EFFECT OF VARIATION OF THE STAY PRESTRESSING FORCES ON THE SEISMIC RESPONSE OF CABLE-STAYED BRIDGES

G.E. Valdebenito[1,2] and A. C. Aparicio[3]

[1] *Professor, Facultad de Ciencias de la Ingeniería, Universidad Austral de Chile, Valdivia, Chile*
Email: gvaldebe@uach.cl
[2] *PhD Candidate, Dept. of Geotechnical Engineering and Geosciences, Technical Univ. of Catalonia, Barcelona, Spain*
Email: galo.valdebenito@upc.edu
[3] *Professor, Dept. of Construction Engineering, Technical Univ. of Catalonia, Barcelona, Spain*
Email: angel.carlos.aparicio@upc.edu

ABSTRACT :

Due to severe damage to bridges caused by the recent strong earthquakes around the world, very high ground motion is now required in the new bridge design specifications, in addition to the relative frequent earthquake motion specifications by which old structures were designed and constructed. For that reason, in order to investigate if variation of the stay prestressing forces are important regarding the seismic response of cable-stayed bridges, two concrete bridge models were analyzed, with a very similar main span length, but with differences in their stay cable layout, deck type, deck level and stay spacing. For the variation of the cable forces, two load conditions were considered: an original load condition where the stay prestressing forces were the obtained from the static analysis under service loads, and an optimal load condition, where the stay prestressing forces were modified to an optimal value minimizing the structural displacements. Results of this investigation show that the seismic response of the bridges is not very sensitive with regard to the influence of low-to-moderate variations of the stay prestressing forces. The main differences come from the maximum vertical deflections and axial forces of the decks, as long as differences with regard to the seismic response of the towers are less sensitive.

KEYWORDS: Cable-stayed bridge, seismic response, stay prestressing forces, response spectrum method

1. INTRODUCTION

Bridges are very vulnerable structures, and essential for transportation systems, consequently the understanding of their seismic behaviour is fundamental. Cable-stayed bridges, due to their large dimensions and flexibility, usually experience very long fundamental periods, which is an aspect that differentiates them from other structures, and of course, that affects their dynamic behaviour. However, the flexibility and dynamic characteristics of that kind of bridges depend on several parameters such as the main span length, stay system and their layout, support conditions and many other things. This structural typology is complex, consisting on several structural components with different individual stiffness and damping properties. They experience more flexibility than normal girder bridges, and consequently, they need a detailed dynamic analysis for their seismic design.

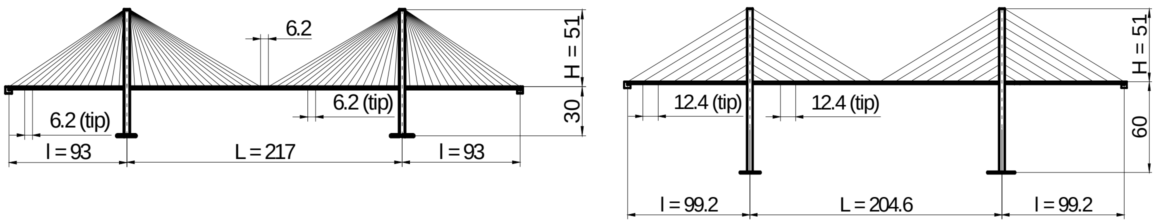
The effect of cable vibrations on cable-stayed bridges has been studied mainly with the aim to understand and control the effect of the rain and wind on those structures, and of course, with an aerodynamic point of view. However, the effect of cable vibrations on the seismic response of cable-stayed bridges sometimes can be important, reason why some researchers began to study this effect since the early 90´s, with the works of Abdel-Ghaffar (1991) and Abdel-Ghaffar and Khalifa (1991). They suggested a formulation for the cable modelling using a multi-element cable discretization, with the mass distribution along the cable modelled and associated with extra degrees-of-freedom to take into account the spatial vibration of the cables, and the interaction with the whole structure. Similar investigations were conducted focusing on the cable modelling and the effects of the cable vibrations on the seismic response of cable-stayed bridges [Tuladhar *et al* (1995); Caetano *et al* (1996, 2000a, 2000b); Macdonald and Georgakis (2002) and Cheng and Lau (2002)]. In this sense, it seems to be that an appropriate modelling of cables can increase or reduce the global response of a cable-stayed bridge, depending on the excitation frequency. Likewise, incorporation of additional damping on cables can attenuate the global response under certain circumstances, where the consideration of support flexibility in the analysis of transversally loaded cables can have a significant effect on the results [Förars *et al*, 2000]. Those aspects are necessary to be taken into account, especially when cable-deck interaction is important.

