



Seismic performance and design approach for friction dissipative foundations

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ABSTRACT

The seismic actions transmitted to a structure can be limited by inserting a frictional system between the foundation and the underlying soil. This allows a controlled sliding when the seismic forces reach a given critical value, that should be chosen to provide a desired seismic performance of the structure. Although this type of dissipative foundation has recently been adopted for the towers of two long-span bridges, its design still requires the development of a clear and robust design approach. In this view, the present paper studies the response of dissipative foundations, developing a generalised non-dimensional formulation that can be useful for design. The dynamic response of the foundation is analysed using a Newmark rigid block model, that is modified to account for the multi-directionality of the ground motion and to include a simplified description of the dynamic response of the superstructure. The main factors affecting the dynamic response of the dissipative system are highlighted through an extensive parametric study, leading to the definition of an optimised design approach of the dissipative foundation.

1. Introduction

An attracting energy-dissipating technology was recently implemented in the submerged foundations of the Rion-Antirion cable-stayed bridge in Greece (Pecker 2003 [1]) and the Izmit Bay suspension bridge in Turkey (Zhang et al., 2013 [2]). In both cases the tower foundations are founded on a group of driven steel piles that are separated from the caisson by a thick layer of gravel that provides a frictional interface between the two foundation elements.

For the specific case of the Izmit Bay suspension bridge, the behaviour of the frictional foundations was evaluated with a series of coupled dynamic analyses of a three-dimensional soil-structure numerical model, carried out by Gorini and Callisto (2015) [3] and Callisto and Gorini (2018) [4]. These analyses showed that the seismic response of the foundations is influenced by the presence of the pile group in the foundation soil and by the vertical component of ground motion, that has a strong effect on the instantaneous effective stresses acting on the soil-caisson interface.

At the end of this study it was felt that, although a complex dynamic analysis of the soil-structure interaction can indeed provide some insight into the behaviour of a specific structure, there is the need to develop an approach to the analysis of this innovative frictional device that is simple enough to be used at the design stage. This simplified

approach, based on modifications and generalisations of the rigid-block Newmark approach (1965) [5] to the problem at hand, is described in the following section.

2. Simplified model

Following the procedure recently proposed by Gorini and Callisto (2016) [6], the calculation method is based on a decoupled approach, in which the seismic time-histories at the foundation level are obtained by a free-field site response analysis, that takes into account the eventual presence of piles in the foundation soil through a kinematic modification of the foundation input motion. The seismic motion at the foundation level is then applied to a simplified model for the foundation-structure interaction, based on a generalisation of the Newmark displacement method [5], as shown in Fig. 1. The model includes a rigid block, representing the foundation, coupled with three single-degree-of-freedom (SDoF) systems oriented in three mutually orthogonal directions, which are aimed to reproduce the dynamic response of the superstructure. The dissipative soil-foundation interface is modelled as a frictional contact between the rigid block and the underlying base.

Fig. 2 shows a schematic representation of the two-dimensional model. The foundation block is characterised by its mass m_f only while the response of the superstructure is defined by the mechanical

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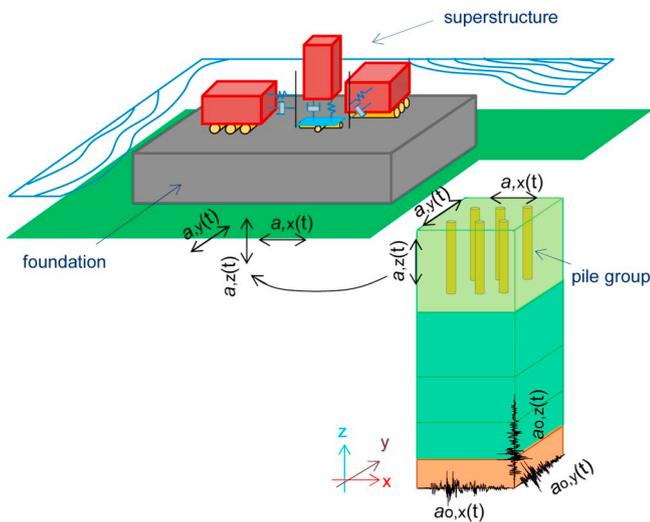


Fig. 1. Scheme of the analysis method and of the modified Newmark model.

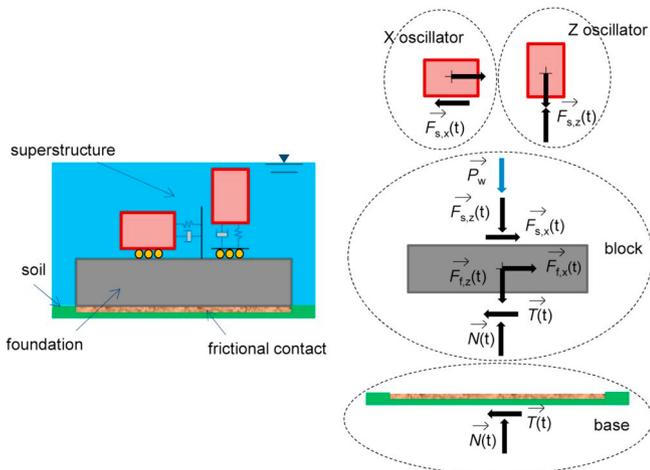


Fig. 2. Load pattern of the modified Newmark model for the bi-dimensional problem (plane X-Z).

properties of the two SDoF systems. The interface behaviour at the soil-foundation contact is rigid-perfectly plastic: the relative motion between the soil and the foundation occurs when the horizontal acceleration at the base is larger than the instantaneous value of the critical acceleration and the motion comes to an end when the relative velocity becomes equal to zero.

