Seismic Response of Cable-Stayed Bridges for Different Layout Conditions: A Comparative Analysis

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SUMMARY:
This work presents a numerical comparative seismic analysis of the response of cable-stayed bridges for different stay cable arrangements, in which the main objective is to propose the best structural configurations from a seismic point of view. Firstly, eight symmetric concrete theoretical cable-stayed bridge models based on the well-known Walther's Bridges are defined considering variations of the cable arrangement, deck level and stay spacing. As a starting point, a nonlinear static analysis is performed for all the cases in order to compute the main geometric nonlinearities involved with the overall change in the bridge geometry, nonlinear cable sag effect and axial force-bending moment interaction in towers and girders. After that, the dynamic characterization of the models is carried out by means of a modal analysis considering the modified stiffness matrix obtained from the nonlinear static analysis. In order to compare the maximum seismic responses as function of the main variations considered, a response spectrum analysis is performed for all the structures considering strong ground motions according to Eurocode 8.

Keywords: Cable-stayed bridge, seismic response, modal analysis, response spectrum analysis

1. INTRODUCTION

Bridges are very vulnerable structures, and essential as life-lines, consequently the understanding of their seismic behaviour is fundamental. Cable-stayed bridges, due to their large dimensions and flexibility, usually experience long fundamental periods, aspect that makes the difference with respect to other structures, and of course, that affects their dynamic behaviour. However, their flexibility and dynamic characteristics depend on several parameters such as the main span length, stay system and their layout, support conditions and many other things [Walther, 1999].

Modal analysis results on cable-stayed bridges are discussed in many papers, with emphasis on the seismic behaviour. First vibration modes show a very long period, in the order of several seconds, and they are fundamentally deck modes. They are followed by cable vibration modes, coupled with the deck. The tower modes are usually of higher-order, which can be coupled with the deck depending on the support conditions. Undoubtedly, the modes are very difficult to separate when they are sufficiently coupled [Morgenthal, 1999]. For typical cable-stayed bridges strong coupled modes (like bending and torsion) in the three orthogonal directions can be appreciated. This coupled motion makes the difference regarding suspension bridges, in which pure vertical, lateral and torsional motion, very easy to recognize, is experienced. This implies a three-dimensional system modelling [Wethyavivorn and Fleming, 1987]. An exact analysis of natural frequencies and modal shapes on cable-stayed bridges is very important, not only for the study of the seismic response, but also for wind action and traffic loads.

Due to their nature, long-span cable-stayed bridges have a predominant non-linear behaviour. The static non-linear analysis under dead loads is essential as a starting point for the non-linear seismic analysis, taking the deformed state for dead load as previous condition for a dynamic analysis [Abdel Ghaffar, 1991]. For main span bridges longer than 600 m, geometric and material non-linear analyses are necessary when the structures are subjected to strong motions. Those material nonlinearities depend on the specific structure, but geometric nonlinearities are present in almost all cable-stayed bridges, especially in the stay cable sag effect, the compressive action in deck and towers, and the large deflections effect due to the flexibility of this kind of structures [Morgenthal, 1999]. In fact, the investigation developed by Ren (1999) gives a good analysis of the effects and importance of both kinds of nonlinearities on cable-stayed bridges.

Since last decade, research regarding the seismic behaviour of cable-stayed bridges has been focused on the study of the seismic response of towers (Hayashikawa *et al*, 2000), the spatial variability effects (Soyluk and Dumanoglu (2004), Abdel-Raheem *et al* (2011)); the effect of the cable vibrations on the seismic response (Caetano *et al*, 2000), the effect of the vertical component (Button *et al*, 2002; Jia and Ou, (2008)), the seismic response of multi-span cable-stayed bridges (Ni *et al*, 2005) and the effect of the stay prestressing forces on the seismic response (Valdebenito and Aparicio, 2008). Calvi *et al* (2010) have published the basis of the conceptual seismic design of cable-stayed bridges. On the other hand, a state-of-the-knowledge regarding the seismic protection of cable-stayed bridges can be found in the works of Valdebenito and Aparicio (2006, 2009).

