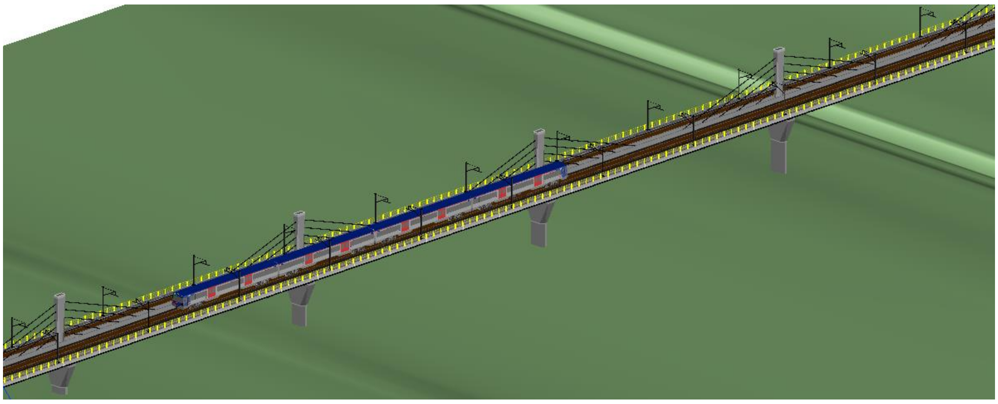

PONTE FERROVIÁRIA COM PRÉ-ESFORÇO EXTRADORSAL

PROJECTO BASE E ESTUDOS ESPECIAIS

Railway Extradosed Bridge

Base Case design and Special Studies

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"Extended Abstract"

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ABSTRACT

This thesis studies a structural solution in a preliminary level of an extradosed railway bridge for high-speed trains, it uses a pre-stressed concrete deck with a box girder section. We proceeded to the characterization of the construction process and it's modelling, studying the stresses during the construction and the bridge life span, taking more interest in the cables and the girder.

For definition of the actions and general safety criteria is used Eurocodes, the more important verifications are the fatigue and the limit states.

Beyond this, we calculated the deck's dynamic response during the crossing of high-speed train's compositions according Eurocode 1, evaluating the behaviour during the circulation.

For the vertical concrete elements analyses were used artificial accelerograms to calculate the dynamic response of the bridge, there were used viscous dampers and was studied its behaviour. For complementing the analyses was made a cracking study involving moment-curvature graphics for the longitudinal direction.

There was also made some interaction N-M diagrams for the piers design, and also was evaluated the unsymmetrical bending.

Finally, we resort to strut and tie models to solve some concrete elements that have a complex geometry.

Keywords: Railway Extradosed Bridges, Construction Process, Fatigue, High Speed Trains, Viscous Dampers, Interaction Diagrams, Strut and Tie Models

1 INTRODUCTION

In this thesis studies the structural behaviour of a railway extradosed bridge with high-speed trains, focussing in the following aspects:

1. Evaluate the structural behaviour in service conditions
2. The study of the construction methods;
3. The analysis of the more suitable static model for the connection between the deck and pylons to the infra-structure to have the best performance during service conditions and accidental seismic events;
4. The dynamic behaviour of the deck and pylons during the circulation of high-speed trains;
5. The study of the transmission of elevated forces in reinforced concrete using strut and tie models.

2 THE BASE CASE DESIGN SOLUTION

2.1 Extradosed Solution

This type of bridges uses extradosed cables anchored to pylons or towers to achieving a higher prestress effect, compressing the deck with a greater force than the convectional prestress cables configuration. They similar to the to stay-cables bridges and differentiate in the pylon/tower height and the vertical suspension component. The extradosed bridges pylons are shorter and the vertical component is carried by the cables and the deck unlike the cable-stayed bridge that all the lode is carried by the tendons.

So the box-girder must have a high stiffness to achieve a good service behaviour without being to flexible. This type of bridges are less susceptible to fatigue problems, because the high stiffness of the girder.

