Effect of the type of connection used between the deck and the piers on seismic response of extradosed bridges

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SUMMARY:
This paper presents the influence of the type of connection used between deck and piers, piers height, and the seismic hazard of the area where a bridge is built, on the seismic response of an extradosed bridge with a main span length of 100 m. For the first parameter, a monolithic connection and a simple supported scheme were studied. The seismic load, defined in accordance with the Colombian Bridge Design Code, consists of earthquake-response spectra for three different regions with high, intermediate and low seismic hazard. Results show that a monolithic connection is more suitable for regions with low seismic hazard. For the other two regions, due to better deck behaviour when this is supported on the piers, the supported scheme is preferred.

Keywords: Extradosed Bridge, seismic response, deck-pier connection, height of the piers.

1. INTRODUCTION

The Extradosed bridge is an established structural type in Asian countries like Japan, China and South Korea, where the seismic activity is an aspect of concern. In Europe and The Americas, its use has been increasing with good acceptance, see Fig. 1. This fact points to a future application in areas with a wide range of seismic risk which demands studies in order to understand its seismic response.

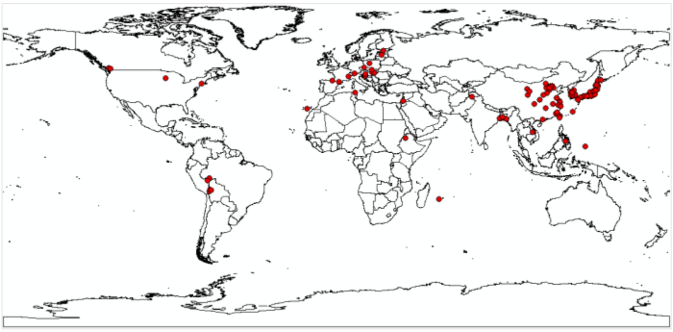

Figure 1.1. Global distribution of extradosed bridges (constructed, in construction and in phase project)

As in other structural bridge types, the seismic behaviour of the extradosed bridge depends on several factors like the type of earthquake input, mass and stiffness distribution, type of supports, connection between the deck and the piers, among others. For the last parameter mentioned, Chio (2000) y Meiis (2007) studied the bridge behaviour under static loads, finding that monolithic connections are favourable. For the severe seismic conditions of Japan, Otsuka et al. (2002) analyzed the behaviour of extradosed bridges with main span lengths of 150, 200 and 250 m. Despite the fact that the authors included the deck-pier connection as a variable, they did not show results regarding this parameter because the study was mainly focused on comparing construction costs with cable-stayed bridges and PC-box girder bridges.

This work presents the influence of the type of connection used between the girder and the piers, the height of the piers and the seismic hazard of the area where the bridge is built on seismic response of an extradosed bridge with a total length of 220 m. This kind of preliminary studies is helpful to understand the seismic behavior of the bridge and will establish a basis for future definition of design criteria. The paper describes the characteristics of the parametric study and the results for main structural elements. Finally, based on results and a shallow calculation of quantity of materials, recommendations regarding the type of connection between the deck and piers, depending on seismic hazard, are provided.

2. PARAMETRIC STUDY

2.1. Geometry and Numerical Modelling

Geometric characteristics of the bridge were defined from the design criteria presented in Benjumea, Chio, and Maldonado (2010). The bridge has a main span length (L) of 100 meters and two side spans of 60 m, see Fig 2.1. The deck consists of a single cell box section with constant height of 2.9 meters, see Fig 2.2. The height of the towers is L/10, half the required for a cable-stayed bridge. The piers are supposed to be fixed at the base. At the top, two types of connection were studied: a scheme with the deck supported on the piers using POT-bearings (which establish a pinned connection) and a monolithic connection. Through this paper, these patterns are named *A* and *M* respectively. The heights of the piers (H_p) studied are L/4, 3/8L and L/2. At abutments, the bridge is supported on roller-supports, although for bridges constructed in high seismic areas and with a piers height equal to 50 meters, because of excessive longitudinal displacements, it was necessary to adopt elastomeric supports. The elastomeric stiffness was chosen so that longitudinal displacements for the bridges were under 10 cm.

