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SEISMIC DESIGN OF BRIDGES

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Seismic isolation is recognized as one of the most efficient design strategies to reduce seismic demand and potential damage on bridge structures. Three seismic bridge case studies are presented and compared herein. A different seismic structural design approach has been adopted for each case. In the first case study, lead core elastomeric bearings (LCEB) are used to isolate the structure and provide additional damping to reduce seismic forces. In the second case study, a balanced cantilever bridge with a monolithic connection between the superstructure and the substructure is analyzed by using a displacement-based design and considering a cracked section for the piers. The third case study depicts the seismic design of a cable-stayed bridge with a 330 m main span and towers reaching up to 142 m in height, in which fluid viscous dampers have been designed as main elements to transmit and minimize the impact from seismic forces.

Introduction

The three case studies presented represent three different bridges designed by Pedelta in Colombia and recently completed. Colombia has high seismic hazard zones located mainly on the Pacific and Andina regions of the country where more than two-thirds of the national population live. Therefore, the main urban areas of the country and the highway network are in permanently exposed to potentially severe earthquakes. This hazardous natural condition makes seismic isolation a viable alternative to be considered in the design of structures and specifically bridges [1]. Bridge designers can either reduce the seismic demand by isolating the bridge or design the structure to behave in a ductile way under large seismic events without seismic isolators. Both design approaches are accepted by bridge design codes, but have a different cost in construction, maintenance and repair after an earthquake. When seismic isolators are used, no damages are expected in the structure under the seismic design event.

Example 1. Lead core elastomeric bearings (LCEB)

In this first example, a short span bridge is isolated by using lead core elastomeric bearings. Lead core elastomeric bearings are seismic isolators with a high energy dissipation capacity. The LCEB consists of a steel reinforced elastomeric bearing with a central lead core that deforms plastically under shear forces, dissipating the seismic energy. In addition, due to the elasticity of the elastomer, the structure gets back to the initial position after the seismic event.

Road 44 bridge is located in Colombia, close to the Pacific Ocean shoreline, within the Pacific and Nazca subduction area. This is an area of high seismicity, with a ground seismic acceleration of 0.25 g, for a return period of 475 years that is equivalent to an exceedance probability of 10% in 50 years. The bridge is 120 m in length with a main span of 35 m. The deck is a post-tensioned concrete voided slab 1.6 m deep and 12.4 m wide constructed span-by-span (*Figure 1*).

Seismic isolation reduces the earthquake demand by modifying the structural dynamic response, typically by increasing the oscillation period, leading to a substantial reduction of seismic forces. This is achieved by means of devices with very low stiffness called seismic isolators, which are usually placed between the superstructure and the substructure [2].

Table 1 summarizes the seismic force reduction achieved in five concrete bridges located in Colombia using different types of seismic isolators.

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View of the Gazapa balanced cantilever bridge.

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Figure 1. Typical cross section.

Table 1. Examples of bridges designed by Pedelta where elastomeric bearings have been used as seismic isolators.

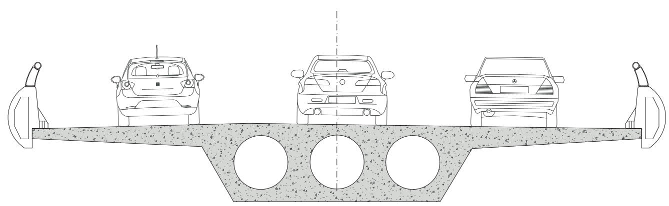

Figure 1

Bridge name and year	***Isolator type***	***Seismic force reduction due to bearing flexibility***	***Seismic force reduction due to hysteretic cycle (%)***
Portachuelo Bridge (2009)	Elastomeric	60%	–
Redoma San Mateo (2007)	Elastomeric	45%	–
PR-13 Bridges (2015)	Elastomeric, lead core	44%	34%
El Rosal Bridge (2012)	Elastomeric, high dumping rubber	30%	40%
Road 44 Bridges (2011)	Elastomeric, lead core	25%	34%

Table 1

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Figure 2. Comparison between design accelerations spectrum for 465 years return period and 5% damping. Local seismic hazard assessment values are higer and are shown as round points.

