Some Issues in Seismic Analysis and Design of Blockwork Wharves

R. PASQUALI¹, C. G. LAI², and M. CORIGLIANO²

¹Italian Ministry of Economic Development, Roma, Italy
²European Centre for Training and Research in Earthquake Engineering (EUCENTRE), Università degli Studi di Pavia, Pavia, Italy

During past decades, a number of ports worldwide have suffered extensive earthquake-related damage. As seaports play a key role in the world’s economy, their seismic performance should be enhanced, clearly stated and reliably pursued by designers. This work focuses on seismic vulnerability of wharves in Italy. According to a recently carried out statistical study, most of the existing wharves are gravity-type, made of superimposed, dry-connected blocks, particularly in older facilities. Such technology is widely in use worldwide but it has not attracted much research interest so far. In the present study, the validity of current, simplified design approaches has been investigated by comparison with the results of inelastic dynamic time-history analyses. Permanent displacements of the wall blocks have been calculated. Available performance criteria have been reviewed. A real wharf structure in the Port of Ancona (Italy) has been studied in depth, as a methodological example. For such structure, a parametric study has been conducted with the aim of investigating the role played by different design parameters and to assess the validity of the widely in use pseudo-static method.

Keywords Gravity Wharves; Performance-Based Design; Time-History Analyses; Retaining Walls

1. Introduction

Despite its being a relatively young discipline, earthquake engineering has marked very significant progress in recent years. Nevertheless, earthquake occurrences still show inadequate performance of many structures worldwide. Ports have shown to be highly vulnerable to earthquakes. Seaports are crucial elements in the export and import of goods and on the flow of travellers in the tourism industry of many Countries, including Italy. The presence of a port has always represented a remarkable, at times decisive, factor in development, able to produce direct and indirect effects on the economy and on the social and environmental context of the interested area. A wharf out of service in a major port is likely to cause an economic loss exceeding by far the cost of repair. The importance of sea transports, and therefore of ports, is particularly evident for a country like Italy, with its long tradition of navigation and hundreds of ports of all sizes existing along its coasts which total about 8,000 km in length.

Many of the Italian seaports are located in zones characterized by moderate to high seismicity. Figure 1 shows the seismic hazard affecting the major seaports in Italy according to the new Italian seismic code (DM January 14th, 2008). Moreover, the seismic vulnerability of port structures in Italy could be high since most of the existing

Received 16 December 2008; accepted 20 April 2009.
Address correspondence to C. G. Lai, European Centre for Training and Research in Earthquake Engineering (EUCENTRE), Università degli Studi di Pavia, via Ferrata 1 Pavia, Italy; E-mail: carlo.lai@eucentre.it
facilities were built several decades ago, without specific seismic design provisions. The combination of hazard, vulnerability, and exposure of port structures leads to a possibly high seismic risk. In fact, the consequences of earthquake-induced damage are not only related to life safety and repairing cost of the structures, but especially to the interruption of port serviceability after an earthquake. Experience gained from recent seismic events (e.g., 1989 Loma Prieta in USA, 1995 Hyogoken-Nanbu, and 2003 Tokachi-Oki in Japan earthquakes) has dramatically demonstrated the potential economic loss due to earthquake damage to seaports.

In Italy, the Department of Civil Protection has funded a research project meant to develop a methodology for the seismic design of new wharves and assessment of existing structures at seaports located in areas characterized by medium to high seismicity. The first part of the project consisted in a detailed census of the Italian major seaports which was carried out using purposely devised questionnaires meant to identify, among a variety of technical data, the existing wharf typologies [Gentile and Lai, 2007; Bartolomei et al., 2008].

For the above reasons, new design tools appear to be necessary to improve reliability of seismic performance assessment methods for port structures. At present, performance-based design methods and specifically displacement-based design approach (considered in a broad sense) appear to be the most promising solutions.

Block wharves are commonly designed using simplified, pseudo-static, force-based design approaches. This work investigated the validity of such approach. Extensive dynamic time-history analyses have been carried out in parametric fashion, using advanced numerical codes. The results of pseudo-static analyses (according to the prescriptions of EN 1998-5:2005), Newmark analyses and nonlinear time-history analyses have been compared. A real wharf blockwork structure in the Port of Ancona, in Central Italy, has been assessed as a methodological example. Furthermore, a sensitivity study has been conducted with the aim of investigating the role played by key design parameters.

2. Wharf Structures

A wharf is a structure whose purpose it is to allow berthing and mooring of ships, as well as embarking/discharging of passengers and loading/unloading of cargoes. Essentially, a wharf has to provide a transition between onshore land and a several meters deep water pool, where ships can float. Wharf structures are traditionally classified according to the following types: closed-type, open-type, and also floating wharves.
Open-type wharves (see Fig. 2a) are essentially pile-supported decks, below which the soil slopes down from deck level to the bottom of the dock.

Closed-type wharves (see Fig. 2b) feature a continuous, vertical wall bounding the dock. Such walls are subject to the thrust of backfill soil and may be either sheet pile type or gravity type. Sheet piles are generally preferred when the available space in plan is limited, but only when water depth is not excessive, because just one row of anchors can be employed (anchors below the waterline are not used).

Floating wharves may either be built at locations where tidal oscillations are particularly high or considered for minor, low-impact infrastructure like marinas. Such kind of wharves will not be considered herein.

Gravity wharves have been in use for many centuries. Among the gravity-type retaining structures, those composed of superimposed rigid blocks are possibly the most ancient. In fact, before reinforced concrete was invented, pile-supported wharves had to be made of wood. Therefore, for important infrastructure, masonry was preferred.

Nowadays, gravity wharves are made either of superimposed concrete blocks, or of monolithic reinforced concrete caissons. The use of dry-connected, superimposed blocks is often still preferred for smaller wharves, where the investment for caisson production (drydock, transportation etc.) is not justified or when the contractor does not have the required know-how. On the other hand, caissons allow building deeper docks, which

FIGURE 2 Examples of: a) open-type wharves; b) closed-type wharves (from up to down: anchored sheet pile wall, caisson wall, blockwork wall) (after Werner, 1998; PIANC, 2001).
would require blocks too large to be handled. Both kinds of walls require a good foundation soil, because the contact pressure at the base is always quite high. For this reason, gravity retaining structures are not suitable when the foundation soil is too compressible or highly non uniform. Indeed, recent case-studies have shown seaports where extensive soil replacement was carried out (at foundation level) to make the construction of a caisson wall possible.

Compared with closed-type structures, pile-supported wharves have a major advantage: they are not retaining structures and therefore increasing water depth has only a minor influence on structural member size. Thus, this technology is often preferred when deep docks are needed. In addition, the absence of a vertical reflecting boundary helps reduce residual waves in the dock. On the other hand, a rockfill dam is needed below the dock, which can be a problem if quarries are not available nearby. In such cases, closed-type wharves are often preferred because gravel or even sand is sufficient for backfilling. In the past, also silt was sometimes employed, leading to potential liquefaction problems.

As mentioned above, one of the objectives of the census recently carried out on the major Italian seaports, was the identification of the most common structural typology for wharves. A sample of 5 (out of 25) major Italian seaports was studied for a total of 46 wharves. It turned out that the majority of Italian wharves are made of superimposed concrete blocks (Fig. 3). Hence, it appears that such typology is worth careful study, with the aim of improving vulnerability assessment.


