Evaluación de la vulnerabilidad sísmica de puentes extradados durante su construcción por voladizos sucesivos

Seismic vulnerability assessment of extradosed bridges during cantilever construction

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Resumen

El trabajo presenta el estudio de la vulnerabilidad sísmica de los puentes extradados durante construcción por voladizos sucesivos. Para esto fueron desarrollados modelos de elementos finitos de un puente extradado de tres luces (60 + 100 + 60 m), a partir de los cuales se determinó la respuesta sísmica durante seis etapas constructivas, y dos en estado de servicio, antes y después de que ocurran los efectos de fluencia y contracción del concreto. Siguiendo el enfoque propuesto en Wilson y Holmes (2007), se comparó la respuesta sísmica durante construcción y en servicio para determinar los elementos estructurales con mayor vulnerabilidad sísmica, además de detectar las etapas constructivas críticas. Los resultados encontrados permiten concluir que la vulnerabilidad sísmica del puente extradado durante construcción es mayor que la del estado de servicio, ya que durante construcción se tienen probabilidades de excedencias anuales mucho mayores al 0.21% que se adopta para el diseño sísmico del puente. Además, se encontró que los cables extradados y las torres son los elementos con mayor vulnerabilidad, y que las etapas previas al cierre del puente son las más críticas.

Palabras Clave: Puentes extradados, vulnerabilidad sísmica, construcción, voladizos sucesivos, probabilidad de excedencia anual

Abstract

This paper presents the study of seismic vulnerability of extradosed bridges during cantilever construction. The seismic response during six construction phases and two in service, before and after creep and shrinkage of concrete takes place, of a 3-span extradosed bridge (60 + 100 + 60 m) were determined by means of finite elements models. Using the approach proposed by Wilson and Holmes (2007), seismic response during construction and in service were compared to determine structural elements with higher vulnerability and to detect critical stages during construction. The results show that the seismic vulnerability of the bridge during construction is higher than in service, this is supported on the fact that during construction annual exceedance probabilities are higher than 0.21%, a commonly adopted value for the seismic design of bridges. Furthermore, it was found that extradosed cables and the towers are the most vulnerable elements and that the stages before the closure of the bridge during construction are the most critical.

Keywords: Extradosed bridges, seismic vulnerability, construction, cantilever, annual exceedance probability

1. Introduction

The construction of extradosed bridges is commonly executed by employing the balanced cantilever method. This method provides a high degree of vulnerability on bridges facing adverse conditions, like wind, form traveler or segments collapse, or cantilever rollover. Such actions are considered and reviewed during the bridge design phase. However, seismic events that might take place during construction are not taken into consideration and, the bridge seismic design is generally planned for the whole structure. This fact is supported by the low design seismic exceedance probability during the construction phase. For example by using the design philosophy in the Colombian Code of Bridges Seismic Design (AIS, 1995), which proposes a seismic design with probability of exceedance (P) of 10%, for a period (t) of 50 years, equivalent to an event with annual exceedance probability (p) of 0.21%, there would be an exceedance probability of 0.32% resulting from Equation 1, for the same event but in a construction period assumed in 1.5 years.

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If instead of using the total construction period, the maximum execution period of each construction stage is used, as suggested by Calvi, Sullivan and Villani (2010), the probability will be lower.

$$P(t)=1-(1-p)^t \quad (1)$$

However, seismic events of lower magnitude than the design earthquake taking place during construction are likely to provoke considerable forces on structural elements due to the low redundancy of structural system and because of the great mass percentage of the bridge. This fact was studied by Wilson and Holmes (2007), who determined from the seismic response at the columns base of a cable-stayed bridge, that the forces obtained by the whole bridge for the seismic design with $p=0.21\%$ are matched during construction by lower magnitude seismic events that have higher annual exceedance probabilities almost reaching 20%, thus exposing a high seismic vulnerability degree for such typology.

By using the same approach presented by Wilson and Holmes, the current study assesses seismic vulnerability of extradosed bridges during construction stage, typology that shares morphological similarities with cable-stayed bridges. However, such typology design and behavior are different in regards to the higher stiffness of the deck. To this effect, the seismic response of an extradosed bridge with 100 meters mid-span, numerically modeled by means of finite elements, is analyzed. The columns height has been included as a variable. The study describes the research characteristics and shows the results not only from columns, but also from other main elements such as deck, towers and cables, so that a wider idea can be obtained about the bridge vulnerability. At the end, the main conclusions drawn from this study will be presented.

2. Characteristics of the study

2.1 Bridge Description

The main geometrical characteristics of the bridge were defined from the design criterion presented by Benjumea, et al., (2010) and also considering the current trends for extradosed bridges presented by Benjumea et. al. (Benjumea, et al. (2012). The bridge has a mid-span of 100 m and two side spans of 60 m. The towers are solids elements of 2.5 x 1.5 m, of 10m height, see Figure 1. The deck has a constant depth of 2.5 m (slenderness equal to $L/40$) and corresponds to a single-cell- girder box type with cantilevers, with variant lower slab thickness; see Figure 2 and Table 1. The first extradosed cable is anchored to the deck at a distance of 21.5 m in regards to the tower axis; the remaining six cables are placed every 5.5 m, matching the dowels ends with the cables anchorage nodes. Such elements are made up of 12 tendons of $\varnothing 0.6"$, which are pre-tensioned at an average of 0.42fpu. The columns correspond to a hollow rectangular section, which is presented by Figure 3.

