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A COMPUTATIONAL FRAMEWORK FOR THE ASSESSMENT OF EARTHQUAKE-INDUCED ROCKING IN CIDH PILE SUPPORTED BRIDGES

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Abstract. *The scope of this study is to investigate the response of RC bridges, founded on single cast-in-drilled-hole (CIDH) piles, under simultaneous earthquake-induced translational and rocking excitation. This rotational excitation results from pile bending under vertically propagating S-waves which, in turn, is dependent on pile's relative flexibility with respect to the surrounding soil, a phenomenon known as "kinematic interaction". Typically, the rotational component of seismic excitation of the superstructure is not taken into consideration; neither is it prescribed in any of the modern seismic codes, despite the fact that analytical solutions have been proposed in the literature. Moreover, the potential impact of such a pile-induced rocking of CIDH supported bridges has also not been quantified yet. Along these lines, an effort is made in this paper to present an analytical and computational framework for parametrically studying both: (a) the nature of the rotational excitation component and: (b) the additional displacement demand imposed to the superstructure. For this reason, a Matlab-based program is developed and the lateral response of multiple 4-DOF oscillators representing typical bridge structures is analytically studied, for various scenarios of excitation frequencies, superstructure height and soil stiffness. The resulting displacement demand is then compared to the displacements that would develop by ignoring the rotational component of ground excitation). From the set of parametric analyses conducted, it is concluded that ignoring kinematically induced rocking, transverse deck displacements may be significantly increased, especially in frequencies associated with the dynamic characteristics of the soil and the superstructure.*

1 INTRODUCTION

Bridge construction industry comprises nowadays the second most demanding construction sector in terms of overall investment; therefore, the safety of bridges constitutes a field of extensive research worldwide. Following a series of catastrophic earthquakes around the world (San Fernando 1971, Loma Prieta 1989, Costa Rica 1990, Northridge 1994, Kobe 1995, Kocaelli 1999, Chile 2010, Japan 2011) that caused serious damage and collapse in numerous bridges, the safety issue becomes more emerging. The reason is that bridges are part of commercial and transportation networks whose vulnerability determines the level of social impact that can be caused by a seismic event. As a result, nearly all modern seismic codes prescribe means to ensure a target level of performance related to bridge integrity and serviceability for various levels of earthquake loading, so that the probability of massive human loss is reduced and the disruption of the social and financial activity is as limited as possible.

Bridges, despite their relatively simple structural system compared to buildings, may exhibit quite complex seismic response due to their larger dimensions, their various non-linear mechanisms (stoppers, shear keys, gaps, bearing-type connections), the more significant contribution of higher modes, their higher sensitivity to the spatially variable properties of the surrounding soil and ground motion, the high soil compliance, as well as to the overall topography of the area crossed. As a result, it is not uncommon that the overall superstructure-foundation-subsoil system is studied as a whole and the dynamic interaction among its subcomponents is taken into consideration. Research on the dynamic interaction of such systems has long been studied; significant progress was achieved thanks to field observations, structural health monitoring data and strong ground motion records obtained during major seismic events [1-3]. Moreover, analytical solutions and advanced numerical simulation models have been developed [4-6] with particular emphasis on the response of pile foundations [7-14], lateral spread of soil [15], lateral excitation in layered deposits [16-19] and soil liquefaction [20-23]. Experimental results, involving complex bridge structures and pile foundations, are also currently available [24-26] while significant research effort was shed light in the nature of kinematic soil-pile interaction [27-31].

Despite the extensive research, an issue that has not yet been thoroughly studied is the additional rocking that is imposed to the bridge superstructure due to earthquake-induced pile bending. In particular, it is well known that the presence of a pile foundation modifies the amplitude and frequency content of the incoming seismic waves, thus resulting into a “Foundation Input Motion” that is different from the free field one, while analytical expressions have been proposed for computing the aforementioned additional pile head rotation [32]; still, however, there is no comprehensive approach available for practical purposes that can simultaneously account for the translational and rotational component of seismic acceleration neither has this effect ever been quantified for the case of realistic structures. This approach would be of particular use, especially for bridges supported on cast-in-drilled-hole (single) pile foundations, a common design alternative, primarily in the U.S.

Along these lines, the scope of this paper is to:

(a) present a comprehensive methodology and computational framework for considering the translational and rotational excitation of the soil-pile-superstructure system, and

(b) highlight those cases where ignoring the rotational component of seismic excitation, the displacement demand imposed to the superstructure may be significantly underestimated.

The fundamental concepts of the approach as well as the parametric analysis scheme and the subsequent results, are presented in the following.

2 PROBLEM VARIABLE DEFINITION

The study of pile-induced pier rocking requires definition of different variables that can be grouped into four major categories, specifically relating to the soil, the foundation, the superstructure (in terms of both material and geometry) and seismic excitation (primarily defined in terms of frequency). It is noted that some of these variables are dimensional, while others are dimensionless, as summarized in Table 1 below. The fundamental dimensions of the variables involved are mass (M), time (T) and length (L).

Variable category	Variable	Dimensions	Fundamental dimensions	Kind of variable
Super-structure	Modulus of elasticity, E	kN/m^2	M,T, L	Dimensional
	Mass of superstructure, m	t (of mass)	M	Dimensional
	Pier height, h	M	L	Dimensional
	Pier density, ρ	kg/m^3	M, L	Dimensional
	Pier material damping, ζ	%	-	Dimensionless
Founda-tion (RC CIDH pile)	Modulus of elasticity, E_p	kN/m^2	M,T, L	Dimensional
	Pile diameter, d_p	M	L	Dimensional
	Poisson's ratio, v_p	-	-	Dimensionless
	Density, ρ_p	kg/m^3	M, L	Dimensional
	Material damping, ζ_p	%	-	Dimensionless
Soil	Modulus of elasticity, E_s	kN/m^2	M,T, L	Dimensional
	Stratum thickness, H_s	M	L	Dimensional
	Poisson's ratio, v_s	-	-	Dimensionless
	Density, ρ_s	kg/m^3	M, L	Dimensional
	Material damping, ζ_s	%	-	Dimensionless
Excitation	Cyclic frequency, ω	T	T	Dimensional

Table 1: Problem Variables

Due to the fact the seismic excitation is assumed sinusoidal, its cyclic frequency, ω, is used to characterize the excitation; the amplitude of the pulse is irrelevant since the analysis is linear elastic. Based on the above, a total number of 11 dimensional and 5 dimensionless variables is derived, in particular: E, m, h, ρ, E_p, d_p, ρ_p, E_s, H_s, ρ_s, ω and ζ, v_p, ζ_p, v_s, ζ_s respectively. Eventually, it is only 9 out of 11 different dimensional variables that remain as part of the problem, due to the fact that the (concrete) material between the superstructure and the pile is identical, hence, it can be assumed that $E=E_p$.