2.2 Stifness of the estradosed cables and deck

Extradosed bridges can be differentiated from the stay-cable bridges using a ratio parameter. This stiffness ratio (β) translates the load carried by the cables, the higher this value the higher load is carried by the cables. It is important that this value doesn´t exceed 30% because of the fatigue problems.

$$\beta = \frac{Load\ carried\ by\ the\ extradoses\ cables}{Total\ vertical\ load} \tag{2.1}$$

In the studied bridge this value is 14%.

2.3 Deck Design

The bridge has 6 interior spans with 66 m and two lateral spans with 37 m, which makes a total of 470 m of total length.
The deck solution is a 2,5m height prestressed concrete box-girder (Figure 2.1).

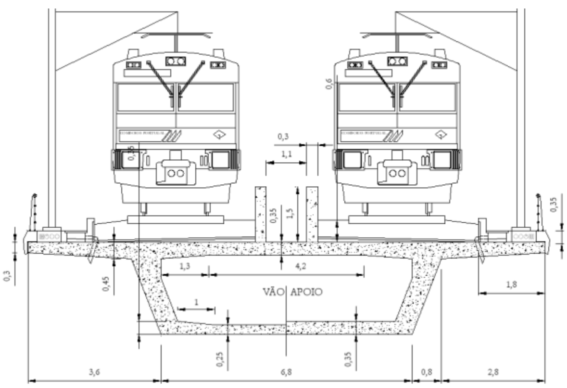

Figure 2.1 - Reinforced concrete deck in the span and support deck sections

The deck uses three types of prestressed cables (i) extradosed cables that are anchored to pylons over the piers with an harp configuration; (ii) Longitudinal prestressed cables with a parabolic profile in the lateral span and a straight cable configuration in the main spans, with 19 strands 0,6"S; (iii) The transversal cables that have a parabolic profile and 4 strands 0,6"N

Additionally in the extradosed cables anchorage site is implemented two steel struts. This solution involves CHS tubes and a thread stress bar (Figure 2.2). It is important that during the service conditions the connection between the metallic tube and the box girder doesn't have alternate tensions.

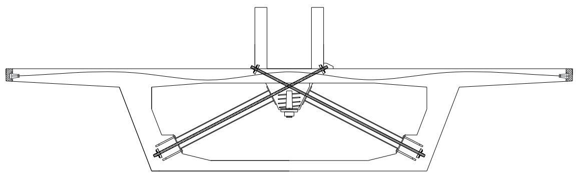

Figure 2.2 - Metallic strut and prestressed cable

2.4 Pylons and the extradosed cables

This bridge has 7 pylons with 11,5 m and 5 extradosed cables with 55 strands 0,6"S. They intersect the pylon with saddles. The structural design is shown in the Figure 2.3.

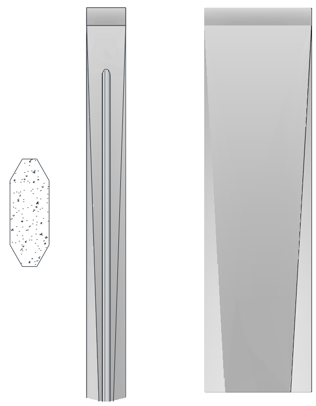

Figure 2.3 – Pylons section and lateral view

The pylon is fixed to the pier, that solution is important to have a more stiff deck and reduce the stress variation due to the live loads in the deck.

The cables are connected to pylon using a saddle solution (Figure 2.4). This way we can do the transition of the cables in the pylon deviating it strand by strand. We can reduce the number of anchors, being only necessary in connection between the cable and the deck. With this solution we can also have a more slender pylon because we the extra space to install the anchors [1].

Figure 2.4 – Saddle to do the cable transition in the pylon

2.5 Piers, Abutments and Foundations

The design of the piers are shown in the Figure 2.5. This elements are connected to the deck with free sliding pot-bearing supports along the longitudinal direction. Because of the connection between the piers and the pylons and because of the low profile of the extradosed cables, we have high horizontal stiffness that request the piers during horizontal applied forces. The implication of this fact is that there is no need to have fixed pot bearings in this direction.

On the transversal direction all the pot-bearings are fixed.

