Seismic Analysis and Design of Blockwork-Wharf Structures

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ABSTRACT:

During the past decades, a number of ports worldwide have suffered extensive earthquake-related damages. As seaports play a key role in the economy of many countries, target performances of wharf structures should be clearly stated and reliably pursued by designers. The present work focuses on seismic vulnerability of wharves in the Italian scenario, where most of the seaports are constituted by old infrastructures. According to a recently carried out census, most of the existing wharves in Italy are gravity-type, made of superimposed, dry-connected blocks, particularly in older facilities. Although such wharf-typology is widely spread worldwide, it has not attracted much research interest. Blockwork wharves are commonly designed using simplified, pseudo-static, force-based approaches. When the need of calculating displacements is recognized, the Newmark's method is considered as the standard procedure. In the present study, the validity of such approaches has been investigated by comparison with extensive dynamic time-history analyses. Permanent displacements of the wall blocks have been calculated and compared with available performance criteria. Application to a real structure in the seaport of Ancona (Italy) has been carried out, as a methodological example. Furthermore, a parametric study has been conducted with the aim of investigating the role played by key design parameters.

KEYWORDS: Blockwork wharves, gravity retaining walls, time-history analysis, Newmark's method, pseudo-static method, performance-based design.

1. INTRODUCTION

Seaports are crucial elements in the export and import of goods and on the flow of travellers in the tourism industry of many industrialised nations, including Italy. The presence of a port has always represented a remarkable, at times decisive, factor of development, able to produce a series of direct and indirect effects on the economy and on the social and environmental context of the interested area. The importance of the sea and the maritime transports, and therefore of the ports, is mostly evident for a country such as Italy, with its long tradition of navigation and hundreds of ports of any size existing along its coasts, totalling about 8000 km length.

Many of the Italian seaports are located in zones characterized by moderate to high seismicity. Figure 1a shows a seismic classification of 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 facilities were built several decades ago, therefore without specific seismic design provisions. The combination of hazard, vulnerability and exposure of the 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 repair costs of the structures, but especially to interruption of port serviceability in the immediate aftermath of an earthquake. Experience gained from recent seismic events (e.g. 1989 Loma Prieta in USA, 1995 Hyogoken-Nanbu and 2003 Tokachi-Oki in Japan) 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 marginal 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 the implementation of a detailed census of the Italian major seaports which was carried out using purposely devised questionnaires aimed to identify the existing wharf typologies (Gentile & Lai, 2007).

Wharf structures are traditionally distinguished between *open-type* and *closed-type*. Open wharves are platforms supported by a series of vertical or battered piles, whereas closed wharves are composed of a deck supported by soil retained by a wall. Within the aforementioned census, structural typology of wharves was investigated. A sample of five (out of 25) major ports was studied for a total of 46 wharves. It turned out that the majority of Italian wharves are made of superimposed concrete blocks (Figure 1b). Hence it appears that such typology is worth being carefully studied, with the aim of improving vulnerability assessment.

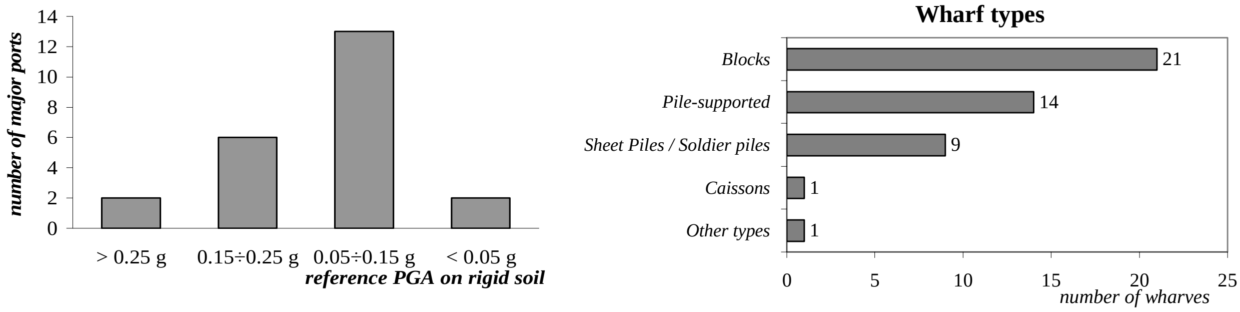


Figure 1 a) Seismic hazard at the 25 major Italian seaports (reference PGA for 475 years return period); b) Wharf typology occurrence at the Italian seaports studied within the EUCENTRE research project.