Fig. 2 depicts the forces acting in the system when a dynamic perturbation is applied to the base. An additional vertical force P_w was considered, to account for the presence of external forces, such as the weight of the water acting on a submerged foundation. The critical acceleration for the foundation is defined as the horizontal acceleration of the soil that mobilises the strength available along the frictional interface. Therefore, in a critical condition, the interface shear force $T(t)$ determined through the global equation of motion in the horizontal direction must be equal to its limiting value $T_{lim}(t)$, the latter expressed using the Mohr-Coulomb failure criterion as:

$$T_{lim}(t) = [N(t) - U(t)] \cdot \text{tg}(\varphi') + C' \quad (1)$$

where $N(t)$ is the resultant normal force acting on the soil-foundation interface while C' and φ' are the total cohesive resistance and the effective angle of shearing resistance along the sliding surface, respectively. For a submerged foundation, the effective vertical force is obtained by reducing $N(t)$ by the resultant of the pore water pressure $U(t)$ acting on the soil-foundation interface. The latter is composed of the

initial value and a time-dependent quantity function of the temporal variation of the interface normal force:

$$U(t) = U_0 + \Delta U(t) = U_0 + \delta \cdot \Delta N(t) \quad (2)$$

hence Eq. (1) reads:

$$\begin{aligned} T_{lim}(t) &= \{N_0 + \Delta N(t) - [U_0 + \Delta U(t)]\} \cdot \text{tg}(\varphi') + C' = \\ &= T_{lim,0} + [\Delta N(t) - \Delta U(t)] \cdot \text{tg}(\varphi') = T_{lim,0} + \Delta T(t) \end{aligned} \quad (3)$$

as the sum of the static limit shear force $T_{lim,0}$ and its dynamic variation $\Delta T(t)$.

It is assumed that sliding occurs along the contact between the foundation and the dissipative gravel at constant volume, with no generation of excess pore water pressures. The amount of effective vertical force transmitted to the superstructure is quantified by the drainage factor δ , ranging from 0 to 1: $\delta=0$ represents fully drained conditions while $\delta=1$ implies undrained conditions.

Any hydrodynamic effect in a submerged foundation may be considered adjusting either the mass of the foundation or modifying the vibration period of the SDoFs attached to the foundation raft.

By setting the relative acceleration between the soil and the foundation equal to zero (incipient motion), for the two-dimensional problem it is possible to define an explicit expression of the horizontal critical acceleration $\ddot{u}_{crit}(t)$:

$$\begin{aligned} \ddot{u}_{crit}(t) &= (1 + \alpha_x \cdot MR)^{-1} \cdot \{g \cdot (1 + MR) + (1 - \delta) \cdot [\ddot{u}_{bz}(t) + \alpha_z \cdot MR \cdot \ddot{u}_{sz}(t)]\} \\ &\cdot \text{tg}(\varphi') + \beta - \alpha_x \cdot MR \cdot \ddot{u}_{sf,x}^r(t) \end{aligned} \quad (4)$$

where α_x and α_z are the horizontal and vertical normalised participating masses of the superstructure, respectively. The term $\ddot{u}_{bz}(t)$ is the absolute vertical acceleration of the soil, assumed equal to the vertical acceleration of the foundation, and $\ddot{u}_{sf,x}^r(t)$ represents the relative acceleration of the superstructure with respect to the foundation in horizontal direction, evaluated through the local equation of motion of the SDoF system. In Eq. (4), the parameters MR (mass ratio) and β are defined as follows:

$$MR = \frac{m_s}{m_f} \quad (5)$$

$$\beta = \frac{Q}{m_f} \cdot \text{tg}(\varphi') + \frac{C'}{m_f} \quad (6)$$

with $Q = P_w - U_0$. The quantities m_s and m_f are the mass of the superstructure and of the foundation, respectively. Eq. (4) evidences that the critical acceleration is not a constant quantity: in addition to the mechanical properties of the system, it is altered by the vertical ground motion and by the dynamic response of the structure that alters the stress state along the interface. The proposed model is intrinsically able to account for the uplift of the foundation caused by vertical accelerations: an effective normal force on the frictional interface equal to zero implies a null critical acceleration.