Figure 3. Comparison between design periods and accelerations for no-isolated configuration and use of LCEB.

Figure 4. View of the bridge elevation.

Figure 5. Comparison between piers reinforcement ratios obtained with the forces and displacements design methods.

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Figure 6. View of the bridge elevation.

Figure 7. Cable-stayed bridge deck cross section.

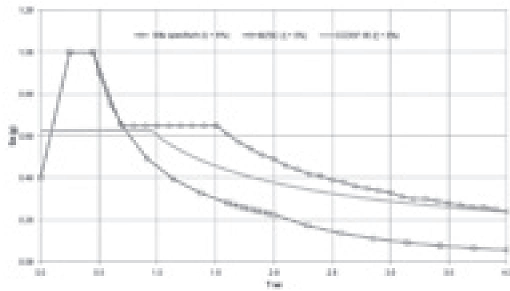

Figure 2

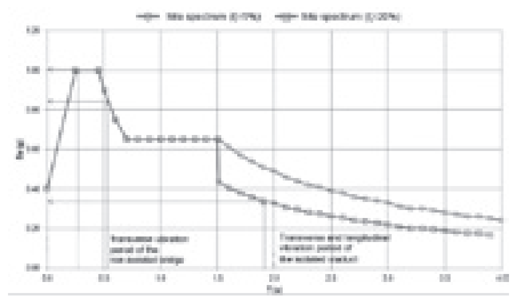

Figure 3

Figure 4

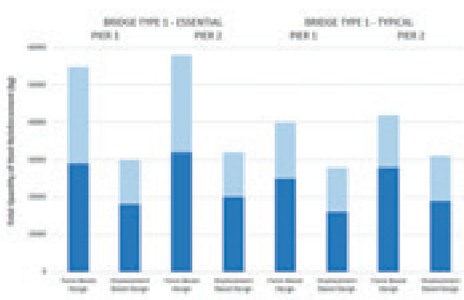

Figure 5

Figure 2 shows the Colombian seismic design code, CCDSP-95, seismic Microzonation and seismic local hazard assessment acceleration response spectrum. The seismic local hazard assessment approach is adopted, which accurately models and reflects the actual bridge behavior during a seismic event. In contrast, the other two examples do not consider the soft ground amplification effects properly nor the bridge response for periods larger than 1 second.

By using LCEB, vibration periods were modified from 0.5 seconds to roughly 2.0 seconds. The seismic force is reduced 60% compared to a non-isolated bridge. Even though this reduction can also be achieved by means of plastic hinges, the use of seismic isolators prevents structural damage to the bridge during a large earthquake. Therefore, a reduction of the pier reinforcement quantity is achieved. Both advantages lead to a cost-effective bridge, which is expected to be in service after an earthquake.

Figure 3 shows a comparison between design periods and accelerations for a non-isolated configuration and for using LCEB. The calculation process is well known and considers the actual pier stiffness including concrete cracking [3]. Since the structural behavior is non-linear, the design process is iterative [2].

Example 2. Ductile substructure with a monolithic connection between superstructure and substructure

This example illustrates the seismic design approach of a cast-in-place concrete bridge built in balance cantilever. *Figure 4* shows a general view of this bridge, which has three spans 61.75 m + 125 m + 61.75 m with an overall length of 250 m. The superstructure is a concrete box 10.3 m wide and variable in depth. The concrete piers, founded with caissons, are g resistance of the piers. Two design methods have been considered – force-based design and displacement-based design.

Seismic codes are now shifting from the force-based design to displacement design for the following reasons:
i) Actual structural behavior or bridge working point. The working point is the intersection of the displacement demand and response demand, which are equal in the acceleration spectrum vs. displacement diagram,
ii) Force-based design does not consider the actual pier reinforcement nor the lack of simultaneity between hinges in different piers.

Some examples of codes considering displacement-based design approaches are AASHTO Guide Specifications for LRFD Seismic Bridge Design [4] and AASHTO Guide Specifications for Seismic Isolation Design [5], even though AASHTO Seismic Bridge Design limits the use of this method to ordinary bridges. Complex bridges must be evaluated by means of a time-history analysis. The analysis approach for the displacement-based design is using a multi-modal linear analysis for bridges in seismic hazard zones A, B, and C, and a pushover analysis for bridges located in seismic hazard zones D.