However, incidence of the variation of the stay prestressing forces on the seismic response of cable-stayed bridges has not been specifically investigated before, aspect that is enlarged in this investigation. Variations of the cable forces can come from slightly variations of the support conditions, static loading or geometric conditions of the bridge, being necessary to study about how much these cable force variations affect the seismic response.

2. GEOMETRY AND STRUCTURAL MODELLING

The seismic analysis of the structures, in order to study the effects of the static variations of the stay prestressing forces, starts with the definition of the bridges and their geometry, materials, loads, structural modelling and some additional considerations.

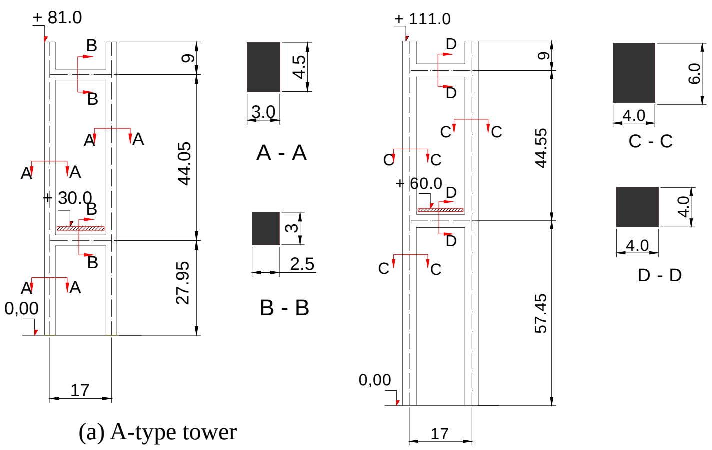
Two theoretical symmetric concrete bridge models were considered in this analysis: Bridge *AB1* is a 217 m – main span length cable-stayed bridge, with two H-type concrete tower (A-type), double-plane fan pattern cable layout, 30 m deck level, slab-type deck and 6.2 m stay spacing. Bridge *AR4* is a 204.6 m – main span length cable-stayed bridge, with two H-type concrete towers (B-type), double-plane harp pattern cable layout, 60 m deck level, hollow-box type deck and 12.4 m stay spacing [Figs. 1(a), 1(b), 2(a) and 2(b)]. For both bridges, the same area of the cross-section of the decks is considered, which implies the same weight.



(a) Bridge *AB1*

(b) Bridge *AR4*

Fig. 1 Longitudinal definition of the bridges



(a) A-type tower

(b) B-type tower

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

The chosen bridges, dimensions and structural specifications were taken from the specialized literature, and specifically, from Walter's Bridges [Walter, 1999] including the recommendations of Aparicio and Casas (2000) and Priestley *et al* (1996). In this sense, dimensions and some special considerations for the selected bridge typologies take into account an elastic seismic behaviour of the materials. In fact, cable-stayed bridges experience very long periods, and due to the high compressive forces of the pylons, ductility of them can be questionable. Likewise, because of the importance of such structures, it is preferable an elastic behaviour of the materials.

