New launching method for steel bridges based on a self-supporting deck system. FEM and DOE analysis

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Abstract

This paper studies a new launching method for steel bridges based on a self-supporting deck system. This new and patent-protected procedure is able to launch bridges of a span length up to 150 m, in an economical and sustainable way. The use of the last span segments to reinforce the critical sections during launching replaces other conventional temporary means applied nowadays. The main objective of this research paper is to numerically analyse the best double deck configuration as well as to define an approach to stiffener distribution in order to avoid the patch loading phenomenon in the slender webs. With this in mind, the pre-design of a triangular cell along the low flange of each web is presented. A three dimensional finite element model (FEM) is built and the design of experiments technique (DOE) is applied to obtain the best bridge configuration. This new construction method can be used together with a continuous launching system in order to increase the velocity of the whole operation and to improve safety during launching. Very good results have been obtained, in terms of deflection, patch loading resistance and vertical load on the pushing device. The comparison with other different construction systems and the application to a real case allows us to ensure the viability of the method described.

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1. Introduction and historical review

The bridge launching technique is defined as a construction system in which the whole deck structure is divided into segments and not directly placed into its definitive position but a nearby location, usually behind one of the abutments (see Fig. 1) [1, 2, 3]. After that, the bridge erected at that moment is moved forward on the prebuilt piers until the final abutment, by means of hydraulic jacks. This movement can be done by horizontal displacement, even giving it its definitive longitudinal slope; the bridge slides, or also rolls, on the supports placed at the top of the piers and in the launching yard. Other intermediate bearings may be used between the piers of the viaduct. In this way the construction of the bridge is completely independent from the ground conditions and it is only necessary to make the accesses to the foundations.

The most important parameters are the maximum span of the structure, and the material of the cross section, e.g. steel or concrete bridges. These variables determine the best system to be used, because the bending moments in a cantilever beam are higher than in a continuous beam, but with opposite signs. There are several launching systems nowadays, and some outstanding structures have been built during the last few years all over the world [4, 5].

Other researchers focused on sensitivity analysis techniques in order to define several construction processes [6], and also to get the most efficient outputs [7]. Some construction trends are trying to improve existing auxiliary means and make them more sophisticated [8].

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The practice of bridge launching worldwide is quite extended. But the need to improve and systemize the construction system is promoted by the fact that today there is a competitive and global market demanding economical, sustainable and safe ways to adapt. There are a lot of studies, which tackle the problem of selecting the best alternative to be built [9]. Simulation techniques are usually carried out in order to design and calculate most bridge construction systems [10].

Finally, one of the most important issues regarding the launching construction system is the analysis of the patch loading phenomenon. The influence of the longitudinal stiffener configuration [11, 12], the use of hybrid sections, and international regulation by means of the different national or international codes [13] are not unified yet. Furthermore, the interaction between the bridge and the pushing device also plays an important role [14].

The main objective of this paper is the investigation and development of a new-patented launching method to use in steel bridge construction [15]. This system allows the launching of longer, higher and wider structures; especially focusing on environmental protection and decreasing costs [16], by means of replacing the temporary elements used nowadays. The relationship between this new bridge launching method (NBLM) and the new continuous pushing system [17] has been also studied.

1.1 Disadvantages of contemporary bridge launching methods

Many common disadvantages must be solved [18] when using the new bridge launching method that is described in the present paper. All of them are related to the essential conditions of the construction system and also to the pushing machinery used to move the bridge. The combination of these issues has provided a lot of solutions in the last fifty years [19]. The main problems are described below.

Firstly, the bridge is subjected to different structural schemes during the construction stage – cantilever span configuration – and when the bridge is completely erected and the serviceability state begins – continuous beam –.

Secondly, the variation of the boundary conditions during the launching process implies the alternate sign of the bending moments along the whole deck, and their absolute values are higher than the service ones; this is the most critical factor for the first two spans from the launching nose (see Fig.1). So different auxiliary means and temporary constructions must be used during the launching, even when the bridge span is not too long, such as [4, 5, 19]: long launching nose, temporary piers, temporary stayed system and so on.
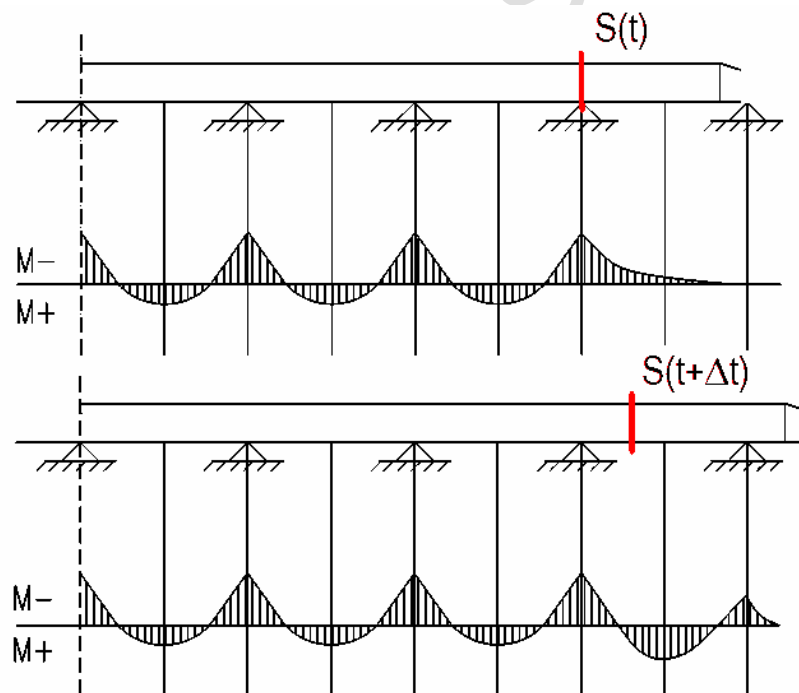
Fig. 1. Alternate sign of bending moments (M) in a specific bridge section (S) between two time steps (t and t+Δt).

The third key aspect related to the design and construction of launching bridges is the fact that every cross section along the bridge is going to pass on the bearings at the top of the piers when the structure is being pushed ahead. Not only the designed support sections, but also any section, must resist the shear stress and patch loading phenomenon. This is an important instability problem for slender webs, such as the ones used for steel bridges [20].

Fourthly, a very important percentage of the global cost of the launching bridge is due to the delays derived from the successive running and stopping phases that all pushing systems imply (the well-known methods are friction jacks, bars, cables, etc., and all of them are pulled out by means of hydraulic machinery) [21]. Besides a readjustment of those elements is needed when each new construction segment is added at the rear of the deck. In this way, a continuous launching system is not possible nowadays and the schedule of the works is frequently delayed or interrupted.

Finally, at present, no global method can be used to launch bridges, because they are highly dependent on particular considerations. Furthermore, the reuse of the auxiliary means is not affordable in an economic or safe way, and they need to be improved by a rigorous maintenance program each time the bridge is launched.

There have been some fatal structural failures in launching bridge construction History [22]. All of them are related to the most critical conditions during the launching procedure, such as the maximum bending moments on the cantilever span, the patch loading phenomenon on the bearing sections [23].

2. Description of the new launching method

2.1 Description of the proposal

The design of the new bridge launching method has a basis, which consists of the placement of the last span of the bridge to be launched on the pushed deck [15]. That segment is called double-deck and collaborates in bending and shear resistance of the weakest sections when the structure is moved and gets the maximum cantilever span. The system is able to transmit the global stresses trough a shear connection mechanism by means of a joint between them, i.e. HFSG bolts. Fig. 2 (a) shows a characteristic launching phase, when the nose is reaching the next pier.
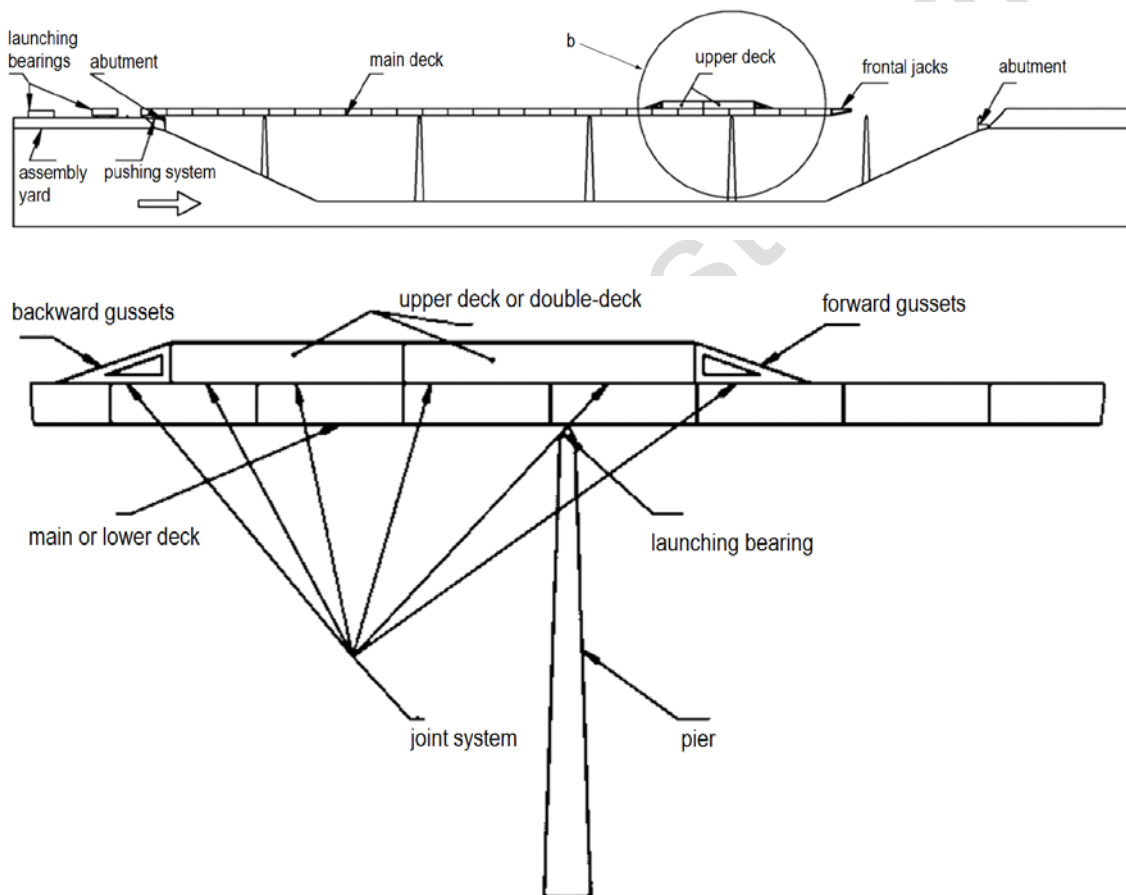

