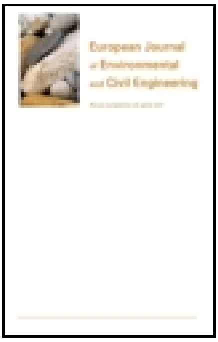

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Seismic response evaluation of bridges under differential ground motion: a comparison with the new Algerian provisions

Nassima Benmansour [a], Mustapha Djafour [a], Abdelmalek Bekkouche [a], Djawad Zendagui [a] & Abdelhakim Benyacoub [a]
[a] RISAM, Department of Civil Engineering, University of Tlemcen, Tlemcen, Algeria

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Nassima Benmansour*, Mustapha Djafour, Abdelmalek Bekkouche, Djawad Zendagui and Abdelhakim Benyacoub

RISAM, Department of Civil Engineering, University of Tlemcen, Tlemcen, Algeria

Seismic response of extended structures, such as bridges, must take into account spatial variability of earthquake ground motion (SVGM). Eurocode 8 (EC8) and the Algerian bridge seismic regulation code (*Règles Parasismiques applicables au domaine des Ouvrages d'Art*; RPOA) are among the rare bridge design codes that introduced simplified approaches for SVGM. This paper aims at evaluating the accuracy of the method proposed by the RPOA through comparison with more refined approaches and the EC8 provisions. Various bridges are considered and the results show that the RPOA's simplified method does not give satisfactory results and clearly overestimates the seismic demand. The solution proposed in this paper is to modify some provisions, and the new approach gives results which are in better agreement with the other methods.

L'effet de la variabilité spatiale du mouvement sismique (SVGM) doit être considéré lors du calcul sismique des structures étendues, telles que les ponts. L'Eurocode 8 (EC8) et récemment le règlement parasismique Algérien des ouvrages d'art (RPOA), sont parmi les rares codes de conception de pont à proposer des approches simplifiées pour tenir en compte de la SVGM. L'objectif de cet article est d'évaluer la méthode proposée par le RPOA en la comparant avec des approches plus raffinées et avec les dispositions de EC8. Pour cela, différents ponts ont été étudiés et les résultats obtenus montrent que la méthode simplifiée du RPOA ne donne pas des résultats satisfaisants dans le sens où elle surestime la demande sismique. La solution proposée dans cet article est d'en modifier quelques dispositions pour obtenir des résultats semblables aux autres méthodes.

Keywords: bridges; dynamic analysis; earthquakes; spatial variability; RPOA

Mots-clés: ponts; analyse dynamique; séismes; variabilité spatiale; RPOA

1. Introduction

Seismic response of extended structures, such as bridges, must take into account spatial variability of earthquake ground motion (SVGM) which can be regarded as an asynchronous ground motion input. The sources of this variation have been identified (Der Kiureghian, 1996) as: (a) wave passage effect, (b) wave coherency loss, and (c) local site effects. During recent decades, models that describe SVGM have been developed based on either empirical or analytical approaches and it has been widely accepted that the coherency function describes the SVGM. Using these models, structural analyses have been performed on numerous structures and have shown the importance of taking

*Corresponding author. Email: benmansournassima@yahoo.fr

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SVGM into account (Burdette & Elnashai, 2007; Lou & Zerva, 2005; Lupoi, Franchin, Pinto, & Monti, 2005; Saxena, 2000; Sextos & Kappos, 2009; Sextos, Kappos, & Mergos, 2004; Wang, Carr, Cooke, & Moss, 2009).

However, the complexity of the problem has not yet permitted the development of specific design provisions that are widely accepted. In most modern codes, the impact of SVGM appears clearly only in the seating length provisions. Eurocode 8 (EC8) (CEN, 2005a) and, more recently, the Algerian seismic code (*Règles Parasismiques applicables au domaine des Ouvrages d'Art*; RPOA) (MTP, 2010) are among the rare codes that provide detailed procedures for taking into account the effect of SVGM in bridge design. Basically, they propose to combine pseudo-static and dynamic components to simulate the effect of SVGM. The dynamic component is obtained from 'traditional' spectral analysis by using the 'classical' pseudo-acceleration response spectrum.

The main objective of this paper is to quantitatively assess these two simplified approaches and to compare them with more refined solutions, i.e. dynamic time history analyses. Theoretically, linear SVGM time history analyses can be conducted by imposing either accelerations or displacements at the bridge support points. However, most FEM (Finite Element Method) software only allows spatially variable displacements to be prescribed; giving the unique possibility to conduct nonlinear SVGM time history analyses.

For the purpose of this study, it is therefore important to simulate seismic motions which enable 'comparable' analyses to be performed, i.e. the response spectra calculated from imposed acceleration and from imposed displacements have to be comparable with the code's spectrum. Basically, the conditional simulation of the seismic input, starting from the code's response spectrum, is achieved by using the well-established method of Deodatis (1997), whereas SVGM is described by an empirical coherency function, namely the coherency model of Harichandran and Vanmarcke (1986). To evaluate the way the aforementioned seismic codes treat the consideration of asynchronous ground motion during the seismic design of bridges, three bridges having different lengths and seating on four types of site conditions are considered. For each case of bridge/site, five types of linear analyses are conducted and the results are compared in terms of demand for internal forces.

2. Conditional simulation of spatially varying ground motions

2.1. SVGM model

SVGM is usually expressed in the literature by the complex coherency function. The complex coherency function describes the correlation between the amplitudes and phase angles of two ground motion time histories in the frequency domain. This function is defined as (Der Kiureghian, 1996):

$$\gamma_{jk}(\omega) = \frac{S_{jk}(\omega)}{\sqrt{S_j(\omega S_k(\omega))}}, \tag{1}$$

where ω is the circular frequency; $S_j(\omega)$, $S_k(\omega)$ are the power spectral density functions of the time histories $g_j(t)$ and $g_k(t)$, respectively; and $S_{jk}(\omega)$ is the cross-power spectral density of the considered time histories.

The coherency function can be rewritten as (Der Kiureghian, 1996):

$$\gamma_{jk}(\omega) = |\gamma_{jk}(\omega)|\exp\left(i - \frac{\omega d_{jk}}{v}\right), \tag{2}$$

where d_{jk} is the projected horizontal distance along the direction of propagation of the waves, which is from station j to station k, and v is the surface apparent velocity of waves, considered as constant over the frequency range of the wave. In general, sensitivity analyses conducted for the evaluation of the SVGM effect on the response of bridges assume a range of apparent propagation velocities from very low (a few hundred m s^{-1}) to infinity, thus reflecting propagation velocity characteristics of different types of waves (surface or body waves).

In Equation (2) $|\gamma_{jk}(\omega)|$ is the absolute coherency function, also called the coherency function. It describes the incoherence effect between stations having a separation distance of d_{jk}.The exponential term represents the wave passage effect. It should be noted that the wave passage effect can be incorporated by a time shift.

