Paper 0123456789

Proceedings of the Fourteenth International Conference on
Civil, Structural and Environmental Engineering Computing,
B.H.V. Topping and P. Iványi, (Editors),
Civil-Comp Press, Stirlingshire, Scotland

Seismic Assessment of Pile-supported Bridges Considering the Rotational Excitation of Earthquake Ground Motion

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Abstract

The scope of this paper is to investigate the response of RC bridges, founded on pile groups in a liquefiable, layered soil, under simultaneous earthquake-induced translational and rocking excitation. The rocking excitation results from pile bending, under vertically propagating seismic S-waves, which, in turn, depends on the relative flexibility between the pile group and the surrounding soil, a phenomenon known as "kinematic interaction". Typically, the rotational component of the seismic excitation is not taken into consideration in the design, neither is it prescribed in any of the modern seismic codes. Based on the previous research of a pile-induced rocking in CIDH pile supported bridges [1], an effort is made to extend the above findings considering soil liquefaction and a pile group foundation. For this reason, the lateral response of a typical bridge in the Egnatia Highway Greece, is analytically studied for various acceleration scenarios. The response of the pile cap, in terms of displacement and rotation time histories, as a result of the kinematic interaction analysis of the foundation system, is the total foundation input motion (F.I.M.) of the superstructure. The resulting displacement demand of the coupled load is then compared to the displacements that would develop by ignoring the rotational component of the excitation. From the set of parametric analyses conducted, it is concluded that ignoring the rocking component of the input motion, transverse deck displacements may differ significantly, based on the dynamic characteristics of the foundation and the superstructure.

Keywords: rotational excitation, pile foundation, kinematic interaction, soil – structure interaction, rocking, liquefaction, translational excitation.

1 Introduction

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 sub-components is taken into consideration. Research on the dynamic interaction of bridge 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 [2], [3], [4]. Moreover, analytical solutions and advanced numerical simulation models have been developed [5], [6], [7] with particular emphasis on the response of pile foundations [8], [9], [10], [11], [12], [13], [14], [15], lateral spread of soil [16], lateral excitation in layered deposits [17], [18], [19], [20] and soil liquefaction [21], [22], [23], [24]. Experimental results, involving complex bridge structures and pile foundations, are also currently available [25], [26], [27] while significant research effort was shed light in the nature of kinematic soil-pile interaction [28], [29], [30], [31], [32].

Despite the extensive research highlighting the significant impact of soil-pile-superstructure interaction in bridge response, an issue that has not yet been thoroughly studied is the additional rocking excitation 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 [33]; 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. Given the paucity of research on the above phenomenon, a study was undertaken from the authors to provide data to address the rocking response of bridges supported on cast-in-drilled-hole (single) pile foundations, a common design alternative, primarily in the U.S [1]. Results have shown 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.

The aim of this paper is to extend the above findings for the case of a pile group within a potentially liquefiable soil. This is an interesting case where two counteracting phenomena take place simultaneously: the rigid cap of the pile group reduces the rotational response of the foundation, hence the rotational input to the superstructure, while soil liquefaction induces significant relative deformations along the pile length thus increasing the rotation of the pile cap.

Along these lines, the aim of this paper is to assess (a) the response of the soil-foundation system due to the rotational component of earthquake ground motion, compared to the conventional translational component, (b) the resulting Foundation Input Motion from the composition of the two distinct (translational and rotational) components and (c) the overall response of the bridge superstructure. The fundamental concepts of the approach, as well as the parametric analysis scheme and the subsequent results, are presented in the following.

2 Overview of the soil – foundation – superstructure system

2.1 Bridge properties and system modeling

A typical ordinary standard bridge, representative of an overpass of Egnatia Highway at northern Greece, is the structure adopted to be studied herein. The bridge under investigation is a three-span, straight structure with a total length of 99.0m. Two single, circular columns of 2.0m diameter and 7.94m and 9.34m height are monolithically connected with the single box-girder superstructure which has a central span of 45.0m. The box section is pre-stressed with a total height of 2.0m and a slope of 7.0% along the longitudinal direction of the bridge. The wall abutments of 5.63m and 5.71m height support the deck through elastomeric bearings with dimensions 350x450x136. The material used for the pre-stressed deck and the reinforced piers is concrete B35 while B25 has been used for the abutments and the foundation. The reinforcement and pre-stressing steel corresponds to Bst500s and 1570/1770, respectively.

Foundation consists of a pile group embedded into the bedrock. The dominant friction bearing capacity of the piles necessitates a rather large pile length of 37.0m. According to design requirements, each pier is founded on a 4x4 pile group of 1.3m diameter and 3.0m spacing. The pile cap is 12x12m with 1.5m thickness. The asymmetry of the bridge, due to the different pier heights, as well as the torsional nature of the first and second mode of the structural system, necessitates the examination of both piers.

A 3-D finite element model of the bridge is developed using the SAP2000 commercial software [34]. The bridge superstructure as well as the piers and the piles are modeled using frame elements as shown in Figure 1, while shell elements are used for the pile cap. The joints connecting the deck to the pier columns are monolithic and modeled as rigid elements. Determination of the properties of the springs representing the soil-pile system is performed through the lateral soil resistance-deflection (p-y) relationship presented in the following sections.

2.2 Soil profile and selected ground motions

The soil profile adopted in this study consists of 6 horizontal layers of silty sands and non-plastic silts (Table 1). Bedrock is located at a depth of 23m while the water table is assumed at the soil surface. One-dimensional site response analysis is performed through the above soil conditions considering the liquefaction-induced deformations in each soil layer. For this reason, a set of 12 earthquake records with horizontal PGA ranging between 0.16g and 0.36g was selected from the Pacific Earthquake Engineering Research (PEER) strong motion database of the University of California, Berkeley [35]. All selected earthquakes have a magnitude greater than 5.5 and rich frequency content in order to excite several modes of the soil profile, the foundation system and the superstructure. Details of the selected ground motions are given in Table 2.

