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**Citation:** Efthymiou, E. & Camara, A. (2021). Inelastic response of cable-stayed bridges subjected to non-uniform motions. *Bulletin of Earthquake Engineering*, 19(6), pp. 2691-2710. doi: 10.1007/s10518-021-01079-z

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# 1 Inelastic response of cable-stayed bridges subjected 2 to non-uniform motions

3 Eleftheria Efthymiou · Alfredo Camara

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5 Received: date / Accepted: date

6 **Abstract** This paper studies for the first time the effect of the Spatial Variability of Ground Motions (SVGGM) with large intensities on the inelastic seismic response of the pylons which are responsible for the overall structural integrity of cable-stayed bridges. The SVGGM is defined by the time delay of the earthquake at different supports, the loss of coherency of the seismic waves and the incidence angle of the ground motion. An extensive study is conducted on cable-stayed bridges with ‘H’- and inverted ‘Y’-shaped pylons and with main spans of 200, 400 and 600 m. The SVGGM is most detrimental to the pylon of the 200-m span bridge owing to the large stiffness of this bridge compared to its longer counterparts. The stiff configuration of the inverted ‘Y’-shaped pylon makes it more susceptible against the multi-support excitation than the flexible ‘H’-shaped pylon, especially in the transverse direction of the response. Finally, the earthquake incidence angle is strongly linked with the SVGGM and should be included in the seismic design of cable-stayed bridges.

20 **Keywords** spatial variability · cable-stayed bridges · pylon · incidence angle ·  
21 non-linear dynamic analysis · incoherence effect · wave-passage effect

## 22 1 Introduction

23 The Spatial Variability of the Ground Motion (SVGGM) also referred to as  
24 multi-support excitation is described by the differential movement of the sup-  
25 ports of long structures. It is present when a structure is long with respect

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Eleftheria Efthymiou  
Department of Civil and Environmental Engineering, Lyle School of Engineering, Southern  
Methodist University, Dallas, TX, USA  
Tel.: +1 (214) 768-3894  
E-mail: eefthymiou@smu.edu

Alfredo Camara  
School of Mathematics, Computer Science and Engineering, City University of London,  
London, UK

to the wavelength of the input motion in the frequency range of importance to the response of the structure. As a result, its supports may be subjected to different excitations [3] and this is known to affect the structural response depending on the amplitude of the seismic motion, the incidence angle of the earthquake relatively to the structure, the geometric characteristics of the structure and the stiffness of the surrounding soil among others [30,35,26,4,51,45,46,49,5,42,47,39,43,33].

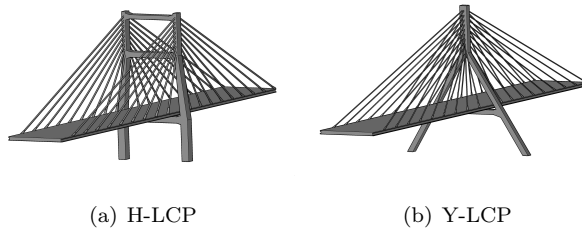
The SVGGM results from the combination of four effects [14,13]; the wave-passage effect which refers to the difference in the arrival times of the ground motion at different supports; the incoherence effect which refers to the loss of coherency of the ground motion due to successive reflections and refractions of the seismic waves in heterogeneous soil media along their path; the site response effect which reflects the modification of the amplitudes and the frequency contents of the ground motions at different supports due to changes in the local site conditions; and the attenuation effect which describes the gradual decrease of the amplitudes of the seismic waves with distance as they travel away from the fault.

The SVGGM can be detrimental on stiff structures whereas, it typically does not significantly affect the response of longer and more flexible structures [4,37]. The increased influence of the SVGGM on stiff structures is due to the pseudo-static component of the structural response [40,52], as opposed to flexible structures wherein the response is dominated by the dynamic component [6]. In cable-stayed bridges that are composed of elements with very different flexibilities, this observation can be extended in the sense that their stiffer components are more vulnerable to the multi-support excitation.

The wave-passage effect is an important component of the SVGGM especially in the long spans that are usually associated with cable-stayed bridges where the time delay of the earthquake at different supports can be more important than the incoherence of the ground motion. Typically lower values of the wave propagation velocity increase the structural response by increasing the pseudo-static forces caused by the SVGGM and by decreasing their dynamic counterpart [4,51,46,47,50,6]. The incoherence effect is usually more pronounced than the wave-passage effect which can be neglected when the seismic waves present a high rate of incoherence [51,45].

More recently, the earthquake incidence angle combined with the SVGGM has gained the attention of the research community, with a limited number of studies stating that the maximum structural response of a cable-stayed bridge may not be obtained when the direction of propagation coincides with the principal axes of the bridge [5,32].

The aforementioned works focus on the elastic seismic response of structures under the SVGGM, but there is lack of works on the inelastic response of the pylons of cable-stayed bridges under large non-uniform ground motions. It is important to understand the response of these structures under extremely large records, which could exceed the design limits. Indeed, important damages have been reported in several cable-stayed bridges after strong earthquakes in the 80's and 90's. This is the case of the Shipshaw Bridge (Canada, 183-m



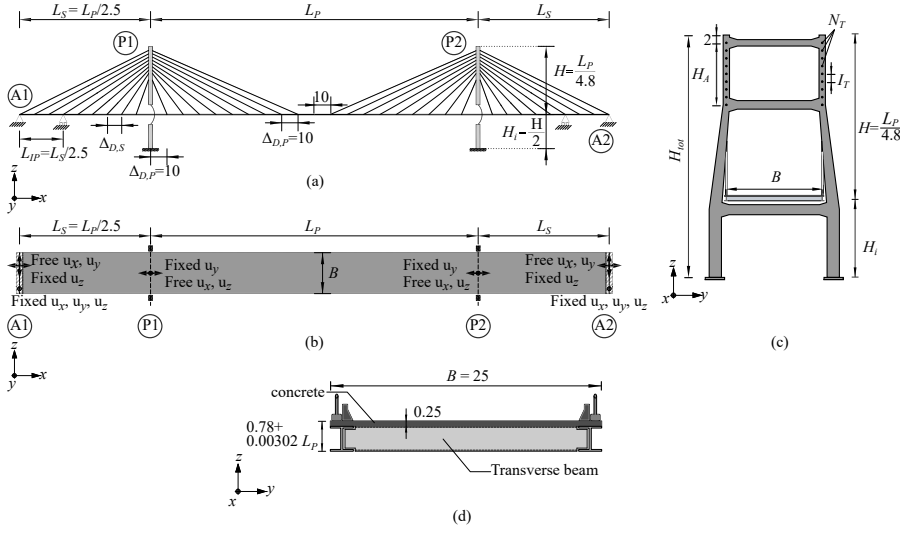
**Fig. 1** Adopted pylon shapes and their reference keywords. The part of the notation before the hyphen reflects the shape of the pylon; whereas the part after the hyphen characterises the cable arrangement. In this work the deck is supported on two lateral cable planes (LCP).

span), damaged at the connection between the deck and the pylon during the 1988 Saguenay earthquake, with moment magnitude  $M_W = 6.0$  [24]. In addition, the piers of the Higashi-Kobe Bridge (Japan, 485-m span) were also damaged during the Hyogo-Ken Earthquake [7]. But perhaps the most significant seismic damage was caused by the 1999 Chi-Chi Earthquake at the pylon of the Chi-Lu Bridge (Taiwan, 120-m span) [11]. Even though these partial failures are deemed inadmissible today, the large social and economic importance of cable-stayed bridges, which emphasizes the need for research on this topic. This paper focuses on the effect of the SVGM and the incidence angle of the earthquake on the inelastic seismic response of the pylons of cable-stayed bridges with different structural configurations and dimensions. Important conclusions are reached through the comparison between the elastic and the inelastic seismic responses of the bridges. The effect of the SVGM on the pylons with variable heights is investigated and the role of the pylon shape as a means to resist the SVGM is also thoroughly discussed. The influence of the multi-support excitation on the seismic response of the pylons is strongly affected by the shape of the pylons, their overall dimensions, the part of the pylon under examination and by the earthquake incidence angle.