As already mentioned, the problem of seismic vulnerability of wharves is basically an economic issue. Continuous serviceability during and after earthquakes may also be an issue (e.g., for facilities that have to be used by rescue units to reach the stricken area). Life safety is rarely a matter of concern [Arulmoli et al., 2008], first of all because few people physically stay on these structures, and furthermore because failure modes are such that human life is hardly at risk (one exception might be the potential overturning of cranes or the falling of special equipment).

Looking at the case histories from past earthquakes and focusing on gravity wharves, more information can be found about caisson wharves than blockwork wharves. However, this does not necessarily imply that blockwork wharves generally behaved
better. Possibly, the reason is that caissons were in more widespread use at seaports struck by recent strong earthquakes.

As a matter of fact, most of the available case histories concerning damage to caisson quay walls refer to ports in South-Eastern Asia, where this kind of structure is widely in use. The Great Hanshin earthquake (formerly known as the Hyogoken-Nambu earthquake) hit the Port of Kobe (Japan), on January 17, 1995, and inflicted severe damage to a number of caisson quay walls (see Fig. 4). The earthquake-induced damage to seaport structures caused by the Great Hanshin earthquake attracted the engineering community’s attention on the potential vulnerability of this typology of structures. However, other earthquakes, before and after 1995, seriously damaged several caissons quay walls, and not only in Japan.

The review of damage suffered by seaport structures worldwide due to earthquakes [PIANC, 2001; Borg and Lai, 2007] shows that most of the damage appears to have been caused by liquefaction. Ports and reclaimed land in general are particularly prone to liquefaction, due to the presence of widespread, saturated loose sand and silt deposits. If liquefaction occurs, the performance of wharves will probably be very poor. It is unlikely that closed-type structures could resist the thrust of a liquefied backfill, while pile-supported wharves will suffer the kinematic actions induced by the lateral spreading of the soil. The effect of liquefaction on wharves depends also on the distance between the liquefied zones and the wharf itself. If there are several tens of meters of non liquefying soil behind the wharf, they will act like a gravity dam to restrain the lateral spreading of the liquefied soil, which will then only settle.

PIANC [2001] listed a number of selected earthquake-related damage case histories, referred to different locations and earthquakes. During the Great Hanshin Earthquake (Japan), a caisson quay wall in the Rokko Island (RC5) underwent 4.2–5.2 m horizontal displacement, associated with 1.5–2.2 m vertical settlement and 4°–5° tilting. Deformation of loose foundation soil was deemed to have caused such impressive damage, while the role played by liquefaction was controversial. The recorded peak

FIGURE 4 Damage of a quay-wall at the Port of Kobe during the 1995 Hyogo-ken Nanbu Earthquake [Nozu et al., 2004].
ground acceleration (PGA) was 0.53 g, and the recorded peak ground velocity (PGV) was 1.06 m/s.

During the 1993 Kushiro-Oki earthquake (Japan), the East Quay, Kita Wharf, East Port District at Kushiro Port displaced horizontally some 1.9 m, with 0.2–0.5 m vertical settlement. The recorded PGA was reported to be equal to 0.47 g, and PGV equal to 0.63 m/s. Liquefaction of the backfill was recognized as one of the causes of damage [PIANC, 2001].

In Taiwan, during the 1999 Chi-Chi earthquake, the No. 1 Quay Wall at Taichung Port underwent 1.5 m horizontal displacement, despite its being subject only to 0.16 g measured PGA. No information is provided about PGV [PIANC, 2001]. The displacement was most likely caused by backfill liquefaction.

Case histories about seismic damage to block wharves also exist, although less extensive and not so well documented. As already mentioned, this is not sufficient for claiming a better seismic response of this kind of structure, possibly because such wharf type is less employed in developed, seismic-prone countries, at least for major port infrastructure.

Pitilakis and Moutsakis [1989] described the case of the main Kalamata harbor quay wall, which was subjected to 0.2–0.3g PGA and 0.4 m/s PGV during the 1986 Kalamata earthquake (Greece). The wharf displaced horizontally about 0.15 m and tilted 4°–5°, but remained serviceable during and after the earthquake.

PIANC [2001] reported the case of a block quay wall in San Antonio Port (Chile) that partly collapsed during the 1985 Chile earthquake. The PGA was 0.67 g, the PGV value is not available. Also, the case of Quay No. 34 at Port of Algiers (Algeria) is described by PIANC (2001). Such block quay wall underwent 0.5 m horizontal displacement and 0.3 m vertical displacement. PGA and PGV of the event are not specified.

According to PIANC [2001], the evidence of damage due to past earthquakes to gravity quay walls suggests the following remarkable points:

- most damage is associated with deformation of soft or liquefiable foundation soil. Liquefaction is a particularly important issue, because of the widespread use of hydraulic filling techniques for walls backfills;
- damage is governed by displacements: most commonly the walls slide seawards, and may also be subject to residual tilting. This is not a catastrophic collapse, but it involves settlement of the apron and consequent damage or failure of supported structures.


Despite the substantial improvements achieved in earthquake engineering, seismic performance of wharf structures is still not satisfactory, as clearly shown by recent earthquakes. Seismic design of gravity wharves is currently performed by using the same methods generally employed for gravity retaining walls, even if the presence of water undoubtedly complicates the analysis. Each of the available design procedures has advantages and drawbacks, as will be briefly outlined below.

4.1. Force-Based Design Approach: the Pseudo-Static Method

The pseudo-static method is the eldest and most widely employed method for the seismic design of retaining structures. It is based on equilibrium check of the wall considered as a
rigid body, in exactly the same way as it is done in non seismic conditions. The effect of the earthquake is simulated by a set of static forces that are added to other non seismic actions. The pseudo-static approach takes into account, in a simplified fashion, the following dynamic phenomena: the inertia of the wall itself, the increment of soil thrust due to seismic shaking and the hydrodynamic effect due to the presence of a large water pool in front of the wall.

The increment of soil thrust due to earthquakes is usually calculated according to Mononobe-Okabe theory [Okabe, 1926; Mononobe and Matsuo, 1929; Kramer, 1996]. The inertia of the wall itself is given by the design acceleration applied to the mass of the wall. The hydrodynamic effect is usually evaluated according to Westergaard’s theory [Westergaard, 1933], which has received wide acceptance and has been incorporated into several seismic codes worldwide (e.g., EN 1998-5:2005).

A delicate point in the implementation of the pseudo-static procedure lies in the choice of an appropriate value of the “seismic coefficient”. Such a parameter is the ratio between the design acceleration (horizontal or vertical component) and the acceleration of gravity. In most seismic codes, it is calculated by applying to the design PGA a so-called “reduction factor”, whose value depends on the amount of displacement tolerable by the structure and which is determined empirically [Hynes Griffin and Franklin, 1984; DM Jan 14th 2008; EN 1998-5:2005; Arulmoli et al., 2008; Pasquali, 2008].

In principle, as the pseudo-static method is based on the free-body diagram for a rigid solid, it seems reasonable to assume that the wall is subject to the design Peak Ground Acceleration (PGA). However, experience has shown that, in high seismicity regions, such an approach would be too conservative, predicting the collapse of most existing gravity walls, which actually survived strong earthquakes. Starting from this empirical observation, engineers have realized that for most gravity retaining walls, and especially for the squatter ones, sliding is the governing failure mechanism. Such a mechanism is associated with a very large (virtually infinite) ductility. In fact, the sliding resistance of the wall is not affected by previous sliding, and after an earthquake the static factor of safety is the same as before, even if the wall did slide meanwhile.