A comparison between a force-based (AASHTO LRFD Bridge Design Specifications [6]) and displacement-based (AASHTO Guide Specifications for LRFD Seismic Design [4]) seismic analysis has been performed. In the force-based analysis, the substructure cracked stiffness is considered, approximate 50% of gross stiffness. The adopted force design reduction factor is 2, so elastic response forces are also reduced to 50%. In the displacement-based analysis, a multi-modal analysis has been conducted and the demand displacement lower than the response displacement condition. A pushover analysis has been carried out with different reinforcement. Cost savings in the pier reinforcement quantities have been achieved with the displacement-based design compared to the force-based design (*Figure 5)*. The plastic hinges length has a significant influence in the response movements evaluation. This length is not well known for rectangular hollow piers, but it is well established for other typical sections including solid circular and rectangular piers. Further study is needed [3].

Example 3. Cable-stayed bridge with seismic dampers

Hisgaura bridge referred to earlier in this publication is a cable-stayed bridge in the province of Santander, Colombia.

Figure 6 shows the bridge elevation which is a 653 m long continuous bridge with 5 spans in total (36.5 m + 36.5 m +125 m + 330 m +125 m, including two approach spans). The main span of 330 m crosses the Hisgaura Creek at a height of more than 70 m above ground. The highest tower is 148 m tall.

Figure 7 shows the cable-stayed superstructure cross section, which consists of two 1.4 m deep post-tensioned concrete edge girders and 250 mm reinforced concrete slab over transverse beams spaced 5 m. The approach

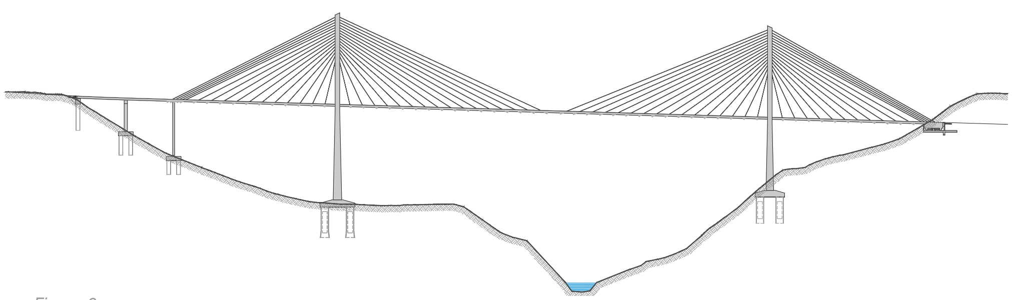

Figure 6

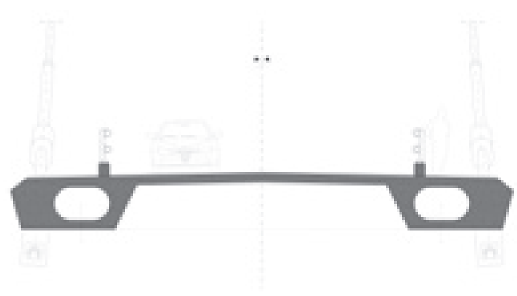

Figure 7

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Figure 8. Viscous damper: force-velocity relation for different α parameter.

Figure 9. Viscous damper hysteretic force-displacement diagram under seismic accelerogram (α = 0.1).

Figure 10. Longitudinal seismic displacement without viscous damper reaching up to 853 mm.

Figure 11. Long. seismic displacement with viscous damper reaching up to 387 mm.

Table 2. Comparison between deck displacements and bending moments at pier 2 and pylons using fixed connection and using viscous dampers.

