13th World Conference on Earthquake Engineering
Vancouver, B.C., Canada
August 1-6, 2004
Paper No. 1858

NON-LINEAR SEIMIC RESPONSE OF AN ARCH BRIDGE WITH FLUID VISCOUS DAMPERS

José de Jesús ALVAREZ[1] and Ángel Carlos APARICIO[2]

SUMMARY

The seismic response of a concrete deck-type arch bridge with linear and non-linear fluid viscous dampers (FVDs) subjected to a ground motion including near-field effects is investigated using a 2D analytical model. FVDs were placed at both ends of the deck connecting the superstructure with the foundation. The force (F)-velocity (V) relation for FVDs can be expressed as $F=C\cdot V^n$; a parametric study aimed to identify the optimum dampers was carried out. The most efficient devices were non-linear dampers with C=15 MN/(m/s)$^{1/4}$ and n=1/4, reducing the longitudinal displacements of the superstructure by up to 72%.

INTRODUCTION

Arch bridges construction has reappeared around the world thanks to the cantilever launching method, and nowadays these structures represent one of the three major types of long-span bridges, with the other two types being suspension and cable-stayed bridges. The arch rib is an element mainly subjected to a large axial compression force caused by dead loads, and that's why arch bridge structures exhibit a complex behavior during strong earthquakes. Furthermore, when a bridge is located close to a fault system, the near-field effects must be considered, since the structure could experiment large displacements.

Several studies have been conducted to evaluate the seismic response of arch bridges. Steel bridges have been studied by Kuranishi [1], Dusseau [2], Nazmy [3] and Torkamani [4]. Regarding the concrete ones, we can highlight the investigations performed by McCallen [5], Kawashima [6] and Sakai [7]. A state-of the-art report is presented in Alvarez [8]. Nevertheless, the seismic response of arch bridges using passive energy dissipation devices has not been evaluated; an arch design is inherently non-ductile [9, p. 154], and thus for energy dissipation it is possible to resort to damping devices. In this study, the nonlinear dynamic behavior of a concrete deck-type arch bridge with viscous dampers and subjected to in-plane ground motion including near-field effects is investigated; a parametric study aimed to identify the optimum dampers was carried out.

1 Associate Professor, Universidad Michoacana de San Nicolás de Hidalgo, Mexico. Nowadays graduate student, Technical University of Catalonia, Spain. Emails: asereno@zeus.umich.mx, jesus.alvarez@upc.es
2 Professor, Technical University of Catalonia, Spain. Email: angel.carlos.aparicio@upc.es

DESCRIPTION OF THE MODEL

Geometry

Figure 1 shows a general view of the model, dimensioned on purpose for this investigation; it represents a reinforced concrete deck-type arch bridge with a main arch span of 400 m. The total length of the bridge is 600 m. The arch is a catenary with a rise to span ratio of 1:5. Both approaches consists of three spans, one of 30 m at the ends, and two more of 35 m. The superstructure is a continuous deck of 20 m width (four-lane highway). Figure 2 shows the cross sections of the structure at midspan and the piers. Along the first 60 m at both springings, the arch is non-prismatic, starting from 7 m deep and decreasing linearly to 5.5 m; the remainder is prismatic, with a 15 m wide and 5.5 m deep two-cell reinforced concrete box section (Figure 2).

