Structural Assessment of a Masonry Quay Wall in Amsterdam Under Traffic Loading

Sharma, Satyadhrik; Longo, Michele; Esposito, Rita; Messali, Francesco

DOI
10.1007/978-3-031-39603-8_13

Publication date
2023

Document Version
Final published version

Published in
Structural Analysis of Historical Constructions

Citation (APA)
Sharma, S., Longo, M., Esposito, R., & Messali, F. (2023). Structural Assessment of a Masonry Quay Wall in Amsterdam Under Traffic Loading. In Y. Endo, & T. Hanazato (Eds.), *Structural Analysis of Historical Constructions: SAHC 2023 - Volume 1* (Vol. 1, pp. 152-164). (RILEM Bookseries; Vol. 47). Springer. https://doi.org/10.1007/978-3-031-39603-8_13

Structural Assessment of a Masonry Quay Wall in Amsterdam Under Traffic Loading

Satyadhrik Sharma(✉), Michele Longo, Rita Esposito, and Francesco Messali

Faculty of Civil Engineering and Geosciences, Technische Universiteit Delft, Stevinweg 1, 2628 CN Delft, The Netherlands
S.Sharma-9@tudelft.nl

Abstract. Canals delimited by masonry quay walls are integral elements of many cities in the Netherlands. Historically built to enable the efficient transportation of goods, today such infrastructure also gives the cities their historical and monumental character. In recent years, many quay walls in the Netherlands have shown substantial deformation and damage, and in few cases even collapse. Historical quay walls, which are constructed in thick multi-wythe unreinforced brick masonry and are supported on a system of timber piles, nowadays sustain traffic loads larger than the one they were designed for. Instances of collapse and severe damage has given rise to a need for research assessing the safety of these structures, which are not appropriately covered by any normative or standardised guidelines. This paper presents a novel methodology to numerically assess the performance of masonry walls in historical quays under the dynamic effect of traffic loads. Application of the proposed methodology to a case study in Amsterdam, the Netherlands, is presented.

Keywords: quay walls · traffic loading · unreinforced masonry · timber piles

1 Introduction

Thousands of kilometres of quay walls can be found globally in city centres, inland waterways, commercial port areas and flood defence systems [1]. Among these, quay walls built in unreinforced masonry, delimiting canals are vital as well as emblematic features of the infrastructure of many Dutch cities, including Amsterdam, where the inner-city canal ring area has UNESCO world heritage status. These structures are often more than 100 years old, a period over which they have suffered damage of different degrees, attributed to overloading, failure or settlements of their pile foundations, aging and deterioration of the material, and lack of adequate maintenance, amongst other causes. It is presently unclear how reliable these structures still are and whether they are capable of bearing the traffic loads associated with a modern city to which they are subjected. Such questions have become even more crucial due to several instances of imminent or explicit collapse [2] of such structures. Thus, there is a need to develop dedicated analysis procedures which can be used to assess these structures under the effects of vehicular traffic. Towards that end, a novel assessment procedure for masonry quay

Y. Endo and T. Hanazato (Eds.): SAHC 2023, RILEM Bookseries 47, pp. 152–164, 2024.
https://doi.org/10.1007/978-3-031-39603-8_13

walls under traffic loads is proposed in this paper. The paper is organized as follows: Sect. 2 of the paper introduces the structural configuration of historical masonry quay walls in Amsterdam/the Netherlands. Section 3 of the paper explains novel methodology developed to assess the behaviour of quay wall structures under traffic induced loading. Section 4 presents the application of the proposed assessment procedure to a case study of a quay wall in Amsterdam, the Netherlands. Concluding remarks highlighting current limitations and potential avenues of improvement of the proposed procedure are ultimately presented in Sect. 5.

2 Quay Structures in the Netherlands

Quays are structures that are built next to rivers and canals and can be found throughout cities of the Netherlands. A typical cross section of such structures can be seen in Fig. 1b. The masonry wall is placed on a timber floor against the backfill with a foundation of timber piles. In between the foundation piles and the floor, timber beams called *kespen* are placed. The original function of these quays was (and still is) to accommodate ships and make sure that land is safe against high waters. Keeping these functions in mind, they were designed primarily as gravity retaining walls i.e. to be stable against the pressure of the soil behind them. However, today they are subjected to an additional pressure that they were not originally designed for, arising due to increasing vehicular traffic plying on carriageways constructed on top of their backfill (Fig. 1). The procedure proposed in this paper addresses the assessment of masonry quay walls under the effect of these vehicular loads.