Fig. 2. New method (NBLM) launching phase of main span: (a) overall view; and (b) detail of double deck.

In this way, the critical sections launching are reinforced in order to support the high values of the bending moment and shear forces. Usually, these values are bigger than

the serviceability limit states once the bridge erection is completed. Fig. 2 (b) shows a detail of the structure on one of the piers. The main actions to be taken into account are:

- Bending moment, shear and torsional effects, and patch loading, which are maxima in the section that supports the whole cantilever of the first bridge span.

- The opposite sign of the bending moments in every section of the bridge, when the structure is going over each pier (see Fig. 1).

So this invention makes both the bridge structure and the double-deck work together and the use of the material is highly profitable for the segment on the deck, which is used as reinforcement during the launching phase and is also the last span of the definitive structure (see Fig. 2). Besides it is not necessary to modify any element of the system, neither their position, once the launching has started, nor make any correction in the final system that ensures the two decks work together. Thus, the construction process is safe and stable in all its phases, involving simple and repetitive operations (with a high degree of prefabrication), which are susceptible of automation and continuous monitoring [24]. This leads to a shorter execution time and lower costs [25]. The originality, advantages and effectiveness of the new method with respect to the traditional ILM have been highlighted in Fig. 3.
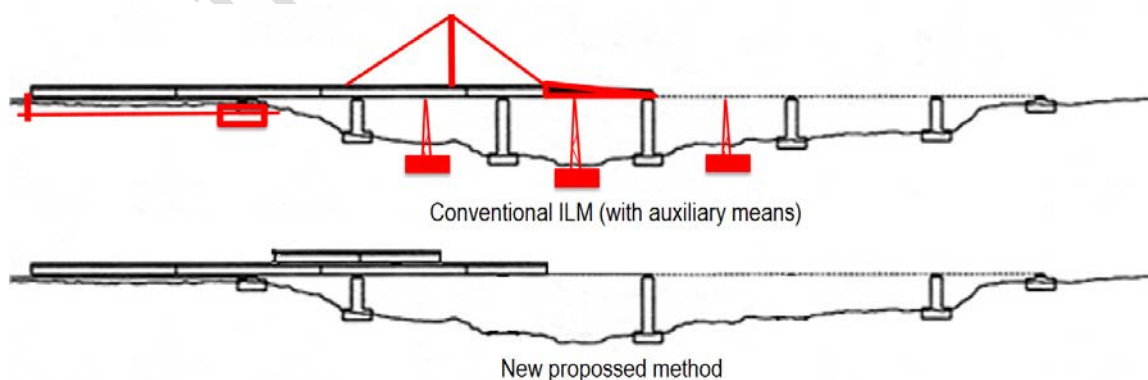
Fig. 3. Comparison with the conventional ILM and the new proposed method showing the auxiliary means in red.

The main characteristics to be achieved with the new proposed method are the following:

- Both steel and composite bridges.

- Cross sections such as simple box, multi-cellular box, two-girder, multi girder and deck frame are suitable; the width of the deck ranges from 14 m to 25 m.

- Constant depth sections (although variable depth may be achieved by means of gussets located at the bottom of the deck, on the location of the piers).

- Spans up to 150 m are designed and bridges up to 1,500 m can be launched.

- Straight or curved guidelines can be launched, both plan and elevation views.

- Concrete slabs (the upper and also lower slab if double composite action is desired) can be casted before the launching.

The proposed method can be supplemented with other structural elements, depending on the maximum span of the bridge, specifically:

- Small launching nose to regain the deflection at the head-on section,

- Devices to disassembly the joint between the upper deck and the main one,

- Construction system to erect the lateral span once the launching is completed (preferably, by means of cranes).
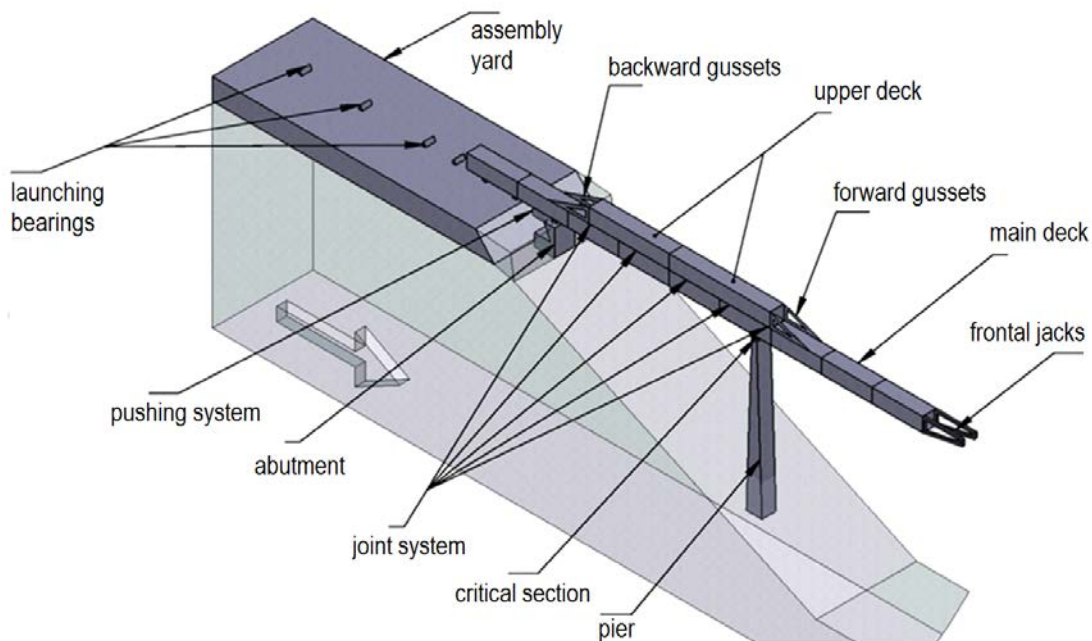
2.2 Main structural elements

The main elements that represent the new method are the following (see Fig. 4):

- **Double-deck**, it is located on the deck of the bridge to be launched, and it configures the lateral span of the final structural scheme. It is 75% of the length

of the lateral span. Its position is calculated trough an optimization methodology. It depends on the nose deflection and the maximum stress on the steel plates of the launching support sections (see chapter 3).

- **Joint system**: these elements make two decks work together and resist the dead load and other load cases, such as operating wind conditions, in the critical section. It is based on M30 high strength friction grip bolts and allows it to be easily disassembled.

- **Gussets**: they are used to transfer the stresses from the main deck to the upper deck, by means of a joint system and shear connection mechanism. The forward elements will also be used as a launching nose if the final assembly of the lateral span is by launching.

Fig. 4. Main structural elements of the new bridge launching method.

- **Frontal jacks:** It is a small device composed of a light truss to compensate the elastic bridge deflection by means of a hydraulic lifting system. In consequence,

it is used to reach the pier in the correct position. The lower flanges are inclined in order to make the passage over the pier easier.

- **Reinforcements of the box sections:** the box section incorporates a plate joining the upper flanges, in order to resist the bending torsion moment. It is needed in the first two spans of the structure, especially in curve guideline bridges.

Other secondary elements can be considered, such as the last span assembly system (rather by cranes) and a new continuous and reversible pushing system [17].

