The Use of High Strength Steel in Bridge Decks

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EXTENDED ABSTRACT

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1 Introduction

For a long time, steel plate girders combined with a concrete slab deck have been using as solution for composite bridge decks for their economic, constructive and structural advantages. In parallel, high strength steel has became available on the last decade and extended its utility in the market of steel structures, taking into consideration its enhanced properties lead to both high yield strength and ductility.

However, it is only useful to use high strength steel when using slender decks which creates buckling issues that need to be investigated. The web thickness becomes a demanding factor in terms of buckling and the fatigue verification becomes an issue since there is an increase in stress ranges when using plates with more reduced thicknesses.

Therefore, the aim of the present work is to investigate the possibilities of using high strength steel in bridge decks. For that purpose, three solutions for steel girder for a railway bridge deck were studied.

The first solution (solution A) consists of twin plate girder deck made of steel S355 NL, commonly used on composite steel-concrete bridges. The bending moments and shear force were calculated for the different actions: permanent loads; traffic loads; thermal gradients and concrete shrinkage. The safety verification was evaluated for the different ultimate limit states (ULS) – resistance to bending moment and shear forces; flange induced buckling; lateral buckling and fatigue resistance – and serviceability limit states (SLS) – web breathing; deformation and in-service stress control.

Solution B is formed by a twin plate girder deck built with high resistance steel (S690 QL). The same analysis done in solution A was performed for solution B, allowing a direct comparison between the two design solutions. The problems induced by the use of the high strength steel were identified and the girder dimensions adjusted so that all safety verifications were fulfilled.

Solution C was then proposed by using a bottom flange composed by a tubular section enlacing a better use of steel S690 in those design aspects where solution B showed to be less structural efficient. Local plate buckling and fatigue resistance.

2 Context

Given the bridge's extension, a model of five spans of the bridge was performed in the program ftool to allow a study of the central span as a representative of the typical span of the bridge deck. The spans are 45 m long and 13 m width, carrying a two railway tracks.

The deck cross section is composed of a reinforced concrete slab supported by two steel plate girders. The slab has a thickness of 0.4 m at mid-span and 0.2 m at the tip of the cantilevers. Steel girders I-shaped are formed by two flanges and a web, welded longitudinally for a total high of 2.6 m.

The Figure 1 and Figure 2 present the deck cross sections of mid-span and over the supports, respectively.

