4.1 (Fig. 3).

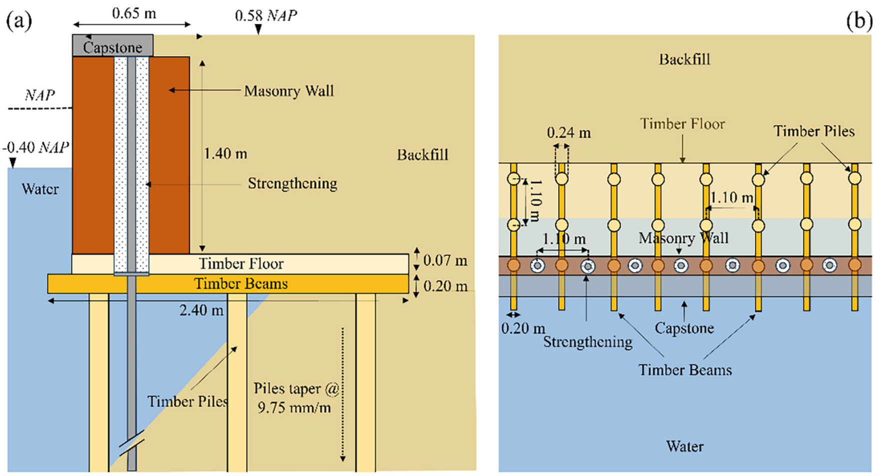

Fig. 3. Geometry of *Marnixkade* assumed for the numerical model with strengthening features inserted: (a) transversal cross-section and (b) plan view.

4.1 Numerical Model of the Quay

All simulations detailed in this paper were conducted using the software package Diana FEA 10.8 [10]. Two 3D numerical models of the quay were developed: one representing its unreinforced state, as per the structural configuration described above, and another depicting its strengthened configuration. A length of 30 m was selected for these models based on a sensitivity study that analysed the effect of model length on the quay's displacements. Given that the *Marnixkade* quay extends to almost 350 m, in-plane restraints were applied as boundary conditions to simulate the confinement provided by the quay sections not included in this model (Fig. 4a).

For the model representing the quay in its unreinforced state, the materials considered are solely masonry and timber. The quay wall's masonry is modelled using solid elements, with an isotropic material model that captures non-linear tensile and compressive behaviour in the principal directions (the Total Rotating Strain Crack model [10]). This constitutive law quantifies cracking through the integral under the stress–strain curve, known as fracture energy. Tensile stresses are modelled to decrease linearly, whereas compression initially hardens before ultimately yielding to softening, described by a parabolic curve. The timber components of the quay wall are simulated as linear-elastic. Shell elements represent the timber floor, while the *kespen* and piles are modelled with beam elements. The interaction between the quay's various structural components is also accurately simulated. A mortar joint between the masonry wall and the timber floor is included, modelled using a non-linear interface element capable of capturing both flexural opening and shear sliding. The connection between the timber floor and the *kespen* is considered fixed. The connection between *kespen* and timber piles is simulated as a spring with limited rotational stiffness, equating to a value of 4E + 08 N-mm/rad, to reflect the weak connection observed between these elements during their inspection [9]. Further details on the numerical model used for the quay in its unreinforced condition are available in [11] (Fig. 4b).

For the model of the quay in its strengthened state, the steel tubular piles and the grout connecting them to the existing masonry of the quay wall are added. The grout is modelled using solid elements, with its non-linear behaviour accounted for by the same isotropic material model used for the unreinforced masonry which captures both non-linear tensile and compressive behaviour in the principal directions, but with different material properties and assuming brittle behaviour in compression. The steel pile is modelled via different element types depending on its location and the surrounding elements of the numerical model of the unreinforced quay with which the pile must interact. When situated within the thickness of the masonry wall (modelled using solid elements), the pile is modelled via shell elements (Fig. 4c). Shell elements are also used to simulate the 10 mm steel disc that prevents the grout from escaping from the quay. Below the location of this steel disc, the steel piles are modelled using beam elements (Fig. 4c). At the floor's location, the tubular steel pile is initially modelled as a circular ring of beam elements, which is then connected to a vertical line of beam elements representing the tubular steel pile within the soil. All steel elements are simulated with non-linear material behaviour based on Von Mises plasticity, employing the Von Mises yield criterion and no hardening function. The interaction between the various elements of the strengthening technique, as well as with the unreinforced quay wall, is accurately simulated. Non-linear interface elements simulate frictional behaviour between the steel of the tubular piles and the grout, as well as between the grout and the surrounding masonry. Since the steel disc is welded to the steel pile, they share nodes, while no connectivity is modelled between the steel disc and the timber floor. Eccentric tyings are utilised between the portion of the steel pile at the location of the floor, modelled as a ring of beam elements, and the portion of the steel pile below the quay, modelled as a line of beam elements. This solution was adopted to ensure a proper transfer of both displacements and rotations while preserving compatibility between the elements.