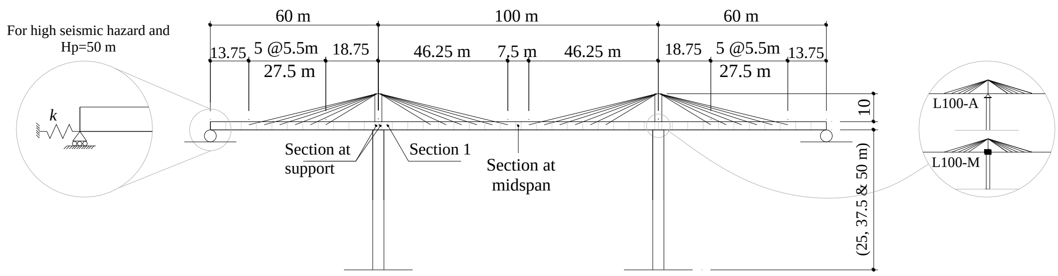

Figure 2.1. Longitudinal view of the bridge

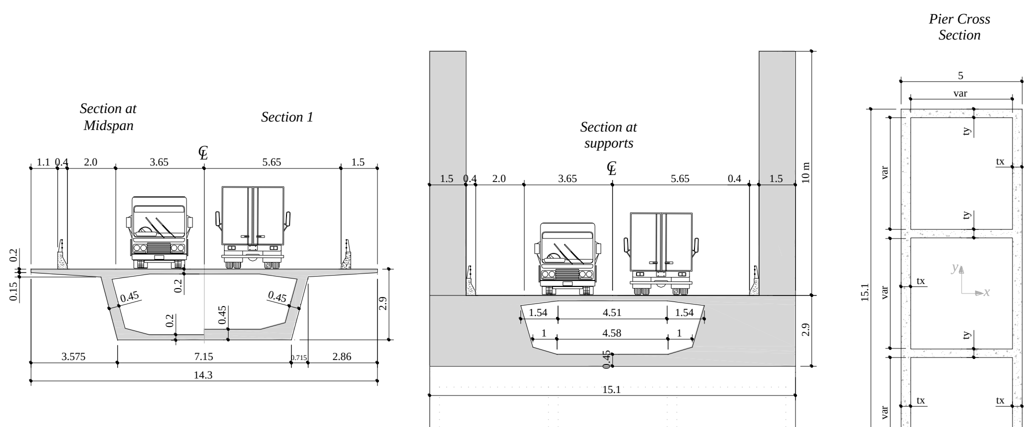

Figure 2.2. Cross section of deck and pier

Numerical models of the bridge were developed and studied by means of linear static and dynamic analysis using the finite element software SAP2000Advanced v.14.2.4. Beam-column elements were used for modelling the deck, towers and piers while cable elements were considered for extradosed cables. Force transmission between extradosed cables and the deck, and between the deck and the piers is achieved through rigid-link elements, see Fig. 2.3. Extradosed bridges with deck supported on piers (pattern *A*) were modelled by releasing rotational DOF at the top of piers. For the modal analysis the eigenvalue problem was solved on the utilization of the stiffness matrix of the bridge in the dead-load deformed state as proposed by Abdel-Ghaffar & Nazmy (1991). A total of 200 modes were considered. These modes are able to represent more than 80% of the total modal mass. In all cases a constant damping value of 2% was used for all modes. This value is slightly higher than that reported by Niihara et. al (2001). To obviate the effects of construction, it has been assumed that the bridge is built on formwork in one step. Finally, Table 2.1 summarizes the characteristics of the bridges studied.

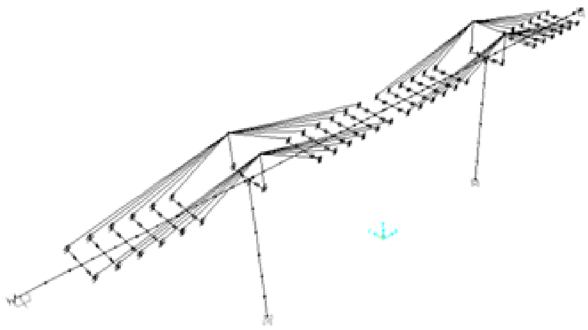

Figure 2.3. Finite element model of the extradosed bridge studied

Table 2.1. Carachteristics of bridges studied

Bridge	Deck-Pier Connection	Height of the piers
L100-Hp25-M	(M) Monolithic	25 m
L100-Hp37.5-M		37.5 m
L100-Hp50-M		50 m
L100-Hp25-A	(A) Deck Supported on Piers	25 m
L100-Hp37.5-A		37.5 m
L100-Hp50-A		50 m

2.2. Materials and Extradosed Cable Design

The mechanical properties of the concrete used for the deck, towers and piers are: f'c = 39.2 MPa, a elastic modulus $E_{c,28}$=2.55x10^4 MPa and a self-weight γ= 23.5 kN/m^3. Stress limits for concrete are presented in Table 1. It is important to point out that the maximum allowable tension stress of concrete has not been limited to a null value; therefore the amount of internal prestress calculated may reflect lower values than those for projects using this type of bridges. However, this does not affect the conclusions as we have used the same design criteria for all bridges studied.

