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Predicting the dynamic response of friction dissipative foundations using a modified Newmark model

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Abstract

This paper analyses the dynamic behaviour of a seismic isolation technology adopted for the foundations of the towers of two recent long-span bridges, consisting of a dissipative layer of granular soil placed at the contact of the foundation caisson with a group of foundation piles. The fundamental aspects of this isolation device is studied through a three-dimensional modified Newmark model that accounts for the multi-directionality of the seismic motion and includes a simplified description of the dynamic response of the superstructure. This simplified model is expressed in a rigorous non-dimensional formulation and is validated against the results of three-dimensional dynamic analysis of the soil-structure interaction for a specific case of a suspension bridge. Taking advantage of the non-dimensional formulation of the simplified dynamic model, a parametric study of general validity is carried out, that highlights the most significant factors influencing the performance of the dissipative device.

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Keywords: dissipative foundations; modified Newmark method; parametric analysis

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1. Introduction

An attractive energy-dissipating device has recently been implemented in the submerged foundations of two long-span bridges, namely the Rion-Antirion cable-stayed bridge [1] and the Izmit Bay suspension bridge [2]. In both cases the bridge towers are founded on a group of driven steel piles; these piles are separated from the overlying caisson by

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a layer of gravel poured onto the top of the pile heads that, under a severe seismic event, is intended to allow a relative horizontal displacement at the caisson-piles contact. This device implicitly limits the maximum shear force transmitted to the superstructure, at the cost of minor horizontal displacements of the base of the towers, than can be easily accommodated by the deformable bridge structure. For the specific case of the Izmit Bay bridge, Gorini & Callisto and Callisto & Gorini [3,4] performed a series of three-dimensional numerical analyses to evaluate the effectiveness of the dissipative foundation, showing that the actual performance of the foundation is strongly influenced by the actual properties of the foundation soils interacting with the pile group, and by the vertical component of the seismic motion.

It is evident that a computationally-demanding three-dimensional analysis cannot be regarded as an actual design tool: therefore, the results of the previous analyses prompted the development of models that could be simple enough to be readily employed for parametric calculations, extremely useful for design purposes, but that would at the same time retain the main features of the dynamic behaviour of the actual foundation. A decoupled approach was envisaged, in which the seismic loads, obtained by a free-field analysis of the site response, are then applied to a simplified model of the foundation, that should on the one hand incorporate explicitly the presence of the dissipative device, and on the other hand consider the dynamic response of the superstructure.

2. Development of the simplified model

The simplified model is based on a generalisation of the displacement method proposed by Newmark [5] to a three-dimensional condition, consisting of a rigid block (the foundation caisson) than can slide along a frictional interface (the dissipative device) and is subjected to a three-component acceleration record at its base. The model includes a three-degrees-of-freedom structural model, aimed to represent the dynamic response of the superstructure in the three mutually orthogonal directions (Fig. 1a). The case of a submerged foundation can be obtained by prescribing values of the pore water pressure in the interface and the corresponding values of the water pressure acting vertically on the caisson.

A detailed description of the mathematical formulation of the model is beyond the scope of this paper. Although the model was implemented for a general three-dimensional case, for sake of conciseness this paper deals only with a two-dimensional case, that considers a seismic motion occurring only in one horizontal direction, in addition to the vertical direction. For such a case, it can be shown that the relative motion at the dissipative interface occurs when the horizontal acceleration at the base is larger than the instantaneous value of critical acceleration $a_{cr}(t)$ given by the following expression:

$$a_{cr}(t)=(1+\alpha_x\cdot\frac{M_s}{M_c})^{-1}\cdot\left\{\left[g\cdot(1+\frac{M_s}{M_c})+\frac{d^2u_{b,v}(t)}{dt^2}+\frac{d^2u_{s,v}(t)}{dt^2}\cdot\alpha_v\cdot\frac{M_s}{M_c}\right]\cdot tg\varphi'-\alpha_x\cdot\frac{M_s}{M_c}\cdot\frac{d^2u^r{}_{sc,x}(t)}{dt^2}+\frac{P_w-PP}{M_c}\cdot tg\varphi'\right\} \qquad (1)$$

where $u_{b,v}$ and $u_{s,v}$ are the vertical displacements of the base and of the structural model, respectively, and $u^r{}_{sc,x}$ is the horizontal motion of the structural model relative to the rigid block; the remaining symbols are defined in Table 1. As in any Newmark-based model, the motion comes to an end when the relative velocity becomes equal to zero.