In the framework of the parametric analysis conducted herein, the modulus of Elasticity of concrete was taken equal to E_p=29GPa, thus corresponding to a concrete strength of f_c=20MPa, while the stiffness of the soil, as expressed by the shear wave velocity V_s, was assumed to parametrically vary between 100 and 250 m/sec. It is recalled out that circular frequency of the soil ω_{soil} is expressed as:

$$\omega_{soil} = 2\pi V_s/4H \tag{1}$$

where H the (uniform) soil stratum thickness, as defined in Table 1.

3 ANALYSIS OUTLINE

Rocking of a bridge supported on CIDH piles occurs along both longitudinal and transverse direction, however the latter is more critical the lower level of redundancy. It is therefore the transverse response of the bridge that is studied herein, assuming a 4-degree-of-freedom oscillator, with translational and rotational DOFs at the locations of its two concentrated masses, one at the top of the pier and one at its bottom as it is seen in Figure 1 (m, J_m, m_p and J_{mp} being the mass and moments of inertia of the pier top and base respectively, h denotes the oscillator height, e the eccentricity that eliminates the conjugation stiffness term K_{xr}, E the pier modulus of elasticity, ζ the damping ratio, b the pier diameter, and D the pile diameter. The spring properties required for providing the translational (K_{xx}) and rotational (K_{rr})

stiffness of the soil-pile system are derived using appropriate equations from the literature [32].

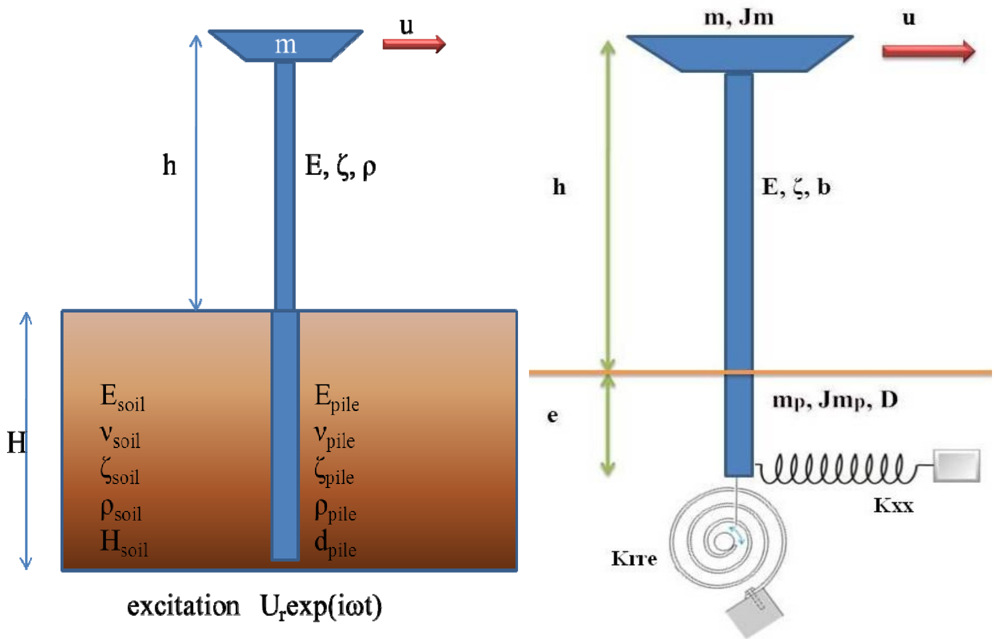

Figure 1: Overview of the system studied and the related variables and degrees of freedom.

The so-called kinematic response factors, I_u for the translational and I_θ for the rocking seismic component which express the frequency-dependent translational and rotational amplification of motion compared to that of the free-field are also derived based on expressions proposed in the literature [32]. As the pile of diameter *d* is assumed to be excited by vertically propagating, harmonic S-waves with amplitude U_r and frequency ω, the rotation that is eventually imposed to the superstructure can be simply derived as [18]:

$$\theta(t) = 2I_\theta(\omega)U_{ff}(z=0)/d_p \tag{2}$$

for each time step, *t* of the incoming harmonic motion. The overall process to derive the rotational excitation time history can be summarized in the following successive steps described below.

1st step: Assessment of the relevant problem variables (related to the soil, foundation, superstructure and seismic excitation) as it described in Table 1.

2nd step: Calculation of the pile moment of inertia and mass:

$$I_p = \pi d_p{}^4/64$$
$$m_p = 25\left(\frac{\pi d_p{}^2}{4}\right)L/g \tag{3}$$

where L: length of the pile (assumed equal to the thickness of the stratum, H_s)
g: acceleration of gravity

3rd step: Assessment of soil modulus of elasticity E_s and shear wave velocity V_s

4th step: Calculation of the free field motion at the surface (z=0) for each frequency ω in the time domain [33]:

$$U_{ff} = U_r e^{i\omega t} = \left(\frac{U_g}{\cos\frac{\omega}{V_s}L}\right)\cos\left(\frac{\omega}{V_s^*}z\right)e^{i\omega t} \tag{4}$$

where U_{ff}: free-field motion amplitude
U_g: amplitude of the bedrock motion (i.e., base of the soil stratum)
z: depth from the ground surface

5th step: Calculation of the foundation impedance functions assuming a Beam on Dynamic Winkler Foundation [34]:

$$S_x = k_x(\omega) + i\omega c_x(\omega)$$

where $k_x = \delta E_s$, $\delta=1.67(\frac{E_p}{E_s})^{-0.053}$

$$c_x(\omega) = 2a_0^{-1/4}\rho_s V_s d_p\left[1+\left(\frac{V_{La}}{V_s}\right)^{5/4}\right] + 2\zeta_s\frac{k_x}{\omega} \tag{5}$$

