Seismic Design of cable-stayed bridges.
The South Crossing Bridge in Guayaquil, Ecuador

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SUMMARY:
Seismic design of long-span Cable-Stayed bridges has received a strong input in the latest decades, from both conceptual design and construction point of view, thanks to latest examples of design and realizations of long-span Cable-Stayed bridges in high seismic areas where some of them have already proved the success of the newly adopted conceptual designs. Within the proposed approach, the structural performance is ensured through the adoption of passive dissipation devices together with structural configurations substantially new for such kind of bridges. The confidence gained towards this innovative approach for seismic protection of long-span bridges further promotes the attempt to adopt those challenging structures as means for bridging morphological and economical gaps which for long time have distressed the natural evolution of commercial flows and economical growth of entire countries, particularly of those highly prone to natural hazards such as earthquakes. The paper illustrates the case study of the South Crossing Bridge in Guayaquil, Ecuador, from preliminary evaluations till the final design solutions adopted for its seismic protection.

Keywords: Cable-Stayed Bridges, Seismic Design, Viscous Damper, Pier-to-deck connection

1. INTRODUCTION

Amongst long span cable supported structures, cable-stayed bridges have become increasingly popular in the last decades firstly because of their aesthetics but, more importantly, because of some fundamental advantages over suspension bridges such as more relaxed foundation requirements, absence of anchoring problems and easier constructions techniques. For such kind of reasons cable-stayed bridges are widely recognized as the most economical structures up to 1000 m spans. In the case of the South-Crossing Bridge (SCB), which features a total length of 1248 m, a cable-stayed structure was appointed as the most valuable design solution, characterized by three main spans 416 m long. The SCB however, only represents a part, though the most complex from a structural point o view, of a wider viaduct network with 48 km of highways and additional 2.6 km of bridges. It clearly acts as a very important link within the country’s economical interests since it will become the main access to the sea port located at the south of Guayaquil, which is the largest and most important in Ecuador. The SCB also stands out from other long span cable-stayed bridges because of it structural configuration characterized by unusual pier-tower arrangement (Y shaped) which allows to accommodate two separate roadways 23.7 m wide each, with the tower being located in between (see Fig. 1). Its seismic design strategy furthermore, intentionally retrace the main concepts already applied in some of the latest cable-stayed bridge designs, i.e. the Rion-Antirion bridge in Greece, though carefully re-evaluated and calibrated for the specific purposes.

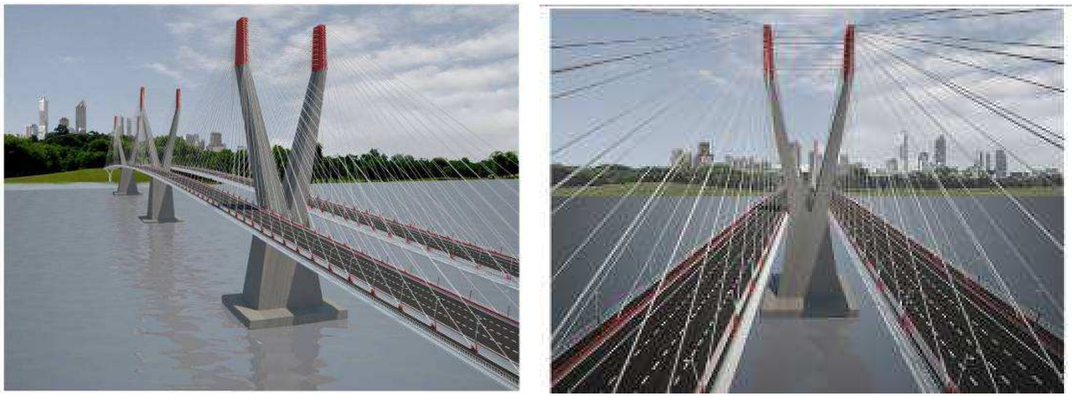


Figure 1. SCB Photo Renderings

2. THE BRIDGE STRUCTURE

2.1. Introduction to the site and the main Bridge location

The SCB spans the Guayas River, in south of Guayaquil, about 250 Km far from the capital of Ecuador, Quito. At the selected location, the river narrows and the distance between the port of Guayaquil and the Durán–Boliche and Boliche–Puerto Inca highways its minimum, as shown in Fig. 2 (a) and (b). The Guayas River is the largest and most important river in the coastal region of Ecuador and it serves the portal area developed on the sloughs around the river.

