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Seismic performance of bridges to a spatially varying horizontal and vertical earthquake ground motion

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ABSTRACT: Considering the spatial variations of ground motions in the design of extended structures, especially bridges, is of importance. In this paper, the effect of spatial variations of the ground motions on bridges regarding the horizontal and vertical components of the earthquake was investigated. A five spans bridge is modeled in OpenSees and 3D nonlinear dynamic time history analysis is performed. The generation of acceleration time histories is in accordance to the spectral-representation-based simulation algorithm which has been presented in previous studies.

Seismic performance of the bridge was studied by considering the identical and differential support ground motions. Shear force, bending moment, displacement of bridge piers in identical and differential excitation supports with different soil conditions were analyzed.

The results showed that by considering the spatial variations of ground motions, internal forces made significant changes at the piers of the bridge. Based on the results by assuming the spatial variation of ground motion, bridge responses in piers will grow considerably; the axial, shear force and bending moment in the bridge piers calculated 1.87، 1.8, 1.97 times, respectively, compared to the identical support ground motion. Furthermore, the influence of soil type of the construction site has been investigated. The results illustrated that the non-homogeneous sites lead to the increase in axial force about 55% in the bridge piers.

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1. INTRODUCTION

During the past three decades, the study of the effect of the vertical components of earthquake on structures, especially bridges, has been the subject of many studies. Recent seismic studies have shown that the ratio of maximum vertical earthquake acceleration to maximum horizontal earthquake acceleration may be greater in near-field earthquakes compared to far-field ones.

Saadeghvariri and Foutch (1991) showed that changes in the axial forces of the bridge piers due to the vertical acceleration component of the earthquake destabilizes the hysteresis loops and increases the ductility demand [1]. The results of the Button et al. (2002) studies indicated that the vertical component of the ground motion has a significant effect on the axial load of the bridge piers and the vertical shear forces of the bridge deck [2]. Kim et al (2011), by studying the concrete bridge response, showed that the simultaneous application of the vertical and horizontal components of the ground motion has a significant effect on the response of the bridge piers at all levels and components. Therefore, they recommend that vertical component of earthquake should be considered in the analysis, evaluation and design [3].

On the other hand, the ground motions are natural phenomenon which varies over the time and space. The effect of ground motions on extended structures such as mat foundations, dams, bridges, transit systems and tunnels are not negligible. The differential support ground motion, compared to the identical support ground motion, can create different response and behavior in the structure, mainly in the bridge with simple supports.

The difference in vibration characteristics of the adjacent bridge structure and unequal support excitation can increase the relative displacement of these bridge structures and result in the damage through the pounding. Bo and Nowawi (2014) have shown that spatial variation of ground motion is one the main causes of relative displacement of adjacent bridge structures which cause pounding and girder unseating. This study demonstrated that spatial variation of ground motion could affect the formation and expansion of plastic joints at the bridge piers. The bridge's elastic analysis shows the high unseating potential [4].

Kaiming & Hao (2012) used numerical finite element simulation of pounding damage among bridge girder and the corresponding abutment in 3D two-span simply-supported bridge by considering the spatial variation of ground motions. Shreesta et al. (2015) investigated the seismic response of multiple-frame bridges in regards to spatial variation of ground motion and the interaction between the structure and the soil. The results indicated that differential support ground

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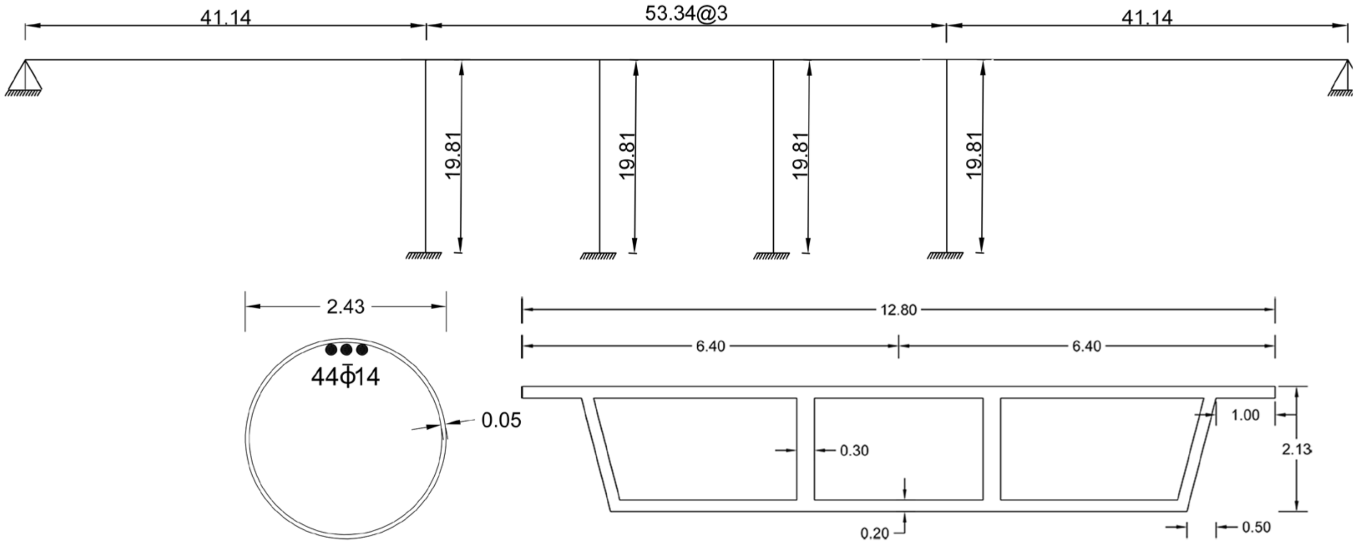