## 2 Modelling and Analysis

### 2.1 Numerical Models

The bridge models employed in this study are based on the work of Camara *et al.* [9] and Efthymiou [15]. The overall bridge arrangement consists of two symmetric reinforced concrete pylons, a composite deck and the cable system. The length of the main span,  $L_P$ , is equal to 200, 400 and 600 m, representing short-span, intermediate-span and relatively long-span cable-stayed bridges, respectively. Pylons with conventional ‘H’ and inverted ‘Y’ shapes have been considered, as shown in Fig. 1 wherein the notation of the pylons is also included and will be followed hereinafter.



**Fig. 2** Parametric definition of the cable-stayed bridges. (a) Elevation, (b) Plan view, (c) sample pylon (the same parametrisation rules are applied to the ‘H’ and the inverted ‘Y’-shaped pylons), (d) deck. All dimensions are in [m].

100 The length of the side spans,  $L_S$ , is described by  $L_P$  as shown in Fig. 2.  
 101  $L_P$  also defines the number of cables,  $N_T$ , in each plane;  $N_T = 9, 19$  and  $29$   
 102 when  $L_P = 200, 400$  and  $600$  m, respectively. The height of the pylons above  
 103 the deck level is  $H = L_P/4.8$  in both pylon configurations. The height of the  
 104 pylons below the deck is  $H_i = H/2$ , resulting in the total height of the pylons:  
 105  $H_{tot} = H + H_i = 62.5, 125$  and  $187.5$  m for the 200-, 400- and 600-m main span  
 106 bridges, respectively. The dimensions of the different members of the pylons  
 107 and of the various cross sections are functions of  $H$  [9].

108 The cables form a semi-harp configuration in the direction parallel to the  
 109 traffic. Two lateral cable planes (LCP) hold the 25-m wide deck that accommo-  
 110 dates four traffic lanes, regardless of the length of the bridge. The deck has an  
 111 open composite cross-section formed of two longitudinal I-shaped steel girders  
 112 at the edges and a 25-cm thick concrete slab on top. The overall stability of the  
 113 deck is provided by transverse I-beams located at fixed intervals ( $\approx 5$  m apart)  
 114 that connect the two longitudinal girders. In the side spans intermediate piers  
 115 at a distance of  $L_{IP}$  from the abutments constrain the vertical displacement of  
 116 the deck and consequently control the longitudinal displacement of the upper  
 117 part of the pylon (see Fig. 2).

118 The abutments constrain the movement of the deck in the vertical ( $z$ ),  
 119 transverse ( $y$ ) and longitudinal ( $x$ // traffic) directions and they also prevent  
 120 its torsional rotation ( $\theta_{xx}$ ) as shown in Fig. 2(b). **The movement of the deck is**  
 121 **constrained in the three orthogonal directions with POT bearings at the edges**  
 122 **of the deck. The torsional rotation is prevented due to the eccentricity of the**  
 123 **POT supports that constrain the vertical movement of the deck relative to**  
 124 **its centerline. The model of these support devices does not account for their**

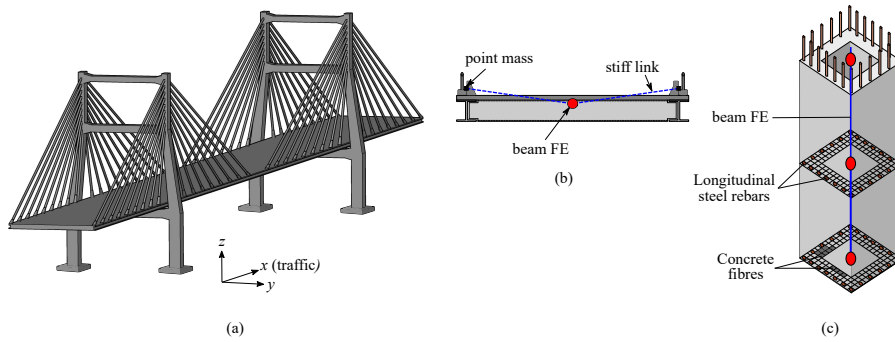
flexibility, i.e. they are modelled as fully-fixed eccentric points linked to the beam element representing the deck at its centroid. This is deemed admissible given the large stiffness of POT devices in the direction in which they restrain the displacement, and the small effect that including their flexibility (e.g. with spring elements) would have in the overall seismic response of the bridge. The piers at the side spans constrain the vertical displacement and the torsional rotation of the deck. At the pylons, the deck is only restrained in the  $y$  direction assuming a floating connection between the deck and the pylon — a common solution in the design of cable-stayed bridges in seismic prone regions.

The soil–structure interaction has been considered by applying systems of springs and dashpots at the base of the pylons. The dynamic impedances are computed for the movement in the three principal directions of the bridge and for the rotation around the  $x$  and  $y$  axes. The spring and dashpot systems have constant properties that are obtained from the work of Gazetas [25] and are calibrated to the mean frequency of the earthquakes,  $f_m$  [48], as:

$$f_m = \frac{\sum_i C_i^2}{\sum_i C_i^2 \left(\frac{1}{f_i}\right)} \quad (1)$$

where  $C_i$  are the Fourier amplitude coefficients and  $f_i$  are the discrete fast Fourier transform (FFT) frequencies between  $0.20 \text{ Hz} \leq f_i \leq 20 \text{ Hz}$ .

The constituent materials (steel and concrete) of the cable-stayed bridges are described through their constitutive models and their properties are defined in the relevant Eurocodes [19–21, 23]. The concrete in the pylons is C40 concrete with characteristic strength of  $f_{ck} = 40 \text{ MPa}$  [19] and it is confined with transverse reinforcement bars following the model of Mander *et al.* [36]. To ensure adequate concrete confinement longitudinal and transverse reinforcement ratios of 2.4% and 0.8%, respectively are considered in the pylons [1, 38]. B500C steel grade is used for the deck and for the reinforcement with yield stress of 552 MPa. The prestressing steel forming the cables has a Young’s modulus of  $E_p = 190 \text{ GPa}$ . In this work the nonlinear material behaviour is only considered in the pylons, which appeared to be the most vulnerable elements in cable-stayed bridges subjected to large ground motions [11], whereas the concrete slab, the steel members of the deck and the cables behave elastically. The structural damping in the dynamic analysis follows a Rayleigh’s damping distribution with a maximum damping ratio of 2% to account for the small structural dissipation in cable-stayed bridges [31] and it is independent of the material (concrete or steel). The range of important frequencies for the structural response of the examined bridges is defined at the lower bound by the fundamental frequency ( $f_1$ ) of the bridges.  $f_1$  is equal to 0.49, 0.33 and 0.20 Hz for the 200, 400 and 600-m span bridges, respectively and it is almost insensitive to the pylon shape and to the type of cable system. The upper bound of the important frequency range is set as 20 Hz in all cases [8].



**Fig. 3** (a) Complete 3D model of the H-LCP bridge with  $L_P = 200$  m, (b) FE model of the LCP deck and (c) FE model of the pylon.

164 The finite element (FE) analysis software Abaqus [2] has been used to  
 165 model the bridges and to conduct the complete sets of dynamic analyses. Fig-  
 166 ure 3 shows that the deck sections are discretised with linear interpolation  
 167 shear-flexible beam-type elements that pass through the centre of gravity of  
 168 the deck sections and account for the structural (reinforced concrete slab, lon-  
 169 gitudinal and transverse steel girders and steel diaphragms) and nonstructural  
 170 (deck asphalt) masses, as shown in Fig. 3(b). Lump mass elements located at  
 171 both cable ends represent the anchorage masses which, at the deck end of each  
 172 cable, also include the parapets. The pylons are modelled with fibre beam ele-  
 173 ments through the centre of gravity of their sections. The nonlinear cyclic  
 174 degradation of the confined concrete is defined through the smeared crack  
 175 concrete model. Finally, the cables are modelled with 3D trusses which use  
 176 linear interpolation of the axial displacements. Each truss element represents  
 177 one cable, ignoring the cable-structure interaction [16].