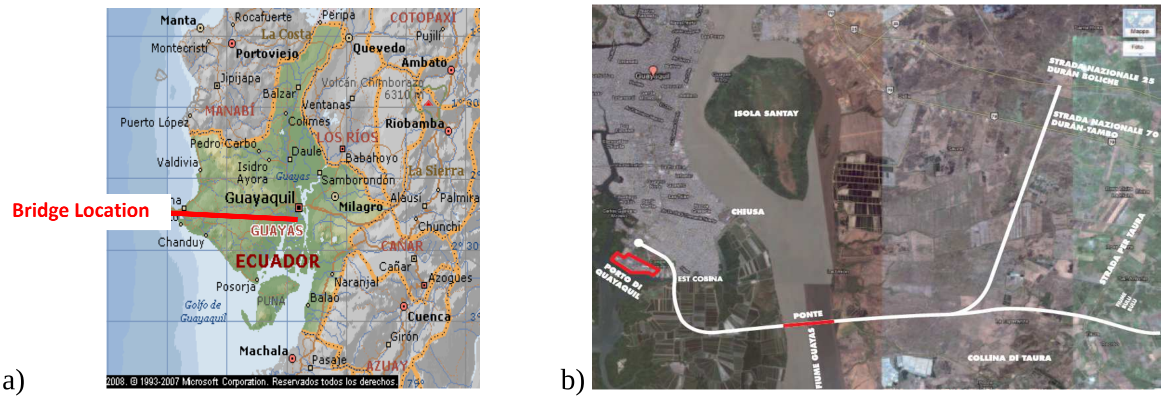
a)  b)

Figure 2. Bridge location (a) – Highway Network for SCB (b).

2.2.Structural configuration

The SCB is a continuous multi-cable-stayed bridge supported by three large pier-tower systems (P1,P2 and P3) resting at the mean sea level, Y shaped and with a double order of cable stays supporting two separate composite decks 23.7 m wide each (see Fig. 3). Drilled piles (48 per Pier) intercepting each of the three piers at the mean sea level from a depth of 80 m were adopted as foundation system. The final span lengths are 208 m and 416 m for the side and the central spans respectively, thus achieving the total length of 1248 m (see Fig. 4). Four concrete towers 78 m tall are built on top of each pier, cantilevering outwards with respect to the pier axis up to 46.6 m maximum spacing. Coupled in groups of two towers, they are connected by a steel-concrete composite tower head which develops further up for 25 m more, thus resulting in a total height of 103 m for the tower system (Fig. 5a).

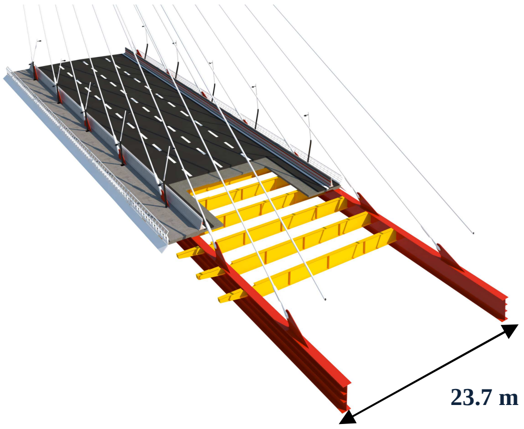
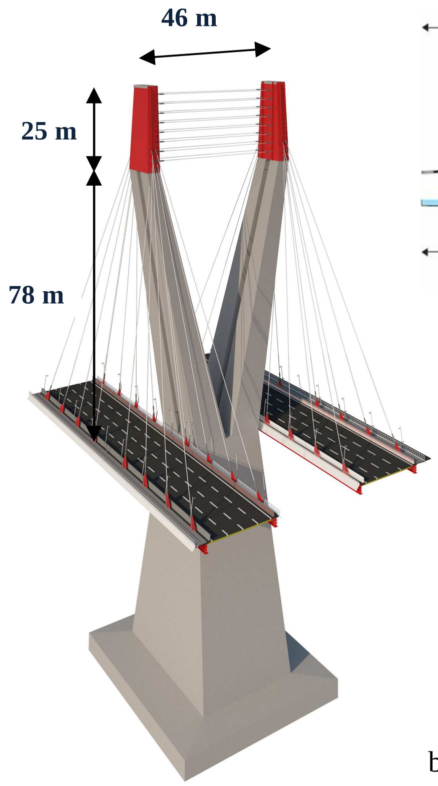


