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SEISMIC RESPONSE OF GRAVITY WHARVES USING MODIFIED NEWMARK MODELLING WITH A DOUBLE-SUPPORT EXCITATION

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ABSTRACT

Seismic design of gravity wharves is most commonly performed by using the pseudo-static approach, which has all the limitations of force-based design methods applied to earthquake engineering. Its "*safety factor*" is purely conventional while performance-based design focuses on the expected wall displacement, thus providing a physical measure of the expected damage (Pasquali et al., 2010). Whereas in recent years the capabilities of advanced numerical methods have improved considerably, dynamic time-history analyses of gravity wharves remain a difficult problem to deal with and it is far from being included in the standard toolbox of practicing engineers. In the framework of performance-based design, the need is felt for a simplified, yet accurate, method of analysis enabling designers to overcome the deficiencies of traditional methods without embarking in complex numerical analyses. Available methods of this kind are based on the sliding block equivalence proposed by Newmark (1965). Such approach, known in the literature as the "*Newmark method*", has been originally developed for the seismic analysis of dams and embankments, however it has been used also for retaining walls. Yet, several differences exist between a soil block sliding along a homogeneous slope and a wharf structure which, differently to the former, is subjected to both a lateral seismic excitation induced by the backfill soil and a base excitation induced by the foundation soil. In this paper, a novel procedure is proposed to improve the applicability of the Newmark method to gravity wharf structures.

Keywords: Gravity wharves; permanent displacement; performance-based design; retaining walls.

INTRODUCTION

Gravity wharves are a typology of seaport structures that is widely used worldwide, especially in countries of ancient civilization (e.g. Mediterranean nations). The seismic vulnerability of gravity wharves is large as demonstrated by recent earthquakes. It is generally exhibited more as a serviceability limit state rather than a collapse problem. The seismic design of these structures is usually carried out using either the pseudo-static or the simplified dynamic (e.g. Newmark type approach) methods. However, both techniques have important drawbacks and limitations if applied to gravity wharves. A key parameter in performance-based design of geotechnical structures is the permanent displacement since it can be directly correlated with damage. For the standard Newmark method, which is based on the analogy with a rigid block sliding on a plane, several differences exist between the simplified model and an earth-retaining wall which, differently from the former, is also subjected to a lateral excitation induced by the backfill. This paper illustrates a novel procedure for assessing the permanent displacement of gravity wharves induced by earthquake loading. The technique is aimed at overcoming the drawbacks of the

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standard Newmark method while retaining most of its simplicity. In this method the wall is represented as a single degree of freedom system with horizontal sliding being the controlling failure mechanism. The wharf is loaded through a double support seismic excitation: from the base foundation according to the standard Newmark approach and laterally from the backfill. The proposed model accounts for both static and hydrodynamic loading of the water. The method, differently from the conventional Newmark model, does not require the use of the pseudo-static approach for the definition of the critical acceleration.

GRAVITY WHARF STRUCTURES

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 travelers in the tourism industry of many Countries. A wharf out of service in a major port is likely to cause an economic loss exceeding by far the cost of repair. 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. 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 struck 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) (Pasquali et al., 2010). Gravity wharf structures belong to closed-type wharves and in countries of ancient civilization they have been used since many centuries. Among the gravity-type retaining structures, those composed of superimposed rigid blocks are possibly the most ancient. Nowadays, gravity wharves are made either of superimposed concrete blocks, or of monolithic reinforced concrete caissons.
Experience gained from recent seismic events (e.g. 1989 Loma Prieta in USA, 1995 Hyogoken-Nanbu, and 2003 Tokachi-Oki in Japan, 2003 Lefkas in Greece earthquakes) has dramatically demonstrated the potential economic loss due to earthquake damage to seaports. In past earthquakes, concrete caisson quay walls subjected to strong ground shaking have repeatedly suffered substantial outward displacement and rotation, but have only rarely completely overturned (Dakoulas & Gazetas, 2008). 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. Ports and reclaimed land in general are particularly prone to liquefaction, due to the presence of widespread, saturated loose sand and silt soil deposits. If liquefaction occurs, the performance of wharves is expected to be very poor. However, 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, this volume of material will act like a gravity dam to restrain the lateral spreading of the liquefied soil which will then only settle (Pasquali et al., 2010). In fact, in several cases (e.g. 1995 Hyogoken-Nambu earthquake in Japan and 2003 Lefkas earthquake in Greece) observations in the field (Towhata et al., 1996; Iai et al., 1998; Gazetas et al., 2005) failed to reveal any signs of liquefaction on the backfill surface near the wall, within a distance of about twice the height of the wall (Dakoulas & Gazetas, 2008).