## 178 2.2 Seismic Action

179 Seven sets of artificial accelerograms have been generated based on the re-  
 180 sponse spectrum of Eurocode 8 [22] with 2% damping,  $a_g = 0.5g$  and for  
 181 ground category D, which represents soft soil conditions. Each set consists of  
 182 four bi-directional accelerograms that are generated for each of the supports  
 183 of the bridges with horizontal restraints (i.e. the two abutments and the two  
 184 pylons). This means that the two horizontal components of the seismic ac-  
 185 tion have been considered — namely ‘Fault Parallel’ (FP) and ‘Fault Normal’  
 186 (FN), the latter coinciding with the direction of wave propagation. **Artificial**  
 187 **accelerograms are not associated to principal components in their generation**  
 188 **process, hence an intensity ratio between the major and minor earthquake**  
 189 **components has been adopted to account for the observed differences in the**  
 190 **propagation of the waves in the directions perpendicular and parallel with re-**  
 191 **spect to the fault.** To this end, for the FP component the response spectrum  
 192 is reduced to 70% of the original FN component [34].

193 The resulting accelerograms have been modified by the empirical coherency  
 194 model of Harichandran and Vanmarcke [27] to account for the loss of coherency  
 195 among the accelerograms at the different supports of the bridges. Following the  
 196 consideration of several empirical and semi-empirical coherency models [17],  
 197 the adopted model of Harichandran and Vanmarcke [27] has been deemed the  
 198 most appropriate for this work because it considers incoherent seismic waves  
 199 even in low frequencies, which are of importance to the response of cable-stayed  
 200 bridges. The coherency is assumed independent of the direction of propagation  
 201 allowing for the same coherency model to be used for the generation of signals  
 202 in the FN and FP directions [44].

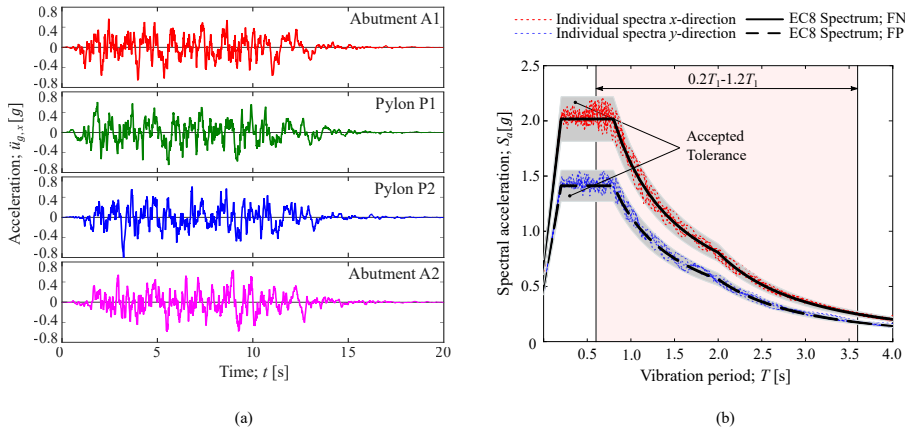
203 The temporal variability of the ground motion (i.e. the wave-passage effect)  
 204 is also accounted for by means of the delay in the arrival of the earthquakes at  
 205 neighbouring supports. The first abutment (A1) is affected by the earthquake  
 206 at time instance  $t_{A1} = 0$  s and then the ground motion travels parallel to the  
 207 deck with propagation velocity  $c = 1000$  m/s reaching the first pylon (P1),  
 208 the second pylon (P2) and the second abutment (A2). [The time delay between  
 209 pylons P1 and P2 reaches 0.2, 0.4 and 0.6 s in the bridges with  \$L\_P = 200, 400\$   
 210 and 600 m.](#)

211 The generation of the spectrum-compatible acceleration histories follows  
 212 the iterative scheme proposed by Deodatis [12]. In this methodology, at the  
 213 end of the  $i^{\text{th}}$  iteration the acceleration history,  $\ddot{u}_{g,j}^{(i)}(t)$ , is obtained for each  
 214 support  $j$  and the response spectrum of  $\ddot{u}_{g,j}^{(i)}(t)$ ,  $\text{RS}_j^{(i)}$ , is compared to the tar-  
 215 get  $\text{RS}_j^{\text{target}}$  and the process is repeated with an updated  $S_{jj}$  until acceptable  
 216 convergence is reached, as follows:

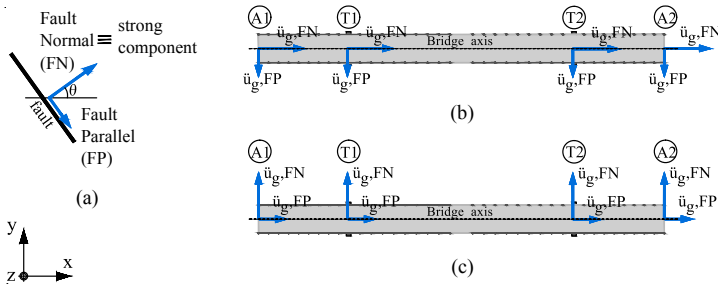
$$S_{jj}^{(i+1)}(\omega) = \left[ \frac{\text{RS}_j^{\text{target}}(\omega)}{\text{RS}_j^{(i)}(\omega)} \right]^2 S_{jj}^{(i)}(\omega) \quad (2)$$

217 where:  $S_{jj}$  is the power spectral density at station  $j$  for the previous ( $i^{\text{th}}$ ) and  
 218 the next ( $i^{\text{th}} + 1$ ) iterations. More details about the generation of spectrum-  
 219 compatible accelerograms that account for the SVGM are included in [15]. In  
 220 this work, the iterations stop when the response spectra of the individual ac-  
 221 celerograms fall within the range 90%-110% of the target spectrum in the range  
 222 of important periods of the studied bridges:  $[0.2T_1, 1.2T_1]$ ;  $T_1$  being the funda-  
 223 mental vibration period of the structure in each case [8]. For the two different  
 224 structural typologies,  $T_1$  is 2.03, 3.02 and 5.01 s on average when  $L_P = 200,$   
 225 400 and 600 m, respectively. The accelerograms are then baseline-corrected to  
 226 ensure there are no residual displacements at the end of the time-histories. An  
 227 indicative set of accelerograms corresponding to the FN component for the  
 228 supports of the 400-m main span bridge is included in Fig. 4(a), where the  
 229 time delay and the loss of coherency can be appreciated. Figure 4(b) shows a  
 230 good match between the FN and FP target spectra and those of the resulting  
 231 signals.

232 Two different orientations of the bridges have been examined aiming to  
 233 address the effect of the angle of incidence,  $\theta$ , of the earthquake. In the first



**Fig. 4** (a) Indicative set of accelerograms when  $c = 1000$  m/s, (b) individual response spectra from the generated accelerograms. The target response spectra are also included.  $L_P = 400$  m.



**Fig. 5** Schematic representation of the angle of incidence ( $\theta$ ) of the seismic waves with respect to the bridges. (a) Principal earthquake components; (b)  $\theta = 0^\circ$  and (c)  $\theta = 90^\circ$ .