Figure 3. Bridge deck detail

Due to their rectangular box section type, towers provide with the access for anchorage positioning and maintenance for the two deck cable-stays groups, each side of the pier. Four vertical clusters of stays with a semi-fan arrangement and a spacing of 12.6 m (longitudinally on the deck) and 1.4 m (vertically on the tower head), link each deck to the tower head segment. Locked-in-coil cable-stays were selected for the advantages they provide in terms of structural efficiency and corrosion protection (Troitsky et al. 1997). A balanced connection between the two composite tower heads is then provided by 14 high-strength steel cables arranged in a double order of 7 horizontal stays.



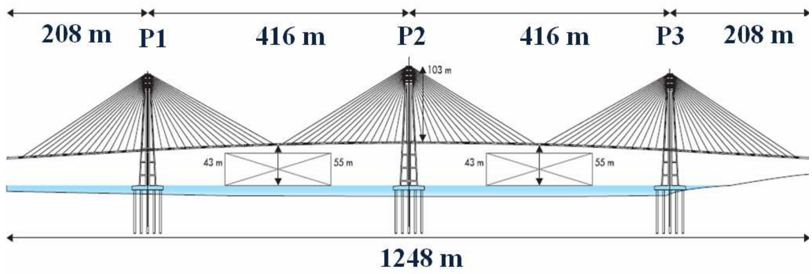
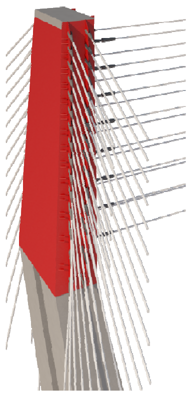


Figure 4. SCB Longitudinal Profile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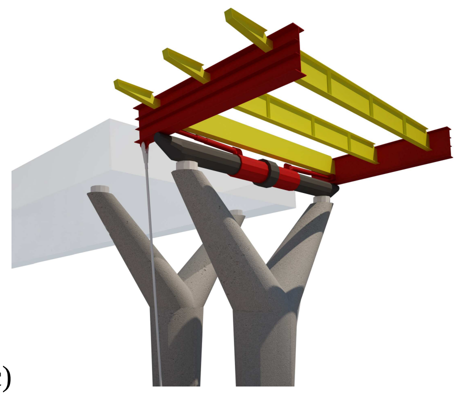

c)

a)

Figure 5. Tower pier system detail a) - Tower Head detail b) - c) Temporary Pier detail

The Bridge has a double deck system 23.7 m wide arranged in single carriageways of 17.6 m width each and 3.5 m wide external walkways for each deck, separated by crash barriers. Such an arrangement results in asymmetric decks, albeit on the whole symmetric if one considers the global transverse elevation of the main bridge. The pier-to-deck connection is realized by coupling rigid fuse elements, fully active during static unbalanced actions due to wind and traffic loads and a system of viscous dampers which become active under seismic loads, after fuses designed failure. For the same purposes other viscous dampers devices coupled with rigid fuse elements are placed at the deck-to-abutment connections, on top of Y shaped temporary piers (Fig. 5c). This arrangement allowed to independently analyze the two structural system composed by approaching viaducts and main bridge structure.

3. DESIGN OF THE SCB

3.1. Guidelines and loads for Static design of the SCB

The global analysis of the SCB under static loads had been carried out with reference to European guidelines (Eurocodes) and in line with the design loads listed in Table 3.1.