$a_0 = \frac{\omega d_p}{V_s}$ and $V_{La} = \frac{3.4V_s}{\pi(1-v)}$

6th step: Calculation of the translational, kinematic interaction factor for a free head single pile [35]:

$$I_u(\omega) = \frac{k_x+i\omega c_x}{E_pI_p(q^4+4\lambda^4)-m_p\omega^2}\cdot\left[1+\frac{1}{2}\left(\frac{\omega}{V_s\lambda}\right)^2\right] \tag{6}$$

where: $q = \frac{\omega}{V_s}$ and $\lambda = \left(\frac{k_x+i\omega c_x}{4E_pI_p}\right)^{1/4}$ (7)

7th step: Calculation of the rocking, kinematic interaction factor for a free head single pile [35]:

$$I_\theta(\omega) = \frac{k_x+i\omega c_x}{E_pI_p(q^4+4\lambda^4)-m_p\omega^2}\cdot\left(\frac{\omega}{V_s}\right)^2\cdot\frac{d_p}{\lambda} \tag{8}$$

8th step: Calculation of the imposed rotation [18]:

$$\theta(t) = \frac{I_\theta(\omega)U_{ff}(t,z=0)}{d_p/2} \tag{9}$$

9th step: Calculation of the translational (K_{xx}) and rotational (K_{rr}) stiffness properties of the soil-pile system springs according to Maravas et al. [32]. It is noted that the discrete sign e which refers to the dynamic impedance terms after appropriate spring-pile head eccentricity and stiffness decoupling:

$$K_{xx} = 4E_pI_p\lambda^3$$
$$K_{xr} = 2E_pI_p\lambda^2$$
$$K_{rr} = 2E_pI_p\lambda$$
$$K_{xxe} = K_{xx} \quad (10)$$
$$K_{xre} = 0$$
$$K_{rre} = K_{rr} - 2K_{xr} \cdot e + K_{xx} \cdot e^2$$
$$e = \frac{K_{xr}}{K_{xx}} = \frac{1}{2\lambda}$$

where K_{xx}: translational stiffness of the soil-pile system along the x direction
K_{xr}: coupled rotational-translational stiffness of the soil-pile system
K_{rr}: rotational-translational stiffness of the soil-pile system
K_{rre}: uncoupled rotational stiffness of the soil-pile system
e: spring-pile head eccentricity

Having defined the translational and rotational foundation input motion and the dynamic properties of the flexible supported bridge structure along the transverse direction, the dynamic response of the 4DOF system can be analytically defined. The equation of motion has the general form:

$$M \cdot \ddot{u} + C \cdot \dot{u} + K \cdot u = p(t) \quad (11)$$

while the initial conditions can be set cast as, $u(0) = 0$ and $\dot{u}(0) = 0$. In this case, the matrix of the displacements relative to the moving base takes the following form, $u = [u\ \theta\ u_p\ \theta_p]^T$ where (u, θ) represent the displacement and rotation of the deck mass and (u_p, θ_p) of the foundation mass respectively. The solution of the equation of motion (11) is given from the well-known Duhamel's integral:

$$q_n(t) = \frac{1}{M_n\omega_{dn}}\int_0^t \tilde{p}_n(\tau) \cdot e^{-\zeta\omega_n(t-\tau)}\, sin\omega_{dn}(t-\tau)d\tau, \qquad n = 1,2,3,4 \quad (12)$$

Thus, the ultimate, geometric coordinates of the vector u(t) can be written as:

$$u(t) = \sum_{n=1}^{N} \varphi_n \cdot q_n(t) \quad (13)$$

Assuming that due to the elastic response of the system considered, the damping matrix, C, equals to $C=2m\zeta\omega$, the mass matrix is quadratic and given as:

$$M = \begin{bmatrix} m & 0 & 0 & 0 \\ 0 & Jm & 0 & 0 \\ 0 & 0 & m_p & 0 \\ 0 & 0 & 0 & Jm_p \end{bmatrix} \quad (14)$$

The computation of the stiffness matrix requires the calculation of its individual values, i.e. the generalized stiffness factors of the system, $K_s = [k_{ij}]$, where i,j=1,2,3,4. The ultimate stiffness matrix diversifies for each excitation frequency, as the foundation impedance is frequency dependent. In general however, stiffness matrix maybe written in the following form:

$$K=\begin{bmatrix} \frac{12EI}{h^3} & \frac{6EI}{h^2} & \frac{-12EI}{h^3} & \frac{6EI}{h^2} \\ \frac{6EI}{h^2} & \frac{4EI}{h} & \frac{-6EI}{h^2} & \frac{2EI}{h} \\ \frac{-12EI}{h^3} & \frac{-6EI}{h^2} & \frac{12EI}{h^3}+K_{xx}(\omega) & \frac{-6EI}{h^2} \\ \frac{6EI}{h^2} & \frac{2EI}{h} & \frac{-6EI}{h^2} & \frac{4EI}{h}+K_{rre}(\omega) \end{bmatrix} \quad (15)$$

It is noted that the external load vector *p(t)* considers both the translational and rotational component of seismic excitation, which for the purpose of linear elastic analysis, can be decoupled and then the individual structural response due to base translation and rotation be superimposed. For instance, the translational seismic component is:

$$-m\cdot\delta\cdot\ddot{u}_0(t) \quad (16)$$

where m is the oscillator (deck) mass, δ the vector of rigid body displacements and $u_0(t)$ the base excitation time history, which is in turn equal to the second derivative of the free-field ground displacement expressed by equation (4). The vector of rigid body displacements, in this case, receives the following form for a unit displacement of the oscillator base.