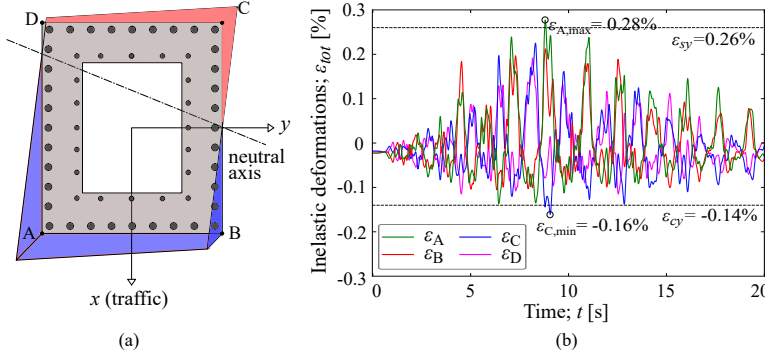
234 case (namely  $\theta = 0^\circ$ ) the FN component, which represents the strong earth-  
 235 quake component, propagates parallel to the bridge and the FP (i.e.  $0.7FN$ )  
 236 component acts perpendicular to the bridge. In the second case ( $\theta = 90^\circ$ ) the  
 237 bridge is rotated by  $90^\circ$  clockwise, making the FN component act transversely  
 238 with respect to the bridge, as shown in Fig. 5.

239 The wave-passage and incoherence effects are dependent on the orientation  
 240 of the bridge with respect to the ground motion. The travelling distance of the  
 241 seismic waves,  $r_{ij}$  with  $i, j = A1, P1, P2$  and  $A2$  and  $i \neq j$ , is a function of  
 242 the distance between supports,  $L_{ij}$ , and of  $\theta$ , as:  $r_{ij} = L_{ij} \cos \theta$ . As a result,  
 243 when  $\theta = 0^\circ$  both the time delay and the loss of coherency between supports  
 244 are maximised. On the other hand, when  $\theta = 90^\circ$  the seismic waves are con-  
 245 sidered completely coherent and the time delay is zero, reducing essentially  
 246 the problem to the identical support motion.

## 247 2.3 Methodology

248 Elastic and inelastic dynamic analyses have been performed to evaluate the  
249 seismic behaviour of the cable-stayed bridges. The geometric nonlinearities  
250 that arise from large deformations, which are inherent in cable-stayed bridges  
251 [4], have been accounted for both in the elastic and the inelastic analyses of the  
252 bridges. The system of dynamics is integrated step-by-step during the analysis.  
253 The direct implicit HHT algorithm [29] is used in the nonlinear analysis of the  
254 bridges with  $\alpha = -0.05$  in order to reduce the high frequency noise that is  
255 introduced to the integration with every time step [2]. The time step for the  
256 integration is selected from a sensitivity analysis as  $\Delta t = 0.01$  s, which is the  
257 same as that of the accelerograms, but can be reduced down to  $\Delta t = 10^{-5}$  s  
258 so that numerical convergence is reached.

259 The effect of the SVGM on the seismic response of the pylons is evaluated  
260 by means of the peak seismic axial load ( $N$ ), the longitudinal ( $V_x$ ) and the  
261 transverse ( $V_y$ ) shear forces along the pylon legs. The deformations of the  
262 concrete and the steel reinforcement ( $\varepsilon_{tot}$ ) in the pylons are also examined.  
263 The results are obtained for the second pylon that receives the SVGM (P2)  
264 because its effect on this pylon has been found more important than on P1  
265 in previous studies on the elastic seismic response of cable-stayed bridges [18].  
266 In fact, the apparent symmetry of the structures described in Section 2.1 of  
267 the manuscript is lost in the seismic response for different incidence angles  
268 ( $\theta$ ) due to: (1) the longitudinal movement of the deck during the earthquake,  
269 which introduces axial compression in one pylon and tension in the opposite  
270 due to the effect of the cable system; (2) the difference in the ground motion  
271 at different supports due to the loss of coherency and the time lag from the  
272 SVGM (Fig. 4(a)); and (3) the lack of symmetry of the support conditions at  
273 the abutments A1 and A2 with respect to the centreline of the deck (axis  
274  $x$ ), as shown in Fig. 2(b). The asymmetry of the response that is observed  
275 both in the SVGM and the SYNC motion is mainly relevant in the longitudi-  
276 nal direction. This is attributed to the effect of the cable system transferring  
277 the longitudinal shear forces from one pylon to the opposite. This effect in-  
278 creases with the restraint of the cable system to the relative (out-of-phase)  
279 longitudinal movement between the pylons. In the transverse direction of the  
280 response the pylons remain relatively unconstrained from the cable system  
281 and the deck (floating deck–pylon connection), and as a result the difference  
282 between the transverse response of the two pylons is negligible. The results  
283 are obtained in the form of time-histories of the forces and strains from the  
284 individual earthquakes at the different regions of the legs and subsequently,  
285 the peak (maximum absolute) values are identified. For the forces, the effect  
286 of the self-weight is subtracted from the time-histories in order to focus solely  
287 on the seismic response. For the deformations of the concrete and the steel re-  
288 inforcement, the strain induced by the self-weight prior to the seismic action is  
289 also considered. The deformations are examined at the corner of the concrete  
290 sections of the pylons, where the maximum strains are recorded, and also at  
291 the corner reinforcement bars, as can be seen in Fig. 6. The positive values



**Fig. 6** (a) Schematic representation of the strain distribution in a typical section of the pylon and (b) Time-histories of the deformation at the corners of the section; H-LCP model;  $L_P = 200$  m;  $\theta = 0^\circ$ ; seismic record #7; SYNC motion.

of the deformations ( $\varepsilon^+$ ) denote tension and the negative values ( $\varepsilon^-$ ) denote compression. The cracking in the concrete sections is considered to be excessive when the steel reinforcement yields in tension ( $\varepsilon_s \geq \varepsilon_{sy} = 0.26\%$ ) which is associated with slight-to-moderate concrete cracking [28], whereas the compressive deformation demand in the concrete is considered inadmissible when the peak compressive elastic limit  $\varepsilon_{cy} = f_{cm}/E_c = 0.14\%$  is exceeded, **and the concrete crushes in compression.**

Eventually, the results are presented in terms of the arithmetic average of the peak seismic forces and the deformations of the materials ( $\mu$ ) from the performed dynamic analyses  $R_i$  (with  $i = 1 \rightarrow 7$  denoting the sets of accelerograms applied). In this work the dispersion of the individual seismic responses is also considered in terms of the standard deviation ( $\pm$ SD) from the mean peak seismic response ( $\mu_{\text{SYNC}}$ ) obtained from the reference synchronous (SYNC) motion of the supports.

The increments in the seismic response due to the SVGM are quantified by the ratio  $\rho$  of the seismic response from the SVGM over the respective response from the SYNC motion:

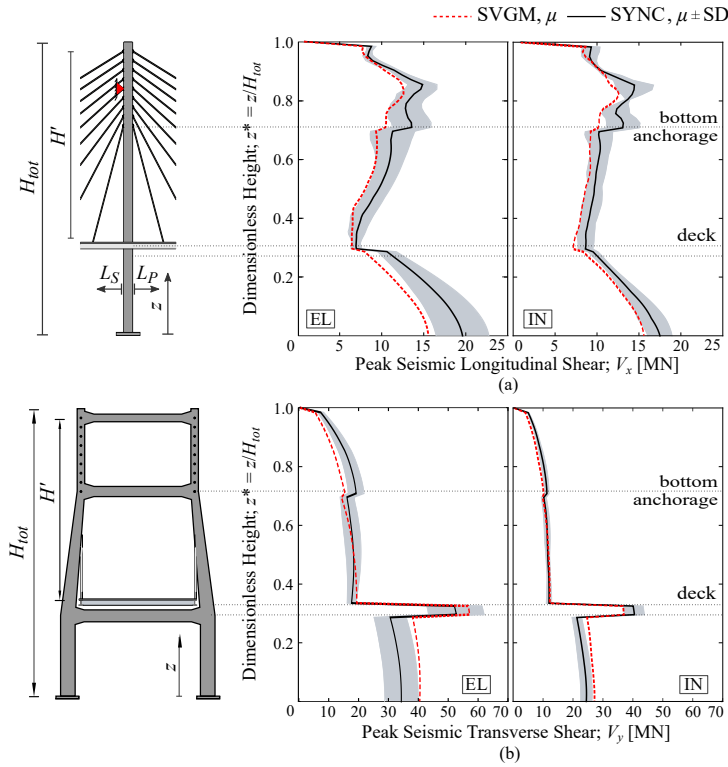
$$\rho_j = \frac{\mu_{\text{SVG M},j}}{\mu_{\text{S Y N C},j}} \quad (3)$$

where  $\mu_{i,j}$  with  $i = \text{SVG M}, \text{S Y N C}$  is the arithmetic mean (from the seven sets of accelerograms) of the peak response:  $j = N, V_x, V_y, \varepsilon_{\text{tot}}$ .