Table 1. Structural and non structural permanent loads adopted for design

Load Case	Load	Description
CPS	4.5 KN/m^2	Non Structural Dead loads
Added Furnitures	5.0 KN/m	Load for added furnitures required for normal bridge operational conditions
CP	183 KN/m	Structural Dead Loads.
PMP	9.15KN/m	Increase in 5% of the structural weight.
CPSM	0.2*CPS	Increase in 20% of the nonstructural Dead Loads.
CPMD	0.208 m	Effects of imposed deformations at the deck cantilevering ends to account for inaccuracies during deck construction

The main phases characterizing the definitive design of the SCB can be summarized in the following steps:

- Analysis of the structural behavior and definition of the pre-stressing forces due to the cable stays in order to achieve the theoretical undeformed shape, under the effect of structural and non structural loads;
- Sizing and strength check of the main structural elements (stays, deck, towers, piles and foundations);
- Analysis of the temporary restraining effects on the design, following a staged construction analysis technique;

The adoption of a staged construction analysis allowed to analyze the real evolution of loading and restraining system acting on the structure, starting from three statically determined structures including piers and cantilevering decks, up to the spans closure where the structural continuity and redundancy that characterize all multi-span cable-stayed bridges are realized.

3.2. Traffic loads

The definition of live loads to be considered for global static analysis of the SCB required specific considerations. In fact, when dealing with long span bridges, reference live loads and load patterns are often not specified within standards or considerably different load models are specified from code to code. Before adopting a specific load model for static global analyses a review of four international guidelines was undertaken, namely the French Regulations (Chaier des prescriptions communes – CPC Fascicule No. 61, titre II, 1961), British Standards (BD 37/01), Eurocodes (also with reference to Italian Regulations NTC 2008) and American Standards (AASHTO, 2007). Within all the analyzed standards at least three of them were adopted for design of well known examples of long span bridges in Europe, namely the Normandie Bridge and the Rion-Antirion Bridge in Greece. In order to carry out a consistent comparison of traffic load models for the analyzed guidelines, the following bridge classification has been considered within the study:

- the bridge class is of road type and each of its two distinct carriageways comprises a total running surface of 17.60 m, situated between two safety fences one of which defines the foot-way boundary.
- the design lane pattern comprises a total number of 4 typical lanes and 1 emergency lane, 3,65m and 3m wide respectively, with one foot-way aside 3.50 m wide.

All values adopted within the comparison refer to factored loads and consider both amplification factors on actions as well as safety factors on material strengths, in order to obtain consistent design values. The approach followed, which takes into account the safety level implicitly defined in each regulation, has allowed a realistic comparison between design requirements in different countries. The comparison shown in Fig. 6 refers to loaded bridge lengths in excess of 200 m where all load models are compared in terms of average load on the full deck section including footways.

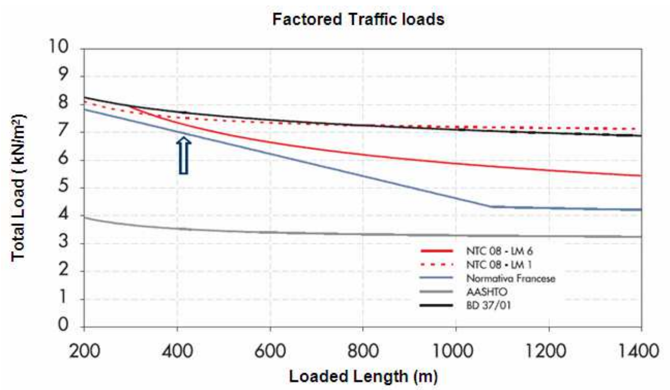


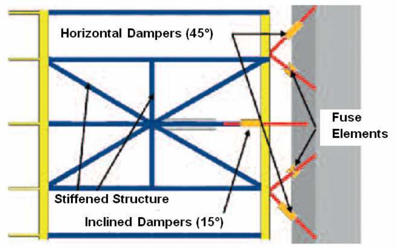
Figure 6. Traffic load comparison for loaded lengths greater than 200 m.

It should be remarked that values adopted for the AASHTO standards are referring to standard bridges since load models applicable to long span bridge are not specifically covered in these regulations and further, that current AASHTO requirements are largely underestimating the design traffic actions when compared to the other international standards. From Figure 6 it can also be highlighted that Eurocodes in general, and Italian Standards in particular, are capable of guarantying a design safety level which is comparable with that of the current British Standards and even conservative with respect to the French regulations. NTC 2008 - LM1 identifies load models to be adopted for loaded lengths less than 300 m, NTC 2008 - LM6 instead, applies specifically for loaded lengths in excess of 300 m. Recognizing that any of the approach analyzed will always be affected by approximations of real loads on long span bridges, as it has been shown by Buckland et al. (1991), and that the ratios of loads in multiple lanes vary with the loaded length and are therefore not constant, for the design of the SCB it is assumed that the traffic actions will be distributed accordingly with the Eurocode guidelines.