Table 2.1. Stress limits for concrete

	Without seismic load	With seismic load
Compression Stress	15.69 MPa (0.4f'c)	23.54 MPa (0.6f'c)
Tension Stress	3.12 MPa	3.90 MPa

For the extradosed cables, a 0.6" steel strand with an ultimate tensile strength f_{pu}=1860 MPa, Es 1.999x10^5 MPa and γ=77.14 kN/m^3 was considered. For the design of cables, the procedure described by Dos Santos (2006) has been used. Fatigue verification in service limit state and allowable stress in the ultimate limit state has been made in accordance with the provisions of the SETRA (2001) design guide-line.

2.3. Loads

The loads are defined for highway bridges in the Colombian Code Seismic Design of Bridges (AIS, 1995). Considered load cases are: the dead load (D), which takes into account self-weight of the elements and a permanent load (barriers and asphalt carpet) of 16.6 kN/m, the prestress forces on extradosed cables (P) and internal prestress (Pi), the traffic load (L), which consists of a uniform load of 11.4 kN/m plus a floating load of 120 kN per lane (four load hypotheses were assumed), and seismic load (EQ), which consists of three design response spectra for three different zones with coefficients of acceleration of 0.05, 0.15 and 0.30, corresponding to areas of low, intermediate and high seismic hazard respectively, see Figure 2.4. These spectra are associated with a soil profile type S2 with a site coefficient equal to 1.2.

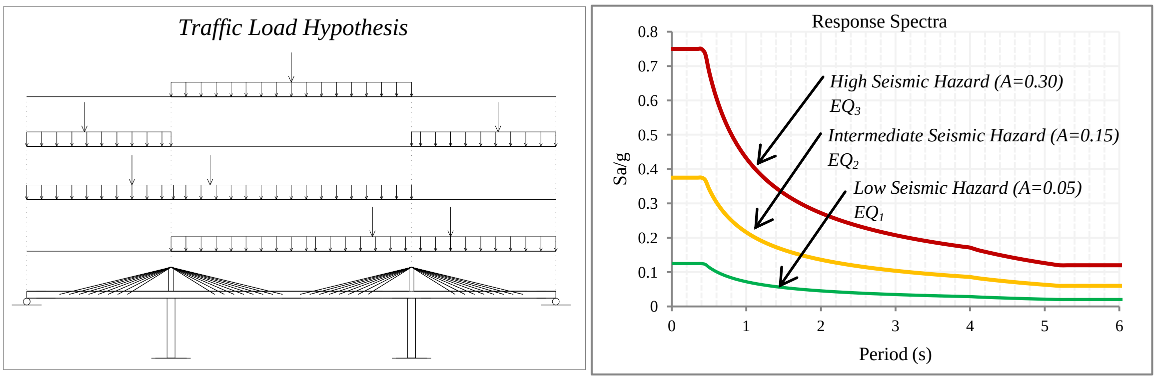

Figure 2.4. Traffic load hypothesis (left) and response spectra (right)

Load combinations for service limit states (SLS) and ultimate limit states (ULS) analyzed are presented in Table 2.2. In all cases for the ELU, the forces in extradosed cables and internal prestress were not factored with the same coefficient of dead load; according to Mermigas (2008) this method is more reasonable for bridges with a rigid deck. Actions such as wind, temperature, and the sudden breakage of extradosed cables were not analyzed, since the primary objective was to study the effects of seismic actions.

Table 2.2. Load Combinations

Combination	SLS	ULS
IA	D + (L) +P + Pi	1.3[β_DD + 2.2 (L)] + P + Pi
IB	D +P + Pi	1.3[β_DD] +P + Pi
VII	D + EQ +P + Pi	1.0[β_DD] + EQ/R + P + Pi
β_D=1.0 and 0.75; R= 3 for flexo-compression design of piers; R=1 for shear design		

3. RESULTS

3.1. Dynamic Properties

Modal shapes corresponding to fundamental periods are shown in Figure 3.1. It can be noted that in bridges with short piers, the first modal shape corresponds to vertical movements due to flexure of the central span of the deck, while after increasing the pier height the modal shape corresponds to an overall longitudinal displacement of the bridge. Figure 3.1 allows us to examine the effect of the type of connection between the deck and the piers: when the deck is supported on piers, there is a reduction in the global stiffness of the bridge. As a result, the periods of vibrations will increase and thus a decrease in the modal acceleration values will be achieved, a favorable effect for bridges constructed in sites with a rock or hard soil profile (Priestley, Seible, & Calvi, 1996).