2.3 Advantages and disadvantages

The most important advantages of the NBLM with respect to the actual disadvantages described in the subsection 1.1 are the following:

- The critical sections, decisively conditioned by the construction system, are reinforced during the launching process and specifically the weakest section in the maximum cantilever position. In addition, the last bridge span does not belong to the forward section (as in the traditional launching systems). In consequence it is not necessary to provide larger cross section or reinforcement with respect to the calculated size in the serviceability limit state.

- The span that can be launched is longer than the today conventional lengths, even using auxiliary means.

- The torsion behaviour of the first two launched spans is greatly improved, as the section is closed with the last lateral span located on the deck, as well as with the thin steel plate to ease the placement of the last span. This plate also serves as a permanent formwork for the slab or to support the prefabricated slabs.

- This reinforcement is mostly done by a part of the final structure, so the use of materials is much better and avoids the use of auxiliary means.

- It also facilitates the launching of irregular or not common geometries (such as straight or circular guidelines), since the last span is assembled independently from the launching technique.

- The construction process is safe and stable in all its phases, involving simple and repetitive operations (with a high degree of prefabrication), which are able to be automated and continuously monitored.

- The new method is suitable for a wide range of cases, such as steel or composite sections and road or railway traffic bridges.

- As a consequence of the span extension, the construction is carried out in the most sustainable way and the environmental conditions are guaranteed, because the impact on the valleys is minimised.

With respect to the disadvantages, the following can be pointed out:

- The reaction on the forward support piers will be higher than the forces obtained in the conventional launching methods, because of both the longer span and the double box weight. The reaction on some of the provisional launching bearings located in the launching yard during the earlier construction phases will be higher. Depending on the pushing system, these issues could be advantages, such as in friction pushing systems.

- There is a minimum span limit, below which the launching process can be carried out without the contribution of the double deck reinforcement. Only needing a launching nose in order to obtain a competitive solution.

- As occurs in any general launching procedure, the two critical phases are the following:

o The first span launched, because there is not enough rear deck in the launching yard or vertical reaction to push the deck by a friction system.

o The last phase, when the lateral span is completed.

However the new proposed system makes these problems more easily solved.

All in all, the proposed method leads to lower execution times and costs.

3. Numerical Models

In order to numerically justify the NBLM, the finite element technique is used. In this way, the numerical simulation was carried out using the ANSYS Workbench software [26]. With this simulation technique, the structural response of the basic parts making up the bridge are analysed in great detail, saving costs and time in relation to experimental tests.

Only the first two spans of the bridge are modelled since the whole deck behaviour can be simulated accurately by adding the corresponding boundary conditions. Figs. 5 (a) and 5 (b) show the geometrical models used, including the main cantilever span of 150 m and the adjacent span from pier nº 1 to the abutment. This model will be widely described and corresponds to the critical phase of launching. Other different models used in previously designed stages are very similar.

A general cross section has been designed, with four longitudinal stiffeners along the webs and the bottom plate. The thickness of the plates has been initially set at 0.025 m (see Fig. 5 (c)). This size was increased to reach its maximum value of 0.035 m, in the bridge segment located over the pier during the critical launching phase. The triangular cell design will be discussed in subsection 3.3.

(a)

(b)
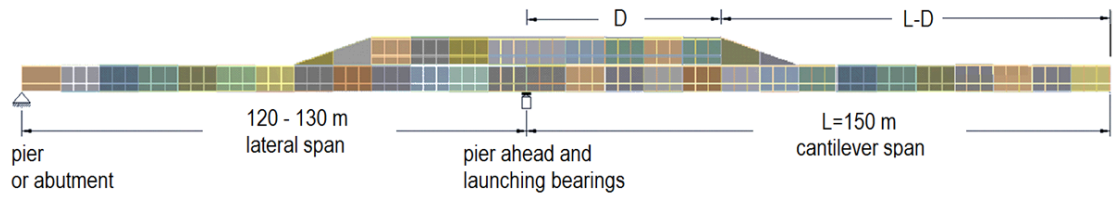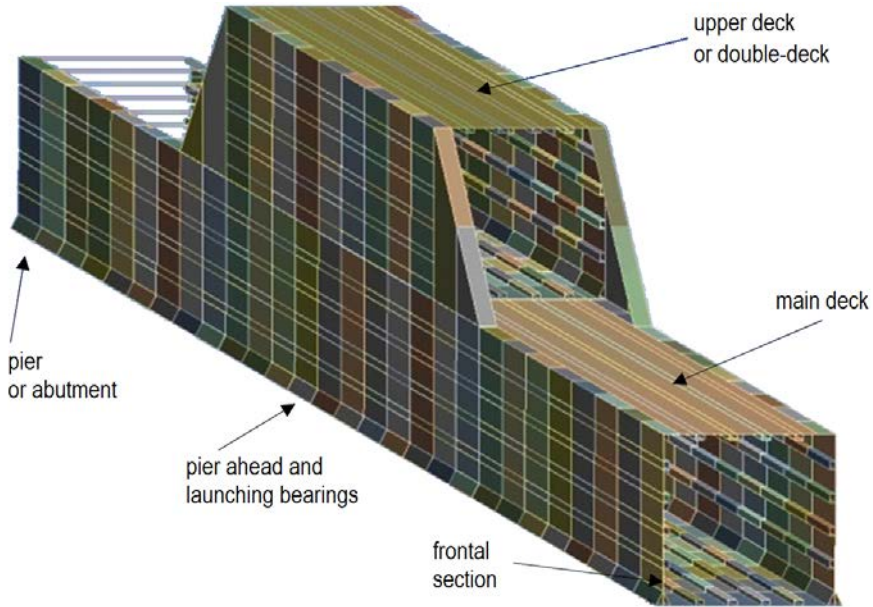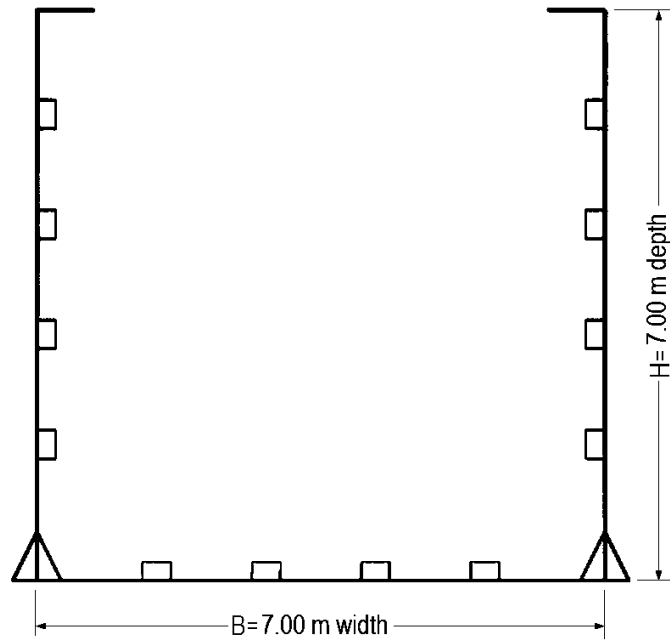
Fig. 5. Geometrical model for the case of the most critical launching phase: (a)

longitudinal view; (b) isometric view and (c) cross section of the deck.

Some construction details are incorporated into the geometrical model, global buckling

modes are avoided and the new analysis allows then exact prediction of the local

buckling modes for the critical launching phase. The final model is made up of:

- Different thickness of the plates along the deck, depending on their location
  (main plates, i.e. web, bottom and triangular cell varying from 0.02 to 0.035 m).

- A very short launching nose (10 m) to support the hydraulic jacks to regain the
  deflection at the forward pier, weighing about $10^4 \times \dfrac{N}{m}$.

- Horizontal continuous plate connecting the upper flanges of the upper deck and
  the upper flanges of the forward main deck.

- Transversal frames with a separation of 10 m one from the other.

- Bracing for the upper part of the box.

In order to explain the numerical models developed to study the NBLM, subsection 3.1

describes the general finite element model.

Subsection 3.2 analyses and optimizes the double-deck system. In order to control the

maximum value of both von Mises stresses and web panel deflection an adequate

planning of the stiffeners - including a triangular cell - is considered. A variation of the

thickness of the bottom plate is defined.

Finally subsection 3.3 shows how the sizing of the triangular cell has been conducted;

the patch loading resistance is ensured by means of this element along the lower flanges

of the bridge. An exhaustive and very complete work about the combination of some techniques of the stiffener system and a lot of related details can be seen in [27].

3.1 Finite Element Model

In this work, the FE model is based on the following:

- Finite elements used: Two types of elements called SHELL 181 and SOLID186 have been used, taking advantage of the bonded-contact models between them, in order to reproduce the behaviour of the welded joints.

- Material properties: In this simulation, one material model is defined: the bilinear plasticity model for steel, with isotropic hardening. The corresponding characteristics are summarized in Table 1.

Table 1: Material properties.

- Meshing: the geometrical model is meshed by a hex dominant method, differentiating two bodies: the bearings and the deck itself. The element size ranges from 0.05 m to 0.03 m near the location of the supports.

- Loads and boundary conditions (see Fig. 6): in order to reproduce the structural behaviour of the bridge during the critical launching phase we have considered the following:

  - Standard Earth gravity of 9.81 m/s^2 value.