For the three-dimensional model, the critical shear force has to be compared with the resultant shear force in the horizontal plane $T(t) = \sqrt{T_x(t)^2 + T_y(t)^2}$ and, developing this equality, it can be shown that the general equation of the relative soil-foundation motion can be written in a compact form as the sum of five terms:

$$\eta_z(t)^2 + \beta^2 = \ddot{u}_{th}(t)^2 + \ddot{U}_{sh}(t)^2 + \ddot{U}_{sf,h}^r(t)^2 \quad (8)$$

in which the main factors controlling the dynamic behaviour of the mechanical model are highlighted:

- $\eta_z(t)^2 = \{g \cdot (1 + MR) + (1 - \delta) \cdot [\ddot{u}_{bz}(t) + \alpha_z \cdot MR \cdot \ddot{u}_{sz}(t)]\}^2 \cdot \text{tg}(\varphi')^2 + 2 \cdot \text{tg}(\varphi') \cdot \beta \cdot \{g \cdot (1 + MR) + (1 - \delta) \cdot [\ddot{u}_{bz}(t) + \alpha_z \cdot MR \cdot \ddot{u}_{sz}(t)]\}$ which depends on the gravitational and vertical inertial forces;
- $\beta^2 = [Q/m_f \cdot \text{tg}(\varphi') + C'/m_f]^2$ which does not vary in time and depends on the mechanical properties of the model and on the resultant vertical force acting on the foundation;

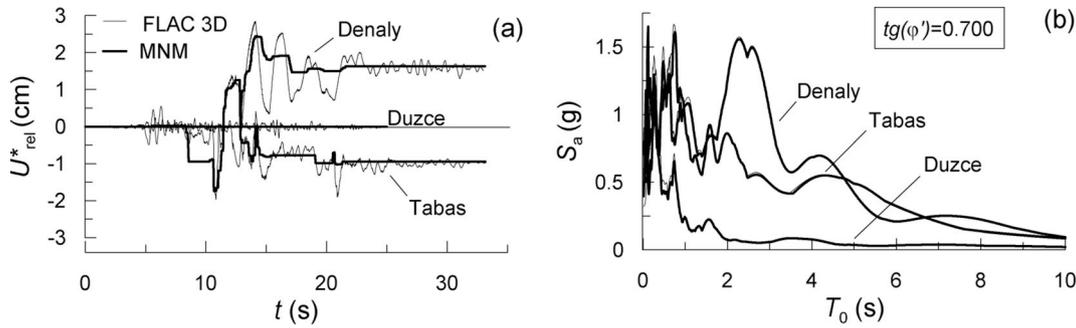


Fig. 3. Model validation: (a) time-histories of the relative horizontal displacements on the soil-foundation contact and (b) 5%-damped elastic response spectra in the caisson for an angle of shearing resistance of the interface equal to 35°.

- $\ddot{u}_{fh}(t)^2 = \ddot{u}_{fx}(t)^2 + \ddot{u}_{fy}(t)^2$ is the square of the absolute acceleration of the foundation in the horizontal plane;
- $\ddot{U}_{sh}(t)^2 = MR^2 \cdot [\alpha_x^2 \cdot \ddot{u}_{sx}(t)^2 + \alpha_y^2 \cdot \ddot{u}_{sy}(t)^2]$ is a term related to the dynamic response of the horizontal SDOF systems;
- $\ddot{U}_{sf,h}(t)^2 = 2 \cdot MR \cdot [\alpha_x \cdot \ddot{u}_{fx}(t) \cdot \ddot{u}_{sx}(t) + \alpha_y \cdot \ddot{u}_{fy}(t) \cdot \ddot{u}_{sy}(t)]$ which depends on the interaction of the block with the SDOF systems in horizontal direction.

In three dimensions, the equation of motion, Eq. (8), is an implicit non-linear relation that cannot be solved analytically in closed form. An iterative procedure is required to find the solution at each time step but, using a time increment sufficiently low, convergence can be obtained with only a few iterations. In two dimensions, Eq. (8) degenerates in the explicit form of Eq. (4).

2.1. Validation of the model

The model was validated against the results obtained with the coupled numerical model developed in Refs. [3,4] of the towers of the Izmit Bay suspension bridge [2]. Fig. 3 shows a comparison between the results of the soil-structure interaction analysis in Refs. [3,4] and those obtained with the present simplified formulation, using three different seismic records representative of the no-collapse design earthquake. Fig. 3(a) shows a comparison of the time histories of the relative displacements along the soil-foundation contact, computed with the full and the simplified model, while Fig. 3(b) compares the 5%-damped elastic response spectra of the motion transmitted to the foundation caisson. It is evident that the simplified model is able to reproduce with a good accuracy the dynamic response of the full soil-structure interaction model.

