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An Investigation on the Parameters Influencing the Pounding in Highway Bridges

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ABSTRACT

The present aim of this study is to investigate the effect of different parameters influencing pounding in highway bridges. Pounding is the result of a collision between two parts of the deck and/or the deck and abutments at the separation distance during the earthquake. In the present study, the period ratio of the adjacent frames, ground motion spatial variation, and soil-structure interaction were considered as the significant parameters influencing the pounding. Accordingly, 144 different models of bridge were generated by changing characteristics of the piers and spans length, and were subjected to non-linear dynamic analysis. The results indicated that ignoring the effects of soil-structure interaction and ground motion spatial variation led to calculate unrealistic responses in the bridges. Finally, it is found that designing bridges including frames with similar or close period is not regarded as an appropriate solution to reduce the pounding effects in the bridges.

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1. INTRODUCTION

The structural design codes propose different forces for the analysis and design of structures depending on the type of application and site conditions. However, some of the forces acting on the structures are not emphasized or briefly addressed in the codes. Due to lack of deep understanding of the exerted forces, especially the dynamic forces, many damages have been observed in the bridges built during the last two centuries. In the last few decades, especially after earthquakes such as Northridge, Kobe and Chi-Chi and their devastating effects on bridges, the force caused by the pounding phenomenon received more attention by researchers. Pounding is created at the separation distance by the impact between two parts of the deck and/or between deck and the abutments, due to out-of-phase responses during major earthquakes. The pounding in bridges can cause either damage in the deck and abutment or the unseating of the deck from its support. In conventional bridge design by procedures in which a few centimeters of separation distance has been considered, the pounding during severe earthquakes will be unavoidable [1-3]. The difference in the periods of adjacent frames is regarded as one of the most important factors in creating out-of-phase responses. Therefore, in order to prevent out-of-phase responses, some design codes propose to design the adjacent frames with equal or close fundamental frequencies in such a way that the fundamental frequency of the more flexible frame should be at least 0.7 times more than the frequency of the more stiff adjacent frame based on the CALTRANS code [4, 5]. On the other hand, lifeline structures such as bridges and pipelines are affected by the non-uniform seismic excitations in their multi-supports in longitudinal direction [6, 7]. Further, the non-uniform excitations, known as the "ground motion spatial variation" leads to out-of-phase responses in the adjacent frames.

Moreover, in the conventional method of dynamic analysis of structures, the ground motions recorded on the free field are inserted to the structure by assuming that the structure is fixed on the ground as a rigid media. Such assumptions are nearly correct for the rocky and stony or relatively hard grounds while it is not true for soft site. Regarding the soil-structure interaction, a large number of studies indicated the effect of this phenomenon on seismic responses of bridges, especially on the total motion of the deck due to sway and rocking motions in the foundation [8-11]. As the total deck

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displacement results in closing the separation distances and pounding in bridges, consideration of this effect in dynamic analyses can provide more accurate results in studying the pounding phenomenon. In addition, the effect of soil properties and the dimensions of the piers and foundations in creating out-of-phase responses are the other effects of considering the soil-structure interaction in the analyses.

Many studies were conducted on the pounding and its influencing parameters in structures such as buildings and highway bridges. For instance, Ruangrassamee and Kawashima [12] calculated the relative displacement response spectrum by considering the pounding effect and indicated that the pounding in bridges increases the size of the support needed for the deck by increasing the relative displacement. In this study, the nonlinear behavior of materials, soil-structure interaction and ground motion spatial variation were ignored. In another work, Desroches and Muthukumar [13] studied the effects of the frame period ratio, the effective period of the earthquake, frame ductility, and the use of restrainers on the pounding response of bridge frames. However, the effects of soil-structure interaction and ground motion spatial variation were neglected in this study. Chouw et al. [2] analyzed the effect of separation distance size on bending moment of pier and maximum pounding force by modeling the bridges with the separation distances of 1, 3 and 5 cm by considering the effect of soil-structure interaction and the ground motion spatial variation. The results indicated that the common size of separation distance, even for the bridges with similar periods was not enough to avoid pounding. Chouw et al. [14] calculated the minimum separation distance required to avoid the impact of the adjacent girders in bridges with modular expansion joint system. In this study, the effects of soil-structure interaction and ground motion spatial variation were considered, irrespective of the modeling of the abutments and the nonlinear behavior of the piers. In another study, Bi et al. [8] by using random vibration method estimated the minimum required separation distance to prevent the pounding between the two parts of the deck and between deck and abutments by considering elastic behavior assumption, irrespective of soil-structure interaction. Furthermore, they extended their study and concluded that soil-structure interaction plays a significant role on the required separation distance values to preclude the pounding [15]. Bi and Hao [16] analyzed the damage mechanism in a two-span simply supported bridge under the non-uniform excitation of the piers to study the pounding effect by using three-dimensional modeling. However, the effect of soil-structure interaction was neglected. In another research, Zhang et al. [17] analyzed the pounding between the deck and abutments by modeling a two-span bridge, irrespective of the soil-structure interaction effect. In addition to the aforementioned studies, some studies were also conducted to remove or reduce the pounding effects in bridges by using the control systems, restrainers and bumpers [18-25].

As it can be seen from the above literature, in most of these studies, for simplicity, some significant parameters influencing the out-of-phase responses of bridges have been ignored. Therefore, the present research is aimed to study the effects of the span length, the ratio of piers height and the period ratio of the adjacent frames in the seismic responses and the pounding in bridges by considering the soil structure interaction and ground motion spatial variation. Accordingly, various samples of bridges were subjected to non-linear dynamic analysis.