Their height was modified and lengths of 25, 37.5 and 50m are under study. It is assumed that the bridge is supported by end rollers, although transverse direction movements have been restricted. It is assumed that the columns are embedded in a foundation and are rigidly connected to the deck.

The steel properties of extradosed cables are: f_{pu} of 1860 MPa, E_{ps} of 1.999×10^5 MPa, and γ_{ps} equal to 77.14 kN/m³. Fatigue control, under servicing limit condition and allowable stress under ultimate limit condition, was done in accordance with regulations established by SETRA (2001). For the structural analysis the stress relaxation of such elements was not considered. The deck, tower and columns concrete's properties are: f'_c equal to 39.2 MPa, E_c equal to 2.55×10^4 MPa, and γ_c equal to 23.5 kN/m³. For the analysis performed during construction stage, concrete creep and shrinkage effects were taken into consideration, following the specifications by CEB-FIB Model Code 1990 (CEB-FIP, 1993). Ageing effects on the concrete resistance and elasticity module were not included by the analysis.

Figura 1. Vista longitudinal del puente estudiado
Figure 1. longitudinal view of the bridge under study

Figura 2. Sección típica del tablero (izquierda) y sección en apoyo sobre pilares (derecha)
Figure 2. typical deck section (left) and section supported by columns (right)

Tabla 1. Propiedades geométricas de las dovelas del puente**Table 1.** Geometrical properties of deck's dowels

Dovela/Dowel	t^* (m)	A (m ²)	J (m ³)	I _{xx} (m ³)	I _{yy} (m ³)
D _{Ap} (en apoyo sobre pilares/ Dap supported by columns)	0.45	8.33	17.72	8.49	99.31
D1	0.44	8.26	17.69	8.44	99.04
D2	0.41	8.05	17.56	8.28	98.22
D3	0.39	7.92	17.45	8.15	97.69
D4	0.36	7.71	17.26	7.95	96.88
D5	0.34	7.56	17.1	7.81	96.36
D6	0.31	7.37	16.84	7.57	96.57
D7	0.29	7.24	16.62	7.4	95.05
D8	0.26	7.04	16.24	7.11	94.27
D9	0.24	6.9	15.93	6.9	93.76
D10	0.23	6.84	15.76	6.8	93.5
D _{Cl} (cierre en centro de luz/ closure of mid-span)	0.22	6.77	15.58	6.68	93.24

Figura 3. Sección de los pilares**Figure 3.** Columns section

2.2 Numerical Analysis

The analysis was performed by using the SAP2000 software. For this purpose, the employed elements were beam-column on the deck, towers and columns and; cable elements which consider the catenary effect of extradosed cables. Force transmission from cables to the deck and connection between deck and columns, was achieved by means of the use of link-type elements, which are rigid elements and their mass is almost void (CSI, 2010), see Figure 4.

The first step for the study of seismic vulnerability during construction stage consisted of developing a constructive and post-constructive analysis (between $t=0$ and $t= 50$ years after closure), including geometrical non-linearity and differed effects throughout time. Considered actions and construction periods, as well as the results obtained from such analysis, can be reviewed on Benjumea (2012).

For the brand new bridge ($t=0$), and then when losses have taken place ($t=50$), a spectral modal analysis is conducted to determinate seismic forces. The same procedure is executed on the analyzed constructive stages, presented by Figure 5, which solution for the eigenvalues problems is performed by using the bridge stiffness matrix under deformed condition for each constructive stage. So as to calculate the bridge seismic vulnerability, the internal response was studied for some deck data points (the anchorage area of the first extradosed cable on the side span and mid-span and the zone connecting with columns), as well as the columns and towers bases and three cables on the mid-span, see Figure 6. It is important to emphasize that this research study has not analyzed the problems of bridge structural elements capacities, because calculations are based on relative comparisons of the bridge structural response and not on absolute ones.

Figura 4. Modelos de elementos finitos desarrollados

Figure 4. Developed Finite Elements Modelling

<p>EC1</p> <p>Pilares + Dovela 0 + Torres Columns + Dowel 0 + Towers</p>	<p>EC2</p> <p>Última dovola sin atirantar Last dower non cable-stayed</p>	<p>EC3</p> <p>Primera dovola atirantada First cable-stayed dower</p>
<p>EC4</p> <p>Tercera dovola atirantada Third cable-stayed dower</p>	<p>EC5</p> <p>Última dovola Last cable-stayed dower</p>	<p>EC6</p> <p>Mitad del puente sin dovola de cierre Mid-bridge without closure dower</p>
<p>ES₀ y ES₅₀</p> <p>Puente completo ($t=0$ años) y ($t=50$ años) ES₀ and ES₅₀ the whole bridge ($t=0$) and ($t=50$ years)</p>		

Figura 5. Estados del puente considerados

Figure 5. Bridge conditions taken in to consideration

2.3 Seismic activity

In order to determine seismic activity during construction stage and during service, the acceleration recurrence curve from the city of Bucaramanga, located at the northeast area of Colombia, was employed, located at the northeast area of Colombia, see Figure 7. From this curve it is observed that, for a return period of 475 years, the corresponding horizontal acceleration is 0.22g approximately. This value was used to create the acceleration spectrum, in accordance with the Bridge Seismic Design of the Colombian Code, which is presented by Figure 8.