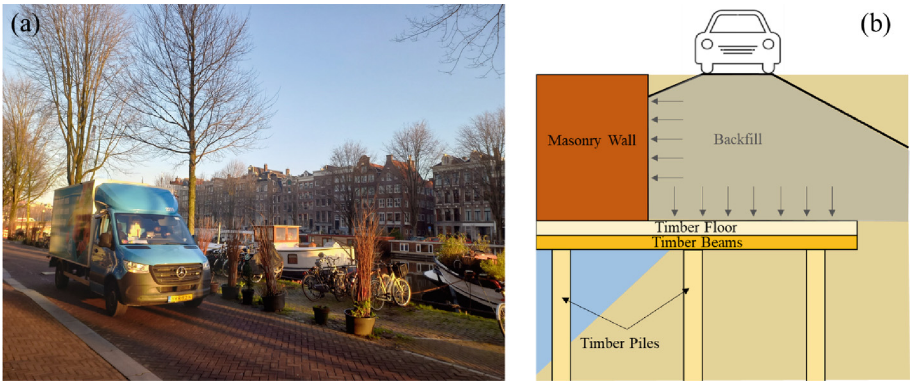

Fig. 1. (a) Vehicular traffic on carriageways constructed on the backfill of masonry quay walls in Amsterdam, the Netherlands and (b) schematic showing how this vehicular traffic creates pressure distributed by the soil on quay wall structures.

3 Novel Assessment Procedure

The assessment of masonry quay walls should also account for the possible redistribution of forces along the length of the wall and any other hidden non-linear reserves in them. This is to prevent unnecessary exceptional maintenance or even renewal, which have high costs and impact on the monumental character of the structure. Another aspect to be considered is that interventions are typically carried out on historical masonry quay walls when monitored displacements exceed a certain threshold. Consequently, any developed assessment procedure should also aim to reliably estimate displacements of the quay wall as well as correlate displacements and structural capacity. A novel methodology to numerically assess the non-linear performance of masonry quay walls under the effect of dynamic traffic loads is proposed to address these goals.

A two-tier sub-structured modelling approach, in analogy to the method used for buildings [3], is adopted in this methodology. Here, a sub-structured approach refers to the fact that the system constituting of the quay wall structure and the adjacent soil is divided into two subsystems analysed separately. The first sub-system in tier 1 and its associated numerical model consists of just the soil and the road pavement on which the vehicles travel. Linear-elastic calculations are performed in this tier. This is used to simulate the propagation of loads resulting from vehicular traffic through the soil onto the quay wall structure. The second sub-system in tier 2 consists of just the quay wall structure with the effects of soil and its interaction with the structure modelled in a simplified fashion using springs/boundary interface elements. The loading input applied to this sub-system is obtained from tier 1. This is in the form of pressure-time series recorded at the locations of the wall and the timber floor in tier 1. Non-linear calculations are performed in this tier. Both these tiers are described in more detail in the following sub-sections.

3.1 Tier 1

In tier 1 of the proposed procedure, the passage of a vehicle (corresponding to the considered traffic load model) is simulated over a limited distance, Δd. This simulation is obtained by performing dynamic analyses that consider only the first sub-system, i.e. the 3D solid model of soil and road pavement. The soil block is extended in one direction from the waterfront, longer than the width of the carriageway over which the vehicle travels while in the other direction is terminated where the masonry wall meets the water (Fig. 2). Absorbing boundary conditions [4] are adopted at the external faces of the soil block to simulate the effect of an infinite medium and prevent the reflection of outward waves. No absorbing boundary conditions are modelled on the waterfront, where the quay wall meets the water. A stiffer layer corresponding to the paving of the carriageway over which the vehicle travels is also modelled. Linear-elastic properties are used for all the elements of this model (Fig. 2).

The passage of the vehicle over a limited distance (Δd) is simulated by applying the weight of the vehicle via each wheel impulsively. The normalized compressive stress impulse used for this purpose corresponds to a haversine impulse fitted to experimentally readings by Loulizi *et al.* [5]. These experimental readings were measured in an instrumented pavement during the passage of a loaded moving truck over a pressure cell. Δd in the simulation described in this work corresponds to the diameter of the circular pressure cells used by Loulizi *et al.* [5] above which the vehicle travelled when the compressive stress impulse was recorded. As the wheel approaches the pressure cell, the amplitude of vertical stresses on it starts increasing. It reaches its maximum value when the wheel is vertically above the centre of the pressure cell and starts decreasing as the wheel moves further away (Fig. 3a). The tier 1 model shown in Fig. 2 corresponds to a 3 axle, 6 wheeled fire truck used in Amsterdam, the Netherlands (Fig. 3b). This is the traffic load model considered in Sect. 4 to demonstrate the application of the proposed methodology. The truck is modelled at a distance of 4 m from the quay wall (measured from the inner edge of the quay wall to the longitudinal central axis of the fire truck). Since each wheel load is modelled individually, any traffic load (in terms of axle loads and spacing configurations) can be considered in this tier and, consequently, in the proposed assessment procedure.

It can be easily inferred from Fig. 1b that vehicles travelling on carriageways constructed on the backfill of quay wall systems create a pressure on both masonry wall and timber floor of the quay. The masonry wall and the timber floor are not modelled in tier 1, but the normal compressive stress flow through two surfaces located at their positions (σ_{Wall} and σ_{Floor} in Fig. 2) is continuously recorded. From these recordings, a selection is made of the instant at which stress distributions on these surfaces result in the maximum net compressive force acting on them. The stress distributions at this instant, truncated to get rid of normal tensile stresses which occur simply as a result of the model being linear elastic, defines the loading in tier 2 of the methodology. In the tier 1 model for the fire truck in Fig. 3b, the net compressive force obtained by integrating the σ_{Wall} and σ_{Floor} distributions maximise simultaneously at the same instant and are provided in Fig. 4. The time-history of the maxima stresses (σ_{Max} in Fig. 4) in these distributions are also recorded for use in step 2 of the methodology.