Seismic design of gravity wharves

Seismic design of gravity wharves is currently performed by using the same methods generally employed for gravity earth-retaining structures, even if the presence of water undoubtedly complicates the analysis. One of the most widely employed method for the seismic design of earth-retaining walls is the pseudo-static technique. It is based on an 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 (pseudo) static forces that are added to the other non-seismic actions. The safety factor is directly calculated as the minimum value among the ratios of stabilizing versus destabilizing actions. Different methods have been established in order to calculate the actions and the resistant forces, but in the end a

static equilibrium check is the core of the procedure first proposed by Mononobe and Matsuo (1929) (Pasquali et al, 2010; Corigliano et al., 2010).
The weak point is, evidently, the definition of a static force capable of reproducing the effects of a real acceleration time history. As a matter of facts, this correspondence is entirely empirical and also difficult to corroborate, short of real-scale experiments (Pasquali, 2008). A delicate point of the procedure lies in the choice of the "*seismic coefficient*". Such parameter is defined as the ratio between the design acceleration (horizontal or vertical component) and the acceleration of gravity. In principle, as the pseudo-static method is based on the free-body diagram of a rigid body, it seems reasonable to assume the wall subjected to the design Peak Ground Acceleration (PGA). However, experience has shown that, in high seismicity regions, such approach would lead to an overconservative design, predicting the collapse of most existing gravity walls, which actually behaved well in strong earthquakes. Starting from this empirical observation, engineers have realized that for most gravity earth-retaining structures, and especially for the squatter walls, *sliding* is the governing failure mechanism (Pasquali et al., 2010). Thus, exceedance of the sliding resistance of the wall will result in a permanent displacement of the wall, which may turn to be acceptable or even unnoticeable after the earthquake. For such reason, in most seismic codes, the seismic coefficient is calculated by applying to the design PGA a so called "*reduction factor*", whose specific value depends on the amount of displacement tolerable by the structure. The direct consequence of this reasoning is that checking equilibrium under seismic loading is not really necessary because the key point lies in calculating the permanent displacement of the wall and ascertaining if this is acceptable (Pasquali et al., 2010). Sliding at the wall base can be a stable and safe ductile behaviour because, in most cases, the sliding resistance of the wall is not affected by previous movement, and after an earthquake the static factor of safety regain the same value assumed prior to the seismic event, even though the wall did slid meanwhile. Thus, temporary loss of equilibrium of an earth-retaining wall that fails by sliding during an earthquake may be acceptable as long as it leads to a tolerable amount of displacement.

The "*safety factor*" computed using the pseudo-static approach is purely conventional and it does not allow the assessment of the permanent displacement of the structure. However in the framework of performance based-design the focus is on the expected wall displacement which provides a measure of the expected damage. A gravity earth-retaining wall can fail through three different mechanisms: by excessive sliding, by overturning, and by failure of the foundation soil. The first of these three mechanisms is inherently ductile. Therefore, it is desirable that, through an appropriate *capacity design*, sliding be the governing failure mechanism of the wall. If this is ensured, then the concept of "*failure*" becomes strictly connected to the concept of "*allowable displacement*". In fact, for a massive and rigid structure such as a gravity earth-retaining wall or a wharf, the permanent displacement at the end of the earthquake is the only measurable form of damage (Corigliano et al., 2010). A simplified method for analyzing the seismic stability of embankments and slopes in terms of displacements was outlined by Newmark in his remarkable 1965 Rankine Lecture. The method is based on the analogy with a rigid block sliding on a plane (Newmark, 1965), and despite of being initially developed for evaluating the seismic response of dams and embankments, it has been extended without conceptual changes also to the seismic analysis of earth-retaining walls (Kramer, 1996). The method requires the definition of a threshold ("*yield*") acceleration that would induce incipient movement (i.e. factor of safety of one) of the rigid block under study. Integrating twice the part of acceleration time history exceeding the yield acceleration, allows one to calculate the permanent displacement undergone by the block.