Block wharves are commonly designed using simplified, pseudo-static, force-based approaches. This work investigated the validity of such approach. Extensive dynamic time-history analyses have been carried out, using advanced numerical programs. The results of pseudo-static approach implemented according to the prescriptions of Eurocode 8 Part 5 (EN 1998-5:2005), Newmark approach and non-linear time-history analyses have been compared. A real structure in the Port of Ancona, in Central Italy, has been assessed as a methodological example. Furthermore, a parametric study has been conducted with the aim of investigating the role played by key design parameters.

2. SEISMIC DESIGN OF GRAVITY WHARVES

Despite the substantial improvements achieved by earthquake engineering, seismic performance of wharf structures is still not satisfactory. In recent years, substantial earthquake-related damage has been recorded, even in most developed countries. Among the gravity-type retaining structures, those composed of superimposed blocks are possibly the most ancient. Blocks are dry-connected and rely on friction at the joints to achieve a sufficiently monolithic response. Nowadays, a reinforced concrete beam is always cast on top of the wall, with the function of distributing the mooring and berthing internal forces over a sufficient length. Different design procedures are available for designing gravity retaining walls, each of them has advantages and drawbacks, as will be briefly outlined below.

3.1. Force-based method

Most commonly, seismic design of gravity wharves (both monolithic and blockwork), as well as of gravity retaining walls in general, is performed according to the pseudo-static approach (also known as the *Mononobe-Okabe* method), which is required or suggested by the large majority of seismic codes worldwide. According to such procedure, seismic actions are represented by static forces, which are then included in the global equilibrium free body diagram. In this way, seismic and non-seismic design is carried out following the same procedure. Following this method, seismic actions are usually represented by three terms: the inertia of the wall itself, the increase of soil thrust due to seismic shaking, and the hydrodynamic effect which arises when a water pool exists in front of the wall. The aforementioned representation of seismic actions is clearly conventional. Its use is justified by its simplicity and by the satisfactory performance of walls designed in such a way when subject to real earthquakes. This satisfactory performance has been mostly related to earth-retaining structures.

For wharves, unfortunately, the picture is quite different. First of all, poor performance has been recorded after recent earthquakes (e.g. the Hyogoken-Nanbu, Japan, in 1995) and in addition, for wharves, more than for retaining walls, it is important not only to avoid collapse, but also to control displacements in view of strict serviceability requirements. This means moving towards performance-based design, aiming at substituting a conventional measure of safety (the traditional safety factor) with the assessment of the real response of the structure hit by an earthquake (i.e. permanent displacements).

3.2. Displacement-based methods

A fundamental difference exists between seismic and non-seismic actions. The former exists only for few seconds, this means that, if equilibrium is violated because of seismic actions, such equilibrium loss will last only for a very short amount of time. As a consequence, the permanent movement due to equilibrium loss may be limited and, at the end of an earthquake, we may find our retaining wall displaced of a few centimetres, but still serviceable. This example helps to realize that while equilibrium must be guaranteed with respect to static soil thrust, this is not necessarily required as far as seismic action is concerned. Indeed, this is a good news, because if realistic levels of ground shaking are to be considered, then it may turn out very difficult to satisfy equilibrium at each instant of time during an earthquake in high seismicity zones. Moreover, seismic performance of these structures during past earthquakes indicates that soil, foundation and structural deformation together with the state of stress are the key design parameters and, unlike the conventional limit equilibrium-based methods, some residual deformation may be acceptable in design (PIANC, 2001; EN 1998-5:2005; Iai et al., 2008). An estimate of earthquake-induced displacement may be obtained by using simplified dynamic analyses (i.e. Newmark approach) or alternatively advanced non-linear time histories analyses.