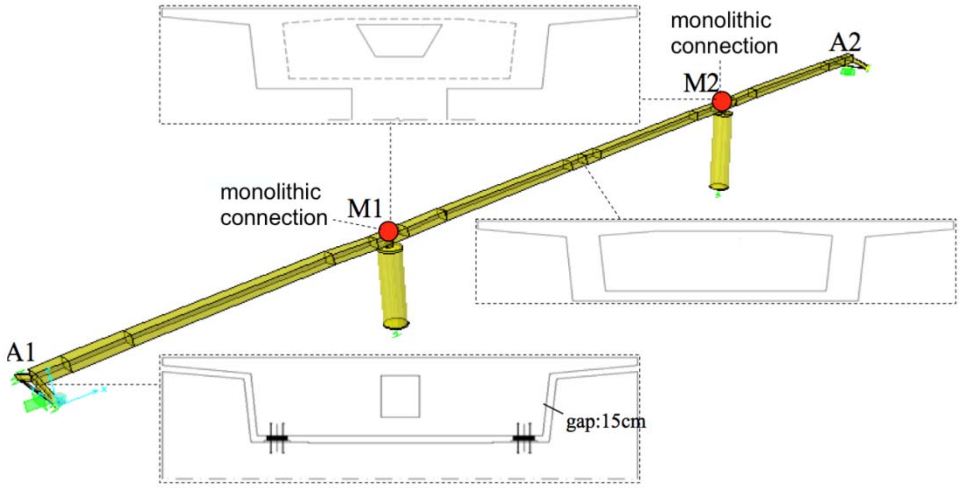

Figure 1: Overpass of Egnatia Highway under study.

Depth (m)	Vs (m/s)	ρ (t/m^3)	E (MPa)	Dr (%)
0 - 3	179	1.9	6.0	60
3 - 7	165	1.9	6.0	60
7 - 11	198	1.9	6.0	60
11 - 15	228	1.9	6.0	60
15 - 19	215	1.9	6.0	60
19 - 23	255	1.9	6.0	60
> 23 (bedrock)	500	2.1	6.0	60

Table 1: Overview of the soil profile.

3 Analysis outline

Rocking of pile-supported bridges in liquefiable soil occurs along both the longitudinal and the transverse direction; however the latter is deemed more critical given the relatively lower level of redundancy. It is therefore the transverse response of the bridge that is studied herein. The twelve acceleration time histories [1] selected from PEER Strong Motion database are utilized to excite the soil – pile group – superstructure system described in the prior sections. The analysis follows the principle of the substructure method, studying separately the foundation and the superstructure. The overall process to derive the superstructure response due to the combined translational and rotational excitation can be summarized in the following successive steps described below, appropriately modified to account for the potential liquefaction.

ID	Earthquake Event	Date	M_w	Station	Fault rupture (km)	PGA (g)
1	Loma Prieta	18/10/1989	7.1	1652 Anderson Dam (Downstream)	21.4	0.244
2	Northridge	17/1/1994	6.7	24389 LA-Century City CC North	25.7	0.222
3	Kern County	21/7/1952	7.7	1095 Taft Lincoln School	41	0.178
4	Cape Mendocino	25/4/1992	7.1	89509 Eureka-Myrtle & West	44.6	0.178
5	San Fernando	9/2/1971	6.6	126 Lake Hughes #4	24.2	0.192
6	Landers	6/28/1992	7.4	23559 Barstow	36.1	0.132
7	Imperial Valley	15/10/1979	6.9	6604 Cerro Prieto	26.5	0.169
8	Taiwan	11/14/1986	7.8	29 SMART1 M07	39.0	0.160
9	Superstitn Hills	11/24/1987	6.6	5052 Plaster City	21.0	0.121
10	Northridge	17/1/1994	6.7	24303 LA-Hollywood Stor	25.5	0.358
11	Loma Prieta	18/10/1989	7.1	1678 Golden Gate Bridge	85.1	0.233
12	Livermore	27/1/1980	5.5	57T02 Livermore-Morgan Terr	8.0	0.198

Table 2: Selected earthquake ground motions.

1st step: 1-Dimensional nonlinear site response analysis using the selected acceleration records considering the liquefaction induced deformations in each soil layer.
2nd step: Excitation of the soil-foundation system with the (depth-dependent) displacement time histories that have aroused from the above site response analysis, to predict, through kinematic interaction, the pile cap response expressed in terms of translational and rotational acceleration time histories.
3rd step: Excitation of the flexibly-supported superstructure, with the above Foundation Input Motion.

4th step: Comparison of the deck displacements, along the transverse direction, resulting from the assumption of the conventional (i.e. translational only) and the combined (translational - rotational) excitation approach.

3.1 Seismic response of multi-layer, liquefiable soil

Site response analyses are performed with Cyclic 1D [34, 35] software, a pre- and post-processor for execution of site response simulations considering liquefaction-induced lateral deformations. The liquefaction model employed in the program is formulated within the framework of multi-yield-surface plasticity, shown in Figure 2. In this model, emphasis is placed on controlling the magnitude of cycle-by-cycle permanent shear strain accumulation in clean medium to dense sands [36, 37]. Specifically, the experimentally observed accumulation of permanent shear strain was modeled using strain-space parameters. Furthermore, appropriate loading-unloading flow rules were devised to reproduce the observed strong dilation tendency, and resulting increase in cyclic shear stiffness and strength (the "Cyclic Mobility" mechanism).

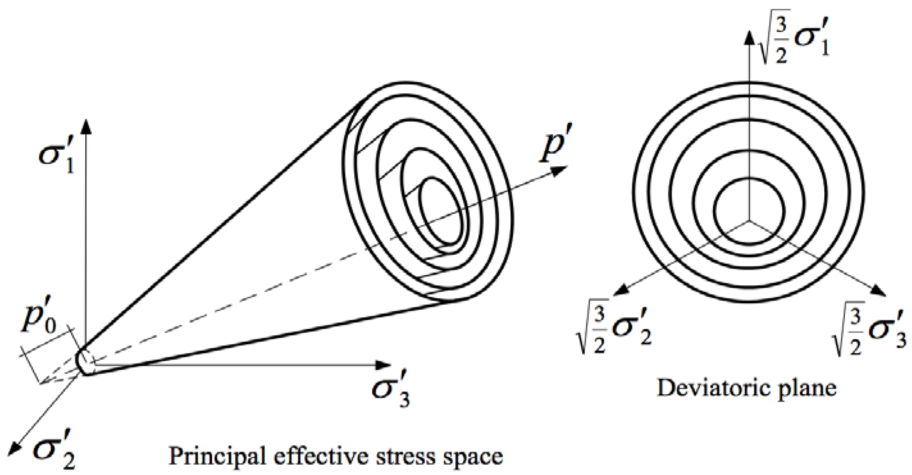

Figure 2: Multi-yield surfaces in principal stress space and deviatoric plane. Liquefaction model of Cyclic 1D, [36, 37].