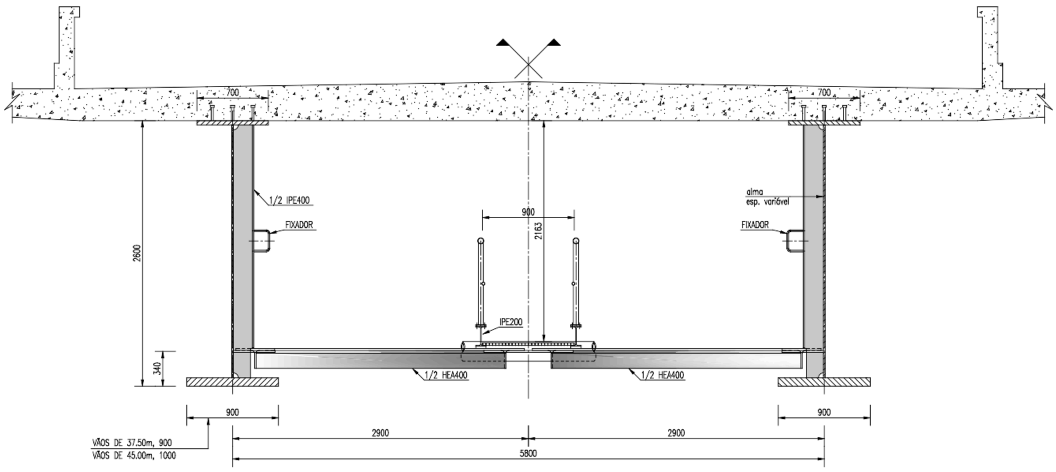

Figure 1 – Deck mid-span cross-section

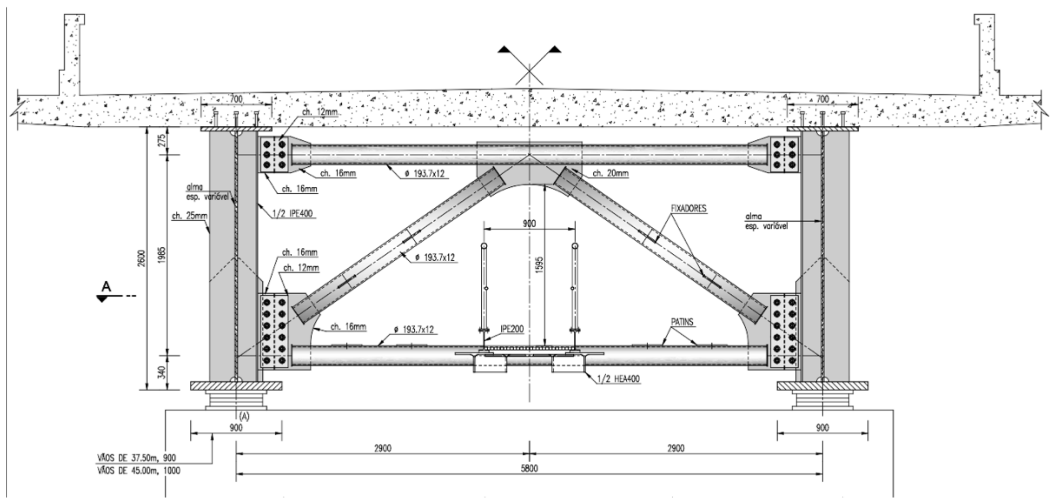

Figure 2 – Deck cross-section over the supports

3 Loads

The loads considered in the design are: permanent loads (self-weight and non structural bridge equipments); traffic loads; thermal gradients and concrete shrinkage.

The design values for the self-weight of one steel girder is 10 kN/m and the assotiated load of the reinforced concrete slab is 52.15 kN/m. An additional 10% increase of the self-weight of the steel girders was considered to account diaphrags, vertical stiffeners and bracings. The design value for non structural equipment attached to one girder is 83.6 kN/m.

The traffic load to be considered is “load model 71” (LM71) which represents the static effect of vertical loading due to normal heavy rail traffic. This load model is given in the EC1-2 [1] and consists of 4 concentrated vertical forces of 250 kN each and a distributed vertical force of 80 kN/m, as shown in Figure 3.

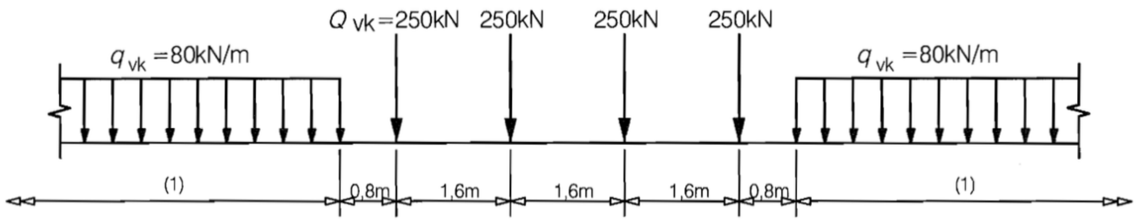

Figure 3 - Load Model 71

This load must be multiplied by a factor to have in consideration the dynamic aspects of the load. According with the EC1-2 [1], this dynamic factor is given by:

$$\phi = \frac{2,16}{\sqrt{L_{\emptyset}} - 0,2} + 0,73$$

However, considering the dimensions of the span and the number of spans the factor is equal to 1.0 having no influence on the effects of this traffic load.

The thermal gradient to be used was adopted from EC1-1-5 [2]. The national annex indicated the approach 1 should be used, which consists of applying an equivalent linear temperature variation between top and bottom face of the deck cross-section. The values used are 12ºC for determining internal forces and bending moments at the span sections and -18ºC for obtaining the same forces over the supports.

The concrete shrinkage is equivalent to a negative strain of the slab during time. Therefore, the effect of this load was analyzed by introducing in the model the equivalent forces induced by shrinkage deformation in the slab. Thus, the restraining force of the concrete slab is given by:

$$F_{ret} = \varepsilon_{ret} \cdot E_{c,eq} \cdot A_c$$

This is equivalent to a force with the same value and a bending moment applied at the centroid of the composite deck cross-section. Over the supports, where the concrete is considered to be fully cracked, the effects of shrinkage may be neglected. Thus, the force and bending moment applied in the model are located at 0,15L from the supports.

The shrinkage effect was only considered in the supports region as it is conservative not to consider it in the mid-span section, and only the hyperstatic effects were considered.

4 Design of deck solutions A, B and C

The first solution (solution A) is formed by two I-shaped beams made of steel S355 NL, commonly used on bridge decks, attached to a concrete slab by three connectors of 22 mm each, conceding a full connection between the steel girders and the concrete.