### 3 Influence of the SVG M on the Seismic Response of the Pylons

#### 3.1 Effect of the Material Nonlinearity

Figure 7 shows that the material nonlinearities reduce the seismic-induced  $V_x$  (Fig. 7(a)) and  $V_y$  (Fig. 7(b)) in the bridge with  $L_P = 400$  m, compared to their respective magnitudes when the materials (concrete and reinforcement steel) have linear elastic behaviour. This reduction is attributed to the plastic



**Fig. 7** Peak elastic (left) and inelastic (right) seismic response in pylon P2. (a)  $V_x$  and (b)  $V_y$ . H-LCP bridge;  $L_P = 400$  m;  $\theta = 0^\circ$  (i.e. FN // traffic).

317 dissipation of the input seismic energy that provides an additional damping  
 318 mechanism in the pylons. Focusing on the SYNC motion first, the longitudinal  
 319 seismic shear force,  $V_{x,IN}$ , is reduced by 10% at the base of the pylon compared  
 320 to  $V_{x,EL}$  and  $V_{y,IN}$  is reduced by 25% from  $V_{y,EL}$  at the area between the deck  
 321 and the lower strut when material nonlinearities are considered. The larger  
 322 reduction in  $V_y$  is associated with the transverse reaction that the deck exerts  
 323 to the pylon, which is dominated by the fundamental transverse vibration  
 324 mode of this bridge [10]. Looking at the response from the SVGM, the reduction  
 325 in  $V_x$  at the base of the pylon when material nonlinearities are considered is  
 326 negligible, which suggests that there is no plastic dissipation at this region  
 327 of the pylon from the longitudinal out-of-phase motion of the pylons. On the  
 328 other hand, the transverse shear force,  $V_{y,IN}$ , from the SVGM is reduced by 47%  
 329 compared to  $V_{x,EL}$  at the deck level, indicating that in this direction there is  
 330 damage from the plastic dissipation of the seismic energy in the pylon, from  
 331 the deck level down to the foundation.

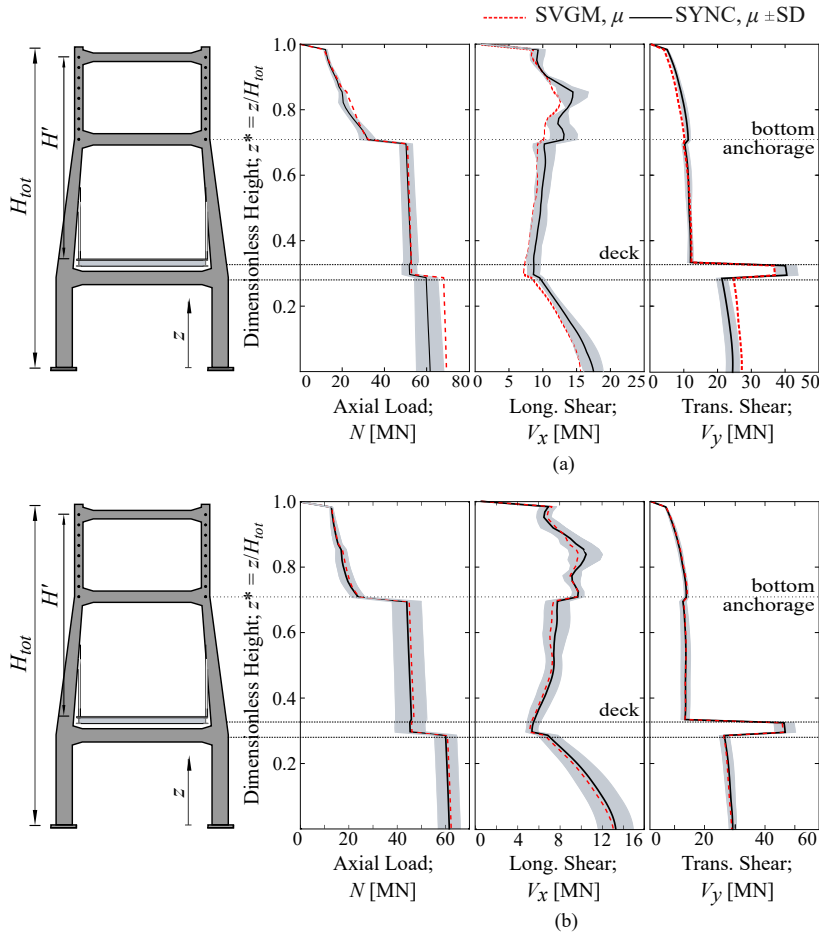
332 The results of the elastic and inelastic analyses of the cable-stayed bridge in  
 333 Fig. 7(a) demonstrate that the SVGM consistently results in smaller  $V_x$  than the  
 334 SYNC motion along the height of the pylon, implying that the multi-support

excitation is not important in the longitudinal response of the pylon. Figure 7(b) shows that when the material nonlinearities in the pylon are considered the transverse response above the deck from the SVGM resembles the seismic SYNC response. This is because the transverse out-of-phase oscillation of the two pylons is not significantly restrained by the cable-system. On the other hand, the SVGM reduces the inelastic  $V_y$  at the deck level of the pylon compared to the SYNC motion, but increases the elastic  $V_y$ . This is because in this region of the pylon the SVGM results in larger damage than considering the SYNC ground motion. The results from Fig. 7 indicate that the SVGM is more detrimental for the transverse response of the pylon than it is for the longitudinal response because in the former case the reduction of the peak shear force demand is greater than that from the SYNC motion. This important observation suggests that the effect of the longitudinal restraint of the cable-system to asynchronous motions is less important than the increased transverse deck-pylon reaction under the SVGM.

### 3.2 Effect of the Incidence Angle of the Ground Motion

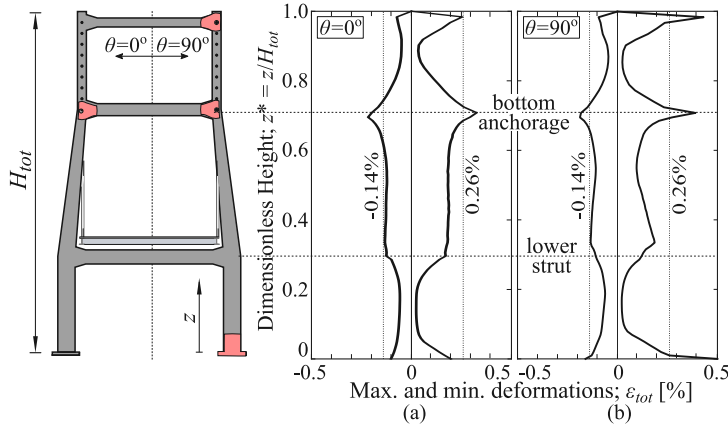
The influence of the earthquake incidence angle is explored by comparing the seismic responses of the intermediate-span bridge when the FN earthquake component is parallel (i.e. when  $\theta = 0^\circ$ ) and perpendicular ( $\theta = 90^\circ$ ) to the traffic. The results are presented in Fig. 8 which shows that the longitudinal shear force,  $V_x$ , is larger when  $\theta = 0^\circ$  than that when  $\theta = 90^\circ$  for the SYNC motion and for the SVGM. Accordingly, the transverse shear force,  $V_y$ , when  $\theta = 90^\circ$  is larger than the obtained response when  $\theta = 0^\circ$  for the SYNC motion as well as for the SVGM. The differences in the response between the two orientations is related to the FP earthquake component being 70% of the FN component. Specifically, Fig. 8 shows that  $V_{x\text{ IN,SYNC}}$  at the base of the pylon is reduced by 25% when  $\theta = 90^\circ$  compared to the responses obtained when  $\theta = 0^\circ$ . On the other hand, the transverse shear,  $V_{y\text{ IN,SYNC}}$  at the deck-pylon connection level is increased by 15%. This shows that the seismic response in each direction is maximised when the strong earthquake component is applied parallel to this direction. The axial load at the base of the pylon is not affected by the different orientations of the bridge in the SYNC motion case. Figure 8 also demonstrates the negligible effect of the SVGM on the three directions of the seismic response when  $\theta = 90^\circ$  compared to the SYNC motion. As discussed in Section 2.2 the wave-passage and the incoherence effects are orientation dependent and hence when  $\theta = 90^\circ$  the problem is reduced to the SYNC motion of the supports.