The behavior of a sliding wall recalls the ideal model of perfect plasticity. In addition, the displacement capacity of a sliding wall is not limited by the structural requirements of the wall itself, but only by serviceability or collapse limits of retained structures. On the basis of such observations, geotechnical engineers have borrowed from structural engineers the concept of ductility, stating that during an earthquake, temporary loss of equilibrium of a retaining wall that fails by sliding may be acceptable, as long as it leads to a tolerable displacement of the wall.

The direct consequence of this reasoning is that checking equilibrium under seismic actions is not really necessary. The key point lies in calculating the permanent displacement of the wall. However, in the early days of earthquake engineering that was not easily done. Even with the simplest mechanical model, the numerical integration of the equations of motion would have required some type of automatic calculation, which was at that time a major problem. In addition, sufficient strong motion accelerograms were not available, so they could not be used as an input.

In contrast, the equilibrium method was a simple and well-established procedure, which could be carried out by hand. So it seemed natural to extend it to the seismic case, by introducing the “seismic coefficient” concept [Kramer, 1996; EN 1998-5:2005]. Although numerical integration of earthquake acceleration time histories is nowadays straightforward, the pseudo-static method is still appealing, partly because of its simplicity, partly because several earth-retaining structures, designed according to this approach, showed satisfactory performance when subject to real earthquakes.
Of course, bad performance also occurred, and gravity wharves seem to have been especially prone. Poor performance has been recorded for these structures in recent earthquakes (e.g., the Hyogoken-Nanbu, Japan, in 1995). Moreover, the pseudo-static approach has all the limitations of force-based design methods applied to earthquake engineering [Priestley et al., 2007] and it only gives a conventional measure of safety, while performance-based design focuses on the actual wall displacement, thus allowing an explicit estimate of the expected damage.

4.2. Performance-Based Design

Compared to other kinds of structures, the implementation of a performance-based design procedure for a gravity wharf is relatively straightforward. In fact, for a generic structure subject to earthquake, it is not easy to identify the governing failure mechanism and to define, at a structural level, the damage criteria related to performance targets [Priestley et al., 2007].

Conversely, for a gravity wharf it is rather easy to obtain sliding as the governing failure mode [Pasquali, 2008]. This can be checked, for example, by a pushover method: a stepwise increasing horizontal force is applied to the wall, in addition to static actions. The force may be applied at 2/3 of the wall height, or even at the top of the wall, to be most conservative. At each increasing force step, equilibrium is checked against the wall sliding, overturning and the foundation soil failure. The governing failure mode is the first mechanism whose associated safety factor drops below unity. If both overturning and excessive residual tilting are excluded, the only parameter to be checked against performance criteria is the residual horizontal displacement of the wall. In addition, for this kind of structures, the maximum and the residual displacement are basically the same (which is not the case for most other structures). This further simplifies the problem. Therefore, the main issue for performance-based design of a gravity retaining structure is the accurate evaluation of the wall displacement; therefore this is the core of the present work.

Even though some general guidance may be useful for preliminary design (see, e.g., PIANC, 2001), performance criteria for gravity wharves should be established on a case-by-case basis, according to a site-specific risk assessment. For example, for wharves supporting cranes the maximum acceptable displacement may be governed by tolerable deflection of the rails. If there are buildings on the apron, vertical settlements related to the wall’s sliding may be the limiting factor. If no structure is supported, then relatively large displacements may be acceptable. Seismic safety of stacked containers is generally governed by equilibrium with respect to overturning, taking into account the possible rocking behavior.

The examples given above show how the design criteria can be customized for each design, according to the specific performance needed.

4.2.1. Simplified Displacement-Based Approaches: Newmark-Type Methods. In his 1965 Rankine lecture, Newmark outlined a method for evaluating seismic behavior of dams and embankments, based on the analogy with a block sliding on an inclined plane [Newmark, 1965]. The method requires the definition of the threshold ("yield") acceleration that would induce incipient movement (i.e., unit factor of safety) of the rigid body under study, which can be computed using a pseudo-static method. A double numerical integration of the part of acceleration time history exceeding the yield acceleration provides an estimate of the final displacement of the rigid body.

In the original Newmark’s work, it is not suggested that designers should carry out such accelerogram integration. Instead, an effort is made to give closed-form expressions
and design charts to be directly used for design. The following expression was suggested as an upper bound for permanent displacements:

\[ d_{perm} = \frac{PGV^2 \cdot PGA}{2a_y^2}, \]  

(1)

where \( PGV \) is the peak ground velocity, \( PGA \) is the peak ground acceleration, and \( a_y \) is the yield (or threshold) acceleration. The formula is claimed to be valid for:

\[ a_y \geq 0.17PGA. \]  

(2)

Newmark’s [1965] work was based on the analysis of only four accelerograms and many parts of his article are nowadays outdated. However, several fundamental concepts of earthquake engineering were stated there and they are still valid, including the idea of performance-based design focusing on displacement evaluation and different performance levels associated to different design earthquakes.

Nowadays, recognizing that many specific features of the input accelerogram may influence the final result (duration, dominant frequencies, etc.), the Newmark’s method is most commonly applied with direct integration of the design accelerogram(s). Free, reliable software is available for efficiently performing such numerical integration (e.g., Jibson and Jibson, 2003), so that users only have the task of calculating the yielding acceleration and selecting suitable input accelerograms. As a consequence, the interest for the above mentioned closed-form expressions is nowadays limited and the case-specific integration of design accelerograms is often referred to simply as Newmark’s method.

After Newmark’s pioneering work, many other researchers followed in his wake, some of them trying to give improved closed-form expressions for slope displacement (a complete set of references is listed in Kramer, 1996). At present, a whole family of formulae exists based on Newmark-type method.

Although the Newmark’s 1965 work was originally referred to dams and embankments, it has been extended without conceptual changes also to the seismic analysis of retaining walls [Kramer, 1996]. Obviously, when calculating the yield acceleration, the most critical failure mechanism needs to be considered. As mentioned above, for gravity retaining walls, the governing failure mechanism is commonly (and desirably) that of base sliding.

Several authors proposed closed-form solutions to predict permanent displacement of retaining walls. Richard and Elms [1979] proposed the following expression for permanent wall displacement:

\[ d_{perm} = 0.087 \frac{PGV^2 \cdot PGA^3}{a_y^3}, \]  

(3)

where \( PGV \) is the peak ground velocity, \( PGA \) is the peak ground acceleration, and \( a_y \) is the yield acceleration for the wall-backfill system. Such expression has been obtained similarly to Eq. (1) and it is reputed to give similar results [Kramer, 1996]. However, it is commonly referred to (and so it will be in the following) as the “Richards and Elms method”.

Starting from the work of Richards and Elms [1979], Whitman and Liao [1985] identified a certain number of modelling errors derived from the simplified assumptions of the Newmark sliding block procedure when applied to earth-retaining structures.
Whitman and Liao [1985] applied a statistical method to combine different sources of modelling errors and found that the permanent displacements were lognormally distributed with mean value:

\[
d_{perm} = 0.37 \cdot \frac{PGV^2}{PGA} \cdot \exp\left(-\frac{9.4a_y}{PGA}\right),
\]

where, as usual, \(d_{perm}\) is the mean value of the permanent displacement of the wall, \(PGV\) is the peak ground velocity, \(PGA\) is the peak ground acceleration, and \(a_y\) is the yield acceleration.