  - Bending moment at the rear of the deck of $1 \times 10^7 \, \text{N} \times \text{m}$, based on a previous 2D analysis in which the force reactions on each pier were calculated. Shear forces at the rear of the deck are absorbed by the supports.

- Two launching supports of 1.0×0.6 m at pier nº 1, in which the rotation angle has been controlled and compared with the previous 2D analysis. Vertical displacements are not allowed and horizontal movement is restrained in one of the bearings.

- Simply support at the rear of the deck, 130 m long for pier nº1, just on the abutment and near the location of the pushing system. Displacements are not allowed.
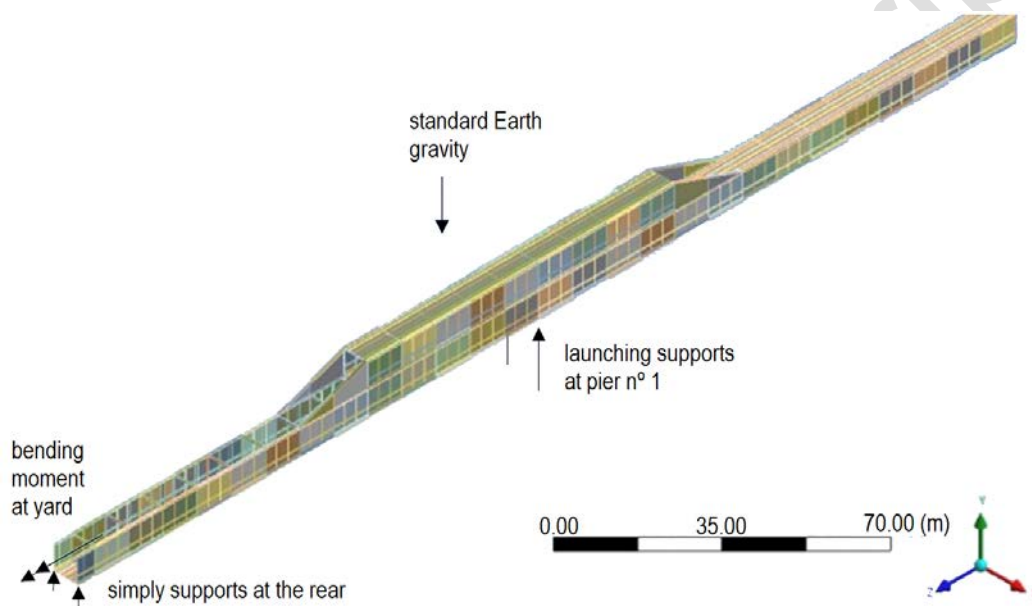

**Fig .6.** Boundary conditions and loads applied to the model.

The present nonlinear static structural problem was solved by the full Newton-Raphson option for all degrees of freedom. A non-symmetric solver including the adaptive descent option [28] is used, discarding the inertial effects. With the aim of achieving an initial solution for the lineal buckling analysis it was necessary to perform a linear static structural analysis. Then a linear buckling analysis is performed and the normalized values of the initial defect of each mode are calculated. Finally, the plasticity of the material and actualization of the geometry in each step load is taken into account to obtain the failure load.

3.2 Numerical analysis of the double deck system

In order to justify the new proposed launching system we show the numerical models and methodology applied to solve this nonlinear problem. Using conventional techniques it is not possible to get spans of around 150 m without any auxiliary means. The shear force, the bending moment and the deflection at the launching nose would be inadmissible and the steel section will collapse.

(a)

(b)
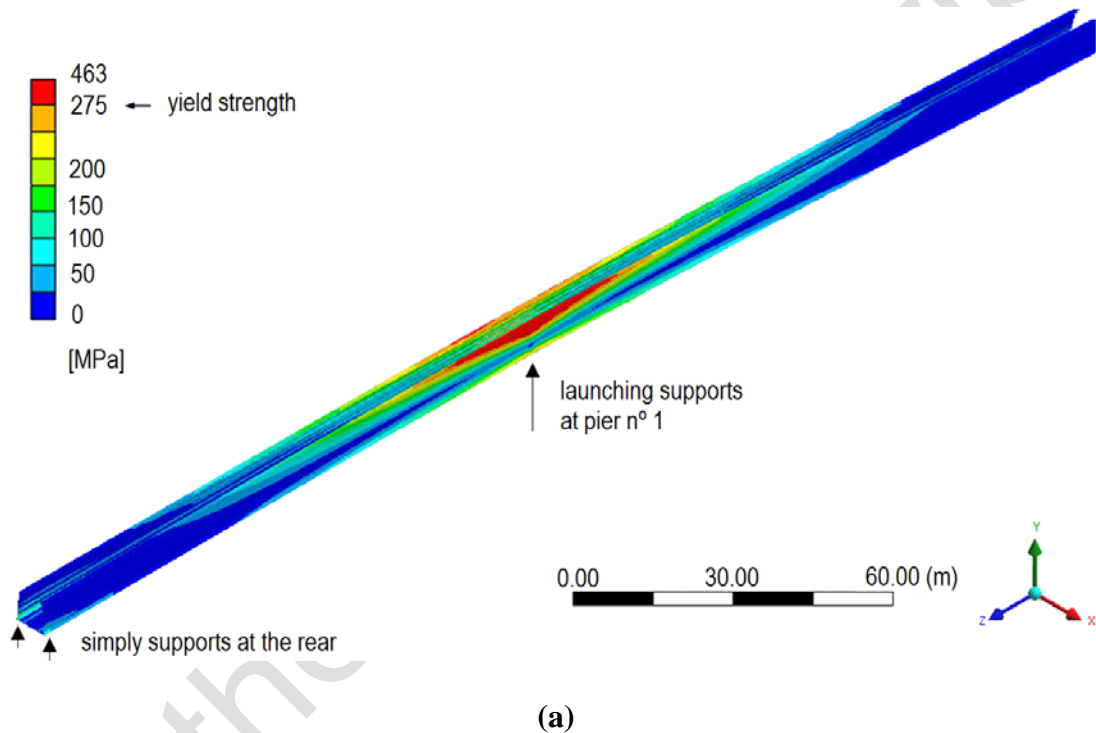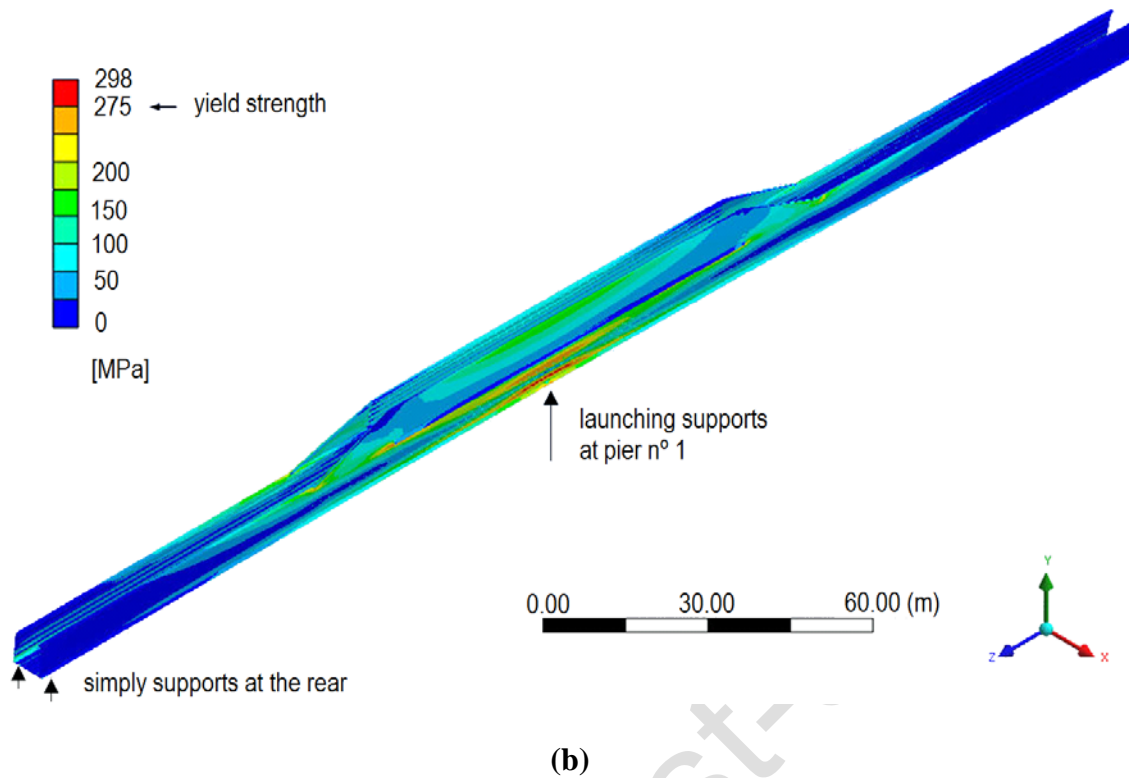
Fig .7. Equivalent von Mises stress distribution in a 150 m cantilever span: (a) without auxiliary means; (b) new self-supporting double deck system.