2. BRIDGE MODELING

2. 1. Pier and Deck Modeling In order to perform the dynamic modeling of bridge, two intermediate frames of a bridge were selected while the effects of the adjacent frames and abutments were ignored to reduce the complexity in interpreting the results. The bridge mass was concentrated on the deck and each of the piers (with deck) was modeled as a single-degree-of-freedom system. In addition, a three-degree-of-freedom system including translational and rotational movements of foundation, and the rotation of trapped mass moment of inertia was considered to model the interaction between the pier foundations and the surrounding soil [26, 27]. Figure 1 illustrates the longitudinal perspective, along with a schematic and simplified model of the bridge. Then, mass, stiffness and damping matrices were calculated and given in Equations (2) to (4), based on the material properties and the dimensions of the bridge components. Obviously, by changing the height, diameter and number of columns for each pier, as well as the bridge deck mass, a change takes place in the period of each frame. Table 1 illustrates the characteristics of all models analyzed in the present study. In this table, T_1, h_1, m_1, T_2, h_2 and m_2 are the period, pier height and mass in the first and second frames of the bridge, respectively. According to this table, 144 different models of bridge are analyzed by changing the parameters of the span length, pier height and period of the frames. The values of different parameters are selected as follows:

- The distance between the two piers in different models is considered to be about 25, 50, 75 and 100 m.
- The height of the left pier is fixed to be about 10 m, while that of the right pier is taken to be about 5, 7.5 and 10 m.
- The left frame includes the periods of 0.5, 1 and 1.5 seconds while the periods in the right frame are selected based on different ratios of T_1/T_2 such as 0.55, 0.7, 0.85 and 1.

Moreover, in all these models, the separation distance between two parts of the deck is taken to be about 5 cm and the structural damping ratio of the bridge is assumed to be 5% [11]. In such bridges, Separation distance is usually between 2.5 and 7.5 cm [1, 2].

The equation of motion of the bridge can be written as follows:

$$[M]\{\ddot{u}\}+[C]\{\dot{u}\}+\{F_S\}+\{F\}=-[M][r]\{\ddot{u}_g\} \tag{1}$$

where $[M]$ and $[C]$ represent the mass and damping matrices, respectively; $\{F_S\}$ is inelastic force vector that depends on the histories of displacement and velocity, $\{u\}$, $\{\dot{u}\}$ and $\{\ddot{u}\}$ are displacement, velocity and acceleration vectors of the deck relative to the ground, respectively; $[r]$ is influence coefficient matrix, $\{\ddot{u}_g\}$ the horizontal component of the earthquake acceleration and $\{F\}$ the pounding force vector in the impact location. The equation is solved by using a computer program written by the authors in MATLAB software environment. The mass, stiffness and damping matrices are as follows:

$$[M]=\begin{bmatrix} m_1 & 0 & 0 & 0 & 0 & 0 & 0 & 0 \\ 0 & m_2 & 0 & 0 & 0 & 0 & 0 & 0 \\ 0 & 0 & m_f & 0 & 0 & 0 & 0 & 0 \\ 0 & 0 & 0 & I_f+\Delta M_\phi & 0 & 0 & 0 & 0 \\ 0 & 0 & 0 & 0 & M_\theta & 0 & 0 & 0 \\ 0 & 0 & 0 & 0 & 0 & m_f & 0 & 0 \\ 0 & 0 & 0 & 0 & 0 & 0 & I_f+\Delta M_\phi & 0 \\ 0 & 0 & 0 & 0 & 0 & 0 & 0 & M_\theta \end{bmatrix} \tag{2}$$

$$[K]=\begin{bmatrix} k_1 & 0 & -k_1 & -k_1h_1 & 0 & 0 & 0 & 0 \\ 0 & k_2 & 0 & 0 & 0 & -k_2 & -k_2h_2 & 0 \\ -k_1 & 0 & k_h+k_1 & k_1h_1 & 0 & 0 & 0 & 0 \\ -k_1h_1 & 0 & k_1h_1 & k_\phi+k_1h_1^2 & 0 & 0 & 0 & 0 \\ 0 & 0 & 0 & 0 & 0 & 0 & 0 & 0 \\ 0 & -k_2 & 0 & 0 & 0 & k_h+k_2 & k_2h_2 & 0 \\ 0 & -k_2h_2 & 0 & 0 & 0 & k_2h_2 & k_\phi+k_2h_2^2 & 0 \\ 0 & 0 & 0 & 0 & 0 & 0 & 0 & 0 \end{bmatrix} \tag{3}$$

$$[C]=\begin{bmatrix} c_1 & 0 & -c_1 & -c_1h_1 & 0 & 0 & 0 & 0 \\ 0 & c_2 & 0 & 0 & 0 & -c_2 & -c_2h_2 & 0 \\ -c_1 & 0 & c_h+c_1 & c_1h_1 & 0 & 0 & 0 & 0 \\ -c_1h_1 & 0 & c_1h_1 & c_\phi+c_1h_1^2 & -c_\phi & 0 & 0 & 0 \\ 0 & 0 & 0 & -c_\phi & c_\phi & 0 & 0 & 0 \\ 0 & -c_2 & 0 & 0 & 0 & c_h+c_2 & c_2h_2 & 0 \\ 0 & -c_2h_2 & 0 & 0 & 0 & c_2h_2 & c_\phi+c_2h_2^2 & -c_\phi \\ 0 & 0 & 0 & 0 & 0 & 0 & -c_\phi & c_\phi \end{bmatrix} \tag{4}$$

where m, k, c, h and I display the mass, stiffness, damping, height and moment of inertia, respectively. Subscripts 1, 2 and f indicate the first frame, second frame, and foundation, respectively and subscripts h and ϕ are related to translational and rotational degrees of freedom.