2.2. Simulation of stationary time series

A stationary time series implies that the stochastic descriptors of the motions do not depend on absolute time, but are functions of time differences only.

In this paper the stationary time series are simulated using the method described by Deotatis (1997) which is as follows.

From Equation (1), the cross-spectral density matrix $S_0(\omega)$ for the stationary process g_j(t); j = 1,2,...,n. is given by:

$$S_0(\omega) = \left[S_j(\omega)S_k(\omega)\gamma_{jk}(\omega)\right]; \quad j,k = 1,2,...,n. \tag{3}$$

In order to simulate samples of the n-variant stationary stochastic process $g_j(t)$; j = 1,2,..., n its cross-spectral density matrix $S_0(\omega)$ given in Equation (3) is factorised into the following product using Cholesky's decomposition method:

$$S_0(\omega) = H_0(\omega)H^{T*}(\omega). \tag{4}$$

The elements of $H(\omega)$ can be written in polar form as:

$$H_{jk}(\omega) = |H_{jk}(\omega)|\exp\left(i\theta_{jk}(\omega)\right); \; j>k, \tag{5}$$

where:

$$\theta_{\boldsymbol{jk}}(\omega) = \boldsymbol{tan}^{-1}\left(\frac{\boldsymbol{Im}[\boldsymbol{H_{jk}}(\omega)]}{\boldsymbol{Re}[\boldsymbol{H_{jk}}(\omega)]}\right). \tag{6}$$

Using Equations (5) and (6) the stationary stochastic vector process g_j(t); j = 1,2,...,n can be simulated by the following series as $N \rightarrow \infty$.

$$g_j(t) = 2\sum_{m=1}^{n}\sum_{l=1}^{N}|H_{jm}(\omega)|\sqrt{\Delta\omega}\,cos(\omega_l - \theta jm(\omega_l) + \Phi_{ml}); \tag{7}$$

where:

$$\omega_l = l\Delta\omega;\ l = 1, 2, ..., N \tag{8}$$

$$\Delta\omega = \frac{\omega_u}{N}; \tag{9}$$

N represents the number of the frequency step $\Delta\omega$ needed to reach the upper cut-off frequency ω_u.

The $\{\phi_{ml}\}; m = 1, 2, ..., n; \quad l = 1, 2, ..., N;$ appearing in Equation (7) are n sequences of independent random phase angles distributed uniformly over the interval $[0, 2\pi]$.

2.3. Simulation of non-stationary time series

An extensive list of publications addressing the topic of simulations of non stationary time series has appeared in the literature. A number of these studies generate stationary random fields and apply an envelope modulation function to obtain non-stationary time series (cf. Zerva & Zervas, 2002). However it is widely recognised that these envelope functions disturb the properties of the simulated accelerograms (Shama, 2007) and affect the structural response (Zerva, 2009).

Non-stationary seismic ground motions can also be generated from predefined acceleration time history using time and/or frequency-domain segmentation (cf. Heredia-Zavoni & Santa-Cruz, 2000; Liao and Zerva, 2006; Shama, 2007). In this study, the predefined or ‘reference’ time history includes synthetic accelerograms compatible with the RPOA spectrum. The time series are divided into nearly stationary segments with different durations. Each segment is used as a reference time series and stationary conditional simulations are carried out for each segment using the methodology defined in §2.2. The stationary conditionally simulated segments are joined together to obtain the entire non stationary and spatially variable acceleration time histories. A time shift is incorporated for the wave passage effect.

However, when considering SVGM, FE (Finite Element) dynamic analysis is generally performed using the displacement time histories as an input. In other words, the displacement time histories need to be evaluated from the spatially variable simulated accelerations. But experiences show that direct integration of the acceleration data often causes unrealistic drifts in the derived velocity and displacement. A correction scheme must be used to ensure compatibility between simulated accelerations, velocities and displacements time histories, i.e. their properties must be compatible in terms of power spectral density function, peak of displacement and response spectrum. For these purposes, a computer tool, RISAM (Benmansour, 2004; Boukli Hacene & Rachedi, 2010), was developed to automatically derive the displacement sets required for FE analyses.

3. Overview of the RPOA and EC8 provisions regarding SVGM

3.1. RPOA provisions

The RPOA (MTP, 2010) is the first Algerian code for the seismic design of bridges. It clearly recognises that the differential ground motion can induce significant additional stresses in bridge structures and that their seismic design cannot be based only on the effects of uniform motion. RPOA is one of the few seismic codes that provide a detailed set of guidelines for explicitly tackling the problem of SVGM on the seismic response of bridges.

According to RPOA, the effects of asynchronous ground motion are negligible except when one of the following conditions applies: (a) the structure crosses an active fault; (b) the soil properties vary along the bridge; (c) the length of the bridge is very important. In this case, an evaluation of the effect of SVGM on the structural response is to be performed. In the simplified method, the code recommends that the designer first conducts a dynamic analysis of the structure under uniform seismic excitations, using spectral analysis. The second step is to achieve a pseudo-static analysis based on a pattern of prescribed differential displacements at the bridge supports. Finally, the results are combined using the SRSS (square root of the sum of squares) rule.

On a site without marked mechanical discontinuity, the prescribed differential displacement, denoted d at a support of the bridge is given by (MTP, 2010):

$$d = \eta A g X \qquad \text{for} \quad X < L_M \tag{10}$$

$$d = A g D_M \sqrt{2} \qquad \text{for} \quad X \geq L_M, \tag{11}$$

where X is the horizontal distance of the support from the reference support measured in the longitudinal direction;

$$\eta = \frac{D_M}{L_M}\sqrt{2}; \tag{12}$$

Ag is the design seismic acceleration on type S1 ground; g is the acceleration of gravity; L_M is the distance beyond which the motions of the two supports can be regarded as independent; and D_M are absolute displacements; they are given for unit acceleration (1 m s^{-2}).

The values of D_M and L_M are given in Table 1 for the four ground types in RPOA, S1 to S4, which are classified on the basis of the shear wave velocity V_S.

Figure 1 provides an example of bridge with four supports subjected to differential displacements d. If the bridge supports are sited on the same ground type but are located on both sides of a marked topographic discontinuity (valley), and in absence of more rigorous assessment, the value of d is increased by 50%.

If two bridge supports are located on both sides of a marked mechanical discontinuity, the displacement d is given by Equation (13) (MTP, 2010):

$$D = Ag\sqrt{D_{M,1}^2 + D_{M,2}^2}, \tag{13}$$

Table 1. Values of L_M and D_M (MTP, 2010).