In a first stage of the work, the simplified model was validated against the results of the three-dimensional dynamic analyses of the soil-structure interaction for the case of the Izmit Bay bridge. It was found that, in order to obtain a good response from the simplified model, it is necessary that the analysis of the site response considers the increase in soil stiffness and strength provided by the driven piles. As an example, Figure 1b shows, for input signals corresponding the no-collapse earthquake, a satisfactory comparison between the time-histories of the relative displacements U^*_{rel} computed along the gravel interface using the complete and the simplified models, while Figure 1c shows a comparison between the 5%-damped elastic response spectra computed with the two methods.

In a subsequent stage the simplified model was used to carry out a number of parametric analyses. To this purpose, a rigorous non-dimensional formulation was developed for the problem at hand: from the 16 quantities that describe the model in two-dimensional conditions (Table 1) one can derive the 13 non-dimensional groups listed in Table 2. The reference values used for these groups are inspired to the properties of the Izmit Bay bridge. A simplified input was used in the parametric analysis, consisting of a Ricker wavelet for each component of the seismic motion, characterised by its maximum amplitude and a given predominant period. The possibility of a phase difference between the vertical and the horizontal components of the seismic motion was also considered. The model results can be expressed concisely using two output quantities, that represent non-dimensional values of the relative displacement U_{rel}, and of the spectral acceleration of the caisson motion S_{a} (Table 2).

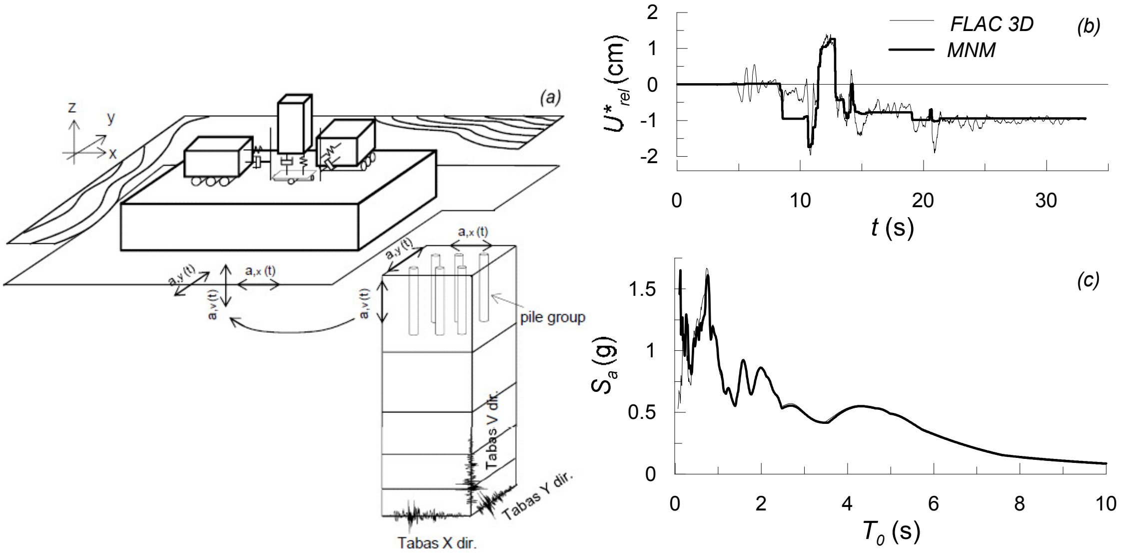

Fig. 1. (a) Scheme of the modified Newmark model (MNM) and of the procedure to determine the seismic input motion; validation of the model against the results of a complete finite difference model (FLAC 3D): (b) time histories of the relative horizontal displacements at the caisson-pile interface and (c) 5%-damped elastic response spectra in the caisson.