In the above-mentioned formulae, preliminary calculation of the yield acceleration is required. The user is free to choose the most suitable method for calculating such a value, but in practice the threshold acceleration is normally calculated according to the pseudo-static method, by looking to the value of the seismic coefficient that makes the safety factor to sliding equal to unity. This implicit problem can be easily solved by means of a spreadsheet as it was done in the present study. Closed-form formulae like the ones provided by Richards and Elms [1979] and Whitman and Liao [1985] can then be used for the direct design of the wall instead of the usual trial-and-error procedures. Examples of this kind are provided by Ebeling and Morrison [1992] and Kramer [1996].

4.2.2. Inelastic Time-History Analyses. The computation of the seismic-induced deformations of earth-retaining structures can be done by Inelastic Time Histories Analysis. This method, often referred to with the acronym ITHA, represents a procedure for solving the equations of motion of a mechanical system subject to a specific set of forces.

Unless very simple systems are considered, the equations of motion have to be solved numerically. Finite difference (FDM) or finite element (FEM) methods are commonly in use to build a numerical model of the physical system at hand. The earthquake is described by an imposed load or displacement/velocity time-history. Most commonly, the accelerograms are used, but also velocity, stress or displacement time-histories can be imposed at some nodes or boundaries of the model.

This kind of analysis gives as an output the time-histories of a number of kinematic and mechanical user-selected quantities (such as accelerations, velocities, displacements, stress, strain, and so forth). Obviously, the quality of the results depends on the accuracy and reliability of all the input parameters, particularly the soil constitutive parameters and the soil-structure interface coefficients.

The modeling of nonlinear, dynamic soil-structure interaction, taking also into account the presence of water, is one of the most complex computational problems in civil engineering. However, nowadays, commercial software implementing advanced material modelling and robust numerical algorithms is available. As a result, nonlinear THA of large mechanical models can be efficiently carried out.

Each available computer program includes its suites of soil constitutive models, some of them allowing effective stress dynamic analyses and explicit modelling of highly nonlinear phenomena such as strain localization and liquefaction.

Actually, advanced constitutive modelling of such complex phenomena is still more art than science, in that the analyst has no precise guidance concerning fundamental issues such as the geometric extension of the model needed to avoid spurious effects, mesh size, optimal values to be adopted for viscous damping of soils, etc.

In many cases, the need to provide a practical answer to an engineering problem drives the analyst to perform numerous trial tests for the purpose of validating the model.
This usually starts by trying to reproduce known solutions of simple problems. Once this is achieved, and therefore confidence has been gained about the basic features of the model, this one is gradually refined, by introducing additional sources of complexity [Itasca, 2005].

5. Case Study: A New Wharf at Ancona Seaport in Italy

In order to investigate the capabilities and limitations of available analysis methods to gain an insight into the seismic response of blockwork wharves, a real structure was studied in details, using state-of-the-art techniques. The Fincantieri wharf is a new berthing structure that is going to be built at the port of Ancona (Italy). All relevant parameters have been taken from the original drawings and design reports, which were made available by the Port Authority of Ancona. A schematic cross section of the wharf is shown in Fig. 5.

The traditional pseudo-static method gives as an output the safety factors with respect to sliding, overturning and foundation bearing failure. Newmark’s method and ITHA give, with different accuracy, the final displacement of the wall. In order to obtain comparable results, a procedure was developed to estimate the wall displacement also from current pseudo-static approach, as described below.

According to Eq. 7.1 and Table 7.1 of Eurocode 8 Part 5 (but several other seismic codes have a similar approach), when applying the pseudo-static method, the maximum allowable wall displacement must be chosen a-priori, in order to select the appropriate PGA reduction factor (see EN 1998-5:2005). Then a standard equilibrium analysis is carried out.

If the so-calculated safety factor is equal to 1, it means that the wall will undergo the maximum acceptable displacement. If the factor of safety obtained is greater than unity, it is reasonable to assume that the permanent displacement will be proportionally smaller. This is an assumption, not backed by the code, but it is a reasonable one. If we accept it, we may say that, in general:

\[ d_{\text{actual}} = \frac{d_{\text{allowable}}}{FS}, \]

where \( FS \) is the safety factor of the pseudo-static method, \( d_{\text{actual}} \) is the final displacement, and \( d_{\text{allowable}} \) is the allowable displacement according to EN 1998-5:2005.

![FIGURE 5 Cross section of the new Fincantieri wharf at Port of Ancona, Italy.](image-url)
In other words, even if the safety factor is greater than 1, it is not sure that the wall will not move: it will displace less than initially assumed, but it will not remain in its initial position, unless no reduction factor is considered (i.e., the wall is checked for the real peak design acceleration).

Within this framework, the only assumption underlying Eq. (5) is the following: if the factor of safety is greater than unity, then the displacement of the wall can be, to a first approximation, considered to be proportionally smaller than the allowable displacement. Following this assumption, also a factor of safety smaller than unity does have a meaning, which is that the wall will displace more than the designer had assumed.

For the wharf under study, the acceptable displacement has been set equal to 60 mm, consistently with the original design documents where a reduction factor equal to 2 was used. The proposed formula allows converting a static factor of safety into a displacement estimate. This approach, despite its certainly being not accurate, is consistent with the assumptions of current code-compliant implementation of the pseudo-static method. It was used in this study only as an attempt to compare the results of methods that are different in principle, but share the same final purpose: the design of retaining structures.

The authors are aware that trying to extract an estimate of displacement from a pseudo-static approach may sound a bit foreign. However, if the different design approaches are seen as different paths leading to the same place (the design of the wall), there is ground to argue that there should be a criterion to judge which path is best to follow. In this case, the criterion is felt to be the ability to forecast seismic performance and seismic performance means, indeed, displacement.

### 5.1. Seismic Input

Time history analyses and Newmark approach require the seismic input to be specified in terms of accelerograms. According to Eurocode 8 Part 1, the accelerograms may be either recorded in real earthquakes, artificial (i.e., generated using stochastic algorithms), or simulated (i.e., generated by modelling the earthquake mechanics).

For time-history analyses applied to geotechnical problems, the use of real accelerograms is preferred because they are more realistic as to the frequency and energy content, number of cycles, duration, and temporal correlation between the vertical and horizontal components of ground motion [EN 1998-1,5, 2005, Bommer and Acevedo, 2004].

A set of accelerograms used for dynamic analyses should be spectrum-compatible, meaning that the average response spectrum computed from all the accelerograms should match a target spectrum (e.g., obtained from a probabilistic hazard study or prescribed by a seismic code) within a certain tolerance, over a specified range of periods. Spectral compatibility is not simply achieved when recorded accelerograms are used; however, it is an important requirement in order to avoid using records that are inconsistent with code prescriptions or with the results of a seismic hazard study [Dall’Ara et al., 2006].

In this work, the seven accelerograms to be adopted for dynamic analyses were selected from strong motion record databases, by imposing the constraint of spectrum compatibility with a code-based target spectrum. The spectral shape of Eurocode 8 type 1 was used, considering ground type A (rock site $V_{S30} \geq 800\text{m/s}$).