No soil is modelled in either of the numerical models. However, considerations must be made to account for its presence. To simulate the presence of the soil adjacent to the masonry wall and below the timber floor, boundary interface elements are adopted. These interface elements are modelled with non-linear no-tension behaviour. The soil around the piles is replaced with linear elastic boundary interface elements that simulate the subgrade reaction. Additionally, the soil at the tip of the timber as well as steel piles in the strengthened model is simulated using non-linear no-tension boundary point elements. It is also to be noted here that in addition to the non-linearities mentioned above, geometrical non-linearities are also considered in every performed analyses.

Regarding material properties, the values used for masonry are sourced from NPR9998 [12], the seismic assessment guidelines of the Netherlands. Timber piles are classified as C24 grade according to [9], with their material properties obtained from Eurocode 5 [13]. For the strengthened model, steel is assumed to be of grade S235 and grout is assumed to have the material properties of concrete class C12. Their material properties are sourced from Eurocode 2 [14] and Eurocode 3 [15] respectively. The material properties adopted for the masonry and grout, as well as for the timber and steel in the numerical model, are summarized in Tables 1 and Table 2, respectively. The soil material properties used to calculate the stiffness of springs simulating the soil's influence in the model, are derived from Cone Penetration Tests (CPTs) [9].

Table 1. Input material properties used for masonry and grout.

		Masonry	Grout
Property	Unit	Value	Value
Young's modulus	MPa	5000	18962
Poisson's ratio	–	0.25	0.2
Density	Kg/m^3	1950	2350
Tensile strength	MPa	0.1	1.57
Fracture energy in tension	N/mm	0.01	0.12
Compressive strength	MPa	8.5	20
Fracture energy in compression	N/mm	20	Brittle

Table 2. Input material properties used for steel and timber.

		Steel	Timber
Property	Unit	Value	Value
Young's modulus	MPa	205000	11000
Poisson's ratio	–	0.35	0.35
Density	Kg/m^3	7350	420
Yield Stress	MPa	235	N/A

4.2 Representation of Traffic Loading in the Numerical Model

While the soil itself is not modelled explicitly, the influence of vehicular loads transmitted through the soil is accounted for. This analysis is based on an analytical formulation developed by Frazee [16], enhancing Boussinesq's work [17], to calculate both horizontal