3.2 Tier 2

In tier 2, the entire structural part of the assessed quay wall is modelled 3D. In contrast to tier 1, the soil is left out from the model. However, the impedance of the soil block adjacent to the wall and floor is still accounted for through springs/boundary interface elements. This model is used for the structural assessment of the quay wall and is consequently modelled as non-linear. More details of the non-linearities and boundary conditions adopted for the tier 2 model of the selected case study are provided in Sect. 4.

The effect of vehicular traffic is simulated dynamically along the entire quay wall length by applying the stress distributions σ_{Wall} and σ_{Floor} in combination with the time-histories of their respective σ_{Max}, as derived from tier 1 as normal pressure loads on both masonry wall (horizontal pressure) and timber floor (vertical pressure). The pressure loads are applied to a sequence of consecutive sections, each with a length Δd, the same limited distance over which the passage of the vehicle was simulated in tier 1. A time difference of Δt is maintained between load applications on subsequent sections with Δt calculated as the time required by the vehicle (at the speed to be considered) to transverse a distance of Δd. Since the maxima of stresses occurring during the transit of the vehicle over the distance of Δd recorded in tier 1 are used, this procedure is simplified yet conservative.

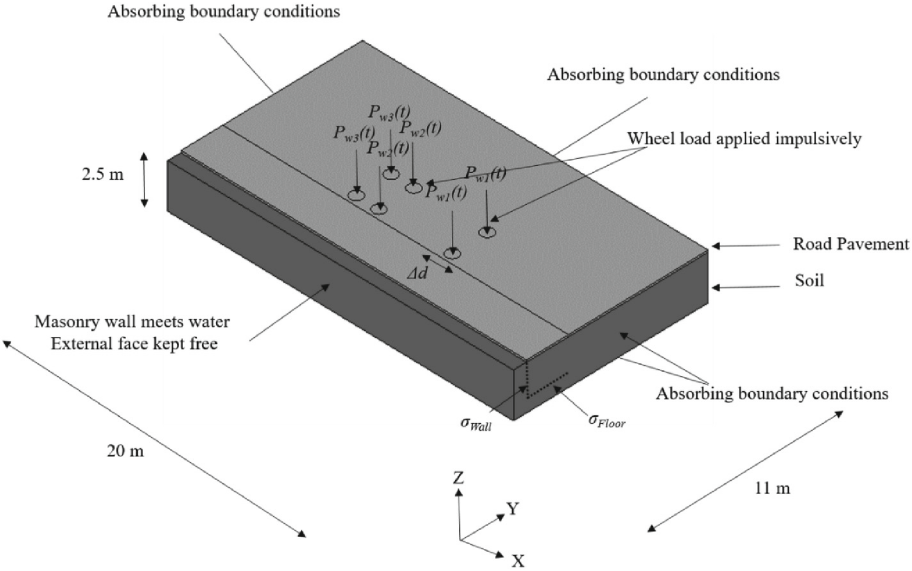

Fig. 2. Numerical model adopted for step 1 of the tier-based assessment procedure for a six wheels fire truck.

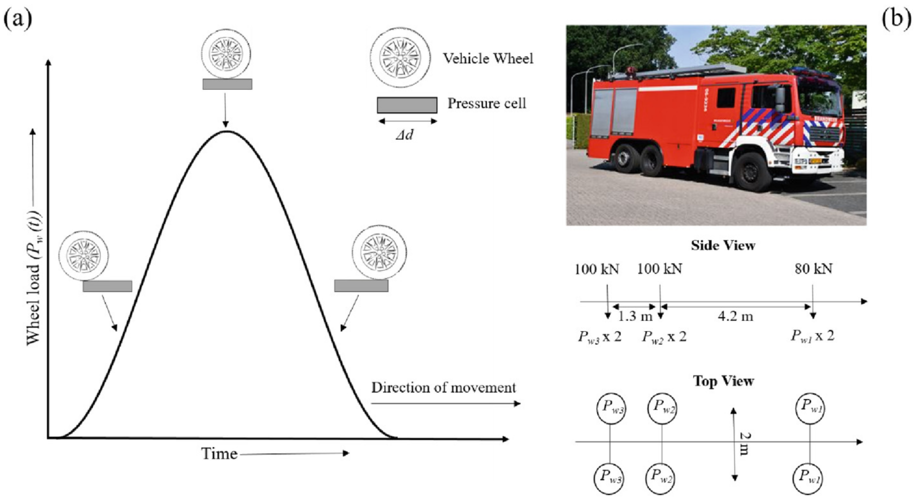

Fig. 3. (a) Relation between a wheel load impulse ($P_W(t)$) and the passage of a wheel over a distance Δd; (b) a 3 axle, 6 wheeled fire truck used in Amsterdam, the Netherlands, adopted as vehicular load for the case study reported in the paper.