Fig. 1. Longitudinal and cross section of the bridge deck & pier of TY0H Bridge (Dimension in meters) [13]

motion and the interaction between soil and structure have a significant effect on the relative displacement of the adjacent bridge components [5]. The effect of the spatial variation of ground motion on the seismic crash of cable bridges was investigated by Zhong et al. (2017) [6]. The effect of this phenomenon was seen as a damage-exceedance probability index in the structure. The section ductility demand at the pylon is affected by the spatial variations of ground motion.

Experimental studies and three-dimensional finite element model (FEM) of pounding by considering the spatial variation of ground motion were investigated by He et al (2017). For the experimental model, two-bridges with a scale of 1:6 were designed. The results showed that non-uniform excitations and foundation torsion can dramatically increase relative displacements and pounding responses [7].

In order to evaluate the effect of spatial variation of ground motion on spatially extended structures, numerical studies were conducted by Özcebe et al. (2018) with Park field earthquake data in the UPSAR acceleration network. The spatial variations by using standard coherency functions even in conventional structures with a length of 300 meters on homogeneous stiff soil conditions can increase the engineering demands parameters by up to 50%, based on this study [8]. Falamarz-Sheikhabadi & Zerva, (2017) proposed a simplified differential displacement loading patterns to consider the effects of spatial variations of ground motions in seismic design codes and compared the results with the proposed European Seismic Design Code (EC). Based on the results, this fact that when the adjacent bridge piers move in different directions, the proposed European seismic loads can provides unrealistic responses for differential ground motions [9]. Stationary and transient analyzes of suspension bridges under the influence of spatial variations of ground motions with the consideration of the site's effect clearly showed that the responses of the bridges in a heterogeneous soil condition is more than homogeneous soil condition [10].

The present study investigates the response of bridge regarding the simultaneously effect of the vertical earthquake component and the spatial variation of ground motion in different sites condition with different Soil type. In this study a five spans bridge is modeled in OpenSees and 3D nonlinear dynamic time history analysis is performed. Earthquake accelerations have been generated based on spectral-representation-based simulation algorithm [11]. Considering this fact that Eurocode 8 [12] is one of the few design codes that in addition to including horizontal spectrum, presents the vertical component spectrum, this design code has been used for the design spectrum. Various parameters including shear force, bending moment, displacement of bridge piers in identical and differential excitation supports with different soil conditions has been studied.

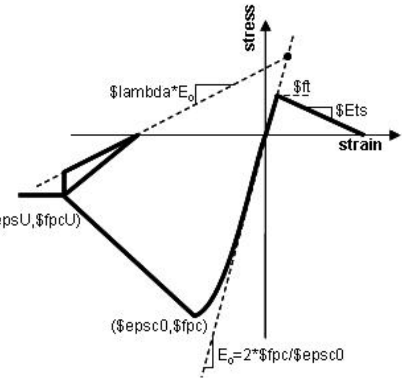
2. MODEL SPECIFICATIONS

The studied bridge is TY0H Highway Bridge, California, USA [13] which has five spans with a total length and width of 242.3 m and 12.80 m, respectively. The length of the first and the last span are 41.14 m and the length of the mid spans is 53.34 m. The cross section of the bridge is shown in Fig. 1. The bridge consists of four circular columns, diameter of 2.43 m, and height of 19.81 m.

