SEISMIC PERFORMANCE OF A LONG SPAN ARCH BRIDGE TAKING ACCOUNT OF FLUCTUATION OF AXIAL FORCE

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ABSTRACT :

The seismic behavior of a long-span arch bridge is investigated based on a series of nonlinear dynamic analyses. The bridge was modeled with ADINA by means of elements with nonlinear behavior Since the axial force and bending moment interaction is significant in the arch rib under strong ground motions, this effect is included in the analyses by frame elements varying their moment-curvature relations. It was found from the analyses that large axial forces, as much as 1.5 times the dead axial force are developed in the arch rib during excitations, so moment-curvature relations change in the arch rib and hence should be considered in design.

KEYWORDS: Arch bridges, nonlinear dynamic analysis, seismic performance.

1. INTRODUCTION

Arch bridges are one of the oldest structures to span topographical accidents. They are, even at the present time, broadly built in several countries because every time there are more and better constructive methods, besides the technological advance of the different structural materials and devices of seismic protection, which allows that, this kind of larger bridges can be built. At the moment there are bridges with more than 400 m of main span, as the case of the world records: the Wanxian Bridge with 425 m and the Chaotianmen Bridge with 552 m for concrete and steel bridges, respectively (Virola, 2008), both in China.

The arch bridges behavior is very contrasting because on one hand they are very efficient to transmit the high compression forces caused by gravitational load to the supports, which causes a limited ductility and it can take to a fragile failure; on the other hand they present a complex behavior during the occurrence of strong earthquakes, and although there is not registration of severe damages in the past due to these kind of events, the bridge can be vulnerable after a seismic event against possible replicas.

In the last 20 years several studies have been conducted to determine and clarify the seismic response of arch bridges. Recently, Torkamani and Lee (2002), Nazmy (2003) and Usami *et al.* (2004) analyzed several aspects of the steel arch bridges behavior. Regarding to concrete arch bridges, it must be mentioned the studies of McCallen *et al*. (1999), Kawashima and Mizoguti (2000), and Sakai and Kawashima (2002). In spite of the scarce number of studies related to this topic, a complete state-of–the-art is presented by Alvarez and Aparicio (2003). The studies pointed out the importance of an adequate evaluation of gravitational and accidental loads to assess the vulnerability and possible structural damage during the bridge life-cycle.

One of the most important factors to consider in the arch bridges dynamic behavior is the fluctuation of the axial force in their main element during an earthquake, since the interaction with the flexural moment can cause that the arch rib presents flexural yielding in some sections of its length, in such a way that is necessary to study more meticulously the nonlinear behavior of the bridge due to this kind of internal forces combination.

The study of the inelastic behavior of a reinforced concrete arch bridge under strong ground motions is presented in this work. The seismic movements used in this investigation are the records of Campano-Lucano (Italy, 1980), Michoacan (Mexico, 1985), Northridge (USA, 1994), Kobe (Japan, 1995), and Chi-Chi (Taiwan, 1999), all lineally scaled to 0.5g in the longitudinal direction (L) as shown in Table 1, and also the transversal

(T) and vertical (V) directions are presented.

Table 1 Scaled seismic records

Ground motion	M_W	Station	Direction	PGA (g)	PGV (m/s)	PGD (m)
Campano-Lucano, Italy, 1980	6.5	Sturno	L	0.500	0.858	0.254
			T	0.334	0.512	0.211
			V	0.364	0.308	0.105
		Torre del Greco	L	0.500	0.450	0.192
			T	0.342	0.400	0.150
			V	0.300	0.433	0.283
Michoacan, Mexico, 1985	8.0	Caleta de Campos	L	0.500	0.671	1.619
			T	0.497	0.860	2.059
			V	0.315	0.577	1.692
		Mesa vibradora	L	0.500	1.410	1.346
			T	0.462	1.410	4.551
			V	0.256	1.167	2.038
Northridge, USA, 1994	6.7	Castaic-Old Ridge Route	L	0.500	0.459	0.083
			T	0.452	0.460	0.131
			V	0.191	0.109	0.053
		Riverside-Airport	L	0.500	0.242	0.039
			T	0.461	0.211	0.023
			V	0.172	0.180	0.031
Kobe, Japan, 1995	6.9	KJMA	L	0.500	0.495	0.108
			T	0.365	0.452	0.122
			V	0.209	0.233	0.063
		OKA	L	0.500	0.296	0.130
			T	0.364	0.198	0.099
			V	0.235	0.154	0.105
Chi-Chi, Taiwan, 1999	7.6	TCU052	L	0.500	1.413	2.938
			T	0.415	1.897	2.200
			V	0.288	1.319	1.951
		TCU014	L	0.500	0.900	1.000
			T	0.387	1.613	2.493
			V	0.120	0.413	0.540