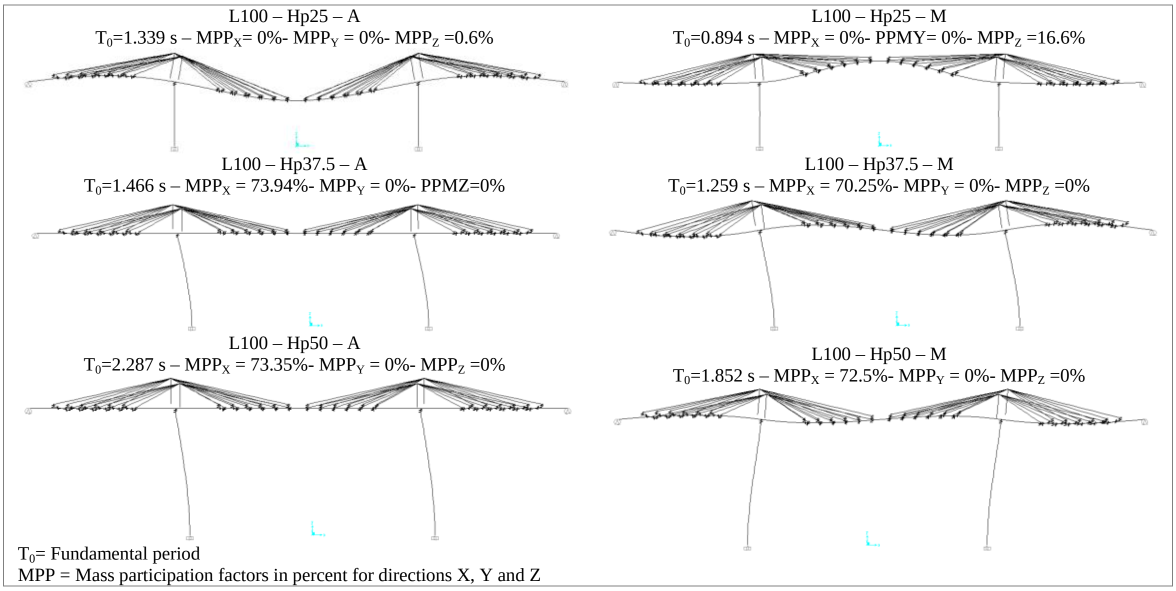

Figure 3.1. Modal shapes corresponding to fundamental periods

3.2. Cable Behaviour

Although the stress oscillations in cables due to earthquakes can become larger than those of the traffic load, see Fig. 3.2, the fatigue problem will be controlled by the latter, since the frequency of occurrence of large magnitude earthquakes is low and therefore the accumulated damage, if any, would be much lower. For this reason, it is clear that the fatigue problem in extradosed cables will be controlled by traffic load, regardless of the seismic hazard of the area where the bridge is built. For bridges with an A-type pattern, the stress change due to traffic load in cables ($\Delta\sigma_L$) is about 55 MPa, whereas an M-type bridge scheme presents stress changes on the order of 44 MPa. This fact obliges the design engineer to reduce cable presolicitation for bridges with A pattern, conducing to a less efficient use of material. Regardless of the seismic zone and the height of the piers, the total weight of extradosed cables for bridges with M-patterns remained constant (228.5 kN) while in bridges with A-patterns an increase in seismic hazard and the height of the piers obliged to increment the area of the cables located near the tower because of higher stress in those. As a result, bridges with A-patterns constructed in high seismic zones exhibited a total extradosed cable weight of 239.7 kN.

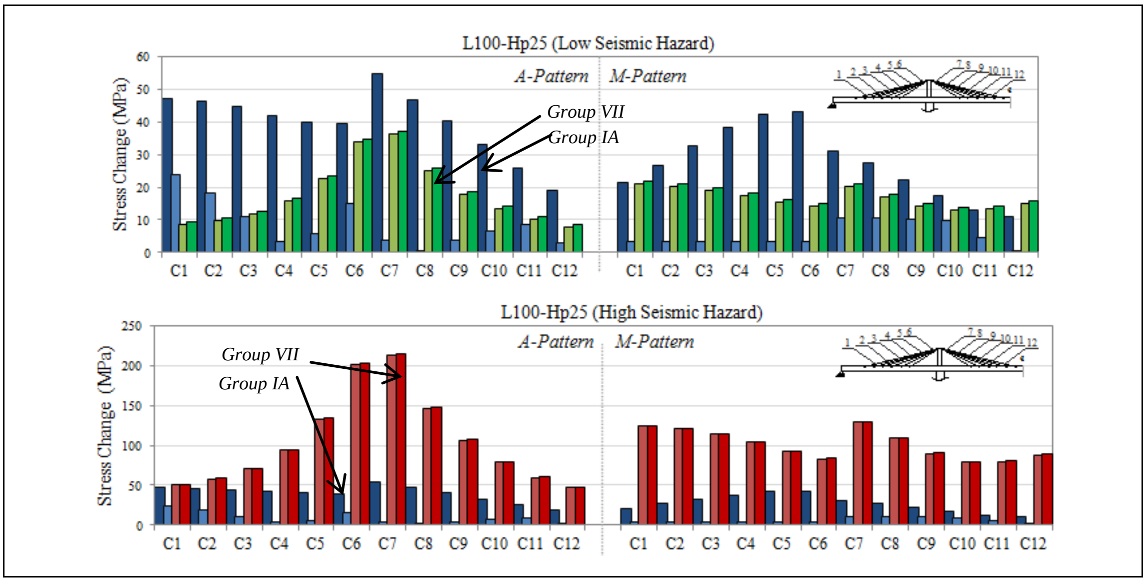

Figure 3.2. Comparison of stress change in cables due to traffic and earthquake loads