The Newmark method is simple and robust and it has gained increasing popularity in the geotechnical community. It requires the use of a computer but it has been implemented in freeware software (e.g. Jibson and Jibson, 2003). Yet, it is characterized by important drawbacks and limitations when applied to earth-retaining structures. Here below are some of the most relevant (Pasquali et al., 2010; Corigliano et al., 2011):

- seismic waves propagate from the bedrock to the foundation soil and to the backfill. The backfill is usually loose and of poorer mechanical properties if compared with those of the foundation soil. The ground motion is thus likely to be amplified. Even though the retaining structure is subjected to two sources of excitation – the base-foundation and the backfill – standard application of the Newmark method considers only the seismic input coming from the base-foundation. The latter is less severe and in general out-of-phase with respect to that of the backfill. A soft backfill may undergo significant amplification and induce "*whiplash effects*" on the wall. While these phenomena can be explicitly taken into account in advanced numerical analyses, they are completely disregarded by the Newmark method, which inherently assumes that the earth-retaining wall is subjected only to a base excitation;
- the Newmark method requires preliminary definition of the yield acceleration. The user is free to choose the most suitable method for calculating such parameter, but in practice the threshold acceleration is usually calculated according to the pseudo-static method, by looking at the value of the seismic coefficient that makes the factor of safety to sliding equal to unity. Using the pseudo-static approach for the definition of the yield acceleration implies that the inconsistencies and uncertainties of the force-based approach are somehow carried over to the displacement-based method;
- the effect of vertical shaking of the ground and of the structure is usually disregarded in the standard application of the Newmark method. It can in principle be taken into account by assuming that the vertical accelerogram is a scaled copy of the horizontal record. Otherwise, the threshold acceleration should be re-computed at each time step, which is still possible in theory, but very rarely implemented;
- if in front of the earth-retaining structure there is a water pool, as it happens for wharves, hydrodynamic effects should be taken into account. These phenomena can be simulated by an added mass applied to the wall, to be considered only for horizontal inertia and not for gravity action (Westergaard, 1933). The standard Newmark method does not allow these effects to be taken into account.

NEWMARK METHOD APPLIED TO GRAVITY WHARVES WITH DOUBLE-SUPPORT EXCITATION

To overcome the limitations of the Newmark method while retaining most of its simplicity, Corigliano et al. (2010) and Corigliano et al. (2011) proposed a novel approach aimed at estimating the permanent displacement of a gravity earth-retaining structure using a double-support excitation: one from the foundation soil and the other from the backfill. The method was originally developed for gravity earth-retaining walls in the absence of water (i.e. assuming dry soils). In the present work the technique is modified to account for the hydrostatic and hydrodynamic effects which need to be taken into account in case of wharf structures. The gravity wharf is idealized as a single degree of freedom system having only the capability to slide (horizontally). The gravity wharf is subjected to the geostatic earth pressure, the inertia of the wall itself, the inertia force in the backfill, the dynamic increment of earth pressure due to the shaking of the backfill, and the hydrostatic and hydrodynamic effects due to the presence of the water. Hence the gravity wharf is modeled as a rigid body subjected to a double support seismic input. One source of excitation comes from the foundation base as in the standard Newmark method, the other acts laterally and is due to the soil backfill. The gravity wharf slides when the sum of the horizontal forces acting on the wall is greater than the frictional resistance between the wall and the soil. The governing dynamic equilibrium equation is therefore the following:

$$\sum F_H(t) > F_V \cdot \tan\delta = (W - U_b) \cdot \tan\delta \qquad (1)$$

where W is the weight of the wharf, δ is the friction angle between the soil and the wall and U_b is the water pressure at the base of the wall which will be defined below. The vertical component of the soil thrust is neglected. Figure 1 shows the forces acting on the gravity wharf reported in Eqs. (1) and (3). The *critical acceleration*, which is the threshold acceleration above which the wall slides, is defined as:

$$a_{crit} = \frac{F_V \cdot \tan\delta}{M} \tag{2}$$

where M is the mass of the wharf.

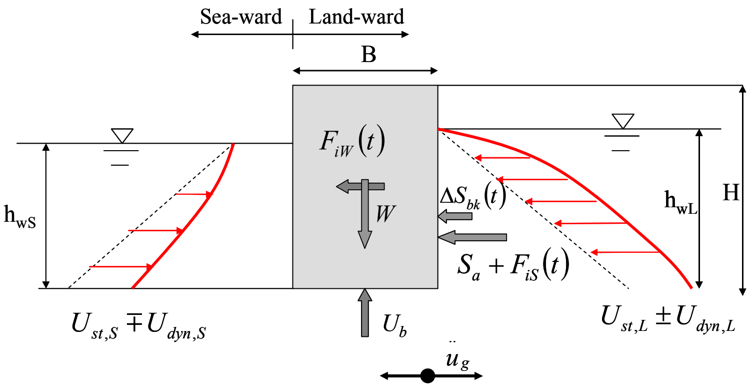


Figure 1. Forces acting on a gravity wharf during a seismic excitation

The resultant horizontal force is the sum of several contributions:

$$\sum F_H(t) = S_a + F_{iS}(t) + F_{iW}(t) + \Delta S_{bk}(t) - \left[U_{st,S} \mp U_{dyn,S}\right] + \left[U_{st,L} \pm U_{dyn,L}\right] \tag{3}$$