$$\delta=[1\ 0\ 1\ 0]^T. \quad (17)$$

Similarly to the above, the rocking excitation component is induced separately in the right-hand side of the equation of motion, while the rocking acceleration results from the double derivative of the rotation, with respect to time, as it has been calculated from equation (9) above. For a unit rotation of the pier base, vector δ takes the following form:

$$\delta=[h\ 1\ 0\ 1]^T. \quad (18)$$

and the two equations of motion for the two distinct translational and rotational base excitation mechanism can be written as:

$$M\cdot\ddot{u}(t)+2m\zeta\omega\cdot\dot{u}(t)+K\cdot u(t)=-M\cdot\delta\cdot\ddot{u}_{ff}(t,z=0)\cdot I_u$$
$$M\cdot\ddot{u}(t)+2m\zeta\omega\cdot\dot{u}(t)+K\cdot u(t)=-M\cdot\delta\cdot\ddot{\theta}(t)=-M\cdot\delta\cdot I_\varphi(\omega)\cdot\frac{\ddot{u}_{ff}(t,z=0)}{d_p/2} \quad (19)$$

4 COMPUTATIONAL FRAMEWORK

The above analysis steps are implemented computationally through a specifically developed graphical MatLab environment, a sample of which is illustrated in Figure 2. The user defines the properties of the pile (i.e., modulus of elasticity, diameter, length, density and damping ratio), soil data (expressed in terms of the ratio E_p/E_s, Poisson's ratio, damping, density and shear waves velocity) and the desired excitation frequency range. In turn the program returns the variation of the translational and rotational kinematic interaction factors (I_u and $I_φ$) with the dimensionless frequency $ωd/V_s$. The translational and rotational excitation time histories are also computed to be used as the foundation input motion to the (spring-supported) superstructure.

Following the computation of the kinematic interaction factors, the seismic response of the 4DOF system representing the pier-foundation-soil is derived for each excitation frequency (Figure 3). The fundamental period of both the fixed and flexibly supported structure is also computed and the maximum response quantities are derived and stored. The resulting displacement maxima are compared to the deck displacements that would have been derived if the conventional approach (i.e., translation-only foundation input motion) was followed. Results are plotted in appropriate diagrams of dimensionless quantities as will be described below.

5 ANALYSIS RESULTS

Having established the analytical and computational framework, a detailed parametric analysis scheme was formed and the effect of rotational excitation was investigated. In particular, the parametric study was performed by modifying four analysis parameters, related to the:

- *Soil stiffness*: of the upper soil layer (taken uniform along the pile height) expressed in terms of shear wave velocity V_s which was taken equal to 100m/sec or 250m/sec, thus corresponding to very soft and medium soft soil conditions respectively. It is noted that the above variation of soil stiffness corresponds to a modulus of Elasticity of the soil E_s, between 2.9MPa (very soft clay) to 29MPa (moderately soft clay), in other words, to a dimensionless variable Ep/Es lying within the range of 100-1000. (*2 cases*)
- *Pier Height*: taken equal to 5m, 7.5m, 12.5m and 20m leading, after appropriate mass modification, to fundamental periods in the transverse direction between the range 0.4-2.5sec. (*4 cases*)
- *Frequency of excitation*: harmonic pulses were used ranging between (0.1-2.0sec), that is, having frequencies 0.5,1,2,3,4,5,6,7,8,9 and 10Hz. (*11 cases*)
- *Excitation component*: the flexibly supported pier was excited in the transverse direction by (a) the translational, (b) the rotational and (c) the combined translational and rotational components of the foundation input motion. (*3 cases*)

The analysis of the above 2 x 4 x 11 x 3 = 264 parametric analysis were plotted in a normalized ratio, versus the dimensionless frequency term ($ωD/V_s$). The ratio adopted for illustrating the relative effect of the different (i.e., translational and rotational components of the kinematically modified seismic wave field) was:

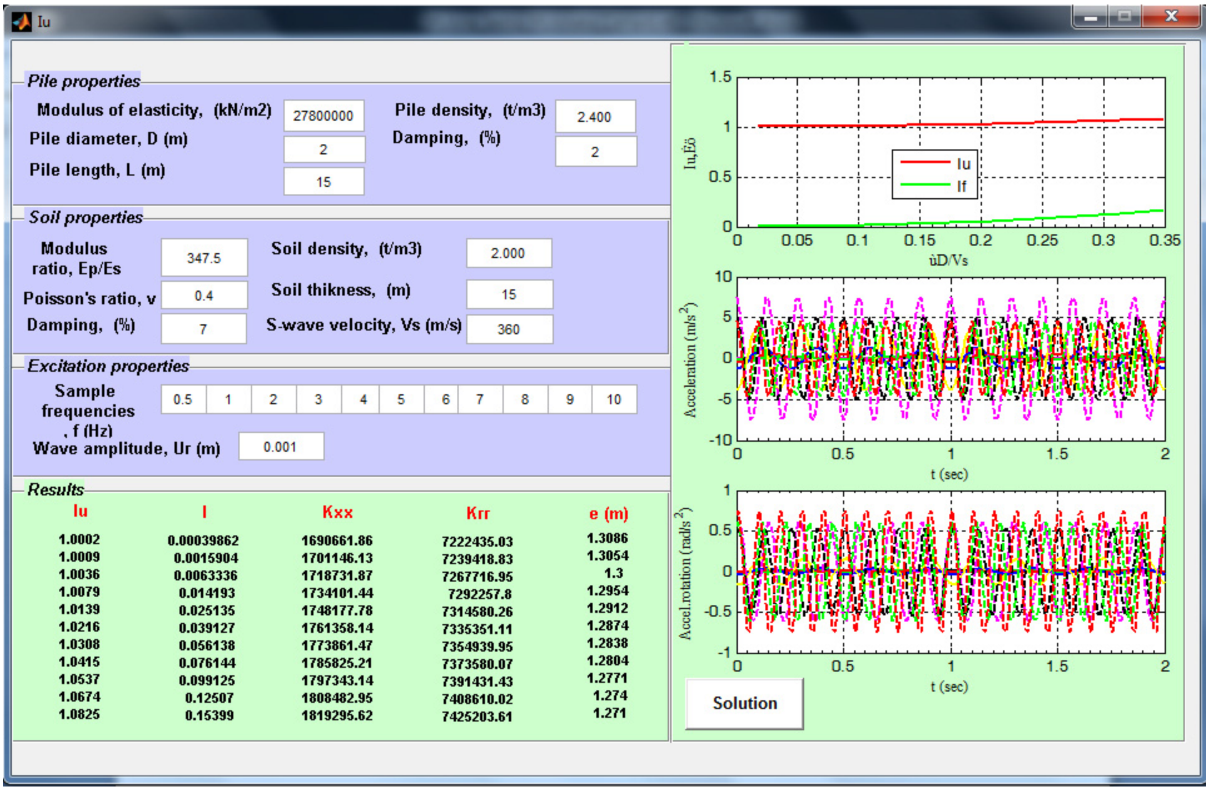

Figure 2 : MatLab application for computing the translational and rotational interaction factors (I_u, I_φ) and the subsequent uncoupled dynamic impedance matrix terms (K_{xx}, K_{rre}).