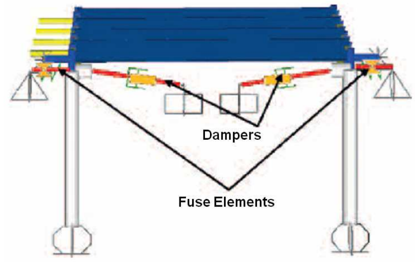
4. SEISMIC DESIGN OF THE SCB

4.1. Conceptual Seismic Design of the SCB

Seismic protection of the SCB has been perceived as the governing aspect of the entire project since the very first stages of conceptual design. In order to achieve high levels of seismic protection of the structure the '*Total Suspension Concept*' (Virlogeux et al. 2001) has been adopted as key of the conceptual design. This concept was already successfully implemented within the seismic design of the Rion-Antirion bridge in Greece (Combault et al. 2005), and implies the absence of any structural connection between deck and piers in case of seismic events, apart from seismic dissipation devices allowing the "free" oscillation of the deck subjected to seismic forces. The adopted solution necessarily implies an increase of the structural system flexibility, which from a static point of view has been resolved by increasing the structural stiffness of the pier-tower system in the longitudinal direction, whereas in the transverse direction, by adding rigid restraints between the decks and the piers acting as fuses (see Fig. 7a and 7b). Those elements represent static bidirectional restraints for the decks, avoiding both longitudinal and transverse relative displacements between decks and piers under wind loads and eccentric unbalanced traffic loads. During seismic excitations instead, all fuse elements are calibrated for a design force failure level which initiates relative displacements between decks and piers and allows energy dissipation after the activation of nonlinear viscous dampers.



a)



b)

Figure 7. Nonlinear Viscous dampers arrangement at main Piers (a) – and Temporary Piers (b).

Parametric analyses have been performed in order to calibrate all nonlinear viscous dampers capacities and to validate the structural concept and the preliminary sizing criterion implemented, both derived from a Displacement-Based approach (Calvi et al. 2010). Viscous dampers with nonlinear behavior have been adopted in order to achieve a better control on the level of forces developed within the connected structural elements and also, considering the outcomes of a specific seismic hazard assessment as well as the geotechnical properties of the soil at the project site, as a design measure against near faults effects characterized by high velocity pulses. The parametric study involved the dynamic characterization and analysis of the bridge under several connection arrangements, ranging from free to rigid and intermediate pier-to-deck connections. The latest scenario was obtained by estimated properties of nonlinear viscous dampers at relative displacement of 1.0 m and 0.5 m respectively for main piers and transition piers dampers.

Table 2. Effects of pier-to-deck connection types on forces transmitted to the tower base

Connection Type	Longitudinal Shear (MN)	Transverse Shear (MN)	Transverse Bending Mom. (MNm)
Intermed.	244	223	468
Rigid	313	277	580
Free	251	258	476

Table 3. Effects of pier-to-deck connection types on deck displacements monitored at Pier P2.

Connection Type	Longitudinal Displ. (m)	Transverse Displ.(m)	Vertical Displ.(m)
Intermed.	0.50	0.68	0.24
Rigid	0.12	0.06	0.05
Free	0.17	0.78	0.28

Results obtained (see Table 2 and Table 3), show that the intermediate connection solution results in the most suitable seismic behavior since it allows a significant reduction of both relative displacements and forces induced to the structural elements.

4.2. Definition of Seismic Loads

The seismic design of the SCB was performed starting from a preliminary review of the seismicity of the area by means of a probabilistic seismic hazard analysis (PSHA) aiming to determine the main features of the seismic inputs to be adopted in the analysis of the structure.