Ground type	S1	S2	S3	S4
$V_s \cdot \left(\frac{m}{s}\right)$	$V_s \geqslant 800$	$400 \leqslant V_s \leqslant 800$	$200 \leqslant V_s \leqslant 400$	$V_s \leqslant 200$
$L_M \cdot (m)$	600	500	400	300
$D_M \cdot (m)$	0.03	0.05	0.07	0.09

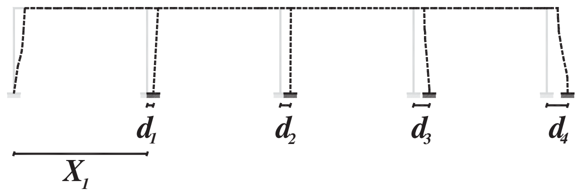

Figure 1. Example of a bridge subjected to differential ground motion *d*.

where $D_{M,1}$ and $D_{M,2}$ are the absolute displacements at supports 1 and 2, respectively.

3.2. EC8 provisions

EC8-Part2 (CEN, 2005a) specifies that SVGM should be considered in the design of bridge with a continuous deck when one or both of the following two conditions apply: (a) the soil properties vary along the bridge and there is more than one ground type supporting the bridge; (b) the soil properties along the bridge are approximately uniform, but the length of the deck exceeds the limiting length L_{lim}; the recommended value of L_{lim} is equal to $L_g/1.5$, where L_g is the distance beyond which the motions can be considered as independent. The values of L_g are presented in Table 2 for the various ground types of EC8-Part1, A to E (CEN, 2005b).

EC8 provides a detailed framework for considering the effect of SVGM in bridge design, prescribing a simplified approach. In the general case the latter should be followed, unless a more accurate analysis is carried out. To this end, an analytical method is presented in an informative annex to the code.

In the simplified method, the code recommends that the designer conduct first a dynamic analysis of the structure caused by uniform seismic excitations based on prescribed response spectra, then a pseudo-static analysis based on two patterns of prescribed displacements at the bridge supports, termed 'Set A' and 'Set B'. Finally, the dynamic response is combined with the worst scenario pseudo-static response by means of the SRSS rule. The two displacement sets are as follows:

Set A consists of application of simultaneously relative displacements d_{ri} with the same sign at all piers in the horizontal direction; d_{ri} is given by Equation (14) (CEN, 2005a):

$$d_{ri} = \varepsilon_r L_i \leq d_g \sqrt{2}, \tag{14}$$

where:

$$\varepsilon_r = \frac{d_g \sqrt{2}}{L_g}, \tag{15}$$

Table 2. Values of L_g (CEN, 2005a).

Ground type	A	B	C	D	E
L_g. (*m*)	600	500	400	300	500

i is the support identification number, L_i is the distance (projection on the horizontal plane) of support i from a reference support and d_g is the design ground displacement corresponding to the soil conditions underneath support i and provided in Part 1 of the EC8 (CEN, 2005b) as:

$$d_g = 0.025 a_g S\ T_C\ T_D. \tag{16}$$

In Equation (16) a_g is the design ground acceleration on type A ground, S is the soil factor, T_C is the upper limit of the period of the constant spectral acceleration branch, and T_D is the value defining the beginning of the constant displacement response range of the spectrum.

Set B considers the effect of ground displacements occurring in opposite directions. The differential displacements Δd_i are first evaluated at each intermediate support i considering that its adjacent supports i-1 and i + 1 are undisplaced at adjacent supports; Δd_i is given by Equation (17) (CEN, 2005a):

$$\Delta d_i = \pm \beta_r \varepsilon_r L_{av,i}, \tag{17}$$

where $L_{av;i}$ is the average of the distances $L_{i\text{-}1,i}$ and $L_{i,i+1}$ of intermediate support i from its adjacent supports i - 1 and i + 1 respectively. For the end supports (0 and n) $L_{av,0} = L_{0,1}$ and $L_{av,n} = L_{n\text{-}1,n}$. β_r is a factor to account for the amplitude of the ground displacement occurring in opposite directions at the adjacent supports; the recommended values are β_r=0.5 when all three supports are located on the same site conditions, and β_r=1.0 if one of the supports is located on a ground type different than the ground type of the other two supports. ε_r is as defined for set A. If there is a difference in the ground type underneath the two supports, then the maximum value of ε_r should be used.

The Set B displacements d_i are then calculated on the basis of the already derived displacements Δd_i as follows (CEN, 2005a):

$$d_i = \pm \frac{\Delta d_i}{2} \text{ and } \Delta d_i = \pm \frac{\Delta d_{i+1}}{2} \tag{18}$$

4. Evaluation of the RPOA simplified approach

According to the RPOA's simplified approach for bridge analysis under asynchronous ground motion, the bridge response is obtained by combining a static response induced by prescribed differential displacements and a dynamic response induced by synchronous ground motion. As shown in §3, two parameters are important: the limiting length L_M and the ground types. In this paper, three examples of bridges having different overall lengths are studied. For each one, four ground types are considered (S1, S2, S3 and S4).

4.1. Bridges model

The bridges considered in this study have almost the same structural configuration but have different overall lengths: 200 m for Bridge 1, 400 m for Bridge 2, and 600 m for Bridge 3. Their structural configuration was obtained from that of design example No. 1 from the Federal Highway Administration seismic design examples (FHWA, 1996). This example was selected because all of its structural data are readily available. The span length is 50 m. The number of spans is equal to 4 for Bridge 1, 8 for Bridge 2 and 12 for Bridge 3.

The selected bridges have a continuous deck. Its superstructure is a 22.48 m wide post-tensioned continuous box girder. Seat-type abutments are selected for the bridge with space behind the end diaphragm to accommodate free longitudinal movement of the superstructure. The superstructure and the columns are connected using a cap beam. The bents consist of three columns fully connected with square spread footings underneath. The columns have circular cross sections of diameter equal to 1.219 m and heights equal to 8.56 m. The girder cross section of the model of bridges is shown in Figure 2, and the elevation of Bridge 1 is shown in Figure 3.

The finite element model of the bridge consists of six equal-length 3D elastic beam elements per span and four beam elements per pier. The uncracked section properties are used for both the superstructure and columns. The superstructure and the columns are connected by rigid elements. The shear stiffness of the bearings is assumed to provide no restraint in the longitudinal direction. In the vertical direction, the bearings are considered fully restrained due to the gravity forces of the superstructure. The rigid element at each end of the bridge is restrained in the transverse direction by springs, which represent the effect of the girder stops at both ends of the bridge.

The stiffness of each bent foundation is modelled by six soil springs at the lower end of the footing elements, which were determined using an elastic half-space approach (FHWA, 1996). Finally, 5% Rayleigh damping is utilised.

The first three natural periods of studied bridges are shown in Table 3.

4.2. Simulation of seismic ground motions

According to RPOA provisions (MTP, 2010), the highest seismic risk in Algeria is defined by peak ground acceleration on rock (PGA) equal to 0.4 g. The corresponding pseudo-acceleration response spectra for the four ground types S1 to S4 are given by Figure 4 assuming a viscous damping of 5%.