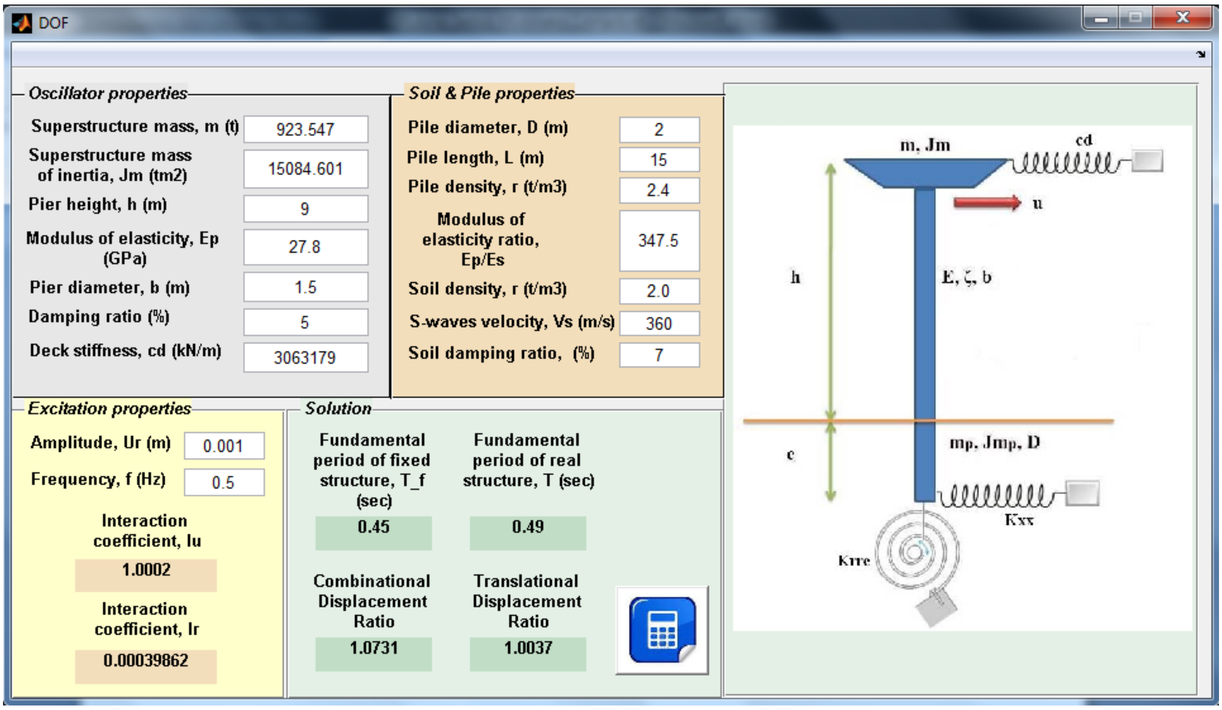

Figure 3 : MatLab-based solution of the soil-pile-superstructure system under combined translational and rotational excitation.

$$I_u = \frac{u_{max,\ FIM,u}(t)}{u_{max,ff}(t)} \tag{20}$$

expressing the maximum displacement in time of the deck (pier top) when the kinematic interaction between the soil and the pile is taken into consideration and the deck is excited exclusively by the translational component of the foundation input motion (F.I.M.), over the maximum displacement in time of the deck due to free field base excitation (a ratio essentially identical to the standard kinematic interaction factor I_u of equation (6)), and

$$I_{u+\theta} = \frac{u_{max,\ FIM,u+\theta}(t)}{u_{max,ff}(t)} \tag{21}$$

expressing the maximum displacement in time of the deck (pier top) when the kinematic interaction between the soil and the pile is taken into consideration and the deck is excited by both the translational and the rotational component of the foundation input motion (F.I.M.), over the maximum displacement in time of the deck due to free field base excitation.

The absolute values of pier top displacements are also plotted so that the relative contribution of each excitation component can be assessed. The results are discussed in the following.

5.1 Very Soft Soil, V_s=100m/s

The first set of figures (Figures 4,6,8,10) illustrates the kinematic interaction factors for translational or combined translational base excitation of the bridge pier under study, together with the corresponding absolute displacements of the deck (Figures 5,7,9,11) for decreasing pier heights (i.e., 20m, 12.5m, 7.5m, 5.0m) and a constant, uniform soil profile of V_s=100m/sec. A first observation is that the kinematic interaction factors are significantly higher in the case of combined translational and rotational foundation input motion compared to the conventional approach of translational base excitation only, for the whole dimensionless frequency range $0<\omega D/V_s<1.4$. These factors, $I_{u+\theta}$, can exceed the value of 10, at specific frequencies, for the case of a very flexible pier (h = 20m) and decay to 6.5 for the case of stiffer bridge structures (h = 5m) in contrast to the translational kinematic interaction factor which does not exceed 2 in the entire frequency range. This situation is also confirmed by the absolute displacement depicted in Figures 5, 7, 9 and 11 where it is clear that the deck displacements are dominated by the base rotational excitation.