In facts, a set of Uniform Hazard Spectra (UHA – see Fig. 8a) was determined in terms of pseudo-acceleration, for 5% damping ratios, arbitrary direction of excitation and for two main return periods, 500 and 2000 years, respectively related to a 2% and 10% probability of exceedance in 50 years. The peak ground accelerations at the two design return periods were found to be around values of 0.25g and 0.43g respectively for 500 and 2000 years return period (see Fig. 8a). Values of PGA obtained also reflect the seismic hazard range for the town of Guayaquil identified by seismic hazard maps in the Ecuadorian area (Dimaté et al. 1999). The following selection of natural records to be adopted for structural analyses was performed starting from disaggregation analysis of the UHS for each of the design return period and within the range of estimated vibration periods of the structure (8.5 long and 6.5 sec. transv.). Scenarios mostly contributing to the specific hazard provided with the selection criteria in terms of event magnitude and distance, while ground properties in terms of $V_{s,30}$ were provided by in situ tests.

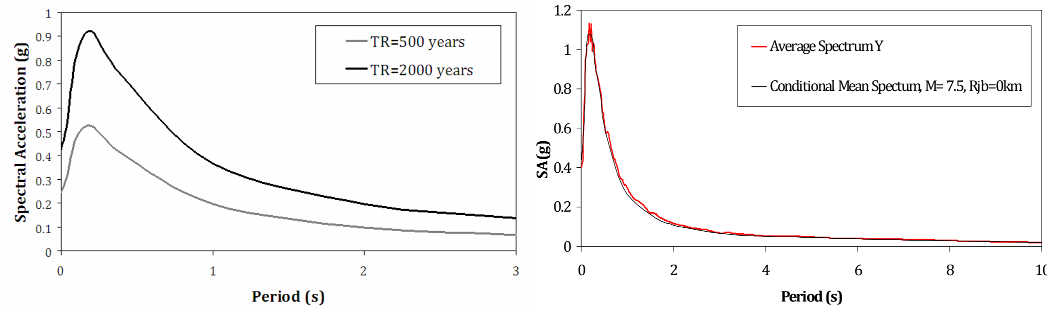


Figure 8. UHS for 500 and 2000 years Return Periods a) - Target Spectrum match (Y - transv. direction).

Natural records selected from PEER strong motion database were then consistently modified to match the design response spectrum (Fig. 8b), by means of wavelet modification techniques (Hancock et al. 2006), but preserving the main non-stationary properties of the original signal (SeismoMatch - Seismosoft, 2010). Three main seismic intensities have been selected to define seismic design objectives:

– Safety Evaluation Earthquake (SEE)
– Functionality Evaluation Earthquake (FEE)
– Construction Evaluation Earthquake (CEE).

The performance of the structure for the aforementioned seismic intensities was characterized by means of two main design limit states, Ultimate Limit Sate (ULS) and Serviceability Limit State (SLS), and two performance levels, Immediate Service Level and Damage Level, which in turn is defined by Minimum and Repairable Damage levels. Table 4 shows deformation limits adopted for structural performance assessment of the bridge concrete members subjected to seismic actions. As for piers and towers, also for decks, stays and expansion joints, sufficient strength should be ensured without or with only limited ductility demands following to SEE. Such a requirement, for cable stays implies to limit strains within values of 0.01%-0.015%, which generally corresponds to a 70-80% ratio of the ultimate strength capacity of the stay (f_{uk}) as certified by the producer (Gimsing et al. 1998). Lastly, residual displacement at expansion joints have to be limited to values not greater than 300 mm.

Table 4. Design limit States: Deformation limits for R.C. confined sections.

Structural Limit State			
SLS	ULS		
Concrete (compr.)	Reinforcement (tens.)	Concrete (compr.)	Reinforcement (tens.)
Elastic (<0.4%)	Elastic (<1.0%)	0.4-0.6%	1.0-1.2%

4.3. Finite Element Modelling

The structural model adopted for all structural analysis, both static and dynamic, adopted finite element type 'frame', 'cable', 'shell' and 'n-linear link' to best simulate the structural behavior, analyzed and assessed with the commercial software Sap 2000 v.11. Mass distributions assumed within the analyses are summarized in Table 4.