The three components of the earthquakes are used for the analysis, those which, they conform a representative collection because they have different duration and frequencies content. The three-dimensional dynamic behavior of the bridge was evaluated in order to know the temporary behavior of its main parameters in different areas of the structure.

The bridge was modeled in 3D using the ADINA code (ADINA R&D, 2005), which allows taking account of the geometric and material non linearity, the last one is modeled by configuring the elements behavior by means of several moment-curvature relationships (M-C) corresponding to different levels of axial force; this way it is possible to study the effect of the axial force fluctuation interacting with the flexural moment by means of step-by-step nonlinear dynamic analysis.

2. ANALYTICAL MODEL

The bridge model under consideration, dimensioned on purpose for this investigation (Alvarez, 2004), represents a reinforced concrete deck-type arch bridge shown in Figure 1, with a total length of 600 m and a main span of 400 m. The arch is a catenary with a rise to span ratio of 1:5. The superstructure is a continuous deck with 20 m width (four-lane highway). Figure 2 shows the cross sections of the arch bridge; as we can see, the deck is connected to the arch and the piers by two multidirectional bearings, and also a transversal shear key to avoid relative displacement between the deck and the support element was considered.

Fixed bearings in bottom of piers P1 to P3 and piers P13 to P15 are assumed. It is well accepted this assumption when the bridge foundation is over bedrock sites, as is the case in the majority of this type of bridges constructed. As a consequence, it is not expected an important contribution of the soil-structure interaction. Piers P3 and P13 are rigidly connected to the deck superstructure while a pinned connection is selected among piers P2, P4, P12 y P14 and the upper deck superstructure. The piers over the arch rib are also rigidly fixed to the superstructure. Horizontal free movement is provided by PTFE bearings in the other piers and in both abutments. In all piers and abutments with sliding bearings, a shear key to constraint transverse displacements is proposed (Figure 2). Thus, in longitudinal direction, piers P2, P3, P4, P12, P13 and P14 support earthquake inertial forces while the arch rib, abutments and all piers support inertial forces in transverse direction

In order to take into account the effect of fluctuation of axial force during an earthquake, two models were used in this investigation. The *primary model* consists of fiber elements for modeling the arch rib and three-dimensional beam elements for piers and deck; due to the complexity and time-consuming of the former, the *final* one models the entire bridge by three-dimensional beam elements. Both models are shown in Figure 3. The *final model* was calibrated from the *primary model* comparing the natural mode shapes, the total bridge weight and the vertical displacement at the arch crown.