2. BRIDGE MODELS AND CONSIDERATIONS

This research takes into consideration eight concrete *3-D* symmetric bridge models for an adequate parametric analysis. The chosen bridges were taken from Walther's Bridges [Walther, 1999]. The examination is based on a symmetrical reference structure with multi-stays, two concrete pylons and a main span of about 200 m, with the materials and mechanical properties shown in Table 1. Two stay cable layouts were selected: fan-type (Figures 1a to 1d) and harp-type (Figures 1e to 1h). The semi-harp pattern was rejected because this typology is an intermediate pattern, and both harp and fan patterns are enough for an adequate analysis. Moreover, the deck pattern considers two cases: a slab-type deck and a hollow-box deck, both made of prestressed concrete. The first type, due to its inherent flexibility, considers a stay spacing of 6.20 m. In the second case, 12.40 m - stay spacing is considered.

Materials and their mechanical properties have been chosen according to general specifications and regulations for bridge design, and taking into account seismic considerations [Priestley *et al*, 1996; Walther, 1999; Aparicio and Casas, 2000]. For the seismic design, high strength concrete is employed, with a characteristic strength (f_{ck}) of 40 MPa. For the steel for reinforced concrete, welding steel *B-400SD* with special characteristics of ductility is employed, with an elastic limit (f_y) of 400 MPa. The stays have been considered applying parallel-strand cables, with an ultimate tensile strength (f_u) of 1900 MPa.

Table 1. Material data for major components of the bridges

Component	Modulus of elasticity (MPa)	Volumetric weight (kN/m^3)	Poisson's ratio	Thermal expansion coeff. ($1/^0C$)
Concrete (Decks, pylons, struts)	36000 (28 days)	25	0.20	1.43×10^{-5}
Steel (for reinforced concrete)	2.1×10^5	78.5	0.30	1.1×10^{-5}
Steel (for cables)	1.9×10^5	78.5	0.30	1.1×10^{-5}

Moreover, the geometric properties of the selected decks are shown in Table 2, where A is the area of the cross-section, A_{vy}, A_{vz} are the shear areas with regard to the principal axes y and z, J_T is the torsion constant and I_y, I_z are the moments of inertia with regard to the principal axes.

The selected tower, for all cases, is a concrete frame-type tower, with deck levels of 30 and 60 m from bottom (Figure 2). The height of the towers is 81 m and 111 m respectively.