Table 4. SCB - Mass distribution

Structural Component	**Pier P1 (Kt)**	**Pier P2 (Kt)**	**Pier P3 (Kt)**
Decks	29.35	29.46	29.35
Stays	1.64	1.75	1.64
Towers	7.22	7.22	7.22
Piers	43.08	53.07	43.08

Damper link element type were adopted to model 'dynamically' controlled connections between pier and decks, whereas multi-linear link element were used to model static restraint provided by fuse elements. In the specific case, 6 nonlinear viscous dampers and 4 fuse elements were adopted on each of the main piers, whereas 2 dampers and 2 fuse elements were placed on each of the temporary piers. Due to the adopted geometrical configuration (see Fig. 7a and 7b), the action of each pier-to-deck connection resulted effective in all principal directions, X, Y, Z, though with different contributions in terms of resulting damping. The force displacement relationships assumed for the dampers are characterized by different level of expected forces and displacements depending on their specific location:

- Main pier: $\mathbf{F}_{\mathbf{damper}}= Cv^{\alpha}$ where $\alpha= 0.15$, C= 4.5MNs/m$^{\alpha}$; $\mathbf{F}_{\mathbf{fuse}}$ = ky where F= 5MN a y= 0.1m
- Transition Pier: $\mathbf{F}_{\mathbf{damper}}= Cv^{\alpha}$ where $\alpha= 0.15$, C= 3MNs/m$^{\alpha}$ $\mathbf{F}_{\mathbf{fuse}}$ = ky where F= 2MN a y= 0.1m

Soil Structure Interaction (SSI), which originates at foundation level, was also accounted for within the global analysis model, in terms of equivalent stiffness springs, representative of the full pile system adopted for the foundation. The spring equivalent properties, were derived from specific SSI studies where nonlinear analyses were performed by means of simplified modeling of the superstructure and detailed modeling of the pile foundation-soil system. In particular 'beam column' and 'nonlinear spring' elements, available in 'Open- Sees v.2.0' were adopted to model piles and soil behavior. Values adopted to implement the foundation stiffness matrix of each of the pile system in Sap 2000, where derived from the point of maximum expected displacement, which was estimated to be around values of 0.4 m.

4.4. Design and Strength checks of the main Structural elements

Strength checks were performed for two main design limit states (SLS and ULS) and for both static and dynamic load combinations. It was found that for most of the structural members, action derived from the seismic combination, obtained as the average response of seven nonlinear time histories, represented the governing scenario for the definitive design of the bridge. All ULS checks for composite decks were performed by direct comparison of demands and capacities obtained in terms of interaction diagrams (M-N diagrams in Fig.9a). The full set of checks allowed to observe a response globally elastic of the composite deck when subjected to seismic forces due to the Safety Evaluation Earthquake (SEE). For those cases where the demand point is quite close o the yielding surface of the analyzed sections, indeed (i.e. the segment linking two main spans - see Fig. 9a), local yielding of the structural steel and of the reinforcing steel should be expected as well as limited cracking of the concrete slab. Considering strain values implicitly adopted for concrete and steel materials within the definition of such capacity surfaces, it can be stated that all performance objectives related to the Repairable Damage level are fully satisfied following the structural sizes and details adopted for composite deck sections. Strength checks related to the SCB towers have required a minimum number of 4 control sections throughout the height of each tower leg due to their atypical cantilevering and tapered configurations. Interaction diagrams were obtained based on a minimum reinforcing steel ratio required for the relevant strength check verifications. From diagrams shown in Figure 9b, it can be appreciated that all seismic demands can be satisfied with reasonable amounts of reinforcing steel and more importantly, without any significant ductility demand for the designed sections. All demand points are in fact well inside the interaction surface, whose definition is obtained by assuming steel and concrete strain values respectively equal to 1% and 0.35%, which are consistent with limitations adopted for performance assessment of concrete members at ULS.