In order to investigate the sensitivity of the results to the severity of ground shaking, two different values of PGA were considered, 0.20 and 0.25 g: in the original design of the Fincantieri wharf, a PGA value equal to 0.20 g had been adopted. Table 1 summarizes the most important seismological characteristics of the selected records, which were all scaled to the target PGA (i.e., 0.20 and 0.25 g). Figures 6 and 7 show, respectively, time
Table 1: Main seismological characteristics of the selected accelerograms

<table>
<thead>
<tr>
<th>Earthquake</th>
<th>Distance (km)</th>
<th>Data</th>
<th>M_L</th>
<th>M_S</th>
<th>M_W</th>
</tr>
</thead>
<tbody>
<tr>
<td>Friuli (aftershock)</td>
<td>16</td>
<td>11/09/1976</td>
<td>5.7</td>
<td>5.52</td>
<td>5.6</td>
</tr>
<tr>
<td>Montenegro</td>
<td>16</td>
<td>15/04/1979</td>
<td>–</td>
<td>7.04</td>
<td>–</td>
</tr>
<tr>
<td>Kalamata (Southern Greece)</td>
<td>10</td>
<td>13/09/1986</td>
<td>5.5</td>
<td>5.75</td>
<td>–</td>
</tr>
<tr>
<td>Erzincan (Turkey)</td>
<td>13</td>
<td>13/03/1992</td>
<td>–</td>
<td>6.75</td>
<td>–</td>
</tr>
<tr>
<td>Ionian (Greece)</td>
<td>18</td>
<td>23/03/1983</td>
<td>5.5</td>
<td>6.16</td>
<td>–</td>
</tr>
<tr>
<td>Parkfield</td>
<td>11.6</td>
<td>28/09/2004</td>
<td>–</td>
<td>–</td>
<td>6.0</td>
</tr>
<tr>
<td>Parkfield</td>
<td>14</td>
<td>28/09/2004</td>
<td>–</td>
<td>–</td>
<td>6.0</td>
</tr>
</tbody>
</table>

histories and Fourier amplitude spectra of the seven unscaled accelerograms. Figure 8 shows the response spectrum of the seven selected accelerograms and their average spectrum, compared to the Eurocode 8 type 1 spectrum scaled to a PGA equal to 0.25 g. Table 2 summarizes the PGV values for the seven records for the two levels of PGA.

5.2. Parametric Analyses

The wharf at the seaport of Ancona (Italy) was studied using the pseudo-static approach, the Newmark sliding block analysis and the Inelastic Time Histories Analysis (ITHA).

FIGURE 6 The seven unscaled accelerograms selected for dynamic analyses of the wharf structure at the Port of Ancona, Italy.
FIGURE 7 Fourier amplitude spectra of the seven unscaled records selected for dynamic analysis of wharf structure.

FIGURE 8 a) Elastic response spectrum for the seven selected records and their average response spectrum for PGA = 0.25 g; b) Comparison of the Eurocode 8 type 1 acceleration response spectrum (target spectrum) and average response of the set (damping ratio 5%).
The set of accelerograms described in the previous section was used for both the sliding-block analyses and the ITHA. The analyses were carried out considering both polarities for each record, leading to 14 runs for each set of analyses.

In order to allow the results from all methods to be compared, only the horizontal component of ground motion was considered in all of them, as the Newmark method in its classic formulation cannot account for vertical shaking.

Actually, there are ways to include the effect of vertical ground motion in the Newmark approach: if a constant ratio of vertical to horizontal acceleration is assumed, then the effect of vertical shaking in reducing the yield acceleration is constant over the whole time-history, and can be easily accounted for. On the other hand, when using recorded triplets of accelerograms (which should be the standard practice), two independent acceleration time histories are normally to be considered in the vertical and horizontal direction. In such a case, the yield acceleration must be recalculated at each integration step, and although in principle this can certainly be done, nevertheless it is not easily achieved and would require specifically developed software. For such reason, as this research aimed to investigate those methods which are available to practicing engineers, it was decided to consider only the design approaches that could be pursued by using commercially available software, i.e., neglecting the effect of vertical earthquake. Nonetheless, the reader is warned that this is an important issue when code-compliant design has to be carried out.

Pseudo-static analyses were carried out according to the prescriptions of Eurocode 8 – Part 5 [EN 1998-5:2005]. To investigate the effects of the main design parameters such as PGA, wall typology (e.g., blockwork or monolithic), friction angle at the base, friction angle at the block-to-block interface, angle of shearing resistance of the backfill, a thorough sensitivity analysis was carried out. The main objectives of these analyses are twofold.

1. Assess the performance of the available methods for analysis and design of monolithic, gravity retaining walls, and especially gravity wharves. In order to help the designer to choose the most suitable analysis tool, it is essential to understand the cost/benefit ratios;
2. Compare the seismic response of blockwork wharves to the corresponding monolithic structures, since simplified methods can only be used for monolithic walls, while explicit modelling of blockwork structures requires the use of numerical analyses and advanced computer programs. Therefore, for design purposes, it would be interesting to investigate to which extent a blockwork wall may be studied by using simplified methods as if it were monolithic.

In order to achieve these goals, different logic tree scenarios were considered. The geometry and the material properties of the original documents were taken as a reference. The geometry was kept unchanged while the main parameters of the soil-structure model were varied to generate different branches of the logic tree. Figure 9 summarizes the

---

### TABLE 2
Peak ground velocity of the seven records selected for dynamic analysis of wharf for 2 levels of PGA

<table>
<thead>
<tr>
<th></th>
<th>EQK 1</th>
<th>EQK 2</th>
<th>EQK 3</th>
<th>EQK 4</th>
<th>EQK 5</th>
<th>EQK 6</th>
<th>EQK 7</th>
</tr>
</thead>
<tbody>
<tr>
<td>PGA=0.20g</td>
<td>0.100</td>
<td>0.298</td>
<td>0.246</td>
<td>0.529</td>
<td>0.086</td>
<td>0.118</td>
<td>0.135</td>
</tr>
<tr>
<td>PGA=0.25g</td>
<td>0.125</td>
<td>0.372</td>
<td>0.308</td>
<td>0.661</td>
<td>0.108</td>
<td>0.147</td>
<td>0.169</td>
</tr>
</tbody>
</table>
studied cases. The first group of analyses investigated the influence of friction at the base of the wall for two values of PGA (0.20 and 0.25 g) and for two assumptions about the wall behaviour (e.g., blockwork or monolithic) for a total of 8 cases which, considering the number of records, amounted to a total of 112 dynamic analyses.

The second group of analyses investigated the influence of the friction between blocks, adding 2 more cases which required 28 dynamic analyses. The last group of analyses investigated the influence of the backfill strength, adding 4 more cases which amounted to 56 dynamic analyses. ITHAs were performed for each of the branches shown in Fig. 9, whereas pseudo-static and Newmark-type analyses were performed only for the case of monolithic wall.

5.2.1. Pseudo-Static Method. In principle, when performing pseudo-static analyses of a block quay wall, the non monolithic nature of the structure should be taken into account. As the wall is composed of stacked blocks, the stability check should be carried out for each possible block failure mechanism.

With respect to sliding, the top blocks may slide over those beneath. Such a failure mechanism may have a safety factor lower than the one associated to the whole wall.

![FIGURE 9](logic_trees.png) Logic trees assumed for the parametric study carried out for the Ancona (Italy) wharf structure.
sliding at foundation level, due to the fact that the weight of the wall (and therefore the resisting force due to friction at the interfaces) may increase with height more rapidly than the sum of horizontal actions.

On the contrary, with respect to overturning, it is generally appropriate to consider only the case of the entire wall, as if it were monolithic. In fact, for standard geometries, the height-to-width ratio of the potentially overturning rigid mass increases with height, leading to lower values of the safety factor as the number of blocks increases.