Fig. 7 shows the von Mises stress distribution for the simple and double deck solutions. It is possible to observe the collapse or failure of the structure because the colour red shows the plasticization zone. The section on the pier collapses because of the local yielding failure mode at 40% of the maximum load (1.5×10^7 N) if the double deck system is not used (see Fig 7 (a)). However if it is used Fig 7 (b) shows how the collapse load reaches 60% through the local buckling failure of the web above the supports. This is the main reason why the self-supporting double deck was designed.

3.2.1 Double-deck optimization based on DOE analysis

In order to determine the most efficient location of the double deck, a design of experiments (DOE) analysis using a central composite design (CCD) is carried out. The

initial conditions are the long span - 150 m - and the total length of the lateral span - 120 m -. Besides, the deck segment on the main structure must be 90 m long because the last launching phase finishes when the launching nose passes approximately 30 m past the last pier.

The limits of the parameters are chosen starting from the objective to be achieved (150 m span). Structural response surfaces are calculated from the FEM models through the DOE method and the influence of each input parameter on the structural behaviour is obtained. In this way, it is possible to obtain the optimal values of the parameters considered in the simulation and the best configuration of the double-deck.

The input parameters that will be defined are the span length (L), deck depth (H), the distance from nose to forward plates (L-D) and the thickness of the plates of the box section (e). Their variation range is shown in Table 3.

Table 3: Input parameters for the double-deck position DOE analysis.

The influence exerted by each of the parameters is shown in the sensitivity analysis in Fig.8. The deflection/span ratio at frontal section and the maximum equivalent von Mises stresses (lineal stress) are selected as output parameters.
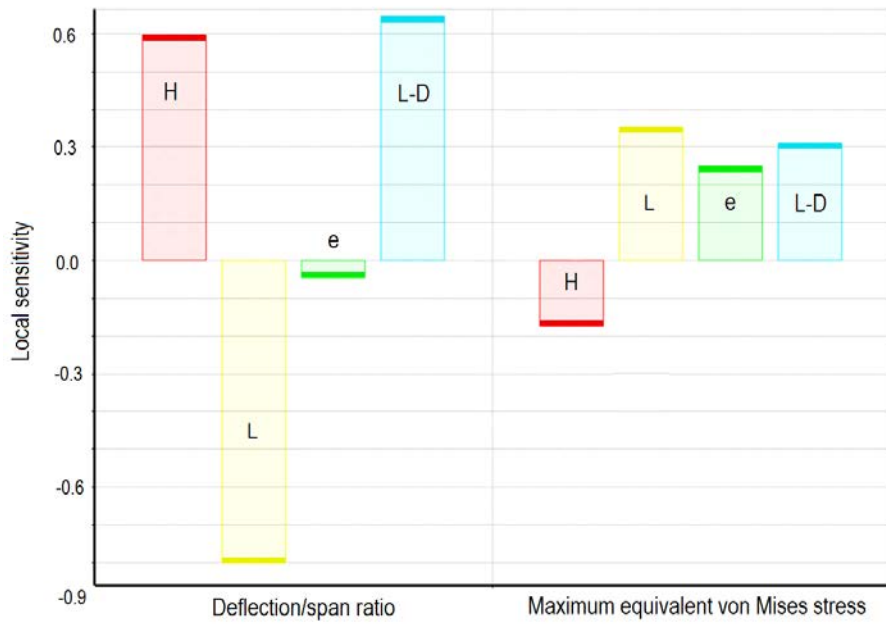
Fig. 8. Sensitivity analysis to determine the best double-deck configuration.
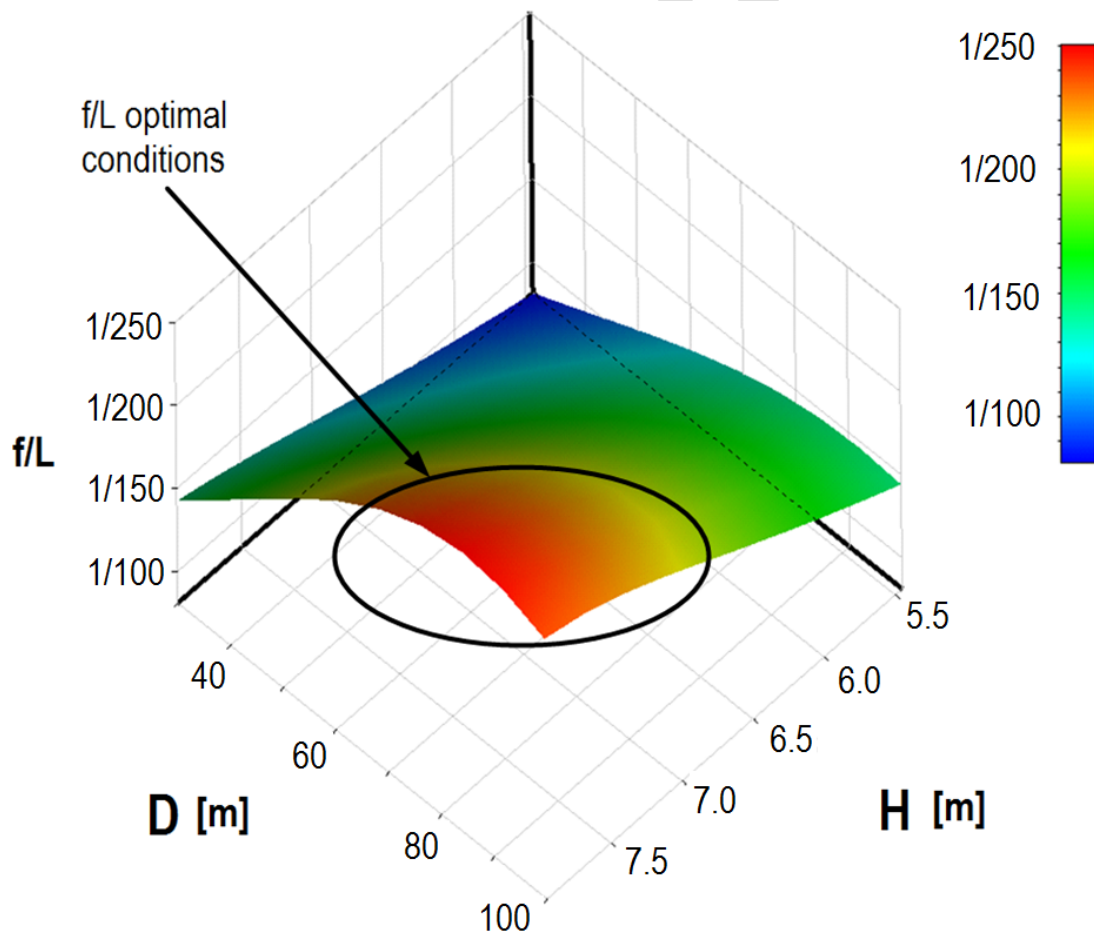
As a result of this study (see Fig. 9), the optimum depth of the deck is 7 m, taking into account that the more slender structure generally implies more expense. The position of the double deck segment, measured form the launching nose ranges from 100 m to 110 m. The exact value of this parameter must be defined in each case, and combined with the launching procedure, the lower concrete slab, etc.

In addition four triangular gussets 20 m long are assembled at the beginning and the end of the upper deck in order to attain a smooth transfer between them.

3.2.2 FEM analysis

From the previous double-deck location analysis distribution of 55% forward and 45% back of the upper segment relative to the support point is studied. Plasticity of the material, non-linearities and large deflection are taken into account.
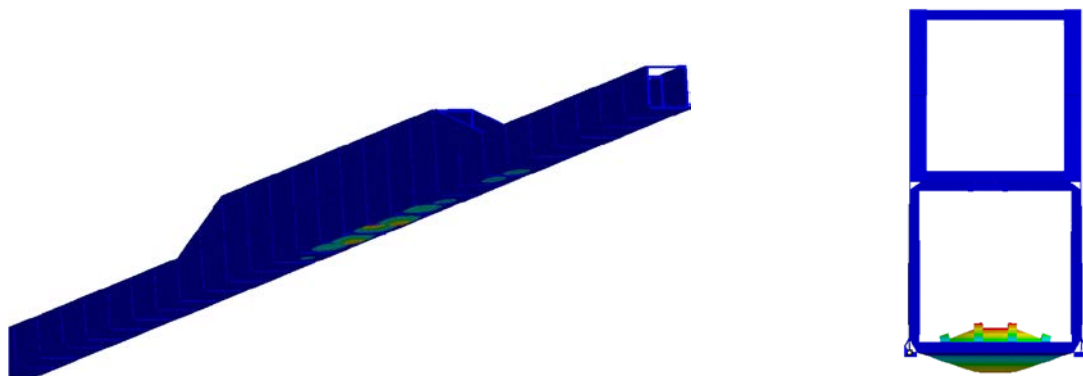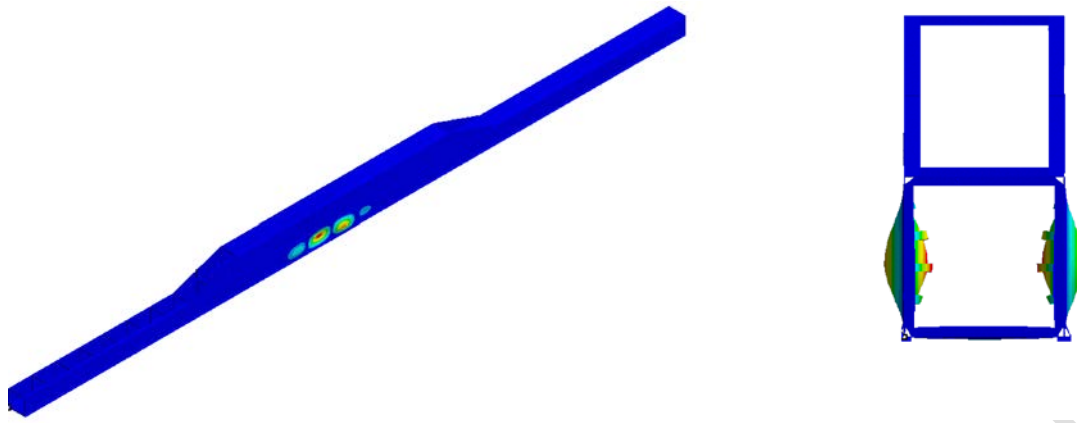
Fig. 10. First two buckling modes of the structure.