Figura 6. Zonas de interés y fuerzas internas estudiadas

Figure 6. Interest zones and internal forces under study

Figura 7. Curva de recurrencia de aceleraciones para la ciudad de Bucaramanga (AIS et al., 1996)

Figure 7. Accelerations recurrence curve for the city of Bucaramanga (AIS et al., 1996)

Figura 8. Espectro de aceleración para la ciudad de Bucaramanga

Figure 8. Acceleration spectrum for the city of Bucaramanga

In order to calculate the seismic forces, a 1.5% damping ratio has been assumed for all modes of the bridge in service and during construction. This value was provided by Niihara et. al., (2001), which is the result from environmental vibration studies in three extradosed bridges, built in Japan. The number of modes included in the modal analysis was selected in such a way that the following premises are guaranteed: (a) that the percentage of modal mass, in vertical direction, shall be higher or equal to the ratio of deck mass in regards to the structure, looking for a high participation of the deck in the vertical direction response; (b) that modal mass percentage shall be higher or equal to the 80% of horizontal direction responses and, (c) that the percentage of modal mass, for three main directions, shall remain relatively constant for all conditions of the analyzed bridge, including those with the bridge under service. In all such cases, the modes corresponding to vertical movements of columns and the additional mass of the bridge form traveler have been taken into account, during the constructive stages.

3. Seismic response during construction stage

After determining the bridge seismic response under the design earthquake, during the construction and in-service stages, they are compared by using Equations 2 and 3 derived by Wilson and Holmes (2007). In such equations the term E_c indicates the maximum seismic response for the analyzed constructive stage. While the terms $E_{s,0}$ and $E_{s,50}$ indicate the value of seismic response analyzed for the whole bridge in $t=0$ and $t=50$ years, respectively. Since the seismic activity considered for constructive and in-service stages is the same, the terms $\psi_{E,0}$ and ψ_{E50} provide a good idea of the elements and the critical constructive stages, thus being identified as having higher values than one, which indicates that the structural seismic response during construction stage exceeds the response of the bridge in-service. The assessment of Equations 2 and 3 is presented by Tables from 2 to 4.

$$\psi_{E,0} = \frac{E_c}{E_{s,0}} \quad (2)$$

$$\psi_{E50} = \frac{E_c}{E_{s,50}} \quad (3)$$

Tabla 2. Valores de $\psi_{E,0}$ y $\psi_{E,50}$ para el puente con $H_p=25$ m**Table 2.** Values of $\psi_{E,0}$ and $\psi_{E,50}$ for the bridge with $H_p=25$ m

$\Psi_{E,0}$		EC1	EC2	EC3	EC4	EC5	EC6	
Pilares Columns	P	0.48	0.46	0.5	0.5	0.51	1.39	
	ML	0.58	0.69	0.71	0.78	1.05	0.96	
	VL	0.56	0.6	0.64	0.9	1.23	1.04	
	MT	0.49	0.87	1.11	1.43	1.2	1.18	
	VT	0.57	0.88	1.11	1.41	1.17	1.21	
Torres Towers	ML	1.29	0.86	0.92	1.81	2.8	1.94	
	VL	1.28	0.85	0.91	1.78	2.76	1.92	
Cables 6 3 1	P	-	-	1.27	1.84	1.36	2.32	
	P	-	-	-	1.66	1.62	2.45	
	P	-	-	-	-	1.33	1.9	
Tablero Deck	VL	P	-	-	0.09	0.55	1.25	1
		ML	-	-	0.01	0.35	0.99	0.99
		VL	-	-	0.28	0.92	0.74	1.2
	VC	P	-	-	0.09	0.56	1.28	1.38
		ML	-	-	0.01	0.36	1.01	1.2
		VL	-	-	0.58	1.84	1.49	1.78
	AP	P	0.2	0.41	0.48	0.84	1.35	1.28
		ML	0.1	0.97	1.54	2.21	1.84	2.35
		VL	0.29	1.05	1.37	1.31	0.79	0.93

$\Psi_{E,50}$		EC1	EC2	EC3	EC4	EC5	EC6	
Pilares Columns	P	0.48	0.46	0.5	0.51	0.51	1.39	
	ML	0.57	0.69	0.71	0.77	1.05	0.95	
	VL	0.55	0.6	0.64	0.89	1.22	1.04	
	MT	0.49	0.87	1.11	1.43	1.2	1.18	
	VT	0.57	0.88	1.11	1.41	1.17	1.21	
Torres Towers	ML	1.29	0.86	0.92	1.82	2.8	1.94	
	VL	1.28	0.85	0.91	1.79	2.77	1.92	
Cables 6 3 1	P	-	-	1.51	2.19	1.62	2.76	
	P	-	-	-	1.66	1.62	2.46	
	P	-	-	-	-	1.33	1.91	
Tablero Deck	VL	P	-	-	0.09	0.55	1.25	1
		ML	-	-	0.01	0.36	0.99	0.99
		VL	-	-	0.28	0.92	0.74	1.2
	VC	P	-	-	0.09	0.56	1.28	1.38
		ML	-	-	0.01	0.36	1.02	1.21
		VL	-	-	0.58	1.85	1.5	1.79
	AP	P	0.2	0.41	0.48	0.84	1.35	1.28
		ML	0.1	0.97	1.54	2.22	1.84	2.35
		VL	0.3	1.06	1.38	1.32	0.79	0.94

Tabla 3. Valores de $\psi_{E,0}$ y $\psi_{E,50}$ para el puente con $H_p=37.5$ m**Table 3.** Values of $\psi_{E,0}$ and $\psi_{E,50}$ for the bridge with $H_p=37.5$ m