Summing up, for a wall composed of \( n \) stacked blocks, \( n \) sliding checks should be carried out, considering \( n \) walls (composed of 1, 2, \ldots, \( n \) blocks) from top to bottom. Therefore, the total number of required calculations may become large and some type of automatic computing is desirable. In current design practice, the outlined procedure is rarely implemented. More often, blockwork walls are analyzed by the pseudo-static method as if they were monolithic. Such an approach was also followed in the original design of the Fincantieri wharf and for the sake of comparison it was adopted also in this study.

Pseudo-static analyses were carried out using a spreadsheet implementing the procedure of Eurocode 8 Part 5 [EN 1998-5:2005] and require no clarification.

5.2.2. Newmark-Type Methods. As already mentioned, Newmark-type analyses were carried out by considering the wharf as a monolithic block. In theory, it could be possible to consider separately each possible sliding mechanism, as discussed in the previous section referring to the pseudo-static method. However, this would be impractical (\( n \) analyses would be needed for a wall made of \( n \) blocks) and, most importantly, the problem would arise of defining the input motion for the upper blocks. In fact, the sliding of the underlying blocks would modify the acceleration time-history transmitted to the superimposed blocks with respect to the foundation input motion. For this reason it was decided to avoid complications that may only give the illusion of better accuracy.

Rigorous block sliding Newmark type of analyses were carried out by using the Jibson and Jibson [2003] code (in Fig. 18 this was shown with the label “Newmark THA (USGS)”, as the code is distributed by the USGS website). The results of the Newmark analyses have been checked using a different software package. The maximum difference in terms of average results for each considered scenario (average of 14 runs) is less than 10% and it is larger when it comes to smaller displacements. This is probably due to different integration schemes adopted by the two programs and it is negligible in practice.

Moreover, closed form solutions (i.e., Richards and Elms, 1979; Whitman and Liao, 1985) were used for direct calculation of the final displacement without carrying out the integration of input accelerograms. Table 3 summarizes the values of the yield acceleration used for the Newmark-type analyses and closed-form solutions. The results will be discussed in Sec. 5.3.

| Table 3 Yield accelerations computed using the pseudo-static approach |
|-----------------|-----------------|----|
| \( \delta = 25^\circ \) | \( \varphi = 30^\circ \) | 0.081 |
| \( \delta = 25^\circ \) | \( \varphi = 40^\circ \) | 0.099 |
| \( \delta = 30^\circ \) | \( \varphi = 40^\circ \) | 0.126 |

\( \delta \) = friction angle between soil and wall.
\( \varphi \) = angle of shearing resistance of the backfill.
5.2.3. Inelastic Time-History Analyses (ITHA). Dynamic time-history analyses were performed using the finite difference code FLAC v5 [Itasca, 2005], which is a worldwide known and extensively validated program, particularly suited for nonlinear dynamic analyses of geotechnical systems. A two-dimensional model of the blockwork wall under plane strain conditions (see Figs. 10 and 11) was adopted.

The soil was modeled using a linear elastic-perfectly-plastic constitutive law, with the Mohr-Coulomb yield criterion and a non associated flow rule (i.e., the soil dilatancy angle was assumed to be equal to zero) with cohesion equal to zero and angle of shearing resistance equal to 30° or 40° according to the group of parametric analyses showed in Fig. 9. The blockwork wall was modeled using linear elastic, isotropic plane strain elements. The material parameters used for THA are listed in Table 4.

Hydrodynamic effects due to the presence of the water pool in front of the wharf were taken into account by introducing an additional mass distribution along the height of the wall, according to the Westergaard’s theory [Westergaard, 1933; EN 1998-5:2005]. The soil strength parameters have been factored according to the prescriptions of Eurocodes, the partial safety factors specified in EN 1998-5:2006, 3.1 were applied (i.e., $\gamma_{cu} = 1.4$, $\gamma_{\phi'} = 1.25$). Elastic moduli were not factored.

![FIGURE 10](image1.png)

**FIGURE 10** Fincantieri wharf: schematic model assumed for pseudo-static analyses consistent with original design.

![FIGURE 11](image2.png)

**FIGURE 11** (a) soil column used for comparison with SHAKE results; (b) final FLAC model; (c) wall close-up.
Interface elements between blocks were used to model the blockwork wall. In order to simulate the state of stress into the soil under static conditions before the earthquake occurrence, the stages of excavation and wall construction were explicitly simulated. Further details concerning the modeling of the wharf structure, including also a discussion of the underlying assumptions are out of the scope of this article. The interested reader is referred to Pasquali [2008].

Numerical modelling of dynamic soil-wall interaction problems, taking also into account the presence of water, is not a trivial problem. Although significant progress has been made in recent years in computational geotechnics, several issues are still debated among the scientists, for instance how to correctly simulate energy dissipation within the model (i.e., radiation, intrinsic, and viscous damping), or how to achieve the optimal balance between accuracy of results and computational efforts.

The geotechnical model in Fig. 11 was built step by step, starting from a simple model and gradually increasing the degree of complexity after consistent and stable results were obtained for the previous phase of the analysis.

The amount of viscous damping to be employed was determined by a trial-and-error procedure. For such purpose, a simple 1D linear viscoelastic soil column was considered, featuring the same soil layers and properties of the full model (bedrock plus backfill; see Table 4 for details). This simple model was used to run quickly SH wave propagation analyses, following the same procedure to be used for the full model and varying the values of damping. The results of such analyses were compared with those obtained with SHAKE [Schnabel et al., 1972] for the same soil column and level of seismic excitation. The value of viscous damping to be adopted in FLAC was chosen as the one allowing the best match with the results of SHAKE (the following parameters were finally chosen: 0.01% Rayleigh damping centred at 1 Hz, plus the hysteretic damping option included in FLAC).

The optimal geometrical size of the model (Fig. 11b) was achieved after few successive iterations by imposing that, far from the wall, the response (in terms of displacement, velocity, and acceleration time-histories at a series of points) matched the free-field response obtained with SHAKE. In this way, it was ensured that the whole volume of soil actually interacting with the wall was included in the model.

5.3. Results

When studying the seismic response of gravity retaining walls composed of superimposed blocks, one of the major modelling issues concerns the possibility of simulating the relative sliding between blocks (which in the pseudo-static and Newmark approaches is disregarded). Thus, the first aspect that was investigated in the parametric study was the influence of friction between the concrete blocks compared to the limit case of a monolithic wall.

### Table 4

Reference material properties used for ITHA (resistance parameters unfactored)

<table>
<thead>
<tr>
<th>Material</th>
<th>Weight for unit volume</th>
<th>Angle of shearing resistance</th>
<th>Cohesion</th>
<th>Bulk modulus</th>
<th>Shear modulus</th>
</tr>
</thead>
<tbody>
<tr>
<td>Bedrock (&quot;Schlier&quot; marlstone)</td>
<td>20 kN/m³</td>
<td>0º</td>
<td>0.25 MPa</td>
<td>875 MPa</td>
<td>403 MPa</td>
</tr>
<tr>
<td>Backfill (sand)</td>
<td>17 kN/m³</td>
<td>40º (1)</td>
<td>0.00</td>
<td>66.67 MPa</td>
<td>40 MPa</td>
</tr>
<tr>
<td>Concrete</td>
<td>24 kN/m³</td>
<td>–</td>
<td>–</td>
<td>16667 MPa</td>
<td>12500 Mpa</td>
</tr>
</tbody>
</table>

(1) Subject to parametric study.
Figures 12 and 13 show the final displacement profile resulting from time history analyses respectively for blockwork (considering friction angle between blocks equal to $30^\circ$) and monolithic wall, for a PGA equal to 0.20 g, angle of shearing resistance of the backfill equal to $40^\circ$ and friction angle at the wall base equal to $30^\circ$. Comparing the response of blockwork and monolithic walls, one can conclude that for the blockwork wharf some sliding occurs at each interface between the blocks.