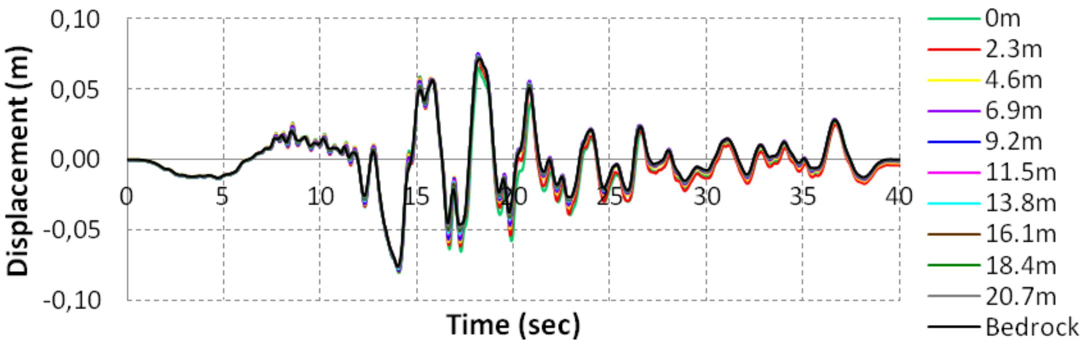

Figure 3: Displacement time histories of the soil layers for record #8.

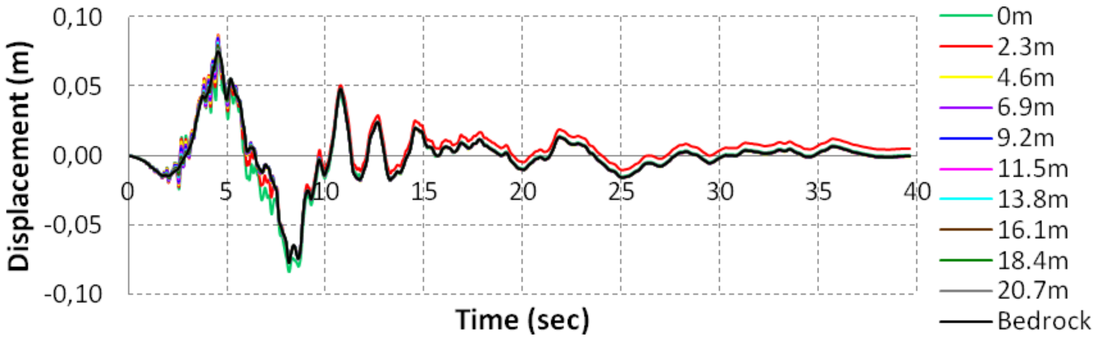

Figure 4: Displacement time histories of the soil layers for record #1.

Figures 3 and 4 illustrate the absolute displacement time history of each soil layer, for record #8, Taiwan (29 SMART1) with a peak ground acceleration of 0.160g and record 1, Loma Prieta (Anderson Dam) with 0.244g, respectively. The differences among ground motions in different depths are not substantial in the time domain, yet, as will be illustrated in the following sections, they are adequate to generate non-negligible rotational excitation to the superstructure.

3.2 Soil – pile group – bridge interaction analysis

3.2.1 Kinematic interaction analysis

Kinematic interaction analysis is carried out first by computing the response of the pile group foundation, in terms of transverse displacement and rotation of the pile cap, under the lateral dynamic loading the piles with the displacement time history prescribed above for each soil layer. The lateral soil resistance-deflection (p-y) and the corresponding damping at each specific depth are derived according to [40]. It is noted that in order to account for localized liquefaction, the spring constants k_h of the liquefiable layers have been appropriately reduced according to the methodology proposed by the Japanese Road Association [41]:

$$k_{h,liq} = D_E k_h \tag{1}$$

where D_E denotes a reduction factor which is a function to depth, liquefaction safety factor and shear resistance ratio. Based on the depth-dependent soil properties, the pile group characteristics and the selected ground motions, the dynamic impedance coefficients are computed (Table 3; liquefiable layers are marked in italics). Note that the effect of ground motion intensity (in terms of PGA) does not affect the liquefaction safety factor, thus resulting in almost identical soil-pile stiffness for all the excitations examined.

The kinematic interaction analysis is performed in SAP2000 finite element software [34]. The 4x4 pile group is modeled with linear finite elements for the piles and shell elements for the pile cap, while the soil springs follow the discretization of the piles.

Based on the layers of the soil profile, the displacements obtained from the site response analysis, are imposed along the piles in time domain.
As already mentioned, the response of the pile cap, in terms of displacement and rotation time histories, is the Foundation Input Motion of the superstructure. Figures 5 and 6 illustrate the displacement (u) and rotation (θ) of the pile cap, indicatively for records #1 and #8. It is seen that in both cases, rotation and translation time histories remain generally in phase. Note that, rotation values are multiplied by 10 in order to plot the two components of the input motion in the same diagram.

Depth (m)	k_h D (kN)
0 - 4	*5200*
4 - 5	*3900*
5 - 6	*2600*
6 - 7	*3900*
7 - 10	*5200*
10 - 12	6500
12 - 13	7800
13 - 14	6500
14 - 16	5200
16 - 17	6500
17 - 24	7800
24 – 28	32500
28 - 34	39000
(bedrock) 34 - 37	71500

Table 3: Depth-dependent stiffness of the soil – pile system.