2.2. Non-dimensional formulation

Through a non-dimensional analysis of the problem at hand using the Buckingham pi-theorem, we have defined the 14 non-dimensional groups, listed in Table 1, that characterise completely the mechanical model. The reference values of these groups were inspired by the specific case study of the Izmit Bay bridge [2–4]. Ricker wavelets were chosen as the dynamic input for the structural model, that are characterised by only the maximum amplitude and predominant period. The amplitudes $A_{i,x}$ and $A_{i,y}$ of the horizontal input motion were taken equal to the acceleration of gravity for analysing instead the variability of the ratio $A_{i,z}/A_{i,x}$ between the amplitudes of the vertical and horizontal motion. The first vertical vibration period of the superstructure T_z was considered equal to $1.3 \times T_{i,z}$, with $T_{i,z}$ the period of the vertical input motion, as for the Izmit Bay Bridge. The effect of the excited mass of the superstructure was studied by varying the participating mass coefficient α_x , while α_y and α_z were kept constant equal to 0.25 and 0.85, respectively, representative of the participating masses associated with the respective first modes of the towers of the Izmit Bay bridge. An

Table 1
Non-dimensional groups.

Non-dimensional groups	Definition	Reference values	Range
Interface shear strength	$tg(\varphi')$	0.577	0.364–0.7
Mass ratio	m_s/m_f	0.35	0.0–2.0
X-direction excited mass ratio	α_x	0.25	0.0–1.0
X-direction interaction ratio	$T_x/T_{i,x}$	0.6	0.1–5.0
Y-direction interaction ratio	$T_y/T_{i,y}$	0.0	0.5–2.0
Damping ratio	ξ	2%	2%–10%
Drainage factor	δ	0.0	0.0–1.0
Pore pressure factor	PP/P_{tot}^a	0.0	0.0–0.8
X-V input motion amplitude	$A_{i,z}/A_{i,x}$	0.0	0.5–1.0
X-Y input motion ratio	$T_{i,y}/T_{i,x}$	0.0	0.25–10.0
X-V input motion ratio	$T_{i,x}/T_{i,z}$	0.0	1.0–7.0
X-V input motion phase difference	$\Delta t_{xz}/T_{i,x}$	0.0	0.0–0.39
Relative displacements U_{rel}	$U_{rel}^*/(g \cdot T_{i,x}^2)$	response parameter	
Non-dimensional spectral acceleration S_a	S_a^*/g	response parameter	

^a $P_{tot} = (m_f + m_s) \cdot g + P_w$.

additional vertical force $P_w = 0.6 \times PP$ was taken into consideration in the parametric analysis, with PP the pore water pressure acting on the soil-foundation interface (submerged foundation).

The seismic performance of the dissipative foundation was concisely described by the non-dimensional soil-foundation relative displacement U_{rel} and the non-dimensional spectral acceleration S_a .

3. Seismic performance of the friction dissipative foundation

The simplicity of the above model allowed to carry out a wide parametric study on the seismic performance of the friction dissipative foundation. All the results were expressed in a non-dimensional formulation.

3.1. Effect of the interaction between the horizontal dynamic input and the structural dynamic response

Let the mechanical system perturbed by a horizontal Ricker wavelet be the model used as reference, as shown in Fig. 2, and assume that the soil-foundation interface be dry. As a first result, the maximum relative displacements $U_{rel,max}$ occurring along the interface are plotted in Fig. 4(a) as a function of $T_x/T_{i,x}$ for a fixed value of the friction coefficient, equal to 0.577 ($\varphi' = 30^\circ$) and for different values of the normalised participating mass of the superstructure α_x , in which the curve related to the reference case of the Izmit Bay Bridge (α_x equal to 0.25) is highlighted; $\alpha_x = 1$ corresponds to a full participation of the superstructure mass to the dynamic response of the system, while $\alpha_x = 0$ means that the mass of the superstructure contributes exclusively to modify the contact pressure acting on the soil-foundation interface. It is

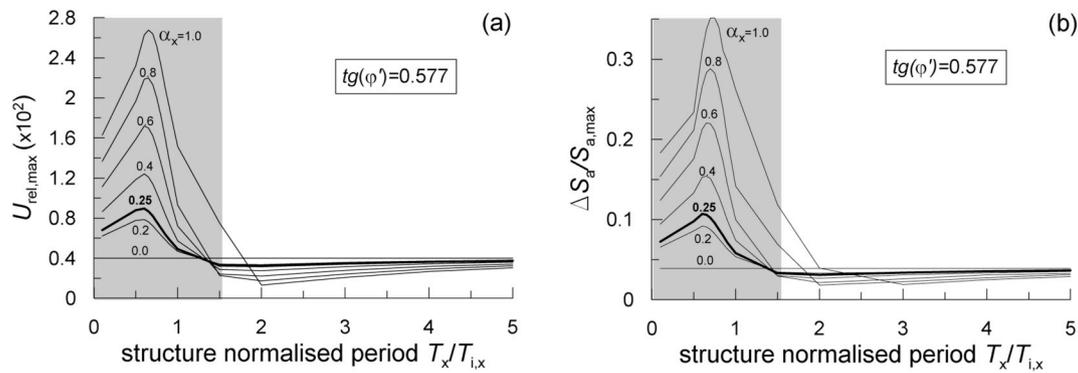


Fig. 4. Normalised maximum displacement (a) and normalised reduction of the maximum spectral acceleration in the foundation (b) plotted as a function of the structure normalised period $T_x/T_{i,x}$ for different values of the normalised participating mass of the superstructure α_x .