In the proposed methodology, the entry (and conversely also the exit) of the vehicle from the portion of quay wall considered in the simulations can be easily accounted for. As an example, the length ΔL of the wall affected by the stresses generated in tier 1 by the considered vehicle when travelling for a length Δd at a distance of 4 m from the

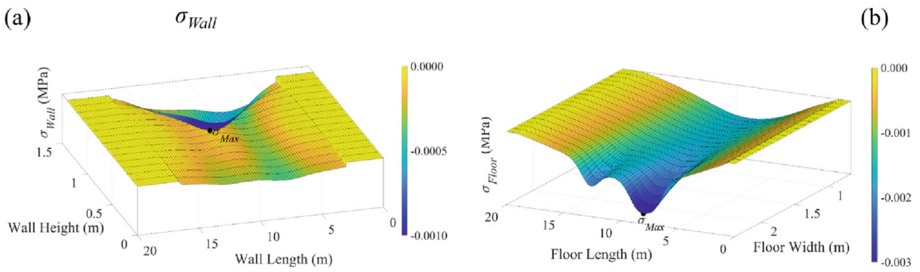

Fig. 4. Normal stress distributions at the instant of occurrence of maximum net compressive force (σ_{Max}) at: (a) masonry quay wall (σ_{Wall}) and (b) timber floor for a fire truck (σ_{Floor}).

quay wall is approximately 13 m (Fig. 4a). Since the value of ΔL largely exceeds the length of the vehicle, it is observed that a significant load pressure is generated on the wall and floor even when none of the vehicle wheels are adjacent to the modelled portion of the quay wall. To simulate this *"far-field"* effect of vehicular loads, at the beginning of the simulation in tier 2, the quay wall is completely unloaded. In the next step, only the outermost Δd long portion of the entire stress distributions is applied to the wall and floor. Consequently, the entire stress distributions are gradually introduced in Δd sized portions to simulate the entry of the vehicle to the vicinity of the quay wall and then onwards. Similarly, the stress distributions are also gradually reduced step-wise in Δd sized portions to simulate the exit of the vehicle.

4 Application of the Proposed Procedure to a Case Study

4.1 Reference Case: Marnixkade in Amsterdam

The case study to which the proposed procedure is applied to is the *Marnixkade* quay located along the *Singelgracht* canal in the North-West side of Amsterdam. According to archival data, this quay was constructed at the end of the 19th century and is now approximately 130 years old. The quay construction consists of a masonry gravity retaining wall with a capstone standing on a timber floor. The floor is supported on timber piles. 3 rows of timber piles are present parallel to the waterway with timber beams (*kespen*) on top of them perpendicular to the waterway. This quay was chosen as a case study due to a detailed inspection of its superstructure as well as foundations having been conducted in 2016 for investigating the feasibility of constructing an underground parking garage in its vicinity.

From this inspection [6], the masonry gravity wall is measured to have a height of 1.4 m and a thickness of 0.65 m. Several discrepancies can be found between the original design dimensions in archival records and the information gathered from the field inspection. These differences are mainly in the case of timber elements which are exposed to the water. Environmental actions accumulated over time have led to a reduction of their cross section. In particular, the timber floor was measured to have a thickness of 59 mm while originally it was 70 mm thick. Each timber beam above the piles (*kesp*) is 2.4 m long and was measured to have a cross-section of 189 × 189 mm which was originally 200 × 200 mm. The piles have a circular cross-section and are also tapered i.e. their diameter decreases with depth/along the length of the pile at a rate of approximately 9.75 mm/m. Originally their cap diameters varied between 200 and 260 mm with an average diameter of 235 mm. From the inspection, these diameters were measured to vary between 88 and 242 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 distance of 1100 mm was measured.

The ground level at *Marnixkade* is at a height of 0.58 m *Normaal Amsterdams Peil* (*NAP*) while the water level is at −0.40 m *NAP*. Here, *Normaal Amsterdams Peil* refers to the reference plane for height in the Netherlands. A *NAP* height of 0 m is approximately equal to the average sea level of the North Sea [7]. The soil conditions underneath the quay wall have also been investigated via cone penetration tests. Poor soil conditions are found until the first sand layer is detected. The tip of the foundation piles are assumed to reach this layer at approximately −13 m *NAP*. This results in each pile being approximately 12 m long. Based on the information summarized above, the geometry of the *Marnixkade* quay is determined.

4.2 Numerical Model

All simulations reported in this paper have been performed in the software package Diana FEA 10.5 [8]. The tier 1 numerical model adopted for the case study has already been described in Sect. 3.1. In this sub-section, a more detailed description of the tier 2 numerical model which was adopted for the assessment of *Marnixkade* is provided. A length of 30 m (*L* in Fig. 5) was chosen for this model on the basis of a sensitivity study analysing the effect of length of this model on the displacements of the quay. Since the *Marnixkade* quay is much longer i.e. almost 350 m, in-plane restraints are applied as boundary conditions to simulate the confinement provided by the adjacent portions of the quay not considered in this model.