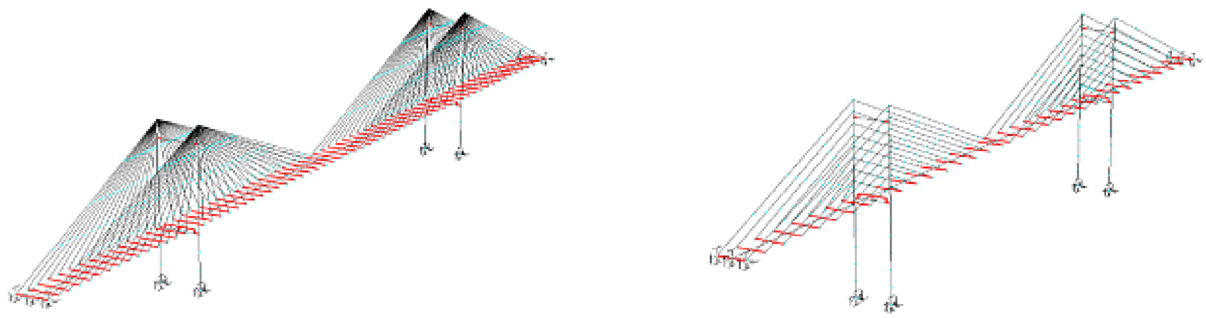
Materials and their mechanical properties have been chosen according to the general specifications and regulations for bridge design, taking into account seismic considerations [Priestley *et al*, 1996; Walter, 1999; Ministerio de Fomento, 2000; Aparicio and Casas, 2000]. Material data can be summarized in Table 1.

Table 1 Material data

MATERIAL	Charact. Strength (f_{ck})	Elastic limit (f_y)	Modulus of elasticity (E)	Poisson`s ratio (ν)	Volumetric weight (γ)	Thermal exp. Coeff (α)	Ult. tensile strength (f_u)
Concrete	40 MPa		36000 MPa	0.20	25 KN/m^3	1.43x10^{-5} (1/ºC)	
Steel (reinf. concrete)		400 MPa	2.1x10^5 MPa	0.30	78.5 KN/m^3	1.1x10^{-5} (1/ºC)	
Steel (cables)			1.9x10^5 MPa	0.30	78.5 KN/m^3	1.1x10^{-5} (1/ºC)	1900 MPa

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 for both bridges. In fact, the use of beam elements can be more useful to assess 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. Figs. 3(a) and 3(b) shows the complete *3D* structural modelling.



(a) *AB1* pattern

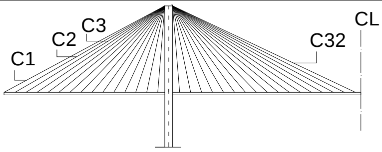
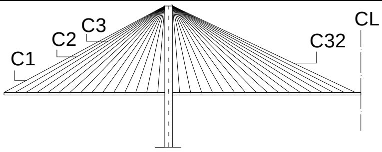
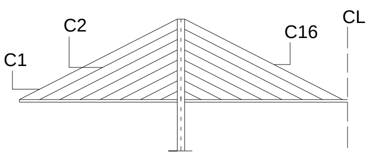
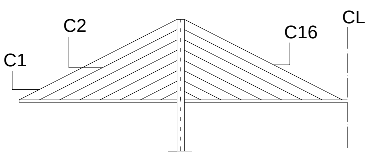
(b) *AR4* pattern

Fig. 3 Complete *3D* structural modelling of the bridges

3. ANALYSIS AND SEISMIC RESPONSE COMPARISON

For the static variation of the stay prestressing forces, two extreme conditions were considered: an *original load condition*, where the cable forces were the obtained from the static analysis under service loads; and the second condition, so-called *optimal load condition,* where a rectification of the back stay forces was introduced to minimize values of longitudinal displacements of the tower-top, and the vertical displacements of the deck at the mid-span. This correction was carried out applying an iterative procedure in which the cable forces were gradually increased, controlling the displacements of the structures. As a result, an increase of 20% for the back stay forces of the cables *C1, C2* and *C3* (bridge *AB1*) and an increase of 12% for all the back stay forces of the bridge *AR4* were obtained. Table 2 summarizes the previous idea.