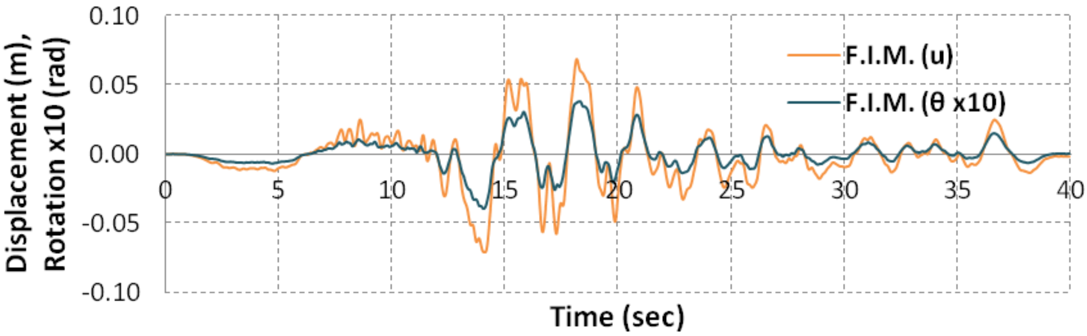

Figure 5: Kinematic interaction response of the pile cap (record #8).

3.2.2 Inertial interaction of the superstructure with the supporting soil-foundation system

The inertial response of the bridge-foundation system can be computed under the substructure excitation obtained from the kinematic interaction step. For computational convenience and conceptual simplicity, the inertial interaction stage is further subdivided into two independent analysis steps, i.e., (a) computation of the dynamic impedances at the pile-group cap associated with the swaying, rocking and cross-swaying-rocking motion of the foundation and (b) dynamic analysis of the superstructure supported on the above springs and dashpots subjected to the Foundation Input Motion.

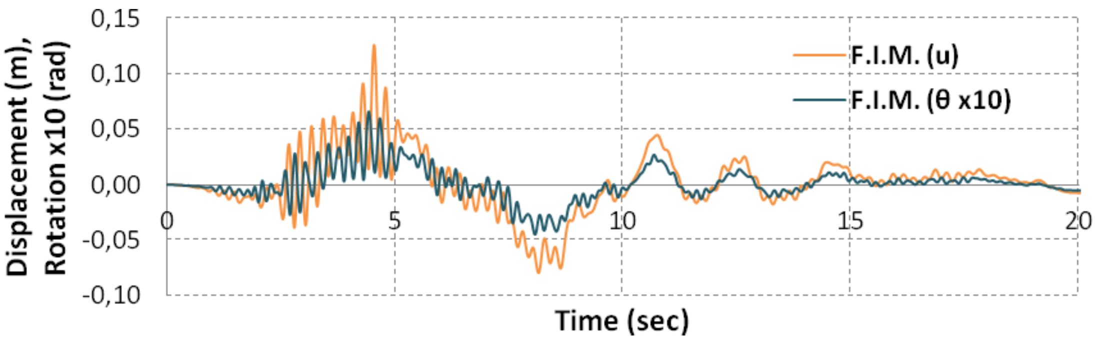

Figure 6: Kinematic interaction response of the pile cap (record #1).

The calculation of springs and dashpots is performed through a Matlab based software based on analytical solutions [18] and taking into consideration the pile group properties (number of piles, pile length, pile spacing, material, diameter, pile cap, pile to pile interaction), soil conditions (V_s, E_s, depth, damping, poisson's ratio) as well as the dominant frequency of the excitation (Table 4). Having determined the two components (translational and rotational) of the foundation kinematic response and the pile-group dynamic impedances, the response of the bridge pier, namely, the (absolute) displacement and rotation of the pier top at the deck level, are computed through linear elastic analysis.

4 Analysis results

4.1 Structural response due to translational and rotational excitation

The results portrayed in Figures 7 and 8 compare the maximum displacements of the deck at the location of piers M1 and M2, along the transverse direction of the bridge, under two distinct excitation scenarios, i.e., purely translational excitation and combined translational and rotational excitation, respectively. Figure 7 in particular, illustrates the maximum top displacement of pier M1, along the transverse direction of the bridge, when subjected to the corresponding translational component of the foundation input motion for the selected records (i.e., U_{max} due to F.I.M.$_{(u)}$), compared to the top displacement of pier M1 due to both the translational and rotational components of foundation response (i.e., U_{max} due to F.I.M.$_{(u+\theta)}$).
It is seen that the pier displacements in case of the conventional approach, that is, purtely translational base excitation, are in most cases higher, by 4% to 16%, compared to the response due to the combined foundation input motion. This trend

is even more pronounced in case of pier M2 where the rotational excitation of the foundation leads to approximately 30% lower maximum displacements. It is interesting to note herein that pier M2 is higher than M1 (9.34m vs. 7.94m), hence, more sensitive to rocking modes of vibrations.

Exceptions, for both piers M1 and M2, are analyses with records #1 (Loma Prieta_1652) and #12 (Livermore) where the combined translational and rotational components of excitation leads to a minor increase in top displacements. Apparently, due to the limited number of records studied herein, it is not feasible to draw generalized conclusions, however, these results yield interesting the investigation of the fundamental mechanisms behind the coupling of the two ground motion components that will be attempted in the following sections.

To understand better the reasons behind the detrimental effect of the rotational excitation for the case of record #8 (Taiwan) as opposed the case of record #1 (Loma Prieta), the displacement time history of the top displacements of pier M2 are comparatively plotted in figures 9 and 10.

	Record #8 (Taiwan)		**Record #1 (Loma Prieta)**	
DOF	Springs (kN/m)	Dashpots (kNsec/rad m)	Springs (kN/m)	Dashpots (kNsec/rad m)
$U_{vertical}$	405760	11640	404300	11850
$U_{longitudinal}$	108890	4140	106520	3830
$U_{transverse}$	108890	4140	106520	3830
DOF	Springs (kNm/rad)	Dashpots (kNm sec/rad^2)	Springs (kNm/rad)	Dashpots (kNm sec/rad^2)
$R_{longitudinal}$	12454200	92680	12441830	86070
$R_{transverse}$	12454200	92680	12441830	86070

Table 4: Dynamic impedance of the pile-supported bridge superstructure.