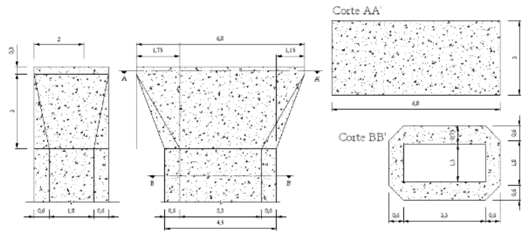

Figure 2.5 - Piers design

The abutments design is shown in Figure 2.6.

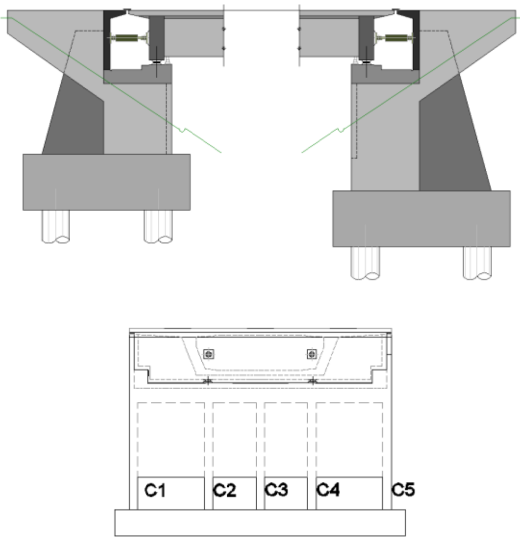

Figure 2.6 – Abutments design

The connectivity in the transversal direction is fixed and free in the longitudinal direction. This is due to creep and the shrinkage time dependent and the uniform temperature variation effects that generates high stresses because of the elevated stiffness of this element. To the abutments as seismic resistant elements we resorted to viscous dampers. The abutments have 2 dampers with 5000 kN of maximum force each (Figure 2.7).

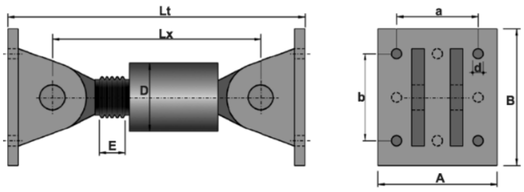

Figure 2.7 - Viscous dampers type ALGASISM FD

The foundation of the piers have with 8 reinforced concrete piles with 1,5m diameter (Figure 2.8) and the foundation of the abutments have only 6 piles.

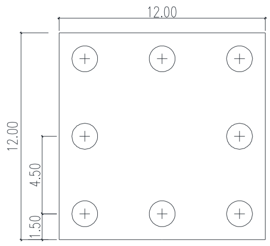

Figure 2.8 - Piles design

2.6 Construction methods

The construction process resorts to temporary framing of the entire bridge, the concreting is staged and done in two construction fronts.

First is done the preparation of the terrain, than executes the foundations and the piers, and finally the concreting begins near the abutments.

During the construction of the deck is intended to install the extradosed cables for the purpose of make parts of the deck self-supporting, the construction is symmetric. After each stage the last segment is done connecting the deck to the other structure already made.

Because of the length of the bridge, the smaller piers are subjected to high stresses due to creep and the shrinkage time dependent and the uniform temperature. To solve this problem during the construction last stages are installed hydraulic jacks to introduce reactions in the direction of the abutments before the last segment is done. This way we can counteract the shrinkage effect.

During the construction to counteract the shrinkage effect the abutment is fixed, this way because the stiffness centre is situated near the abutment, the bridge will move in his direction.

3 MATERIALS AND DESIGN CRITERIA

There are used several materials in this bridge, (i)for the pylons and deck it's used concrete C40/50; (ii) foundations, abutments and piers C30/37; (iii) the CHS 273,2x16,0 is done in S355 steel; (iv) thread stress bar S1081 ø57 mm; (v) rebar A500NR.