Forces acting on the wharf due to the presence of water:

- $U_{st,L} = \frac{1}{2}\rho_W \cdot g \cdot h_{wL}^2$ is the hydrostatic force on the land-ward side of the wharf and γ_W is the unit weight of the water;
- $U_{st,S} = \frac{1}{2}\rho_W \cdot g \cdot h_{wS}^2$ is the hydrostatic force on the sea-ward side of the wharf;
- $U_{dyn,S} = \pm \frac{7}{12} k_h \cdot \gamma_W \cdot h_{wS}^2 = \pm \frac{7}{12} \ddot{u}_{b,ff} \cdot \rho_W \cdot h_{wS}^2$ is the hydrodynamic force acting on the vertical face on the sea-ward side of the wharf calculated using *Westergaard's (1933) theory*, k_h is the seismic coefficient, $\ddot{u}_{b,ff}$ is the acceleration of the soil in free-field conditions (see Figure 4);
- $U_{dyn,L} = \pm \frac{7}{12} k_h \cdot \gamma_W \cdot h_{wL}^2 = \pm \frac{7}{12} \ddot{u}_{bk} \cdot \rho_W \cdot h_{wL}^2$ is the hydrodynamic force acting on the vertical face on the land-ward side of the wharf which is present for the case of *dynamically highly pervious soils* (i.e. the internal water is free to move with respect to the soil skeleton) whereas is zero for *impervious* soils (i.e. no drainage can occur under the seismic action). According to Eurocode 8 – Part 5 for a coefficient of hydraulic conductivity less than $5 \cdot 10^{-4}$ m/s, the pore water pressure is not free to move with respect to the soil skeleton and the soil may be treated as single-phase medium;
- $U_b = \frac{1}{2}(u_L + u_S)B$ is the water pressure at the base of the wharf (Bellezza and Fentini, 2008), B is the width of the wharf, u_L is the pore water pressure at the base of the wharf on the land-ward side and u_S is the pore water pressure at the base of the wharf on the sea-ward side. They are given by:

$$u_L = u_{L,stat} + u_{L,dyn} = \rho_W \cdot g \cdot h_{wL} \pm 7/8\, \ddot{u}_{bk}\, \rho_w h_{wL} (\chi - 1) \tag{4}$$

$$u_S = u_{S,stat} + u_{S,dyn} = \rho_W \cdot g \cdot h_{wS} \mp 7/8\, \ddot{u}_{b,ff}\, \rho_w h_{wS} \tag{5}$$

where χ is a dummy parameter equal to 2 for pervious soil and equal to 1 for impervious soil. $\ddot{u}_{bk}$ and $\ddot{u}_{b,ff}$ are acceleration time histories which will be defined below. In the present formulation, the effect of the excess of pore water pressure in the backfill soil is neglected.

Forces acting on the wharf due to wall inertia and soil backfill:

- $S_a = 1/2 K_{ah} \cdot \gamma^* \cdot H^2$ is the horizontal component of the static earth pressure computed according to the Coulomb (1776) and Müller-Breslau (1906) classical theory, H is the height of the wall and γ^* is the equivalent soil unit weight for partially submerged backfill (Kramer, 1996):

$$\gamma^* = \gamma' \cdot \lambda^2 + \gamma_d \left(1 - \lambda^2\right) \tag{6}$$

  where $\lambda = h_{wL}/H$, γ' and γ_d are the buoyant and dry unit weight of the soil respectively.
- $F_{iS}(t) = m_{stat} \cdot \ddot{u}_{bk}$ is the inertia force in the backfill, where $\ddot{u}_{bk}$ is the backfill acceleration and m_{stat} is the mass within the static Coulomb wedge of failure which will be defined below (see Figure 2);
- $F_{iW}(t) = k_h \cdot W = M \cdot \ddot{u}_g$ is the inertia force of the wall, k_h is the seismic coefficient and $\ddot{u}_b$ is the base acceleration of the ground;
- $\Delta S_{bk}(t)$ is the dynamic thrust increment induced by the backfill, which will be defined below.