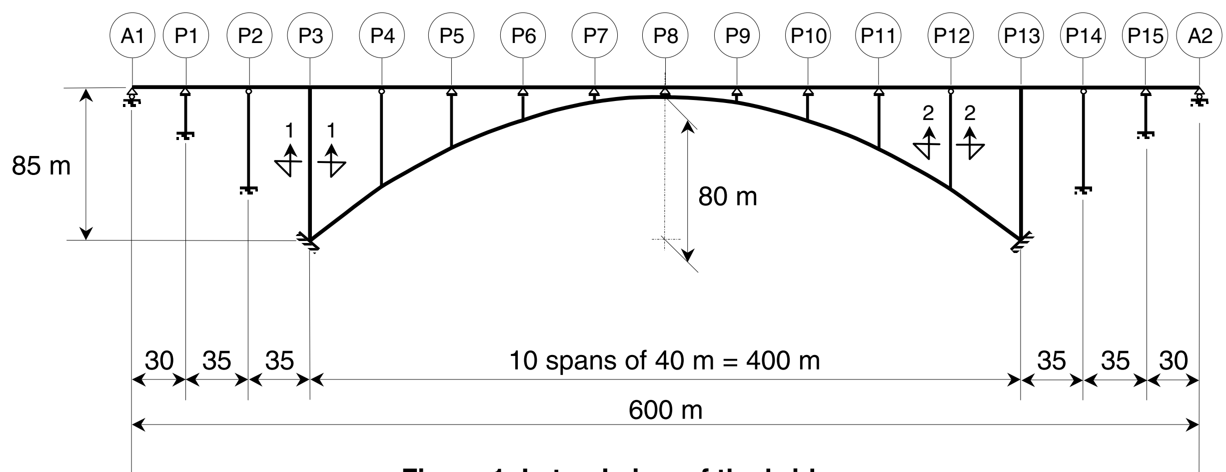
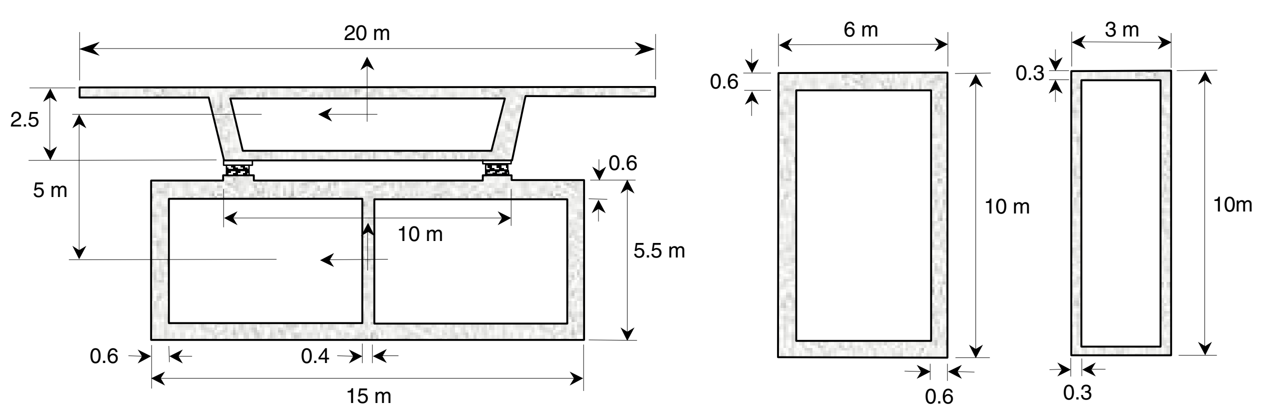


Figure 1. Lateral view of the bridge

Piers P4 to P12 are continuous with the arch rib. The main piers (P3 and P13) are continuous with the superstructure. The adjacent piers at each side of the main ones (P2, P4, P12 and P14) are pin-connected to the deck; we decided to consider it this way because of their slenderness. The remaining piers (P1, P5 to P7, P9 to P11 and P15) and both abutments are equipped with longitudinal PTFE bearings without seismic restraints. The special case of connection between the arch crown and the deck will be described later.



a) Deck and arch rib at midspan

b) Section 1-1. Piers 3 and 13

b) Section 2-2. Piers 1, 2, 4 to 12, 14 and 15

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

The superstructure was modeled by means of frame elements. As we considered a prestressed superstructure, frame elements were modeled with gross cross-sectional properties, as cracking should be negligible. The same assumption was made for the arch rib, because of the large compression force caused by dead load.

Bridge piers were also modeled as frame elements. Flexural and shear stiffness of the pier cross sections were taken as 70% of the gross cross-sectional stiffness to account for distributed cracking. Axial and torsional stiffness were based on gross cross-sectional properties. Two additional sets of finite elements were used for the top-end nodes of piers equipped with PTFE sliders: (1) no-mass elements that model the PTFE sliders on the top of the piers, and (2) rigid, no-mass elements that join the PTFE sliders to the center of gravity of the superstructure.

On the other hand, evaluation of non-linear behavior of hollow arch rib sections subjected to large flexural moments under significant fluctuation of axial force is complex [6]. Therefore, as a first trial, all frame elements were considered to be linear elastic. A concrete with compression strength of 40 MPa, modulus of elasticity of 30,891 MPa, Poisson's ratio of 0.2 and volumetric weight of 24.5 kN/m^3 was assumed.

