DESING OF 10 FOOTBRIDGES IN NEW HIGHWAY “EXPRESS PASS OF CUERNAVACA”, MEXICO

Alberto PATRON

CEO
Consultora Mexicana de Ingeniería
Mexico City, Mexico

apatrons@gmail.com

Clemente POON

Director
Dirección General de Carreteras, SCT.
Mexico City, Mexico

cpoon@sct.gob.mx

Agustín MELO

Adjoin Director
Dirección General de Carreteras, SCT.
Mexico City, Mexico

amelo@sct.gob.mx

Ernesto MORALES

Civil Engineer
Consultora Mexicana de Ingeniería
Mexico City, Mexico

ing.emf@gmail.com

Eduardo REYES

Technical Director

Consultora Mexicana de Ingeniería
Mexico City, Mexico

eduardoreyesg@gmail.com

Summary

The Cuernavaca city is located 80 KM, south of Mexico City. The highway Mexico-Acapulco pass through the Cuernavaca city, and traffic jams are very important, particularly during weekends and holydays. For that reason the Ministry of Transports of Mexico (*Secretaría de Comunicaciones y Transportes*) started a program to expand the highway from 4 to 10 circulation lanes in the 14.5 KM of the highway zone that crosses the city of Cuernavaca. In that context 10 footbridges, placed over the highway were demolished and replaced with new ones with spans from 37 to 47 m. The bridges are arch type in steel. This type of structure was selected for aesthetic and structural reasons. Due to the high flexibility of the bridges, the dynamic behavior under pedestrian loading was studied and modifications to the original design were made in order to avoid unacceptable vibrations. The bridges are also placed in a high seismic region, and time history calculations considering non-linear behavior of the concrete piers were made in order to evaluate the dynamic response of the bridges under strong earthquakes. This paper presents the main issues of the design process of the bridges, and some important results concerning the dynamic and seismic design of them.

Keywords: Aesthetics; dynamics; vertical vibration; seismic design.

1. Introduction

The Cuernavaca City is located 80 KM, south of Mexico City. The highway Mexico-Acapulco was constructed on the 1960’s. Since then the city has experienced an important growth, and at the present time the Mexico-Acapulco highway crosses practically the center of Cuernavaca City. During holydays and weekends the traffic jams are very important. In that context the *Secretaria de Comunicaciones y Transportes* (Mexico´s Ministry of Transports) decided to expand the highway from 4 to 10 circulation lanes, were the 4 central lanes are confined and are intended for long itineraries (without exits for Cuernavaca) the other 6 circulation lanes are for local traffic. The total length of the Express Pass of Cuernavaca is 14.5 Km, and the width of the highway passed from 21.0 m to 36.0 m. Both sides of the highway are densely populated. For this reason several bridges had to be replaced by new ones with larger spans. This was also the case for 10 footbridges distributed along the 14.5 Km of the highway.

2. Design of the bridges

The design of the bridges presented several challenges: on one hand, due to the important width of the new highway the spans of the bridges will be relatively important, on the other hand, the city of Cuernavaca is located in an area of high seismicity, and finally a pleasant aesthetic for them was required.

Fig.1. Location of the bridges site

2.1 Description of the bridges

The 10 footbridges have spans comprised between 37 and 47 m. The summary of the main characteristics of the bridges is presented on Table 1.

Bridge	Localization(Km)	Span (m)	Height (m)
1	80+020	37.0	9.0
2	82+790	47.0	11.0
3	84+850	38.0	8.0
4	86+200	47.0	8.5
5	86+780	47.0	8.2
6	89+685	41.0	8.8
7	90+485	39.0	8.6
8	91+150	47.0	7.0
9	91+600	38.0	7.9
10	93+920	44.0	10.3

Table 1.Summary of bridge dimensions

The bridges are steel arches, with reinforced concrete deck (Figure 2). The main arches are composed by two tube arch structures, joined by transverse tube elements. The main arches are inclined transversally for architectural reasons. The load of the concrete pedestrian way is transmitted to the arches by means of tubular elements (hangers); it was decided to give a slight inclination in longitudinal direction to these elements also for architectural reasons. The foundations and piers are both in reinforced concrete.