It has to be noted that such a significant effect of rotational excitation cannot be generalized, as the case studied clearly represents the extreme condition where the structure is excited at a single and critical frequency (in contrast to the broad frequency content of an actual earthquake loading), in the transverse direction which is more prone to base rotations due to limited redundancy, while the pier height is significant (h = 20m) and at the same time, the soil is very soft (V_s = 100 m/sec). One could even argue that in such cases of soft soil profiles, a single CIDH pile wouldn't be a desirable foundation type anyway. As a general trend though, the aforementioned diagrams are

indicative of the fact that the earthquake –induced head rotation of a (single) CIDH pile imposes, apart from the inertial component, an additional rigid body motion to the pier-deck superstructure which has a direct effect on the pier deck displacements as it is proportional to the pier height (i.e., the displacement vector $\delta_{rocking}=[h\ 1\ 0\ 1]^T$). As a result, the importance of rotational foundation input motion in piles casted in soft soils is non-negligible under certain circumstances and has to be further studied.

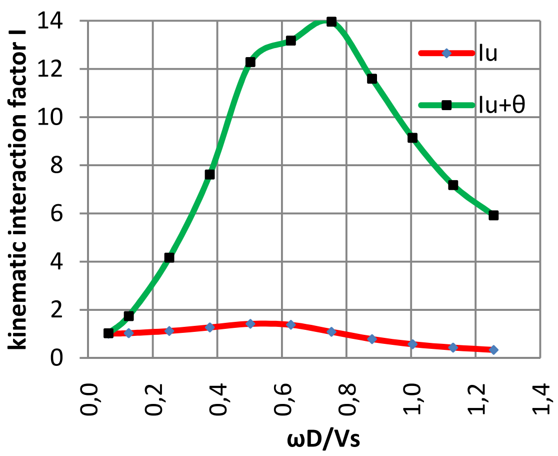

Figure 4: Displacements ratio as function to dimensionless frequency for V_s=100m/s, h=20m.

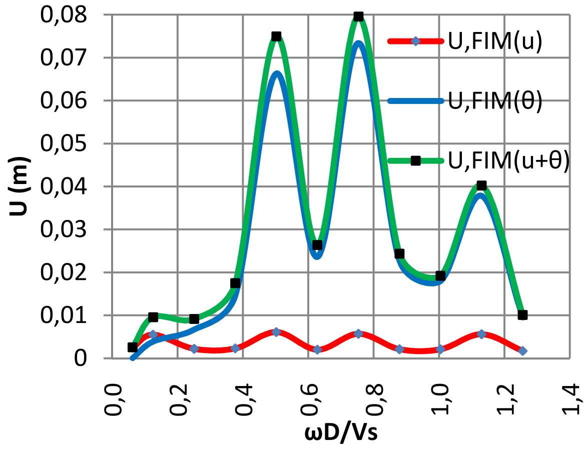

Figure 5: Absolute displacements as a function of dimensionless frequency for V_s=100m/s, h=20m.

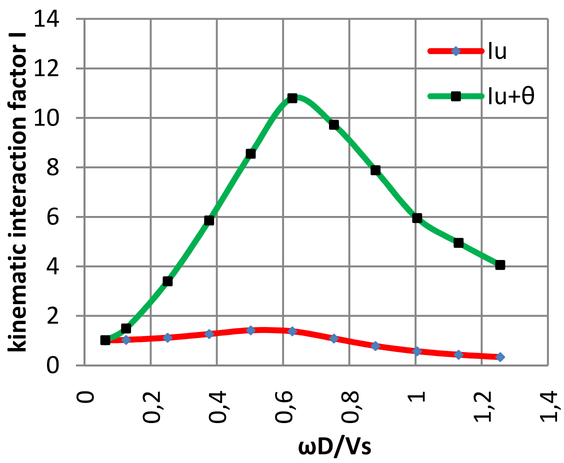

Figure 6: Displacements ratio as a function of dimensionless frequency for V_s=100m/s, h=12.5m.

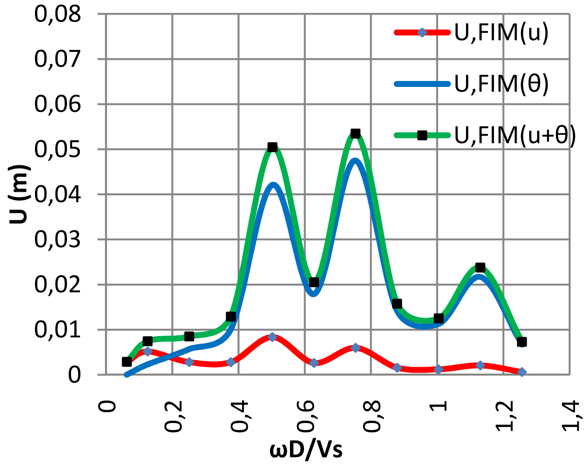

Figure 7: Absolute displacements as a function of dimensionless frequency for V_s=100m/s, h=12.5m.

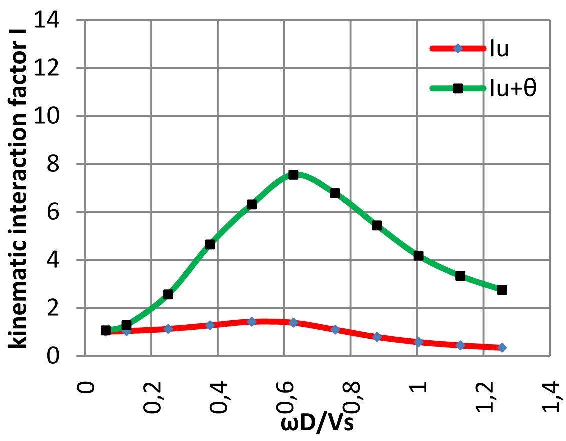

Figure 8: Displacements ratio as a function of dimensionless frequency for V_s=100m/s, h=7.5m.

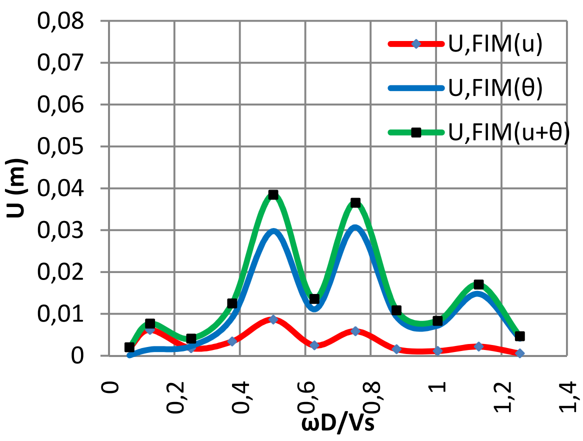

Figure 9: Absolute displacements as a function of dimensionless frequency for V_s=100m/s, h=7.5m.