Analytical model

Figure 3 shows the analytical model of the prototype. The bridge was modeled with *SAP2000 Nonlinear* [10], which allows to simulate viscoelastic devices based on the Maxwell model (which consists of an elastic spring and a viscous damper connected in series), by means of *Nllink* elements. At the ends of continuous members, *end offsets* was considered with a rigid-end factor equal to 0.5. Furthermore, *end releases* tool was used to disconnect the necessary degrees of freedom, suitable with the described prototype. About the boundary conditions, piers and arch springings were considered fixed, in view of such bridges are generally founded on rock or stiff soil, and in order to concentrate on structural response.

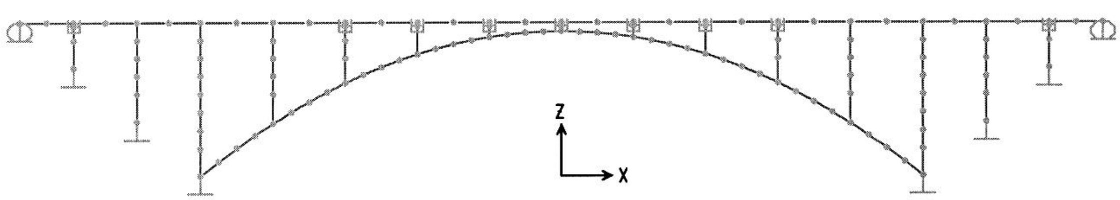


Figure 3. Analytical model of the bridge

With regard to the connection between the arch crown and the superstructure, two options were considered: (1) arch crown pin-connected to the deck, and (2) arch crown equipped with longitudinal PTFE bearings. With the first choice it is possible to control the longitudinal displacements, but we can expect large bending moments on the arch rib. With the second one, bending moments in the arch rib should be smaller, but longitudinal displacements of the superstructure would be larger. To decide which was more suitable, a modal seismic analysis of the prototype was performed using the Eurocode 8 design spectra in longitudinal direction, considering a ground acceleration equal to 0.25g (maximum in Spain according with the local code), subsoil class A, elastic behavior (q=1), importance factor equal to 1.3 and 3% of modal damping. In relation to this last parameter, a damping value of 5% of critical damping is commonly assumed for concrete structures [9, p. 176], but recent studies [11], [12] shows smaller values, so we decided to use 3%. Some of the results that allowed us to select the second configuration are presented in Table 1. We wanted to protect the arch rib as much as possible, and in any case, longitudinal displacements of the superstructure could be controlled by placing dampers at the ends of the deck, as it is described later. So, we decided to continue this work with the second configuration (deck resting over PTFE sliders placed over the arch crown).

Table 1. Displacements and forces caused by Eurocode 8 design spectra

	Arch crown and deck pin-connected	Arch crown with PTFE bearings
Peak longitudinal displacement of the superstructure	0.176 m	0.70 m
Peak axial force on arch springings	104.3 MN	86 MN
Peak bending moment on arch springings	1,124.1 MN-m	661 MN-m

Dynamic characteristics

Table 2 shows natural periods and modal participating mass ratios for the first ten in-plane modes. These properties include the P–Delta effect, considering the axial forces caused by dead load; this effect can be quite important in arch structures where the structural system is essentially a compression structure. It is observed that cumulative effective mass is not enough to accurately evaluate the response of the model, so in the different analysis the first 75 modes by means of Ritz vectors were considered, exceeding in this way the 99% of cumulative mass. Figure 4 shows the mode shapes for the first three modes; the first one corresponds to longitudinal displacement of the superstructure. Coupling between longitudinal and vertical motion was observed in several modes (as in mode 2). Mode 3 corresponds to vertical displacements.