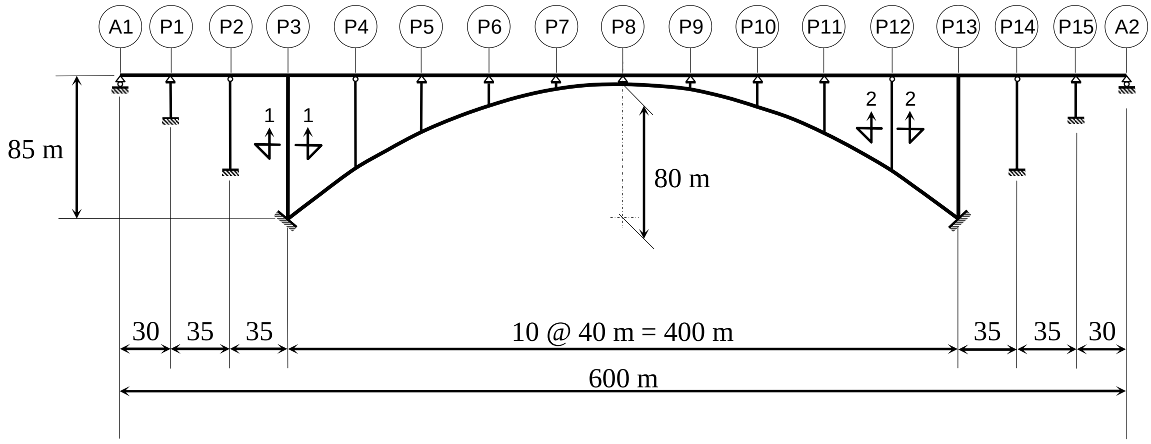


Figure 1 Longitudinal view of the bridge

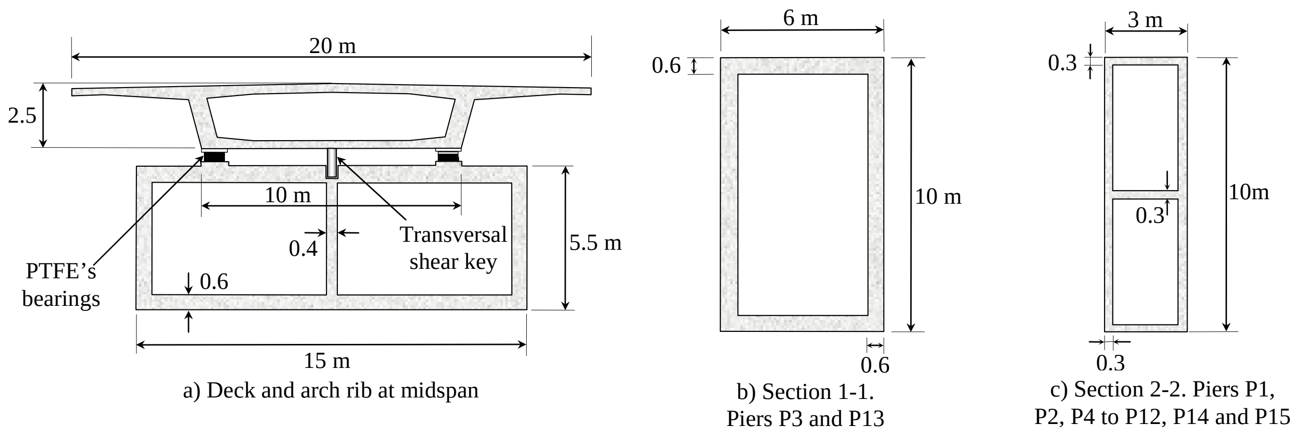
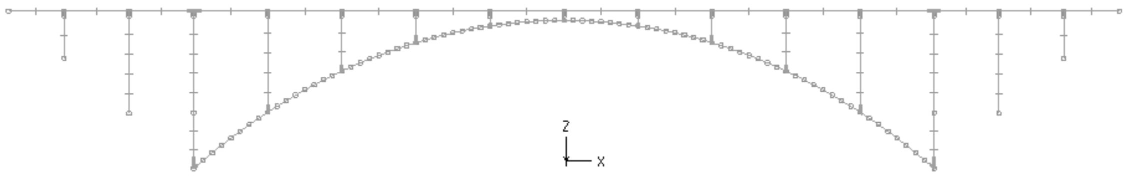


Figure 2 Cross sections of the deck, arch rib and piers

a). Primary Model



b). Final Model

Figure 3 Analytical models of the bridge

Tables 2 and 3 compares the main results used to calibrate both models. Table 2 shows natural periods and modal participating mass ratios for the first nine modes: as expected, the mode shapes corresponded to the same directions. For instance, the first mode has a longitudinal shape; the second one is transversal and so on. Also, the periods and the accumulated mass are very similar between the modes of each model. About the weight and vertical displacement, Table 3 shows the results that are identical referring to the weight and only 3 mm of difference talking about the maximum displacement at the arch crown due to gravity load.