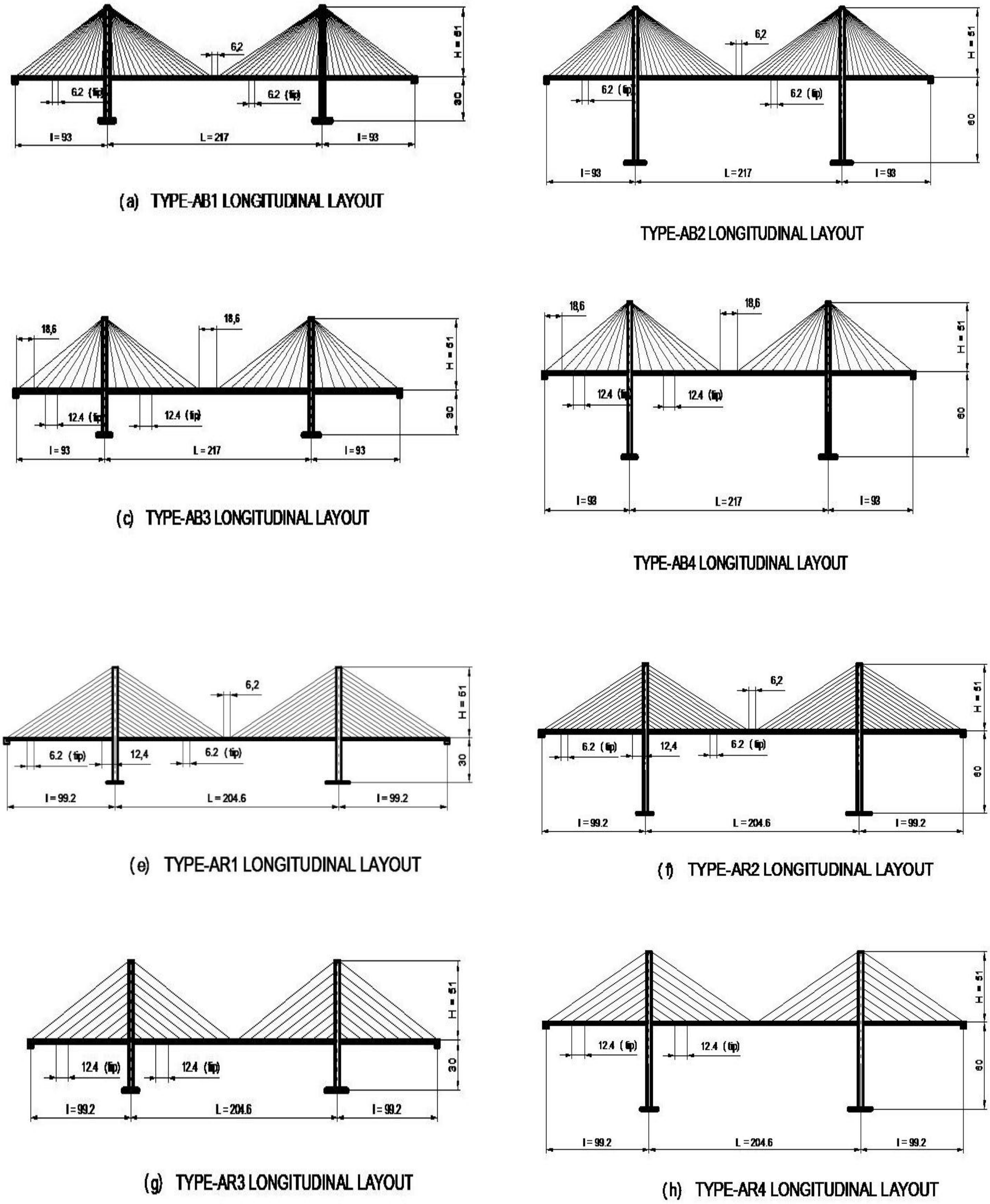
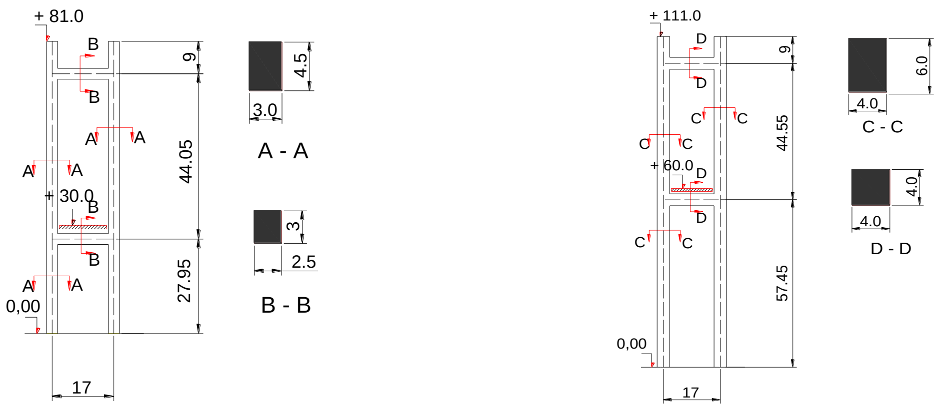


Figure 1. Longitudinal layout of the bridges

Table 2. Geometric properties of the selected decks

Deck-type	Height (m)	A (m^2)	A_{Vy} (m^2)	A_{Vz} (m^2)	J_T (m^4)	I_y (m^4)	I_z (m^4)
Slab	0.40	5.20	4.33	4.33	0.272	0.0693	73.23
Hollow-box	2.25	5.20	3.20	1.06	9.735	3.710	63.489



(a) A-type tower (b) B-type tower

Figure 2. Selected towers for the analysis (dimensions in metres)

For definition of the actions in this research, the criteria of the *Dirección General de Carreteras de España* [Ministerio de Fomento, 1998] and the specific regulations of Eurocode 8 - Part 2 [CEN, 1998a] regarding the seismic action on bridges, were applied. In this investigation, the bridges were considered with a medium importance and normal design traffic. These considerations involve a seismic importance factor $\gamma_I = 1.00$ according to Eurocode 8 – Part 2, and a live load factor $\psi_{21} = 0$, according to Eurocode 1 - Part 3 [CEN, 1998b]. By this way, to study the seismic response of the bridges, the only considered actions were the permanent loads (q_{PL}), the stay prestressing forces (q_{SPL}) and the seismic action of course (q_E). These considerations are reasonable because the permanent loads of a cable-stayed bridge may contribute 80 – 90% to total bridge loads [Ren and Obata, 1999].