evident that the amplitude of the relative soil-foundation displacement depends on the interaction parameter $T_x/T_{i,x}$ and this dependence is emphasised by the normalised participating mass of the superstructure. Similar findings can be inferred looking at the actions transmitted into the structure during seismic motion: Fig. 4(b) plots the normalised maximum reduction of the spectral acceleration $\Delta S_a/S_{a,max}$ as a function of $T_x/T_{i,x}$. The difference ΔS_a represents the reduction of the spectral acceleration in the foundation with respect to the relative maximum value of the input motion $S_{a,max}$. Also in this case, for $T_x/T_{i,x}$ ranging from 0.1 up to about 1.5, the dynamic response of the dissipative system is emphasised compared to the response of a rigid block ($\alpha_x = 0$). In this range of the normalised structural period, the reduction of the seismic action transmitted to the superstructure is enhanced at the cost of greater relative displacements on the interface and this is due to the dynamic coupling between the input motion and the response of the superstructure. For the specific value of the mass ratio $m_s/m_f = 0.35$ taken as reference in these results, the peak of the dynamic amplification curves occurs when the normalised period of the superstructure is about 0.7. Moreover, it can be observed that, for every value of α_x , the two output quantities, generally denoted as Y_i , show a very similar dependence on $T_x/T_{i,x}$, and therefore are both a good indicator of the performance of the dissipative foundation.

The value of the normalised period of the superstructure corresponding to the optimal performance of the dissipative system is denoted $(T_x/T_{i,x})_{max}$. Fig. 5(a) plots $(T_x/T_{i,x})_{max}$ as a function of the composed non-dimensional parameter $\alpha_x \times m_s/m_f$, which represents the ratio between the excited mass in the superstructure and that in the foundation (relative excited mass). It can be seen that the parameter $(T_x/T_{i,x})_{max}$ starts from a value of roughly 0.6 and increases linearly up to one for $\alpha_x \times m_s/m_f = 0.5$, remaining constant for larger values of

$\alpha_x \times m_s/m_f$. In fact, the larger the excited mass of the superstructure, compared to the foundation mass, the larger $(T_x/T_{i,x})_{max}$, because the behaviour of the global structural system is increasingly more controlled by the superstructure. When the excited mass in the superstructure becomes greater than the half of the foundation mass ($\alpha_x \times m_s/m_f > 0.5$), the maximum amplification is entirely controlled by the dynamic characteristics of the superstructure and the resonance occurs for a constant value of $(T_x/T_{i,x})_{max} = 1.0$, as in the case of a massless foundation.

It is now interesting to consider the variation of the generalised response quantity Y_i as a function of the same composed parameter $\alpha_x \times m_s/m_f$, as depicted in Fig. 5(b). Also in this case, every output quantity shows a bi-linear trend, with a first transition part and, starting from $\alpha_x \times m_s/m_f = 0.5$, a steady behaviour in which the dependence from the composed parameter becomes much less pronounced.

3.2. Effect of the shear strength of the soil-foundation interface

This section explores the effect of the strength properties of the dissipative interface, assuming that it may be reduced by the use of smoother sliding surfaces. Specifically, the effect of the frictional coefficient of the dissipative interface is examined for values of the friction angle ϕ' ranging between 20° and 35° . Looking at the results illustrated in Fig. 6, the trend of the maximum reduction of the spectral acceleration $\Delta S_a/S_{a,max}$ moves to significantly greater values when the friction coefficient decreases. More in detail, the maximum value of $\Delta S_a/S_{a,max}$ decreases with ϕ' with a gradient that depends on $\alpha_x \times m_s/m_f$. For a given frictional coefficient of the interface, the maximum response of the system increases markedly with $\alpha_x \times m_s/m_f$ but this increment becomes less evident for $\alpha_x \times m_s/m_f > 1$.

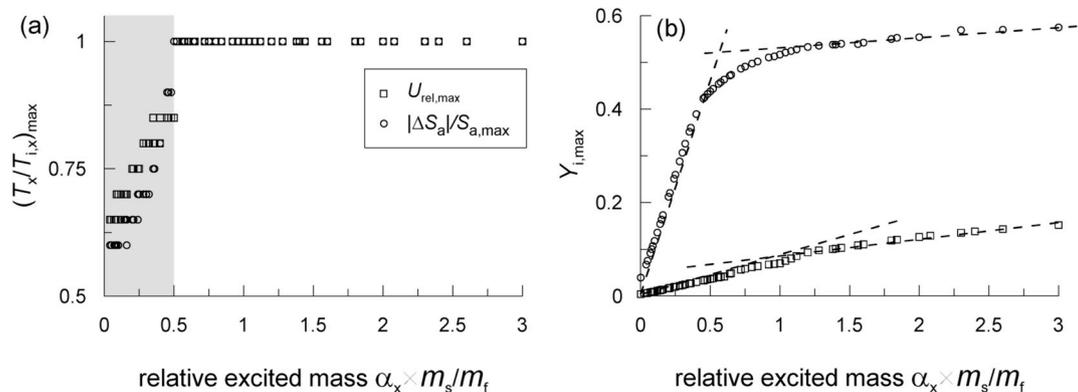


Fig. 5. (a) Variation of the parameter $(T_x/T_{i,x})_{max}$ corresponding to the maximum efficiency of the dissipative system as a function of the composed parameter $\alpha_x \times m_s/m_f$ for the two output quantities considered ($U_{rel,max}$, $|\Delta S_a|/S_{a,max}$); (b) maximum dynamic response of the system $Y_{i,max}$, in terms of $U_{rel,max}$ and $|\Delta S_a|/S_{a,max}$, plotted as a function of $\alpha_x \times m_s/m_f$.

