Seismic analysis of a cable stayed bridge in Talavera de la Reina, Spain

S. Hernández[1], L. E. Romera[1], F. Pereira[1] & A. Díaz[2]
[1]*Structural Mechanics Group, School of Civil Engineering, University of Coruña, Spain*
[2]*SOM, Skidmore, Owings and Merrill LLP, San Francisco, USA*

Abstract

A cable stayed bridge with a signature design destined to be an icon in the region has been recently built in the city of Talavera de la Reina in central Spain. Although Spain is a country with no significant seismic peril, the recent earthquake that took place in the city of Lorca in the year 2011, with a Richter magnitude value of 5.1, has promoted studies of several structures of social and economical relevance. Therefore, a comprehensive analysis of seismic effects of the mentioned cable stayed bridge has been performed.

The study carried out included seismic analysis in both the frequency and time domain, considering several sources of nonlinearity as cable sag, cable prestress distribution and large displacements. Nonlinear seismic time history analyses were conducted considering artificial earthquakes in the three spatial directions.

In addition to the dynamic analysis a computer video was produced showing the expected deformation of the bridge for each of the natural modes of vibration and the seismic event. The bridge model developed for the visualization is very precise and contains many details, so the perception of the viewer of the computer video will be very similar to what will be seen if a real earthquake would happen in the bridge site.

Keywords: cable-stayed bridge, nonlinear dynamics, seismic analysis, visualization tools.

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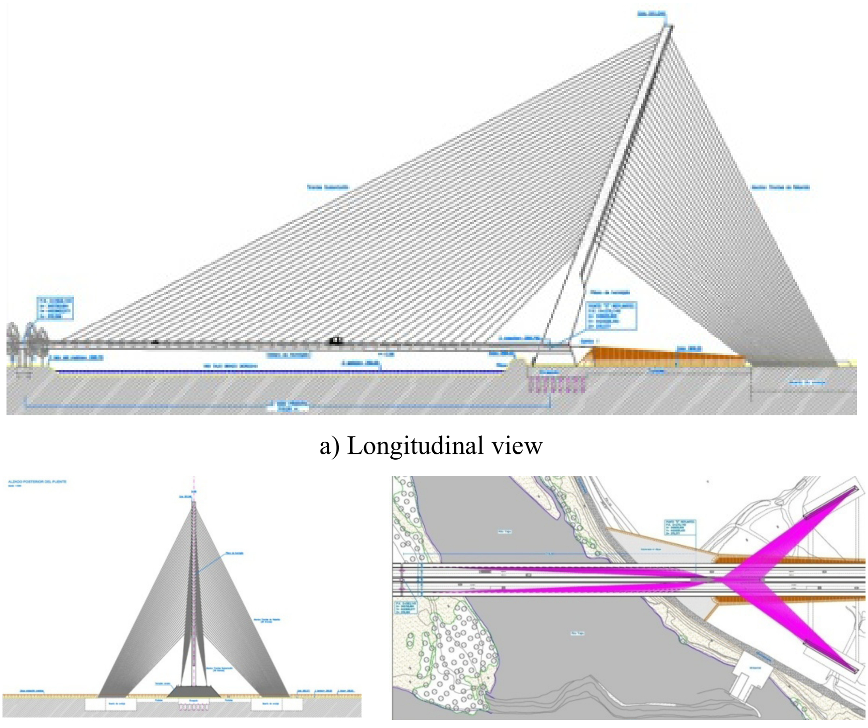
1 Introduction

Cable stayed bridges are one of the most used bridge typologies for spans between 200 m to about 1100 m, due to their structural efficiency, cost and aesthetics. They consist of a deck supported by a set of stay cables connecting it to one o more towers; this geometry combined with high strength materials used in its construction results in very slender and flexible structures, very sensitive to traffic, wind and seismic loads, being necessary detailed studies of their response to these loads [1, 2].

The bridge under study consists of a single span of 318 m, and an inclined tower at one end with a very elegant geometry and 164 m high (fig. 1). Two oblique planes of stays connect the tower with the deck; another two groups of stays connect the rear of the tower with the foundation balancing the loads on the tower.



a) Longitudinal view

b) Transversal view

c) Plan view

Figure 1: Bridge data and views.