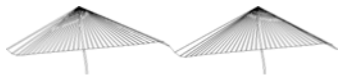
Table 2 Prestressing forces of the stays

Bridge	Stay Prestressing Forces [KN].								
 **(a)** *AB1*- Original Load Condition	**C1**	**C2**	**C3**	**C4**	**C5**	**C6**	**C7**	**C8**	**C9**
	4635	1460	1330	1220	1160	1100	1000	960	900
	C10	**C11**	**C12**	**C13**	**C14**	**C15**	**C16**	**C17**	**C18**
	850	800	760	710	780	490	490	780	710
	C19	**C20**	**C21**	**C22**	**C23**	**C24**	**C25**	**C26**	**C27**
	760	800	850	900	960	1020	1090	1160	1240
	C28	**C29**	**C30**	**C31**	**C32**				
	1300	1400	1470	1600	1600				
 **(b)** *AB1* – Optimal Load Condition	**C1**	**C2**	**C3**	**C4**	**C5**	**C6**	**C7**	**C8**	**C9**
	5560	1750	1600	1220	1160	1100	1000	960	900
	C10	**C11**	**C12**	**C13**	**C14**	**C15**	**C16**	**C17**	**C18**
	850	800	760	710	780	490	490	780	710
	C19	**C20**	**C21**	**C22**	**C23**	**C24**	**C25**	**C26**	**C27**
	760	800	850	900	960	1020	1090	1160	1240
	C28	**C29**	**C30**	**C31**	**C32**				
	1300	1400	1470	1600	1600				
 **(c)** *AR4* – Original Load Condition	**C1**	**C2**	**C3**	**C4**	**C5**	**C6**	**C7**	**C8**	**C9**
	2570	1970	2790	2930	2850	2630	2000	760	890
	C10	**C11**	**C12**	**C13**	**C14**	**C15**	**C16**		
	2000	2580	2760	2800	2700	2600	2300		
 **(d)** *AR4* – Optimal Load Condition	**C1**	**C2**	**C3**	**C4**	**C5**	**C6**	**C7**	**C8**	**C9**
	3080	2200	3120	3280	3190	2950	2240	850	890
	C10	**C11**	**C12**	**C13**	**C14**	**C15**	**C16**		
	2000	2580	2760	2800	2700	2600	2300		

The input ground motion was characterized by use of the response spectrum method defined by Eurocode 8 – Part 2 [CEN, 1998a]. Although this procedure, on the basis of a performed modal analysis, can be questionable because of the supposed linearity involved in this strategy, in this case, with the aim to compare results of the static and dynamic structural analysis and to obtain maximum values for the seismic response of the structures, it is adequate to employ this method. In fact, conclusions taken from a time history analysis can be difficult to obtain, and strongly depend on the considered earthquake database, being confused in the present analysis. Likewise, because of the elastic response of those structures, here this strategy is preferable. The 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 (A-type soil), and the considered maximum effective ground acceleration is $0.5g$ for the horizontal component, and $0.35g$ for the vertical component, because these values are representative of strong ground motion for structures located in high seismicity areas founded on bedrock. Because of the modal shapes of the bridges and in accordance with the damping values obtained from the dynamic analysis [Kawashima and Unjoh, 1991], a critical damping ratio of 1.7% was selected. Likewise, the η-damping correction factor, for values different from the classical 5%-critical damping ratio, was applied in this case.

The models were analyzed for each load condition, in which the seismic action was combined with the permanent loading applying the *30% rule* according to Eurocode 8, that is to say, to add the earthquake input, 100% of one component was added to 30% of the other components of the seismic action and considering all possibilities. The static and dynamic analyses were carried out applying the code RAM advance [RAM International, 2003] considering 30 modes for each bridge model, which implies over 85% of the effective translational mass. With regard to the combination rule, due to the strong modal coupling that cable-stayed bridges experience, *CQC* modal combination rule was applied. Table 3 summarizes the dynamic characterization of the bridges.