Regarding the bridge modelling, the analysis was carried out considering the use of beam and cable elements. In fact, the use of beam elements can be more useful to compute forces on members, with clear graphical results and a considerable decrease of the computing time, especially when non-linear behaviour is considered. The decks were modelled using a single spine with the exact mass and inertias passing through the centroid of the cross-section, applying linear beam elements and including zero-mass transverse rigid-links to simulate the anchor of cables. In the same way, the towers were represented by three-dimensional portal frames, with tower legs and struts modelled by linear elastic beam elements based on gross cross-section properties, and the application of rigid-links for the strut-leg connection. The cables were idealized applying a multi-element cable formulation with a discretization employing 5-node isoparametric cable elements, based on a Lagrangian formulation [Ali and Abdel-Ghaffar, 1995; Förars *et al*, 2000]. In order to keep longitudinal displacements to minimum values, fixed hinge connections between decks and towers as well as roller supports at the deck-ends were employed. The towers were founded to bedrock and their bases were treated as being fixed in all degrees-of-freedom at the piers. Because of the inherent non-linear behaviour of cable-stayed bridges, mainly of geometric type, some nonlinearities were accounted for, and specifically, the non-linear behaviour of towers and girders due to axial force-bending moment interaction ($P - \Delta$ effects), and the non-linear cable sag effect due to the inclined cable stays which governs axial elongation and the axial tension. This non-linear behaviour of cables is considered by a multi-element cable formulation with *tension-only* members, in order to take into account the spatial vibrations of them. All the analyses were computed using the code SAP2000 (Computers & Structures, 2007).

Although the subsequent analyses are based on the modal and response spectrum analysis, the apparent contradiction that implies the consideration of the nonlinear behaviour previously exposed, has the main objective of include those nonlinearities in the evaluation of the stiffness matrix (Valdebenito and Aparicio, 2009).

3. COMPARATIVE STUDY

3.1. Modal analysis

The total number of modes was selected to reach at least 90% of the effective translational mass, in which more than 300 modes were necessary, although a total of 30 natural periods for each bridge were obtained that range between 0.24 and 2.94 sec, with important participative masses in some cases, as can be seen in the previously work by Valdebenito and Aparicio (2009). Depending on the relative amplitudes of the modal shapes, these modes were classified into the following groups: vertical modes (*V*), transverse modes (*Tr*), longitudinal modes (*L*) and torsional modes (*Tor*) for deck and towers.

For all the bridges, first vibration modes correspond to deck modes. They are followed by tower or cable modes, depending on the geometric configuration. Likewise, for almost all the bridges, higher vibration modes are associated with the vertical motion of the deck. In general terms, it was observed that longitudinal deck motion governs the first modal shape, except the bridges *AB3* and *AR3* governed by the vertical deck motion and the bridge *AB1* governed by the transverse deck motion, that is to say, the tallest bridges are controlled by the longitudinal deck motion. Likewise, it seems that for 30 m deck level with stay spacing of 12.4 m, vertical deck motion governs the first modal shape. Another interesting observation can be appreciated with the modal shapes related to the deck torsion. It seems that the deck torsion is coupled with a longitudinal motion of the tower legs, independently on the deck type, stay spacing, stay cable layout and deck level. This implies that the torsion generated by the eccentricity of the cross-section of the deck can be ignored. Cable-deck coupling can be observed in some cases, specifically for the bridges *AB2*, *AR1*, *AR2* and *AR4*. It seems to be that the harp pattern shows evidences of cable-deck interaction in this case. On the other hand, it was observed strong modal coupling, a very important characteristic of the dynamic behaviour of cable-stayed bridges.

It is interesting to observe that the first ten modes for all bridges are associated with periods that range between 0.4 and 2.94 sec, with important participative masses in some cases, that is to say, those structures can be more affected by velocity than acceleration or displacements according to Eurocode 8 [CEN, 1998a]. In fact, this code explains that velocity-sensitive region corresponds to periods in the range between $0.4<T<3$ *sec.*These results can be observed in detail in the work of Valdebenito and Aparicio (2009).