3.3. Deck Behaviour

From Figure 3.3, which is representative of all bridges studied, it can be concluded that regardless of the type of connection between the deck and the piers, in zones with low seismic hazard the traffic load will govern the maximum deflections in the deck. For that zone, bridges with A-patterns exhibit higher deflections values. However, in zones with high seismic hazard, bridges with M-patterns show greater seismic deflections than those for A-patterns. This effect results from the transmission of bending moments in the deck-pier connection joint for bridges with monolithic connections. This behavior is similar to that found by Tuladhar and Dilger (1999) in their studies for cable-stayed bridges.

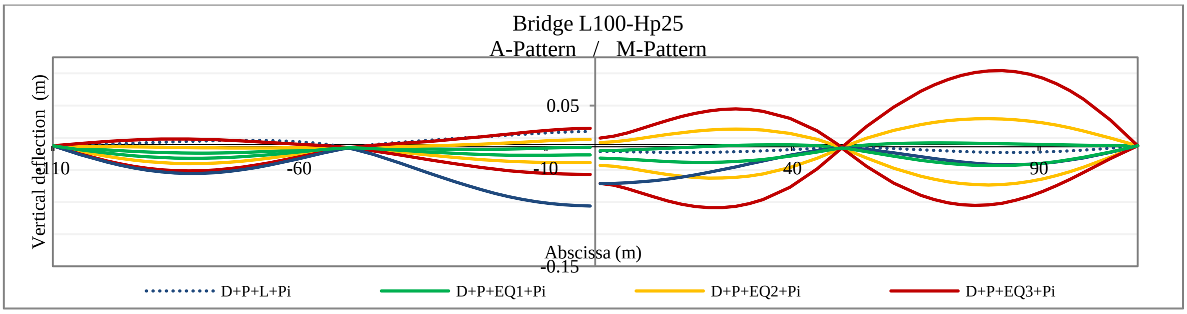

Figure 3.3. Comparison of vertical deflection in function of seismic hazard and deck-piers connection pattern

Figure 3.4 illustrates longitudinal bending moments and axial forces for bridges L100-Hp37.5-A and L100-Hp37.5-M, which are representative for the remaining bridges. From that figure, it can be observed that in areas of low seismic hazard, the bending moments are governed by the traffic load, being lower for the M-pattern. When the bridge is projected on a zone with high or intermediate seismic hazard, the monolithic connection introduces seismic bending moments of great magnitude, which are 1.5 to 2 times higher than those that occur on bridges A-pattern.

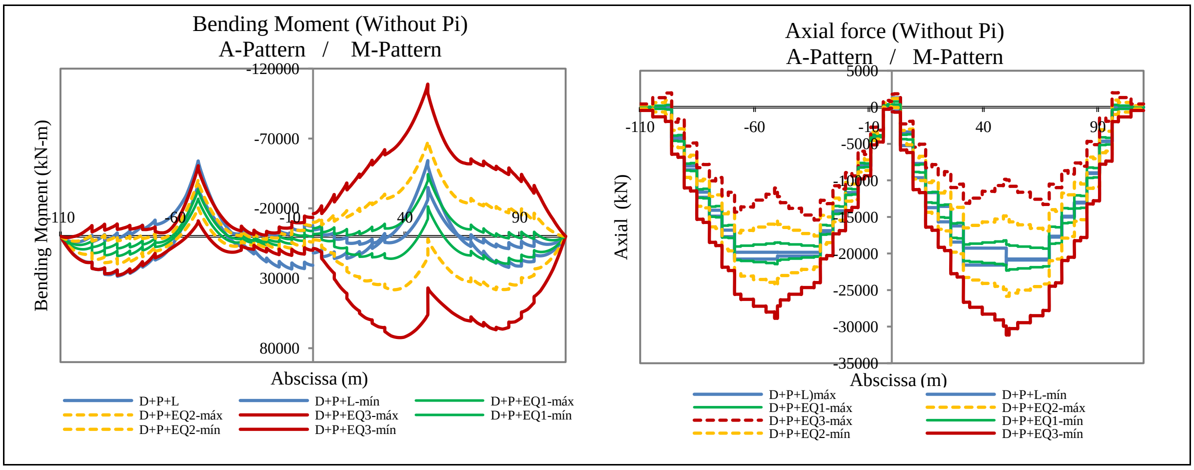

Figure 3.4. Comparison of deck behaviour in function of seismic hazard and deck-piers connection pattern

If we analyze the maximum bending moment in terms of seismic hazard and the ratio Hp/L, see Fig. 3.5, it can be concluded that for traffic loads, regardless of the type of connection used between the deck and the piers, the influence of the height of the piers in the behavior of the superstructure is not significant. However, for seismic actions and M-pattern bridges, an increase in the height of the piers produces an increase in the bending moments on supports and central span, an effect that does not take place in A-pattern bridges.