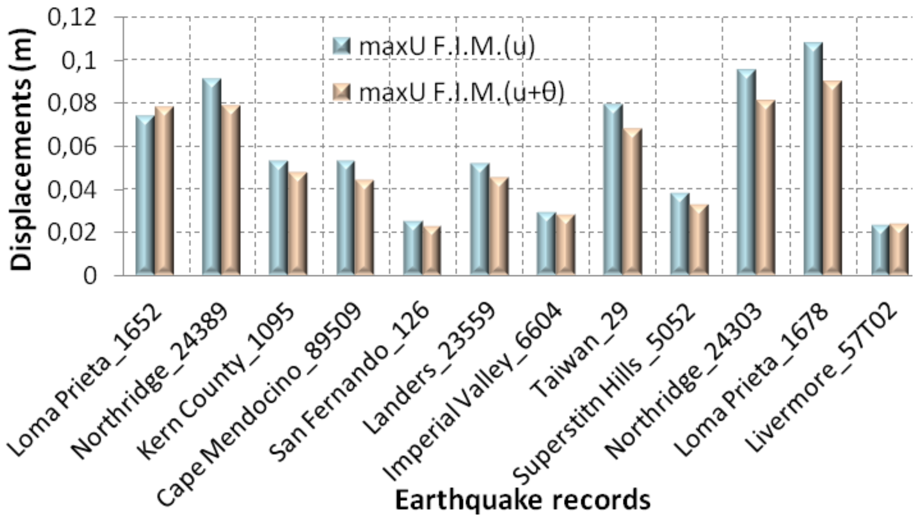

Figure 7: Maximum displacements of pier M1 for purely translational (F.I.M.$_{,u}$) and combined translational and rotational excitation (F.I.M.$_{,u+\theta}$).

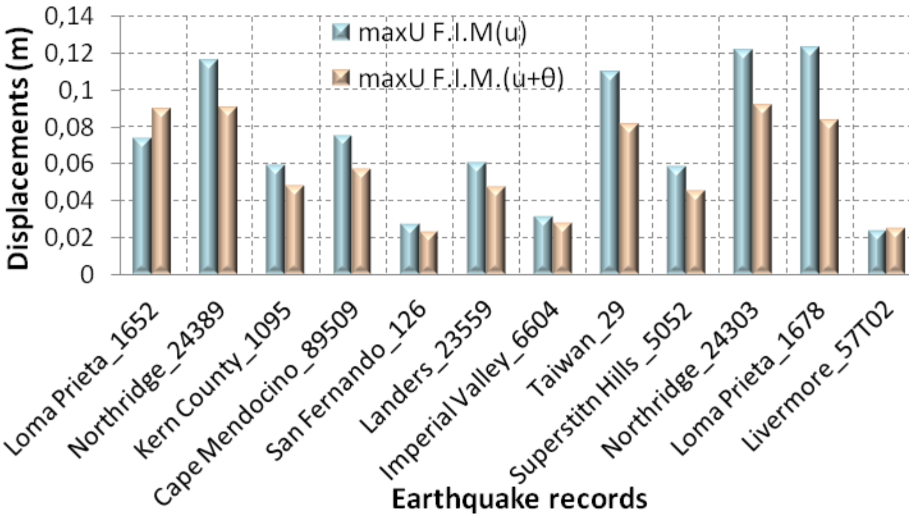

Figure 8: Maximum displacements of pier M2 for for purely translational (F.I.M.$_{,u}$) and combined translational and rotational excitation (F.I.M.$_{,u+\theta}$).

The figures illustrate the three response time histories that correspond to the three distinct excitation scenarios of the superstructure, i.e. translational, rotational and the combined translational and rotational effect. In case of record #8 (Taiwan, figure 9), the response of pier M2 due to pure translation is clearly higher compared to the combined, translational and rotational excitation, a fact that was also depicted in figure 8. Notably, the displacements due to the rotational acceleration ($U_{,F.I.M.(\theta)}$) are clearly not in phase with the ones emerging from the translational excitation ($U_{,F.I.M.(u)}$). This is a clear indication that the two ground motion excitation components are essentially counteracting, thus reducing the superstructure response. On the contrary, in case of record #1 (Loma Prieta record), the two components remain in phase, at least during the first five seconds where the strong ground motion takes place. Naturally, the *maximum* top displacement of pier M2 is higher under the combined (translational and rotational) excitation as they latter act in synergy ($U_{max,F.I.M(u)}$=6.9cm and $U_{max,F.I.M.(\theta)}$=2.2cm). After t=5sec, the response under pure translation and pure rotation are clearly out of phase, however, this does not affect the maximum in time pier top displacement. The above observations though, provide evidence that the potentially detrimental effect of the rotational component of ground motion is phase-dependent. This mechanism will be further investigated in the following section.

4.2 The effect of the rotational Foundation Input Motion on the superstructure

Figures 11-12 and 13-14 present the acceleration time histories and their Fourier amplitude spectra of the translation and rotational components of ground motion for record #8 (Taiwan) and record #1 (Loma Prieta), respectively. Note that rotation values are multiplied by 10 to facilitate plotting of both results in the same diagram. It is clearly seen that in case of record #8 (Taiwan), the rotational component of ground motion excites a significant number of high frequencies, which were not triggered by the translational component. In contrast, the foundation input motion (i.e., the response of the soil-pile foundation system) under record #1 (Loma Prieta) does not trigger any higher modes of vibration. Naturally, the F.I.M. in the first case associated with significantly richer frequency content, excites rocking-related modes of vibration of the pile group and enhances the impact of the rotational excitation to the superstructure. It is noted that this activation of higher modes can be attributed to the asynchronous nature of pile group excitation (i.e., pile imposed displacements are spatially variable with depth) and is a phenomenon that is also encountered in bridges [43].

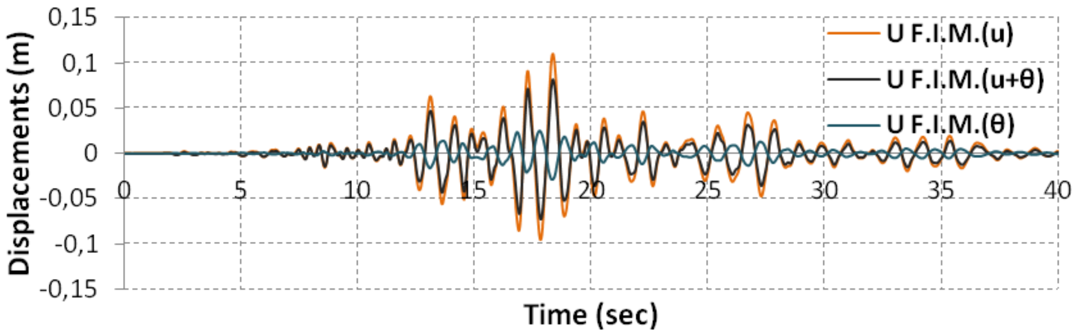

Figure 9: Displacement time histories of pier M2 under purely translational, purely rotational and combined ground motion excitation.