Solution B is also formed by two I-shaped beams but made of high strength steel S690 QL, allowing a slenderer structure. The other deck characteristics were maintained – bracings, diaphragms and the concrete slab.

Solution C is created by two beams with alternative bottom flanges composed by a tubular section that allows the girder not to be so subjected to buckling without as much steel as solutions A and B.

In Figures 4 and 5 is illustrated the mid-span and support cross-sections of the three solutions that verify the safety, respectively.

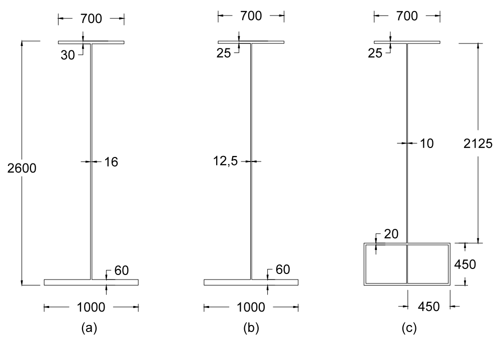

Figure 4 - Mid-span cross-section for (a) Solution A (b) Solution B (c) Solution C [mm]

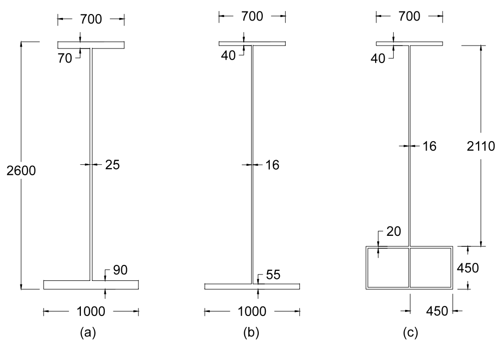

Figure 5 - Support cross-section for (a) Solution A (b) Solution B (c) Solution C [mm]

5 Ultimate Limit State Safety Verifications

5.1 Resistance of cross-sections

By comparison of the three solutions studied it was found that both solution B and C offer a better efficiency as they produce a higher bending resistance using less quantity of steel than solution A.

As to shear resistance, contrary to solutions A and C, solution B does not verify the safety over the supports, as a consequence of the reduction of the web thickness. However, this problem is easily solved by adding a transverse reinforcement to the web panels near the supports.

Solution C presented itself to be a better solution in terms of this ultimate limit state as it has a high bending resistance and, although it has the same web thickness as solution B, its new girder form allows it to ensure a higher shear resistance.

The bending resistance were calculated excluding the resistance offered by the web. Therefore, all the web resistance is guaranteed for the shear forces and there is no need to verify the interaction between shear force and bending moment.

5.2 Flange induced buckling

Flange induced buckling consists in the buckling of the flange in the surface of the web and, according to EC3-2 [3], its verification is done by limiting the ratio h_w/t_w to:

$$\frac{h_w}{t_w} \leq K \cdot \frac{E}{f_{yf}} \cdot \sqrt{\frac{A_w}{A_{fc}}}$$

However, this expression is too conservative making it critical for solution B and C where the web slenderness is higher. Therefore, the expression was changed to have into consideration:

- The non symmetrical shape of the cross section which defined the position of the neutral axis not exactly at the middle;
- The real tension installed in both flanges.

The new expression, by which the safety is verified for solutions B and C, is given by:

$$\frac{h_w}{t_w} \leq K \cdot \frac{E}{\beta \cdot f_{yf}} \cdot \sqrt{\frac{A_w}{A_{fc}}}$$

where the factor β is a function of $\frac{h}{h_i}$ and $\frac{\sigma_{Ed}}{f_{yf}}$:

$$\beta = \frac{h}{3h_i} \cdot \left(\frac{\sigma_{Ed}}{f_{yf}} + 0{,}5\right)$$

5.3 Lateral Buckling of the bottom flange over the supports

This ultimate limit state is defined by the buckling of the bottom flange in the deck support sections, since it has a high compressive stress installed. The safety to lateral buckling was verified both in solution A and B by ensuring that the resultant internal axial force is lower than the critical axial force.

As lateral buckling did not represented a problem in the use of high strength steel it was not analyzed in solution C. However, the new form of the bottom flange used in solution C has a lower lateral slenderness than the traditional I-shaped girders so it also verifies the safety to this ultimate limit state.