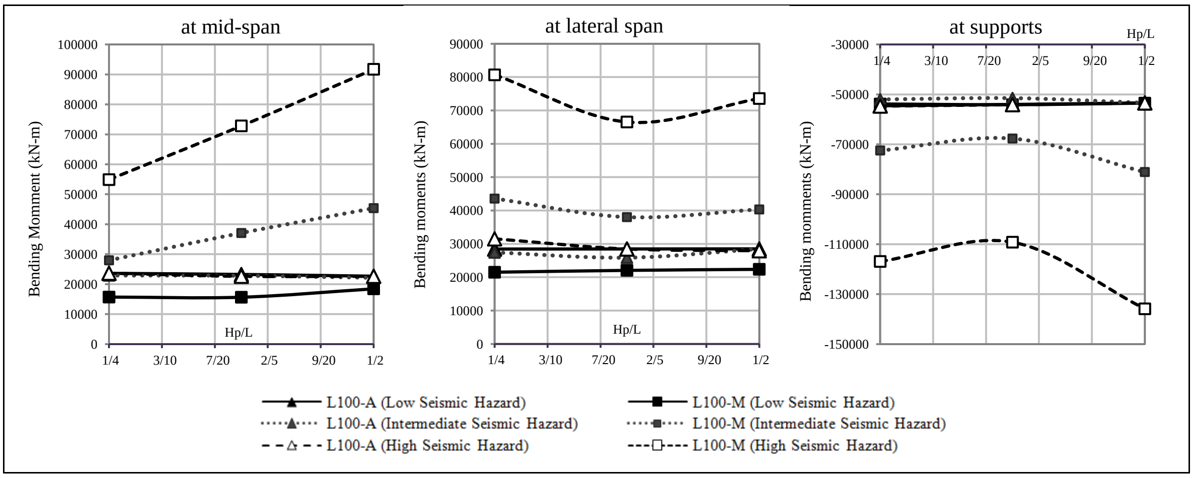

Figure 3.5. Maximum values of bending moment on the deck in function of ratio Hp/L, seismic hazard and deck-piers connection pattern

3.4. Piers Behaviour

Typical bending moment diagrams, shear forces and deflection of the piers for both types of connections are plotted in Figure 3.6. This figure allows us to observe that in bridges with A-patterns, seismic actions will always govern the design of these elements, unlike M-pattern bridges, in which the design will be governed by traffic load in zones with low seismic hazard, and by the earthquake load in zones with intermediate and high seismic hazard. For both types of connections studied, the maximum longitudinal deflections occur for earthquake loads, being greater for A-pattern bridges because of a higher flexibility.