The bridge deck (fig. 2), with four traffic lanes and one central lane for service is formed by a concrete box section with several cells and two lateral cantilevers for pedestrian traffic. Figure 3 shows two views of the bridge after construction.

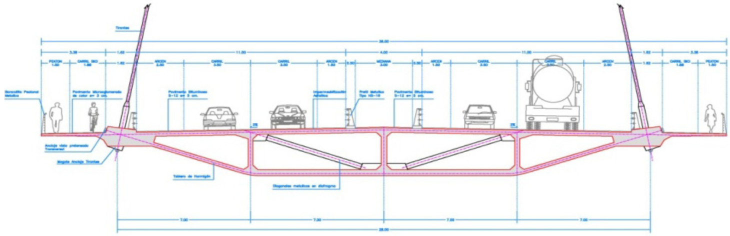


Figure 2: Bridge deck detail.

Figure 3: Views of the bridge after completion.

Among the most important nonlinear effects to consider in the design of a cable stayed bridge are cables and deck wind vibrations [3, 4], seismic analysis considering soil-structure interaction and multiple base excitations [5, 6], and pushover analysis to determine the resistant capacity [7]. Due to the dimensions and foundation soil of the bridge of Talavera de la Reina, in the seismic analysis performed have not been considered the effects of soil structure interaction and multiple base excitation.

The loads considered in design specifications are: dead loads (permanent and non-structural loads), live loads and seismic loads.

2 Structural model and methodology

To carry out the static and seismic analysis a global three-dimensional model of the complete bridge was developed using the commercial finite element program SAP-2000 [8]. The numerical model has a total of 4178 elements and 2265 nodes (fig. 4), where X is the longitudinal axis, Y the transversal axis and Z the vertical axis.

Each cable has been modelled with 40 beam elements in order to calculate its deformed geometry under prestressing loads in detail. The tower is modelled with 50 bar with different sections to take into account the variations of the section with height. To model the deck, longitudinal beam elements located in the center of gravity of each section and with the geometrical and mechanical properties of the corresponding transversal sections were used. Finally, transversal beams, with high rigidity and without mass, are arranged connecting the stays with the longitudinal beam elements [9].

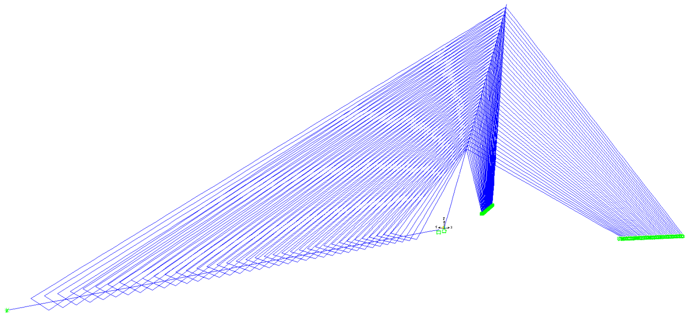


Figure 4: Structural model.

Prestressing of the cables, modelled as equivalent thermal loads, is calculated iteratively considering dead loads and a percentage of live loads, balancing the movement of the deck and the tower.

Materials considered are high strength steel for cables (elasticity modulus E = 1.8 x 10^8 kPa, Poisson coefficient ν = 0.2, thermal expansion coefficient α = 1.2 x 10^{-5} and specific weight γ = 77 kN/m^3), and high strength concrete in the tower and deck (elasticity modulus E = 4.2726 x 10^7 kPa, Poisson coefficient ν = 0.2, thermal expansion coefficient α = 1.2 x 10^{-5} and specific weight γ = 24.5 kN/m^3). Linear material behaviour was considered in the numerical model with the full stiffness of the sections, since in the prestressed deck ductility demand is very low and for the tower considering a strength design this approach gives conservative results.

Since the mass of the deck is associated only with the longitudinal beam elements, rotational masses are included in each node of the deck with a value of 31.3 kNms^2/rad in *X* direction to model adequately torsional mass inertia.