3.2.1 Newmark's method

Newmark (1965) first recognized that the seismic stability of a slope could be assessed by taking into account the dynamic nature of seismic actions, even though in a simplified way. His method has then been extended from slopes to retaining walls, without changing the core concept (Kramer, 1996). Newmark suggested calculating the threshold ("yield") acceleration that would induce incipient movement (i.e. unit factor of safety) of the rigid body under study, by means of the pseudo-static method. Then, a design accelerogram is needed: acceleration values exceeding the threshold acceleration are integrated twice to obtain the final displacement of the rigid body. Obviously, when calculating the yield acceleration, the most critical failure mechanism needs to be considered. For gravity retaining walls, the governing failure mechanism is commonly (and desirably) that of base *sliding*. However, that this is indeed the case, should be checked on a case by case basis.

3.2.2 Inelastic time-history analyses

By the term "Time History Analysis" (THA) engineers (and physicists) indicate the procedure of solving the equations of motion (Newton's second law) of a mechanical system subjected to a specific set of forces (boundary conditions). Then, the configuration of the system at each time step is obtained (*Lagrangian* approach). Unless very simple systems are considered, the equations of motion have to be solved numerically. Finite difference or finite element methods are commonly used to build a numerical model of the physical system at hand. Nowadays, commercial software implementing advanced material models and robust numerical algorithms are available. As a result, non-linear THA of large mechanical models can be efficiently carried out.

3.3. Force-based vs. displacement-based methods

The pseudo-static method has one major advantage: it is quick and simple for the designer. In fact, within such approach, the major problem is left in the hands of the code writer. How to find a static force *equivalent* (in terms of its effects) to the forces produced by seismic shaking. Such static force is usually expressed in terms of a *seismic coefficient* multiplying a gravity force (EN 1998-5:2005; Pasquali, 2008).

A possible choice for the seismic coefficient could be that of adopting the peak ground acceleration (PGA) of the design earthquake. However, in medium to high seismicity zones, experience has shown that such approach would be too much conservative. In fact, peak accelerations occur for a very short time, so that even if temporary equilibrium loss occurs, the resulting movement of the wall is usually very modest.

For this reason most seismic codes introduce a *reduction factor*, to be applied to the peak ground acceleration in order to obtain the seismic coefficient. Such reduction factor depends on the amount of acceptable wall's displacement and it is somehow similar conceptually, to the reduction factor used in structural design to account for ductility. So the core problem in the force-based approach is to reliably assess the reduction factors, which can only be done empirically. On the contrary, if a displacement-based method is used, there is no need for introducing empirical parameters and the final displacement of the wall is directly calculated, which appears more rational.

3. REAL CASE STUDY: A NEW WHARF AT THE ANCONA SEAPORT

In order to investigate the capabilities of available methods of analyses, 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 simplified cross section of the wharf is shown in Figure 2.

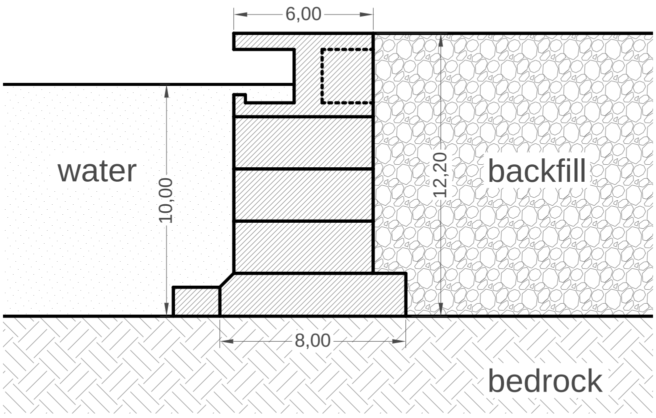


Figure 2 Cross section of the blockwork wharf analyzed in this study.