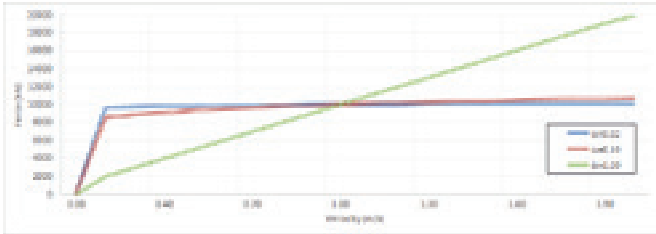

Figure 8

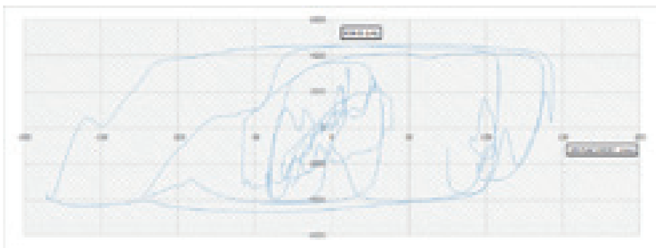

Figure 9

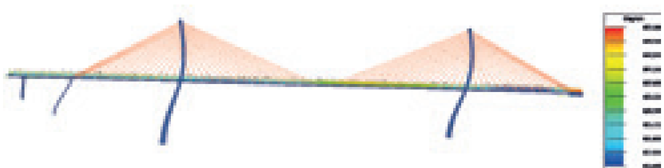

Figure 10

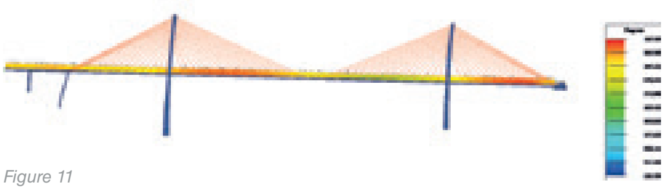

Figure 11

Type of connection between deck and pylons	***Deck longitudinal displcmt. (mm)***	***Max. bending moment Pier 2 (kN*m)***	***Max. bending moment Pylon 1 (kN*m)***	***Max. bending moment Pylon 2 (kN*m)***
Deck fixed at pylons	690	69,500	330,000	390,000
Viscous damper at abutment 2	400	40.000	100,000	70,000
Reduction by using viscous dampers	42%	42%	70%	82%

Table 2

spans have a multi-cell post-tensioned cross-section, which has the same outer shape as the cable-stayed span cross-section.

At the towers, the deck is free to move vertically and longitudinally, being restrained only in transverse direction. In the longitudinal direction, the deck is fixed at one of the piers (pier 2), which serves also as an anchorage point for the retaining stay cables. Pier 2 becomes a critical point of the structure, so it is able to resist a high structural demand and be flexible enough to accommodate the deck's longitudinal movements. This pier is vertically post-tensioned. Many studies can be found in technical literature to assess the impact of the analysis type and the structural configuration in the seismic response of a cable-stayed bridge [7,8], but the site-specific conditions and geometry are the most relevant parameters and a specific analysis is always required.

The south abutment also allocates four 2,500 kN capacity fluid viscous damper units (with a design stroke of +/-500 mm) connecting the deck and the abutment. These devices allow free displacements in the longitudinal direction to slow movements, while providing damping to dissipate earthquake energy and displacement control under fast movement (i.e. seismic event).

The Hisgaura bridge is located in a high seismic region and close to an active seismic fault. The seismic design uses a site-specific acceleration response spectrum and a series of time histories ground motions. To reduce the seismic demand in the longitudinal direction, the bridge is seismically protected with fluid viscous dampers that dissipate over 30% of the energy induced by a seismic event and control the longitudinal deck displacement under a predefined target value. In the transverse direction, the deck is laterally restrained at the supports with shear keys. To ensure a fully functional bridge and to enable opening the bridge immediately following an earthquake, towers, piers and foundations are designed to remain elastic.

The seismic demand was determined with two different methods:
i) an iterative analysis by using a multimodal linear spectral analysis with a modified design response spectrum to account for the additional damping provided by isolators, and
ii) a non- linear time-history analysis by using site-specific acceleration inputs.

The bridge is classified as "essential". Due to its proximity to an active seismic fault a local response study was required. This provides a design response spectrum based on actual seismic event data for the region. Several configurations were studied in a first iteration. The first configuration consisted of fixing the deck at the pylons, which resulted in high demands on the foundations during the seismic event that made this option not feasible. Another studied configuration incorporated elastomeric bearings as seismic isolators, which allows a small damping, but it leads to excessive deck longitudinal displacements and demands on the pier 2. The selected design configuration uses viscous dampers.