Table 2 Natural periods and cumulative modal mass

Mode	Primary Model				Final Model			
	Period	Cumulative modal mass (%)			Period	Cumulative modal mass (%)		
		X	Y	Z		X	Y	Z
1	3.213	48.70	0.00	0.00	3.578	52.94	0.00	0.00
2	2.417	48.70	57.29	0.00	3.572	52.94	57.21	0.00
3	1.756	54.62	57.29	0.00	2.598	54.40	57.21	0.00
4	1.195	54.62	57.29	0.00	1.571	54.40	57.21	0.00
5	1.098	54.62	57.29	1.28	1.465	54.40	57.21	0.23
6	0.918	54.62	77.50	1.28	1.092	54.40	70.84	0.23
7	0.747	54.62	77.50	1.28	0.842	56.70	70.84	0.23
8	0.695	57.26	77.50	1.28	0.811	56.70	70.84	0.23
9	0.668	57.26	77.50	44.53	0.709	56.70	70.84	34.56

Table 3 Weight and displacement at the each crown due to gravity load

Model	Weight (MN)	Vertical displacement (mm)
Primary	654.636	149
Final	654.636	146

3. RESULTS

Once the dynamic analyses were performed, the main bridge responses were compiled and compared. In this work, as a summary, results from Figures 4 to 8 are presented. The complete information is presented in Chavez (2008). Figure 4 compares the average values of the maximum longitudinal displacements and the corresponding ones to each seismic excitation, with and without considering the axial force (N) and bending moment (M) interaction. As seen, the displacements considering N-M interaction in the analyses are larger than the responses without N-M interaction. Referring to the maximum absolute acceleration, Figure 5 shows the vertical component where is evident the effect that axial force fluctuation (N-M interaction) has in incrementing the magnitude of the acceleration, hence, larger internal forces may occur.