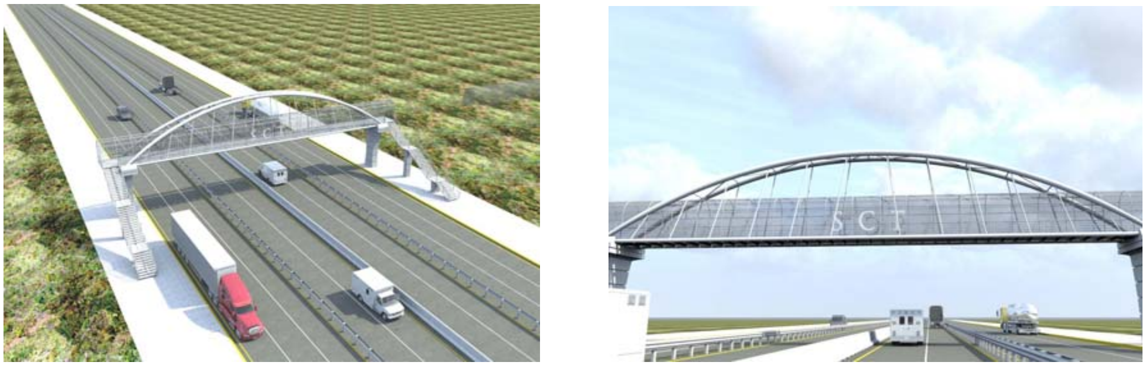


Fig. 2. Render images of a typical footbridge.

2.2 Seismicity of bridges site

The Cuernavaca city is placed in a high seismic risk zone, 250 Km north of Acapulco, near the area where the “Cocos” plate is subducted into the “North American” plate, originating earthquakes of great magnitude. The ground along the road is very variable and in the places of the bridges it goes from relatively firm ground to basaltic rock. The maximum seismic coefficient on this zone is 0.75 g and design spectrum is shown in Figure 3. The seismic forces are the most important action for the foundations and piers design.

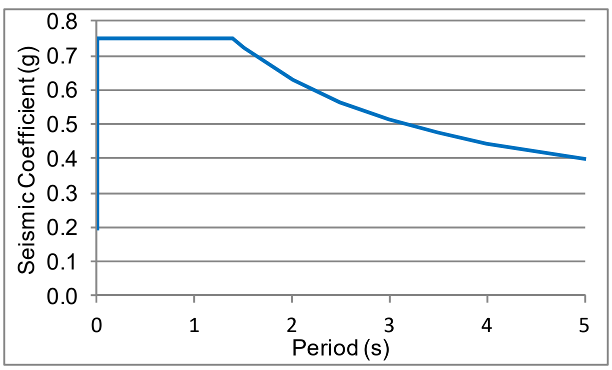


Fig. 3.Design spectra for the bridges

The preliminary designs of bridges were made using the modal spectral method, but the results were very conservative. In order to optimize de foundations and piers design, a time-history non-linear method was used to evaluate the seismic forces on those elements. For that reason a set of (5) “synthetic” accelerograms were generated from the design spectra [1].

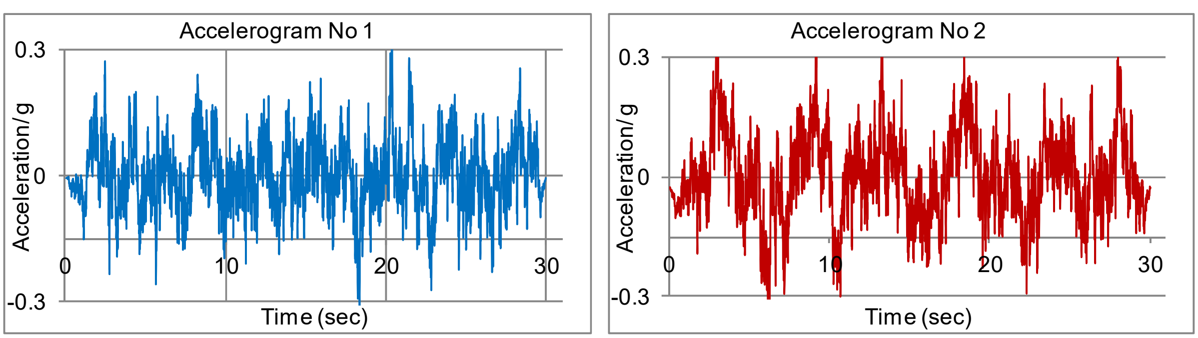


Fig.4. Examples of synthetic accelerograms

Figure 4 shows an example of the accelerograms obtained. A response spectra was calculated from the synthetic accelerograms in order to verify their agreement with the target spectra. These results are shown on Figure 5.