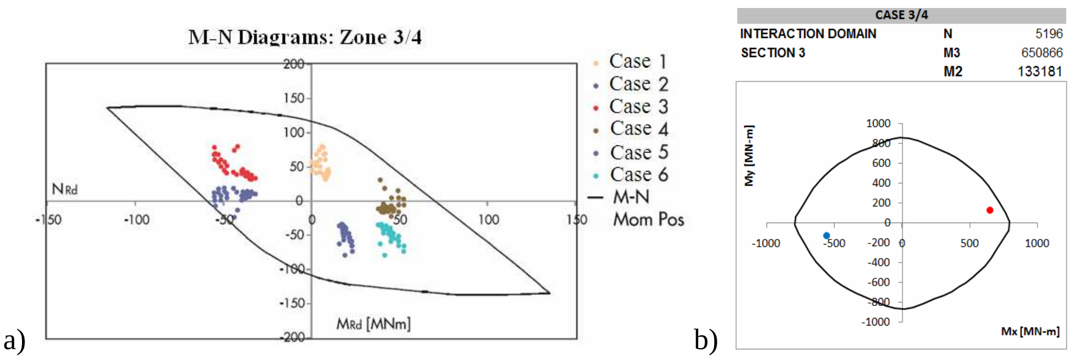


Figure 9. M-N interaction Diagrams: a) Composite deck – b) Tower ½ - ρ_{sl} = 3.5%, $A_{sl,tot}$ = 9765 cm^2

Seismic performance of nonlinear viscous dampers were lastly assessed in terms of required displacement and force capacities observed during nonlinear time history analyses, and preliminary estimated from a displacement-based approach (Calvi et al. 2010). Results obtained shows a very good agreement between the expected values (1 m - 4.5 MN e 0.5 m - 3 MN) and the average response recorded from NTHA. In Figure 10a e 10b are shown hysteretic loops for some of the most stressed dampers, located at the central pier of the bridge (pier P2) whereas Table 5 summarizes the average displacement demands.

Table 5. Dampers Average displacement demands.

	Design displacement	Damper ID
DESIGN SPECIFICATIONS	500 mm for P1 and P3	15° inclined Vertical Dampers
	1000 mm for P2	
	800 mm for P1 and P3	45° inclined Horizontal Dampers
	1100 mm for P2	
	300 m	Transition Pier

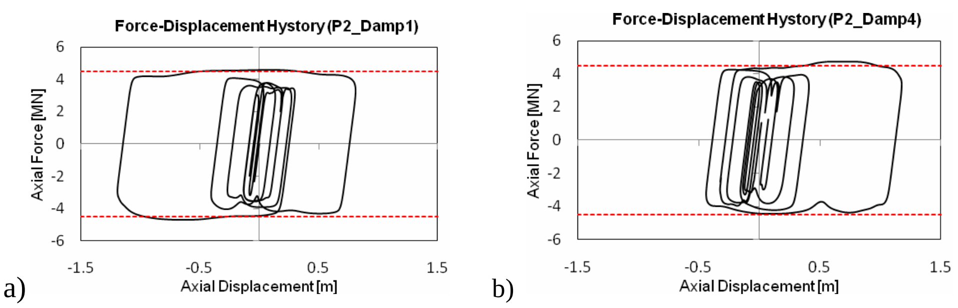


Figura 10. Nonlinear Viscous Damper hysteretic loops: a) Pier P2 - Horiz. Damper , b) Pier P2 – Verit. Damper

A detailed study by means of advanced solid models was finally undertaken for the tower head segment, undoubtedly the most complex structural compound of the entire structure because of its geometrical complexity, and due to the amount of concentrated stresses acting on it. For details of the study, the reader is referred to Calvi et al. 2011.

CONCLUSIONS

The design experience related to the South Crossing Bridge project has been briefly described, a long span cable-stayed bridge located in Ecuador, a region characterized by high seismic hazard. A carefull evaluation of the design strategies for some of the latest long span cable-stayed bridges, some of which also located in high seismicity regions, has allowed the identification of solutions to many of the challenges implied by designs of structures of political and economical relevance. Firstly the choice of appropriate design guidelines, usually dictated by local authorities or common practice can significantly affect core design choices when dealing with long span cable-stayed bridges. The level of safety as well as the comprehensive amount of design reference observed within the Eurocode guidelines, led to their adoption as reference standard for structural design of the SCB. The design approach undertaken for seismic design moreover, including the preliminary sizing criteria followed for the dissipation devices, lies outside the common practice range and represents therefore, one of the few examples of global approach to the seismic design of a relevant structure. Results obtained from nonlinear dynamic analyses and structural strength checks, proved the success of the seismic protection strategy, characterized by the adoption of the ‘Total Suspension Concept’, which together with implementation of dissipation devices, allowed to strike the balance between reduction of relative displacements and forces induced to the structural elements ensuring, at the same time, the full functionality of the structure at a serviceability level.

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