TABLE 1. The properties of the bridges under study

Model No.	Span length (m)	Left frame (P1) T_1 (s)	h_1 (m)	m_1 (ton)	Right frame (P2) $\frac{T_1}{T_2}$	$\frac{h_2}{h_1}$	$\frac{m_2}{m_1}$
1-48	25, 50, 75 ,100	0.5	10	1500	0.55, 0.7, 0.85, 1	0.5, 0.75, 1	1
49-96	25, 50, 75 ,100	1	10	1500	0.55, 0.7, 0.85, 1	0.5, 0.75, 1	1
97-144	25, 50, 75 ,100	1.5	10	1500	0.55, 0.7, 0.85, 1	0.5, 0.75, 1	1

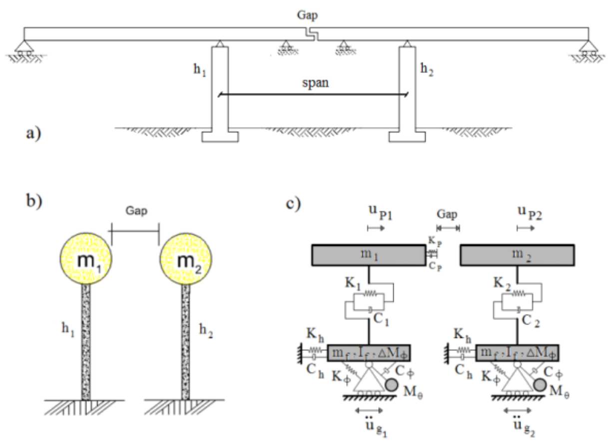

Figure 1. The bridge model selected in this study: a)

Generally, under moderate to severe earthquakes, bridges undergo beyond their elastic region and show plastic behavior. Discrete plastic hinge and the fiber models are among the analytical models to analyze the non-elastic behavior of the members and the structures. In this study, discrete plastic hinge model was considered at the end of the pier columns on the footing, and the nonlinear behavior of the materials in the pier was modeled by a rotational nonlinear spring. In order to determine the characteristics of the hinge, the moment-curvature curve was used to calculate the amount of displacement in the member. The amount of displacement in the top end of the pier just at the yield point was obtained from the following equation:

$$D_y=\varphi_y\frac{h^2}{3} \tag{5}$$

where φ_y is the effective yield curvature, and h is the member length.

In order to express the behavior of reinforced concrete under cyclic seismic loads, researchers use different hysteresis models including simple models like elastoplastic and bilinear models, the models such as

Takeda and Q-hystin which hardness reduction is emphasized, and Pivot model which focuses on the effects of hardness reduction, strength deterioration and pinching [28].

Strength deterioration phenomenon under axial loadings and pinching is observed in the members involving shear deformations, while the members with dominant flexural deformations approximately present stable hysteresis loops [28]. In the present study, the Q-hyst model was used to explain the flexural behavior of the reinforced concrete piers.

As shown in Figure 2, the model consists of four parts [29]:

1- Elastic behavior region: the pier elastic stiffness is calculated by the following equation in this study:

$$K=\frac{3EI_{eff}}{h^3} \tag{6}$$

where E is the concrete modulus of elasticity; and I_{eff} is the effective moment of inertia of the cross section of pier column [5].

2- Strain hardening region: it occurs after the yield point, is expressed as a proportion of primary stiffness (post-yield stiffness).

$$K_{yu}=\beta K \tag{7}$$

where K is the primary stiffness of the pier column and β is the strain hardening parameter. Various studies have shown that axial load, tensile reinforcement and concrete strength affect post-yield stiffness values. The β value is usually between 0.01 and 0.2. In present study, based on the amounts proposed by researchers in similar works the value of 0.05 is considered [13].

3- Unloading: the stiffness decreases during unloading the system compared to the primary stiffness.

$$K_r=K\left(\frac{D_y}{D_{max}}\right)^{\alpha} \tag{8}$$

where D_y is the yield displacement, D_{max} the maximum displacement attained in the loading and α unloading stiffness degradation parameter. The value of α was considered to be about 0.4

4- Loading: it increases the load after unloading stage.

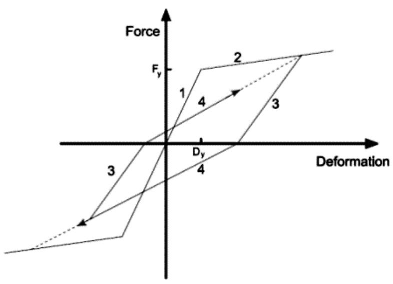

Figure 2. Q-hyst model

2. 2. Soil-structure Interaction Modeling The vibration of the structure (above the ground) is affected by the flexibility of the soil materials beneath the foundation. Accordingly, the deformation and displacement of the foundation will be influenced by the vibrations of the structure above the ground. This mutual practice of the soil and structure is called soil-structure interaction.

Based on the seismic design codes, applying the effect of the soil-structure interaction in a structural design may be required depending on the soil and the structure properties. FEMA 356 [30] proposes soil-structure interaction in cases which increasing period leads to an increase in the spectral accelerations of the structure such as near field sites and soft soil. However, the effect can be disregarded in other cases. European seismic design code [31] suggests that the interaction effects should be considered in a) those structures in which second order effects ($P-\Delta$) play an important role in seismic responses, b) the structures with heavy or deep foundations such as bridges and silos, c) tall and slender structures, and d) the structures on soft soils with average shear wave velocity of less than 100 m/s. The CALTRANS seismic design criteria classified the soil type into competent soils, marginal soils and poor soils. The fixed base assumption for foundation is not appropriate for the last two categories.