Table 1. Quantities describing the simplified model

	symbol	dimension	description
Mechanical system	g	L/T²	*Acceleration of gravity*
	tan(φ')	-	*Interface shear strength*
	M_{c}	M	*Mass of the caisson*
	M_{s}	M	*Mass of the structure*
	α_{x}	-	*Participating mass coefficient of the structure in X direction*
	α_{v}	-	*Participating mass coefficient of the structure in V direction*
	T_{x}	T	*Vibration period of the structure in X direction*
	T_{v}	T	*Vibration period of the structure in V direction*
	ξ	-	*Damping ratio*
	P_{w}	M L/T²	*Weight of the water column over the caisson*
	PP	M L/T²	*Pore pressure resultant on the interface*
Input motion	$A_{\mathrm{i,x}}$	L/T²	*Input motion amplitude in X direction*
	$A_{\mathrm{i,v}}$	L/T²	*Input motion amplitude in V direction*
	Δt_{xv}	T	*Phase difference of the X-V input motion*
	$T_{\mathrm{i,x}}$	T	*Period of the input motion in X direction*
	$T_{\mathrm{i,v}}$	T	*Period of the input motion in V direction*

The parametric study presented in this paper considers a variation of a limited number of non-dimensional groups within the intervals reported in Table 2. Specifically, a variation was considered for the ratio of the natural period of the superstructure to the predominant period of the horizontal input signal (X- direction interaction ratio XIR), the ratio of the excited masses in the X direction (XEM), the pore pressure factor (PPF), the horizontal-vertical motion ratio (XVM), and the horizontal-vertical phase difference (XVP).

3. Representative results

Figure 2a shows, for a range of values of the normalized period of the superstructure *XIR*, the temporal variation of the normalised relative displacement U_{rel} occurring along the dissipative interface, together with the time-history of the normalised input motion $A_{\mathrm{i,x}}$, showing that the relative displacements accumulate during the central part of the Ricker signal. Figure 2b plots the maximum value of U_{rel} as a function of *XIR*, for several values of the normalized participating mass of the superstructure *XEM*. It can be seen that the vibration period of the structural element influences the magnitude of the permanent displacement occurring along the dissipative interface, and that this influence increases with increasing participating mass of the structural element. The maximum relative displacements occur when the normalized period of the superstructure *XIR* is equal to about 0.6.

The beneficial effect of the dissipative device can be quantified by looking at the maximum spectral acceleration of the horizontal displacements computed in the rigid block, that are proportional to the maximum shear force transmitted to the base of the superstructure. Figure 3a shows that, for the reference case, a sustained attainment of the interface strength during the seismic motion results in a reduction ΔS_{a} of the spectral acceleration for normalized periods $T/T_{\mathrm{i,x}}$ in the interval of 0.4 to about 1.2, while at normalized periods of 0.2 to 0.4 the attainment of the interface strength produces an appreciable increase of the normalized spectral ordinates. The normalized reduction of the spectral acceleration show a variation with *XIR* and *XEM* that is very similar to that computed for the normalized relative displacement (Fig. 3b), demonstrating that both quantities (U_{rel} and $\Delta S_{\mathrm{a}}/S_{\mathrm{a,max}}$) are effective indicators of the efficiency of the dissipative interface.

Table 2. Non-dimensional groups and output quantities

non-dimensional groups	definition	reference values	range
Interface Shear Strength ISS	$tg(\varphi')$	0.577	-
Mass Ratio MR	M_s/ M_c	0.35	-
X-direction Excited Mass ratio XEM	α_x	0.25	0.0-1.0
V-direction Excited Mass ratio VEM	α_v	0.85	-
X-direction Interaction Ratio XIR	$T_x/ T_{i,x}$	0.6	0.1-5.0
V-direction Interaction Ratio VIR	$T_v/ T_{i,v}$	1.34	-
Damping Ratio DR	ξ	2%	-
Embedment Effect Factor EEF	P_w/PP	0.6	-
Pore Pressure Factor PPF	PP/P_{tot}*	0.0	0.0-0.8
X input Motion Amplitude XMA	$A_{i,x}/g$	1.0	-
X-V input Motion Amplitude XVA	$A_{i,v}/ A_{i,x}$	0.0	0.0-1.0
X-V input Motion ratio XVM	$T_{i,x}/ T_{i,v}$	-	1.0-2.0
X-V input motion Phase difference XVP	$\Delta t_{xv}/ T_{i,x}$	-	0.0-0.39

*P_{tot}=(M_c +M_s) ·g+ P_w

output	definition
U_{rel}	$U^*_{rel}/(g\cdot T_{i,x}^2)$
S_a	S_a^*/g

U_{rel}^*: caisson-gravel relative displacement [L];
S_a^*: spectral acceleration [L/T²].