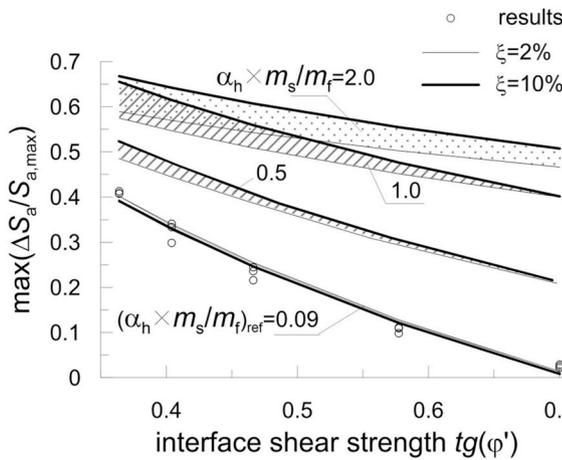


Fig. 6. Dependence of the maximum value of $\Delta S_a/S_{a,max}$ on the interface shear strength $tg(\varphi')$ and on the damping ratio of the superstructure ξ .

Fig. 6 also shows the effect of the damping ξ in the superstructure: an increment of the dissipative capabilities of the superstructure enhances significantly the response of the soil-foundation interface for values of $\alpha_x \times m_s/m_f > 1$. When the damping rises, in fact, the dynamic response of the superstructure is more attenuated and its motion tends to follow more closely the foundation motion, leading to inertial forces equally oriented that favour the activation of the sliding along the soil-foundation contact.

3.3. Effect of the vertical ground motion and drainage conditions

As explained in previous works [3,4], the vertical motion can affect the performance of the friction dissipative system, because it produces a continuous alteration of the effective normal stresses acting on the interface during seismic motion, resulting in a time-dependent critical acceleration. This effect is quantified herein by referring to an additional performance index of the system $U_{rel}/U_{rel,ref}^H$, defined as the ratio of the relative soil-foundation displacement that occurs when the structure is perturbed by a bi-component motion, in horizontal and vertical direction, to that produced by a solely horizontal signal. As noted in Section 3.1, the performance index can be also expressed in terms of the reduction of the seismic actions in the structure.

Fig. 7(a) plots the performance index as a function of three parameters: $T_{i,x}/T_{i,z}$ and $\Delta t_{xz}/T_{i,x}$, quantify the different frequencies and the phase difference between horizontal and vertical motion respectively, while $A_{i,z}/A_{i,x}$ represents a vertical-horizontal amplitude ratio. All the curves refer to the value of the interaction parameter that produces the maximum response in the reference case ($T_x/T_{i,x} = 0.7$ and $\alpha_x \times m_s/m_f = 0.09$). Looking at the results obtained for $A_{i,z}/A_{i,x}$ equal to 1, when the frequency of the two components of motion are similar ($T_{i,x}/T_{i,z} = 1$), the effect of the vertical motion is important and can produce either remarkable improvements of the seismic performance of the dissipative foundation, up to $U_{rel}/U_{rel,ref}^H = 5.0$ when the phase difference $\Delta t_{xz}/T_{i,x}$ is limited, or a reduction of the energy dissipated, with relative displacements roughly halved when the vertical motion occurs in phase opposition ($\Delta t_{xz}/T_{i,x} = 0.39$). The above effect, however, becomes much less important when $T_{i,x}/T_{i,z}$ increases, becoming negligible for values of $T_{i,x}/T_{i,z} > 6.0$, that is, for frequencies of the vertical motion much larger than those of the horizontal component. The intensity of the vertical motion has a relevant influence on the response for low values of $T_{i,x}/T_{i,z}$.

The combined effect of the dynamic response of the structure, the vertical motion and the presence of pore water pressures acting on the interface is illustrated in Fig. 7(b). In this case, the performance index $U_{rel}/U_{rel,ref}^H$ is more conveniently referred to the maximum displacement occurring for a bi-component motion ($U_{rel,ref}^H = 5$ in Fig. 7(a)) to highlight the effect of drainage conditions. It is clearly shown that the presence of pore water pressure enhances the dissipative capabilities of the frictional contact, because it produces a decrease of the critical acceleration. The amplitude of the performance index is minimum when the drainage is impeded because, in this condition, the effective normal force on the dissipative interface is not altered by the vertical component of the seismic motion.