Natural frequencies and mode shapes, table 1, are calculated using the full geometric nonlinear stiffness matrix ($\boldsymbol{K}_{\mathrm{NL}}$) obtained from an initial nonlinear static step with dead and prestressing loads:

$$\begin{aligned}&\left(\boldsymbol{K}_{NL}-\omega^2\boldsymbol{M}\right)\boldsymbol{\Phi}=\boldsymbol{0}\\&\boldsymbol{K}_{NL}=\boldsymbol{K}_{L}+\boldsymbol{K}_{\sigma}+\boldsymbol{K}_{G}\end{aligned}\tag{1}$$

where $\boldsymbol{K}_{\mathrm{L}}$ is the linear stiffness matrix, $\boldsymbol{K}_{\sigma}$ is the geometric stiffness due to initial stresses, $\boldsymbol{K}_{\mathrm{G}}$ is the nonlinear stiffness due to node displacements, $\boldsymbol{M}$ is the lumped mass matrix, $\boldsymbol{\Phi}$ is the modal matrix and ω are the natural frequencies. If live loads are included in the calculation of $\boldsymbol{K}_{\mathrm{NL}}$, the variations of vibrations modes are not significant.

Figure 5 shows the first vertical deck vibration mode with a period of 3.45 s. To reach mass participating factors over the 90% in each spatial direction up to 58 modes of vibration should be included in the analysis.

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Table 1: Vibration periods T and modal participating mass ratios percentages in each direction u_x, u_y and u_z.

Mode	T (s)	% u_x	% u_y	% u_z	Description
1	3.45	3.01	0	38.36	First deck vertical
2	2.21	2.73	0	2.01	Second deck vertical
3	1.69	0	55.37	0	First tower transversal
4	1.67	0	22.21	0	First deck transversal
5	1.42	0.24	0	6.39	Third deck vertical

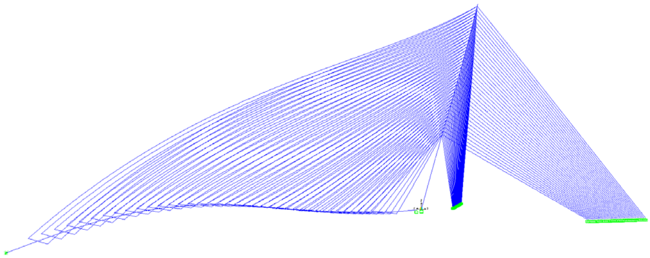

Figure 5: First vibration mode.

Regarding the vibration modes associated with the cables is interesting to compare the results of the numerical model and the analytical nonlinear formulation [3] using the equation

$$f = \frac{1}{2L\sqrt{m/gP}} \tag{2}$$

where L is the cable length in meters, m is the cable mass per unit length, P is the cable prestressing and g is the acceleration of gravity. Table 2 shows the results for five cables vibration frequencies, the differences between the two formulations are always less than 10%.

Table 2: Comparison of five cable vibration frequencies.

Cable	L (m)	m (Ns^2/m^2)	P (kN)	Analytical f (hz)	Numerical f (hz)	Difference %
1	90.09	1314.38	10190	1.53	1.65	7.29
2	98.47	344.17	2570	1.37	1.52	9.32
3	106.78	381.94	2800	1.26	1.38	9.24
4	115.16	433.47	3100	1.15	1.27	9.23
5	123.60	471.24	3385	1.07	1.17	7.92

3 Spectral dynamic analysis

Considering the bridge location and the Spanish seismic bridge code NCSP-07 [10], the elastic design spectrum for ultimate limit state (fig. 6), has a peak of 0.14g for periods in the range between 0.16 s and 0.64 s, and a peak ground acceleration (PGA) of 0.051g.

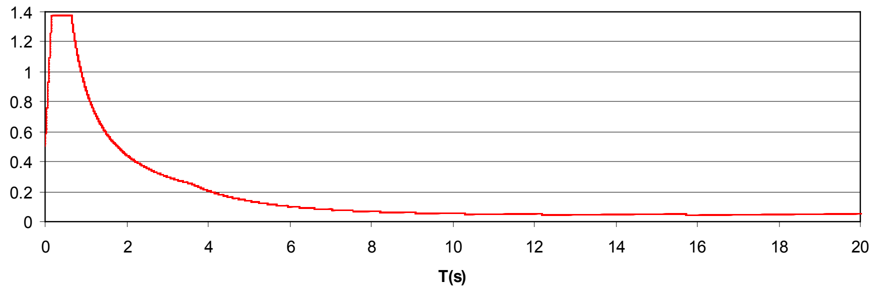

Figure 6: Pseudo-acceleration elastic design spectrum.