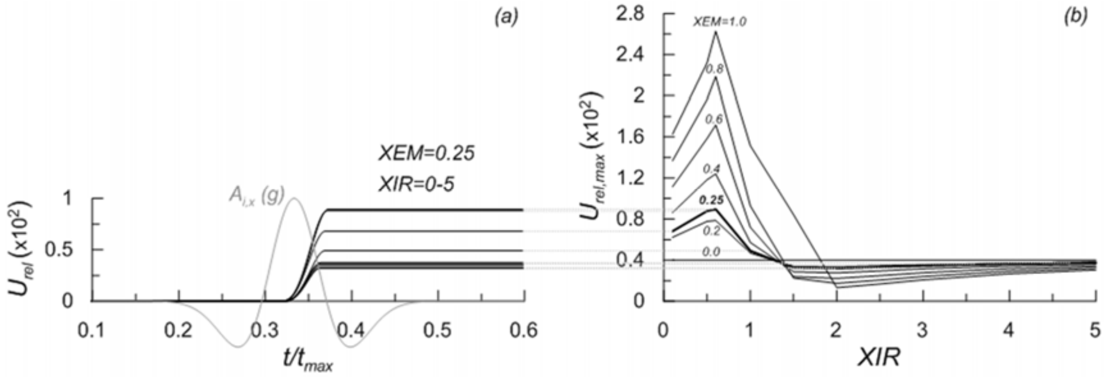

Fig. 2. (a) Time histories of the normalised horizontal displacements at the caisson-pile interface for different values of the parameter XIR; (b) maximum normalised displacement plotted as a function of XIR for different values of XEM.

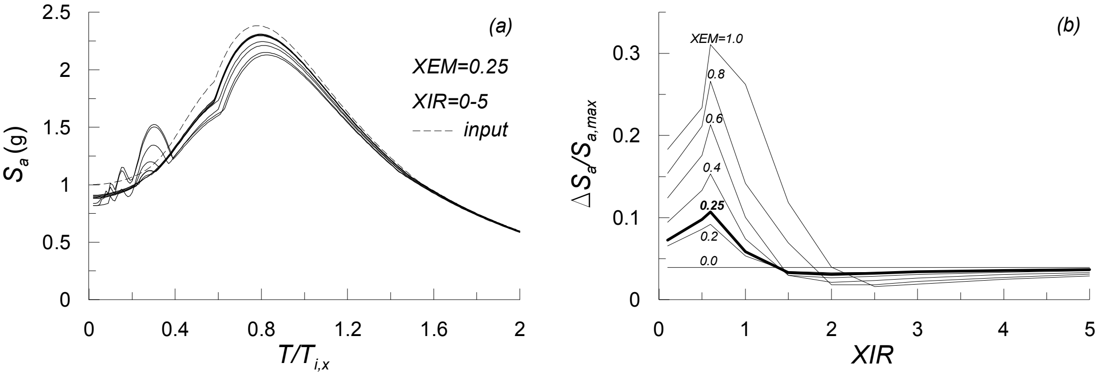

Fig. 3. (a) 5%-damped elastic response spectra in the caisson for different values of XIR; (b) the reduction of the maximum spectral accelerations plotted as a function of XIR for different values of XEM.