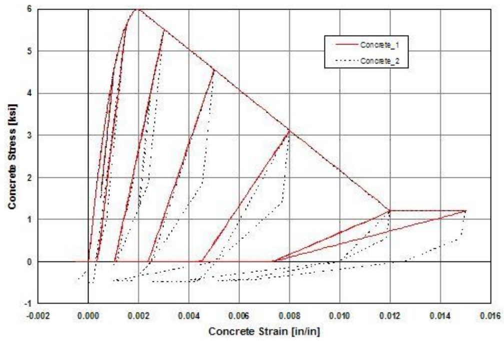
3. MODELING OF BRIDGE AND ANALYSIS

In this study, OpenSees software and nonlinear dynamic analysis with fiber elements have been used to investigate the nonlinear behavior of bridge piers. The TY0H Bridge is a reinforced concrete bridge with single column and the deck is constructed on precast concrete girders. The deck on the columns axis has no expansion joints. Rigid elements are used to connect decks and columns.

For accurate and reliable investigation of the bridge behavior, the 3D analysis method was used. Concrete behavior



(a)



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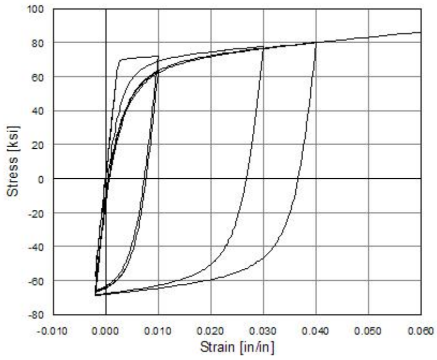

(c)

Fig. 2. (a) Stress-strain relation and (b) hysteresis behavior of "Concrete02" (c) uniaxial Giuffre-Menegotto-Pinto model "Steel02" (OpenSees)

is modeled by a uniaxial material object with tensile strength and linear tension softening (Concrete02) and in compression is defined by a maximum compressive strength f_{pc} for the strain ε_{co} and the residual strength f_{pcu} achieved at the ultimate strain ε_{cu}. Part of the relation that describes the tensile behavior is determined by the maximum tensile strength f_t and the slope coefficient that determines the decrease of the tensile strength E_{ts}. Model Concrete02 has typical hysteresis behaviors shown in Fig. 2. Steel behavior is represented by a uniaxial Giuffre-Menegotto-Pinto model (Steel02).

The best method to distribute the mass between the elements of the bridge is distribution in regards to the length of the elements. Transitional masses in longitudinal, transverse, and vertical directions were assigned to nodes based on their effective length. To evaluate the distributed mass between elements with concentrated mass in the nodes, the sufficient number of nodes in the model is defined.

To model the deck, the linear beam-column element was used and a 3D fiber nonlinear Beam-Column Element was applied to model the columns in the OpenSees software.

The longitudinal and transverse periods of model respectively are 1.43 and 1.72 second which have difference of 1.4% and 1.7% with Kim & Feng (2003)[14].

4. SIMULATION OF GROUND MOTION COMPATIBLE WITH THE RESPONSE SPECTRUM

The method that is used in this study to produce ground motion acceleration time histories is a simulated algorithm based on the design spectrum which is compatible with three quantities: response spectrum, coherence function and module function [11]. By this method and according to the spectral density matrix, acceleration, speed, and displacement time histories in different situations on the surface of ground are produced. This method is based on the algorithm which proposed by Shinozuka (1987) [15], Shinozuka-Deodatis (1988)[16] and Li and Kareem (1991) [17].

According to the proposed method, the time histories of acceleration in *n* nodes on the ground [11, 18] were formulated with a non-stationary stochastic vector process with *N* variable. To do this, a spectrum of target acceleration response for *n* nodes was considered ($RSA_j(\omega)$, j = 1,2, …, n). In addition, complex coherence functions (j, k = 1, 2... n, j ≠ k) and module functions ($A_j(t)$, j = 1, 2, …, *n*) were assigned between any two nodes and at each node, respectively. Afterwards, simulated time history accelerations were created according to the repetitive plan, shown in Fig. 2. This procedure was repeated as long as it converged. In most cases, less than 10 repetitions are required to achieve

Table 1: outline for simulating of time history acceleration compatible with response spectrum at n points on the free field [11]