The quay is constructed in masonry and timber. The masonry of the quay wall is modelled via solid elements, using an isotropic material model which accounts for both non-linear tensile and compressive behaviour in the principal directions (the Total Rotating Strain Crack model in [8]). The cracking phenomenon in this constitutive law is quantified by the integral under the stress–strain diagram, denoted as fracture energy. Tensile stresses are assumed to diminish linearly, while under compression an initial hardening gives way ultimately to softening defined by a parabolic curve. The timber elements of the quay wall are modelled as linear-elastic. However, the failure of timber elements is still evaluated with an indirect check comparing the maximum stresses that develop in the model with the material strength related to different failure mechanisms. Shell elements are used for the timber floor while the *kespen* and piles are modelled using beam elements. The tapering of the piles is taken into account by discretizing the beam elements used for the piles into five segments, each having a different diameter which reduces along with the depth. It is to also be noted here that in this work, the original average dimensions of the timber cross-sections mentioned in Sect. 4.1 are employed.

The interaction between the different structural components of the quay is also appropriately simulated. A mortar joint is present between the masonry wall and the timber floor. The presence of this mortar joint is simulated using a non-linear interface element which can capture flexural opening as well as shear sliding. The connection between the timber floor and the *kespen* is modelled to be fixed. The connection between the *kespen* and the timber piles is modelled as a spring with limited rotational stiffness, corresponding to a value of 1E+08 N-mm/rad, to simulate the weak connection observed between these two elements during their inspection in [6].

No soil is modelled in the tier 2 numerical model. However, considerations must be made to account for its presence. To simulate the presence of the soil adjacent to the masonry wall ($k_{soil,wall}$ in Fig. 5) and below the timber floor ($k_{soil,floor}$ in Fig. 5), boundary interface elements with stiffness calculated as per the method summarised by NEHRP [9] based on the partially empirical formulation developed by Gazetas [10] are adopted. These interface elements are non-linear and act only in compression. The soil around the piles is replaced with linear elastic boundary interface elements that simulate the subgrade reaction (k_h accounting for the horizontal and k_v for the vertical component of subgrade reaction in Fig. 5). Though simple in its definition, the modulus of subgrade reaction has proved to be a difficult parameter to evaluate. This is because it cannot be measured in laboratory tests, but must be back-calculated from full-scale field tests. Investigations have shown it to be variable not only with the soil type and mechanical properties, but also with stress level and the geometry of the pile. In the absence of detailed information, the coefficient of subgrade reaction may be estimated by several methods. The selected formulations for timber piles are taken from Vesić [11]. Additionally, the soil at the tip of the timber piles is also simulated using non-linear no tension boundary point elements (k_{tip} in Fig. 5). In this case, the stiffness of the interface element is calculated using Boussinesq's solution for a rigid footing resting on an elastic half-space as reported by Poulos and Davis [12]. 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 analysis.

Regarding material properties, the experimental characterisation of the multi-wythe masonry of Amsterdam quay walls and inner city bridges is the subject of an ongoing experimental campaign, complementary to this work [13]. However, given the lack of complete experimental characterisation at the time of running the simulations reported in this paper, material properties for the masonry were adopted from NPR9998 [14], the Dutch code of practice for seismic assessment of structures. The values adopted from [14] in this study for the masonry refer to standard quality clay brick masonry constructed prior to 1945. The timber piles can be correlated to a C24 structural grade. Material properties for C24 strength grade of timber are taken from Eurocode 5 [15]. Material properties of soil required to calculate the stiffness of springs ($k_{soil,wall}$, $k_{soil,floor}$, k_h, k_v and k_{tip}) simulating the impedance of the soil block adjacent to the wall/below the timber floor and the subgrade reaction are adopted from CPTs performed at *Marnixkade*. All values of material properties used in the tier 2 model are provided in Table 1.

4.3 Loads

The primary traffic load considered in the investigation of the structural behaviour of *Marnixkade* is the passage of the 3 axle, 6 wheeled fire truck shown in Fig. 3b. The truck is assumed to be travelling at a velocity of 30 km/h. Additionally, a parking strip runs along the quay at ground level. This parking strip consists of spaces for parking

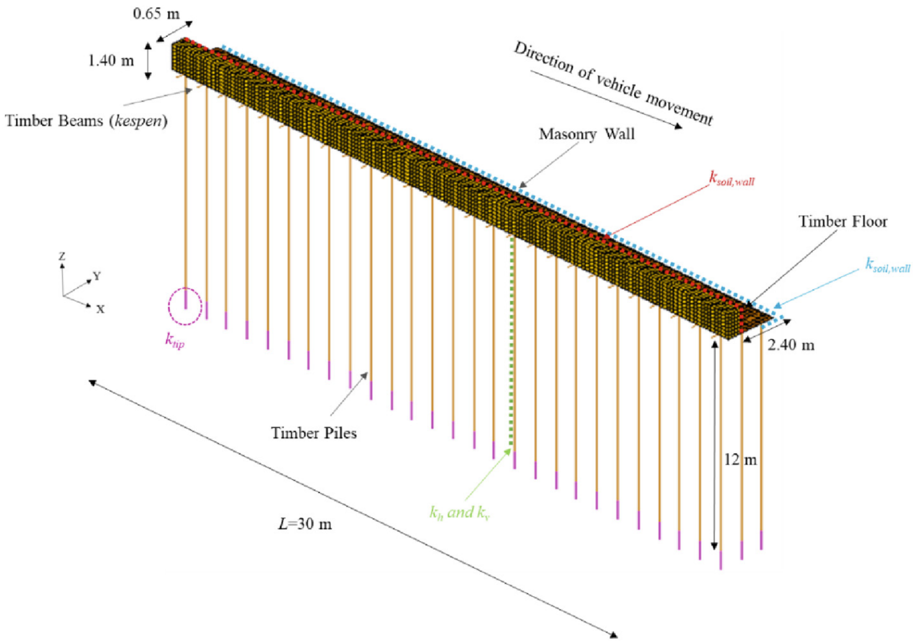

Fig. 5. Numerical model adopted for tier 2 of the procedure to assess the response of *Marnixkade* under traffic loading.