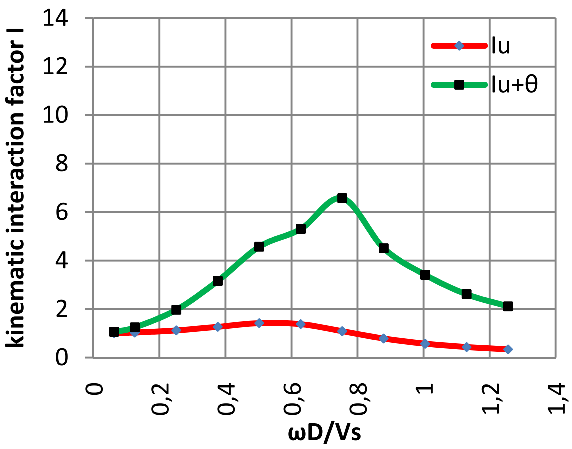

Figure 10: Displacements ratio as a function of dimensionless frequency for V_s=100m/s, h=5m.

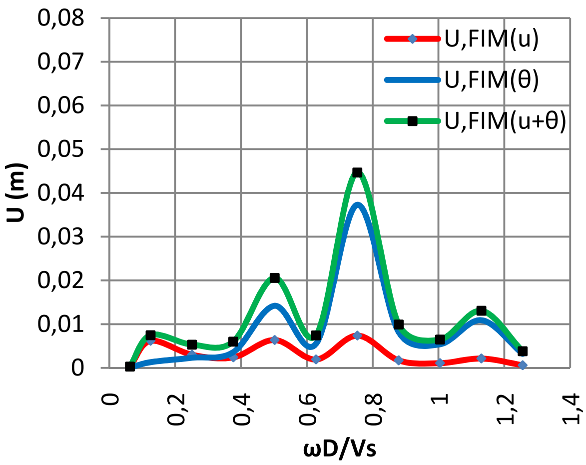

Figure 11: Absolute displacements as a function of dimensionless frequency for V_s=100m/s, h=5m.

5.2 Medium Soft Soil, Vs=250m/s

Similarly to the case of soft soil, the same set of parametric analyses were conducted for the case of medium soil, characterized by a shear wave velocity of 250m/sec. The corresponding variation of the kinematic interaction factors with and without the effect of rotational excitation as well as the variation of the absolute bridge deck displacements with the dimensionless frequency is illustrated in Figures 12,14,16,18 and 13,15,17,19 respectively. Again, the impact of rotational excitation on the overall pier top displacements is clearly visible: the combined kinematic interaction factor $I_{u+\theta}$ ranges from 3.8 to 8, always being higher than the conventional I_u which does not exceed 1.1 along the entire frequency range and for all pier heights. On the other hand, as anticipated, this effect of the rotational component of foundation input motion, although significant, is lower than the one observed for the case of soft soil, and as such it is expected to be smaller for stiffer soils as well. The overall (i.e., rigid body and inertial) mechanism described previously, in which the rotational excitation affects the transverse bridge deck displacements, is again confirmed.

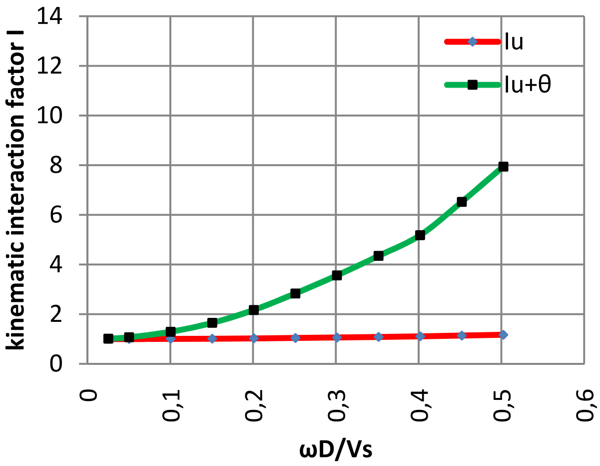

Figure 12: Displacements ratio as a function of dimensionless frequency for V_s=250m/s, h=20m.

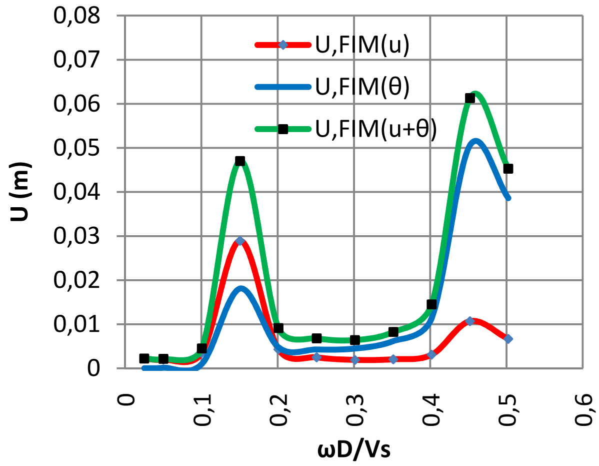

Figure 13: Absolute displacements as a function of dimensionless frequency for V_s=250m/s, h=20m.