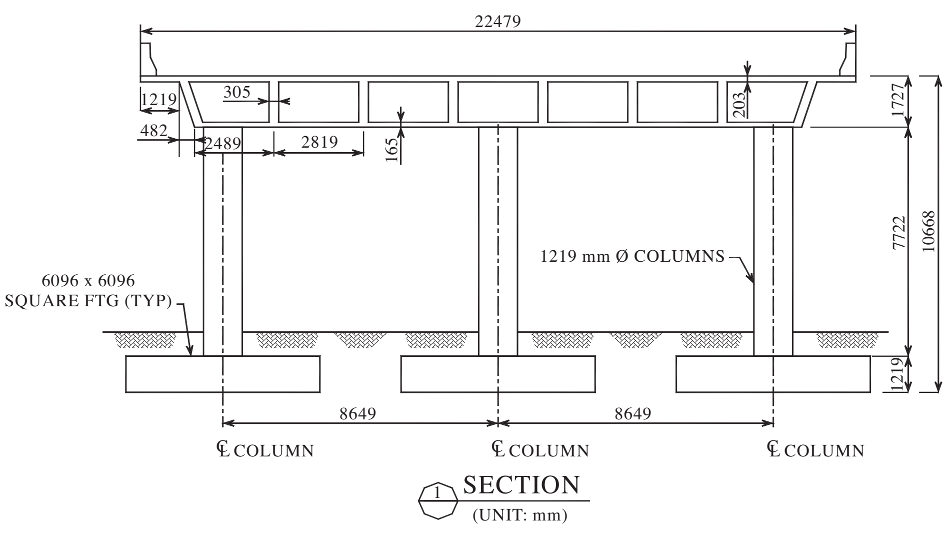

Figure 2. Girder cross-section (FHWA, 1996).

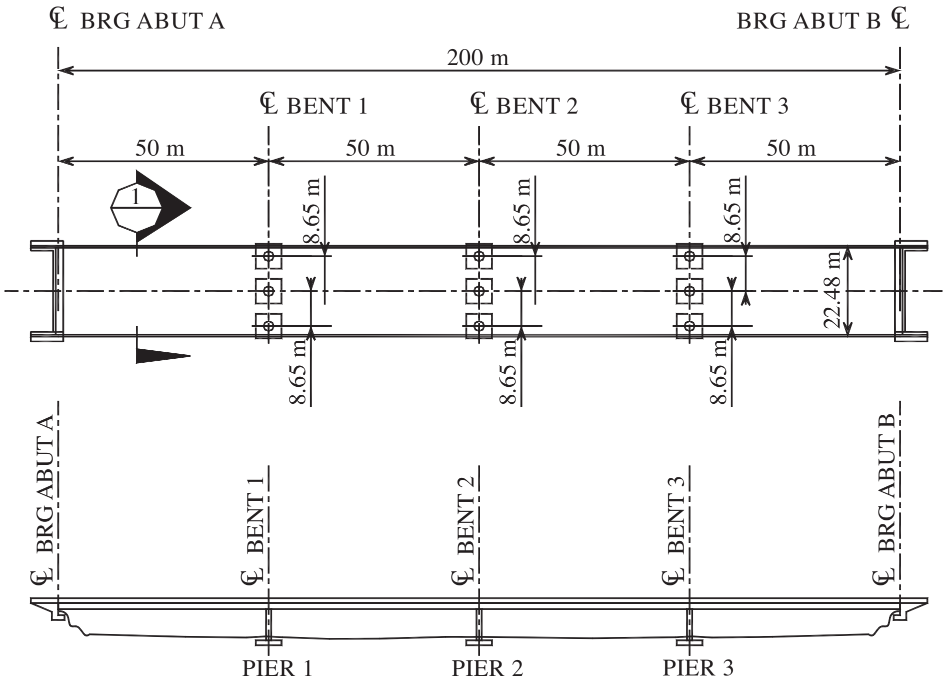

Figure 3. Elevation of Bridge 1 (FHWA, 1996).

Table 3. First three natural periods of studied bridges.

		Periods (s)		
Bridge	Ground type	Mode 1	Mode 2	Mode 3
Bridge 1	S1	0.914	0.812	0.764
	S2	0.919	0.813	0.771
	S3	0.942	0.818	0.800
	S4	1.030	0.910	0.830
Bridge 2	S1	0.892	0.831	0.809
	S2	0.898	0.840	0.810
	S3	0.926	0.879	0.813
	S4	1.031	1.030	0.907
Bridge 3	S1	0.886	0.836	0.819
	S2	0.892	0.845	0.828
	S3	0.921	0.885	0.865
	S4	1.037	1.031	1.006

From these response spectra, 'reference' acceleration time histories were simulated using the TARSCTHS software (Papageorgiou, Halldorsson, & Dong, 2002). The target response spectra may represent the ground motion of a point. An epicentral distance of 17 km from an earthquake with moment magnitude of 6.50 has been considered. The simulated reference time histories have a time step $dt = 0.01$ s, and duration $T = 30$ s.

To simulate non-stationary spatially variable ground motions, the reference accelerations were subdivided into three successive windows. It should be noted that an interpo-

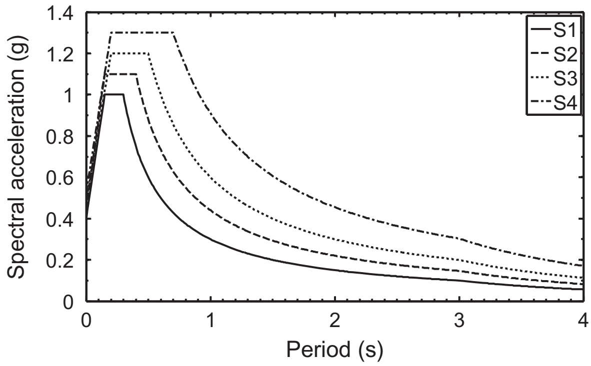

Figure 4. Pseudo-acceleration response spectra for the four ground types S1, S2, S3 and S4 (MTP, 2010).

lation in time domain is needed to satisfy the fast Fourier transform requirements. The power spectral densities were calculated for each segment and were utilised to simulate stationary and spatially variable acceleration time histories for each segment according to the method presented in §2.2. The coherency model of Harichandran and Vanmarcke (1986) was adopted:

$$|\gamma_{jk}(\omega, d_{jk})| = A.exp\left(-\frac{2(1-A+\alpha A)|d_{jk}|}{\alpha\theta(\omega)}\right) + (1-A).exp\left(-\frac{2(1-A+\alpha A)|d_{jk}|}{\theta(\omega)}\right) \tag{19}$$

$$\theta(\omega) = k\left[1+\left(\frac{c}{2\pi\omega_0}\right)^b\right]^{-\frac{1}{2}} \tag{20}$$

The following parameters of the model are used: A = 0.736, α = 0.147, k = 5210 m, ω_0 = 6.85 rad s^{-1} and b = 2.78, which correspond to data recorded during Event 20 at the SMART-1 array, Lotung, Taiwan. Since the span length is the same for all bridges, it was decided to simulate stationary SVGM every 50 m for each segment. Figure 5 gives a plot of the coherency function versus the frequency and the distance.