With structural damping of 4% according to NCSP-07 code for this type of bridges subjected to seismic ULS, and the action of two horizontal components with the elastic spectrum of figure 6, combined with one vertical seismic component with a value of 70% of the horizontal components, numerical results obtained are shown in tables 3 and 4, along with the static results by dead loads, live load of 4 kN/m^2 and a concentrated load of 600 kN in the center of the span.

Given the low seismicity and the essentially elastic performance expected for the bridge structure for the design earthquake, no behavior modification factor is utilized and seismic demands are extracted directly from the elastic response spectrum analysis.

Table 3: Maximum strength demands in the bridge.

	Tower longitudinal moment (kNm)	Tower transversal moment (kNm)	Deck transversal moment (kNm)
Seismic horizontal components	81816	163151	339011
Seismic vertical component	-	83173	-
Gravity design comb.	-	333336	-

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Table 4: Maximum displacements (mm) in the bridge.

	Tower longitudinal displacement	Tower transversal displacement	Deck transversal displacement
Seismic horizontal components	9.48	19.54	48.22
Seismic vertical component	11.35	-	59.63
Gravity design comb.	88.35	-	-

4 Nonlinear time-history seismic analysis

Artificial earthquakes shown in figure 7, with duration of 20 s and a time step of 0.005 s, have been used for the seismic non-linear time-history analysis. These earthquakes were generated previously for seismic analysis of a dam near the bridge.

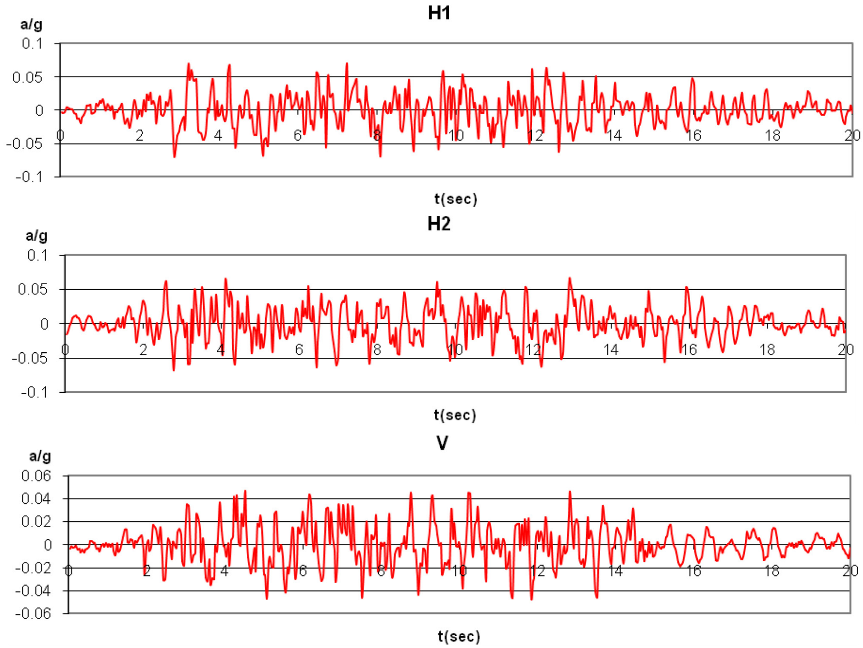

Figure 7: Artificial earthquakes (horizontal components H1, H2 and vertical component V).

Direct integration of nonlinear coupled dynamic equilibrium equations was made using Newmark's method with average acceleration integration (g = 0.5, b = 0.25), to achieve unconditional stability. Rayleigh damping was considered fitting a 4% damping for the period of 3.45 s and 0.36 s [10].

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Seismic analysis was carried out in two steps; the first to apply dead and live loads and then the earthquake load was applied with the response integrated with time steps of 0.005 s during 70 seconds. Results obtained are greater than those obtained in spectral analysis and are shown in figure 8 and tables 5 and 6. This is expected as the ground motions utilized from the nearby site were not scaled nor spectrally matched to the bridge structure and site. As a reference, compare the PGA for the design spectrum of 0.051g, versus the higher PGA of approximately 0.065g for the horizontal ground motions shown in figure 7.