For the monolithic wall, the final displacement profiles are straight lines, as shown in Fig. 13, as slippage between blocks cannot occur. The profiles are sloping because the soil foundation undergoes permanent deformation during the seismic excitation, mainly beneath the seaward edge of the wall foundation, and therefore the wall undergoes permanent tilting. However, for the examined case, the foundation soil was stiff and thus the amount of residual tilting was small (less than 0.1%).

Figures 12 and 13 also show a high variability of the final displacement profile, both for the blockwork and the monolithic wall, depending on the specific seismic input. Since each accelerogram is scaled to the same PGA, this clearly indicates that PGA is not the only controlling parameter and that other characteristics of the ground motion play a

**FIGURE 12** Displacement profile for blockwork wall, PGA = 0.20 g, friction at wall base = $30^\circ$, friction at block/block interface = $30^\circ$, angle of shearing resistance of the backfill = $40^\circ$.

**FIGURE 13** Time-history analyses results for monolithic wall (glued interfaces), PGA = 0.20 g, friction at wall base = $30^\circ$, angle of shearing resistance of the backfill = $40^\circ$. 

---

According to the text, the displacement profiles for both the blockwork and monolithic walls are presented. The blockwork wall shows variability due to different seismic inputs, while the monolithic wall has a more consistent profile due to the lack of slippage between blocks. The foundation soil's stiffness affects the amount of residual tilting, which is relatively small for the examined case (less than 0.1%).

---

Some Issues in Seismic Analysis and Design of Blockwork Wharves
key role in determining the final displacement of the wall, e.g., the frequency content, the
duration, etc.

As already mentioned, the spectrum compatibility required by the EC8 for THA was
achieved considering the average response spectra of the selected set of accelerograms.
Since the probabilistic response spectrum (as the EC8 one used for this study) reflects the
contribution of different seismogenic zones, it also reproduces the effects of an earth-
quake at different distances and with different magnitudes and consequently covers a
wide range of frequency content. On the contrary, the real accelerograms are records of
the strong motion generated from a deterministic seismic event, with specific magnitude
and distance values, and with a specific frequency content.

The strong influence of the accelerograms used for the analysis on the seismic response of
the retaining wall could be related to a resonance-like phenomenon, for which the top
displacement of the retaining wall increases as the frequency content of the input is closer to
the main frequency of the system retaining wall-backfill. However, the applicability of the
concept of “resonance” to an inelastic phenomenon such as block sliding is not straightforward.

The high dispersion of the displacement profile is further confirmed by Fig. 14,
which shows the average final displacement profile plus and minus one standard devia-
tion. Graphs similar to Figs. 12 and 13 were obtained for each branch of the parametric
program shown in Fig. 9. The full set of results is reported in Pasquali [2008].

If the wall is designed to fail by sliding instead of overturning, the parameter which may
affect the ultimate or the serviceability limit state of the wall is the displacement of the top of
the wall, while the shape of the displacement profile is no matter of concern as long as the
wall’s top displaces more than the bottom (which is always the case in all the analysed
scenarios). Therefore the parameter chosen for the assessing the wharf seismic performance is
the permanent horizontal displacement at the top, seawards corner of the wall.

Figure 15 shows the comparison between blockwork and monolithic wall response for
different values of friction at the wall base and for two levels of PGA. Focusing on the
average results, one can make the following remarks. A similar trend for both levels of
PGA (0.20 and 0.25 g) is found when friction between the concrete blocks is larger than
friction at the soil-foundation interface. In fact, in this case, the final displacement at the top
of the blockwork wall is very similar to the one that would occur if the wall were
monolithic.
A closer look at the displaced shape of the wall would show that, in such conditions, the blockwork wall experiences both global sliding at the base (smaller than the one that would occur for the corresponding monolithic wall) and block sliding at block-to-block interfaces.

The total displacement of the top block is the sum of these two contributions and it nearly equals the displacement of the corresponding monolithic wall, which can only slide at the base. Conversely, if friction at the base of the wall increases and becomes equal to friction between the wall concrete blocks, then preferential sliding occurs between blocks and the difference in the response between blockwork and monolithic wall becomes significant. The increment of friction angle at the base reduces the permanent displacement of a monolithic wall, whereas it (almost) does not affect the final displacement of the blockwork wall.

**FIGURE 15** Influence of friction at the wall base – PGA = 0.20 and 0.25 g.

A closer look at the displaced shape of the wall would show that, in such conditions, the blockwork wall experiences both global sliding at the base (smaller than the one that would occur for the corresponding monolithic wall) and block sliding at block-to-block interfaces.

The total displacement of the top block is the sum of these two contributions and it nearly equals the displacement of the corresponding monolithic wall, which can only slide at the base. Conversely, if friction at the base of the wall increases and becomes equal to friction between the wall concrete blocks, then preferential sliding occurs between blocks and the difference in the response between blockwork and monolithic wall becomes significant. The increment of friction angle at the base reduces the permanent displacement of a monolithic wall, whereas it (almost) does not affect the final displacement of the blockwork wall.
The above observation is further confirmed by the results of Fig. 16, showing the influence of friction between concrete blocks. Comparing the first and the second column of the figure and paying attention to the different scales of the graphs, it can be observed that when friction between blocks drops below the friction at the wall base, the final displacement at the top increases dramatically. A closer look at the displaced shape of the wall shows that, under such conditions, preferential sliding occurs above the block placed at the bottom [Pasquali, 2008].

Both Figs. 15 and 16 show that the phenomenon being examined is highly nonlinear with respect to both PGA and friction. Therefore, unfortunately, it does not seem possible to extrapolate the solutions found for a certain scenario, i.e. an array (PGA; friction), by simply applying a scaling factor to final displacements.

FIGURE 16 Influence of friction between concrete blocks.
Figure 17 shows the influence of the backfill shear strength on wall displacements. For both levels of PGA (0.20 and 0.25 g), the backfill with lower values of shearing resistance appears to produce an increase in the permanent displacement of the wharf; however, the ratio between the displacements observed in the blockwork wall and in the monolithic wall does not change significantly. As already mentioned, it can be observed that when friction between concrete blocks is larger than friction at the wall base, the displacement at the top of a blockwork wall is similar to the one of a monolithic wall.

Figure 18 shows a comparison between the final permanent displacement of the wharf obtained using pseudo-static, Newmark, and time history analyses for the case of a
monolithic wall. Obviously, neither pseudo-static nor Newmark methods allows an estimate of the wall rotation.