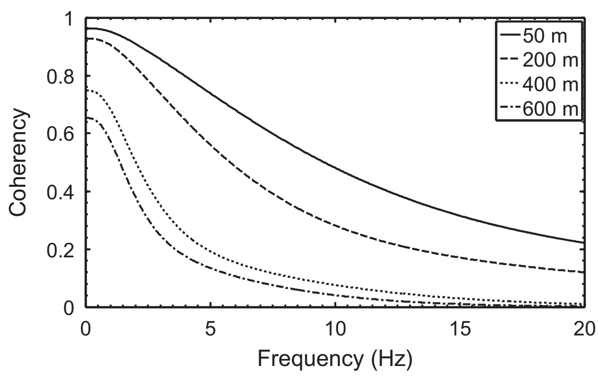

Figure 5. Harichandran and Vanmarcke (1986) coherency function.

The simulated segments are corrected and joined together to obtain the entire acceleration time histories, into which a systematic time shift to account for the apparent wave propagation of the motion is incorporated. In this study, an apparent propagation velocity v = 750 m s^{-1} was used, which represents an average level of wave passage effect.

Finally velocity and displacement time histories are evaluated by integration of acceleration. This procedure is achieved using RISAM software (Boukli Hacene & Rachedi, 2010).

For Monte Carlo simulation needs, the procedure is repeated 10 times. Figure 6 gives one set of non-stationary SVGM displacements corresponding to ground type S1 which were simulated for the longest bridge (600 m, i.e. 13 support points). In other words, 520 = 10 × 13 × 4 displacements are utilised to study the longest bridge when considering the four ground types.

Figure 7 shows the target response spectra (pseudo-acceleration) compared to the mean response spectra of the 10 simulations at the 13 support points, and for each soil type. It is shown that the spectral values for the simulated motions are in good agreement with that of the target response spectra.

4.3. Analysis cases

In this paper, the bridge models presented in §4.1 are analysed using the assumptions of uniform and spatially variable ground motions. For the uniform case, two analyses are performed: conventional response spectrum analysis (URSA) and time history analysis (UTHA). For SVGM, the simplified methods of RPOA (VRPA) and EC8 (VEC8) are used in addition to the time history analysis (VTHA). It should be noted that for both assumptions, the TH analyses are performed using the displacement time histories as an input. Table 4 summarises the analyses cases.

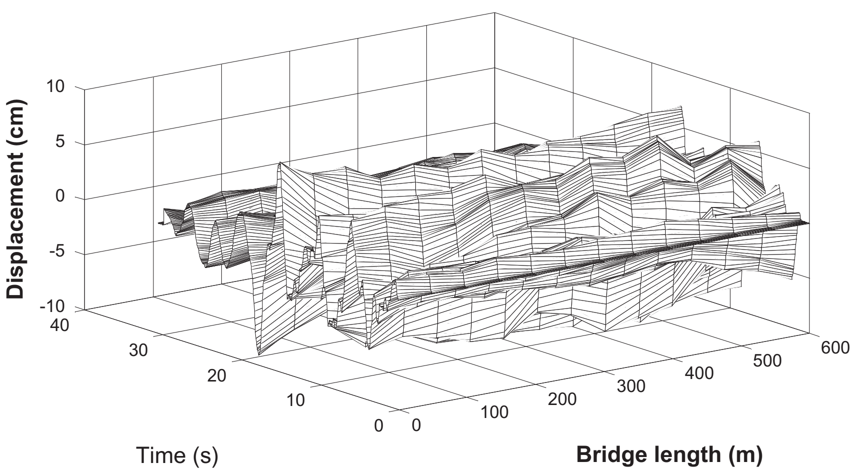

Figure 6. One sample of non-stationary SVGM displacements corresponding to ground type S1.

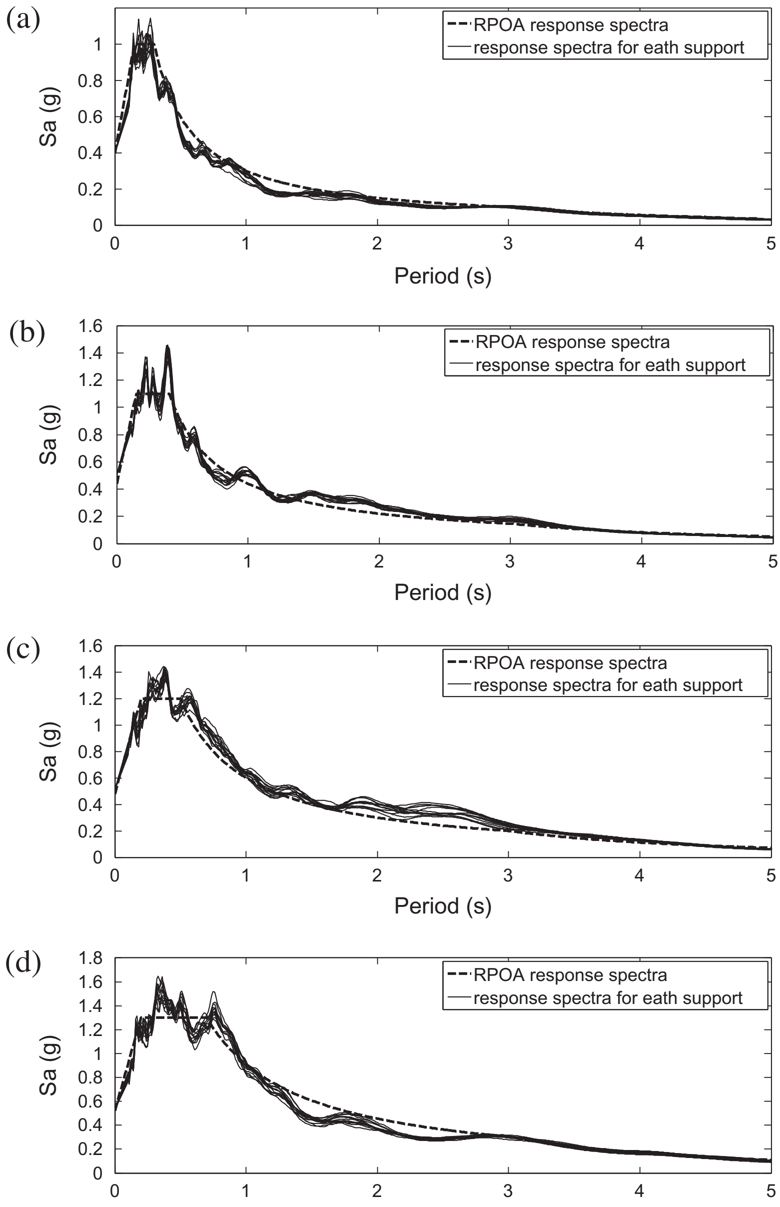

Figure 7. Comparison between target and simulated motion acceleration response spectra: (a) S1, (b) S2, (c) S3 and (d) S4.