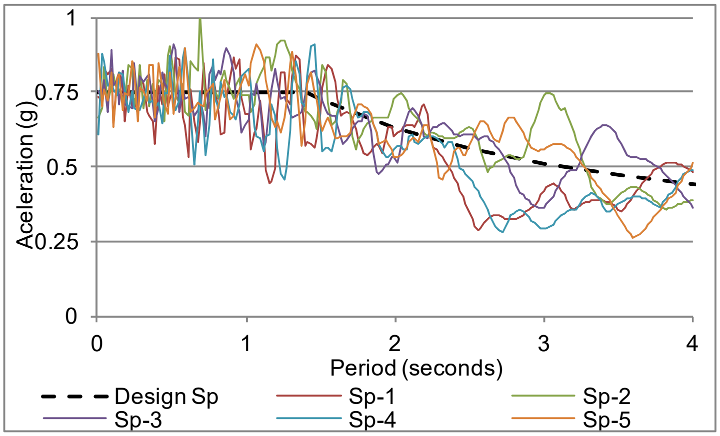


Fig.5. Verification of the compatibility of the accelerograms with source spectra

Seismic design

The design method for the foundations and piers was the following:

I. Elaboration of FEM model of the bridge (see Figure 6).

II. Modal spectral calculation of the seismic response to evaluate de zones were de plastic hinges on piers will develop.

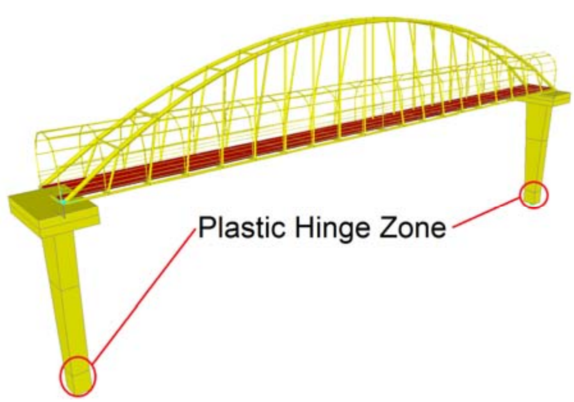
III. Calculation of Moment-Curvature diagrams of the piers sections were the plastic hinges will appear.

IV. Introduction of the plastic hinges in the FEM of the bridge.

V. Time-history non-linear seismic analysis of the response of the bridge for the set of 5 different simulated earthquakes (synthetic accelerograms).

VI. Verification of the resistance of the reinforce concrete sections of the piers,

VII. Verification of the foundations.



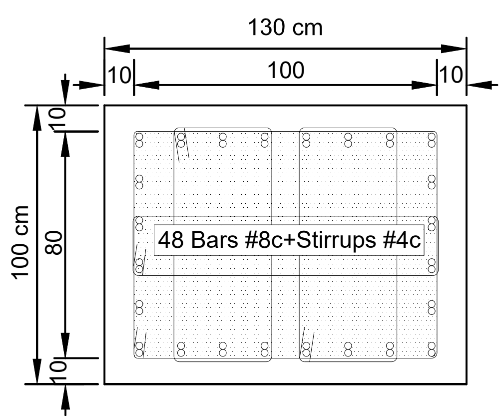
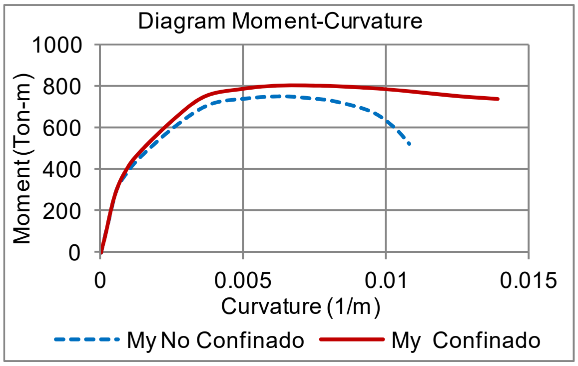


Fig. 6.FEM model of one bridge

Fig. 7 .Typical reinforcement of the piers at their base

Figures 7 and 8 shown the typical reinforcement of the piers at their base (were the seismic moments are maximum), and the corresponding moment curvature diagram. The results presented in Figure 8 show the importance of the transverse reinforcement of the pier for confinement in the area of the plastic hinge [2]. The idealized plastic hinge, considered in the FEM model is represented on Figure 9.