$\Psi_{E,0}$		EC1	EC2	EC3	EC4	EC5	EC6	
Pilares <i>Columns</i>	<i>P</i>	0.82	0.79	0.74	0.79	0.77	1.58	
	<i>ML</i>	0.59	0.71	0.76	0.81	0.86	0.83	
	<i>VL</i>	0.56	0.61	0.64	0.7	0.9	0.81	
	<i>MT</i>	0.71	0.96	1.05	1.18	1.19	0.91	
	<i>VT</i>	0.79	1	1.07	1.18	1.16	0.89	
Torres <i>Towers</i>	<i>ML</i>	0.61	0.45	0.52	0.96	2.07	1.34	
	<i>VL</i>	0.6	0.45	0.51	0.94	2.04	1.32	
Cables 6 3 1	<i>P</i>	-	-	1.18	1.79	1.78	3.74	
	<i>P</i>	-	-	-	1.11	1.45	2.64	
	<i>P</i>	-	-	-	-	1.17	2.02	
Tablero <i>Deck</i>	VL	<i>P</i>	-	-	0.07	0.37	1.07	1.02
		<i>ML</i>	-	-	0.01	0.28	1.01	0.99
		<i>VL</i>	-	-	0.37	1.22	1.53	1.05
	VC	<i>P</i>	-	-	0.07	0.38	1.03	1.41
		<i>ML</i>	-	-	0.01	0.27	0.92	1.26
		<i>VL</i>	-	-	0.48	1.58	1.83	2.41
	AP	<i>P</i>	0.16	0.36	0.4	0.53	0.98	1.1
		<i>ML</i>	0.06	0.6	0.91	1.46	1.82	2.5
		<i>VL</i>	0.4	1.32	1.65	1.87	1.46	1.98

$\Psi_{E,50}$		EC1	EC2	EC3	EC4	EC5	EC6	
Pilares <i>Columns</i>	<i>P</i>	0.82	0.79	0.74	0.79	0.77	1.58	
	<i>ML</i>	0.59	0.71	0.76	0.81	0.86	0.83	
	<i>VL</i>	0.56	0.61	0.64	0.7	0.9	0.81	
	<i>MT</i>	0.71	0.96	1.05	1.18	1.19	0.91	
	<i>VT</i>	0.79	1	1.07	1.18	1.16	0.89	
Torres <i>Towers</i>	<i>ML</i>	0.61	0.46	0.52	0.97	2.08	1.34	
	<i>VL</i>	0.6	0.45	0.51	0.95	2.04	1.32	
Cables 6 3 1	<i>P</i>	-	-	1.18	1.79	1.79	3.74	
	<i>P</i>	-	-	-	1.11	1.45	2.65	
	<i>P</i>	-	-	-	-	1.18	2.02	
Tablero <i>Deck</i>	VL	<i>P</i>	-	-	0.07	0.37	1.07	1.02
		<i>ML</i>	-	-	0.01	0.28	1.01	0.99
		<i>VL</i>	-	-	0.37	1.22	1.53	1.05
	VC	<i>P</i>	-	-	0.07	0.39	1.03	1.41
		<i>ML</i>	-	-	0.01	0.27	0.93	1.27
		<i>VL</i>	-	-	0.48	1.57	1.83	2.4
	AP	<i>P</i>	0.16	0.36	0.4	0.53	0.98	1.1
		<i>ML</i>	0.06	0.6	0.91	1.45	1.81	2.49
		<i>VL</i>	0.4	1.32	1.65	1.88	1.46	1.98

Tabla 4. Valores de $\psi_{E,0}$ y $\psi_{E,50}$ para el puente con $H_p=50$ m

Table 4. Values of $\psi_{E,0}$ and $\psi_{E,50}$ for the bridge with $H_p=50$ m

$\Psi_{E,0}$		EC1	EC2	EC3	EC4	EC5	EC6	
Pilares <i>Columns</i>	<i>P</i>	0.92	0.87	0.86	0.83	0.78	1.32	
	<i>ML</i>	0.64	0.75	0.81	0.9	0.93	0.98	
	<i>VL</i>	0.63	0.66	0.69	0.73	0.78	0.85	
	<i>MT</i>	0.74	0.9	0.96	1.08	1.18	0.94	
	<i>VT</i>	0.84	0.95	0.98	1.09	1.14	0.92	
Torres <i>Towers</i>	<i>ML</i>	0.39	0.29	0.29	0.59	1.47	0.69	
	<i>VL</i>	0.38	0.29	0.29	0.58	1.44	0.69	
Cables 6 3 1	<i>P</i>	-	-	0.98	1.31	1.69	2.79	
	<i>P</i>	-	-	-	0.77	1.25	1.78	
	<i>P</i>	-	-	-	-	0.99	1.32	
Tablero <i>Deck</i>	VL	<i>P</i>	-	-	0.06	0.29	0.81	1.36
		<i>ML</i>	-	-	0.01	0.21	0.72	1.28
		<i>VL</i>	-	-	0.39	1.03	1.31	0.97
	VC	<i>P</i>	-	-	0.06	0.28	0.88	0.96
		<i>ML</i>	-	-	0.01	0.19	0.74	0.78
		<i>VL</i>	-	-	0.41	1.09	1.49	1.49
	AP	<i>P</i>	0.15	0.31	0.36	0.44	0.78	0.73
		<i>ML</i>	0.04	0.48	0.63	0.88	1.37	1.41
		<i>VL</i>	0.39	1.49	1.65	1.63	1.64	1.75