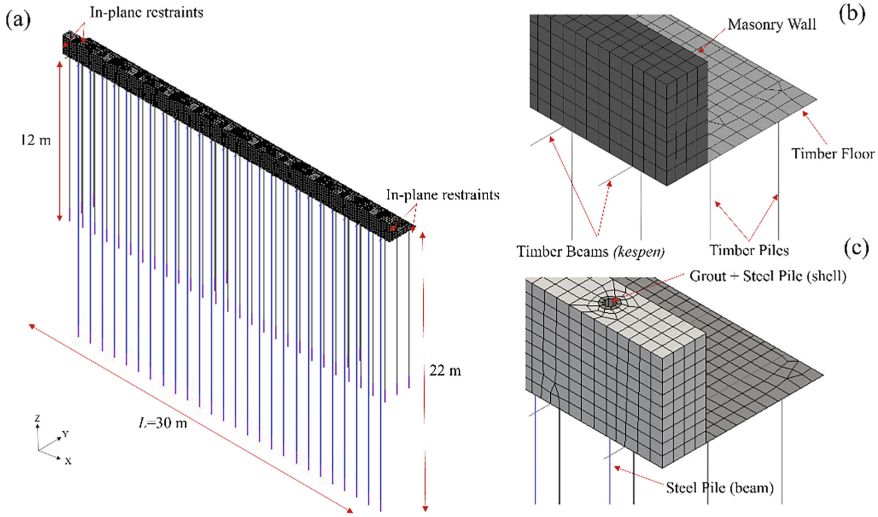

Fig. 4. Numerical model adopted to assess the response of *Marnixkade* under traffic loading: (a) general isometric view of the strengthened model; (b) close-up of the general isometric view of the unstrengthened model; (c) close-up of the general isometric view of the strengthened model.

and vertical stresses, denoted as $q_h(y,z)$ and $q_v(x,y)$, caused by a point load Q at a specified distance x, as detailed in Eq. (1a, b). The approach assumes the soil behaves as an elastic, isotropic, infinite half-space.

$$q_h(y, z) = \frac{\psi Q}{2\pi}\left[\frac{3x^2 z}{\left(x^2 + y^2 + z^2\right)^{\frac{5}{2}}} - \frac{1 - 2\nu}{\left(x^2 + y^2 + z^2\right) + z\sqrt{x^2 + y^2 + z^2}}\right] \tag{1a}$$

$$q_v(x, y) = \frac{3Q}{2\pi}\frac{z^3}{R^5} = \frac{3Qz^3}{2\pi}\left(x^2 + y^2 + z^2\right)^{-\frac{5}{2}} \tag{1b}$$

Figure 1 illustrates that vehicular traffic induces horizontal stresses on the quay's masonry wall, while vertical stresses impact the timber floor. Additionally, it was also discovered that vehicular loads transmitted through the soil can also subject the piles (both timber and tubular steel) to horizontal stresses. The analyses carried out in this paper considers a 3-axle, 6-wheeled fire truck commonly used in Amsterdam, the Netherlands, positioned 4 m away from the quay wall. This distance is measured from the masonry quay's inner edge to the vehicle's longitudinal central axis. Using the formula presented in Eq. 1, the horizontal stresses on the masonry wall (σ_{Wall} in Fig. 5a) and piles (σ_{Piles}) and the vertical stresses on the floor (σ_{Floor} Fig. 5b) due to this fire truck are calculated by linearly superposing the stress distribution from each of the truck's six wheels, as outlined in Eq. 1. To assess the quay's structural capacity, the calculated stress distributions are monotonically and statically increased until the quay's failure is reached.

In addition to the traffic load caused by to the fire truck, which is monotonically increased, other loads applied to the quay include the system's dead load as well as soil and water pressure from the canal water and backfill. Additionally, a parking strip runs along the quay wall at ground level. This parking strip consists of spaces for parking diagonally at an angle of 45 degrees, with some trees interspersed. A uniformly distributed load (UDL) of 5 kN/m^2 is considered for this parking load over a distance of 2.5 m inland from the quay wall. The effect of the parking load is also applied to the quay, considering its distribution through the soil, similar to the approach for the fire truck, adopting formulation for uniformly distributed loads (in place of the formulation for concentrated loads in Eq. 1a, b) in [16]. However, it should be noted that the UDL is maintained constant throughout the analysis, and not increased monotonically together with the fire truck load.