Table 5: Maximum strength demands in the bridge.

	Tower longitudinal moment (kNm)	Tower transversal moment (kNm)	Deck transversal moment (kNm)
Seismic horizontal components	130178	-295011	560788
Seismic vertical component	-	83173	-

Table 6: Maximum displacements (mm) in the bridge.

	Tower longitudinal displacement	Tower transversal displacement	Deck transversal displacement
Seismic horizontal components	19.72	40.65	77.64
Seismic vertical component	11.35	-	59.63

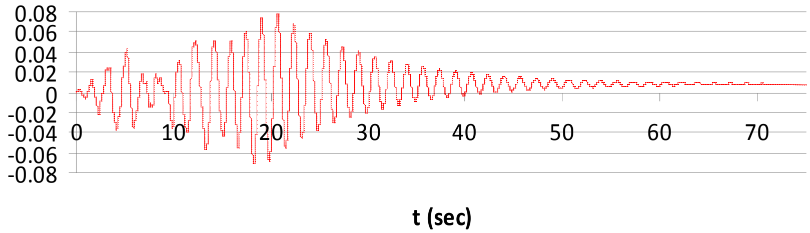

Figure 8: Transversal midspan deck movements (mm).

5 Seismic visualization

The use of a detailed digital model for visualizing the seismic motion of the bridge allows us to appreciate the vibration caused by the earthquake in the most realistic way, so the perception of the viewer of the computer video will be very similar to what will be seen if a real earthquake would happen in the bridge site [11].

MAYA software [12] was used for seismic visualization response (fig. 9, using the technique of animation based on lattices or Free Form Deformers (FFD) incorporated in this program. Using self-developed codes movements obtained in every time-step of the seismic analysis has been transferred to MAYA for animation.

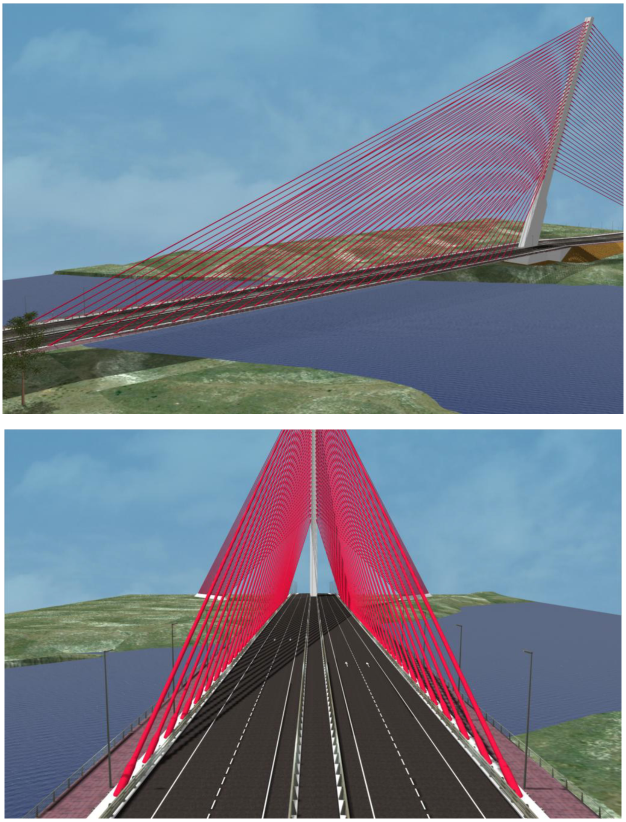

Figure 9: Detailed visualization model.

6 Conclusions

Results of nonlinear structural seismic analysis and visualization of cable-stayed bridge of Talavera de la Reina have been presented. Due to the location of the bridge in an area of low seismicity, the seismic analysis results are not critical but should be considered in the design of the structure.

Time-history seismic analysis results are more unfavourable than the spectral results due to the artificial earthquakes considered.

Current visualization tools can generate high quality realistic animations to visualize seismic movements in structures.

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