In this way, the main local buckling shapes of the structure corresponding to both webs and bottom plates failures near the location of the bearings, are clearly be seen (see Fig. 10). As it was mentioned above, this is the critical factor determining the sizing of the launching method, due to high and concentrated loads produced by general bending and support force reactions. Cross sections that are not designed as a final support section and that do not have any special transversal frame, are also subjected to this load.
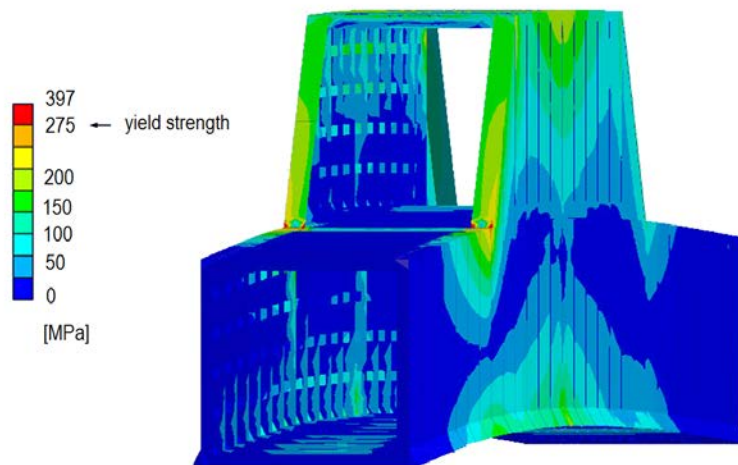
(a)
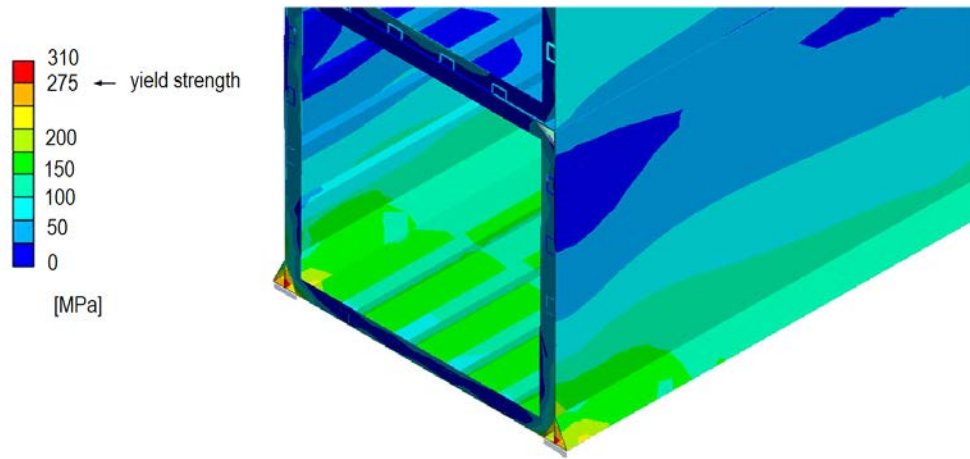
(b)

Fig. 11. Equivalent von Mises stresses in a 150 m cantilever span: (a) structure deflection; (b) detail of the triangular cells of the supports.

The maximum deflection at the launching nose is 2.3 m, i.e. the deflection at the nose - maximum cantilever span ratio is in the range of 1/130 – 1/140. However, the large stresses on the provisional support section are controlled because the general shear forces and bending moments are distributed over a bigger section, and this also contributes to increase the patch loading resistance, which is dependent on the compression stress at the bottom of the section. 100% of the design force reaction $(1.5 \times 10^7$ N) is calculated, and reinforcement during the launching procedure is a part of the structure itself. The von Mises stresses results are shown in Fig. 11.

Summarizing, the self-supporting double deck method solves the most important problems during the launching phases, and it is not used at present [29]

3.3 Numerical analysis of the triangular cell

A simplified numerical model is used to study the stiffness of a single panel or segment subjected to a point load such is a bearing plate. This segment is 5 m in length, 7 m in

depth and 0.025 m thick. The provisional launching bearing plate dimensions are 1.0×0.6 m, usually located on the definitive bearings. The contribution of this element together with the longitudinal and transversal stiffeners will be taken into account in the global model.

Two different configurations are studied. On the one hand, a double triangular cell is modelled. Fig. 12 (a) shows the cell formed by two plates (one inside the box and the other outside the web). The triangular stiffener is so efficient that it counterbalances the higher expense due to the second plate and the welding along the whole deck.
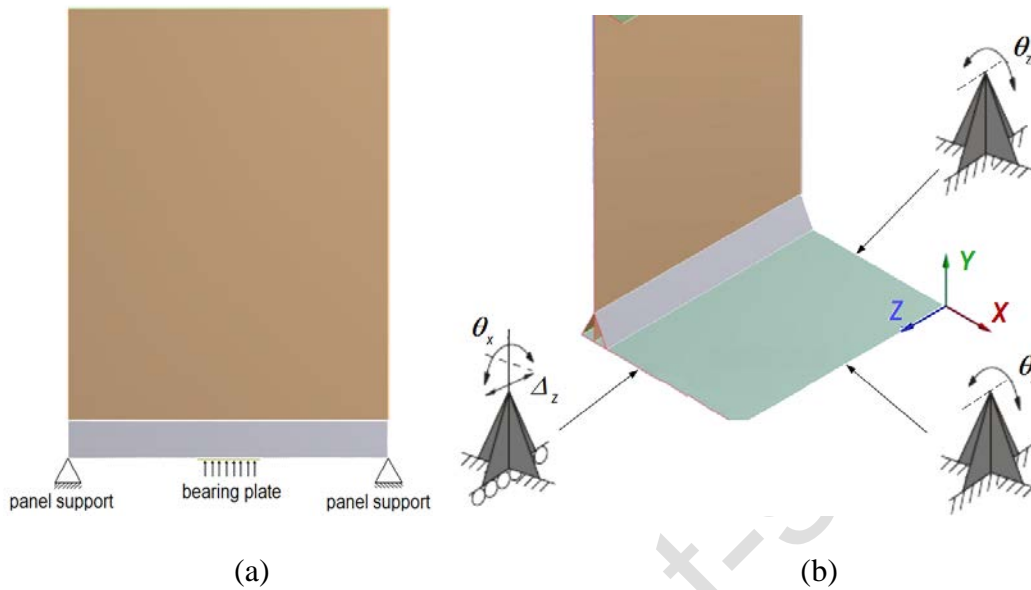
On the other hand, a triangular cell with only one plate, inside the box section, is shown in Fig. 12 (b). This stiffener is a conventional solution, but it requires thicker plates in order to control the maximum local stress.