Differently than with the classical Newmark method, the proposed approach does not require the definition of the critical (“*yield*”) acceleration, therefore it does not involve the use of the pseudo-static approach. At this purpose the dynamic thrust increment induced by the backfill (ΔS_{bk}) is obtained as the product of the backfill acceleration ($\ddot{u}_{bk}$) with the dynamic mass increment (Δm_{bk}) which is defined from the inclination of the failure surface in static and dynamic conditions. The inclination of the failure surface affects the magnitude of the lateral force acting on the retaining structure. Steedman and Zeng (1990) introduced the effects of phase change in the calculation of the dynamic thrust. The inclination of the failure surface obtained by these authors depends on the frequency content of the signal and on the shear wave velocity of the backfill. Steedman and Zeng (1990) showed that the dynamic wedge obtained with their model is smaller in size than the active wedge obtained using the classical Mononobe-Okabe theory. Unfortunately there is no closed-form solution for the computation of the inclination of the failure surface proposed by Steedman and Zeng (1990). Figure 2 shows the inclination of the failure surface under both static and dynamic loading.
The inclination under static loading conditions has been calculated according to the Coulomb (1776) and Müller-Breslau (1906) theory:

$$\tan \alpha_C = \left[\frac{(\sin\varphi' \cos\delta)^{1/2}}{\cos\varphi' \{\sin(\varphi' + \delta)\}^{1/2}} \right] + \tan\varphi' \tag{7}$$

The inclination under dynamic loading conditions was calculated using the Mononobe-Okabe theory (Okabe, 1926; Mononobe and Matsuo, 1929) since a closed-form solution actually exists in this case, and also because it represents an upper bound solution of the true inclination. This implies a conservative approach since it leads to a larger value of the horizontal forces induced by the backfill.

The closed-form solution of the angle of inclination of the failure surface predicted by the Mononobe-Okabe theory (α_{MO}) is given by the following relation derived by Zarrabi-Kashani (1979):

$$\alpha_{MO} = \varphi' - \psi + \tan^{-1}\left[\frac{-\tan(\varphi'-\psi) + C_1}{C_2}\right] \quad (8)$$

where

$$C_1 = \sqrt{\tan(\varphi'-\psi)[\tan(\varphi'-\psi) + \mathrm{cotg}(\varphi'-\psi)][1 + tg(\delta+\psi)\mathrm{cotg}(\varphi'-\psi)]} \quad (9)$$

$$C_2 = 1 + \{tg(\delta+\psi)[\tan(\varphi'-\psi) + \mathrm{cotg}(\varphi'-\psi)]\} \quad (10)$$

The value of the seismic angle ψ depends on the hydraulic conductivity of the soil. For *dynamically highly pervious soils* it can be computed from:

$$\tan\psi = \left[\frac{\gamma\lambda^2 + \gamma_d(1-\lambda^2)}{\gamma'\lambda^2 + \gamma_d\left(1-\lambda^2\right)} k_h\right] \quad (11)$$

whereas for *impervious soils*:

$$\tan\psi = \left[\frac{\gamma_d\lambda^2 + \gamma_d(1-\lambda^2)}{\gamma'\lambda^2 + \gamma_d\left(1-\lambda^2\right)} k_h\right] \quad (12)$$

where k_h is the horizontal seismic coefficient given by:

$$k_h = \ddot{u}_g / g \Rightarrow k_h(t) \Rightarrow \alpha_{MO}(t) \quad (13)$$

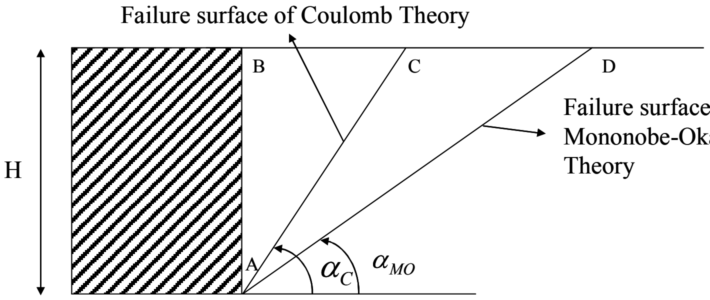


Figure 2. Failure surfaces in the backfill under both static and dynamic loading conditions