$\Psi_{E,50}$		EC1	EC2	EC3	EC4	EC5	EC6	
Pilares <i>Columns</i>	<i>P</i>	0.92	0.87	0.86	0.83	0.78	1.31	
	<i>ML</i>	0.64	0.75	0.81	0.9	0.93	0.98	
	<i>VL</i>	0.63	0.66	0.69	0.73	0.78	0.85	
	<i>MT</i>	0.74	0.9	0.96	1.08	1.18	0.94	
	<i>VT</i>	0.84	0.95	0.98	1.09	1.14	0.92	
Torres <i>Towers</i>	<i>ML</i>	0.39	0.29	0.29	0.59	1.47	0.69	
	<i>VL</i>	0.38	0.29	0.29	0.58	1.44	0.69	
Cables 6 3 1	<i>P</i>	-	-	0.99	1.32	1.69	2.79	
	<i>P</i>	-	-	-	0.77	1.25	1.79	
	<i>P</i>	-	-	-	-	0.99	1.32	
Tablero <i>Deck</i>	VL	<i>P</i>	-	-	0.06	0.29	0.81	1.36
		<i>ML</i>	-	-	0.01	0.21	0.72	1.28
		<i>VL</i>	-	-	0.39	1.03	1.31	0.97
	VC	<i>P</i>	-	-	0.06	0.28	0.88	0.96
		<i>ML</i>	-	-	0.01	0.19	0.74	0.78
		<i>VL</i>	-	-	0.41	1.08	1.49	1.49
	AP	<i>P</i>	0.15	0.31	0.36	0.44	0.78	0.73
		<i>ML</i>	0.04	0.48	0.62	0.87	1.36	1.4
		<i>VL</i>	0.4	1.5	1.67	1.65	1.67	1.77

From Tables 2 up to 4, it is observed that in cables and towers, the deck seismic response will always be exceeded at least during one stage of construction phase. Such elements have the highest $\psi_{E,0}$ and $\psi_{E,50}$ values (2.80 for the tower longitudinal bending moment and, 3.74 for the axial of the first extradosed cable). In columns, the whole bridge internal seismic forces, in transverse direction, will be exceeded at least during constructive stages EC4 and EC5. In longitudinal direction, higher seismic responses are also achieved during construction phase, although with a lower rate than in transverse direction. For both directions the highest $\psi_{E,0}$ and $\psi_{E,50}$ values were obtained for the bridge supported by short columns. As far as axial force is concerned, only during construction stage EC6, it exceeds the axial seismic force for the whole bridge. Deck maximum seismic responses during construction phase take place in the columns connection area ($\psi_{E,0}$ between 1.41 and 2.35 for bending moment; $\psi_{E,0}$ between 1.31 and 1.98 shear). In mid-span, maximum seismic responses occurs for longitudinal bending moment ($\psi_{E,0}$ between 1.21 and 1.27) and for longitudinal shear ($\psi_{E,0}$ between 1.49 and 2.41). In the side span, a maximum seismic response was achieved for the longitudinal shear ($\psi_{E,0}=1.53$), although the other analyzed responses are close to the response of the deck in-service condition.

The results from Tables 2 to 4 also show that $\psi_{E,0}$ and $\psi_{E,50}$ values are quite similar. This is so, because the structure modal response for these two bridge stages does not significantly changes, see Figure 9, which shows the relation between vibration periods under $E_{S,0}$ and $E_{S,50}$ condition, for the first 100 modes.

Figura 9. Relación entre los periodos de vibración para los estados ES_0 y ES_{50}

Figure 9. Relation between vibration periods for ES_0 and ES_{50} conditions

If Equations 2 and 3 are modified by introducing the effect of selfweight and dead load during construction stage (D_c) and in-service stage (D_s), see Equations 4 and 5, it is possible to obtain a clearest idea about the bridge vulnerability and to confirm the trend previously determined, since that load combination is far more real than the seismic activity working by itself.

$$\psi_{E+D,0} = \frac{E_c + D_c}{E_{s,0} + D_{s,0}} \quad (4)$$

$$\psi_{E+D,50} = \frac{E_c + D_c}{E_{s,50} + D_{s,50}} \quad (5)$$

The assessment of Equation 4 and 5 is presented by Tables 5 to 7. The following records are mentioned: (a) Towers and extradosed cables are structural elements showing higher vulnerability against seismic activity during construction stage. Since, these two elements are relevant for the extradosed bridge behavior special attention shall be paid during design phase; b) in columns seismic forces in transverse direction show the highest vulnerability during construction phase. Axial and forces in longitudinal direction achieved lower or equal values to those obtained for the bridge under full condition; (c) the area showing the highest vulnerability in the deck is the connection with columns. In this area the longitudinal moment reaches the most critical values, which tend to increase as long as the columns stiffness decreases, because the deck absorbs more seismic load. In mid-span and side spans, values may exceed the ones obtained for the bridge in full condition, however vulnerability is lower than the one shown by the area connecting with columns; (d) in general, the constructive stages EC4, EC5 and EC6 are the most vulnerable of the whole bridge. In these stages, the structure has a lower redundancy than that of the complete bridge and a considerable percentage of total mass.

Tabla 5. Valores de $\Psi_{E+D,0}$ y $\Psi_{E+D,50}$ para el puente con $H_p=25$ m.