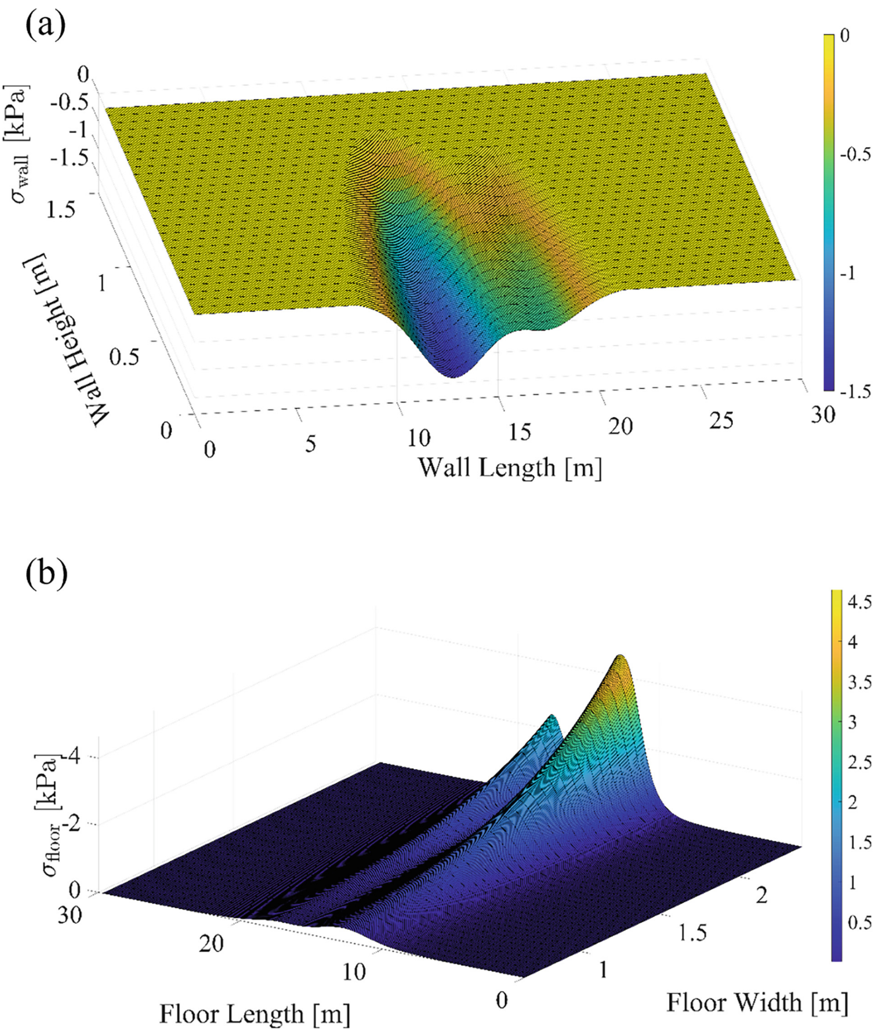

Fig. 5. Stress distributions due to traffic loading applied on: (a) the masonry wall and (b) the timber floor of *Marnixkade*.

4.3 Results

This section presents the results from simulations employing the numerical models described in Sect. 4.1, subjected to the loads specified in this section. The structural performance of the quay is evaluated in both its strengthened and unstrengthened states, with a focus on the modelling considerations that are crucial for accurately replicating the intended design behaviour of the strengthening technique. Specifically, simulating the installation stages of the strengthening measure is identified as essential, which may be achieved through an appropriately defined phased analysis. Initially, the dead load and soil and water pressure are applied to the unstrengthen configuration of the quay, causing the quay to tilt outwards (Fig. 6a). The next phase simulates drilling the borehole for the tubular steel pile by removing the existing masonry at its location. Removing the masonry's dead weight reduces the outward tilt (Fig. 6b). Although the tubular pile is present in the model, it is supported temporarily (Fig. 6c). The connection between the tubular pile and the masonry, using grout and non-linear interfaces, is established after clearing the displacements in the numerical model due to these phases to ensure the strengthening pile is compressed before the application of traffic-induced loads (Fig. 6d). This approach prevents numerically induced bending stresses in the steel pile that could arise if it were to tilt with the quay under gravity and soil and water pressures without proper simulation of the installation phases.