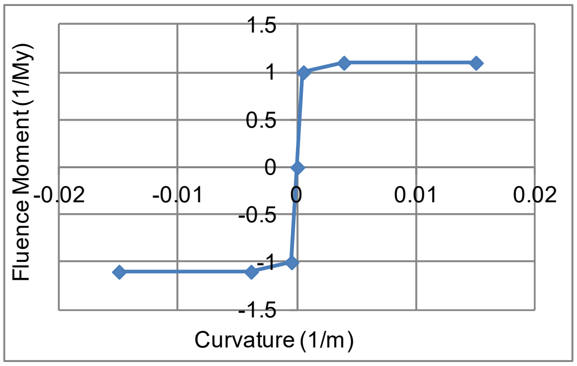
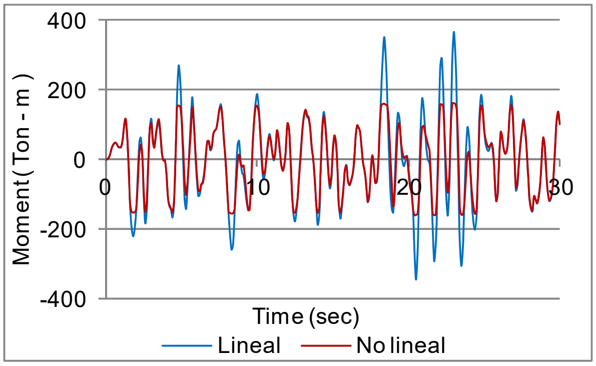


Fig.8. Calculated moment-curvature-diagram

Fig.9. Idealized moment curvature diagram on FEM

The mains results of the non-linear FEM seismic calculations are shown on figure 10, on these figures the time history of longitudinal moments at the base piers during two of the simulated earthquakes are plotted and compared with the results of the corresponding linear FEM calculations. The results shown an important reduction of the response due to the dissipation of energy originated by the non-linear behavior of the plastic hinges at base of piers.



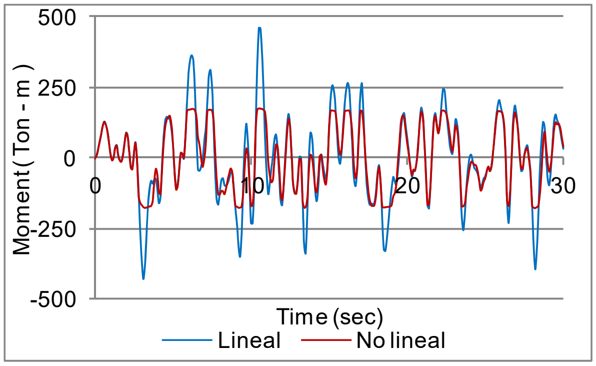


Fig.10. Time history response of longitudinal moments during 2 simulated earthquakes

The verification of the resistance of the reinforced concrete sections at the base of piers is illustrated on Figure 11, by the comparison of the interaction diagram and the calculated history of transverse and the history of longitudinal moments at the base of pier. The elastic response of the structure during simulated earthquakes overpasses the resistance of the section. Nevertheless, considering the non-linear behavior of the bridge, due to the formation of plastic hinges at their base, the response of the structure (moments) is under the resistance of the section, and consequently the proposed reinforcement is considered correct.

It is important to note that the preceding results are based on the assumption of the formation of the plastic hinge at the base of piers. In order guarantee this assumption the transverse confinement reinforcement of concrete core of sections on the plastic hinges zones was calculated to meet requirements seismic design codes [3].