Table 5. Values of $\Psi_{E+D,0}$ and $\Psi_{E+D,50}$ for the bridge with $H_p=25$ m.

$\Psi_{E+D,0}$		EC1	EC2	EC3	EC4	EC5	EC6	$\Psi_{E+D,50}$		EC1	EC2	EC3	EC4	EC5	EC6		
Pilares <i>Columns</i>	P	0.48	0.58	0.64	0.74	0.88	1.02	Pilares <i>Columns</i>	P	0.48	0.58	0.64	0.74	0.88	1.02		
	ML	0.5	0.6	0.62	0.68	0.93	0.85		ML	0.49	0.59	0.6	0.66	0.91	0.83		
	VL	0.5	0.54	0.58	0.8	1.1	0.94		VL	0.47	0.51	0.54	0.75	1.03	0.88		
	MT	0.49	0.87	1.11	1.43	1.2	1.18		MT	0.49	0.87	1.11	1.43	1.2	1.18		
	VT	0.57	0.88	1.11	1.41	1.17	1.21		VT	0.57	0.88	1.11	1.41	1.17	1.21		
Torres <i>Towers</i>	ML	1.12	0.75	0.81	1.59	2.44	1.74	Torres <i>Towers</i>	ML	0.92	0.62	0.67	1.31	2.01	1.44		
	VL	1.11	0.74	0.8	1.57	2.42	1.73		VL	0.92	0.61	0.66	1.3	1.99	1.43		
Cable 6 3 1	P	-	-	1.55	1.61	1.65	1.74	Cable 6 3 1	P	-	-	1.18	1.23	1.26	1.33		
	P	-	-	-	1.15	1.22	1.27		P	-	-	-	1.24	1.33	1.37		
	P	-	-	-	-	1.18	1.21		P	-	-	-	-	1.33	1.36		
Tablero <i>Deck</i> VC	VL	P	-	-	0.23	0.62	1.06	0.98	Tablero <i>Deck</i> VC	VL	P	-	-	0.25	0.67	1.15	1.06
	ML	-	-	0.11	0.41	0.73	0.83	ML		-	-	0.12	0.48	0.84	0.96		
	VL	-	-	0.37	0.82	0.54	1.04	VL		-	-	0.34	0.75	0.5	0.95		
	P	-	-	0.14	0.38	0.65	0.64	P		-	-	0.18	0.49	0.84	0.83		
	ML	-	-	0.12	0.46	0.78	0.98	ML		-	-	0.11	0.42	0.71	0.89		
	VL	-	-	0.47	0.99	0.87	0.98	VL		-	-	0.42	0.89	0.78	0.88		
AP	P	0.16	0.39	0.53	0.78	1.06	1.03	AP	P	0.2	0.51	0.69	1.02	1.38	1.34		
	ML	0.08	0.99	1.29	1.79	1.82	2.08		ML	0.07	0.88	1.14	1.58	1.61	1.84		
	VL	0.3	0.77	0.88	0.89	0.77	0.81		VL	0.29	0.75	0.85	0.87	0.75	0.78		

Tabla 6. Valores de $\Psi_{E+D,0}$ y $\Psi_{E+D,50}$ para el puente con $H_p=37.5$ m

Table 6. Values of $\Psi_{E+D,0}$ and $\Psi_{E+D,50}$ for the bridge with $H_p=37.5$ m

$\Psi_{E+D,0}$		EC1	EC2	EC3	EC4	EC5	EC6	
Pilares <i>Columns</i>	P	0.59	0.68	0.72	0.81	0.94	1.03	
	ML	0.62	0.75	0.8	0.87	0.92	0.9	
	VL	0.53	0.58	0.61	0.67	0.86	0.78	
	MT	0.71	0.96	1.05	1.18	1.19	0.91	
	VT	0.79	1	1.07	1.18	1.16	0.89	
Torres <i>Towers</i>	ML	0.58	0.43	0.5	0.94	1.99	1.28	
	VL	0.57	0.43	0.49	0.92	1.95	1.26	
Cable 6 3 1	P	-	-	1.07	1.12	1.23	1.43	
	P	-	-	-	1.05	1.19	1.33	
	P	-	-	-	-	1.14	1.25	
Tablero <i>Deck</i> VC	VL	P	-	-	0.22	0.58	1.02	0.98
		ML	-	-	0.1	0.37	0.76	0.85
		VL	-	-	0.46	0.94	1.18	0.87
	VC	P	-	-	0.16	0.41	0.71	0.74
		ML	-	-	0.11	0.39	0.74	1.05
		VL	-	-	0.41	0.84	1.08	1.33
		AP	P	0.15	0.4	0.54	0.74	1.02
ML	0.05		0.62	0.77	1.18	1.67	2.06	
VL	0.33		0.81	0.9	1	0.95	1.08	