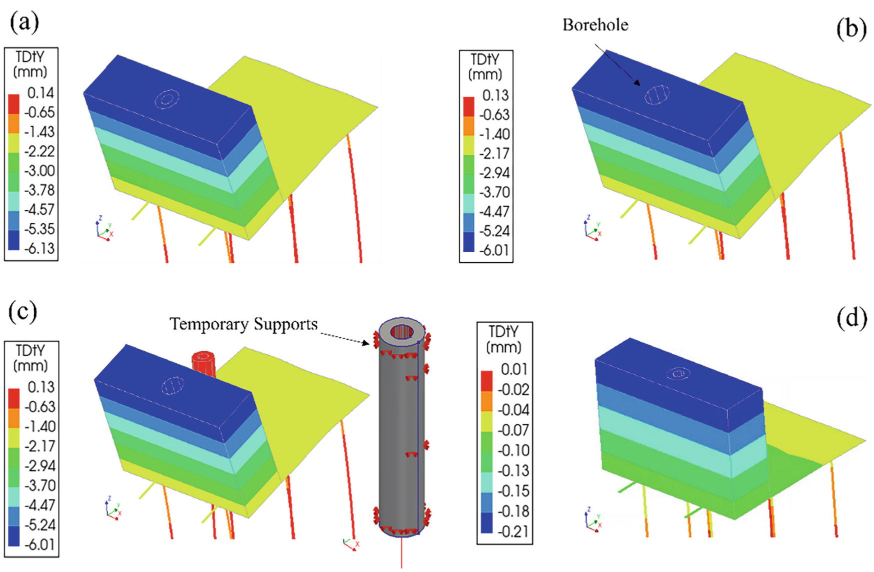

Fig. 6. Phased Analysis for simulation of the installation of the strengthening technique: (a) initial application of dead load and soil and water pressure; (b) simulation of borehole drilling for the tubular steel pile; (c) temporary support of the tubular steel pile within the model before establishing connections and (d) clear displacements in the model, finalise connection between the tubular steel pile and the masonry and apply parking load (Deformation Scale Factor = 100).

After the installation phase has been accurately simulated, traffic loads transmitted by the backfill are applied to the quay. This involves applying horizontal stresses on the masonry wall (σ_{Wall}) and piles (σ_{Piles}), along with vertical stresses on the floor (σ_{Floor}), which are monotonically increased by multiplying them with a scalar load multiplier (*LM*) until the quay fails. The structural performance of the quay, in both its unstrengthened and strengthened states, is compared by examining capacity curves. These curves are constructed by plotting the reaction forces sustained by the entire quay and the *LM* applied to it against the largest out-of-plane displacement of the quay obtained for each LM. Additionally, the failure mechanism is analysed in terms of principal crack widths and out-of-plane displacements of the quay.

From the capacity curves plotted for both the strengthened and unstrengthened quay in Fig. 7, it is evident that the strengthening technique significantly enhances the force capacity of the quay. The reaction forces of the system plotted in Fig. 7 are normalized with respect to the length of the model. The peak force capacity for the unstrengthened quay is reached at an out-of-plane displacement of approximately 13 mm with a *LM* value of 39.5 (Fig. 8a). At this stage, extensive cracking in the wall, indicative of imminent local overturning of a portion of the quay, is observed in the unstrengthened system (Fig. 9a). Conversely, for the same LM value, the strengthened quay demonstrates very limited displacements (<5 mm) (Fig. 8b) and crack widths (Fig. 9b). A maximum out-of-plane displacement of approximately 13 mm for the strengthened quay is achieved at an LM value of 79 (Fig. 10a). Although the capacity curve indicates that the strengthened system can sustain additional loads, crack widths in the existing masonry suggest its imminent local collapse (Fig. 10b). This is corroborated by monitoring the reaction forces solely sustained by the existing masonry in the strengthened quay, which shows a decrease in its force-carrying capacity. Beyond this stage (i.e., out-of-plane displacements of 13 mm), the forces that both strengthened and unstrengthened models can sustain are borne by the surrounding undamaged portions of the system, owing to 3D load redistribution effects in the numerical model and a portion of the structure is already heavily damaged. It should be noted that the selected static application of vehicular traffic loads may promote the localized failure of masonry, as observed in [18] for unstrengthened quays. Therefore, alternative loading procedures, such as those developed by the authors of the paper in [11], should be utilized to further investigate the structural performance of the strengthened quay.