diagonally at an angle of 45° with some trees in between. A uniformly distributed load (UDL) of 5 kN/m^2 is considered for this parking load for a distance of 2.5 m from the quay wall. The effect of the parking load is also applied to the quay using the proposed tiered methodology. Assumed to be time-invariant, in this case the UDL is applied statically to the tier 1 model. σ_{Wall} and σ_{Floor} are recorded after application of the UDL and applied as a preload to the tier 2 model prior to simulating the dynamic passage of the fire truck. Other static loads applied directly to the tier 2 numerical model before simulating the passage of the fire truck include the gravity load which is automatically calculated by the software used based on the density assigned to each material and the dead load due the weight of the capstone. Soil and water pressure are applied horizontally to the quay wall. The vertical components of soil and water pressure are also applied on the top and bottom of the floor.

4.4 Results

The out-of-plane (OOP) displacements of the quay wall at three different instants during its passage are shown in Fig. 6. These instants are (a) when the fire truck has entered the quay and is at a location *L/4* where $L = 30$ m, (b) when the fire truck is at *L/2* and (c) when the fire truck is heading towards exiting the quay and is at a location *3L/4*. Very limited displacements can be observed and the wall remains in an undamaged state.

In this work, to evaluate the structural capacity of the quay, the load associated with the vehicle is progressively increased. In the context of the proposed methodology, this

Table 1. Summary of material properties adopted in the tier 2 numerical model of *Marnixkade*.

Property	Unit	Masonry	Timber	Interface Stiffness			Value
		Value	Value	Property	Direction	Unit	
Young's modulus	MPa	5000	11000	$k_{soil,wall}$	Normal	N/mm^3	0.018
Poisson's ratio		0.25	0.35		Tangential	N/mm^3	0.014
Density	Kg/m^3	1950	420	$k_{soil,floor}$	Normal	N/mm^3	0.027
Tensile strength	MPa	0.1			Tangential	N/mm^3	0.022
Fracture energy in tension	N/mm	0.01		k_h*	Horizontal**	N/mm^3	0.004–0.008
Compressive strength	MPa	8.5		k_v*	Vertical**	N/mm^3	1.678E−08–3.36E−07
Fracture energy in compression	N/mm	20		k_{tip}	Normal	N/mm^3	0.415

*Varies along the depth of the pile, range of values is provided.
**Refers to the direction of component of subgrade reaction.

requires just scaling up the σ_{Wall} and σ_{Floor} recordings made from the tier 1 model by multiplying them with a scalar *Load Multiplier*. A plot of *Load Multiplier* vs. the maximum OOP displacements recorded at mid-length ($L/2$) along the quay wall is provided in Fig. 7a. The value of *Load Multiplier* can be observed to increase monotonically, simply because it is an input to the analysis. However, the same observation does not hold true for reaction forces developed in the wall, which is an output of the analysis and the wall is modelled as non-linear. The plot of maximum developed wall reaction force in an analysis (normalised with respect to the length of the quay wall being analysed i.e. $L = 30$ m) against the same displacement values is provided in Fig. 7b. It is to be noted here that each data point in these curves corresponds to a separate analysis in which the wall was undamaged to start off with and was then subjected to the passage of a fire-truck whose wheel loads were multiplied by the *Load Multiplier*.

It can be inferred from Fig. 7 that the wall exhibits seemingly linear elastic behaviour until a *Load Multiplier* value of 5. Even at a *Load Multiplier* value of 10, the maximum OOP displacements are still limited to about 10 mm. Diagonal cracks can be observed to form in the masonry wall as the truck makes its passage though they are of limited crack width. However, what is of more interest is the observation that these cracks can close when the truck progresses along the quay wall and is no longer in its vicinity (Fig. 8). This behaviour is possible due to the dynamic nature of the loading applied. Dynamic loading, in addition to being representative of the actual nature of loading in this case, is also necessary for reliably estimating displacements: unlike static procedures, it accounts for strength-stiffness decay under repeated loading cycles and crack closure (and conversely opening) upon transition from tensile to compressive states [16].

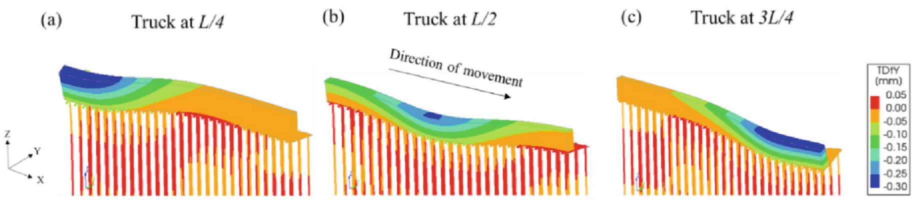

Fig. 6. Out-of-plane displacements of the quay wall during the passage of the fire-truck at different instants: (a) truck at *L/4*; (b) truck at *L/2* and (c) truck at *3L/4*. (Deformation scale factor = 2000).