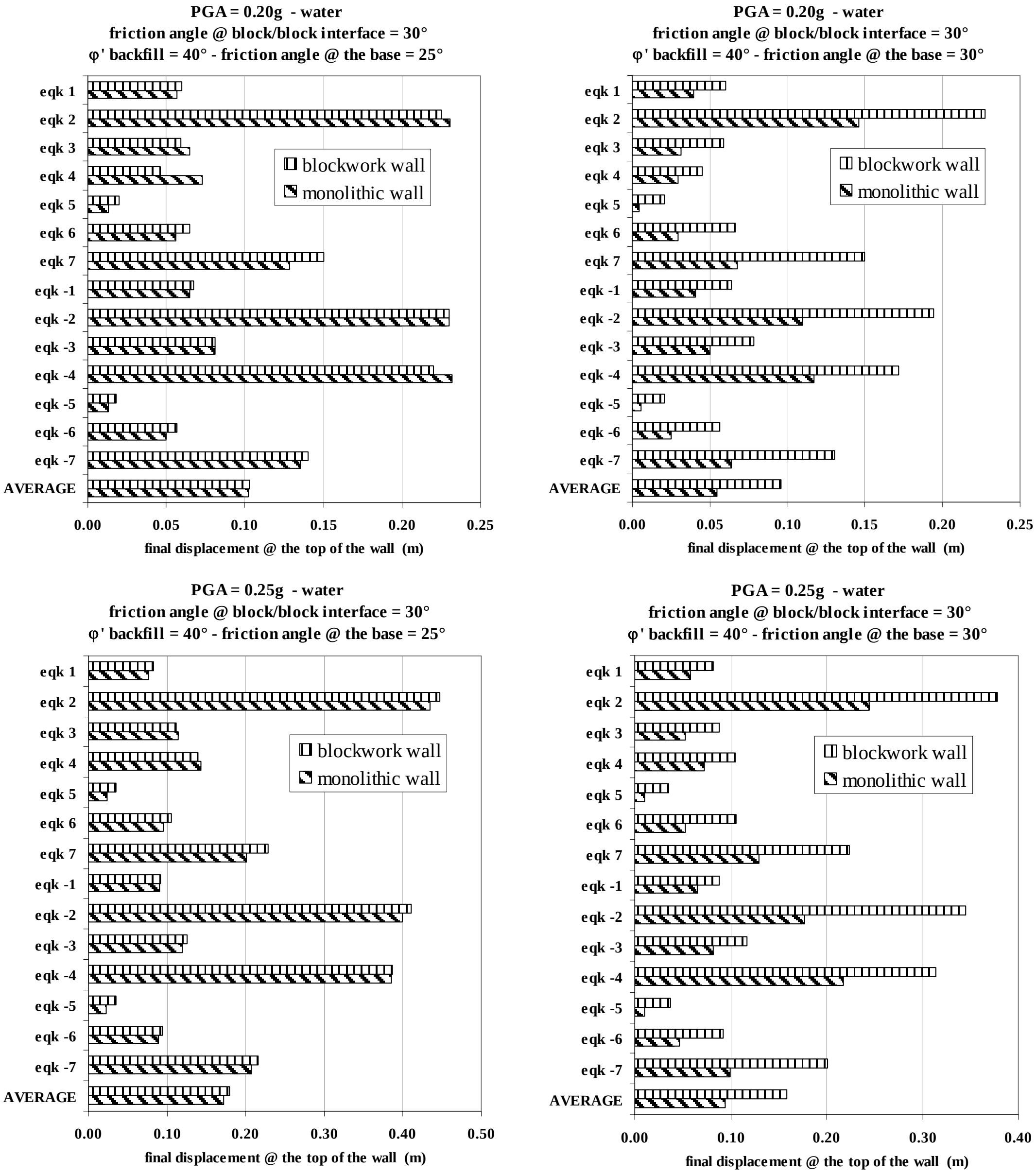
For the Port of Ancona a site-specific seismic hazard study was carried out by EUCENTRE (2006). Within such work, design seismic input was determined as a set of seven real, spectrum-compatible earthquake records. Such accelerograms have been adopted in this work. They have been scaled to two different levels of PGA (i.e. 0.20*g* and 0.25*g*) and used, with both polarities, for both THA and Newmark analyses. Pseudo-static analyses have been carried out too, according to the prescriptions of EN 1998-5:2005 (Eurocode 8 – Part 5).
In order to allow results from all methods to be compared, only horizontal seismic input was considered in THA and in pseudo-static analyses. Dynamic time histories analyses have been performed using the finite difference code FLAC (Itasca, 2005). The soil has been modelled 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 has been assumed to be equal to zero). The blockwork wall has been modelled using linear elastic, isotropic plane strain elements. Hydrodynamic effect was taken into account by introducing an additional mass according to the Westergaard's theory (Westergaard, 1933; EN 1998-5:2005). The soil strength parameters have been factorized according to the prescriptions of Eurocode 8. A thorough discussion of some modelling issues including the issue of soil damping, and other details are described in the work by Pasquali (2008). Pseudo-static analyses have been implemented through a spreadsheet. Newmark analyses have been carried out using the Jibson and Jibson (2003) software and checked with a home-made program. The yield acceleration has been found numerically, by imposing unit factor of safety in corresponding pseudo-static analysis. Two major issues have been investigated. The first goal was to investigate the peculiar behaviour of a blockwork wall and to compare it with the behavior of a corresponding monolithic wall. The second goal was to compare the results of THA, Newmark-type analyses and pseudo-static analyses, referring only to the case of monolithic wall (because standard Newmark-type methods cannot account for block walls). Parametric studies have also been carried out, in which the influence of friction angle at the block-to-block interface, friction angle at the wall base, angle of