The definition of actions and general design criteria are in accordance with the Eurocodes. For the ULS were used the combinations in the (Figure 3.1). The first two combinations refer to static loads and the last two are seismic combinations.

$$E_{Sd} = 1{,}35(E_{cp} + E_{c.extrad.} + E_{pe}) + 1{,}50(E_{sc} + 0{,}80\, E_{arranq.fren})$$

or

$$E_{Sd} = 1{,}35(E_{cp} + E_{c.extrad\cdot} + E_{pe}) + 1{,}50\, (E_{arranq.fren} + 0{,}80\, E_{sc})$$

$$E_{Ed} = E_{cp} + E_{c.extrad.} + E_{pe} + E_{Ed,Tranversal} + 0{,}3\, E_{Ed,Longitudinal}$$

$$E_{Ed} = E_{cp} + E_{c.extrad.} + E_{pe} + 0{,}3\, E_{Ed,Tranversal} + E_{Ed,Longitudinal}$$

Figure 3.1 - Ultimate limited state combinations

For the SLS were used different criteria for the deck, piers and extradosed cables.

The deck used the following criteria:

1) Decompression in all the deck sections for the following combinations

   -Permanent loads

   - $CP + \psi_2 \Delta T_d$

   - $CP + \psi_1 Sob$

2) All sections will not crack for the following combinations

   -$CP + \Delta T_d + Sob$

The extradosed cables for not susceptible to fatigue problems the tension in each cable should not be higher than 0.5 f_{puk}, for service combinations.

For the fatigue criteria was uses EC3-1-11, is important to define the damage equivalent factors because they define the level stresses that the cables are subjected during the service conditions. The extradosed cables, the thread stress bar and the CHS tube were verified to fatigue problems.

4 ANALYSE OF THE SUPERSTRUCTURE

The deck analyses uses three different models, one for transversal analysis, other for longitudinal analysis and the last one to the behaviour for high speed circulation.

4.1 Transversal Analyses

The transversal analyses resort to a three dimensional model (Figure 4.1) using finite elements. In this model we will analyse the metallic struts with the variation of the stresses for a static live load action. Also evaluates the decompression and cracking of the top slab and its reinforcement. Finally we do the fatigue verification.

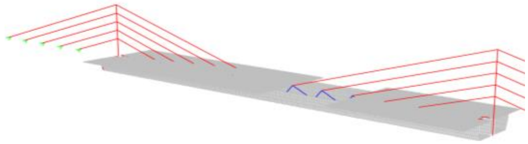

Figure 4.1 - Three dimensional model using *SAP2000*

The bending moments envelopes are shown in the Figure 3.2 on the left is the minim values and on the right the maxim values.

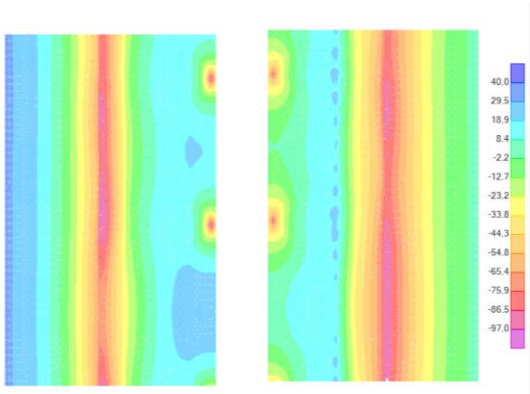

Figure 3.2 – Shell stresses for frequent combination

The decompression for the frequent combination is verified and the non-cracking analyses for the characteristic combination is also verified.

The fatigue verification is shown above in the Figure 4.2, Δσ is referred to the highest stress variation and $\Delta\sigma_E$ is the stress variation affect by the damage equivalent factors. The parameter Δσc is the material resistance.

Δσ(MPa)	$\Delta\sigma_E$(MPa)	$\Delta\sigma_c/\gamma_{mf}$ (MPa)
34,13	19,35	160/1.35=118,51

Figure 4.2 - Fatigue verification for the CHS

4.2 Longitudinal Analyses

The model for the longitudinal analyses uses bar finite elements (

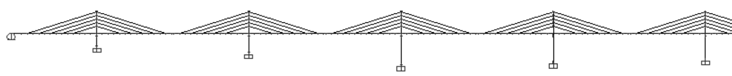

Figure 4.3).