Table 2. Natural periods and modal participating mass ratios

Mode	Period (s)	Cumulative sum (%)	
		Longitudinal	Vertical
1	4.232	55.6970	0.0000
2	3.131	56.9392	0.0000
3	1.673	56.9392	0.0006
4	0.962	59.0139	0.0007
5	0.807	59.0139	41.6281
6	0.637	59.0139	61.1799
7	0.531	59.0260	61.1799
8	0.497	59.0260	61.4524
9	0.455	59.0260	61.4669
10	0.450	60.9256	61.4669

INPUT GROUND MOTION

In the 1930's, a number of reinforced concrete arch bridges were built along the coast of California, U. S. A.; nevertheless, these structures tend to be seismically deficient by recent design standards. The California Department of Transportation (*Caltrans*) has recently developed a program of seismic evaluation and retrofit design for a number of these arch bridges. The Bixby Creek Bridge [5] was part of this program.

The Palo Colorado–San Gregorio fault system is considered to pass at a distance of approximately 1 km from the Bixby Creek site with a maximum credible earthquake potential of magnitude 7.5. Ground motion time histories for the site including near-field terms were developed by *Caltrans*, since of particular concern was the potential for large ground displacement pulses; these time histories have three separate components: fault normal, fault parallel and vertical. Time histories for fault normal component, which caused large displacements to the Bixby Creek Bridge, were selected for our 2D simulation analysis; these time histories are shown in Figure 5.

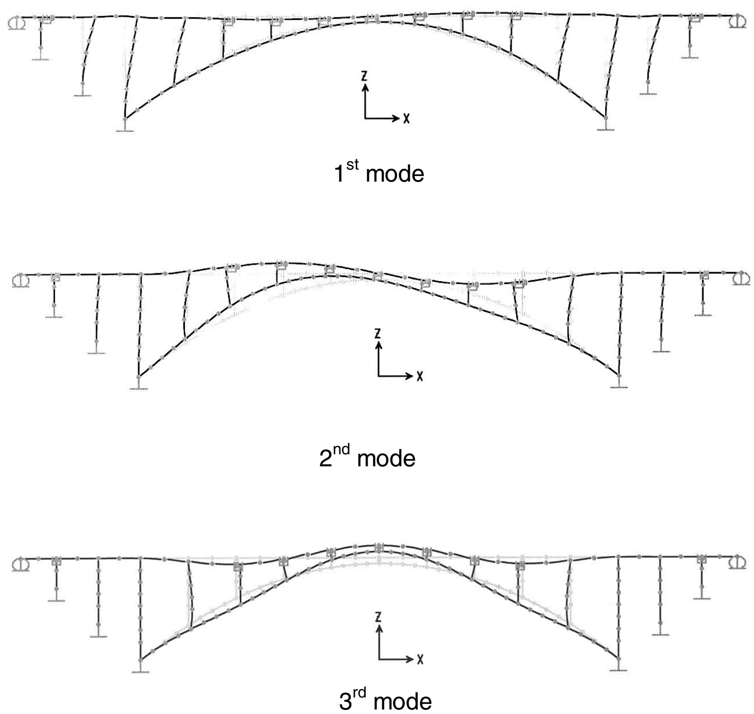


Figure 4. Natural mode shapes

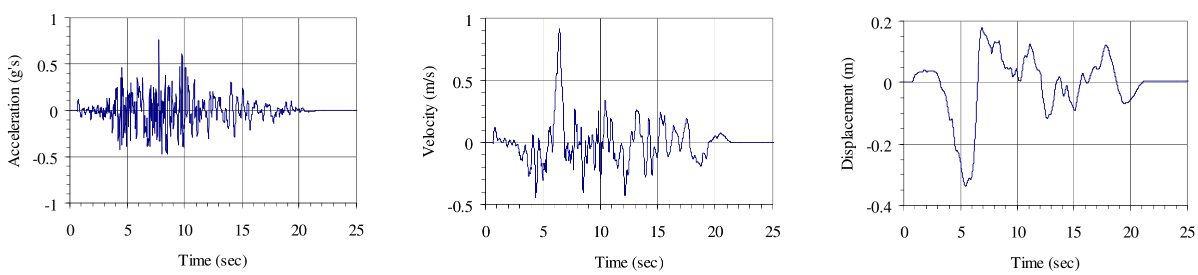


Figure 5. Time histories including near-field effects

DAMPING DEVICES

In order to reduce the seismic response of bridges, it is possible to resort to seismic isolation or energy dissipation devices. One of the better options in continuous long-span viaducts is the use of FVDs, because they can accommodate slow temperature displacements without forces at the devices. FVDs operate on the principle of flow of silicon fluid through orifices at high velocity [13, pp. 190-195]; these devices provide a restoring force (*F*), which is a function of the relative velocity (*V*). The constitutive law is