5 Concluding Remarks

This paper presents a preliminary numerical structural assessment of a proposed strengthening solution aimed at extending the service life of Amsterdam's quay walls. The assessment suggests that the strengthening technique could significantly enhance the force capacity of the system under vehicular loadings. However, the displacement capacity appears to remain consistent with that of the unstrengthened structure, raising the possibility of local structural collapse at similar displacement levels. However, this observation might also result from the static application of vehicular traffic loads in our study, which may lead to more localized structural collapse as observed in [18] or unstrengthened quays. This highlights the need for further investigation through dynamic moving load analyses using procedures developed by the authors of the paper in [11].

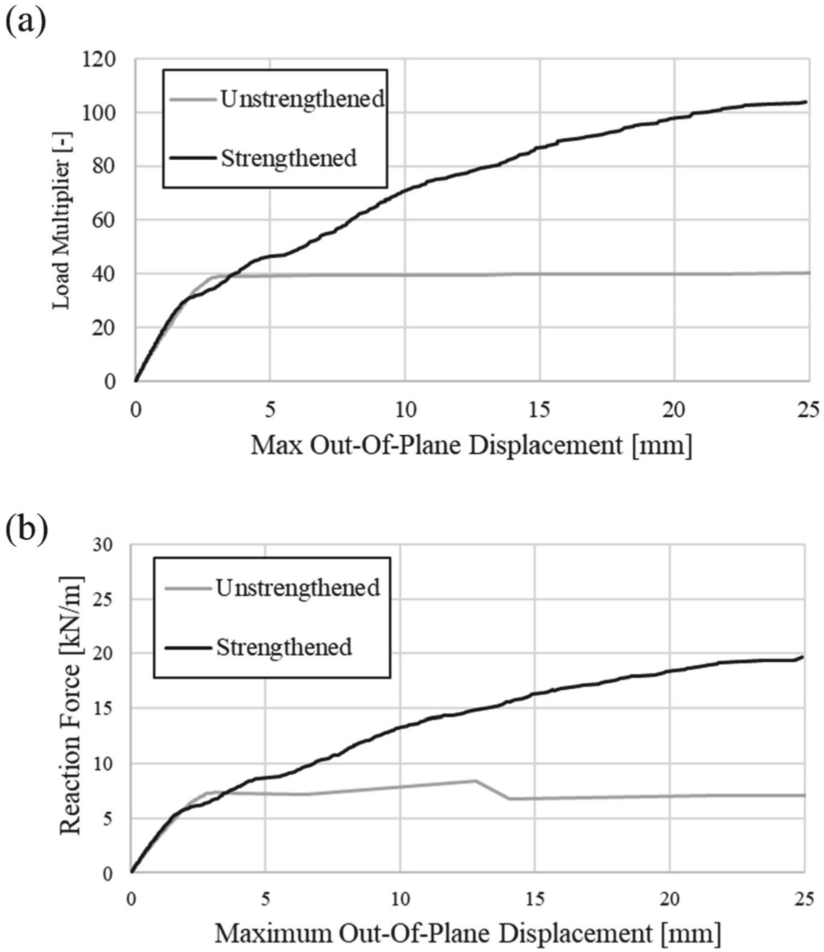

Fig. 7. Capacity curve of the quay in terms of: (a) *LM* (Load Multiplier) applied and (b) reaction forces sustained by the quay vs. maximum out-of-plane displacements recorded.

The analyses underscored the critical importance of accurately simulating the installation stages of the strengthening measure. Equally crucial is the interaction among the various structural components, simulated through the use of interface elements. The mechanical behaviour of these interfaces warrants detailed examination through mechanical characterisation tests. Given the geotechnical function of the proposed strengthening measure and the documented degradation of timber pile foundations in Amsterdam's quays, future research should explore the structural response improvements for a quay wall with modelled foundation degradation. Such investigations will provide more valuable insights into the effectiveness of the proposed strengthening solution in real-world conditions.