Table 3 Summary of the main dynamic properties of the bridges

BRIDGE	Predominant period (sec)	Nature of modal shape	Modal participation (%)	Damping (%)
AB1	1.66		63	1.73
AR4	2.53		75	1.67

A brief summary of the maximum seismic response of the bridges for the original and optimal load conditions are shown in Tables 4 and 5. M_{max1} corresponds to the maximum bending moments of the deck at the mid-span; M_{max2} are the maximum bending moments at the deck-ends; M_{max3} are the maximum bending moments of the tower legs in the longitudinal direction (that occurs at the base level); N_{max1} corresponds to the maximum compressive force of the deck (that occurs at the tower-deck level) and N_{max2} is the maximum compressive force of the tower legs (that occurs at the base level). In the same way, Δ_1 corresponds to the vertical displacement of the deck at the mid-span; Δ_2 is the longitudinal displacement of the deck at the mid-span; Δ_3 is the transverse displacement of the deck at the mid-span; Δ_4 corresponds to the longitudinal displacement at the deck-ends; Δ_5 is the longitudinal displacement of the tower-top and Δ_6 is the transverse displacement of the tower-top. Bending moments are shown in MN.m; forces in MN and displacements in cm.

Table 4 Main values of the maximum seismic response for bridge *AB1*

Original Load Condition	M_{max1}	M_{max2}	M_{max3}	N_{max1}	N_{max2}	**Base Shear**
	10.7	10.5	336	37.1	93.4	57.6
	Δ_1	Δ_2	Δ_3	Δ_4	Δ_5	Δ_6
	19.3	15.5	51.2	17.5	31.4	28.9
Optimal Load Condition	M_{max1}	M_{max2}	M_{max3}	N_{max1}	N_{max2}	**Base Shear**
	10.8	10	337.6	37.4	93.5	57.6
	Δ_1	Δ_2	Δ_3	Δ_4	Δ_5	Δ_6
	15.8	15.5	51.2	17.5	29.7	28.9

Table 5 Main values of the maximum seismic response for bridge *AR4*

Original Load Condition	M_{max1}	M_{max2}	M_{max3}	N_{max1}	N_{max2}	**Base Shear**
	44.8	43	490	42.4	158.2	54
	Δ_1	Δ_2	Δ_3	Δ_4	Δ_5	Δ_6
	5.7	25.1	61.1	27.1	44.0	32.6
Optimal Load Condition	M_{max1}	M_{max2}	M_{max3}	N_{max1}	N_{max2}	**Base Shear**
	45.6	40.3	496	43	158.2	54
	Δ_1	Δ_2	Δ_3	Δ_4	Δ_5	Δ_6
	3.5	25.1	61.1	27.1	42.7	32.6

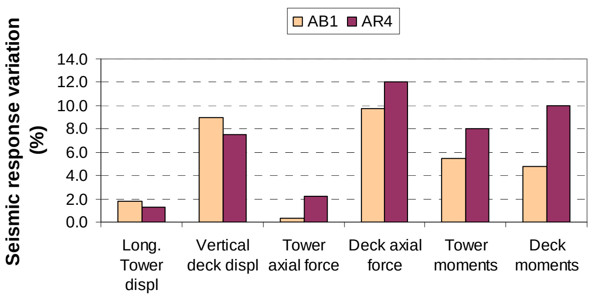


Fig. 4 Average seismic response variation

It is easy to see that there are not significant differences in the seismic response comparing the original load condition and the optimal load condition for the selected points of the structures. A comparison of the average seismic response variation shows some differences, as can be seen in Fig. 4. In this sense, variations of the maximum bending moments on bridge *AB1* are in the order of 5.5% for the towers, and 4.8% for the deck. A similar condition can be found for bridge *AR4*, with main differences for bending moments in the order of 8% (towers), and 10% (deck).