$$F = sign(V)C|V|^n \tag{1}$$

The exponent *n* defines the type of device, whereas the damping constant *C* controls the force that will be developed. These damping devices can be manufactured with a wide range of *C* and *n* values. The cost of the devices is generally proportional to the maximum damping force required [14]. Different values for the exponent *n* were evaluated; exponents n = ¼, ½ and 1 were considered. For each of these exponents, several values of *C* were examined in a parametric study aimed at identifying the optimum dampers; the adopted range of *C* values was chosen in order to obtain solutions with an equivalent linear damping ratio between 3% and 30%. For determining this equivalence, the modal energy plus the dampers energy of the model with FVDs, at the time when maximum potential energy occurred, was matched with the modal energy of the model without dampers at this same time, and with linear damping ratios between 3% and 30%. Two dampers were placed in each case, one at each end of the deck (Figure 6), connecting the superstructure with the abutment, which was assumed to be rigid.

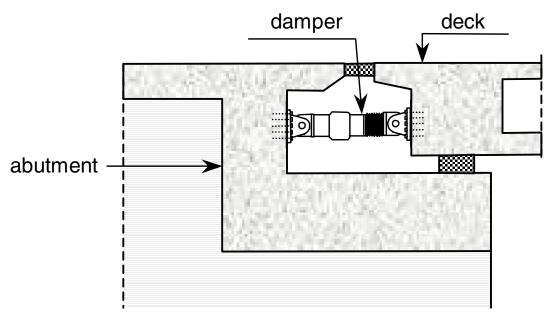


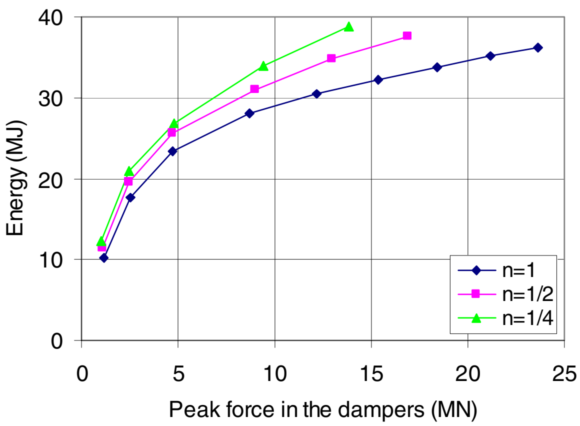
Figure 6. Location of the FVDs

ARCH BRIDGE RESPONSE WITH FVDs

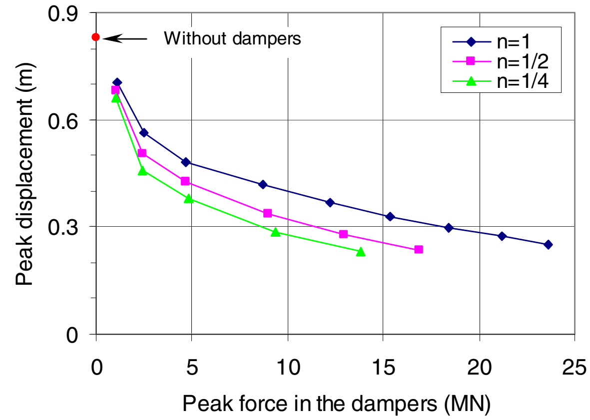
The in-plane seismic response of the bridge was studied, subjecting the model to the ground motion described earlier; the acceleration time history shown in Figure 5 was applied just in longitudinal direction. A total of 21 transient nonlinear analyses were carried out. The response of the bridge was monitored in several points of interest.

One of the main results of these analyses was the energy dissipated by the dampers. Figure 7.a shows the total energy dissipated by the dampers as a function of the peak damper force. Each point in this graph provides the results from one dynamic analysis using a set of values for *C* and *n*. The trend is quite clear; dampers with a small exponent *n* are preferable. For the same force capacity, they dissipate more energy than dampers with larger exponents. This advantage in energy dissipation is translated into a larger reduction in displacements and stresses. Figure 7.b shows the maximum displacement of the superstructure, and therefore of the dampers, as a function of the peak damper force. Again, dampers with *n*= ¼ are more efficient than those with *n*= ½ or 1. They achieve the same reduction in relative displacements with a smaller force. The results shown in Figures 7.c and 7.d are the maximum force and velocity in the dampers, in terms of *C* and *n*; these are important values for the design of the dampers.