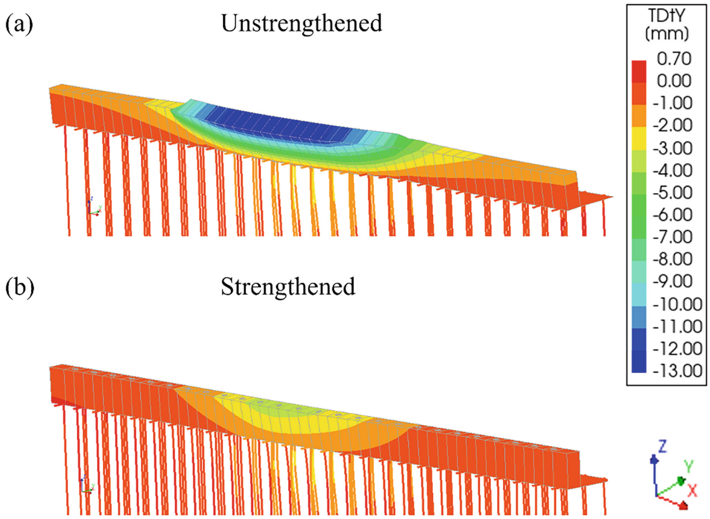

Fig. 8. Comparison of out-of-plane displacements in the numerical models at $LM = 39.5$: (a) unstrengthened and (b) strengthened configuration (Deformation Scale Factor = 100).

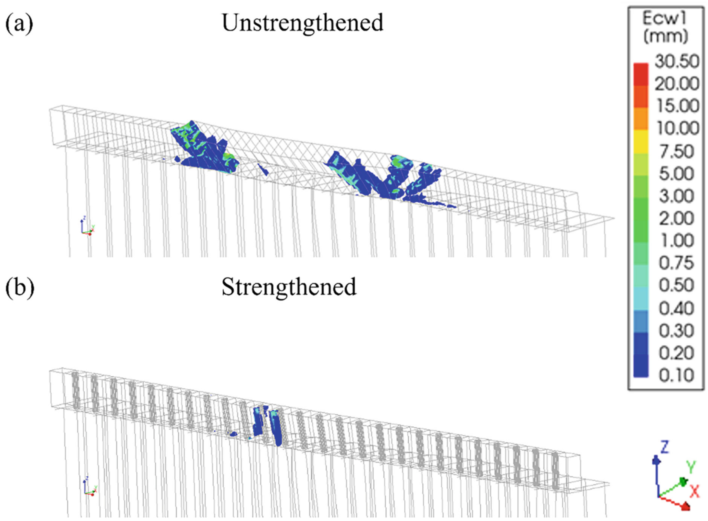

Fig. 9. Comparison of principal crack widths in the numerical models at $LM = 39.5$: (a) unstrengthened and (b) strengthened configuration (Deformation Scale Factor = 100).

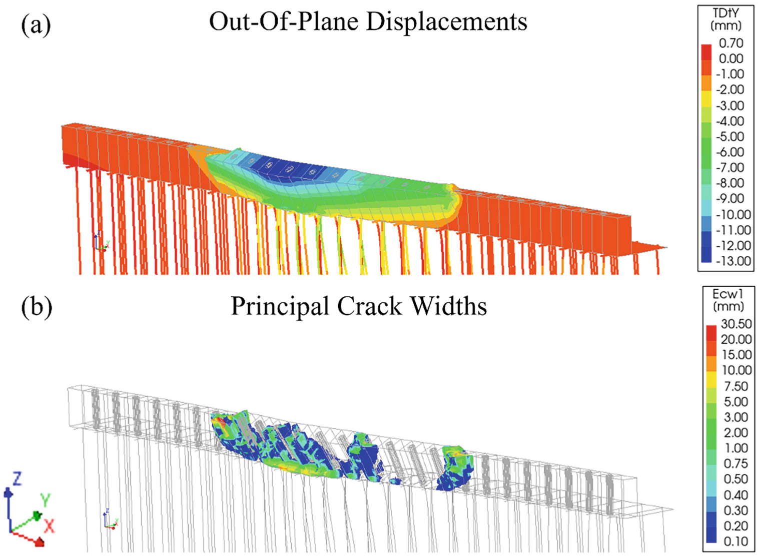

Fig. 10. Failure mechanism in the strengthened numerical model at $LM = 79$ in terms of: (a) out-of-plane displacements and (b) principal crack widths (Deformation Scale Factor = 100).

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