friction resistance of the backfill and PGA have been investigated. Conversely, in all cases the geometry has been kept fixed. Again, details about the assumed material properties can be found in the already cited work by Pasquali (2008).

3.1. THAs results: monolithic Vs. blockwork wall

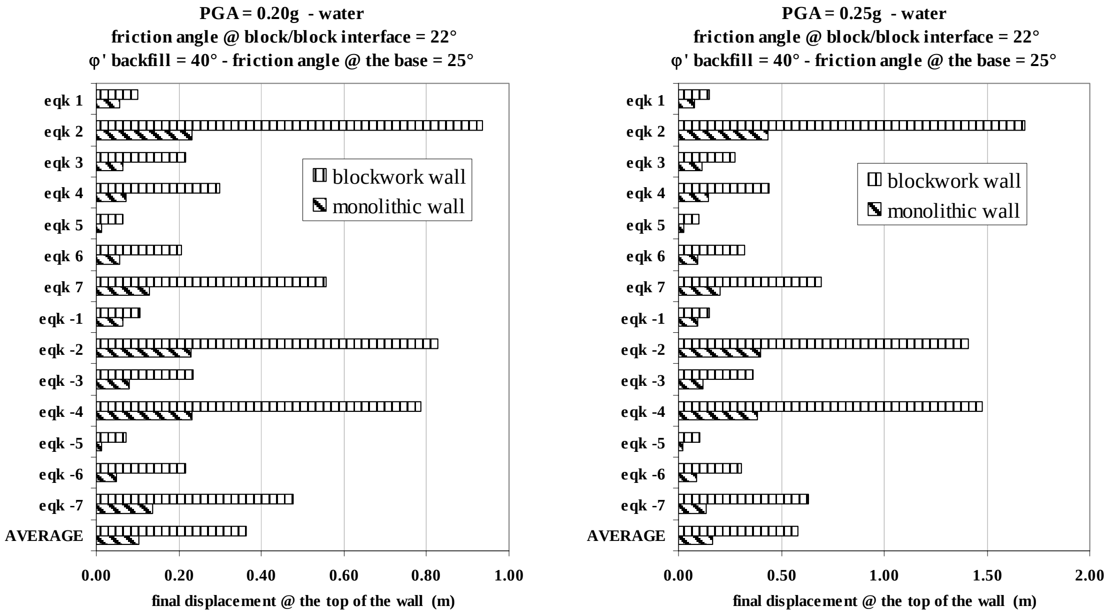
The parameter chosen for the assessment of wharf performance is the permanent horizontal displacement at the top of the wall. Table 3.1 shows the influence of friction at the wall base for different levels of PGA. A very similar trend for both levels of PGA (0.20*g* and 0.25*g*) is found when friction 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. Conversely, if friction at the base of the wall increases and becomes equal to friction between the wall's concrete blocks, then preferential sliding occurs between blocks and the difference in the response between blockwork and monolithic wall becomes very significant. The increment of friction angle at the base reduces the permanent displacement of a monolithic wall, while it almost does not affect the final displacement of a blockwork wall.