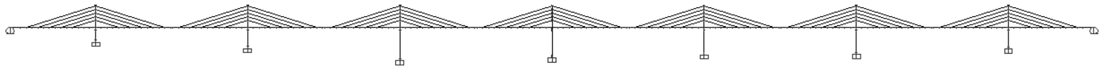

Figure 4.3 - Longitudinal Model

The decompression of the entire deck wasn't achieved (Figure 4.4) but the non-cracking criteria is verified for all the bridge deck (Figure 4.5).

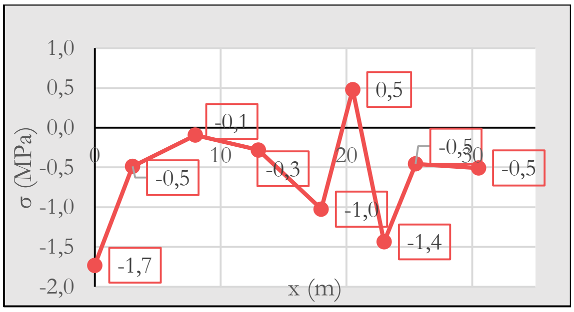

Figure 4.4 - Tension for inferior fibre in the frequent combination, for the deck for the most interior span

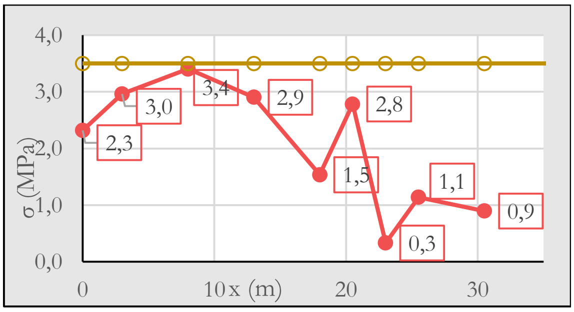

Figure 4.5 - Tension for inferior fibre in the characteristic combination, for the deck for the most interior span

For the ULS we resorted to interaction N-M diagrams. The usage of this analyses is required because the deck is request by a combined axial and bending effects. The axial force due to extradosed cables makes the prestressed concrete analysis that's why the usage of the diagrams. It's show in the Figure 4.6 and Figure 4.7 the interaction diagrams for the two more requested sections of the deck.

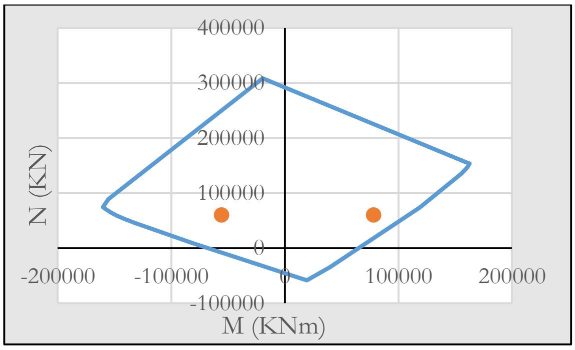

Figure 4.6 - N-M for the a span section

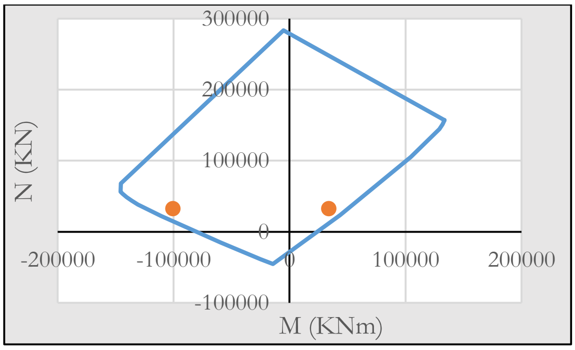

Figure 4.7 - N-M for the deck support section

Finally the fatigue verification of the extradosed cable with the higher stress variation is shown in the Figure 4.8.

$\Delta\sigma$(MPa)	$\Delta\sigma_E$(MPa)	$\Delta\sigma_c/\gamma_{mf}$ (MPa)
185,20	105,00	160/1,35=118,52

Figure 4.8 - Fatigue verification of an extradosed cable

4.3 Deck Behaviour for High Speed Circulation

The deck dynamic response for the crossing of high speed railways is defined in part 2 of EC 1, the objective of this verification is to evaluate the possibility of a high speed train cross over the bridge.