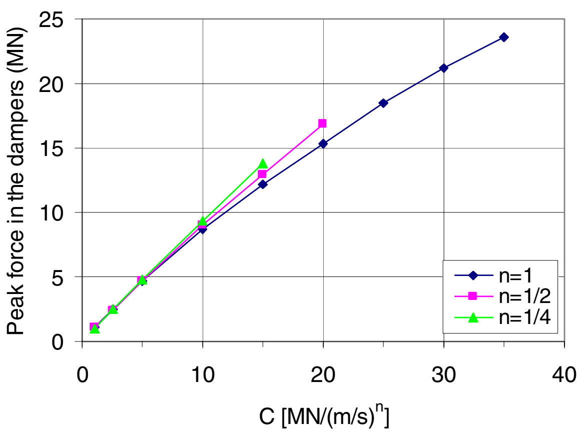
Earthquake maximum internal forces on the main piers bottom (P3 and P13) are shown in Figure 7.e. It is observed that axial force changed just a little, whereas bending moment had a remarkable drop for larger values of the peak damper force. Contribution of the dampers in the reduction of longitudinal displacements of the deck is matched with the reduction of bending moments at main piers, because of piers P3 and P13 were assumed to be continuous with the superstructure, and therefore they were very sensitive to any movement at their top. For the selected structural configuration, the arch rib springings were not as favored; Figure 7.f shows their maximum internal forces. Maximum axial forces almost did not change using dampers, and bending moments decreased about 14% in the best case.