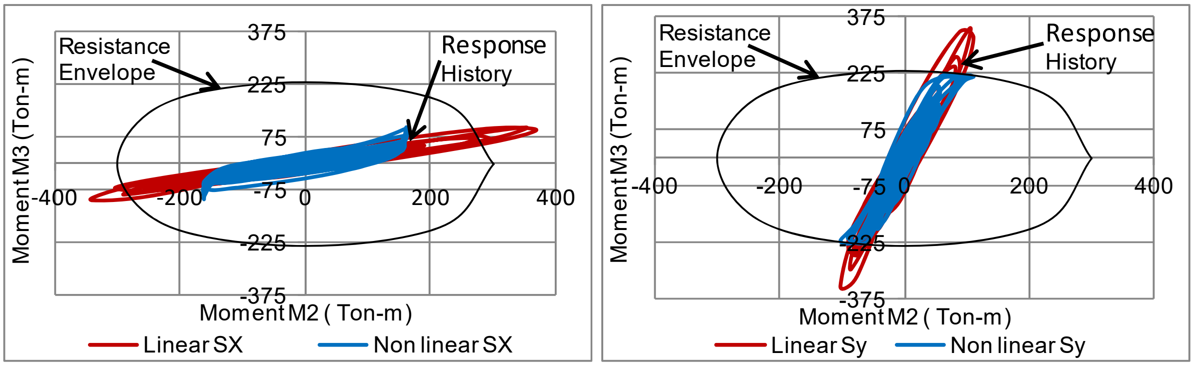


Fig.11. Interaction diagrams of the section at base of piers for transverse a longitudinal seismic load conditions respectively

2.3 Verification of human induced vibrations

The actions induced by pedestrians on footbridges may produce vibrational phenomena. In general, this effect does not have adverse effects on the structure, although the users may fell some discomfort. In the case of the 10 footbridges presented in this paper, the verification of the comfort of user of the bridges under vibration was made following the methodology recommended by SETRA [3]. The first transverse and vertical frequencies of vibration of the bridges were evaluated by FEM calculations. These frequencies were compared with the values indicated in the tables of the reference were the risk of resonance (with pedestrians) is defined as a function of the range of vibratory frequencies of the bridge. As expected the bridges with greater spans were the most sensitive to the vibratory effects of pedestrians.

Tables 2 and 3 shows the main vibrational periods of a 47.0 m bridge span considering different bridge equipment. The first vibration modes of the completed bridge are shown on Figure 12. Since the bridges traverses over an important highway an anti-vandalism cage was proposed in all the bridges. The structure that supports the anti-vandalism cage is formed by steel pipes integrated to the concrete slab of the bridges. The dimensions and thickness of the steel pipes that conforms the main structure of the anti-vandalism cage were defined in order to increase the stiffness of the bridges, and consequently increase the main vibration frequencies of the bridges deck.

Case	Configuration	Frequency (Hz)	Mode type
1	Steel arches with concrete slab only	1.34	First Vertical mode
2	Steel arches with concrete slab only	1.81	Second Vertical mode
3	Steel arches with concrete slab+ antivandalism “cage”	5.38	First Vertical mode
4	Steel arches with concrete slab+ antivandalism “cage”	11.9	Second Vertical mode

Table 2.Summary of main vertical vibration frequencies

During erection the bridges, the construction workers reported that the bridges were very sensitive to pedestrian induced vibrations. Some acceleration measurements were made during the construction stage, and the results of the Tables 2 and 3 were confirmed: the integration of the anti-vandalism cage significantly increases the vibration frequencies of the briges.

Case	Configuration	Frequency (Hz)	Mode type
5	Steel arches with concrete slab only	2.22	First transversal mode
6	Steel arches with concrete slab only	2.83	Second transversal mode
7	Steel arches with concrete slab+ antivandalism “cage”	2.73	First transversal mode
8	Steel arches with concrete slab+ antivandalism “cage”	3.48	Second transversal mode