4.4. Results and comments

The seismic response analysis results of the bridges subjected to the five cases of excitations presented in the previous section are compared in terms of seismic bending moment. This is done for all ground types considered in this study. Figures 8–10 show the absolute seismic force demand envelopes of the extreme column of each bent of the bridges. For VTHA and UTHA analyses, these figures present mean values, plus/minus

Table 4. Analysis cases.

Acronym	Ground motion	Analysis type	Seismic loading
URSA	Spatially uniform	Response spectrum	RPOA's response spectrum.
UTHA	Spatially uniform	Time history	The displacement simulated for the first support point (§4.1) is imposed at all supports.
VRPA	Spatially variable	RPOA's simplified method (MTP, 2010)	RPOA's response spectrum.
VEC8	Spatially variable	EC8's simplified method (CEN, 2005a)	RPOA's response spectrum.
VTHA	Spatially variable	Time history	SVGM displacements simulated in §4.1

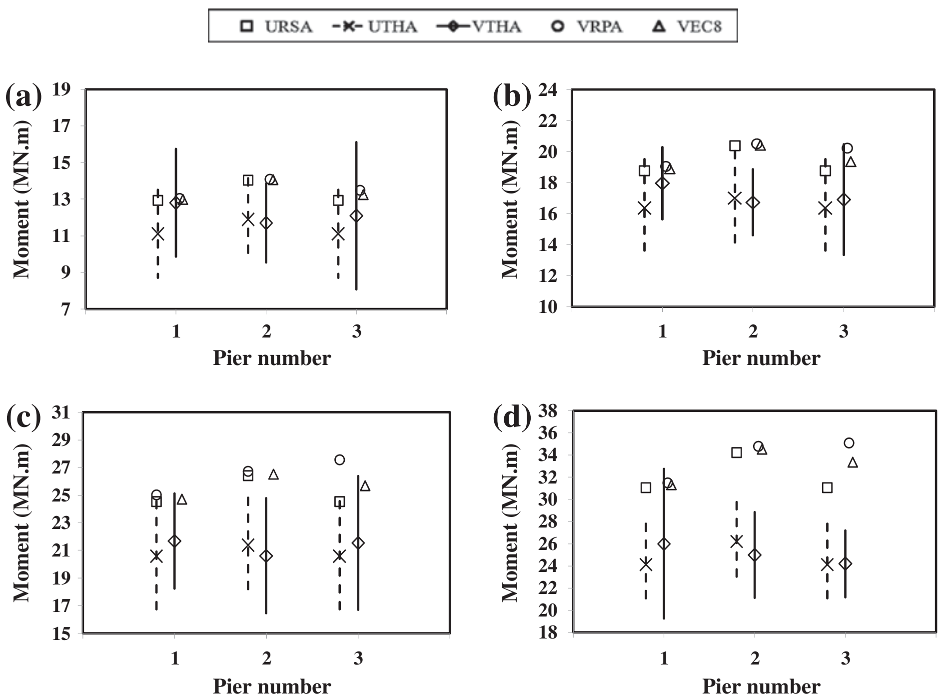

Figure 8. Absolute moment demand envelopes of the bridge column: (a) S1, (b) S2, (c) S3 and (d) S4 (Bridge 1).

standard deviations, (denoted by vertical lines) obtained from 10 time history analyses (realisations). In these figures the horizontal axes indicate pier number. The mean values were obtained using Equation (21) and the standard deviations were calculated using Equation (22).

$$E[x] = \frac{1}{n}\sum_{i=1}^{n} x_i \tag{21}$$

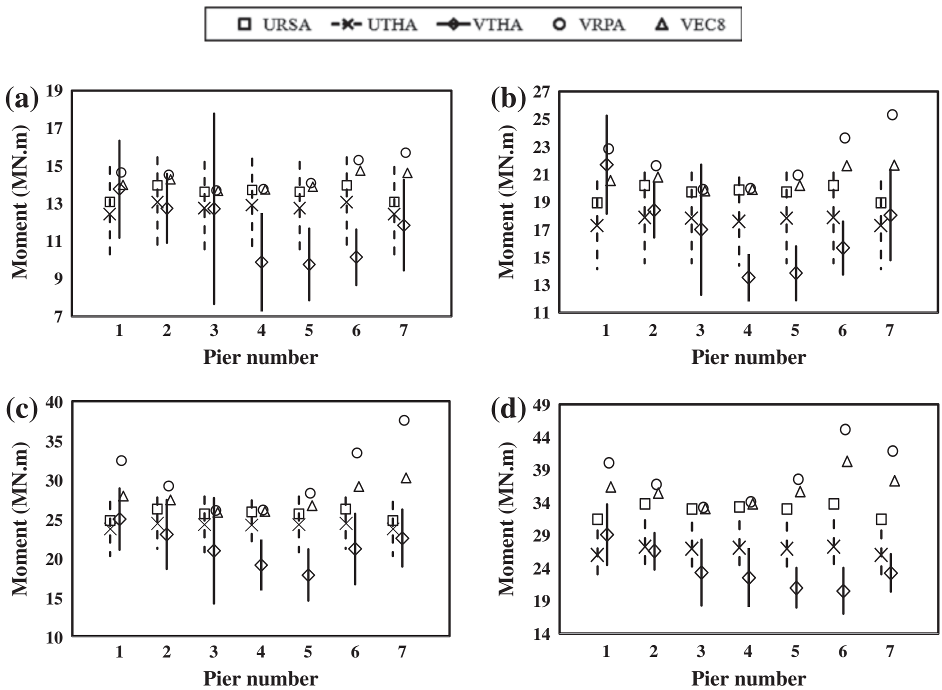

Figure 9. Absolute moment demand envelopes of the bridge column. (a) S1, (b) S2, (c) S3 and (d) S4 (Bridge 2).

$$\sigma_x = E[x^2] - E[x]^2, \tag{22}$$

Where x_i is the bending moment obtained from realisation i.