The dynamic effects are factors that influence this effect are:

- The overload speed;
- The span length;
- The structure mass;
- The structure natural frequencies and its vibration modes;
- The number of axles, its spacing and their corresponding load;
- The vehicle mass and the suspension characteristics;
- The structure damping;
- The track irregularities;
- The vehicle imperfections;
- The existence of ballast

According to EN 1990 A2.4.4.2; for ballasted tracks the vertical acceleration of the deck should not exceed 3,5 m/s^2 for the 10 HSLM. The analyses considered different traveling speeds for a time history analyses. The 10 trains travelled 40 m/s to 117 m/s (from 144 Km/h up to 420 Km/h). In the Figure 4.9 is shown the highest response in the deck and the maximum value wasn't exceeded.

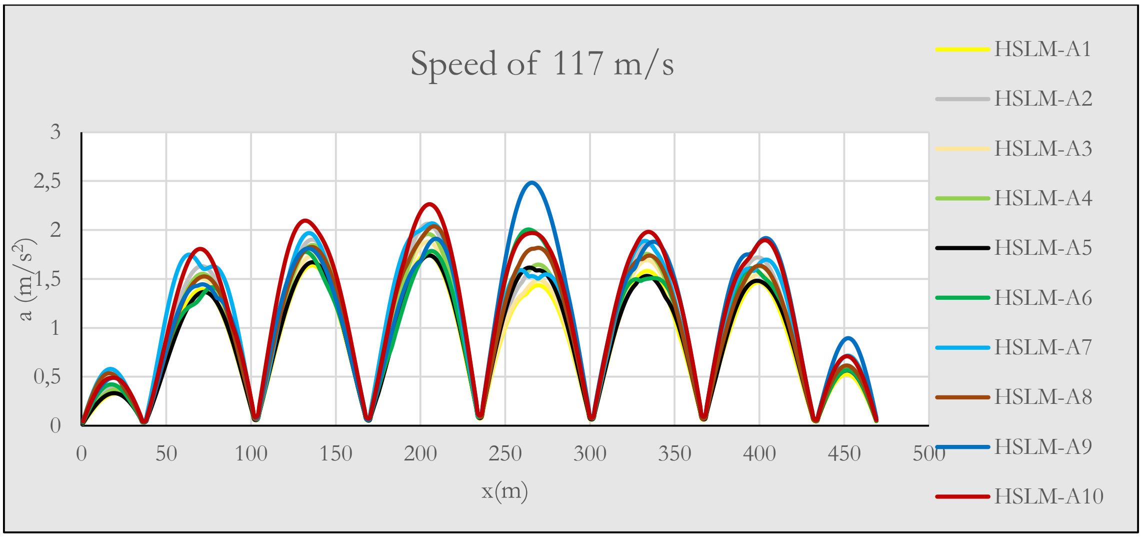

Figure 4.9 - Vertical acceleration of the deck

Besides this evaluation the dynamic bending forces should not exceed the static ones. We should account a higher bending force in the reinforcement of the deck.

As we can see in the Figure 4.10 the static bending moments are higher than the dynamic values.

There was also made in the end the verification of the deck twist. This phenomenon is important because the train might lose contact with the rails if this effect is not accounted for. Because of high girder stiffness this phenomena loses its importance and after the evaluation of this effect was concluded that the conditions were verified.

Finally the deformation of the deck in service conditions were not entirely achieved, the comfort of this bridge reach the acceptable level of comfort.