6 Serviceability Limit State

6.1 Web breathing

The constant change of the web from his natural shape to the deformed shape and back to the initial position caused by the passing of trains causes vibrations that can lead to fatigue problems. Therefore, EC3-2 [3] limits the web slenderness to:

$$h_w/t_w \leq 55 + 3{,}3 \cdot L \qquad com\ L \geq 20m$$

Once again, due to the reduction on the web thickness solutions B and C do not verify this limit forcing the use of a web thickness higher than the necessary for the cross-section resistance. It was also perceived that, although both solution B and C require an increase in the web thickness, solution C demand a lower increase than B.

6.2 Deformation

The Suisse code SIA260 [4] was used for the deformation limit:

- Comfort Limit: deformation due to frequent traffic action $(\delta(\psi_1 Q_{k1}))$ limited to $L/500$;
- Aspect limit: Deformation due to permanent load $(\delta(G_k))$ limited to $\left(\frac{L}{700} - \omega_0\right)$ where ω_0 represents the precamber applied to the structure.

Both solution A and B verified the safety. This serviceability limit state was not studied in solution C, however since the proposed girders present a similar vertical stiffness that solution B, it should also verify this criteria.

6.3 Stress limitations

Stress limits were verified for solutions A and B. It was observed an increase in stresses of the concrete slab and the longitudinal reinforcement when using high strength steel instead of normal strength steel. However, the increase is far from exceeding the limits given in EC2-2 [5].

6.4 Fatigue

This ultimate limit state is the most limiting design criteria. Solution A is already near the stress range limits imposed to the welded joints analyzed. Solution B did not verify the security by a large margin, forcing large thickness increases of the bottom flange, that are almost equal to the thickness used for solution A. This makes the use of high strength steel with practical no benefit in terms of reducing the overall quantity of structural steel.

Solution C showed to be more efficient as it did not need as much steel as solution A and B to verify the fatigue safety requirements. However, it still needs an increase of the minimum steel thicknesses needed for achieving cross-section resistance.

7 Final Conclusions

The use of welded I-Shaped beams in high strength steel S690 QL allows, for the same bending resistance, less steel to be used. However, the girders have a higher slenderness causing local buckling issues of the web and flange plates. In the web: 1) resistance to shear force, 2) web breathing and 3) flange induced buckling, are the important design criteria, as well as fatigue resistance of the welded joints, namely bottom flange welding and web vertical stiffener to flange welding. These effects force the use of an amount of steel higher than the necessary to verify cross-section resistance.

In terms of stress and deformation, although there was an increase when using high resolution steel, the values still are very far from the limits given.

It was verified that the use of a tubular section for the bottom flange in solution C allows a more efficient application of S690 QL steel, making possible the safety of the structure using a less quantity of steel.

The evaluation of the three solutions was done by means of the overall steel quantity used per span to verify the safety the design ULS and SLS listed before. As can be observed in Table 1 and Table 2, the solution C proved to be the most efficient in terms of steel quantities.

Table 1 – Steel quantities per span for Solution A and B

	Solution A	Solution B	% steel saved
Support cross-section area [cm^2]	4000,0	2462,00	38%
Mid-span section area [cm^2]	2148,0	2054,00	4%
Reinforcements and bracings [cm^3]	1722522,3	1722522,3	0%
Steel weight per span [ton]	109,0	90,4	17%
Steel weight per deck area [kg/m^2]	186,4	154,5	17%

Table 2 – Steel quantities per span for Solution A and C

	Solution A	Solution C	% steel saved
Support cross-section area [cm^2]	4000,0	2428,0	39%
Mid-span section area [cm^2]	2148,0	1912,0	11%
Reinforcements and bracings [cm^3]	1722522,3	1722522,3	0%
Steel weight per span [ton]	109,0	86,5	21%
Steel weight per deck area [kg/m^2]	186,4	147,9	21%

References

[1] EN 1991-2. *Eurocode 1: Actions on structures – Part 2: Traffic loads on bridges.* CEN. Brussels. September 2003.

[2] EN 1991-1-5. *Eurocode 1: Actions on structures – Part 1-5: General actions – Thermal actions*. CEN. Brussels, November 2003.

[3] EN 1993-2. *Eurocode 3: Design of steel structures – Part 2: Steel Bridges.* CEN. Brussels, October 2006.

[4] SIA 260. *Bases pour l'élaboration des projets de structures porteuses*. Société suisse des ingénieurs et des architectes. 2003 by SIA Zurich.

[5] EN 1992-2. *Eurocode 2: Design of concrete structures – Concrete bridges – Design and detailing rules*. CEN, Brussels, October 2005.