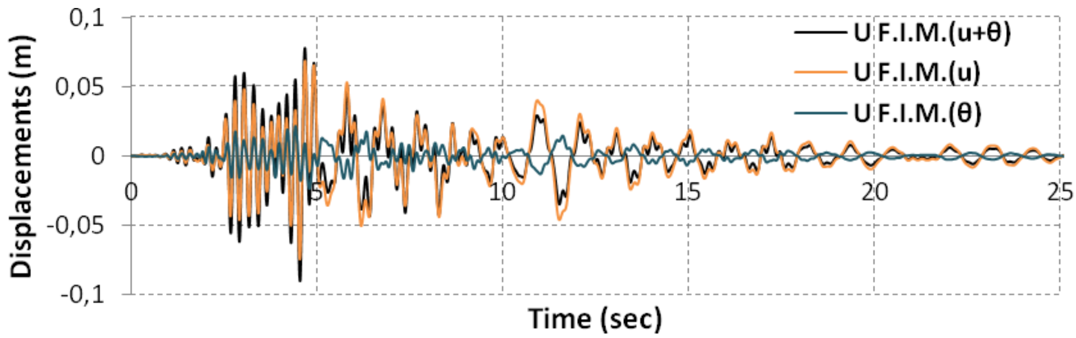

Figure 10: Displacement time histories of pier M2 under purely translational, purely rotational and combined ground motion excitation.

5 Conclusions

An analytical study was presented to investigate the effect of the rotational component of earthquake ground motion on the overall dynamic response of bridge structures on pile groups in multi-layered, liquefiable soil deposits. The study focuses on a real bridge of Egnatia Highway in Greece subjected to simultaneous

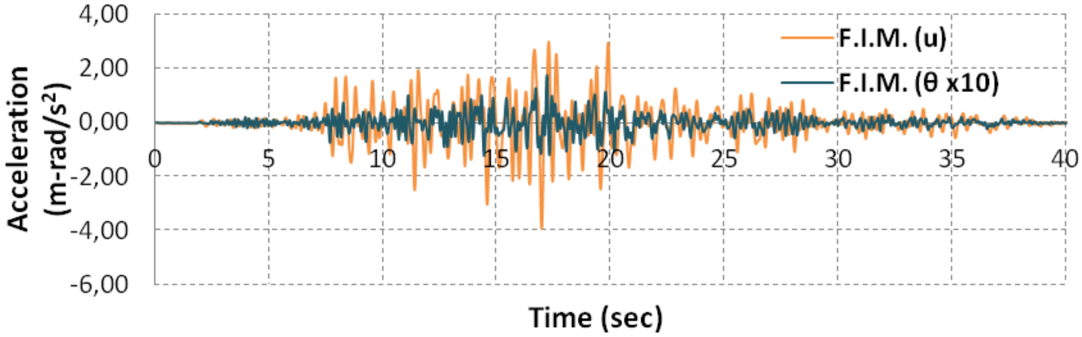

Figure 11: Translational and rotational F.I.M. for record # 8 (Taiwan).

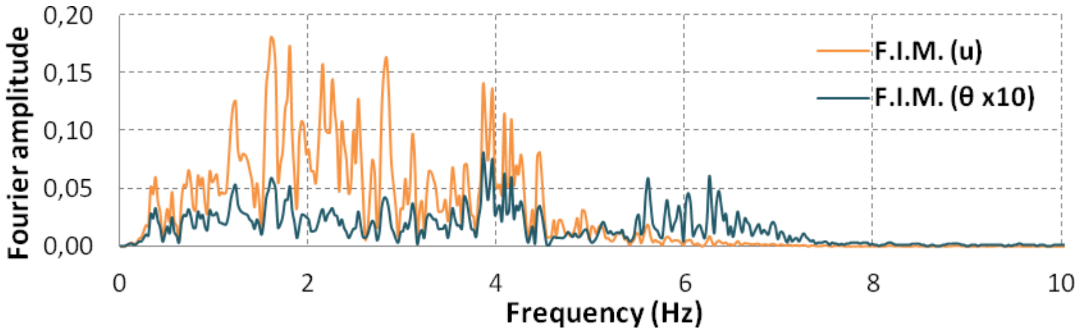

Figure 12: Fourier amplitude spectra of the translational and rotational F.I.M. acceleration time histories for record # 8 (Taiwan).

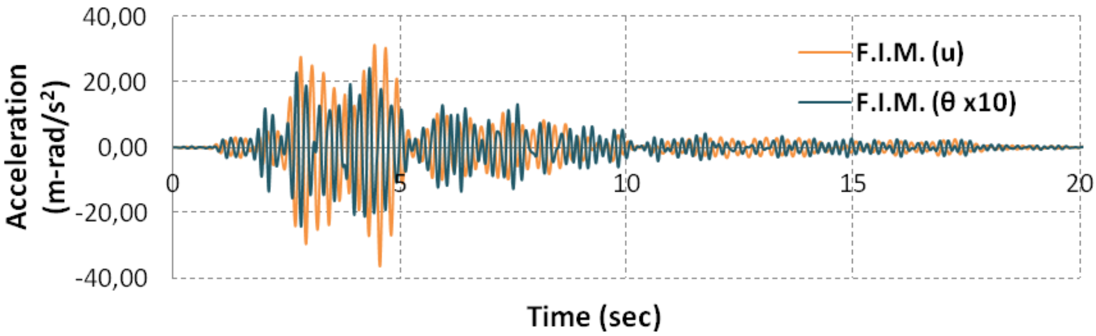

Figure 13: Translational and rotational F.I.M. for record # 1 (Loma Prieta).