The methods used for modeling the soil-structure interaction are divided into two general categories: (1) the direct method by which the soil and structure are modeled using finite element methods simultaneously, and (2) the substructure method by which the soil and structure are separately modeled. The spring-dashpot-mass model belongs to the last category. Usually, due to the complexity of the analysis that arises by considering the frequency-dependent parameters, the soil environment is modeled by linear spring and dashpot (independent of frequency). However, compared to the exact solution, this assumption is only valid for small frequencies, while for frequencies somehow greater than the resonance frequency, it leads to unacceptable results [27].

In this study, the spring-dashpot-mass model is used to model soil and soil-structure interaction effect (Figure 1).

In this model, the coefficients and parameters are calculated using the concept of Cone Models, as presented in the following equations. It is worth noting that only the radiation damping is considered in the present study, and the material damping of the soil is ignored [27].

$$K_h=\frac{8Gr}{2-\nu},\ K_\phi=\frac{8Gr^3}{3(1-\nu)} \tag{9}$$

$$C_h=\pi\rho V_s r^2,\ C_\phi=\frac{1}{4}\pi\rho V_p r^4 \tag{10}$$

$$M_{\phi}=\frac{9\pi^2\rho r^5(1-\nu)}{32}\left(\frac{V_P}{V_s}\right)^2,\ \Delta M_{\phi}=0.3\pi\rho r^5\left(\nu-\frac{1}{3}\right)\ \text{if}\ \nu>\frac{1}{3} \tag{11}$$

$$V_s=\sqrt{\frac{G}{\rho}},\ V_P=\sqrt{2\frac{G}{\rho}\frac{1-\nu}{1-2\nu}} \tag{12}$$

where K_h and C_h are sway stiffness and damping coefficients, respectively. K_{ϕ} and C_{ϕ} are rocking stiffness and damping coefficients, respectively. M_{θ} is the mass moment of inertia, M_{θ} along with a damper on the disc makes the stiffness and the damping of the system, dependent on the frequency of the applied load; ΔM_{ϕ} displays the trapped mass moment of inertia to modify the effect of incompressibility of soil for the values of ν greater than 0.3; ρ, ν, V_s and V_p are mass density, Poisson's ratio, shear wave velocity and longitudinal wave velocity. r is the equivalent circular radius of the foundation and G the shear modulus of the soil which is considered 0.5 to 1 times of the soil initial shear modulus (G_0) depending on the site class and zone of seismicity. The shear modulus of the soil decreases when its strain increases during an earthquake.

The soil properties considered in this study are presented in Table 2.

2. 3. Modeling of Pounding Pounding created by the impact interaction of adjacent structures is a complex and non-linear phenomenon. Stereomechanical and contact element approaches are the main analytical methods for modeling the pounding phenomenon (approaches based on wave theory are also proposed). Stereomechanical approach known as "coefficient of restitution", attempts to model the dynamic poundings based on the instantaneous impact assumption. In this method, Momentum Conservation Principle and restitution coefficient are used to calculate the velocity of bodies after impact [32]. The assumption of instantaneous impact is not very accurate, especially when significant structural changes occur in the duration of the impact.

Contact element method is a force-based approach, which models the pounding phenomenon by spring and damper elements. Based on this approach, several linear and nonlinear pounding models are proposed for point-to-point one-dimensional modeling including linear spring model, linear viscoelastic model (Kelvin model), non-linear elastic model (Hertz model), non-linear viscoelastic model and Hertzdamp model with nonlinear damping [3, 21]. In the present study, Jankowski's nonlinear viscoelastic model was used to study the pounding effect in highway bridge structures. In this method, pounding force can be calculated using the following equation [21]:

$$F(t)=K_P\delta^{1.5}(t)+C_P(t)\dot{\delta}(t)\quad \text{for}\ \dot{\delta}(t)>0,\ \delta\geq 0 \tag{13a}$$

$$F(t)=K_P\delta^{1.5}(t)\quad \text{for}\ \dot{\delta}(t)<0,\ \delta\geq 0 \tag{13b}$$

$$F(t)=0\quad \text{for}\ \delta=(u_1-u_2-Gap)<0 \tag{13c}$$

where $F(t)$ is the pounding force, K_P the impact stiffness parameter, δ the distance between the two bodies (at the moment of impact, δ yields to the resulting deformations), and u_1 and u_2 are the displacement of the first and second bodies. Gap is the separation distance, $\dot{\delta}(t)$ the relative velocity between two bodies and C_P (t) is considered as the damping coefficient of pounding element obtained from the following equation [21]:

$$C_P(t)=2\bar{\xi}\sqrt{K_P\sqrt{\delta(t)}\frac{\bar{m}_1\bar{m}_2}{\bar{m}_1+\bar{m}_2}} \tag{14}$$

where $\bar{m}_1$ and $\bar{m}_2$ are the mass of two adjacent bodies and $\bar{\xi}$ is the damping ratio of the pounding element calculated from the following equation [22]:

$$\bar{\xi}=\frac{9\sqrt{5}}{2}\frac{1-e^2}{e\left(e(9\pi-16)+16\right)} \tag{15}$$

where e is the coefficient of restitution considered to be about 0.65 for the concrete materials or can be obtained from the following equation [23]:

$$e=-0.007\upsilon'^3+0.069\upsilon'^2-0.2529\upsilon'+0.7929 \tag{16}$$

where υ' is the relative velocity of the two bodies before the impact.