Table 3.1 Influence of friction at the wall base for PGA=0.20g and 0.25g.



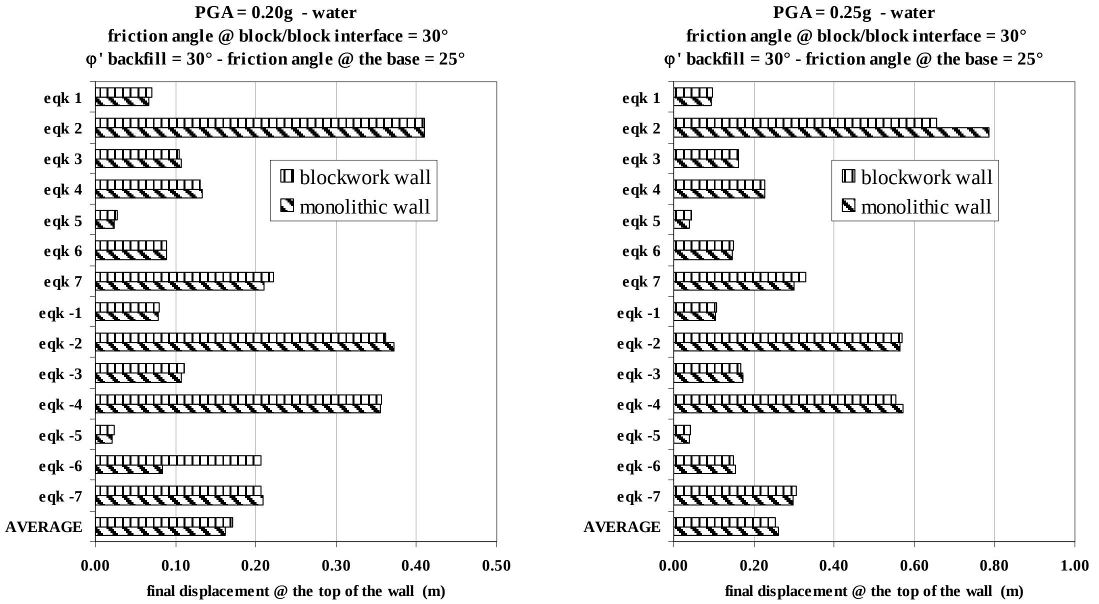
The above observation is further confirmed by the results of Table 3.2, showing the influence of friction between concrete blocks. It can be observed that when friction between blocks drops below the friction at the wall's base, the final displacement at the top increases dramatically. A closer look at the displaced shape of the wall would show that, in such conditions, preferential sliding occurs above the bottom block (Pasquali, 2008).

Table 3.2 Influence of friction between concrete blocks.



Finally, Table 3.3 shows the influence of the angle of friction resistance of the backfill. This table should be compared with the corresponding cases shown at the left-hand side of Table 3.1. For both levels of PGA (0.20*g* and 0.25*g*), the backfill with lower values of shear resistance appears to produce an increase of the permanent displacement, but the ratio between the blockwork wall's and the monolithic wall's displacements does not change significantly. As already mentioned, if friction between concrete blocks is larger than friction at the wall's base, then the displacement at the top of a blockwork wall is very similar to the one of a monolithic wall.

Table 3.3 Influence of backfill shear strength.



In general, all the studied cases show that the permanent top displacement of the wall is strongly influenced by the input accelerogram, although all the accelerograms are scaled to the same PGA. It seems therefore that PGA is possibly not the main parameter governing seismic response of gravity retaining walls (see also Newmark, 1965 and Kramer, 1996). Others parameters such as peak ground velocity, duration and frequency content should be accounted for in order to improve the accuracy of simplified analyses to predict the wall's response.