Input Data: Objective acceleration response spectrum $RSA_j(\omega)$; $j = 1,2,\dots,n$ Coherence functions $\Gamma_{jk}(\omega)$; $j,k = 1,2,\dots,n$; $j \neq k$ Module functions $A_j(t)$; $j = 1,2,\dots,n$		
Creation of spectral density $S_j(\omega)$ as a non-zero constant value		
Production of $g_j(t)$; $j = 1,2,\dots,n$ as a random vector with n variable and with power spectral density (PSD) matrix		
Calculation of the time history accelerations as: $f_j(t) = A_j(t).g_j(t)$, $j = 1,2,\dots,n$		
Calculation of the acceleration response $RSA^{(f_j)}(\omega)$ spectrum according to $f_j(t)$, $j = 1,2,\dots,n$		
Repeat it?	No.	Finish
	Yes	
Improve the PSD function: ,$j=1,2,\dots,n$ $S_j(\omega)$ $S_j(\omega)$ $S_j(\omega) \rightarrow S_j(\omega)\left[\frac{RSA_j(\omega)}{RSA^{(f_j)}(\omega)}\right]^2$		

an accurate convergence at each frequency. Multiplying the stationary time histories to proper envelope function results in non-stationary time histories. Thus, each non-stationary time history is generated independently with respect to the described response spectrum.

Daudatis' (1996) proposed method is to improve the power spectral density functions of vector processes which generate new stationary time histories in respect to the improved power spectral density matrix and the multiplication of these functions to envelope functions resulted in new non-stationary time histories. This improving method will be repeated several times until simulated time histories become compatible with the required response spectrum (table 1) [11].

The acceleration and displacement time histories used in this study were generated by MATLAB software. The ground motion time histories in this study are compatible with the Europe code 8 (EC8) design spectrum. The time history accelerations at six nodes on the ground (Fig. 3) along the main wave propagation line were considered as a non-stationary random vector with six variables.

To generate these acceleration time histories, free surface motion in different locations on the ground surface was assumed as random (Equation 1)

$$f_j(t) = A_j(t).g_j(t) \tag{1}$$

Where $f_j(t)$ is a vector with n components containing of non-stationary free field acceleration and $A_j(t)$, and $g_j(t)$ are the modulus function and component vector which contain random stationary accelerations, respectively.

Eq. 2 shows the Abrahamson model [19]for coherence function of γ between two acceleration time histories of $f_j(t)$ and $f_k(t)$ when $j \neq k$.

$$\gamma_{jk}(\omega) = \frac{1}{1+\left[\frac{\omega}{2\pi c_8(\xi_{jk})}\right]^6} \times \tanh\left\{\frac{c_3(\xi_{jk})}{1+\frac{\omega}{2\pi}c_4(\xi_{jk})+\frac{\omega^2}{4\pi^2}c_7(\xi_{jk})} + \left[4.8 - c_3(\xi_{jk})\right]\exp\left[c_6(\xi_{jk})\frac{\omega}{2\pi}\right] + 0.35\right\} \tag{2}$$

Where ξ_{jk} is the distance between point j , k and:

$$c_3(\xi_{jk}) = \frac{3.95}{\left(1 + 0.0077\xi_{jk} + 0.000023\xi_{jk}^2\right)} + 0.85\exp\{-0.00013\xi_{jk}\}$$

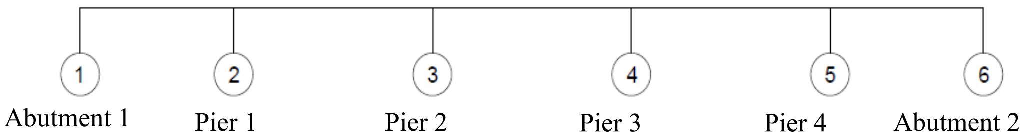

Fig. 3. Layout of stations on the ground compatible with bridge abutment and piers to simulate time histories

$$c_4(\xi_{jk}) = \frac{0.4\left[1 - \dfrac{1}{1+\left(\dfrac{\xi_{jk}}{5}\right)^3}\right]}{\left[1+\left(\dfrac{\xi_{jk}}{190}\right)^8\right]\left[1+\left(\dfrac{\xi_{jk}}{180}\right)^3\right]}$$

$$c_6(\xi_{jk}) = 3\left(\exp\left\{-\frac{\xi_{jk}}{20} - 1\right\}\right) - 0.0018\xi_{jk}$$

$$c_7(\xi_{jk}) = -0.598 + 0.106\ln(\xi_{jk} + 325)$$

$$- 0.0151\exp\{-0.6\xi_{jk}\}$$

$$c_8(\xi_{jk}) = \exp\{8.54 - 1.07\ln(\xi_{jk} + 200)\} + 100\exp\{-\xi_{jk}\}$$

In order to control the duration of strong ground motion, Jennings et al (1968) [20] proposed a model for the modulus function (Eq. 3).