3. GROUND MOTION SPATIAL VARIATION

The characteristics of earthquake waves are altered in different stations due to: a) wave passage effect, i.e. the delay in the arrival of the waves to the stations away from each other, b) the incoherence effect due to the reflection or refraction of waves in the heterogeneous medium and the super-position of waves in the extended source, and c) site effects. The term "ground motion spatial variation" refers to the variation in the amplitude and phase of seismic movements of the ground which occurs as a result of wave propagation [33]. An accurate study of ground motion spatial variations is facilitated by installing of a set of arrays at different areas and

TABLE 2. The soil properties

Density ($\frac{kg}{m^3}$)	Poisson's ratio	$\frac{G}{G_0}$	G_0 ($\frac{N}{m^2}$)
1800	0.3	0.5	$10^6\times115$

distances in different soil types. The array SMART-1[2] is regarded as one of the most important arrays in this context which provided a large amount of data for various seismic events. This array including 37 accelerometers installed in the northeastern Taiwan in Lotung in 1980 [7, 33]. The coherency of the seismic movements between two stations i and j is obtained from the smoothed cross spectral density which is normalized with respect to the corresponding power spectral densities as follows [6]:

$$\gamma_{ij}(\omega)=\frac{S_{ij}(\omega)}{\sqrt{S_{ii}(\omega)S_{jj}(\omega)}} \tag{17}$$

Coherency is a complex function presented as follows:

$$\gamma_{ij}(\xi,\omega)=\left|\gamma_{ij}(\xi,\omega)\right|\exp\left[i\theta_{ij}(\xi,\omega)\right] \tag{18}$$

$$\theta_{ij}=\tan^{-1}\left(\frac{\mathrm{Im}\left[S_{jk}(\omega)\right]}{\mathrm{Re}\left[S_{jk}(\omega)\right]}\right)=-i\frac{\xi\omega}{V_a} \tag{19}$$

where ω is the angular frequency, ξ the distance between the two stations V_a the apparent velocity of the waveand S_{ii} and S_{jj} are the power spectral density function at stations of i and j, respectively. S_{ij} is the cross spectral density function (between two stations i and j), γ_{ij} the coherency function and Re and Im show the real and imaginary parts of the spectral density function, respectively. The absolute value of coherency displays the degree of relationship between two waves by a linear transfer function. This value is applied to express the similarities of the waves in two stations regardless of the arrival time indicating the random variations in ground motions. In the above complex expression, $\exp\left[i\theta_{ij}(\xi,\omega)\right]$ shows the passage effect of the wave.

Various equations are proposed to represent the ground motion spatial variations because of the available differences in recorded seismic data at different sites and various events as well as differences in data processing procedure. In the present study, Harichandran and Vanmarcke’s equation is used to represent the coherency between ground motions in different stations [34]:

$$\left|\gamma(\xi,\omega)\right|=A\exp\left(-\frac{2B|\xi|}{a\nu(\omega)}\right)+(1-A)\exp\left(-\frac{2B|\xi|}{\nu(\omega)}\right) \tag{20}$$

$$\nu(\omega)=k\left[1+\left(\frac{\omega}{2\pi f_0}\right)^b\right]^{-1/2};B=(1-A+aA) \tag{21}$$

where A, a , b , k , and f_0 are parameters which are calculated based on the events 20 and 24 (Events recorded by SMART-1). Based on the event 20, the values of these parameters used in equations 20 and 21 are obtained as follows:

A=0.736, a=0.147, b=2.78, k=5210 m, f_0=1.09 Hz

The apparent velocity of the wave is also considered to be about 500 m/s. Moreover, for the power spectral density function, the relationship given by Clough-Penzien is used as follows [35]:

$$S(\omega)=S_0\left[\frac{1+4\zeta_g^2\left[\frac{\omega}{\omega_g}\right]^2}{\left\{1-\left[\frac{\omega}{\omega_g}\right]^2\right\}^2+4\zeta_g^2\left[\frac{\omega}{\omega_g}\right]^2}\right]\times\left[\frac{\left[\frac{\omega}{\omega_f}\right]^4}{\left\{1-\left[\frac{\omega}{\omega_f}\right]^2\right\}^2+4\zeta_f^2\left[\frac{\omega}{\omega_f}\right]^2}\right] \tag{22}$$

in which S_0 is the power spectral density function of the white noise at bedrock, ω_g and ζ_g are the angular frequency and damping ratio of the first soil layer, respectively and ω_f and ζ_f the angular frequency and damping ratio of second soil layer which plays the role of a low frequency filter. In the present study, by assuming stiff soil, the values of these parameters are selected as illustrated in Table 3 [36]:

The effect of ground motions spatial variation on seismic response of the bridge structures is investigated in both time and frequency domains. Coherency models are used directly as an excitation function in frequency domain analysis. However, in these analyses, the nonlinear behavior of the structures is ignored. Deterministic time history approach is used to study the nonlinear behavior of structures and to analyze complex structural systems. In order to perform the time history dynamic analysis, various methods such as Hao, Abrahamson and Deodatis methods are used to produce the time history of the waves [36].

In the present research, 28 pairs of spatially varying accelerograms are produced for stations at the distance of 25, 50, 75 and 100 meters by using Deodatis method. The non-stationary accelerograms were produced by multiplying the artificial accelerograms by the Jennings envelope function [36]. The generated waves are anchored to the acceleration response spectrum for the soil type D[3], and are subjected to the Butterworth bandpass filter to remove the unwanted noises. Figure 3 illustrates two examples of the generated accelerograms.