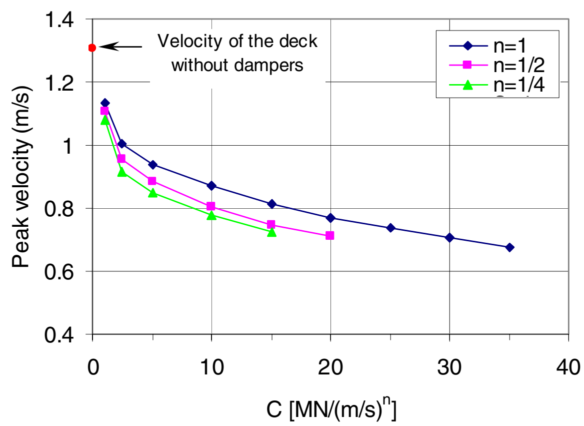
a) Energy dissipated by the dampers



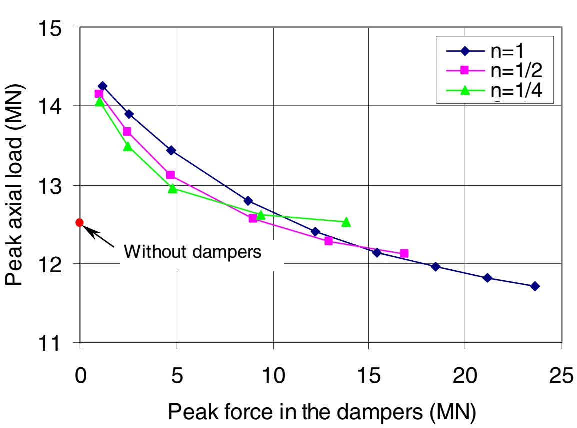
b) Displacement of the superstructure

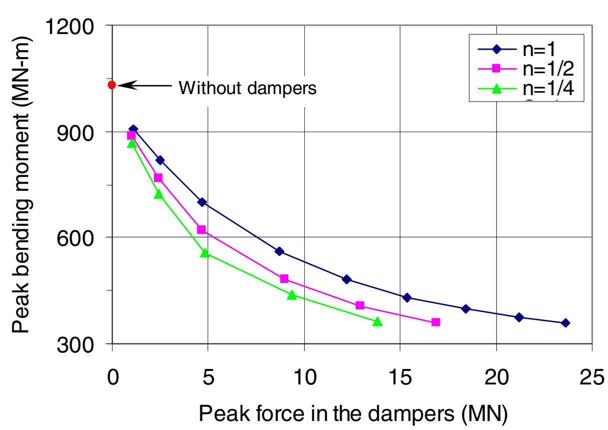


c) Force in the dampers



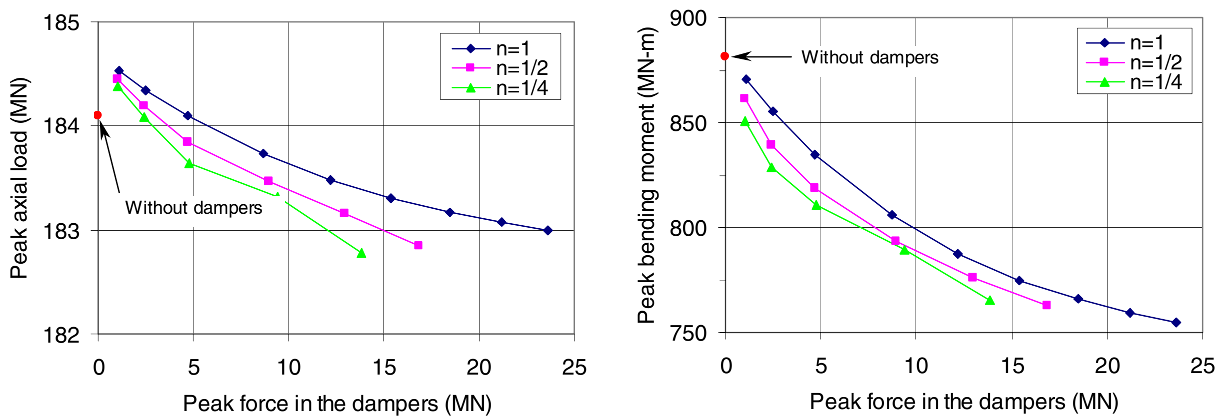
d) Velocity of the dampers





e) Earthquake internal forces on main piers bottom (P3 and P13)

Figure 7. Seismic response of the bridge subjected to a near-field ground motion



f) Earthquake internal forces on arch springings

Figure 7. *(Continued)*

In accordance with these results, the most efficient devices for the model and ground motion in study are nonlinear FVDs with C=15000 kN(s/m)$^{1/4}$ and n=¼, which provide to the structure an equivalent linear damping ratio of 30%; nevertheless, their contribution to the response was not beneficial enough. Figure 8 (left) shows the energy response for the case mentioned above; it is observed that energy dissipated by the FVDs hardly overtook the modal energy, because they did not work in an efficient way in the face of the near-field event, as it is observed in the narrow hysteretic loops (except for one) in Figure 8 (right), and because FVDs, in the selected structural configuration, did not directly contribute to reduce the vibration of the arch rib.