$\Psi_{E+D,50}$		EC1	EC2	EC3	EC4	EC5	EC6	
Pilares <i>Columns</i>	P	0.59	0.68	0.72	0.81	0.94	1.03	
	ML	0.55	0.66	0.71	0.77	0.81	0.79	
	VL	0.52	0.57	0.6	0.65	0.84	0.76	
	MT	0.71	0.96	1.05	1.18	1.19	0.91	
	VT	0.79	1	1.07	1.18	1.16	0.89	
Torres <i>Towers</i>	ML	0.54	0.41	0.47	0.88	1.87	1.21	
	VL	0.54	0.4	0.46	0.87	1.84	1.19	
Cable 6 3 1	P	-	-	1.1	1.16	1.27	1.48	
	P	-	-	-	1.11	1.26	1.41	
	P	-	-	-	-	1.24	1.36	
Tablero <i>Deck</i> VC	VL	P	-	-	0.24	0.63	1.11	1.06
		ML	-	-	0.11	0.4	0.82	0.92
		VL	-	-	0.42	0.85	1.07	0.79
	VC	P	-	-	0.19	0.48	0.84	0.87
		ML	-	-	0.1	0.35	0.67	0.95
		VL	-	-	0.37	0.76	0.98	1.21
		AP	P	0.18	0.46	0.62	0.87	1.19
ML	0.05		0.59	0.72	1.11	1.56	1.93	
VL	0.32		0.79	0.87	0.97	0.92	1.04	

Tabla 7. Valores de $\psi_{E+D,0}$ y $\psi_{E+D,50}$ para el puente con $H_p=50$ m**Table 7.** Values of $\psi_{E+D,0}$ and $\psi_{E+D,50}$ for the bridge with $H_p=50$ m

$\Psi_{E+D,0}$		EC1	EC2	EC3	EC4	EC5	EC6	
Pilares <i>Columns</i>	P	0.64	0.72	0.76	0.84	0.95	1	
	ML	0.63	0.73	0.79	0.89	0.92	0.97	
	VL	0.62	0.64	0.68	0.72	0.77	0.83	
	MT	0.74	0.9	0.96	1.08	1.18	0.94	
	VT	0.84	0.95	0.98	1.09	1.14	0.92	
Torres <i>Towers</i>	ML	0.37	0.28	0.29	0.6	1.44	0.68	
	VL	0.36	0.27	0.28	0.58	1.41	0.67	
Cable 6 3 1	P	-	-	1.02	1.06	1.23	1.33	
	P	-	-	-	0.98	1.16	1.21	
	P	-	-	-	-	1.1	1.13	
Tablero <i>Deck</i>	VL	P	-	-	0.2	0.51	0.88	0.94
		ML	-	-	0.08	0.32	0.62	1.09
		VL	-	-	0.48	0.82	2.43	0.81
	VC	P	-	-	0.16	0.4	0.7	0.69
		ML	-	-	0.09	0.31	0.62	0.71
		VL	-	-	0.38	0.62	0.95	0.92
	AP	P	0.15	0.39	0.54	0.74	0.99	0.96
		ML	0.08	1.04	1.15	1.55	2.68	2.6
		VL	0.33	0.84	0.89	0.93	0.99	1

$\Psi_{E+D,50}$		EC1	EC2	EC3	EC4	EC5	EC6	
Pilares <i>Columns</i>	P	0.65	0.72	0.76	0.84	0.95	1	
	ML	0.61	0.72	0.78	0.87	0.9	0.95	
	VL	0.61	0.64	0.67	0.71	0.76	0.82	
	MT	0.74	0.9	0.96	1.08	1.18	0.94	
	VT	0.84	0.95	0.98	1.09	1.14	0.92	
Torres <i>Towers</i>	ML	0.36	0.27	0.29	0.59	1.43	0.67	
	VL	0.36	0.27	0.28	0.58	1.4	0.67	
Cable 6 3 1	P	-	-	1.06	1.09	1.27	1.38	
	P	-	-	-	1.03	1.23	1.28	
	P	-	-	-	-	1.2	1.22	
Tablero <i>Deck</i>	VL	P	-	-	0.22	0.56	0.95	1.02
		ML	-	-	0.09	0.34	0.65	1.15
		VL	-	-	0.43	0.74	2.2	0.74
	VC	P	-	-	0.18	0.46	0.8	0.79
		ML	-	-	0.08	0.29	0.57	0.65
		VL	-	-	0.34	0.56	0.87	0.84
	AP	P	0.17	0.45	0.61	0.84	1.13	1.09
		ML	0.09	1.15	1.27	1.72	2.96	2.88
		VL	0.32	0.82	0.86	0.9	0.96	0.98

4. Assessment of seismic vulnerability

As previously exposed, the probability of occurrence of the design earthquake during construction is quite low. However, since in some construction stages the bridge has a considerable amount of mass and a lower redundancy compared to the complete bridge, low magnitude earthquakes with higher annual exceedance probability can take place thus generating seismic forces equal to those taking place in the completed bridge. In this section, the main purpose is to calculate the magnitude of such earthquakes, in other words, the value of seismic acceleration coefficient, so that seismic forces during construction phase shall be equal to the ones obtained for the completed bridge in regards to seismic activity, with a return period equal to 475 years. So as to calculate such accelerations, the Equations 6 and 7 are introduced, which use allows obtaining the reduced seismic acceleration coefficient value (a_r). In such equations, the subscript indicate the conditions $t=0$ and $t=50$ years, while a_0 indicates the acceleration coefficient value employed for the bridge design under completed condition ($a_0=0.22$).

$$\text{Para cargas sísmicas/ For seismic loads} \quad a_{r,i} = \frac{E_{s,i}}{E_c} a_0 \quad (6)$$

$$\text{Para cargas sísmicas y permanentes/ For constant seismic loads} \quad a_{r,i} = \frac{E_{s,i} + D_{s,i} - D_c}{E_c} a_0 \quad (7)$$

After assessing these Equations, Table 8 presents the (minimal) critical reduced acceleration values for all the bridges under study. Besides, by using the acceleration recurrence curve, shown by Figure 7, the associated exceedance probabilities are included. Table 8 does not show (a_r) value of cables for the condition “seismic load + permanent load” because concrete creep and shrinkage considerably reduce cables stress, therefore (a_r) values would be negative. Probability values are not shown for accelerations lower than 0.05g, since that information was not available for the recurrence curve.