TABLE 3. Values of the parameters used in Clough-Penzien power spectral density function

Soil type	ω_g (rad/s)	ω_f (rad/s)	ζ_g	ζ_f
Stiff soil	5π	0.5π	0.8	0.8

[2] Strong Motion Array in Taiwan

[3] Soil type D according to the ASCE-7 (183<V_s<366 m/s)

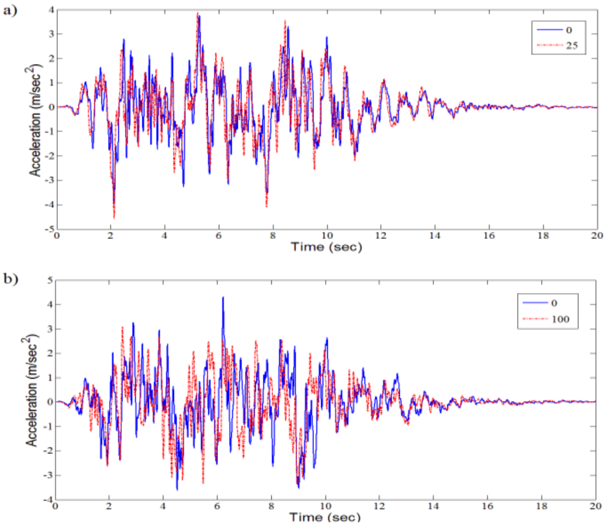

Figure 3. Two examples of the generated accelograms: a) the distance between stations 25 m; b) the distance between two stations 100 m

4. NUMERICAL STUDIES

In order to evaluate the effect of parameters affecting the pounding phenomenon, 144 different types of highway bridges are modeled with various periods, spans and height of piers. Each model was analyzed under the application of seven pairs of different accelerograms generated by the authors, and the average results of the number of impacts and the maximum pounding force were obtained.

4. 1. Analysis of Models under Uniform Seismic Excitation of Piers In this case, the models with various period, period ratio and pier height were subjected to the uniform seismic support excitation. Figures 4 and 5 illustrate the impact number and maximum pounding force for different bridges, respectively. As observed in these figures, no impact occurred under uniform excitation of the piers when the period and height ratios of the frames were unity (T_1/T_2=1 and H_2/H_1=1).

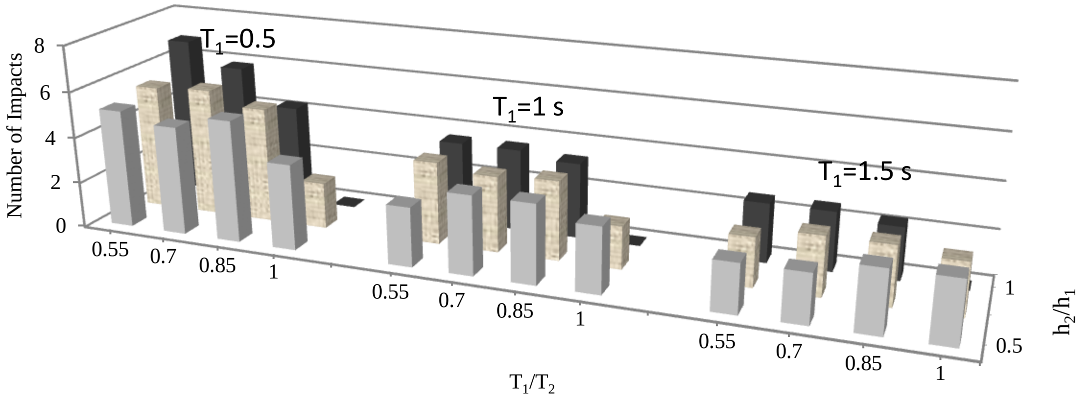

Figure 4. The number of impacts created in bridge in terms of period ratio (T_1/T_2) and piers height ratio (H_2/H_1) under uniform excitation in different periods

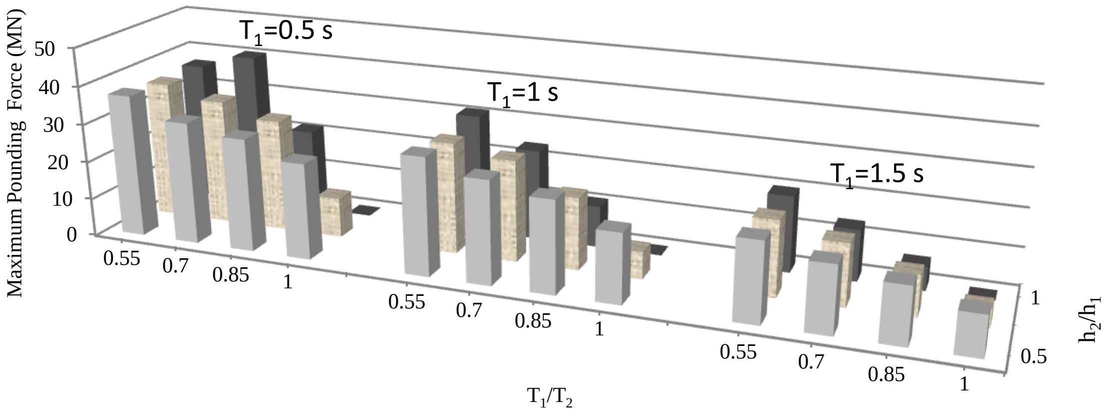

Figure 5. The maximum pounding force in bridges in terms of period ratio (T_1/T_2) and pier height ratio (H_2/H_1) under uniform excitation in different periods

An increase in the difference between the frames period resulted in increasing the impact number and the maximum pounding force in frames with equal height piers (H_2/H_1=1). Further, an increase in the difference between the height of piers, in frames with equal periods (T_1/T_2=1) led to an increase in the number of impacts and pounding force, while no clear behavior was observed in the frames with different periods. Furthermore, the comparison of the results indicated that the bridges with smaller period (T_1=0.5 S) hadhigher values of impact number and maximum pounding force.