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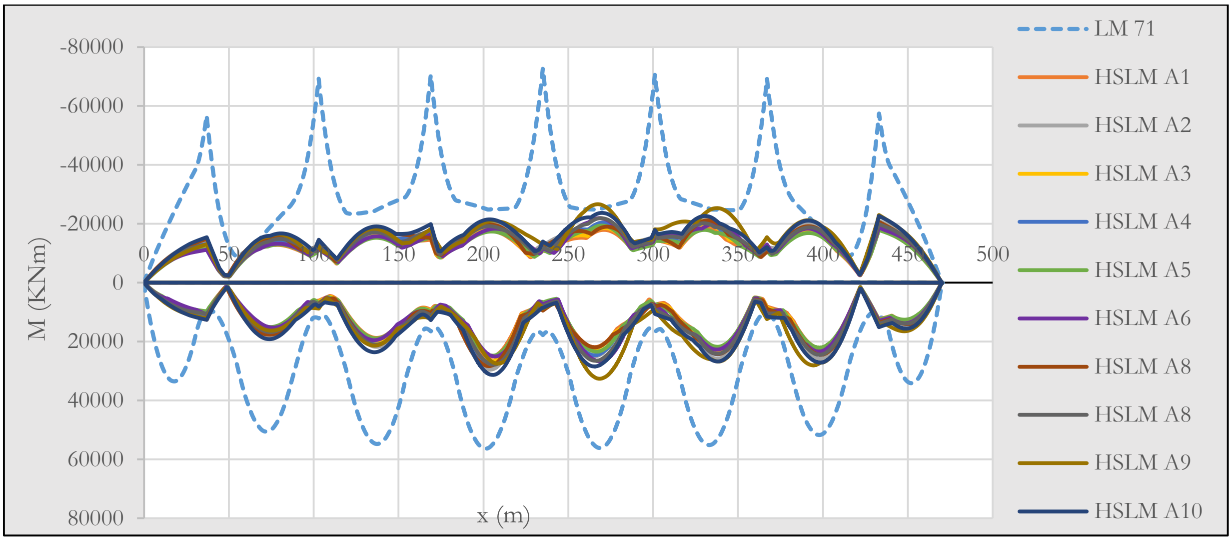

Figure 4.10 – Static and Dynamic bending diagrams

5 VERIFICATION OF PIERS AND ABUTMENTS

5.1 Longitudinal seismic analysis

For this analysis was used 8 artificial accelerograms (Figure 5.1) to model a seism according to EC8 1-1, this was due to the existence of viscous dampers. Because of the nonlinear behaviour of this elements and being speed and time dependent, was needed a time history analysis. To simplify this analysis, was done a modal integration in time because it is faster than the direct integration. The simplified version of this analysis must be only used if the nonlinear behaviour is focused in the viscous dampers.

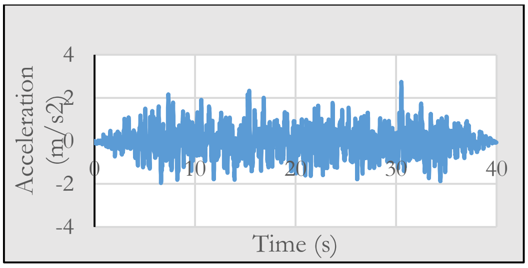

Figure 5.1 - Artificial accelerogram

The viscous dampers parameters are the damping constant (C) that defines the maxim damping force, that depends of the type of damper's holes were the liquid flows. The parameter α that is a characteristic of the liquid viscous, the bigger the value the less sensitive the damper is to the velocity [2]. The behaviour of this element is influenced by the velocity of the deck, so if the movement of the deck is very slow this element will not offer resistance. So for the shrinkage effect the deck over the abutments is free.

5.2 Transversal seismic analysis

This analyses uses the response spectrum according to EC8 part 1, and a behaviour factor of 1,5.

5.3 Moment-curvature

During the seismic event the piers might crack because of the high moments on the base of the piers. Because of this effect the structure will lose stiffness and is reduced the seismic force. To do a cracking analysis was used a moment-curvature graph (Figure 5.2).

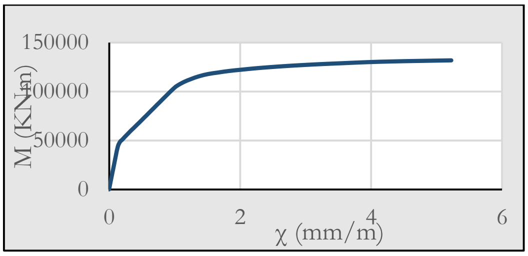

Figure 5.2 - Moment-curvature in the longitudinal direction

5.4 Creep and the shrinkage time dependent

The effect of this phenomena was account during the construction stages. Due to the length of the bride the two piers near the abutments crack under service conditions, but the cracking is under 0,30 mm, so it is acceptable. The stiffness lost was taken into account in the other analysis.