For simplicity the vertical component of the seismic motion has been neglected (i.e. k_v=0), but it may be easily included. Equations (8) and (13) show that the inclination of the failure surface depends on the acceleration time history, therefore it varies with time. The dynamic mass increment Δm_{bk} is obtained as the difference between the inertia of the dynamic wedge m_{dyn} (i.e. triangle ACD of Figure 2) and the static active wedge m_{stat} (i.e. triangle ABC of Figure 2):

$$m_{stat} = \frac{1}{2}\rho^* \frac{H^2}{\tan\alpha_C} \tag{14}$$

$$m_{dyn} = \frac{1}{2}\rho^* \frac{H^2}{\tan\alpha_{MO}} \tag{15}$$

where ρ^* is the equivalent mass density for partially submerged backfill ($=\gamma^*/g$). Thus, the dynamic mass increment is given by:

$$\Delta m_{bk} = m_{dyn} - m_{stat} \tag{16}$$

Finally, the dynamic thrust increment ΔS_{bk} induced by the backfill is given by:

$$\Delta S_{bk}(t) = \Delta m_{bk} \cdot \ddot{u}_{bk} \tag{17}$$

where $\ddot{u}_{bk}$ is the acceleration of the backfill. Figure 3 shows the soil mass enclosed between the static and dynamic active wedges, which induces a dynamic thrust increment to the wharf structure. This is ultimately due to the shaking of the backfill.

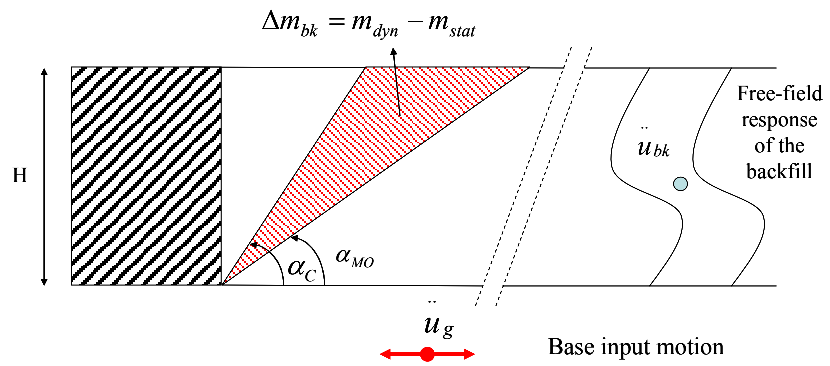


Figure 3. Soil mass inducing a dynamic thrust increment in the wharf due to shaking of the backfill

The proposed model requires the definition of two different time-histories of ground motion. The first is that at the soil-foundation base $\ddot{u}_b$ whereas the second from the backfill $\ddot{u}_{bk}$. Finally, in order to evaluate the hydrodynamic effect on the sea-ward side, the ground motion at the base of the water level $\ddot{u}_{b,ff}$ is required. The three acceleration time-histories can be calculated by exploiting the notion of transfer function using 1D linear wave propagation theory. The calculation is conducted for three, one-dimensional, layered viscoelastic soil profiles overlaying an elastic bedrock (see Figure 4). The first soil profile includes the presence of the wharf (see profile 1 in Figure 4). The second profile is assumed distant from the wall and it includes the stratigraphy of the backfill (see profile 2 in Figure 4). As Figure 4 shows, both acceleration time histories are computed as *inside motion*. The transfer function relating the displacement amplitude at layer *j* to that of rock outcrop is given by:

$$F(\varpi) = \frac{u_j}{u_0} = \frac{\textit{inside soil amplitude}}{\textit{rock outcrop amplitude}} \tag{18}$$

For soil profile 3 the observation point is on the free surface and the transfer function is defined as:

$$F(\varpi) = \frac{u_1}{u_0} = \frac{surface\ soil\ amplitude}{rock\ outcrop\ amplitude} \tag{19}$$

Transfer functions have been obtained assuming the soil deposit consisting of *N* viscoelastic horizontal layers obeying to the *Kelvin-Voigt* constitutive relation, overlaying an elastic layer.