3.2. Seismic response analysis

Response spectra considered in this research were obtained from Eurocode 8 [CEN, 1998a, 1998b]. This code was selected because it considers specific recommendations and the definition of the response spectra for bridges. The structural parameters involved with the definition of the response spectra consider a medium importance for the bridges and an elastic seismic behaviour (behaviour factor q equal to 1.0). The structures are founded on bedrock, and the considered maximum effective ground acceleration is $0.5g$ for the horizontal component, and $0.35g$ for the vertical component, where g is the gravity acceleration. These values are representative for structures located in high seismicity areas founded on bedrock, as usually happens in the subduction zone of the Mexican coast (Pacific ocean) [CFE, 1993]; several areas of the California coast [AASHTO, 1994, section 3.10]; and some areas of Japan [Japan Road Association, 1996, section 6.3]. The vertical component was assessed as a function of the horizontal one, according to Eurocode 8. With regard to the modal superposition, *CQC* modal combination rule was applied because of the strong modal coupling that cable-stayed bridges experience. The bridge models were analyzed for each load condition, in which the seismic components were directionally combined applying the *30% rule* according to Eurocode 8. The importance of the stay prestressing forces on the overall seismic response, is taking into account according to the work by Valdebenito and Aparicio (2008).

The analysis results show some differences regarding the longitudinal displacements of the shortest towers (bridges *AB1*, *AB3*, *AR1* and *AR*3), aspect that is not obvious for the case of the tallest bridges.

In fact, for the shortest towers, maximum displacements can be obtained for the bridges *AR1* and *AB3*. In the same way, longitudinal displacements of the tower of *AB1* bridge are larger than longitudinal displacements of the tower of *AR3* bridge; however, not very interesting conclusions can be formulated according to the above mentioned, as can be seen in Figs. 3 and 4. In the case of the 111 m height-towers, differences regarding the longitudinal displacements of the towers are negligible (Fig. 3). Of course, maximum longitudinal displacements of the towers are obtained for the tallest bridges, with maximum values at the tower-top of about 40 cm for the tallest towers, and 30 cm for the shortest ones. Likewise, it can be appreciated that maximum longitudinal displacements at the tower-top for *AB3* bridge are larger than maximum longitudinal displacements at the tower-top of *AB1* bridge. In the case of *AB4* bridge, maximum top displacements are larger than displacements of *AB2* bridge. A similar situation can be observed with the maximum longitudinal displacements at the tower-top of the bridges *AR3*, *AR1*, *AR4* and *AR2*; concluding that bridges with longer stay spacing experience an increase of the longitudinal displacements of the tower-top.