Assuming that ITHA gives the best results, Fig. 18 shows that the simplified methods are not always conservative (i.e., in some cases they underestimate the final displacement); from this point of view, the Richards & Elms [1979] formula appears to give the

\[
\text{PGA} = 0.20g
\]

friction angle @ the base = 25°
angle of shear. resist. of backfill = 30°
average of 14 analyses (7 eqks, both directions)

\[
\text{PGA} = 0.25g
\]

friction angle @ the base = 25°
angle of shear. resist. of backfill = 30°
average of 14 analyses (7 eqks, both directions)

\[
\text{PGA} = 0.20g
\]

friction angle @ the base = 30°
angle of shear. resist. of backfill = 40°
average of 14 analyses (7 eqks, both directions)

\[
\text{PGA} = 0.25g
\]

friction angle @ the base = 30°
angle of shear. resist. of backfill = 40°
average of 14 analyses (7 eqks, both directions)

\[
\text{PGA} = 0.20g
\]

friction angle @ the base = 30°
angle of shear. resist. of backfill = 40°
average of 14 analyses (7 eqks, both directions)

\[
\text{PGA} = 0.25g
\]

friction angle @ the base = 30°
angle of shear. resist. of backfill = 40°
average of 14 analyses (7 eqks, both directions)

FIGURE 18 Final displacement of the top of the monolithic wharf using pseudo-static, Newmark, and time-history methods of analyses for the parametric program shown in Fig. 9. Standard deviation of the results is also shown.
best results. In particular, the Newmark sliding block method systematically underestimates the displacement and gives worse predictions even when compared with the pseudo-static method. This is somehow surprising and it may be due to the fact that this method cannot account for the hydrodynamic effect, which conversely can be included in the pseudo-static method.

Referring to the scenarios shown in Fig. 18, it is worth noting that only for case (e), the pseudo-static method gives a factor of safety greater than one. Therefore, only such case would be considered as an acceptable design according to current Eurocode 8 prescriptions. As a result, the errors in evaluating the displacement by the pseudo-static method, occurring for example in cases (a) and (b) would not affect real design, as they are referred to situations that would be considered unacceptable (e.g., Safety Factor <1). On the other hand, ITHA allow in all cases an estimate of the displacement, and therefore also case (a) and (b) could be taken as possible design options, as long as the associate displacements are considered to be acceptable.

5.3.1. Performance Assessment: Damage Criteria. The main advantage of performance-based design is the fact that owners and designers can establish the most suitable performance criteria on a case-by-case basis, according to the specific features of the wharf being examined (e.g., type of retained structures and equipment, characteristics of cranes, presence of rails, etc.). A logical procedure that could be used to carry out performance-based design of wharf structures could be the following.

1. Identify all possible seismically-induced failure/damage mechanisms of the wharf under consideration.
2. For each of the identified mechanisms, choose one or more descriptors of the wharf’s performance (i.e., scalar or vector parameters which can be calculated and measured, describing the amount of damage connected to the relevant mechanism; they are usually displacements and/or rotations).
3. Establish some correlation between damage levels and values of the damage parameters defined at point 2.

Some guidance concerning the implementation of this methodology is provided by PIANC [2001]. For gravity wharves, the following damage descriptors are suggested:

1. normalized residual horizontal displacement (residual horizontal displacement at the top of the wall divided by the height of the wall);
2. residual tilting (towards the sea);
3. differential settlement on apron; and
4. differential settlement between apron and non-apron areas;

For different damage levels, PIANC [2001] provides recommendations regarding numerical bounds of damage descriptors to be adopted in design. Concerning the first of the proposed parameters, it is observed that the absolute value of horizontal displacement, instead of the normalized one, may be more directly connected to many kinds of damage (e.g., deviation of rails). Normalization with respect to wall height might not be meaningful as far as the damage to supported equipment is concerned. Moreover, the mechanics of wall sliding does not suggest the wall height and its displacement to be correlated. Concerning the last three proposed parameters, it is observed that they are not independent from the previous ones. In fact, settlements of the apron occur as a consequence of the wall displacement. Therefore, sliding and rotation of the wall appear to be the true controlling parameters.
Finally, as already mentioned, gravity retaining walls should be “capacity designed” to slide before overturning or undergoing excessive deformation of the foundation soil. This would be sufficient to ensure limited residual tilting. Hence, horizontal displacement remains the essential parameter that should be controlled in design. Of course, when assessing the performance of existing wharves, such simplifying assumptions may not be appropriate, and all the possible damage mechanisms (through appropriate descriptors) should be checked.

6. Conclusions

This study has investigated advantages and drawbacks of all the available methods for seismic design of gravity wharves. It has been shown that the complex dynamic behavior of a blockwork gravity wharf may be suitably modeled by nonlinear time-history analyses, while currently adopted simplified methods may underestimate displacements.

In addition, the peculiar behaviour of blockwork walls has been highlighted and compared to the simpler response of monolithic walls. Blockwork wharves are comparatively simpler to build and therefore are often associated to simplified design procedures. However, it has been shown that designing a blockwork wall as if it were monolithic could be a drastic oversimplification, which may be acceptable only if friction between blocks is larger than friction at the wall base or, alternatively, if shear keys are provided between blocks. If such conditions are not met, the final displacement of the blockwork wall top will be larger than the one of the corresponding monolithic wall.

The variability of response to different input accelerograms —scaled to the same PGA— has been highlighted. This has demonstrated that PGA is not the only controlling ground motion parameter for the seismic response of gravity wharves. Therefore the selection of seismic records is perhaps the most delicate step in performing meaningful time history analyses.

To validate these conclusions, experimental verification of the results obtained using numerical simulations would be highly desirable. This would also allow to gain an insight into several technical issues arising in numerical modeling of geotechnical systems subject to earthquake loading.

In the research presented herein, the problem of predicting permanent displacements has been given major emphasis, as this is a general issue. However, for the purpose of performance-based design, reliable estimate of displacements is only half of the work: the calculated values must be checked against appropriate bounds (damage criteria), to be established on a case-by-case basis.

Even though limited to a specific case study, this work is aimed to provide not only a methodological study, but also a quantitative assessment on how different design methods and parameters influence the final design of a wharf structure.

For this reason, an extensive parametric study has been performed. At the same time, this research is by no means intended to be a comprehensive assessment of the seismic performance of blockwork wharf structures, as several modelling issues have been purposely neglected (e.g., cranes on the apron, anchors that sometimes restrain the top blocks, etc.).

Whereas the development of more accurate simplified methods of analyses, capable of overcoming some of the drawbacks of the procedures currently in use, would certainly be most helpful to designers, a more refined calculation using time history analyses is highly recommended when designing earthquake-resistant wharves. In fact, most likely, the strategic importance and the economic value of such structures makes the increased design cost nowadays affordable.
Acknowledgments

The authors would like to thank the Port Authority of Ancona for kindly providing the original design documents of the wharf structure which were used as real case study. The census of Italian port structures has been carried out under the financial auspices of the Department of Civil Protection of Italian Government (Progetto Esecutivo 2005–2008, punto f dell’articolo 3, progetto n. 5) and of the Italian Ministry for Research and Higher Education (MiUR – Ministero dell’Università e della Ricerca) through the FIRB Project No. RBIN047WCL (Assessment and Reduction of Seismic Risk to Large Infrastructural Systems) and. Such support is gratefully acknowledged by the authors.

References


Hynes-Griffin, M. E. and Franklin, A. G. [1984] “Rationalising the seismic coefficient method,” Miscellaneous paper GL-84-13, U.S. Army Corps of Engineers Waterways Experiment Station, Vicksburg, MS.


Itasca Consulting Group Inc. [2005] FLAC (Fast Lagrangian Analysis of Continua), ICG, Minneapolis, MN.


Whitman, R. V. and Liao, S. [1985] “Seismic design of retaining walls,” Miscellaneous paper GL-85-1, U.S. Army Engineer Waterways Experiment Station, Vicksburg, MS.