$$A_j(t,\omega) = A_j(t) = a_1 t\exp(-a_2 t), \quad j = 1,2,\ldots,6 \quad a_1 = 0.68, \; a_2 = 0.25 \tag{3}$$

The spectral density function takes the power of $S_j(\omega)$, j=1,2,…,6 which is equal to a constant value in the frequency range. This value arbitrarily is selected equal to Sj(ω)= 100 cm^2/s^3 j= 1,2,…,6. The simulation was run in 800 time steps (each time step 0.025 sec) and peak ground acceleration (PGA) was considered to 0.35g. The response spectrum of horizontal and vertical components of ground motions with a 5% damping ratio for soil type B according to the EC8 is defined. The generated acceleration records at nodes 1 to 6 for horizontal axe are illustrated in Fig. 4.

To validate the generated acceleration records, adaptation between the spectrum of the EC8 and the calculated response spectrum is controlled. As an instance, this comparison is shown in Fig.s 5 and 6 for some nodes in the horizontal and vertical axes which validate the compatibility of these two spectra.

5.The effect of spatial variations of ground motions on the bridge response

In this study, to analyze the seismic response of the bridge, two different cases have been considered:

Case 1: All bridge supports are located in homogeneous soil condition (Type B) and the bridge response with and without considering the spatial variations of ground motions are compared.

Case 2: bridge supports are located in different soil condition. In this case, some of bridge supports are located in soil type B and some others are located soil type C.

In each case, the seismic performance of the bridge under simulations action of horizontal and vertical components of earthquake with and without considering the spatial variation of ground motion is evaluated. To evaluate the responses obtained from the dynamic time history analysis in two conditions (identical and differential support ground motion) equation 4 can be applied.

$$\alpha = \frac{\text{bridge response under differential ground motion}}{\text{bridge response under identical ground motion}} \tag{4}$$

For α value greater than 1, the calculated response value in differential support ground motion condition is more than uniform support ground motion condition.

For first case, the maximum of axial and shear force and bending moment in the middle and side piers of the bridge by considering the horizontal and the vertical components of earthquake with and without the spatial variations of ground motions are shown in Fig.s 7 to 9.

The maximum axial, shear and bending moment ratios in the TY0H Bridges in both cases, identical and differential support ground motion, have been mentioned in table 2.

The results indicate that the characteristics of the spatial variations of ground motions can greatly affect the bridge responses. Zanardo et al. (2002) [21] reported that by considering the spatial variation of ground motion the internal forces of bridge piers would almost double. It should be noted, that in the present paper the effect of simultaneous action of the vertical and horizontal components of strong ground motion with the assumption of spatial variations of those component are also taken into account.

The second case: In order to investigate the effect of spatial variation of ground motion and site conditions (soil type), it has been assumed that some of the bridge piers are placed in soil type B and some other in soil type C. Then, by considering the spatial variations of ground motion, the seismic response of the bridge was examined. It was assumed that the side piers were placed in a harder soil and the middle piers were placed in a softer one. Table 3 shows the maximum axial force ratio of bridge piers with/without the assumption of spatial variations of ground motion.