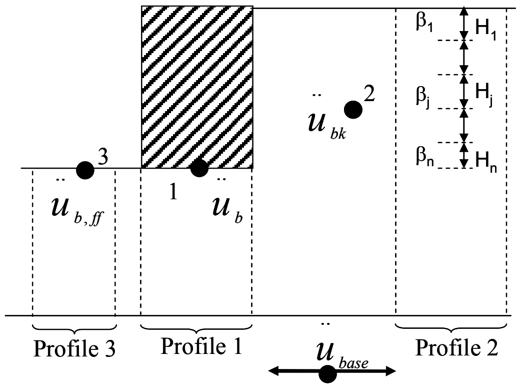


Figure 4. Sketch of the wharf system and location of the points where the time histories are computed by exploiting the notion of transfer function using 1D linear wave propagation theory

To account for the phase difference between the time histories at point 1 and 2 (see Figure 4), a time shift (t_{12}) was applied to the time history at the backfill ($\ddot{u}_{bk}$). This time shift was computed as the time required by the signal to travel between point 1 and 2:

$$t_{1,2} = \sum_{n}^{j} \frac{H_i}{\beta_i} \tag{20}$$

where H_i is the thickness of layer *i* and β_i is the shear wave velocity of the layer *i*. The wharf structure is thus subjected to a double-support excitation which accounts for possible amplification effects in the backfill and phase difference in the seismic input due to the time lag between the two excitations. The effects of the vertical component of ground motion could also be taken into account in the calculation of the normal force at the base of the gravity wharf, but for simplicity in the present work it has been neglected. The current formulation could be easily modified to include this effect. Several studies and recommendations (e.g. Seed & Whitman, 1970; Gazetas et al., 2004; Daukolas & Gazetas, 2008; JSCE,2000; OCDI, 2002; PIANC, 2001) have shown that vertical acceleration has practically no appreciable effect on the response of gravity wharf systems (Daukolas & Gazetas, 2008). Since the wharf is modeled as a SDOF system, base-sliding is the only failure mechanism that has been considered. Safety against overturning and bearing capacity failure mechanisms should therefore be assessed independently using a force-based approach combined with the capacity design philosophy. As discussed at the beginning of this section, the wall slides when:

$$\sum F_H(t) > F_V(t) \cdot \tan\delta \tag{21}$$

During the time lapse in which the horizontal forces acting on the structure are greater than the frictional resistance at the base of the wall, permanent displacement occurs and it is accumulated. The relative acceleration of the wharf can be obtained by dividing the fraction of the total horizontal force exceeding the frictional resistance by the mass of the wall, as follows:

$$a_{rel} = \frac{\left[\sum F_H(t) - F_V(t) \cdot \tan\,\delta\right] > 0}{M} \tag{22}$$

According to the classical Newmark theory, double integration of the relative acceleration obtained from Eq. (22) provides an estimate of the permanent displacement undergone by the gravity wharf.

APPLICATIONS

The model illustrated in the previous section has been validated by Corigliano et al. (2011) through a series of advanced numerical analyses performed using the finite difference-based code FLAC v5 (Itasca, 2005) for dry soil conditions. The presence of water in front of the wall and its influence is investigated in this work together with the permeability conditions in the backfill. The cross-section of the wharf structure is assumed with a rectangular shape, 4 m height and 3 m thick. The soil profile is assumed made by two layers: the bottom layer from the base of the model to the foundation of the wall and the backfill behind the wharf. The angle of shear resistance of the backfill is assumed equal to 30° whereas the soil-foundation friction angle 15°. The elastic parameters adopted in the analyses are listed in Table 1.

Table 1. Viscoelastic parameters assumed for the soil and the wharf structure

Soil properties	Bottom layer	Backfill	Wall
Bulk modulus (MPa)	875	66.7	16700
Shear modulus (MPa)	404	40	12500
Mass density (kg/m^3)	2000	1700	2400
Shear wave velocity (m/s)	450	153	2280
Damping ratio (%)	1	1	1

The seismic input is represented by an accelerogram recorded during the 1994 Northridge earthquake scaled to a PGA equal to 0.264g. The accelerogram was recorded on stiff ground and free-field conditions. The accelerogram was applied two times using forward and inverted polarities. Figure 5 shows the time-history of the resultant of the horizontal forces acting on the gravity wharf for the inverted polarity of the input motion and pervious conditions in the backfill computed using the modified Newmark model. Figure 5 also shows the frictional resistance at the base of the wharf. The figure allows to calculate the time interval during which the total horizontal force is greater than the frictional resistance (illustrated by the square window) with the consequential sliding of the wall. Double integration of the relative acceleration obtained using Eq. (22) provides an estimate of the permanent displacement undergone by the wharf at the end of the seismic excitation. Figure 6 shows for the input motion applied with both forward and inverted polarities, the comparison between the permanent displacements obtained with the modified Newmark method with double support excitation considering three cases:

- dry conditions in the backfill and no water in front of the wall
- impervious conditions in the backfill and water in front of the wall
- pervious conditions in the backfill and water in front of the wall

The figure illustrates the influence of water and hydrodynamic effects in the assessment of permanent displacement of a gravity wharf. Analyses are underway to compare the results of these calculations with those obtained using advanced numerical modeling.