Figure 4a shows, for the reference case, the effect of the pore pressure factor *PPF* on the maximum displacements normalized with respect to those obtained in the reference analysis, $U_{\mathrm{rel}}/U_{\mathrm{rel,ref}}$. Increasing values of *PPF* produce a reduction of the effective stresses along the dissipative interface, resulting in significant increments of the normalized displacements, that can become, for $PPF = 0.8$, as large as 10-20 times the values obtained in the absence of pore water pressure. The effect of the vertical component of the input motion is depicted in Figure 4b, that shows the increment in the permanent displacement produced by a combination of horizontal and vertical Ricker signals characterized by the same amplitude ($XVA = 1$). In this study, relative to the value of $XIR = 0.6$ that corresponds to the maximum efficiency of the dissipative device, the maximum vertical acceleration is directed downwards, producing a decrease in the vertical effective stresses along the interface. For a vertical signal in phase with the horizontal one ($XVP = 0$) and characterized by the same predominant period ($XVM = 1$), the computed displacements are about five times larger than the ones computed in the reference case. As the frequency of the vertical input motion increases (increasing *XVM*) this effect becomes less pronounced. However, the increase in the permanent horizontal displacements due to the vertical input motion can be substantially reduced by a phase difference: for $XVP = 0.23$ the final displacements are roughly halved, while for $XVP = 0.39$ the horizontal and vertical motion occur essentially in phase opposition end the net effect is a decrease of the permanent displacements.

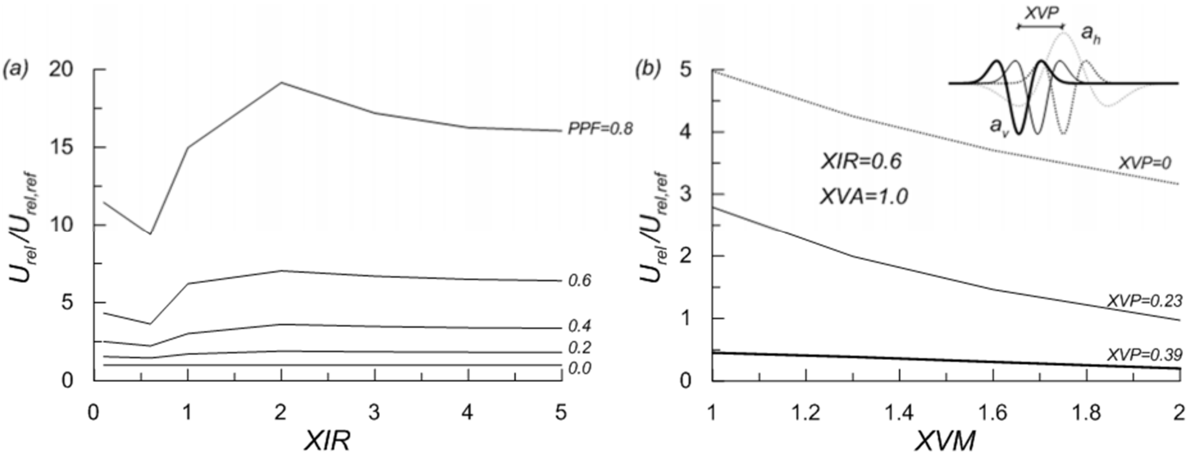

Fig. 4. (a) Effect of the pore water pressure group PPF on the permanent displacements; (b) effect of the vertical component of the input motion

4. Concluding remarks

The main advantage of the present simplified formulation is that it allows a quick evaluation of the response of the system to a combination of horizontal and vertical input signals, that allows to check the performance of the dissipative foundation for the specific interface strength and the main dynamic properties of the superstructure. It was shown that the basic effect of the dissipative device is to reduce the spectral acceleration of the signal transmitted to the structure base, at the cost of a certain relative displacement occurring along the dissipative interface. The spectral accelerations are reduced in a large period interval which in most cases encompasses the fundamental period of the superstructure. However, it was shown that at smaller periods the dissipative device produces in fact an amplification of the input motion, and this may have an effect for stiff structures.

The vertical components of the base motion have a profound effect on the effectiveness of the dissipative device, in terms of amplitude, frequency content and phase difference. Any evaluation of the performance of such a device for an actual structure must consider carefully the coupling between vertical and horizontal components of the seismic motion.

Finally, the validation of the present model against a complete numerical model of the soil-structure interaction showed that the simplified approach presented in the present paper can yield accurate results only if the input motion accounts properly for the characteristic of the foundation soils and for the kinematic effect produced by the pile group: care is needed to make sure that such effect are correctly incorporated in a free-filed analyses aimed to evaluate the base motion to feed into the present simplified model.

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