The geometrical models are meshed by a hex dominant method with a meshing parameter of 0.01 m (see Fig. 12(a)).
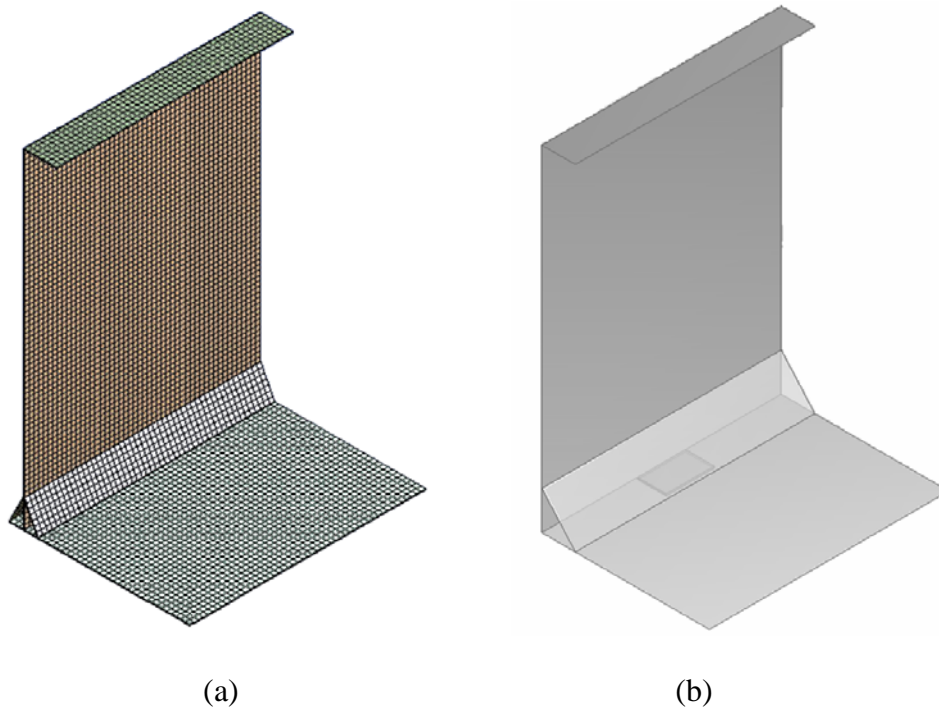
(a)   (b)

(a)   (b)

Fig. 12. Boundary condition details: (a) location of the supports and bearing plate; (b) isometric view of the panel and detail of degrees of freedom allowed.

- Loads and boundary conditions: in order to reproduce the real behaviour of the panel during the critical launching phase, we have considered the following:

  - Force reaction applied to the plate bearings (Fig. 13 (a)): 1×10^7 N each bearing, corresponding to 75% of maximum reaction during the critical launching phase.

  - Boundary symmetry conditions and restrained degrees of freedom are show in Fig. 13 (b).

3.3.1 FEM analysis

A comparative analysis between the designs of the triangular cells is carried out, paying special attention to web stresses and overall deflections. As was said before, the

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equivalent von Mises stresses during the bridge launching process must be lower than 50 - 60% of the stress yield strength value, for there to be no plastic strain.

The sensitivity analysis defines the basic geometry of the cells (see Table 4) taking into account the maximum stress on the webs. The most effective triangular cell distributes the concentrated load over more distance from the support point.

The total depth of the cell is 0.8 m, however the final optimization after the preliminary analysis will define this value. The DOE input data of the cell of the model is shown in Fig. 14 (a, b) and Table 4.

(a)   (b)
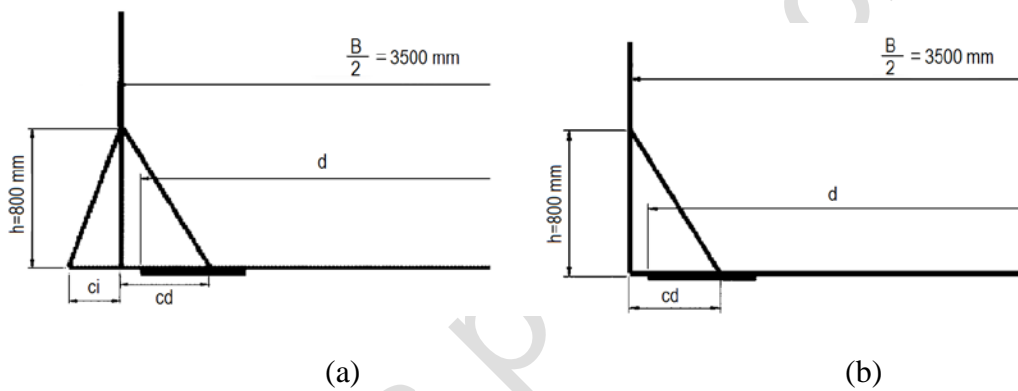
Fig. 14. Geometrical models, dimensions of the triangular cells and bearing location: (a) double cell and (b) single cell.

Numerical results for the two FEM models are shown in Table 4.

**Table 4:** Numerical results of the triangular cell configuration analysis.

3.3.2 Optimization based on DOE analysis

After achieving a suitable FEM model able to reproduce the distribution of the point load, an optimization based on DOE analysis is done. The aim of this analysis is to assess the effect of the most important parameters on the system´s structural response

and define the optimum depth of the triangular cell. It behaves as the first and heaviest longitudinal stiffener of the compression flange of the deck.

The fact of carrying these simulations out using an FEM model makes the pre-sizing easier. Subsequently the highly non-linear problem, including local buckling phenomena, will be undertaken based on the previous design [30].

The most relevant parameters considered and their influence on the plate stress is shown in Fig. 15, which represents the sensitivity study of the bottom, web, outer and inner plates, when a complete triangular cell is used.
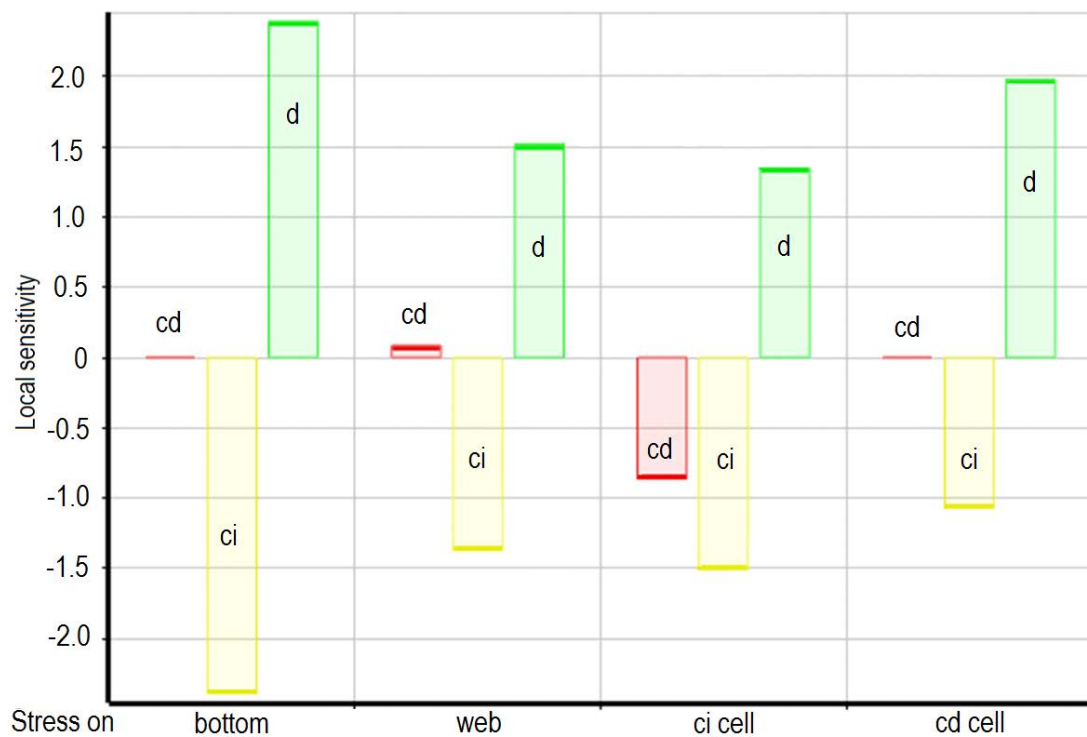

Fig. 15. Sensitivity analysis to determine the optimum triangular cell design for a

complete cell.

The use of the triangular cell is favourable to center the load over the web, so the eccentricity decreases considerably and the most important parameter that influences the buckling phenomena is controlled [31]. The total width of the triangular cell is the other

most influential parameter of the problem, being more relevance when the triangular

cell is only on the inside.

As a consequence of results, a high degree of efficiency of the complete triangular cell

(both inside and outside plates) is obtained. The maximum von Mises stresses decreases

about 30%, and vertical load is well distributed throughout the depth of the web, with

common nominal thickness of 0.025 m and stresses lower than the yield strength.