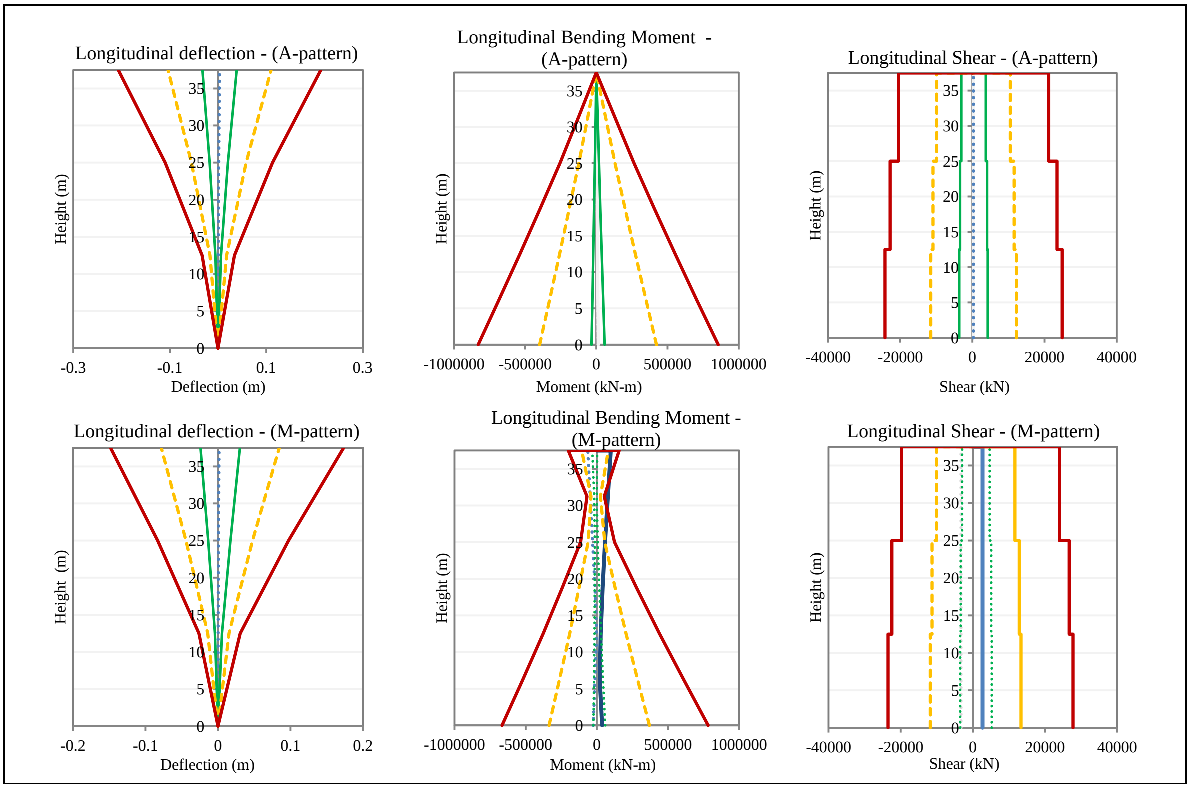

Figure 3.6. Comparison of piers behaviour in function of seismic hazard

When comparing the maximum internal forces on the piers, see Fig. 3.7, it is observed that the longitudinal bending moment remains constant for low and intermediate seismic hazard zones, regardless of the height of piers. However, in zones with high seismic hazard, an increase in piers height produces an increment in bending moments. Regarding shear forces, an increase in piers height leads to a reduction in shear forces, which is a result of the lower stiffness of the piers. For all bridges studied, shear forces are greater in bridges with M-patterns, except in the case of short piers.

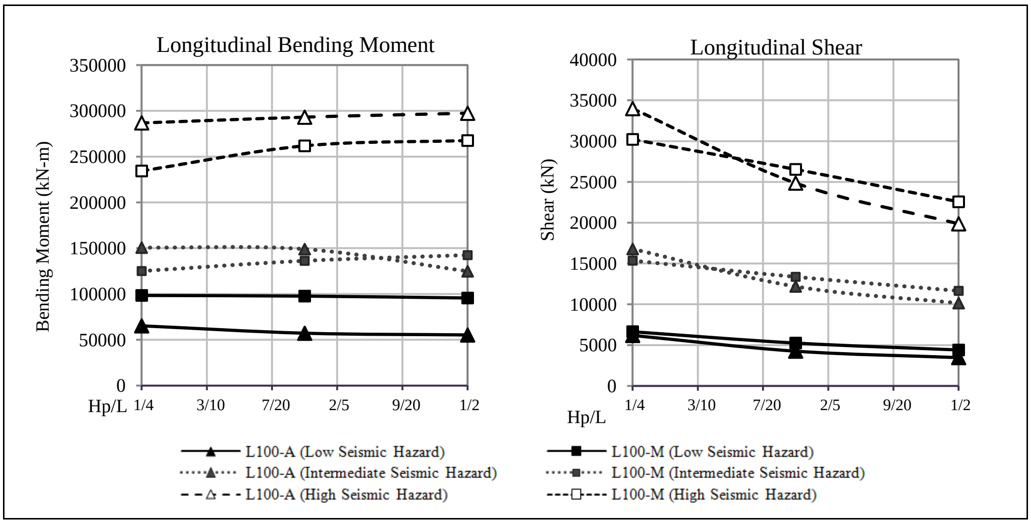

Figure 3.7. Maximum values of bending moment and shear on piers in function of ratio Hp/L, seismic hazard and deck-piers connection pattern

3.5. Quantity of Materials

Table 3.1 presents the total volume of concrete required to build the bridge. This shows that there is no significant effect of the type of connection between the deck and the piers and/or the seismic hazard of the zone. Furthermore, Figure 3.8 illustrates the total amount of steel (active and passive) and the relative percentage difference between the maximum and minimum. This figure offers a glimpse of similar amounts for all bridges built in areas of low seismicity, and some advantage for the A-pattern bridges projected for intermediate and high seismic zones.

With the exception of low seismic hazard zones, where a M-pattern appears more favorable for the structure, these results demonstrate that there is not a type of connection that offers significant advantages compared to the other, and that the particular characteristics of the project will help to decide what type of connection should be used. This result is consistent with the current trend in Extradosed bridges, see Figure 3.9, where a strong parity for both type of connections studied in this paper was found.

Table 3.1. Total Concrete quantity.