Figure 9 presents the plastic strains along pylon P2 of the H-LCP bridge with  $L_P = 400$  m when  $\theta = 0^\circ$  and  $90^\circ$  under the SYNC motion. The sections of the pylon that exceed the strain limits defined in Section 2.3 are highlighted with red colour. In order to distinguish between the two orientations, the plastic strains when  $\theta = 0^\circ$  are noted on the left half of the pylon and the strains when  $\theta = 90^\circ$  are included on the right half. The large plastic strains in the



**Fig. 8** Peak inelastic seismic  $N$ ,  $V_x$  and  $V_y$  in pylon P2 when (a)  $\theta = 0^\circ$  and (b)  $\theta = 90^\circ$ . H-LCP bridge;  $L_P = 400$  m.

378 steel reinforcement bars suggest that the connections of the legs with the upper  
 379 and the intermediate struts along with the base of the pylon are the most  
 380 sensitive regions of the pylon. In these sections, the steel exceeds its yielding  
 381 deformation and the concrete softens in compression. The peak tensile deforma-  
 382 tions are larger than the respective peak compressive deformations because  
 383 when cracking is initiated in the concrete ( $\varepsilon_{c,crack} = 0.0086\%$ ) the neutral axis  
 384 of the section is shifted towards the compression part of the section, resulting  
 385 in higher increments of the tensile deformation. The incidence angle of the  
 386 earthquake influences the amount of cracking, which is larger when  $\theta = 90^\circ$ ,  
 387 as shown in Fig. 9(b). When  $\theta = 90^\circ$  the strong earthquake component is ap-  
 388 plied parallel to the pylon struts and the relative oscillation of its lateral legs  
 389 is more pronounced, which explains the 50%-increase in the inelastic deforma-  
 390 tions at the connection of the legs with the upper transverse struts compared



**Fig. 9** Peak inelastic deformations of the concrete (compression) and the steel reinforcement (tension) in pylon P2 when (a)  $\theta = 0^\circ$  and (b)  $\theta = 90^\circ$ . H-LCP,  $L_P = 400$  m, SYNC ground motion. The highlighted areas in the pylon denote yielding of the materials.

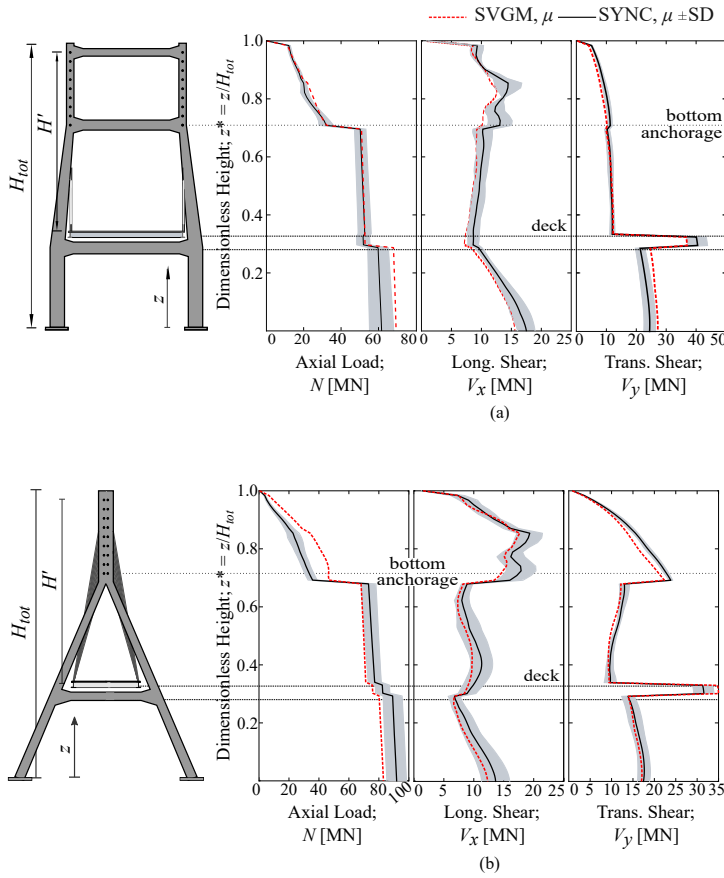
391 to the deformations for  $\theta = 0^\circ$  in Fig. 9(a). Similarly, the plastic deformation  
 392 is increased by 17% at the connection between the legs and the intermediate  
 393 struts and by 150% at the base of the pylon when  $\theta = 90^\circ$ .

### 394 3.3 Influence of the Pylon Shape

395 The peak seismic forces in the 400-m span bridges with ‘H’- and inverted  
 396 ‘Y’-shaped pylons is compared in Fig. 10 when  $\theta = 0^\circ$ . The difference in the  
 397 seismic response between the two pylon configurations is initially examined in  
 398 the SYNC motion. The inclined legs of the inverted ‘Y’-shaped pylon connected  
 399 above the deck result in an overall stiffer pylon configuration compared to the  
 400 relatively flexible ‘H’-shaped pylon [10], which works as a Vierendeel truss in  
 401 the transverse direction. For this reason,  $V_y$  is almost double at the level of the  
 402 bottom anchorage in the pylon of the Y-LCP model than the one at the same  
 403 region of the ‘H’-shaped pylon, and the seismic axial force,  $N$ , is 50% larger.  
 404 This is associated with the reaction of the deck to the pylon legs which, due  
 405 to their outward inclination, is resolved into one component perpendicular to  
 406 the axis of the legs and the other parallel to it. Given the large inclination of  
 407 the legs in the inverted ‘Y’-shaped pylon, the component of the deck–pylon  
 408 reaction that is parallel to the legs is larger than that in the ‘H’-shaped pylon,  
 409 which justifies the larger seismic axial force in the Y-LCP model. On the other  
 410 hand, the force component perpendicular to the legs is larger in the H-LCP  
 411 model due to the slight inclination of the intermediate part of the legs in this  
 412 type of pylons. This explains the 30% increase in  $V_y$  at the deck level of the  
 413 H-LPC model compared to the Y-LCP model.