Table 8 shows that all structural elements have a higher seismic vulnerability during construction phase, because low magnitude earthquakes are likely to take place but having a higher annual exceedance probability of 0.21%, which is used for the design of complete bridge; thus exposing the structure to a high risk of losing a structural element, if the capacity to withstand seismic forces is not enough. For example, it is probable that seismic forces during construction phase can be annually exceeded by 0.41% in columns; in towers by 4.21%, in cables they may be exceeded by 7.81% and, 3.07% in the deck. In some cases, considering the effect of dead loads on seismic vulnerability, there is a positive effect on the deck, demonstrating the dependence on the structure dead load distribution. However, as the effects differed throughout time can significantly modify the bridge structural response under permanent loads, a negative effect is likely to take place, for example in the cables.

Tabla 8. Valores de a_r y probabilidad de excedencia anual para los puentes estudiados

Table 8. Values of a_r and annual exceedance probability for the bridges under study

		Hp=25 m				Hp=37.5 m				Hp=50 m				
		Ec. 6		Ec. 7		Ec. 6		Ec. 7		Ec. 6		Ec. 7		
		ar	%exc.	ar	%exc.	ar	%exc.	ar	%exc.	ar	%exc.	ar	%exc.	
Pilares Columns	P	0.16	0.59	0.2	0.29	0.14	0.88	0.19	0.35	0.17	0.49	0.22	0.21	
	Mx	0.21	0.25	0.24		0.26		0.24		0.23		0.23		
	Vx	0.18	0.41	0.2	0.29	0.24		0.26		0.26		0.27		
	My	0.15	0.71	0.15	0.71	0.18	0.41	0.18	0.41	0.19	0.35	0.19	0.35	
	Vy	0.16	0.59	0.16	0.59	0.19	0.35	0.19	0.35	0.19	0.35	0.19	0.35	
Torres Towers	My	0.08	4.21	0.09	3.07	0.11	1.75	0.11	1.75	0.15	0.71	0.15	0.71	
	Vy	0.08	4.21	0.05	9.2	0.11	1.75	0.11	1.75	0.15	0.71	0.15	0.71	
Cables 6 3 1	P	0.09	3.07	-	-	0.06	7.81	-	-	0.08	4.21	-	-	
	P	0.09	3.07	-	-	0.08	4.21	-	-	0.12	1.37	-	-	
	P	0.12	1.37	-	-	0.11	1.75	-	-	0.17	0.49	-	-	
Tablero Deck VC	VL	P	0.18	0.41	0.09	3.07	0.21	0.25	0.11	1.75	0.16	0.59	0.2	0.29
		My	0.22	0.21	0.23		0.22	0.21	0.25		0.17	0.49	0.18	0.41
		Vy	0.18	0.41	0.21	0.25	0.14	0.88	0.18	0.41	0.17	0.49	0.21	0.25
	VC	P	0.16	0.59	0.71		0.16	0.59	0.34		0.23		0.53	
		My	0.18	0.41	0.23		0.17	0.49	0.21	0.25	0.28		0.33	
		Vy	0.12	1.37	0.22	0.21	0.09	3.07	0.16	0.59	0.15	0.71	0.23	
		P	0.16	0.59	0.03	-	0.2	0.29	0.03	-	0.28		0.05	9.2
	AP	My	0.09	3.07	0.08	4.21	0.09	3.07	0.09	3.07	0.16	0.59	0.03	-
		Vy	0.16	0.59	0.27		0.11	1.75	0.19	0.35	0.13	1.09	0.22	0.21

5. Conclusions

By comparing the seismic response during six constructive stages and two in-service condition (before and after concrete creep and shrinkage took place), the seismic vulnerability of a 3-span extradosed bridge (60+100+60 m) - built with the balanced cantilever method - was evaluated. The study includes the columns height as a research variable.

The achieved results allows us to conclude that extradosed bridges have a higher degree of vulnerability during construction phase than in-service bridges, because in the first stage it is possible that seismic forces with annual exceedance probabilities may surpass the 0.21% exceedance probability adopted for the seismic design of the whole bridge. In this structure, the towers and extradosed cables are the structural elements that show greatest seismic vulnerability during construction and, therefore, special attention shall be paid in the seismic analysis during construction phase. In columns, the internal forces in the bridge transverse direction showed the highest vulnerability, while the axial moment and forces in longitudinal direction demonstrated not being critical. In deck, the highest seismic vulnerability was found for the area connecting with columns, which increases as long as the columns stiffness decreases. This research also allows us to conclude that constructive stages executed after the construction of the intermediate stayed-dowel represent the most vulnerable conditions; since the bridge has a lower structural redundancy than that of the complete bridge and a high percentage of total mass in these stages.

It shall be taken into account the fact that during construction the bridge shows a higher seismic vulnerability than after construction is completed, as it does not indicate that structural elements may reach their ultimate limit condition and they might fail as well, unless their structural capacity is not well able to withstand seismic activity. This research did not determine the structural capacity of the bridge to endure seismic forces during the construction phase. However, it is important to consider this aspect in further researches, so as to determine the safety degree for bridges design.

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