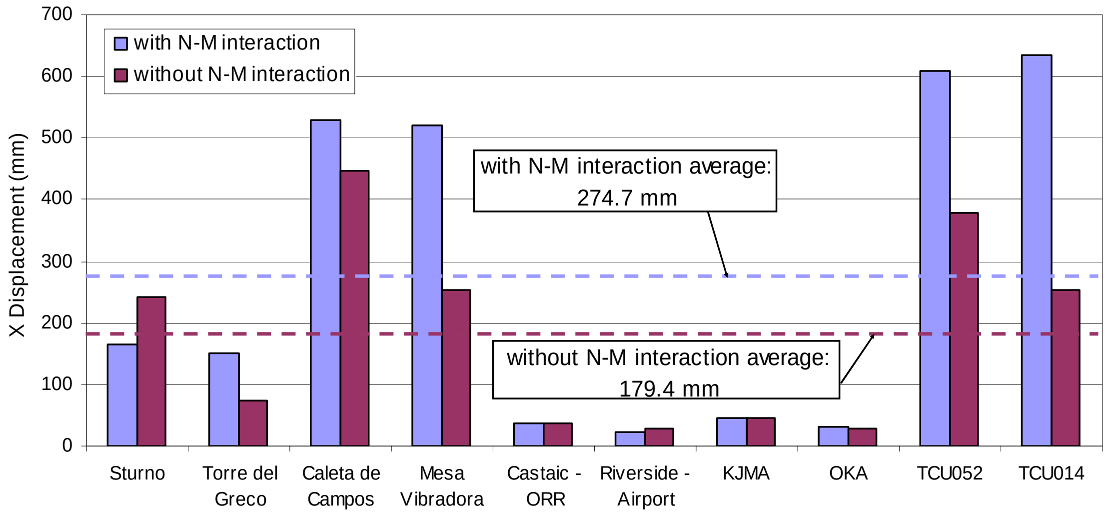


Figure 4 Maximum longitudinal displacements at midspan

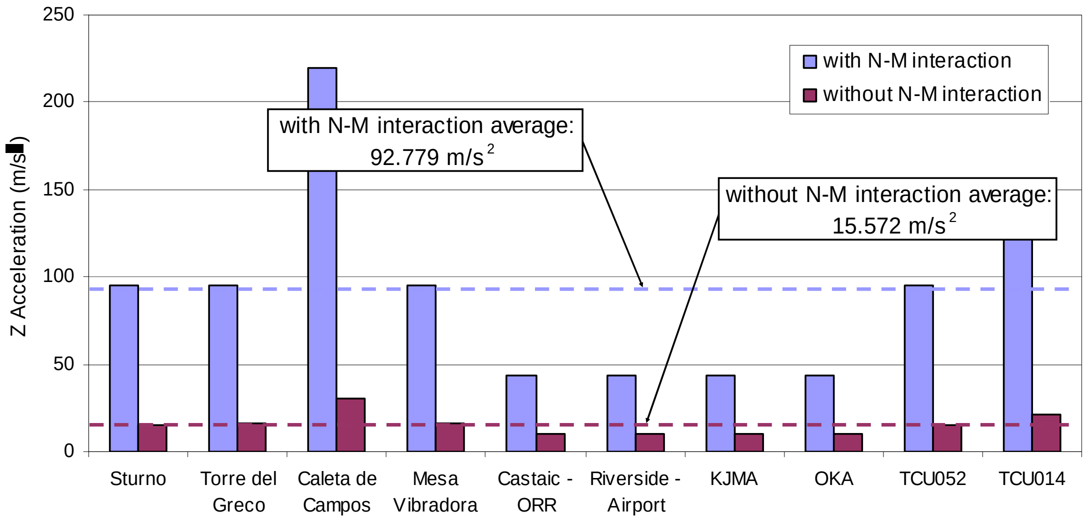


Figure 5 Maximum absolute vertical acceleration at the arch crown

Another important parameter studied along this work was the curvature ductility. For instance, as shown in Figure 6 the capacity of certain sections could be reached or exceeded for some seismic records when considering N-M interaction. The axial force variation in the arch rib was also monitored as shown in Figure 7; as presented, the left springing experimented larger variations of axial force when N-M interaction was taken into account. A similar trend was observed with regard to the bending moments. Figure 8 presents how the yielding and ultimate moments were exceeded by the ones obtained for some seismic records, even if the N-M interaction is ignored (TCU052 and Mesa Vibradora records).

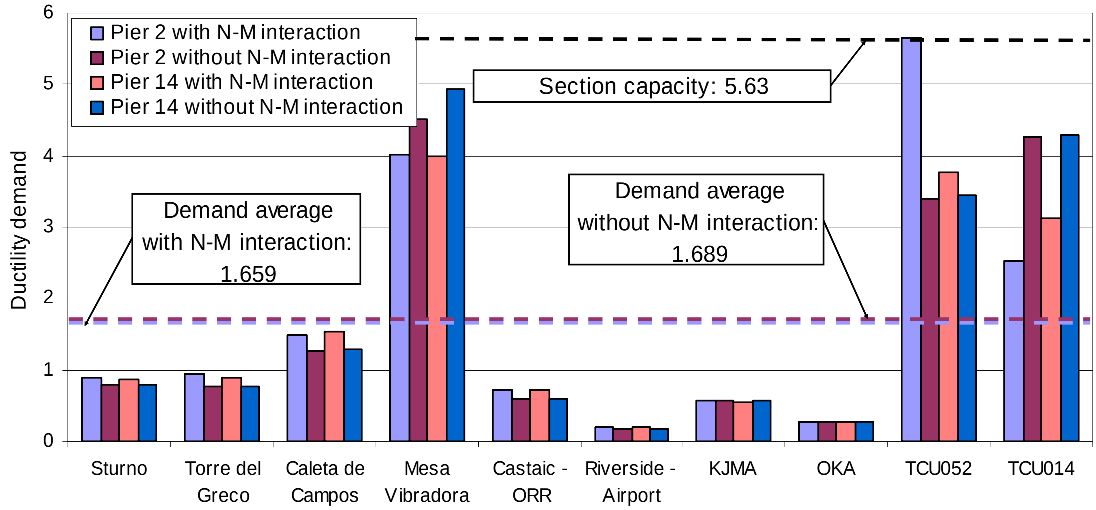


Figure 6 Curvature ductility demand at the bottom of piers P2 and P14 (out-of-plane response)