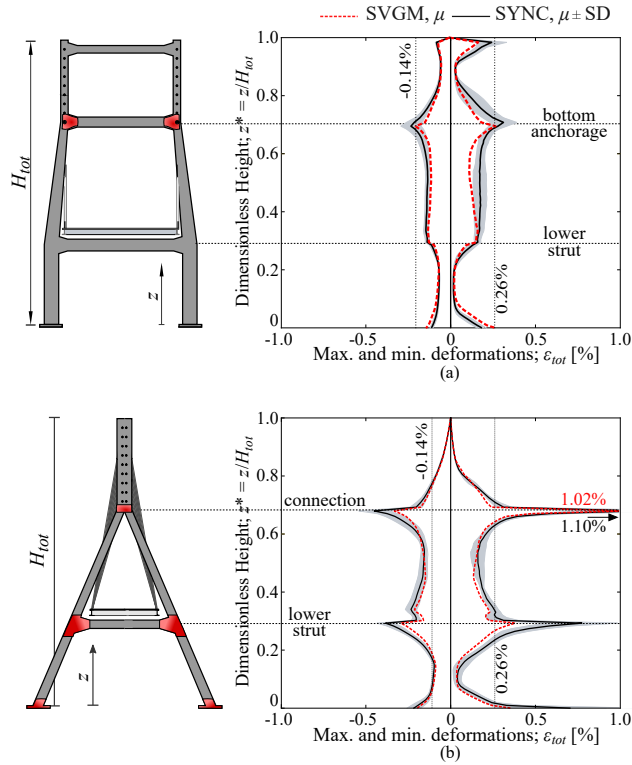
414 The top vertical member of the pylon of the Y-LCP bridge differentiates  
 415 the effect of the SVGM from that obtained in the individual legs of the ‘H’-

416 shaped pylon. The SVGM increases  $N$  by a maximum of 40% compared to the  
 417 SYNC motion at the anchorage area of the pylon with inverted ‘Y’ shape, which  
 418 indicates that the SVGM affects more this bridge than the one with ‘H’-shaped  
 419 pylons. More specifically, in the anchorage area of the pylon of the Y-LCP  
 420 bridge  $\rho_{N,Y-LCP} = 1.41$  whereas  $\rho_{N,H-LCP} = 1.0$  in the same area of the H-  
 421 LCP model. This increment can be explained by the increased stiffness of the  
 422 top part of the inverted ‘Y’-shaped pylon compared to the ‘H’-shaped pylon.  
 423 In pylons of inverted ‘Y’ shape, the transverse movement of the connection  
 424 between the individual legs and the top vertical member is constrained due to  
 425 the inclination of the legs, resulting in a stiffer configuration than that of ‘H’-  
 426 shaped pylons which behave as Vierendeel beams in the transverse direction.



**Fig. 10** Mean peak inelastic seismic forces along the height of pylon P2 in the (a) H-LCP and (b) Y-LCP models.  $L_P = 400$  m;  $\theta = 0^\circ$  (i.e. FN // traffic).

427 Despite the different geometries of the inverted ‘Y’- and the ‘H’-shaped  
 428 pylons, the part of the pylons at the deck level in both configurations is the  
 429 most affected by the SVGM because of the reaction that the deck exerts to



**Fig. 11** Peak inelastic deformations of the concrete and the steel reinforcement in pylon P2 in the (a) H-LCP and (b) Y-LCP models.  $L_P = 400$  m,  $\theta = 0^\circ$ .

430 the pylons. In the Y-LCP model  $\rho_{V_y, Y-LCP} = 1.09$  whereas in the ‘H’-shaped  
 431 pylon  $\rho_{V_y, H-LCP} = 0.92$  at the level of the deck–pylon connection. The transverse  
 432 response ratios suggest that the stiffer transverse configuration of the  
 433 inverted ‘Y’-shaped pylon makes it less capable to accommodate effectively  
 434 the multi-support excitation than the more flexible ‘H’-shaped pylon. In the  
 435 longitudinal direction of the response ( $V_x$ ), the SVGM reduces the seismic response  
 436 of both pylon configurations from the SYNC excitation regardless of  
 437 their shape, showing that the asynchronous excitation of the supports is not  
 438 critical in this direction.

439 Figure 11 compares the peak demand for deformations in the H-LCP and  
 440 Y-LCP models with  $L_P = 400$  m when  $\theta = 0^\circ$ . In the inverted ‘Y’-shaped  
 441 pylon the most critical region is the connection between the inclined legs and  
 442 the top vertical member, where the maximum plastic strains reach 1.02% and  
 443 1.10% for the SVGM and the SYNC motions, respectively. This is explained by  
 444 the connection between the inclined legs at this point which strongly constrains  
 445 the pylon in the transverse direction. The connection between the legs and the  
 446 strut, and the base of the pylon also represent critical sections where there  
 447 is yielding of the steel in tension and softening of the concrete in compress-

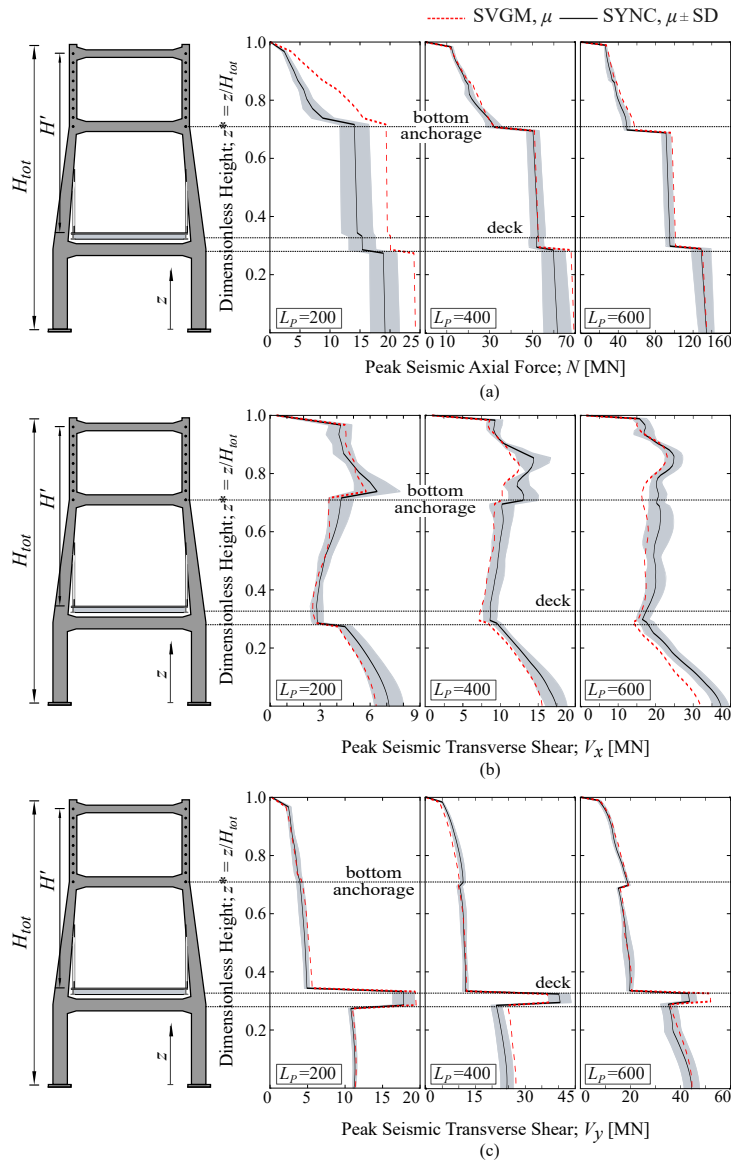
448 sion. However, the peak plastic strains from the asynchronous excitations are  
 449 smaller than the strains from the SYNC motion by  $\rho_{\varepsilon_{tot}} = 0.57$  in the inverted  
 450 ‘Y’-shaped pylon. The stiff configuration of the inverted ‘Y’-shaped pylon re-  
 451 sults in larger deformations at the critical sections compared to the ‘H’-shaped  
 452 pylon, as shown in Fig. 11, suggesting that the ‘H’-shaped pylon is a better  
 453 candidate to accommodate the seismic action in the range of intermediate  
 454 spans with  $L_P \approx 400$  m. This is in agreement with the elastic seismic analysis  
 455 under SYNC motion conducted by Camara and Efthymiou [10] who observed  
 456 that bridges with ‘H’-shaped pylons are less sensitive to the transverse seismic  
 457 reaction of the deck compared to bridges with inverted ‘Y’ pylons. However,  
 458 the SVGM is not critical for the response of either pylon shapes when  $\theta = 0^\circ$ .