3.2. Monolithic wall: THA Vs. Newmark Vs. Pseudo-static (EC8) results

The results of THA for the case of monolithic wall have been compared with simplified methods such as Newmark's and pseudo-static approaches. In order to obtain comparable results, a procedure has been developed to estimate the wall's displacement from current pseudo-static approach, as described below. When applying the pseudo-static method, the maximum allowable wall's displacement must be chosen *a priori*, in order to select the appropriate PGA reduction factor (see EN 1998-5:2005). Then a standard equilibrium check is carried out. If a unit factor of safety is obtained it means that the wall will undergo the maximum acceptable displacement. If the factor of safety is greater than unit, it seems reasonable to assume that the permanent displacement will be proportionally smaller. This is the hypothesis assumed herein. So, in general:

$$d_{actual} = d_{allowable} / FS \qquad (3.1)$$

where FS is the safety factor, d_{actual} is the final displacement and $d_{allowable}$ is the allowable displacement according to EN 1998-5:2005. For the wharf under study, the acceptable displacement has been set equal to 60 mm, consistently with original design documents where a reduction factor equal to 2 was used.
Table 3.4 and 3.5 show the results of the comparison for both PGA values (0.20*g* and 0.25*g*). In general, both the pseudo-static and the Newmark's methods appear to underestimate the final displacement of the wall. However, quite surprisingly, the Newmark's method has shown to yield the worse performance. This may be due to the fact that the Newmark's method in its current formulation cannot a) take into account the hydrodynamic effect, which on the contrary, is accounted for in both the pseudo-static method and in THA, and b) the wall is subjected to a multiple-support excitation (from the foundation base and from the backfill).

Table 3.4 Displacement at the top of the wall according to different methods – PGA = 0.20*g*

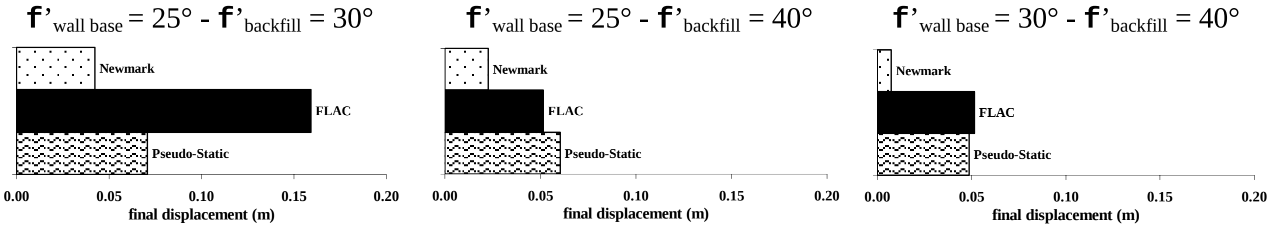
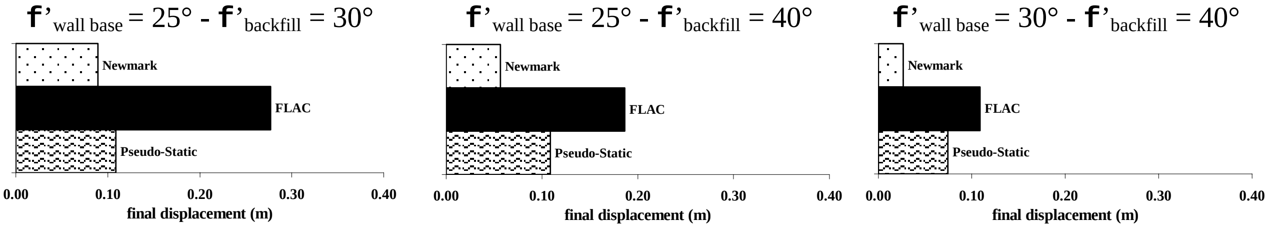

$\varphi'_{\text{wall base}} = 25° - \varphi'_{\text{backfill}} = 30°$ | $\varphi'_{\text{wall base}} = 25° - \varphi'_{\text{backfill}} = 40°$ | $\varphi'_{\text{wall base}} = 30° - \varphi'_{\text{backfill}} = 40°$