Viscous dampers provide a high dissipation of energy. These devices consist of two chambers and a silicone fluid that is forced through an orifice due to a pressure difference between the chambers. During this action, the seismic energy is transformed into heat and dissipated into the atmosphere. The equation characterizing the behavior of these devices is $F=Cv^{\alpha}$, where "v" is the velocity, "C" is the damping constant that characterizes the output force, and "α" is the velocity exponent that characterizes the non-linear behavior of the device. For α =1.0 the output force increases linearly with the velocity, and for a small α (e.g. α =0.02) the output force is approximately constant.

Figure 8 shows different types of behavior of viscous dampers for different values of coefficient α. For slow loads (including wind), the damper behaves passively and no force is transmitted on the abutment. The force-displacement hysteretic behavior of the dampers from the seismic time-history analysis of the Hisgaura Bridge is shown in *Figure 9*.

Figures 10 and 11 respectively show the demands of the configuration fixed at pylons without a damper and free at pylons with damper. *Table 2* summarizes the reduction in longitudinal displacement and the bending demand at piers during the seismic event using the viscous dampers.

With the viscous dampers, an effective damping of 50% was achieved. Several codes such as the Eurocode 8 [9,10] and the AASHTO Guide Specifications LRFD for Seismic Bridge Design [4] indicate that when high damping is used as a seismic isolation of the structure, a response spectrum analysis with an effective stiffness and modified response spectrum may not properly represent the effect of isolation on the response of the structure. Therefore, once the design of the structure was completed, a non-linear seismic time-history analysis was performed as a verification. The design of the structure was carried out with a response spectrum analysis by using a modified spectrum and an equivalent stiffness of the damper device. The use of viscous dampers has shown to be an effective way of reducing the forces induced by the deck on the substructure during a seismic event. The results obtained with the non-linear time-history analysis were found similar to the ones obtained with the adapted linear response spectrum analysis.

Conclusions

Three case studies of different seismic bridge design strategies have been shown in this article, which includes the use of LCEB, the substructure design using displacement-based approach, and the use of fluid viscous dampers. Each strategy has its own advantages. Depending on the overall bridge length, pier height, main span length, and other constraints and conditions, the designer should assess the most appropriate design approach. Site seismic analysis and the use of advanced analysis methods like time-history or pushover analyses are strongly recommended to optimize and to better estimate the actual structural response to seismic events ●

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References:
[1] Féderation Internationale du Béton (fib), Seismic bridge design and retrofit – structural solutions, Lausanne, 2007.
[2] F. Naeim, J.M. Kelly, Design of Seismic Isolated Structures: From Theory to Practice, John Wiley and Sons, New York, 1999.
[3] M.J.N. Priestley, F. Seible, G.M. Calvi, Seismic design and retrofit of bridges, John Wiley and Sons, New York, 1996.
[4] American Association of State Highway and Transportation Officials, AASHTO Guide Specifications for LRFD Seismic Bridge Design, 2nd Edition, Washington D.C., 2011.
[5] American Association of State Highway and Transportation Officials, Guide Specifications for Seismic Isolation Design, 3th Edition, Washington D.C., 2010.
[6] American Association of State Highway and Transportation Officials, AASHTO LRFD Bridge Design Specification, 7th Edition, Washington D.C., 2014.
[7] A. Camara, M.A. Astiz, Typological study of the elastic seismic behaviour of cable-stayed bridges: 8th European Conference on Structural Dynamics (EURODYN 2011), Leuven, Belgium, 2011.
[8] G. Valdebenito, Passive seismic protection of cable-stayed bridges applying fluid viscous dampers under strong motion, Barcelona, 2009.
[9] European Committee for Standardization, EUROCODE 8: Design of structures for Earthquake resistance – Part 1: General rules, seismic actions and rules for buildings, EN 1998-1:2004/AC:2009, Brussels, 2009.
[10] European Committee for Standardization, EUROCODE 8: Design of structures for Earthquake resistance – Part 2: Bridges. EN 1998-1:2005/AC:2009/AC:2010, Brussels, 2010.