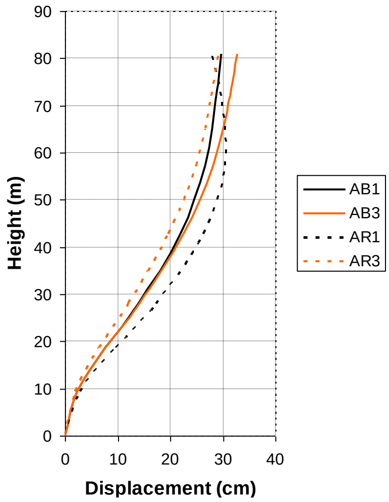


Figure. 3. Maximum Seismic Longitudinal Displacements for 81 m - height Towers

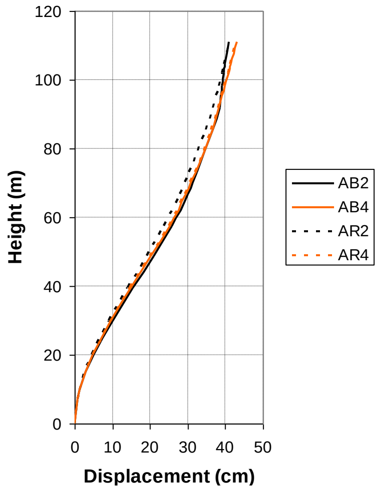


Figure 4. Maximum Seismic Longitudinal Displacements for 111 m - height Towers

With regard to the maximum vertical seismic displacements of the decks, more interesting observations can be formulated. Because of the differences with the flexural stiffness of the decks, the analysis was carried out considering the slab-type deck (bridges *AB1*, *AB2*, *AR1* and *AR2*) and the hollow-box type deck (bridges *AB3*, *AB4*, *AR3* and *AR4*) separately. In the first case, maximum vertical displacements are concentrated in the extreme spans (in the first third-length) and the vicinity of the main-span centre (see Fig. 5), distribution that can be very different from the static condition. Maximum values of the vertical displacements vary from 33 to 40 cm, depending on the bridge model. It is clear that maximum vertical displacements are obtained for the *AR1* model, followed by the bridges *AR2*, *AB1* and *AB2* respectively. In the case of the hollow box-type deck (stay spacing of 12.4 m), the displacement distribution is very different from the slab-type deck, with maximum values concentrated in the half-length of the extreme spans, and the third-length of the main span, as can be seen. Maximum displacements vary from 23 to 33 cm, depending on the bridge model. Likewise, maximum values of deck displacements are obtained for *AR3* bridge followed by the bridges *AR4*, *AB3* and *AB4* respectively (see Fig. 6).

The comparison of internal forces shows interesting conclusions. The analysis of the compressive forces of the tower legs exposes variations for the maximum values (base) of the shortest towers with differences no greater than 5%. The results (Fig. 7) show important differences of the maximum values over the deck level (30 m), and increasing with the altitude. In fact, at the tower-top, differences up to 95% can be found. Also, Fig. 7 exposes that maximum compressive forces are obtained for *AB1* bridge, followed by *AR1* bridge (below the deck level) and *AB3* (above the deck level). The lowest compressive forces are obtained for *AR3* bridge. In a similar situation, the analysis of the compressive forces of the tower legs for the tallest bridges shows variations of the maximum values with negligible differences below the deck level (see Fig. 8), and more important variations above the deck level (60 m).

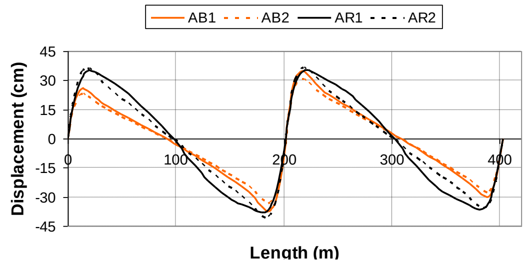


Figure 5. Maximum Vertical Seismic Displacements – Slab-type Deck

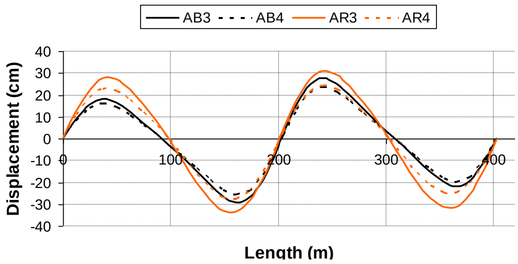


Figure 6. Maximum Vertical Seismic Displacements – Hollow box-type Deck

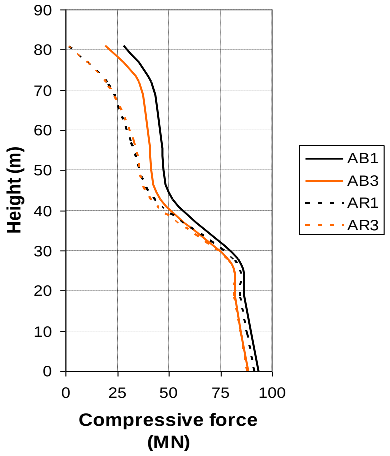


Figure 7. Envelope of Maximum Seismic Compressive Forces for 81 m-Height Towers

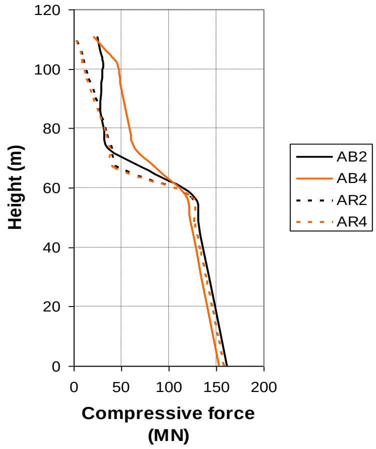


Figure 8. Envelope of Maximum Seismic Compressive Forces for 111 m-Height Towers

The analysis of bending moments for the towers shows a similar behaviour for all cases. As usually happens, maximum bending moments were obtained at the base of the tallest towers, with maximum moments varying from 403 to 496 MN.m (*AR4* bridge), which means a difference of 19%. For the case of 81 m-tower height, maximum bending moments of the towers vary from 178 MN.m (*AR1* bridge) to 337.6 MN.m (*AB1* bridge), which implies a difference of 48%.