### 459 3.4 Influence of the Pylon Height

460 Figure 12 presents the peak seismic response in terms of  $N$ ,  $V_x$  and  $V_y$  in  
 461 the pylons of the H-LCP models with  $L_P = 200, 400$  and  $600$  m and  $H_{tot} =$   
 462  $62.5, 125$  and  $187.5$  m, respectively. It is observed that by increasing the main  
 463 span length ( $L_P$ ), the pylons are less vulnerable to the SVGM in terms of the  
 464 vertical response,  $N$ . This is shown by the fact that the SVGM increases  $N$   
 465 in the relatively short pylon of the 200-m span bridge compared to the SYNC  
 466 motion. Oppositely, both SVGM and SYNC motions result in similar values of  $N$   
 467 in the intermediate and tall pylons of the 400- and 600-m bridges, respectively.

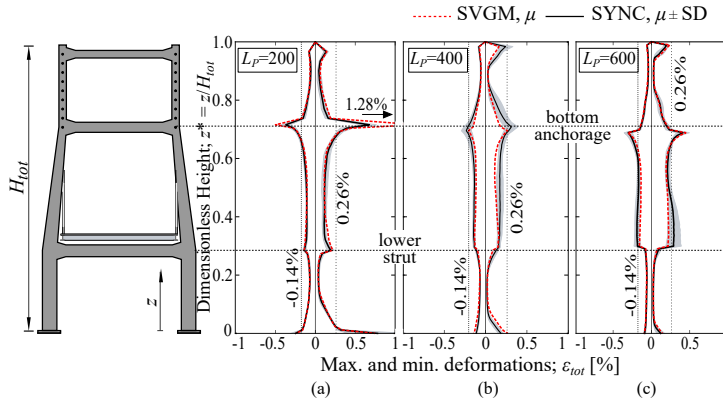
468 The longitudinal response of the pylon is generally reduced by the SVGM  
 469 regardless of the main span length which is reflected in the values of the  
 470 response ratios;  $\rho_{V_x}$  ranges from 0.89 in the 200- and the 400-m span bridges  
 471 to 0.86 in the 600-m span bridge. On the other hand, at the deck level of  
 472 the pylon, increasing  $L_P$  has a considerable effect on the transverse response  
 473 due to the SVGM.  $V_y$  is increased by the out-of-phase motion in the pylons of  
 474 the 200- and the 600-m span bridges but is reduced when  $L_P = 400$  m. The  
 475 large seismic shear force in the pylon of the shortest bridge can be attributed  
 476 to the stiffer configuration of this pylon compared to its longer counterparts  
 477 when  $L_P = 400$  or  $600$  m, given that stiff structures are reportedly vulnerable  
 478 against the pseudo-static forces caused by the SVGM [37,41]. *At the other end,*  
 479 *the largest time-delay and loss of coherency in the 600-m span bridge combined*  
 480 *with the increased height of the pylons and flexibility, result in pronounced*  
 481 *transverse oscillations of the pylons and significant deck–pylon reactions. This*  
 482 *explains the large  $V_y$  at the deck level of the pylon of the bridge with  $L_P = 600$*   
 483 *m. Therefore, it is important to address the effect of the SVGM on the seismic*  
 484 *response of the pylons not only on the basis of the length of the bridge, but*  
 485 *also in terms of the pylons’ height and geometric characteristics.*

486 At the other end, the largest time-delay and loss of coherency in the 600-m  
 487 span bridge combined with the increased height of the pylons and flexibil-  
 488 ity, result in pronounced transverse oscillations of the pylons and significant  
 489 deck–pylon reactions. This explains the large  $V_y$  at the deck level of the pylon  
 490 of the bridge with  $L_P = 600$  m



**Fig. 12** Mean peak inelastic seismic (a)  $N$ , (b)  $V_x$  and (c)  $V_y$  along the height of pylon P2. H-LCP models;  $\theta = 0^\circ$  (i.e. FN // traffic).

491 Figure 13 presents the peak deformations in pylon P2 of the bridges with  
 492 200, 400 and 600 m main spans and 'H'-shaped pylons. With increasing main  
 493 span length, the height of the pylon increases proportionally and it is observed  
 494 that the peak deformations also increase in the connection between the legs  
 495 and the upper strut. However, the short-span bridge ( $L_P = 200$  m) is the  
 496 one subjected to the largest demand of deformation at the region of the con-



**Fig. 13** Peak inelastic deformations of the concrete and the steel reinforcement in pylon P2 in the bridges with (a)  $L_P = 200$  m, (b)  $L_P = 400$  m and (c)  $L_P = 600$  m.  $\theta = 0^\circ$ . H-LCP models.

497 nection between the legs and the intermediate struts above the deck, whereas  
 498 the reinforcement in the pylons of the 400-m span bridge barely exceeds its  
 499 yielding point. The stiff pylon of the 200-m span bridge proves to be the most  
 500 sensitive to the ground motion, especially under the SVGM, in which case the  
 501 deformation ratio is  $\rho_{\varepsilon_{tot}} = \frac{\varepsilon_{tot,SVG M}}{\varepsilon_{tot,SYNC}} = 2$ . At the other end, the flexible pylon  
 502 of the 600-m bridge is also susceptible to damage at the connection between  
 503 the legs and the intermediate struts, but this is due to the relative transverse  
 504 displacement between the legs that is accommodated by the struts. Further-  
 505 more, as the height of the pylon increases, the number of the cable anchorages  
 506 increases proportionally, which adds to the vibration mass at the top of these  
 507 structures and contributes to their relative displacement.

#### 508 4 Conclusions

509 This paper examines the nonlinear response of cable-stayed bridges with dif-  
 510 ferent pylon shapes and span lengths under synchronous (SYNC) and asyn-  
 511 chronous ground motions (SVG M) with different incidence angles. The seismic  
 512 response of these bridges is discussed in terms of the peak seismic forces ( $N$ ,  
 513  $V_x$  and  $V_y$ ) and the deformations ( $\varepsilon_{tot}$ ) that are developed in the pylons. The  
 514 main conclusions drawn from this study are the following:

- 515 1. The shortest bridge ( $L_P = 200$  m) is more sensitive to the SVG M than  
 516 the intermediate- and long-span bridges with  $L_P = 400$  and 600 m, re-  
 517 spectively. The pylons of the 400-m span fall between the stiff pylons of  
 518 the 200-m span bridge and the tall and flexible pylons of the 600-m span  
 519 bridge. In the latter case the effect of the components of the SVG M (wave-  
 520 passage and incoherence) are prominent because they are both functions  
 521 of the separation distance between supports. These findings emphasise the

522 need for code-based provisions in which the importance of the SVGM for  
523 the design and assessment of bridges is established not only in terms of  
524 the total length but also in terms of the geometric characteristics of the  
525 structure.

- 526 2. The shape of the pylon influences the sensitivity to the asynchronous motion. The stiff configuration of the inverted 'Y'-shaped pylon is more susceptible to the SVGM compared to the flexible 'H'-shaped pylon. The most critical parts of the pylons are the connections between the legs and the transverse struts. The lateral flexure of the pylons is accommodated by the struts and the geometry that it conforms with the lateral legs.
- 532 3. The SVGM is detrimental to the transverse response of the pylons because in this direction the pylon shape provides larger geometric stiffness than the one given by the constraint from the cable-system in the longitudinal direction. On the contrary, the longitudinal seismic response is consistently reduced by the multi-support excitation regardless of the height or the shape of the pylon, implying that the SVGM could potentially be neglected in this direction of the structural response.
- 539 4. The earthquake incidence angle with respect to the bridge ( $\theta$ ) is closely linked with the SVGM and with the response quantity under consideration.  $V_x$ , in the pylons is maximised when  $\theta = 0^\circ$  and the peak  $V_y$  is maximised when  $\theta = 90^\circ$ , which is associated with the direction of the strong earthquake component. The wave-passage and incoherence effects are orientation dependent; therefore the effect of the SVGM is maximum when  $\theta = 0^\circ$ .

## 546 Funding

547 This research did not receive any specific grant from funding agencies in the  
548 public, commercial, or not-for-profit sectors.

## 549 Conflict of interest

550 The authors declare that they have no conflict of interest.

## 551 Availability of data and material

552 Data sharing not applicable to this article as no datasets were generated or  
553 analysed during the current study.

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