Average differences for axial forces in the order of 0.3% (tower legs of bridge *AB1*); 9.7% (deck of bridge *AB1*); 2.2% (tower legs of bridge *AR4*) and 12% (deck of bridge *AR4*) can be found. For the displacements, average differences in the order of 1.8% (tower legs of bridge *AB1*); 9% (deck of bridge *AB1*); 1.3% (tower legs of bridge *AR4*) and 7.5% (deck of bridge *AR4*) are obtained. A comparison for displacements considering both load conditions can be observed in Figs. 5 to 8.

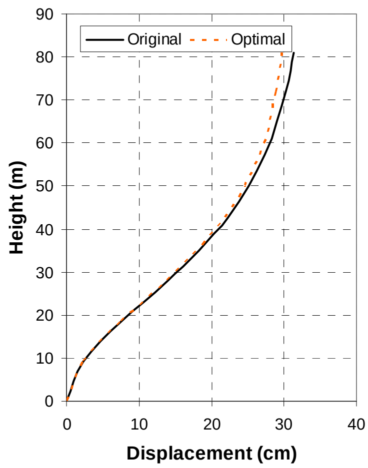


Fig. 5 Longitudinal displacements of the tower – Bridge *AB1*

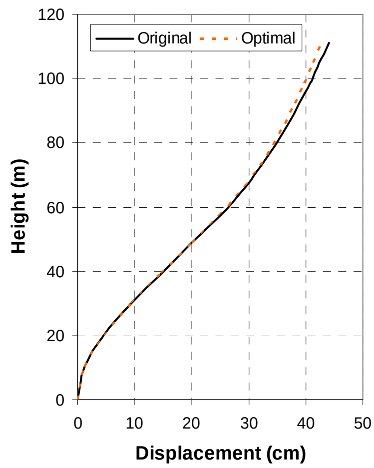


Fig. 6 Longitudinal displacements of the tower – Bridge *AR4*

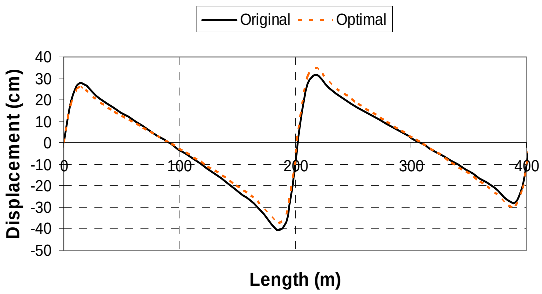


Fig. 7 Vertical displacements of the deck – Bridge *AB1*

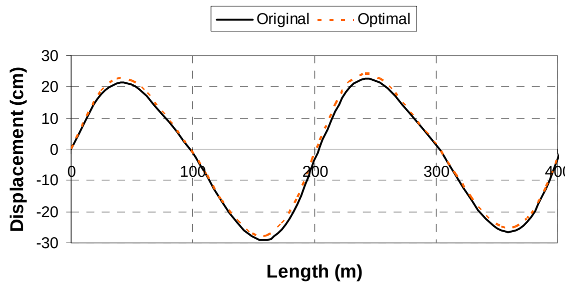


Fig. 8 Vertical displacements of the deck – Bridge *AR4*

Regarding the displacements of the bridges, it can be confirmed that longitudinal displacements of the towers are similar for both conditions, with the maximum at the top for the original load condition. [Figs. 5 and 6]. Of course, maximum longitudinal displacements for the towers occur for the tallest bridge (bridge *AR4*). With regard to the vertical displacements of the decks, the deformed shape is quite different if we compare bridge *AB1* with bridge *AR4*. For bridge *AB1*, maximum vertical deflection occurs near the mid span, as long as for bridge *AR4* maximum vertical deflection occurs of about ¾ of the mid span [Figs. 7 and 8]. These differences come from the incidence of the stay cable layout and from differences in the stay spacing.