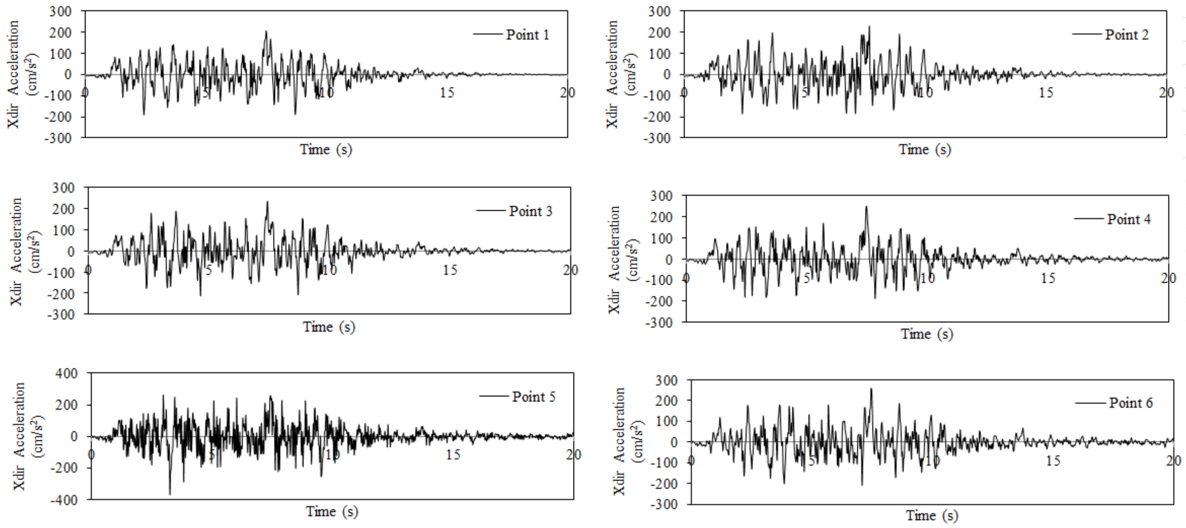

Fig.4. Generated time histories of horizontal acceleration at stations 1 to 6

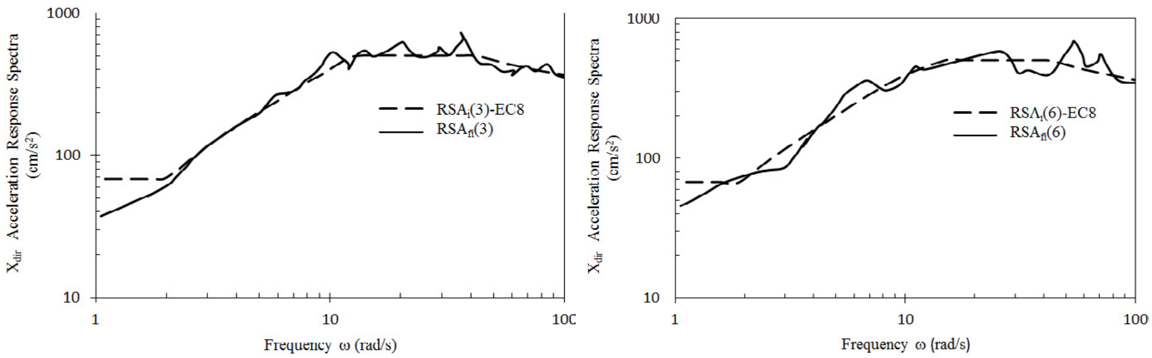

Fig. 5. Comparison of EC8 and response spectrum of generated horizontal time histories

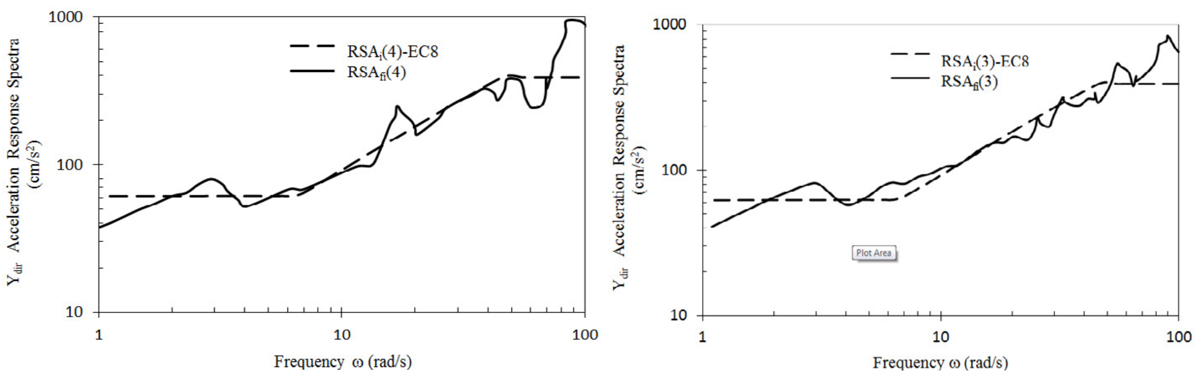

Fig. 6. Comparison of EC8 and response spectrum of generated vertical time histories

5. CONCLUSION

In this study, the seismic response of bridges under silmultamious action of the horizontal and vertical components of ground motion with assumption of spatial variation of ground motion has been investigated. To generate the acceleration records of the ground motion with considering the effect of spatial variations, the simulated algorithm was based on the design spectrum. The design spectrum of EC8 was applied for this study. Also, the synchronic effects of differential support ground motion and soil type on response of piers were studied.

1. Results showed that the differential support ground motions have significant effects on the seismic response of the extended bridges. Based on the results, by considering the effect of spatial variation of ground motion, the maximum of axial force, shear force and bending moment of the bridge piers increased by 1.87, 1.8 and 1.97 times, respectively compare to the response under identical ground motion.