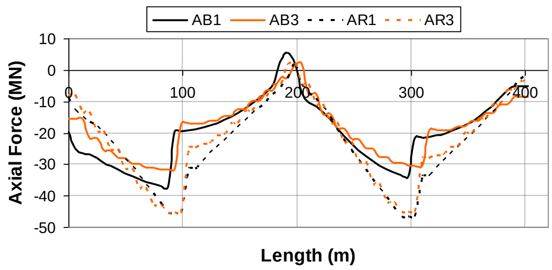


Figure 9. Envelope of Seismic Axial Forces for Decks – 81 m Tower-Height

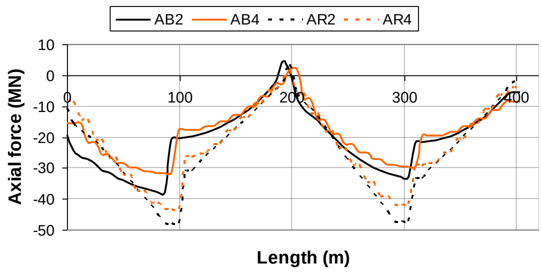


Figure 10. Envelope of Seismic Axial Forces for Decks – 111 m Tower-Height

Comparing the effect of the stay cable layout on the bending moments of the towers, it seems to be that maximum tower bending moments for bridges with fan pattern are higher than maximum tower bending moments for bridges with harp pattern. Regarding the influence of the stay spacing on the tower bending moments, it is easy to see that an increase of the stay spacing decreases the maximum tower bending moments, on the contrary of the harp pattern, for which an increase of the stay spacing implies an increase of the maximum bending moments of the towers.

From Table 3, maximum in-plane tower bending moments were obtained considering the earthquake acting in the longitudinal direction, and of course, in-plane bending moments are lower when the earthquake acts in the transverse direction. In this sense, analysis of out-of-plane tower seismic moments shows that tower moments when the earthquake acts in the transverse direction are lower than in-plane tower moments when the earthquake acts in the longitudinal direction, which implies that the worse analysis condition corresponds to the earthquake acting in the longitudinal direction.

The influence of the analyzed parameters can be summarized in Table 4, which shows the best configurations of the cable-stayed bridges only taking into account the seismic response of such structures applying the response spectrum method under strong ground motion. The most decisive response parameters include maximum displacements of the towers and decks; maximum axial forces of towers, decks and cables and maximum bending moments of towers and decks. As a result, Table 4 shows that reduction of vertical displacements of the deck can be better controlled by using bridges with fan pattern, and especially with high deck level. Longitudinal displacements of the towers are lower by using bridges with the shortest towers and the shortest stay spacing; and the longitudinal displacements of the deck are better controlled by using bridges with low deck level. In general terms, the bridge with the fan pattern cable layout, lowest deck level and shortest stay spacing (*AB1* bridge) is a good choice to reduce displacements; however it is not a good selection to control the internal forces. Regarding the reduction of internal forces, it is clear that the best option is the harp pattern, with the exception of the deck axial forces, for which fan pattern bridges with the longest stay spacing seems to be more adequate. Tower and deck bending moments are better controlled by using the harp pattern with the shortest stay spacing; and the cable forces are lower using the harp pattern with the longest stay spacing. A good choice to reduce both internal forces and displacements is the *AR1* bridge, that is to say, a harp pattern bridge, with the shortest stay spacing and deck level. According to this simplified analysis, the worse conditions are obtained with the fan pattern bridges, and especially the *AB2* and *AB4* bridges, which consider the highest deck level. Of course, because of the complex nature of the seismic phenomena, it is very difficult to reduce displacements and internal forces at the same time, and for that reason, these recommendations are only general guidelines. An only optimal solution does not exist, and this selection necessarily depends on the specific requirements of the bridge prototype. For this reason, the aim of this comparative analysis is to show some results regarding the incidence of some parameters associated with the geometric configuration of the structures on the seismic response of the bridges.