4. 2. Analysis of Models with Equal Period and Pier Height Ratios (T_1/T_2=1 and H_2/H_1=1) under Non-Uniform Excitation of Piers Figures 6 and 7 represent the impact number and the maximum pounding force for models versus span length, respectively. All these models have equal period and pier height ratios. As illustrated in these figures, increasing the span length led to an increase in impact number and maximum pounding force, due to the reduction of the coherence between ground motions as a result of increasing the distance between piers and creating out-of-phase responses.

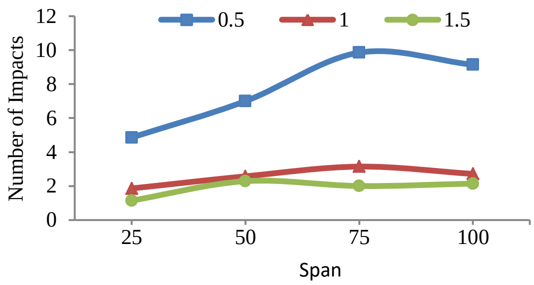

Figure 6. The number of impacts versus the distance between piers in the models with equal periods and piers height

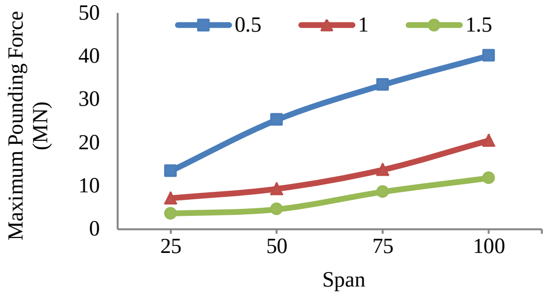

Figure 7. The maximum pounding force versus the distance between piers in the models with equal periods and piers height

4. 3 Analysis of Models under Non-Uniform Excitation of Piers All factors influencing out-of-phase responses are discussed in this section. Figures 8 and 9 illustrate the number of impacts with piers spacing (span) of 50 and 100 meters, respectively. Figures 10 and 11 display the maximum pounding force for the above bridges. Based on the results, by considering the effect of ground motion spatial variations, an increase in difference between periods or piers height does not necessarily result in increasing the values of the impact number and the pounding force. Further, no significant increase was observed in the values of these parameters by increasing the distance between piers if the system had out-of-phase responses due to the difference in periods or piers height. Accordingly, it is worth noting that suggesting a similar or close period for adjacent frames proposed by some seismic design codes such as Caltrans is appropriate for structures subjected to uniform excitation, while it is not valid for non-uniform excitation mode. Thus, it is recommended to adopt other strategies and perform seismic analyses based on the ground motion spatial variation effect in order to reduce the damage caused by the pounding in bridges.

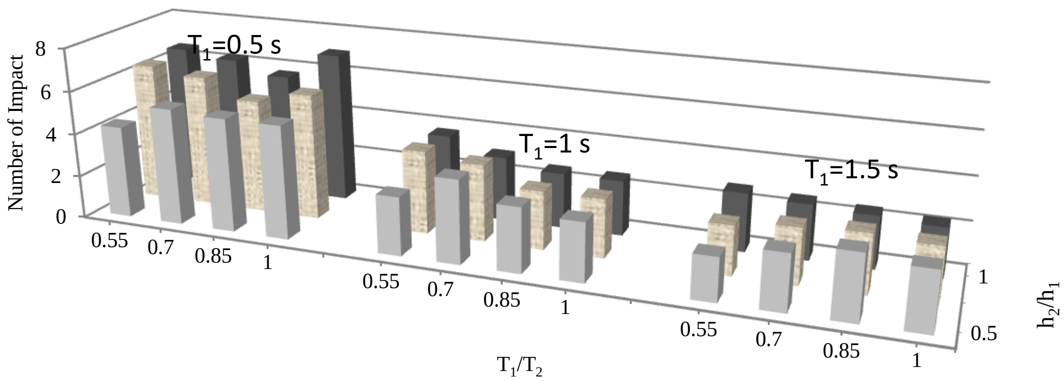

Figure 8. The number of impacts generated in bridges with different periods and piers spacing of 50 m versus on the period ratio (T_1/T_2) and pier height ratio (H_2/H_1)

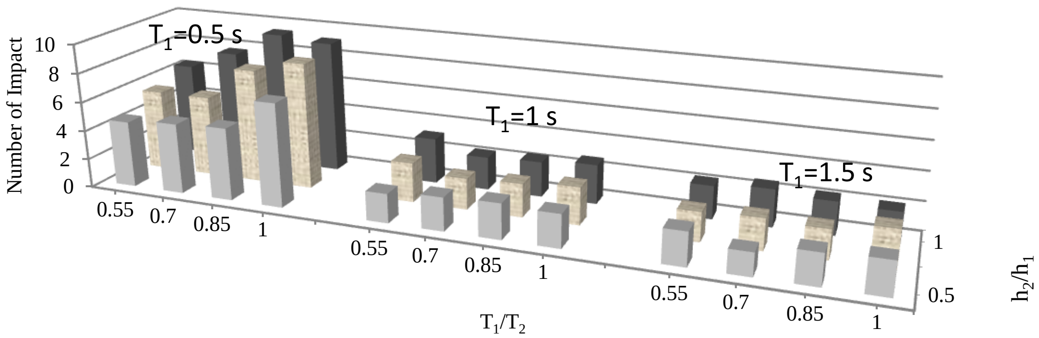

Figure 9. The number of impacts generated in bridges with different periods and piers spacing of 100 m versus the period ratio and pier height ratio