	Seismic Hazard		
	Low	Intermediate	High
Model	Vol. (m^3)	Vol. (m^3)	Vol. (m^3)
L100-Hp25-M	2951.77	2951.77	3124.77
L100-Hp37.5-M	3400.77	3400.77	3526.77
L100-Hp50-M	3849.77	3849.77	3849.77
L100-Hp25-A	2951.77	2951.77	3077.77
L100-Hp37.5-A	3400.77	3400.77	3526.02
L100-Hp50-A	3849.77	3849.77	3849.77

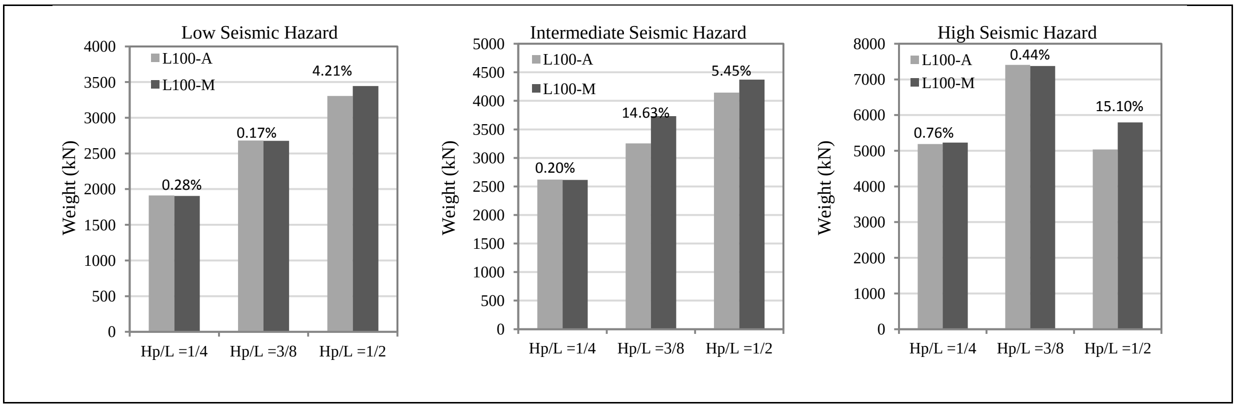

Figure 3.8. Total steel weight as a function of seismic hazard, deck-piers connection patterns and ratio Hp/L

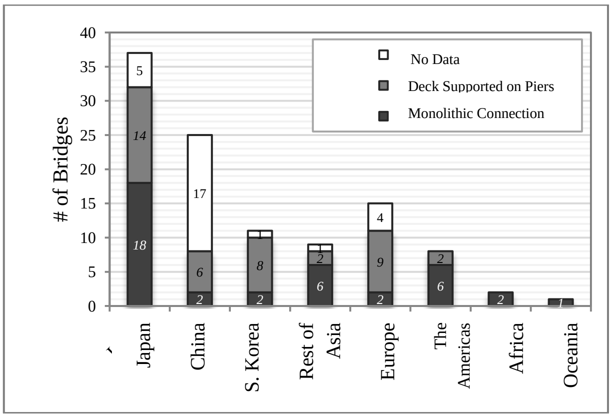

Figure 3.9. Number of bridges constructed and under construction by type of deck-piers connection

4. CONCLUSIONS

A parametric study on the seismic response of an extradosed bridge with main span length of 100 meters and side spans of 60 meters was developed in this paper. The modified parameters are the type of connection between the deck and the piers, the seismic hazard of the area where the bridge is built, and the height of the piers. The main geometric characteristics of the bridge were taken from Benjumea, Chio & Maldonado (2010), the extradosed cables are dimensioned using the method proposed in Dos Santos (2006) and has been verified with the design guide SETRA (2001). The imposed actions are in accordance with Colombian regulations for the design of bridges. The main conclusions of the study are:

- For zones with low seismic hazard, it is advantageous to use a monolithic connection between the deck and piers because this connection improves the performance of the deck and the extradosed cables due to the frame scheme achieved. This connection is reaffirmed as the design of the piers results in very similar values of longitudinal and shear reinforcement compared to the deck-supported-on-piers scheme.

- For areas with intermediate and high seismic hazard, a better performance of the deck when this is supported on the piers is obtained. Nevertheless, both types of connections studied here can be used. When the height of the piers increases, it seems reasonable to use decks supported on those elements, however, only a rigorous study that takes into account the characteristics of the zone, the construction method used, the restriction of the project and actions such as wind and temperature, could help the design engineer to define the type of connection between the deck and piers to be used.

- In bridges with monolithic connections the height of the piers does not affect the structural response of the deck under traffic loads and low earthquakes. This effect occurs because of the relative high stiffness of the deck in extradosed bridges. However, for moderate and high earthquakes, an increase on the piers height induces a rise in the forces on the deck. When the deck is supported on piers, the effect of the piers height is negligible.

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