Table 3 Summary of Maximum Main Forces

BRIDGE	$N_{max\text{-}tower}^{a}$ [kN		$N_{max\text{-}deck}^{b}$ [kN]		$N_{max\text{-}cable}$ [kN]		$M_{max\text{-}tower}^{a,\,d}$ [MN.m]		$M_{max\text{-}tower}^{e}$ [MN.m]		$M_{max\text{-}deck}^{d}$ [MN.m]		Base Shear (kN)
	Stat	Seis	Stat	Seis	Stat	Seis	Stat	Seis	Stat[f]	Seis[a]	Stat[b]	Seis[c]	
AB1	-57200	-93500	-25200	-37400	4800	11600	18.0	337.6	8.15	197.3	1.58	10.8	57600
AB2	-101200	-161500	-25500	-38000	4950	11500	21.2	491.0	10.81	332.6	1.38	9.33	52250
AB3	-52900	-88600	-22100	-31800	5630	9090	15.7	301.5	7.50	195.9	13.3	62.2	56970
AB4	-97000	-153600	-22400	-31700	5770	9100	18.3	465.0	9.09	347.8	12.9	53.6	48360
AR1	-52900	-91400	-36400	-46700	1560	5260	11.0	200.0	6.55	186.0	3.94	7.50	40000
AR2	-97000	-158000	-36900	-47900	1790	6370	23.0	438.8	9.72	347.9	3.81	9.20	53670
AR3	-51300	-88300	-32000	-45500	2700	4660	19.5	287.2	6.36	204.2	22.9	53.1	59560
AR4	-95300	-158200	-32500	-43200	2820	5000	23.5	496.0	8.67	340.8	22.2	45.6	52140

[a] At the tower base
[b] At the tower-deck connection
- Implies compression
[c] Near the mid-span
[d] In the bridge plane
[e] Out-of-plane
[f] At the upper strut level

Table 4. Optimal Configurations to Reduce the Seismic Response Applying the Response Spectrum Method

BRIDGE	Displacements			Internal Forces				
	$\Delta_{3\text{-}V}$	$\Delta_{1\text{-}L}$	$\Delta_{4\text{-}L}$	$N_{max\text{-}tower}$	$N_{max\text{-}deck}$	$N_{max\text{-}cable}$	$M_{max\text{-}tower}$	$M_{max\text{-}deck}$
AB1	good	good	good		good			
AB2	very good				good			
AB3	good		good		very good			
AB4	very good				very good			
AR1		good	good	good		good	Very good	very good
AR2	good			good		good	Very good	very good
AR3			good	good		very good	good	good
AR4	good			good		very good	good	good

3. CONCLUSIONS

A comparative analysis was performed in order to study the main significances of the structural configuration of cable-stayed bridges on the seismic response under strong ground motion. The main conclusions are:

1. The modal analysis shows that first vibration modes correspond to deck modes (longitudinal and transverse oscillations, depending on the bridge configuration). They are followed by cable or tower modes, depending on the geometric layout. Influence of the torsional modes due to eccentricity of the cross sections of the hollow-box type decks can be ignored, and for that reason, only translational lumped masses are enough to be considered in the dynamic modelling of cable-stayed bridges. On the other hand, the close spacing of the natural periods is a vibrational characteristic of cable-stayed bridges, especially for higher order modes, implying strong modal coupling. Likewise, influence of the stay spacing on the determination of fundamental periods shows that variation of the longitudinal stiffness of the bridges is not important for low-to-moderate variations of the stay spacing. In this sense, an increase of the stay spacing not necessarily involves a decrease of the longitudinal stiffness.
2. Application of the response spectrum method shows that the best solution to reduce seismic displacements corresponds to fan pattern bridges with low deck level and short stay spacing. To reduce both internal forces and displacements, the harp pattern seems to be an efficient solution.
3. Response spectrum analysis must be employed only as first approach of the seismic response of cable-stayed bridges, and with comparative purposes. For design and accurate analyses, nonlinear time history analysis is mandatory.

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