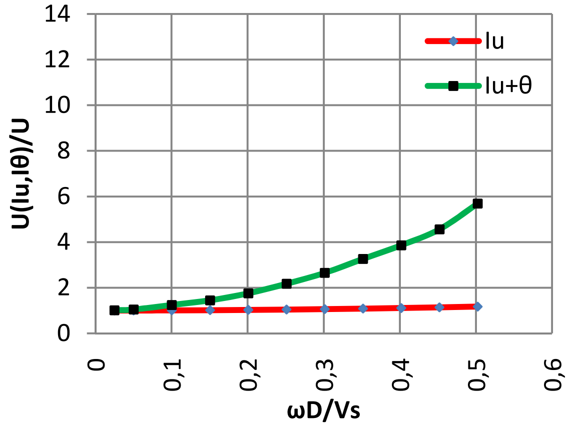

Figure 14: Displacements ratio as a function of dimensionless frequency for V_s=250m/s, h=12.5m

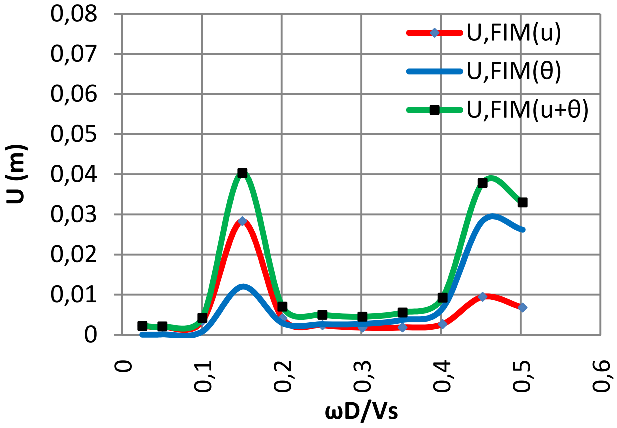

Figure 15: Absolute displacements as a function of dimensionless frequency for V_s=250m/s, h=12.5m.

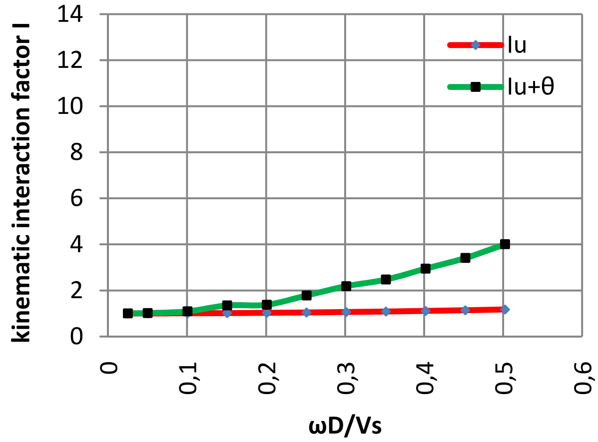

Figure 16: Displacements ratio as a function of dimensionless frequency for V_s=250m/s, h=7.5m

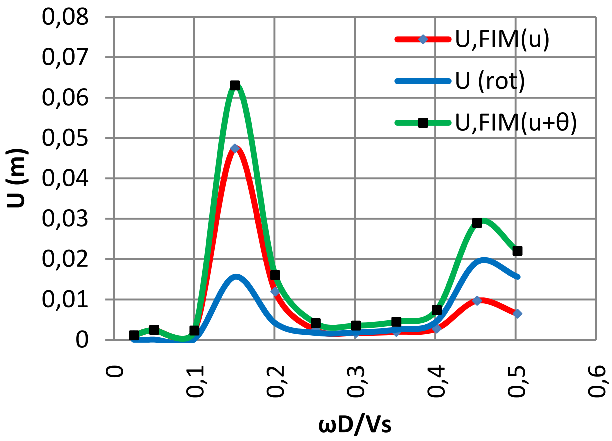

Figure 17: Absolute displacements as a function of dimensionless frequency for V_s=250m/s, h=7.5m.

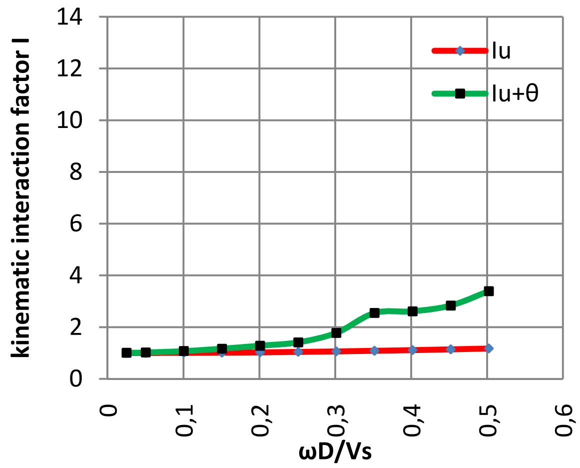

Figure 18: Displacements ratio as a function of dimensionless frequency for V_s=250m/s, h=5m

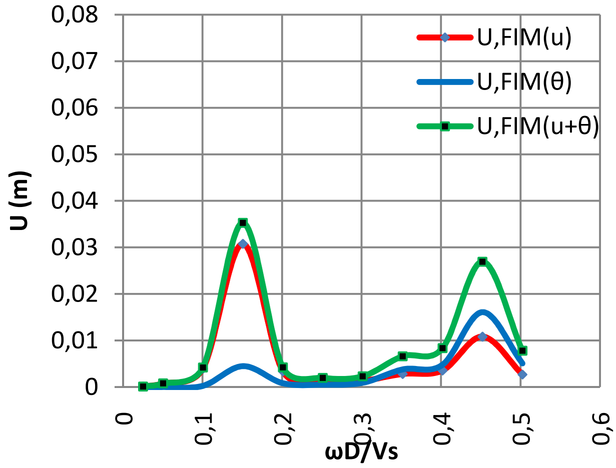

Figure 19: Absolute displacements as a function of dimensionless frequency for V_s=250m/s, h=5m.

6 CONCLUSIONS

A comprehensive analytical study was presented for the rocking effects of seismic motion in reinforced concrete bridges supported on cast-in-drilled-hole piles in homogeneous soil deposits. The study focuses on a 4-degree-of-freedom spring-supported pier subjected to simultaneous translational and rotational foundation input motion. The analytical approach is implemented into a comprehensive computational framework, programmed in Matlab environment, with the use of which a set of linear, elastic analyses was performed for different soil types, dynamic characteristics of the bridge and frequencies of harmonic excitations.

Through the above set of parametric analyses it is concluded that the combined consideration of the translational and rotational seismic components produced by the kinematic response of the pile foundation to vertically propagating S-waves, may reveal specific combinations of excitation frequency content and dynamic characteristics where the additional transverse deck displacements induced by the base rotation can indeed dominate the system response. It was also confirmed that the effect of the rotational component is higher for cases of soft soil profiles and flexible, tall piers.

Further research is certainly needed for quantifying the importance of the rotational excitation, in case of more complex seismic motions, structural systems and foundation types.

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