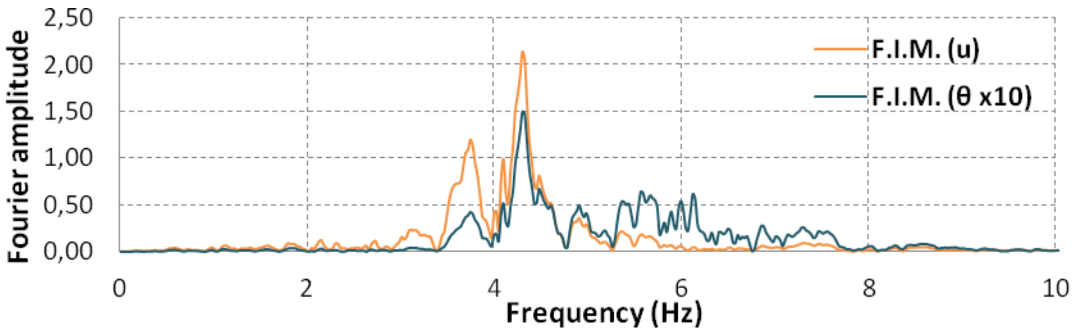

Figure 14: Fourier amplitude spectra of the translational and rotational F.I.M. acceleration time histories for record # 1 (Loma Prieta).

translational and rotational foundation input motion derived from the kinematic interaction analysis. The analytical approach is performed through a comprehensive computational framework, for a set of linear, elastic analyses. 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 of the foundation where the base rotation can indeed change the system response either in a beneficial or a detrimental way. This behaviour is attributed to the excitation of particular higher frequencies and the associated phase shift between the translational and rotational response of the pile group that eventually decrease, the final response of the superstructure. Further research is certainly needed for investigating the coupling between the translational and the rotational component of seismic motion in case of more complex structural systems, soil conditions and foundation configurations.

Acknowledgements

This research has been co-financed by the European Union (European Social Fund – ESF) and Greek national funds through the Operational Program "Education and Lifelong Learning" of the National Strategic Reference Framework (NSRF) – Research Funding Program: THALES: Reinforcement of the interdisciplinary and/or inter-institutional research and innovation.

References

[1] A. G. Sextos, E.-K. Mylona, and G. Mylonakis, "A computational framework for the assessment of earthquake-induced rocking in CIDH pile supported bridges," in *3rd ECCOMAS Thematic Conference on Computational Methods in Structural Dynamics and Earthquake Engineering (COMPDYN), Corfu, Greece*, 2011.

[2] M. Celebi, "Real-time seismic monitoring of the New Cape Girardeau (MO) bridge and recorded earthquake response," in *8th National Conference on Earthquake Engineering, San Fransisco, U.S.*, 2006, no. 158.

[3] G. E. Mylonakis, C. Syngros, G. Gazetas, and T. Tazoh, "The role of soil in the collapse of 18 piers of hanshin expressway in the kobe earthquake," *Earthquake Engineering and Structural Dynamics*, vol. 35, no. 5, pp. 547–575, 2006.

[4] N. Makris, G. Gazetas, and E. Delis, "Dynamic Soil-Pile-Foundation-Structure Interaction: Records and Predictions," *Geotechnique*, vol. 46, no. 1, pp. 33–50, 1996.

[5] A. Kotsoglou and S. Pantazopoulou, "Response simulation and seismic assessment of highway overcrossings," *Earthquake Engineering & Structural Dynamics*, vol. 39, pp. 991–1013, 2009.

[6] A. G. Sextos, K. D. Pitilakis, and A. J. Kappos, "Inelastic dynamic analysis of RC bridges accounting for spatial variability of ground motion, site effects and soil-structure interaction phenomena. Part 1: Methodology and analytical tools," *Earthquake Engineering & Structural Dynamics*, vol. 32, no. 4, pp. 607–627, 2003.
[7] A. G. Sextos, K. D. Pitilakis, and A. J. Kappos, "Inelastic dynamic analysis of RC bridges accounting for spatial variability of ground motion, site effects and soil-structure interaction phenomena. Part 2: Parametric study," *Earthquake Engineering & Structural Dynamics*, vol. 32, no. 4, pp. 629–652, 2003.
[8] Y. X. Cai, P. L. Gould, and C. S. Desai, "Nonlinear analysis of 3D seismic interaction of soil–pile–structure systems and application," *Engineering Structures*, vol. 22, pp. 191–199, 2000.
[9] E. N. Rovithis, K. D. Pitilakis, and G. E. Mylonakis, "Seismic analysis of coupled soil-pile-structure systems leading to the definition of a pseudo-natural SSI frequency," *Soil Dynamics and Earthquake Engineering*, vol. 29, no. 6, pp. 1005–1015, 2009.
[10] B. Maheshwari, K. Z. Truman, M. H. El Naggar, and P. L. Gould, "3-D nonlinear analysis for seismic soil–pile-Structure interaction," *Soil Dynamics and Earthquake Engineering*, vol. 24, no. 4, pp. 343–356, 2004.
[11] G. Gazetas, "Seismic response of end-bearing single piles," *International Journal of Soil Dynamics and Earthquake Engineering*, vol. 3, no. 2, pp. 82–93, 1984.
[12] E. A. Ellis and S. M. Springman, "Modelling of soil-structure interaction for a piled bridge abutment in plane strain FEM analyses," *Computers and Geotechnics*, vol. 28, pp. 79–98, 2001.
[13] N. Makris and G. Gazetas, "Dynamic pile-soil-pile interaction. part II: lateral ans seismic response," *Earthquake Engineering & Structural Dynamics*, vol. 21, pp. 145–162, 1992.
[14] D. Badoni and N. Makris, "Pile-to-pile interaction in the time domain-non-linear axial group response under harmonic loading," *Geotechnique*, vol. 47, no. 2, pp. 299–317, 1997.
[15] K. K. Koo, K. T. Chau, X. Yang, S. S. Lam, and Y. L. Wong, "Soil-Pile-Structure interaction under SH wave excitation," *Earthquake Engineering & Structural Dynamics*, vol. 32, no. 3, pp. 395–415, 2003.
[16] T. Abdoun and R. Dobry, "Evaluation of pile foundation response to lateral spreading," *Soil Dynamics and Earthquake Engineering*, vol. 22, pp. 1051–1058, 2002.
[17] D. Badoni and N. Makris, "Nonlinear response of single piles under lateral inertial and seismic loads," *Soil Dynamics and Earthquake Engineering*, vol. 15, pp. 29–43, 1996.
[18] G. E. Mylonakis and G. Gazetas, "Lateral vibration of internal forces of grouped piles in layered soil," *Journal of Geotechnical and Geoenvironmental Engineering*, vol. 125, no. 1, pp. 16–25, 1999.
[19] Z. Yang and B. Jeremic, "Numerical analysis of pile behaviour under lateral loads in layered elastic-Plastic soils," *International Journal for Numerical*