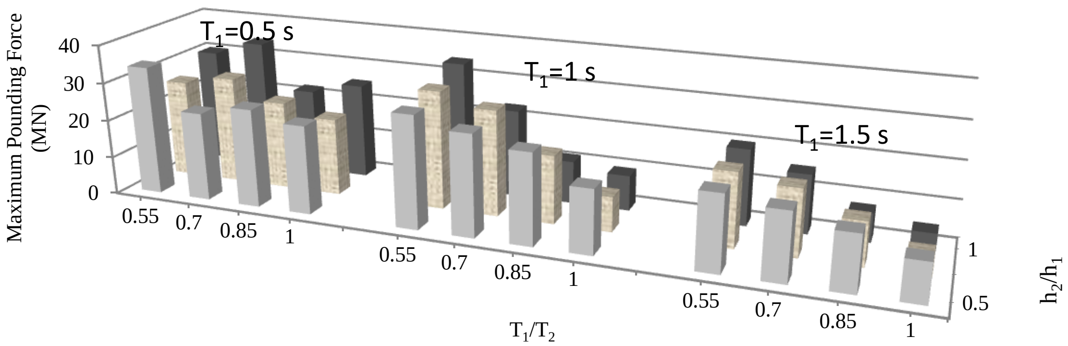

Figure 10. The maximum pounding force generated in bridges with different periods and piers spacing of 50 m versus the period ratio and pier height ratio

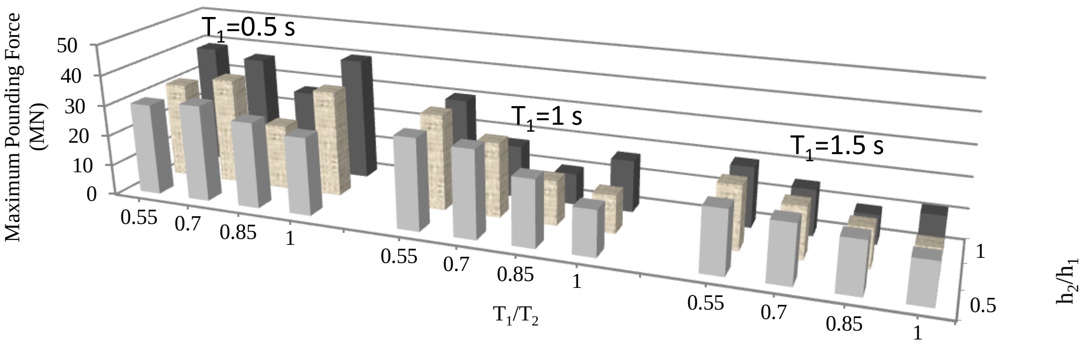

Figure 11. The maximum pounding force generated in bridges with different periods and piers spacing of 100 m versus the period ratio and pier height ratio

5. CONCLUSION

In the present study, the effects of parameters influencing out-of-phase responses and pounding phenomena in bridges were investigated by considering the effects of soil-structure interaction and ground motion spatial variation. Based on the numerical studies conducted in this research, the following conclusions were drawn:

First, an increase in the period of frames (T_1), resulted in decreasing the values of impact number and maximum pounding force.

Second, ignoring the effects of soil-structure interaction and ground motion spatial variation led to the calculation of unrealistic responses in the bridges.

Third, considering just one of the effective parameters on out-of-phase responses shows a clear trend on the resulting impact number and maximum pounding force values. In other words, an increase in the difference in the period, pier height and span length of the frames increases the impact number and maximum pounding force of the bridges. However, with respect to the interactive effects of parameters on each other, no similar trend was observed for the above values.

Fourth, designing the bridges based on the frames having similar or close periods is not regarded as a proper solution to reduce damage caused by the pounding. It can be attributed to out-of-phase responses generated as a result of soil-structure interaction and ground motion spatial variation effects.

Finally, an increase in the difference between the period of frames and height of piers did not necessarily increase the effects of pounding.

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An Investigation on the Parameters Influencing the Pounding in Highway Bridges

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چکیده

این مطالعه به بررسی اثر پارامترهای موثر بر ایجاد ضربه در پل‌ها می‌پردازد. پدیده ضربه در اثر برخورد بین دو بخش از عرشه، یا عرشه و پایه‌های کناری پل در محل درزهای انقطاع به علت پاسخ‌های غیر هم‌فاز در هنگام زلزله رخ می‌دهد. پارامترهای مورد بررسی شامل: نسبت دوره تناوب قاب‌های مجاورهم، حرکات ناهمگون زمین و اندرکنش خاک-سازه می‌باشند. بدین منظور ۱۴۴ نمونه مختلف از پل با تغییر در مشخصات پایه‌ها و اندازه دهانه‌ها ایجاد گردیده و تحت تحلیل‌های دینامیکی غیرخطی قرار می‌گیرند. نتایج نشان می‌دهد که عدم ملاحظه اثر اندرکنش خاک-سازه و حرکات ناهمگون زمین موجب محاسبه پاسخ‌های غیرواقعی در پل‌ها می‌گردد. همچنین، به علت پاسخ‌های غیر هم‌فاز ایجاد شده در اثر حرکات ناهمگون زمین و اندرکنش خاک-سازه، طراحی پل‌ها با قاب‌های دارای دوره تناوب مشابه یا نزدیک به هم، راه‌کار مناسبی برای کاهش خسارات ناشی از پدیده ضربه نمی‌باشد.

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