Table 3.Summary of main transversal vibration frequencies

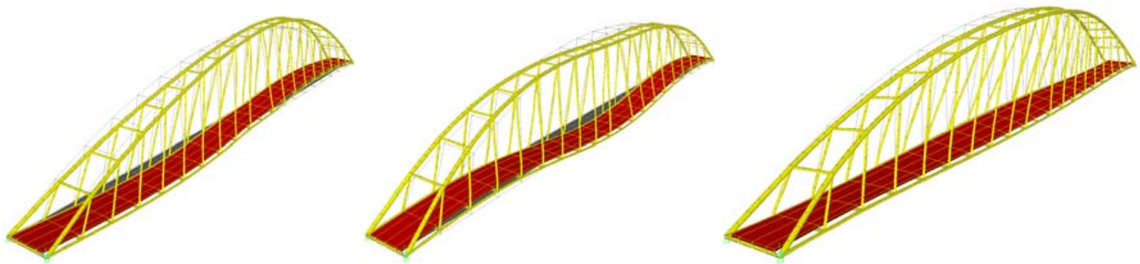
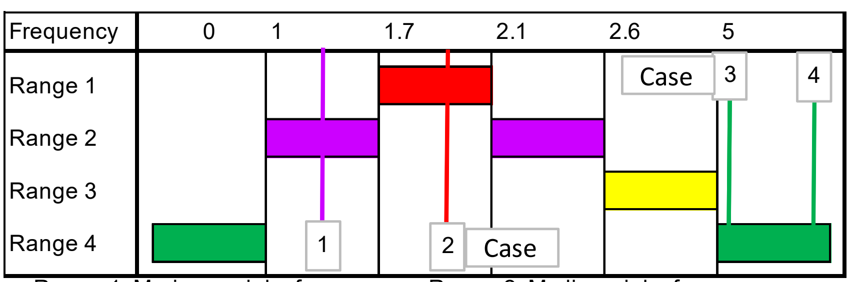


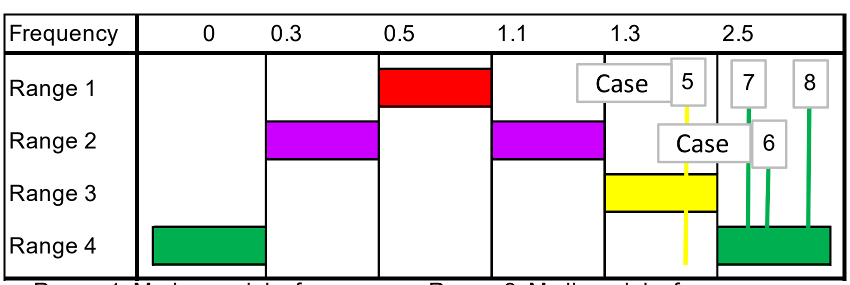
Fig.12. Shapes of the first vibration modes

The figures 13 and 14, show the comparison of the different vibration frequencies of the bridges with the different “resonance risk” zones proposed on the SETRA´s technical guide [3]. As stated earlier the structure without anti-vandalism cage is very flexible and sensitive to human induced vibrations. The incorporation of the anti-vandalism structure increases vibration frequencies of the bridge´s deck, and consequently the risk of vibrations induced by pedestrians becomes negligible.



Range 1: Maximum risk of resonance, Range 2: Medium risk of resonance,
Range 3: Low risk of resonance, Range 4: Negligible risk of resonance.

Fig.13.*Frequency ranges (Hz) of vertical vibrations*



Range 1: Maximum risk of resonance, Range 2: Medium risk of resonance,
Range 3: Low risk of resonance, Range 4: Negligible risk of resonance.

Fig.14.*Frequency ranges (Hz) of transverse vibrations*

3. Conclusions

This article presented the methodology followed for the design of 10 footbridges located along the new highway that crosses the city of Cuernavaca. The bridges have major spans and important heights, and are located in an area of high seismicity. The methodology for the seismic design of the structures was presented. This methodology is based on the modeling of the nonlinear behavior of the structures and the realization of FEM time history non-linear calculations. This allowed the optimization of the design of piers and foundations.

On the other hand, as the bridges have important spans, they are sensitive to pedestrian-induced vibrations. Finite element calculations were performed to evaluate the main vibration frequencies (lateral and vertical) of the bridges, and these values were compared with those recommended by some standards, and the risk pathological vibrations was established. These results showed the importance of stiffening the bridges by means of the integration of the structure of the vandalism cage to the main structure.

Fig. 15. Aerial view of the "Express Pass"

Fig.16. General view of one of the bridges during erection

4. References

[1] Gasparini et al., *SIMKE; A program for artificial motion generation*. Report of the Massachussets Institue of Technology, Boston, USA, 1976.

[2] Wallace, J.W., *A computer for analysis of reinforced concrete and masonry sections*. Repport of the of Clarkson University, Postdam New York, USA, 1992.

[3] Priestley, Seible and Calvi, *Seismic design and retrofit of bridges*. John Wiley Editor, USA.,1996.

[4] SETRA., Technical guide; Assessment of vibrational behavior of footbridges under pedestrian loading, SETRA, Paris, France 2006.