Considering the internal forces, plots for axial forces and bending moments were obtained from the dynamic analysis considering the original and optimal load conditions. Figs. 9 to 12 show a comparison for the compressive forces of the towers and axial forces of the decks. Because of the differences in the variation of the stay prestressing forces for bridges *AB1* and *AR4*, it is not possible to compare differences for displacements and internal forces between those structures. However, it is easy to see that for both bridges, differences regarding the longitudinal displacements and axial forces of the towers are negligible [Figs. 5, 6, 9 and 10], and more important differences can be found for vertical displacements and axial forces of the decks [Figs. 7, 8, 11 and 12]. For both bridges and for both load conditions, maximum axial forces on the decks (compression) occur in the vicinity of the tower-deck connection, with very high values, as usually happens on cable-stayed bridges with fixed hinge connection between the deck and the tower.

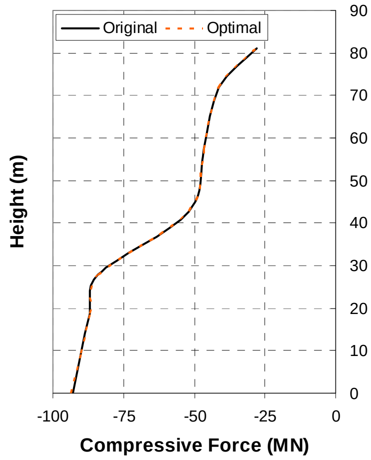


Fig. 9 Compressive forces of the tower – Bridge *AB1*

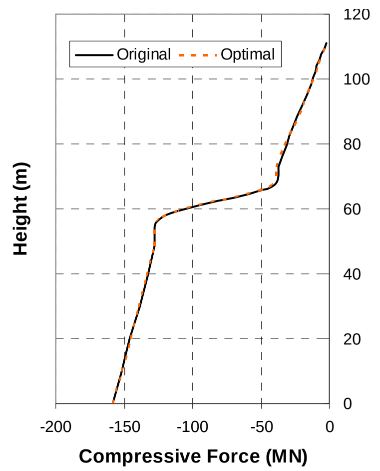


Fig. 10 Compressive forces of the tower – Bridge *AR4*

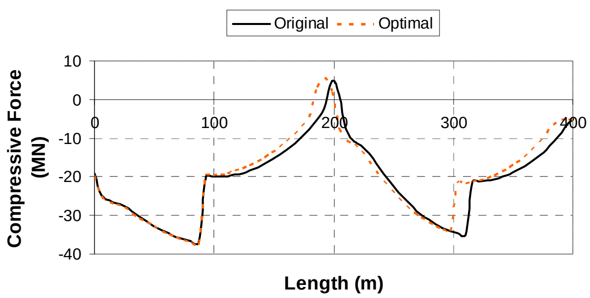


Fig. 11 Axial forces of the deck – Bridge *AB1*

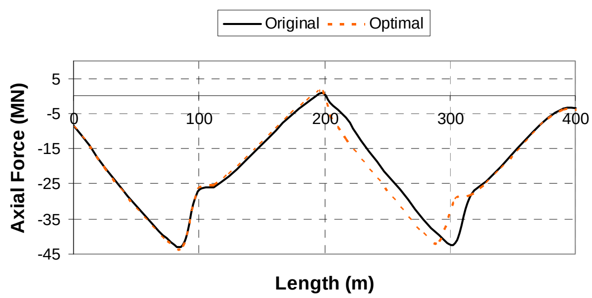
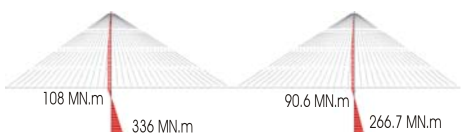
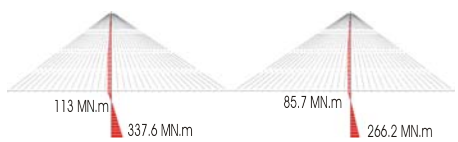