3.4. Bi-directionality of the ground motion in the horizontal plane

The combined effect of the two horizontal components of the seismic motion is studied by determining the dependence of the response quantities on the parameters $T_x/T_{i,x}$, $T_y/T_{i,y}$ and $T_{i,y}/T_{i,x}$. Fig. 8 shows the relationship between the maximum reduction of the spectral acceleration $(\Delta S_a/S_{a,max})_{3D}$ in the X direction, computed when the structural system is perturbed by two horizontal and orthogonal dynamic signals, and the maximum value of $(\Delta S_a/S_{a,max})_{2D}$ obtained when a sole horizontal component of motion is considered. The first evident

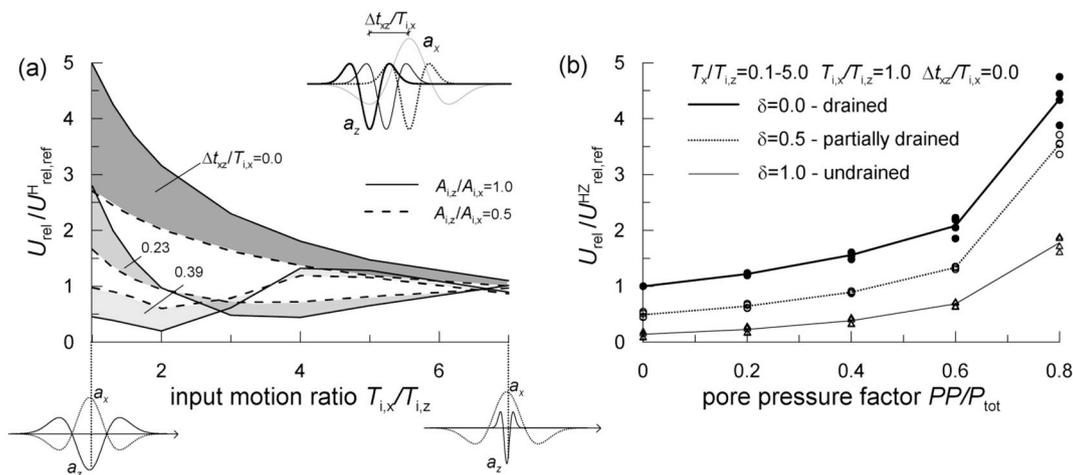


Fig. 7. (a) Effect of the vertical motion on the performance index $U_{rel}/U_{rel,ref}^H$ as a function of the parameters $T_{i,x}/T_{i,z}$, $\Delta t_{xz}/T_{i,x}$ and $A_{i,z}/A_{i,x}$; (b) variation of the dynamic response of the system, expressed in terms of the performance index $U_{rel}/U_{rel,ref}^H$, for the combined effect of the structure dynamic response ($T_x/T_{i,x}$), the presence of the vertical motion ($T_{i,x}/T_{i,z} = 1.0$, $\Delta t_{xz}/T_{i,x} = 0.0$ and $A_{i,z}/A_{i,x} = 1.0$) and the pore water pressure acting on the soil-foundation contact (PP/P_{tot}) for different drainage conditions (δ).

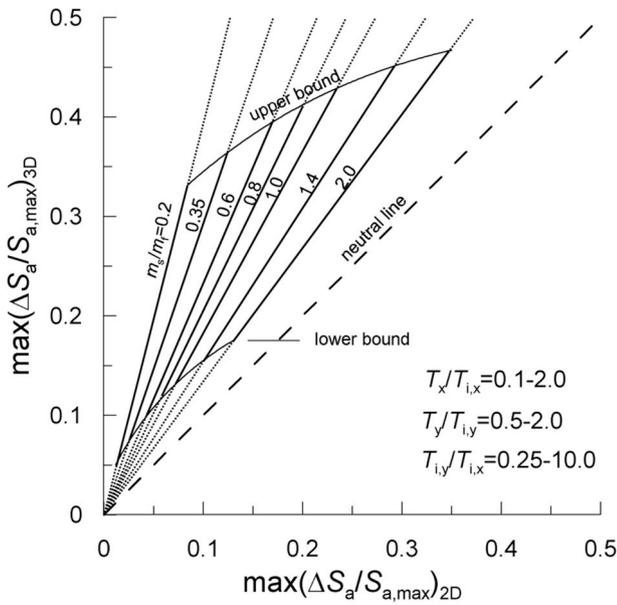


Fig. 8. Relation between the maximum reduction $(\Delta S_a/S_{a,max})_{3D}$ in X direction, related to the structural system perturbed by two horizontal and orthogonal dynamic Ricker wavelets, and the quantity $(\Delta S_a/S_{a,max})_{2D}$, produced by a sole horizontal Ricker wavelet in X direction, varying the parameters m_s/m_f , $T_x/T_{i,x}$, $T_y/T_{i,y}$ and $T_{i,y}/T_{i,x}$.

result is that, for a given dynamic perturbation in the X direction, the bi-directionality of the horizontal motion always provokes an increase of the dissipative response of the foundation. This is due to the reduction of the available shear strength on the soil-foundation contact in X direction because the overall strength is now attained in a different horizontal direction. The amount of this increment in efficiency varies as a function of the mass ratio m_s/m_f . More in detail, accepting a moderate dispersion of the results, it can be reasonably assumed that, for each value of m_s/m_f , $(\Delta S_a/S_{a,max})_{3D}$ increases linearly with $(\Delta S_a/S_{a,max})_{2D}$. Conversely, an increase of the mass ratio leads to a progressive reduction of the slope of these linear trends even if, as demonstrated previously, this implies a more amplified response of the foundation so that the upper and the lower bounds move to greater values of $(\Delta S_a/S_{a,max})_{3D}$ as m_s/m_f rises.