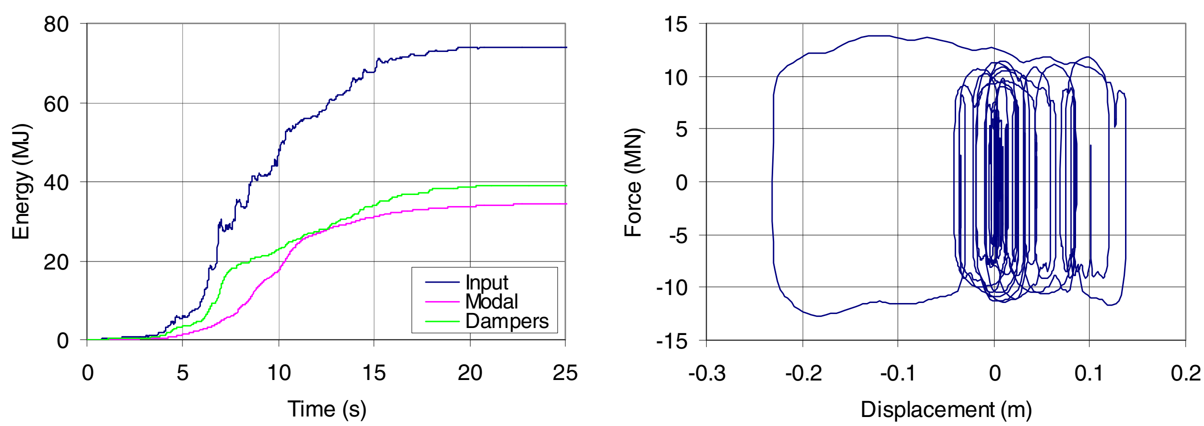


Figure 8. Response using FVDs with C=15 MN/(m/s)$^{1/4}$ and n=1/4

CONCLUSIONS

Arch bridges construction has reappeared around the world. Nevertheless, an arch rib is a structural element inherently non-ductile; in this way, the use of FVDs for energy dissipation is proposed by the writers. The results of a first parametric study have been presented in this work, showing the advantages of using FVDs in the reduction of the seismic response of an arch bridge subjected to a near-field ground motion. First conclusions can be drawn, although more study is needed:

- Viscous devices are suitable for paraseismic applications in arch bridges. FVDs with a force-velocity relationship ($F=C\cdot V^{n}$) with lower n values are the most effective ones for energy dissipation and displacement control.

- Axial forces and bending moments on arch springings and main piers base can be reduced by adding FVDs. This suggests that it is a good option for seismic retrofit of existing arch bridges.

ACKNOWLEDGEMENT

This paper represents a portion of the doctoral dissertation of José de Jesús Alvarez at Technical University of Catalonia; his stay and study has been supported by Universidad Michoacana de San Nicolás de Hidalgo and by the Ministry of Public Education from Mexico.

REFERENCES

1. Kuranishi S, Nakajima A. "Strength characteristics of steel arch bridges subjected to longitudinal acceleration". Structural Engineering/Earthquake Engineering 1986, Japan Society of Civil Engineers; 3(2): 287-295.
2. Dusseau RA, Wen RK. "Seismic responses of deck-type arch bridges". Earthquake Engineering and Structural Dynamics 1989; 18: 701-715.
3. Nazmy AS. "Earthquake-response characteristics of long-span arch bridges". Proceedings of the 11th World Conference on Earthquake Engineering, Acapulco, Mexico. Paper no. 1309. Oxford: Pergamon, 1996.
4. Torkamani MAM, Lee HE. "Dynamic behavior of steel deck tension-tied arch bridges to seismic excitation". Journal of Bridge Engineering 2002; 7(1): 57-67.
5. McCallen D, Noble Ch, Hoehler M. *The seismic response of concrete arch bridges: with focus on the Bixby Creek Bridge*. California: Lawrence Livermore National Laboratory, 1999.
6. Kawashima K, Mizoguti A. "Seismic response of a reinforced concrete arch bridge". Proceedings of the 12th World Conference on Earthquake Engineering, Auckland, New Zealand. Paper no. 1824. Silverstream: NZSEE, 2000.
7. Sakai J, Kawashima K. "Seismic response of a reinforced concrete arch bridge taking account of variation of axial force". Proceedings of the 12th European Conference on Earthquake Engineering, London, United Kingdom. Paper No. 252. Oxford: Elsevier, 2002.
8. Alvarez JJ, Aparicio AC. "Comportamiento sísmico de puentes arco. Un panorama del estado del conocimiento" ["Seismic behavior of arch bridges. A state-of-the-knowledge overview"]. Revista de Ingeniería Sísmica, Mexico, 2003; 68: 21-53 (in spanish).
9. Priestley MJN, Seible F, Calvi GM. *Seismic design and retrofit of bridges*. New York: Wiley, 1996.
10. Computers and Structures, Inc. (CSI), *SAP2000, Integrated Finite Element Analysis and Design of Structures*. Version N7.1, 1997.
11. Wenzel H. "Monitoring the big bridges across the Danube in Austria". Proceedings of IABSE Symposium on Long-Span and High-Rise Structures, Kobe, Japan, 1998. 79: 881-886.
12. Arici Y, Mosalam KM. "System identification of instrumented bridge systems". Proceedings of the 12th European Conference on Earthquake Engineering, London, United Kingdom. Paper No. 541. Oxford: Elsevier, 2002.
13. Soong TT, Dargush GF. *Passive energy dissipation systems in structural engineering*. New York: Wiley, 1997.
14. Murphy TP, Collins KR. "Retrofit of suspension bridges subjected to long period earthquake motion". Proceedings of the 12th World Conference on Earthquake Engineering, Auckland, New Zealand. Paper no. 2310. Silverstream: NZSEE, 2000.