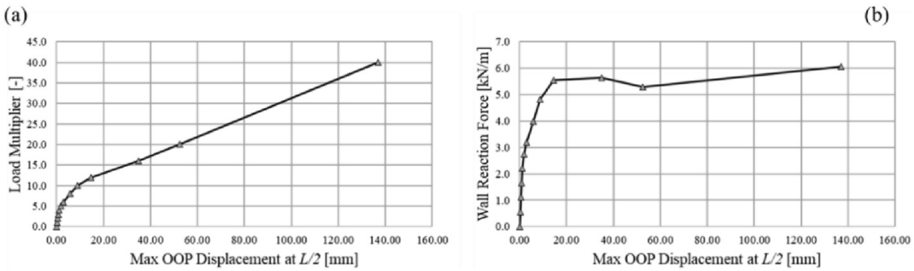

Fig. 7. (a) *Load Multiplier* and (b) wall reaction force (normalised with respect to the length of the wall) vs. max out-of-plane displacements of the quay wall at its mid-length location for each dynamic analysis.

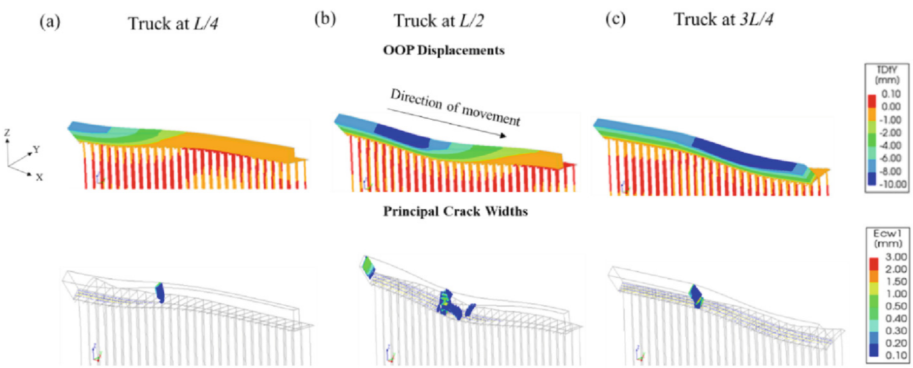

Fig. 8. Out-of-plane displacements and principal crack widths of the quay wall during the passage of the fire-truck (*Load Multiplier* = 10) at different instants: (a) truck at *L/4*; (b) truck at *L/2* and (c) truck at *3L/4*. (Deformation scale factor = 200).

Significant deformation and crack openings, indicative of collapse or near collapse are seen in the quay wall at a *Load Multiplier* value of 40. The entire wall is cracked and unlike for the analysis run with *Load Multiplier* = 10, cracks formed during the passage of the truck remain open even when the truck is not in the vicinity. These cracks also pass through the entire thickness of the wall. The wide flexural crack near the base of the wall also suggests imminent overturning of the wall (Fig. 9). The shear strength of the *kespen* is also checked to be reached at this value of the *Load Multiplier*.

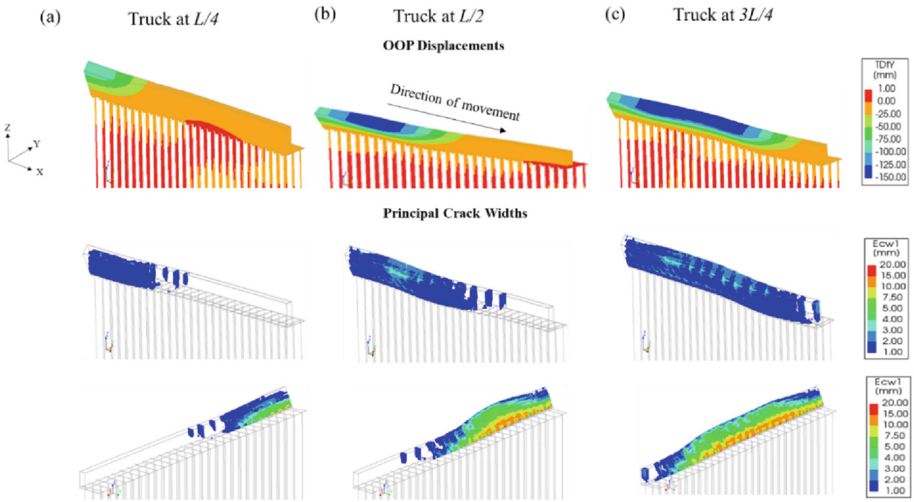

Fig. 9. Out-of-plane displacements and principal crack widths of the quay wall during the passage of the fire-truck (*Load Multiplier* = 40) at different instants: (a) truck at *L/4*; (b) truck at *L/2* and (c) truck at *3L/4*. (Deformation scale factor = 10).