5.5 Ultimate limit states

This analysis was done with the usage of interaction diagrams N-M. Because of the unsymmetrical bending, there was made a study to assess a α factor (Figure 5.3).

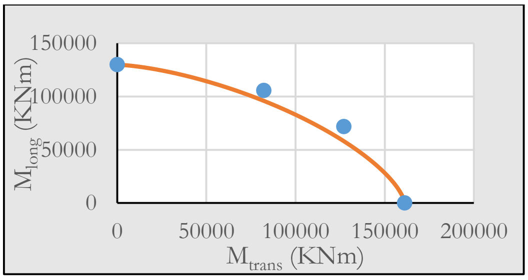

Figure 5.3 - Unsymmetrical bending interaction

5.6 Longitudinal deck movement

Due to damage control it is important to have horizontal stiffness to minimize the lateral movements of the bridge. The maximum value is 20 mm, and the movement is evaluated for a service seismic event and train breaking.

The maximum value registed was 9 mm (Figure 5.4).

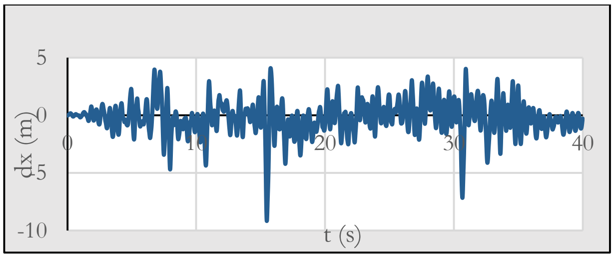

Figure 5.4 - Movement near the abutment during a service seismic event

5.7 Strut and tie models

The abutments and piers capital have a complex geometry so it was resorted to strut and tie models to assess the reinforcement in this elements.

This models is made visualizing the load path and building in top of it a strut and tie elements (Figure 5.4). The general principle is that the equilibrium and strength conditions must be fulfilled.

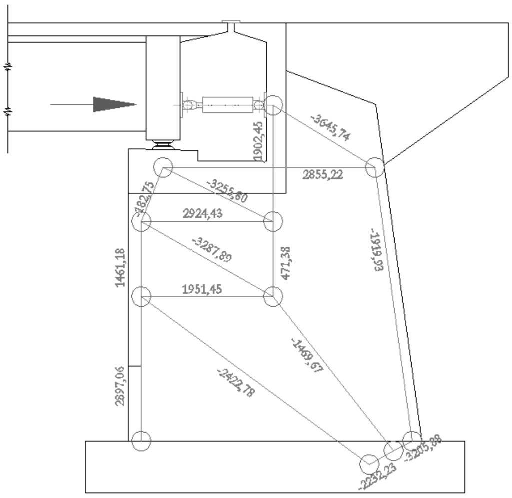

Figure 5.5 - Strut and tie model for the damper under compression

6 MAIN CONCLUSIONS

In this paper was studied a base cased design project of an extradosed bridge. There were done several analysis that are mentioned below:

A. We did a complete construction stage analysis;
B. The analysis of the deck behaviour for high speed circulation;
C. A seismic analysis using time history analysis and studied the viscous dampers behaviour when applied to the abutments;
D. Strut and tie models;

The conclusions from this study must be referred to:

i. All the ultimate limited states were verified;
ii. The bridge response to high speed trains was under the regulated limit 3,5 m/s^2;
iii. We didn't achieve the total decompression of the deck, but under service conditions for all the deck length is verified the non-cracking condition, concluding that it will not be a stiffness loss during the service;
iv. Two piers crack for service conditions, but the crack width is under 0,3 mm, the stiffness loss of this piers were accounted for during the other analysis;
v. Over all the solution is well balanced and have a good response behaviour under vertical and horizontal loads, and static and dynamic effects, with a slender deck and a low profile cables;

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