4. Design approach

The results obtained with the modified Newmark model can be used for a preliminary evaluation of the optimum configuration of the dissipative foundation.

The main target of this anti-seismic technology is to limit the maximum seismic forces transmitted to the superstructure. Hence, suppose that a design requirement specifies that the spectral acceleration at the base of a given structure need to be smaller than a limiting value $S_{a,lim}$. Then, the value of the generalised response quantity $Y_{i,lim}$ can be easily determined as the difference, normalised with respect to $S_{a,max}$, between the maximum spectral acceleration of the seismic input $S_{a,max}$ and $S_{a,lim}$. The preliminary configuration of the superstructure is assumed to be known, as well as the seismic demand for the structure. This implies that the following non-dimensional groups are known: α_x , α_y , α_z , $T_x/T_{i,x}$, $T_y/T_{i,y}$, $T_z/T_{i,z}$, ξ and PP/P_{tot} . The design of the dissipative foundation consists of the evaluation of the remaining groups m_s/m_f and $tg(\varphi')$, that can be accomplished following the procedure described below.

The response quantity $Y_{i,lim}$ can be decomposed in the X and Y directions, $Y_{i,lim,x}^{3D}$ and $Y_{i,lim,y}^{3D}$ respectively:

$$\begin{cases} Y_{i,lim,x}^{3D} = Y_{i,lim} \cdot \cos\theta \\ Y_{i,lim,y}^{3D} = Y_{i,lim} \cdot \sin\theta \end{cases} \quad (9)$$

where θ is the angle between the maximum principal direction of motion, as defined by Arias (1996) [7], and the X direction. The corresponding two-dimensional response quantities $Y_{i,lim,x}^{2D}$ and $Y_{i,lim,y}^{2D}$ can be derived from Fig. 8 by choosing a value of m_s/m_f , and adjusting the foundation mass so that the composed parameter $\alpha_x \times m_s/m_f$ leads to a value of $(T_x/T_{i,x})_{max}$ (Fig. 5(a)) approximately equal to $T_x/T_{i,x}$ of the structural system under examination. Since most of the graphs are related to the reference configuration (mono-component horizontal input motion), the equivalent two-dimensional limit values $Y_{i,lim,x}^{2D}$ and $Y_{i,lim,y}^{2D}$ can be corrected by means of Fig. 7 to also consider the effect of the vertical ground motion and the presence of pore water pressure on the soil-foundation interface. Finally, the shear strength of the dissipative contact can be determined in Fig. 6 according to Eq. (10).

$$tg(\varphi') = \mu = \min\{\mu(Y_{i,lim,x}^{2D}), \mu(Y_{i,lim,y}^{2D})\} \quad (10)$$

5. Conclusions

The adoption of a frictional contact between a foundation and the underlying soil can constitutes an efficient seismic control of the structure, limiting the maximum accelerations transmitted to the superstructure, provided that the properties of the dissipative foundation are appropriately calibrated considering its dynamic interaction with the superstructure: it was shown that a significant dynamic coupling produces an increment of the energy dissipated through the frictional mechanism. The parametric study indicated optimal configurations of the foundation, that maximise its dissipative capabilities. In that regard, a design approach was proposed aimed at identifying optimum values of the foundation mass and of the shear strength along the soil-foundation contact.

The frictional coefficient of the dissipative interface plays a key role in the performance of the foundation. The efficiency of the soil-foundation contact increases as the friction decreases, but this dependence is markedly altered by the dynamic response of the superstructure. It is important to consider that the available shear strength at the frictional contact is affected by any pore water pressure acting within the interface. Pore pressures have always the desirable effect of reducing the seismic actions transmitted to the structure, as a consequence of the decrease of the effective normal force on the interface, that in turn reduces the corresponding limit shear force.

The vertical component of the seismic motion is a relevant source of time-dependent modification of the frictional strength along the interface. It was seen that its effect is important when the main frequencies of the horizontal and vertical motion are similar, although its benefit on the seismic performance of the foundation depends considerably on the phase difference between the two components. The presence of the vertical motion can be neglected when its frequency is much larger than that of the horizontal component.

The bi-directionality of seismic motion in the horizontal plane always provokes an increment of the relative soil-foundation displacement, resulting in lower seismic forces transmitted to the superstructure. This effect is strongly controlled by the relative contribution of the superstructure and the foundation to the overall dynamic response of the structural system.

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