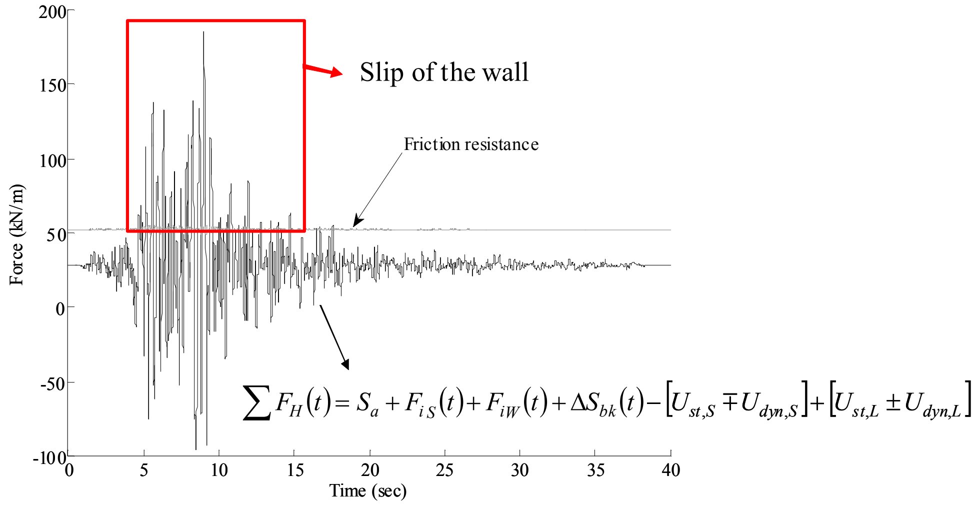
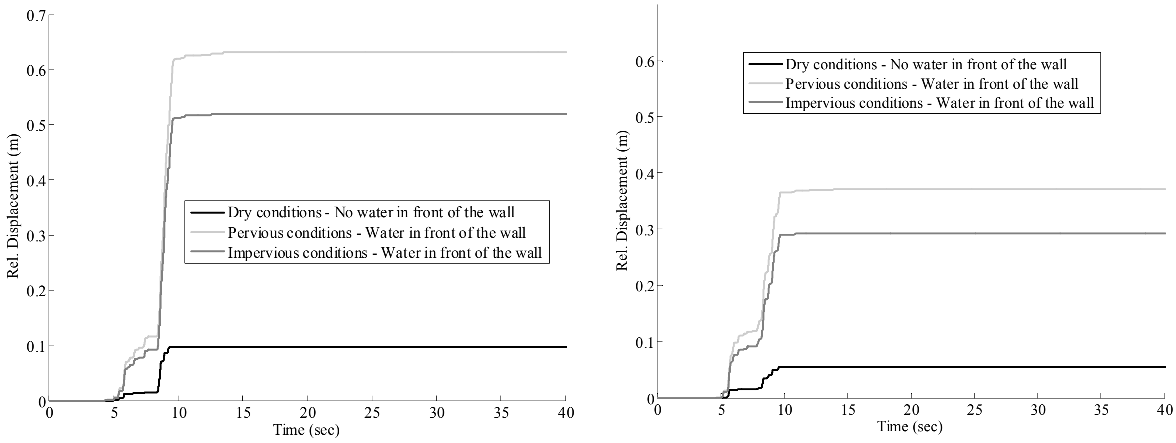


Figure 5. Resultant of the horizontal forces acting on the gravity wharf compared with the frictional resistance at the soil-foundation interface. Case of pervious conditions in the backfill and inverted polarity



a) b)

Figure 6. Comparison among displacement time-histories computed with the modified Newmark method with double support excitation considering – dry conditions in the backfill and no water in front of the wall – Impervious conditions in the backfill and water in front of the wall – Pervious conditions in the backfill and water in front of the wall: a) TH with forward polarity b) TH with inverted polarity

CONCLUDING REMARKS

A novel method is proposed to overcome the main drawbacks of the standard Newmark method applied to the seismic analysis of gravity wharves. Compared to the latter, the proposed model allows introducing the effects of the double-support excitation and of hydrodynamic effects with a modelling effort extremely reduced with respect to that required by advanced numerical time-history analyses.

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