5 Concluding Remarks

This paper proposes a novel methodology to numerically assess the dynamic response of masonry walls in quays under the effect of traffic loads. Adopting simplified yet conservative assumptions, the methodology simulates the passage of vehicles over numerical models of long quay wall systems while avoiding the heavy computational burden associated with the explicit modelling of the soil backfill. It is also envisaged that with appropriate modifications, the methodology can be adopted to investigate the response of other earth retaining structures under traffic loading. Application of the methodology to a case study in Amsterdam, the Netherlands, is presented and shows promising potential towards evaluating the structural capacity of masonry quay walls as well as their displacement response. Load redistribution capacity of the masonry quay walls can be identified, thanks to the adopted time- and space-variance of the applied loading and the three-dimensional model. Near collapse behaviour is eventually observed only for the single passage of a vehicle 40 times heavier than the considered fire-truck.

The current work represents a starting point for the investigation of the structural performance of quay walls under traffic loading and can be developed in several aspects. Multiple passages of vehicles should be considered instead of increasing the traffic load associated with a single vehicle to evaluate structural capacity and the effect of fatigue loading. The effect of degraded timber foundations due to environmental actions on the structural behaviour of the quay wall should be assessed. Also, homogeneous material properties have been assumed in this work for the masonry quay walls while in reality they are multi-wythe constructions partly submerged under water. Avenues for further work also include but are not limited to: cross-checking the proposed methodology against numerical models considering the soil block explicitly and validating it against experimental recordings from an instrumented quay when a vehicle travels on it.

Acknowledgements. This research was funded by the municipality of Amsterdam via the Amsterdam institute for Advanced Metropolitan Solutions (AMS) within the scope of the research programme Bridges and Quay Walls (*Programma Bruggen en Kademuren*).

References

1. Roubos, A.A., Steenbergen, R.D.J.M., Schweckendiek, T., Jonkman, S.N.: Risk-based target reliability indices for quay walls. Struct. Saf. **75**, 89–109 (2018). https://doi.org/10.1016/j.strusafe.2018.06.005
2. Korff, M., Hemel, M.-J., Peters, D.J.: Collapse of the Grimburgwal, a historic quay in Amsterdam, the Netherlands. Proc. Inst. Civ. Eng. Forensic Eng. **175**, 96–105 (2022). https://doi.org/10.1680/jfoen.21.00018
3. Longo, M., Sousamli, M., Korswagen, P.A., van Staalduinen, P., Rots, J.G.: Sub-structure-based ‘three-tiered’ finite element approach to soil-masonry-wall interaction for light seismic motion. Eng. Struct. **245**, 112847 (2021). https://doi.org/10.1016/j.engstruct.2021.112847
4. Lysmer, J., Kuhlemeyer, R.L.: Finite dynamic model for infinite media. J .Eng. Mech. Div. **95**, 859–877 (1969). https://doi.org/10.1061/JMCEA3.0001144
5. Loulizi, A., Al-Qadi, I.L., Lahouar, S., Freeman, T.E.: Measurement of vertical compressive stress pulse in flexible pavements: representation for dynamic loading tests. Transp. Res. Rec. J. Transp. Res. Board **1816**, 125–136 (2002). https://doi.org/10.3141/1816-14
6. NEBEST B.V.: Marnixkade in Amsterdam. Funderingsonderzoek. Ref: 27854-01-01, Vianen, The Netherlands (2016)
7. Rijkswaterstaat. https://www.rijkswaterstaat.nl/zakelijk/open-data/normaal-amsterdams-peil
8. DIANA FEA B.V: DIANA FEA User’s Manual - Release 10.5, Delft, the Netherlands (2021)
9. NEHRP: Soil-Structure Interaction for Building Structures. NIST GCR 12-917-21. U.S. Department of Commerce National Institute of Standards and Technology (2012)
10. Gazetas, G.: Formulas and charts for impedances of surface and embedded foundations. J. Geotech. Eng. **117**, 1363–1381 (1991). https://doi.org/10.1061/(ASCE)0733-9410(1991)117:9(1363)
11. Vesić, A.B.: Bending of beams resting on isotropic elastic solid. J. Eng. Mech. Div. **87**, 35–53 (1961). https://doi.org/10.1061/JMCEA3.0000212
12. Poulos, H.G., Davis, E.H.: Pile Foundation Analysis and Design. Wiley, New York (1990)
13. Li, X., Messali, F., Esposito, R.: Mechanical characterization of multi-wythe masonry bridge in the city of Amsterdam. In: Structural Analysis of Historical Constructions, Kyoto, Japan (2023)
14. NEN: Assessment of structural safety of buildings in case of erection, reconstruction and disapproval – Induced earthquakes – Basis of design, actions and resistances. NPR9998+C1:2020 (2020)
15. CEN: EN 1995-1: Eurocode 5: Design of timber structures - Part 1-1: General - Common rules and rules for buildings (2004)
16. Gatta, C., Addessi, D., Vestroni, F.: Static and dynamic nonlinear response of masonry walls. Int. J. Solids Struct. **155**, 291–303 (2018). https://doi.org/10.1016/j.ijsolstr.2018.07.028