Fig. 12 Axial forces of the deck – Bridge *AR4*

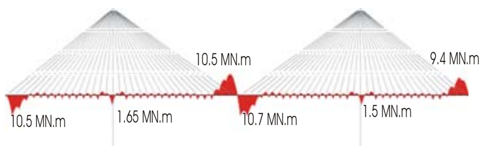
For bending moments, because of the complexity of such plots, it is preferable to show them separately, that is to say, for each load condition and for each bridge, as can be seen in Figs. 13 and 14. Differences for the maximum values of the tower moments are negligible for both bridges [Figs. 13(a) and 13(b); Figs. 14(a) and 14(b)].



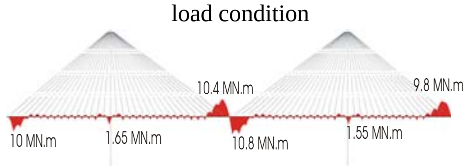
(a) Longitudinal bending moments of the towers – original load condition



(b) Longitudinal bending moments of the towers – optimal load condition



(c) Deck bending moments – original load cond.

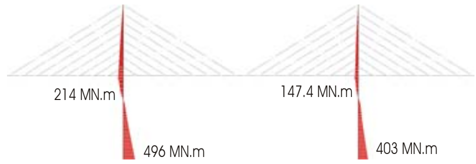


(d) Deck bending moments – optimal load cond.

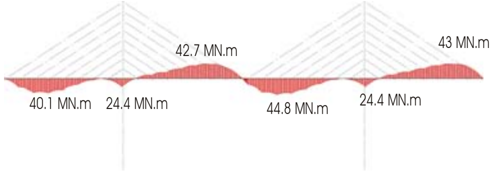
Fig. 13 Envelope of seismic bending moments on bridge *AB1*

The shape of the plot for the deck bending moments is very different between bridge *AB1* and *AR4*. In both situations, maximum values occur near the mid span or near the deck-ends, with very high values for the bridge *AR4* [Figs. 13(c) and 13(d); Figs. 14(c) and 14(d)]. These differences mainly come from differences in the stay spacing. Likewise, not very important differences are obtained if we compare the maximum bending moments for both load conditions.

(a) Tower bending moments – original load cond.



(b) Tower bending moments – optimal load cond.



(c) Deck bending moments – original load cond.

(d) Deck bending moments – optimal load cond.

Fig. 14 Envelope of seismic bending moments on bridge *AR4*

4. CONCLUSIONS

The comparative analysis of the seismic response of two cable-stayed bridge models applying the response spectrum method was carried out with the aim of study the influence of low-to-moderate variations of the stay prestressing forces. The main conclusions are:

1. For bridge model *AB1,* maximum differences of the measured displacements were obtained for the vertical deflections of the deck. Maximum differences of the measured internal forces were obtained for the axial forces of the deck, followed by maximum bending moments of deck and towers. Regarding the bridge *AR4*, maximum differences of the measured displacements were obtained for the vertical deflections of the deck. Maximum differences of the measured internal forces were obtained for the axial forces of the deck followed close by maximum bending moments of deck and towers.

2. As a general conclusion, this study shows that influence of low-to-moderate variations of the stay prestressing forces on cable-stayed bridges, are not very important regarding their seismic response. Likewise, variations in the seismic response when the back stay forces changes, are not very different if the stay cable layout, stay spacing or deck level is changed, and only specific differences regarding the shape of the internal forces or displacements can be found, and specially for the deck. The main differences come from the vertical deflections and internal forces of the deck, as long as differences for the seismic response of the towers are less sensitive, especially the longitudinal displacements and axial forces. These conclusions can be very useful, mainly if small variations in the bridge configuration inducing small variations of the cable forces occur.

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