Table 3.5 Displacement at the top of the wall according to different methods – PGA = 0.25*g*


$\varphi'_{\text{wall base}} = 25° - \varphi'_{\text{backfill}} = 30°$ | $\varphi'_{\text{wall base}} = 25° - \varphi'_{\text{backfill}} = 40°$ | $\varphi'_{\text{wall base}} = 30° - \varphi'_{\text{backfill}} = 40°$

4. CONCLUSIONS

In this paper the results of a parametric study focused on the seismic response of blockwork wharf structures have been illustrated. The study has shown that the complex dynamic behaviour of a blockwork gravity wharf may be suitably modelled by non-linear time-history analyses.

Moreover, the comparison shows that simplified methods, such as the pseudo-static and Newmark's methods, despite being widely used worldwide (also through seismic code prescriptions), offered for the case study a poor prediction of the wall's displacement. Experimental verification through centrifuge and/or shaking table testing and further numerical work are needed. Meanwhile, a final design check by means of THA is highly recommended for important structures.

In addition, the variability of results for different accelerograms scaled to the same PGA has been highlighted. Therefore, the choice of input accelerograms plays a key role in the results of the analyses and of their reliability even when state-of-the art THA are performed.

Finally, it was shown that the response of a blockwork wall may be either very similar or very different with respect to the one of a corresponding monolithic wall, depending on the values of friction between concrete blocks and between soil and foundation. Preliminary rules of thumb are suggested to help in judging if simplified, monolithic models may give acceptable results, but the concept of equivalent monolithic wall behaviour should be used and verified carefully.

ACKNOWLEDGEMENTS

The authors would like to acknowledge the Port Authority of Ancona for kindly providing the original design documents of the case study for the wharf structure. The census of Italian port structures has been carried out under the financial auspices of the Department of Civil Protection of the Italian Government (Progetto Esecutivo 2005-2008, art. 3-f, prog. n. 5) and by 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) which is greatly acknowledged.

REFERENCES

DM 14/01/2008 (2008) Norme tecniche per le costruzioni, Italian Government, Ministry of infrastructures, S.O. n. 30, G.U. Feb. 4th, 2008 n. 29 (in Italian).

EN 1998-5:2005 (2005). Eurocode 8 - Design of Structures for Earthquake Resistance - Part 5: Foundations, Retaining structures and Geotechnical aspects. CEN, Bruxelles.

EUCENTRE (2006). Definizione dell'input sismico per il Porto di Ancona, Research Contract Report N. EU173/2006U (in Italian).

Gentile, R. and Lai, C.G. (2007). Indagine sulla vulnerabilità sismica delle banchine portuali italiane. *Proceedings,* ANIDIS 2007, Pisa, June 10-14, 2007 (in Italian).

Iai, S., Tobita, T. M. and Tamari, Y. (2008). Seismic Performance and Design of Port Structures, Proc. Geotechnical Earthquake Engineering and Soil Dynamics IV Conference, Sacramento.

Itasca Consulting Group Inc. (2005). FLAC (Fast Lagrangian Analysis of Continua), ICG, Minneapolis, Minnesota, USA.

Jibson, R.W. and Jibson, M.W. (2003). "Java programs for using Newmark's method and simplified decoupled analysis to model slope performance during earthquakes", Open-File Report 03-005, U.S. Geological Survey.

Kramer, S.L. (1996). Geotechnical Earthquake Engineering (Prentice-Hall, New Jersey).

Newmark, N. (1965). Effects of earthquakes on dams and embankments. *Geotechnique*, **15:2**, 139-160.

Pasquali, R. (2008). Some issues in seismic analysis and design of wharves. Msc Dis., ROSE School, (Italy).

PIANC (2001). Seismic Design Guidelines for Port Structures. Permanent International Navigation Association, A.A. Balkema Publishers, Rotterdam, The Netherlands.

Westergaard, H.M. (1933). Water pressure on dams during earthquakes. *Transactions of ASCE*, **98**, 418-472.