and Analytical Methods in Geomechanics, vol. 26, no. 14, pp. 1385–1406, 2002.
[20] A. J. Kappos and A. G. Sextos, “Effect of foundation compliance on the lateral load response of R/C bridges,” *Journal of Bridge Engineering*, vol. 6, pp. 120–130, 2001.
[21] S. Miwa, T. Ikeda, and T. Sato, “Damage process of pile foundation in liquefied ground during strong ground motion,” *Soil Dynamics and Earthquake Engineering*, vol. 26, no. 2–4, pp. 325–336, 2006.
[22] J. Berrill and S. Yasuda, “Liquefaction and piled foundations: some issues,” *Journal of Earthquake Engineering*, vol. 6, no. 1, pp. 1–41, 2002.
[23] S. Bhattacharya, S. P. G. Madabhushi, and M. Bolton, “An alternative mechanism of pile failure in liquefiable deposits during earthquakes,” *Geotechnique*, vol. 54, no. 3, pp. 203–213, 2004.
[24] W. D. L. Finn and N. Fujita, “Piles in liquefiable soils: seismic analysis and design issues,” *Soil Dynamics and Earthquake Engineering*, vol. 22, pp. 731–742, 2002.
[25] R. Ranf, H. Shin, M. O. Eberhard, P. Arduino, and S. L. Kramer, “Experimentally based evaluation of soil-foundation-structure interaction for a reinforced concrete bridge,” in *8th National Conference on Earthquake Engineering, San Fransisco, U.S.*, 2006, no. 504.
[26] K. Chau, C. Shen, and X. Guo, “Nonlinear seismic soil–pile–structure interactions: shaking table tests and fem analyses,” *Soil Dynamics and Earthquake Engineering*, vol. 29, no. 2, pp. 300–310, 2009.
[27] N. Kamijo, H. Saito, K. Kusama, O. Kontani, and R. Nigbor, “Seismic tests of a pile-supported structure in liquefiable sand using large-scale blast excitation,” *Nuclear Engineering and Design*, vol. 228, no. 1–3, pp. 367–376, 2004.
[28] K. J. Bentley and M. H. El Naggar, “Numerical analysis of kinematic response of single piles,” *Canadian Geotechnical Journal*, vol. 37, pp. 1368–1382, 2000.
[29] G. E. Mylonakis, S. Nikolaou, G. Gazetas, and A. Nikolaou, “Soil-pile-bridge seismic interaction: kinematic and inertial effects. part I: soft soil,” *Earthquake Engineering and Structural Dynamics*, vol. 26, pp. 337–359, 1997.
[30] F. Dezi, S. Carbonari, and G. Leoni, “Kinematic bending moments in pile foundations,” *Soil Dynamics and Earthquake Engineering*, vol. 30, no. 3, pp. 119–132, 2010.
[31] G. E. Mylonakis and G. Gazetas, “Kinematic Pile Response to Vertical P-wave Seismic Excitation,” *Journal of Geotechnical and Geoenvironmental Engineering*, vol. 128, no. 10, pp. 860–867, 2002.
[32] F. Dezi, S. Carbonari, and G. Leoni, “Static equivalent method for the kinematic interaction analysis of single piles,” *Soil Dynamics and Earthquake Engineering*, pp. 1–12, 2010.
[33] A. Maravas, G. E. Mylonakis, and D. . Karabalis, “Dynamic characteristics of structures on piles and footings,” in *4th International Conference on Earthquake Geotechnical Engineering*, 2007, no. 1672.

[34] CSI, “SAP2000: Linear and Nonlinear Static and Dynamic Analysis and Design of Tree-Dimensional Structures.” Berkeley, California, 2006.

[35] PEER, “Pacific Earthquake Engineering Research,” *University of California, Berkeley*. University of California, Berkeley, 2013.

[36] A. Elgamal, Z. Yang, and J. Lu, “Cyclic 1D: A Computer Program for Seismic Ground Response,” San Diego, 2006.

[37] Z. Yang, J. Lu, and A. Elgamal, “A web-based platform for computer simulation of seismic ground response,” *Advances in Engineering Software*, vol. 35, no. 5, pp. 249–259, 2004.

[38] E. Parra, “Numerical Modeling of Liquefaction and Lateral Ground Deformation Including Cyclic Mobility and Dilation Response in Soil Systems,” Rensselaer Polytechnic Institute, N.Y., 1996.

[39] Z. Yang, “Numerical modeling of earthquake site response including dilation and liquefaction,” Columbia University, N.Y., 2000.

[40] N. Makris, D. Badoni, E. Delis, and G. Gazetas, “Prediction of Observed Bridge Response with Soil-Pile-Structure Interaction,” *Journal of Structural Engineering, ASCE*, vol. 120, no. 10, pp. 2992–3011, 1994.

[41] JRA, *Specifications for Highwaybridges*. 2002.

[42] G. E. Mylonakis and G. Gazetas, “Settlement and additional internal forces of grouped piles in layered soil,” *Geotechnique*, vol. 48, no. 1, pp. 55–72, 1998.

[43] A. G. Sextos, A. J. Kappos, and K. D. Pitilakis, “Inelastic dynamic analysis of RC bridges accounting for spatial variability of ground motion, site effects and soil-structure interaction phenomena. Part 2: Parametric study,” *Earthquake Engineering & Structural Dynamics*, vol. 32, no. 4, pp. 629–652, 2003.