Firstly, it is important to recall that uniform ground motion analysis has been performed twice: (i) using the time history analysis with the generated displacements as input (UTHA); and (ii) using conventional response spectrum analysis (URSA). The latter is the most commonly used method for linear analyses and can be used to globally check the quality of the generated seismic motions. Figures 8–10 show that URSA results are in good agreement with those given by UTHA in almost all the bridge–site combinations. Significant differences have been observed only in Figures 8(c), 8(d) and 9(d) where URSA results were out of the range denoted by the vertical lines. These cases correspond to periods where the spectra of the generated signals deviate locally from the reference spectra (Figure 7). This remark raises the problem of the choice of the seismic input for the TH analyses in order to achieve comparable studies for a very complex problem such as SVGM.

The effects of the model of SVGM used in this study can be evaluated by comparing the results from the time history analyses (UTHA and VTHA). Figures 8–10 show that VTHA can produce force demands lower or higher than UTHA. The difference lay between +25% and –44% but, for the great majority of the columns, the SVGM was beneficial, i.e. a reduction in the resulting bending moments was observed. However, it is important to note that the SVGM always caused an increase in the seismic demand in at least one of the columns of the studied bridges. These observations are in agreement with the findings of previous studies. Generally, the effects of SVGM were con-

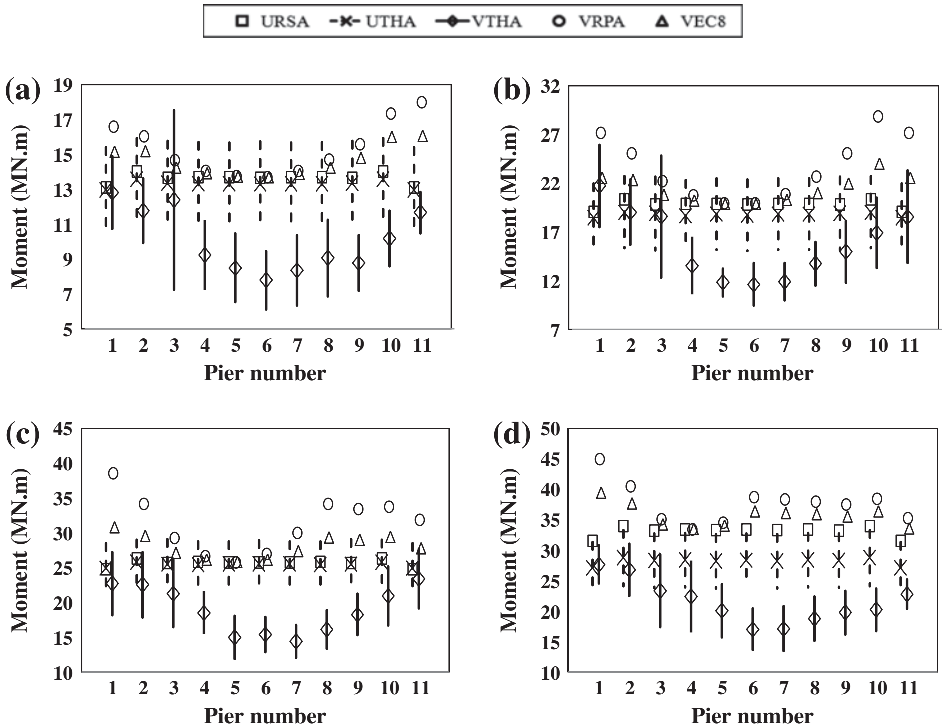

Figure 10. Absolute moment demand envelopes of the bridge column. (a) S1, (b) S2, (c) S3 and (d) S4 (Bridge 3).

sidered negligible (Monti, Nuti, & Pinto, 1996) for the symmetric bridge configuration and uniform soil condition, but recent studies (cf. Burdette & Elnashai, 2007; Lupoi et al., 2005; Sextos, Pitilakis, & Kappos, 2003) observed that this finding cannot be generalised, and concluded that, depending on the characteristics of the SVGM, the bridge configuration and its boundary conditions, spatially variable ground motions can induce a higher or lower response in the structure than the response resulting from uniform ground motions. This conclusion is obviously too general and vague to be practical. The results of the present study indicate that precautions with respect to the SVGM should be taken even if the bridge seems to be simple.

The next objective is to evaluate the simplified methods of RPOA and EC8 for SVGM analysis. Globally, VRPA and VEC8 give results which have the same tendencies but the VRPA results are definitely higher than those of VEC8. VRPA amplified the values of URSA up to 13% in the case of Bridge 1, and up to 50% in the case of Bridges 2 and 3. This amplification is in line with the conclusion of the preceding section but the values, at least for VRPA, seem exaggerated. Another striking fact is the loss of symmetry in the results of the simplified methods, particularly visible in the results of the 600 m long bridge. The bending moment decreases in the central part of the bridge because the loading is a differential displacement which is prescribed at the column bases of which heads are fixed in an infinitely rigid deck which is free to move in the direction of the loading. The loss of symmetry is due to the fact that the prescribed displacement varies linearly until a limiting length, L_M, beyond which it

Table 5. Corrected values for absolute displacement.

Site	S1	S2	S3	S4
D_M	0.025	0.035	0.045	0.07

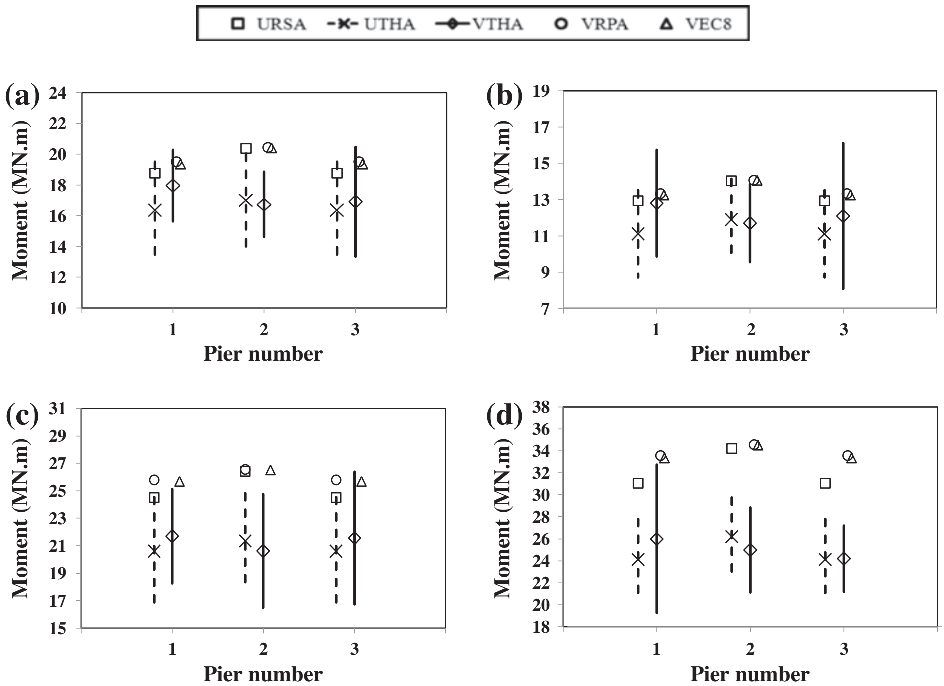

Figure 11. Absolute moment demand envelopes of the bridge column: (a) S1, (b) S2, (c) S3 and (d) S4. Differential displacement is applied in both directions of Bridge 1.