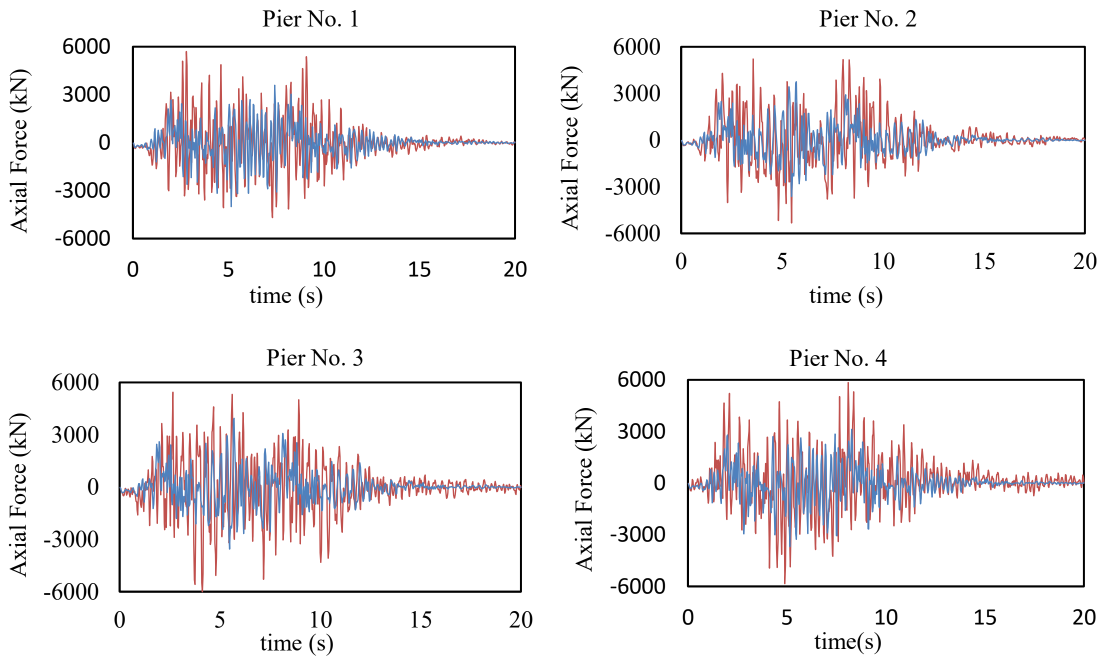

Fig. 7. Axial force of bridge piers under identical (blue line) and differential (red line) support ground motion in the case 1

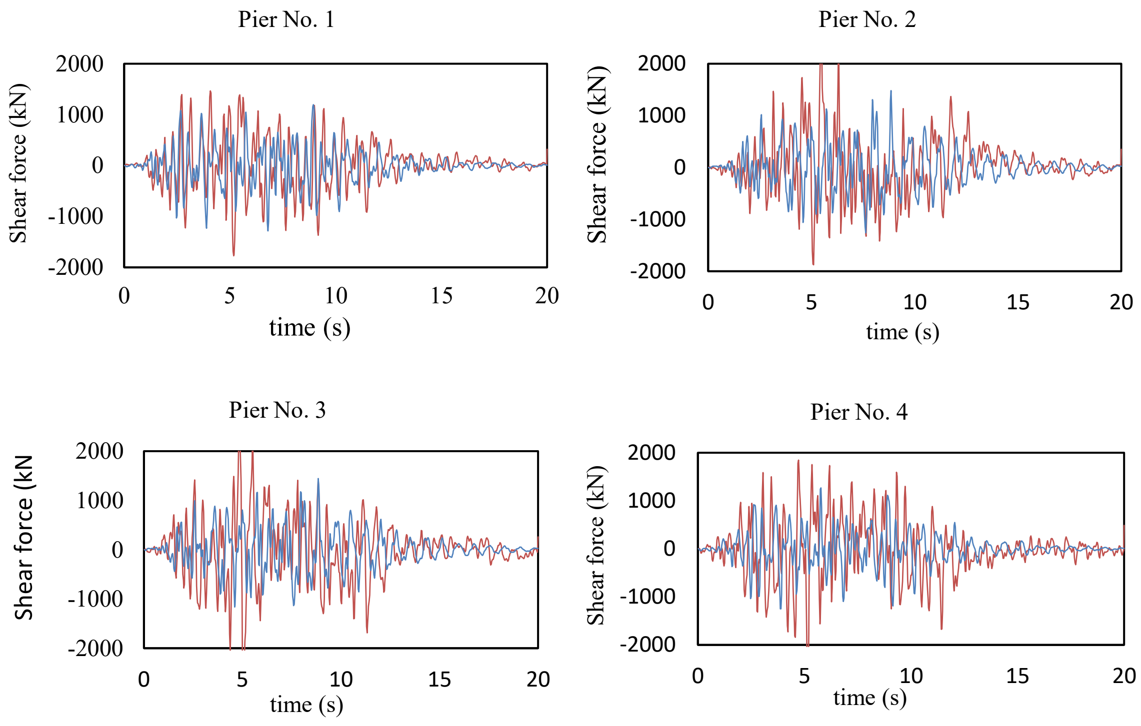

Fig. 8. Shear force of bridge piers under identical and differential support ground motion in the case 1