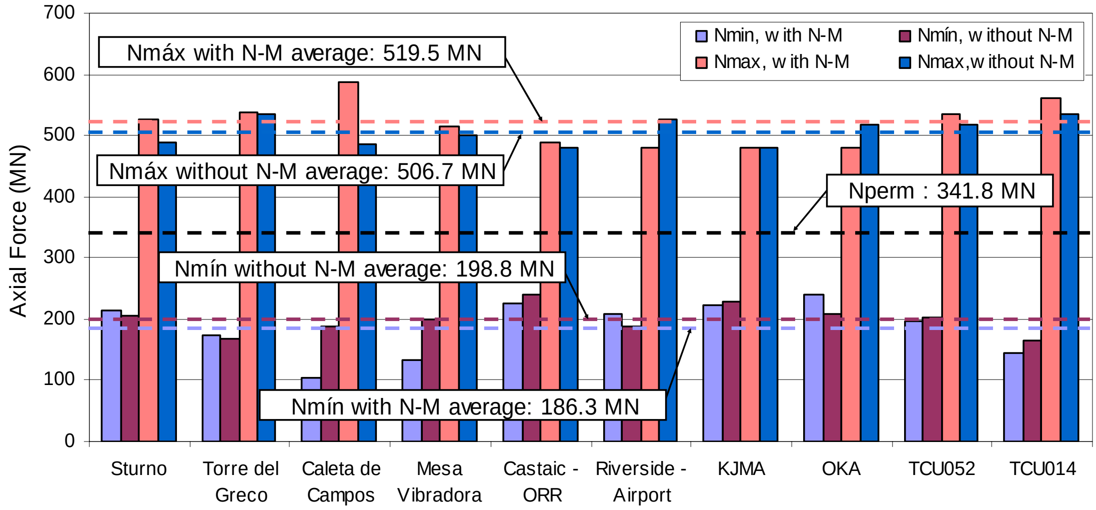


Figure 7 Maximum and minimum axial force at the left springing

4. CONCLUSIONS

- From all the seismic movements, the results showed that the stations with larger velocities produced larger accelerations, displacements, axial force fluctuation, bending moments and ductility demands, so this kind of structures, with long natural periods, are more vulnerable to ground velocity than acceleration.
- As the velocity, considering the N-M interaction in the analyses increases the model response in almost all the parameters monitored in this study.
- It was found from the analyses that large axial forces, as much as 1.5 times the gravity axial force are developed in the arch rib during excitations, so moment-curvature relations change in the arch rib and hence should be considered in design.

- Due to the nature of this kind of bridges, a brittle failure is likely to occur in some sections of the structure. The arch rib and the piers exhibited high levels of ductility demands and responses in general, considering the N-M interaction.

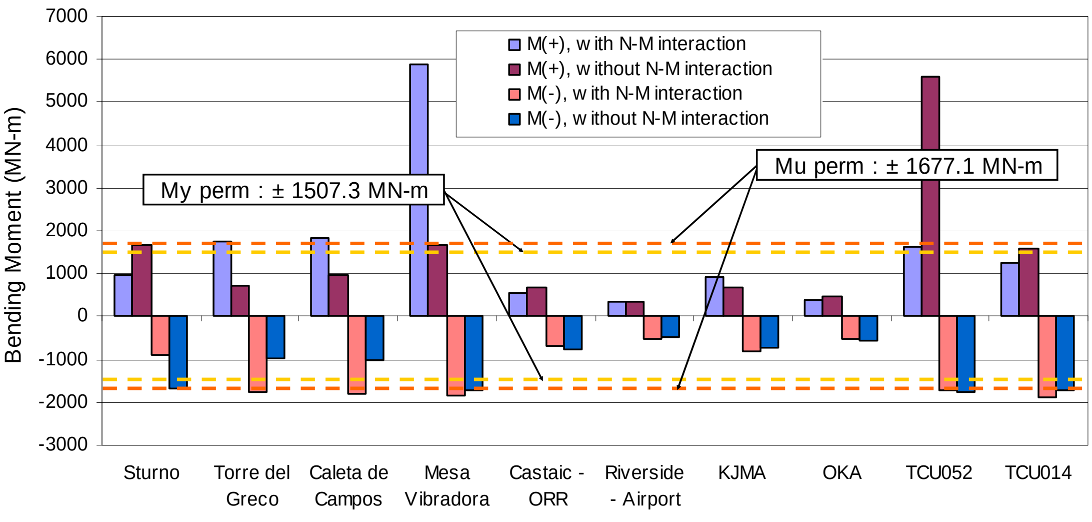


Figure 8 Maximum and minimum bending moment at the left springing (in-plane response)

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