becomes constant. If this variation of differential displacement can be explained by the loss of coherency beyond L_M, the loss of symmetry cannot be justified. It appears obvious that it is necessary to apply differential displacement in both directions of bridge, starting from both reference abutments. This recommendation should be clearly specified in RPOA and EC8. Finally and in order to reduce the results of the VRPA to more reasonable levels, Table 5 proposes new values of D_M, estimated from statistical analysis, correcting those given in Table 1. The results of both recommendations are given in Figures 11–13, which indicates clearly the improvement of the simplified method of RPOA and making it at the same level of performance as EC8. It should be noted that a detailed study and sensitivity analysis are necessary to calibrate more adequately these new values of D_M.

5. Conclusion

Recently, the RPOA code proposed a simplified method to introduce the effect of SVGM in the design of bridges. In this paper, the accuracy of this method is evaluated

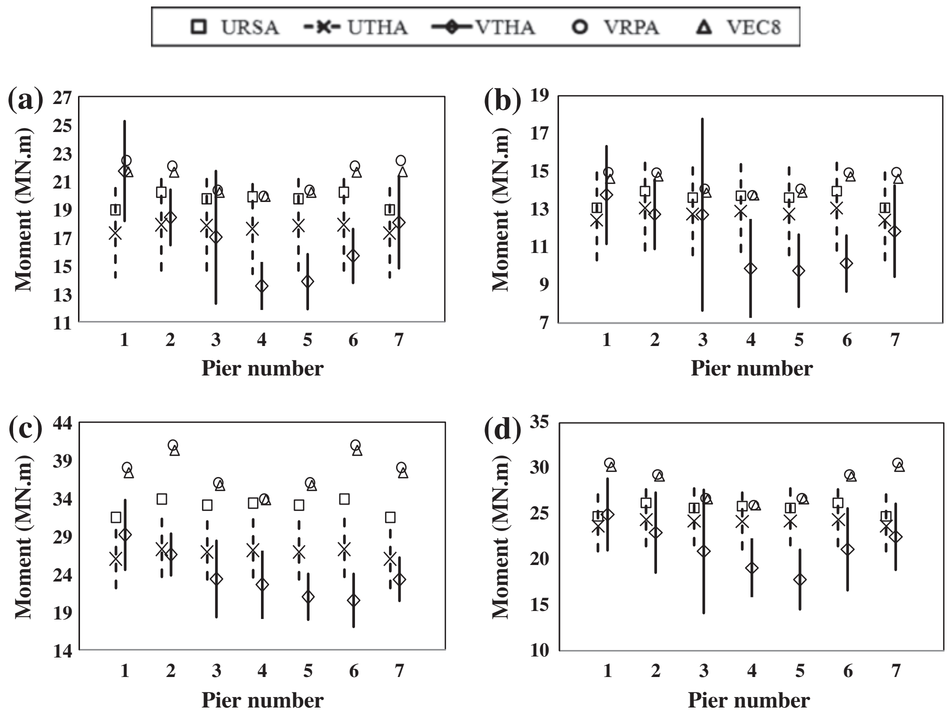

Figure 12. Absolute moment demand envelopes of the bridge column. (a) S1, (b) S2, (c) S3 and (d) S4. DDifferential displacement is applied in both directions of Bridge 2.

through comparison with more refined approaches and the EC8 provisions. Three bridges having different lengths and seating on four types of site conditions are considered. For each bridge–site case, five types of linear analysis are conducted. The spatially variable ground motions are generated using conditional simulation starting from the response spectra of RPOA. The results of these analyses are compared in terms of bending moment demands at piers. Based on this study, the following conclusions can be drawn.

- The spatial variation properties of the earthquake ground motion can locally increase the structural response even in the case of symmetric bridges seating on uniform soil conditions.
- In the simplified methods proposed by RPOA and EC8, the differential displacements should be applied in both directions of the bridge, starting from both reference abutments.
- In order to obtain reasonable results, new values of the absolute displacement D_M are proposed for the simplified method of RPOA.

Last but not least, it is important to mention that the present analysis corresponds only to one model of bridges with different overall lengths and additional research needs to be conducted to cover other models and help enrich this area of investigation.

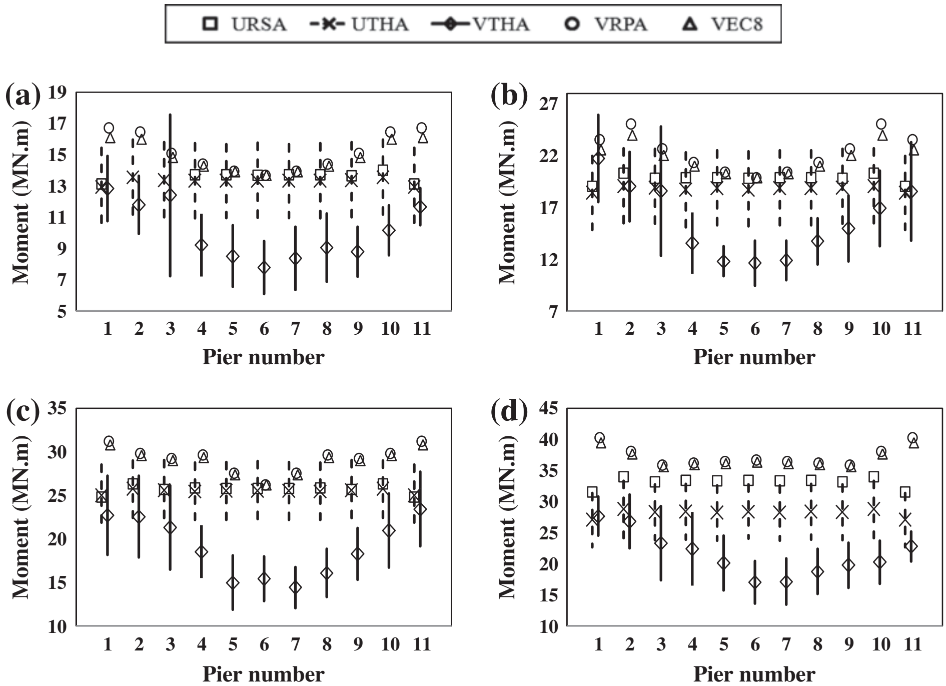

Figure 13. Absolute moment demand envelopes of the bridge column: (a) S1, (b) S2, (c) S3 and (d) S4. Differential displacement is applied in both directions of Bridge 3.

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