2. The results indicate that the spatial variations of ground motion can boost the response of bridge piers by twice.

3. In the other case, the responses of the bridge were evaluated under different site conditions (different soil type) and due to spatial variation of ground motion. In this case, the middle piers placed on the softer soil compared to the side piers. Comparing the axial forces with and without considering spatial variation in the case of different soil type illustrated that the ratios of maximum axial forces in different bridge piers (pier No. 1 to 4) are 1.85, 2.16, 2.6 and 1.93 times.

4. The results showed that assuming different soil type can increase axial force of the bridge piers even up to 55% comparing to homogeneous soil condition.

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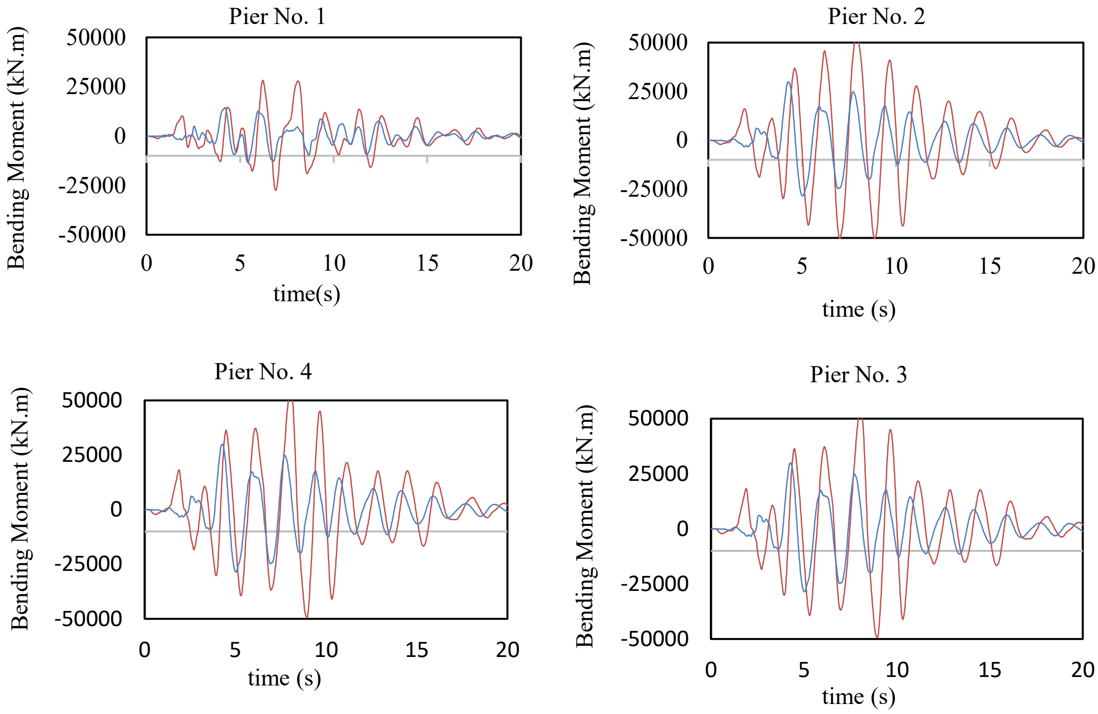

Fig. 9. Bending Moment of bridge piers under identical (blue line) and differential (red line) support ground motion in the case 1

Table 2. Maximum ratio of axial fore, shear force and bending moment of bridge piers under identical and differential support ground motion in the case 1.

Pier No.	Pier No. 1	Pier No. 2	Pier No. 3	Pier No. 4
Axial force ratio	1.59	1.39	1.38	1.87
Shear force ratio	1.23	1.80	1.57	1.46
Bending moment ratio	1.97	1.78	1.76	1.66

Table 3. Maximum ratio of axial fore of bridge piers under identical and differential support ground motion in the case 2.

Pier No.	Pier No. 1	Pier No. 2	Pier No. 3	Pier No. 4
α	1.85	2.16	2.06	1.93

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