Finally, the individualised optimization of the triangular cell is carried out (without any

other rigidity system, such as longitudinal, vertical stiffeners or transversal frames). Fig

16 shows the evolution of von Mises stresses on every plate as a function of the cell

height. The optimum design value is 0.6 m high.
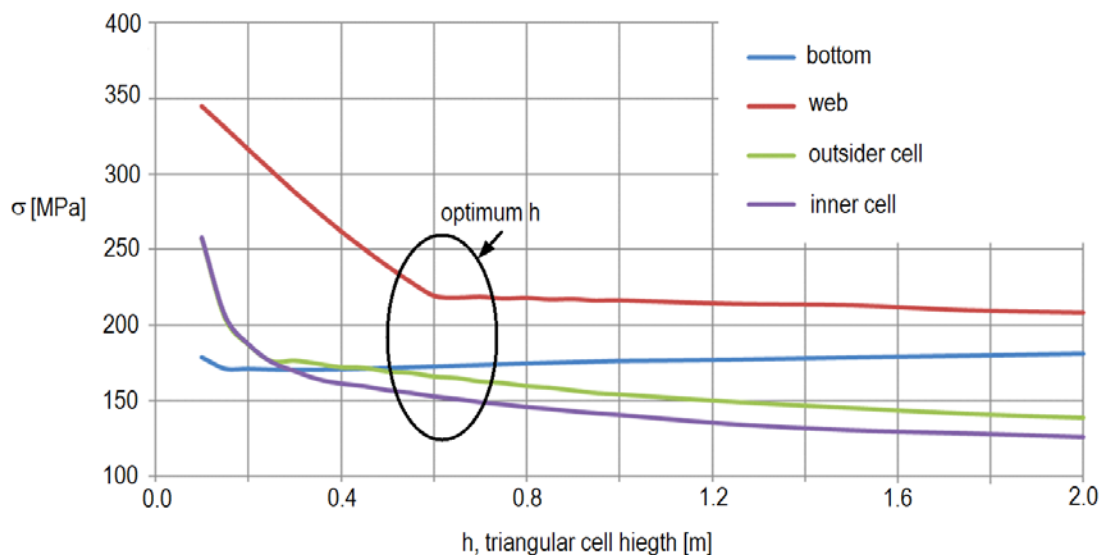
Fig. 16. Von Mises stresses evolution as a function of the cell high.

The final design is shown in Fig. 17. The stress distribution on every element has been

calculated and the maximum von Mises stress is about 2×10^8 Pa, lower than the steel

yield strength limit. Patch loading phenomenon is controlled by means of the triangular

cell and the general longitudinal stiffeners, taking into account the average thickness of the web and cell plates.
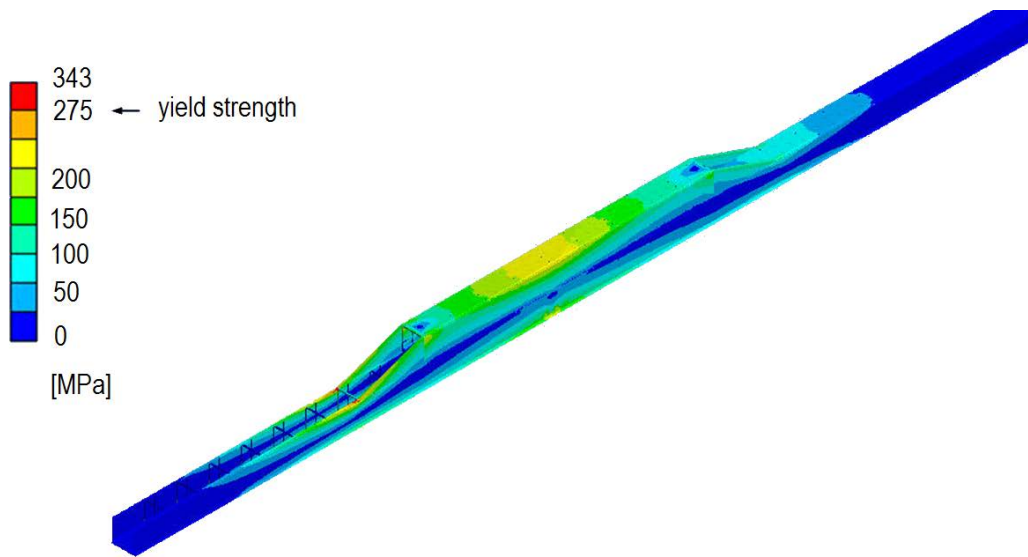
Fig. 17. Von Mises stresses on every plate of the bridge during the critical launching phase, 150 m cantilever span.

4 Case study comparisons: Regueirón Viaduct, Spain

Regueirón Viaduct has been recently built in Asturias (Spain, 2009) [32] and is chosen as an example to apply the simulation of the new construction method described in this paper.

The bridge is 560 m in length and 23 m in width. It is a curved bridge of 600 m radius. The cross section is 4.28 m constant depth and 7 m bottom wide. The total length is divided into 7 spans of 64+84+92+84+84+84+68 m. The elevation guideline is a constant ramp of 5.80%.

A conventional launching method by means of an auxiliary and temporary cable stayed system was carried out. There were six assembly phases (144 + 48 + 80 + 112 + 128 + 48 m) and six launching stages (44 + 84 + 136 + 112 + 48 + 136 m). Fig. 24 shows the fifth launching stage.

**Fig. 18.** Fifth launching stage of the Regueirón Viaduct.

The assembly yard was 140 m long and it was located behind the first abutment. Segments up to 16 m in length were previously transported from the factory to the work site and finally assembled over four concrete provisional slabs. The stayed tower was 20 m in height and was located at 92 m from the nose with a pair of stays of 88 m in length. This auxiliary system was used to minimize the deflection at the launching nose and the huge bending moments during the launching phase. The pushing force was provided by two hydraulic jacks on a cable pulling system.

4.1 Technical comparison: launching possibilities

In order to reproduce the complete launching procedure on the Regueirón Viaduct using the NBLM procedure, a continuous beam model is carried out.

The numerical model of the bridge is 690 m in length with straight plan guideline. Reaction forces on every pier and on the pushing system bearings are calculated, as well as the reactions in service limit state. The maximum force reaction on the pier is about

2×10^7 N. Each bearing must support the half reaction, 10^7 N (see Fig. 19). The calculation that is carried out in this work take into account a 1.5 magnification coefficient due to the possible curved deck guideline. The maximum reaction considered in the FE analysis described in chapter 3 is 1.5×10^7 N on each bearing.
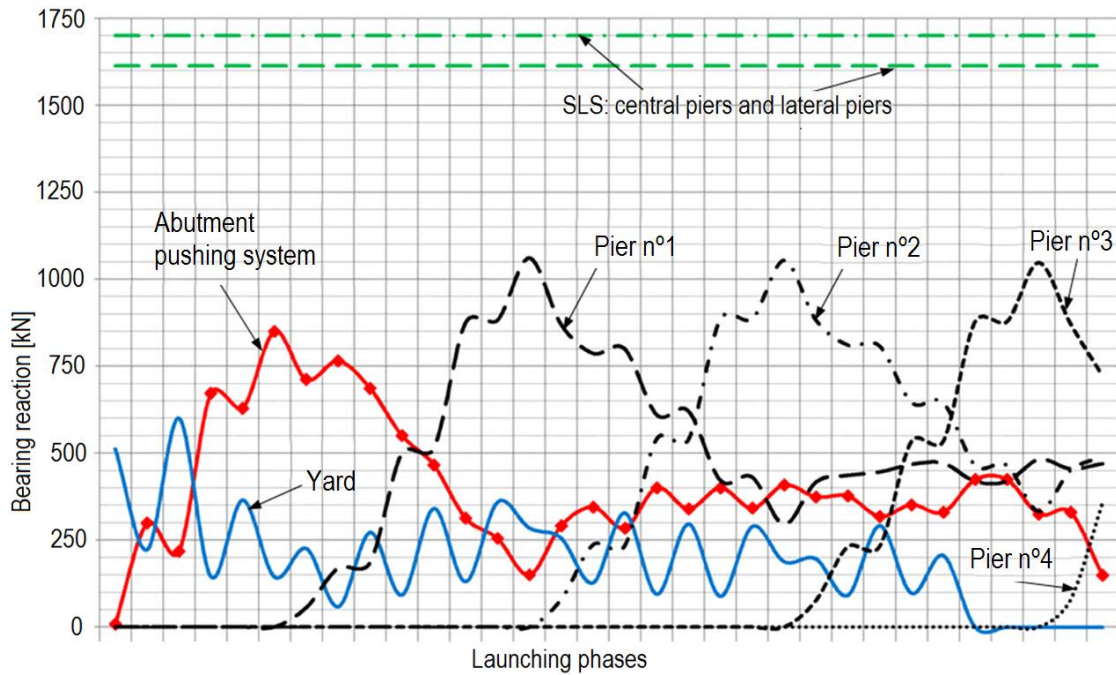
Fig. 19. Vertical reactions on every support section, NBLM simulation.

The technical viability of the NBLM is concluded, and the structure is able to support the stresses due to the construction method. Summarizing:

- The maximum force reaction is about 1.5×10^7 N on each bearing. This value is used to study the patch loading resistance.

- The nose deflection/total span ratio is within the range 1/130 to 1/150. This is a conservative value.

4.2 Economic comparison: limits

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At the end, more relevant economic aspects are taken into account, in order to determine the competitiveness of the new NBLM.

The comparison is done between the Regueirón Viaduct real construction system (92 m long span) and the new method presented in this paper (150 m long span bridge).

Units are summarized in four main issues (see Table 5); the conventional method includes provisional cable stayed, launching yard, abutment configuration and cable pulling system. NBLM considers the equivalent issues, the assembly of the last span of the structure and takes into account the saving due to the decrease in piers and bearings, because the main span is more than 50% longer than the conventional system one. There is also an important saving in the pushing system related with the traditional auxiliary equipment (e.g. long launching nose, cable stayed tower, provisional supports, etc.) is not needed by using NBLM.

Table 5: Comparative costs.

At least, a 3% saving can be achieved after the paid-off time - eight years - has been accomplished.

5. Conclusions

The aim of this paper is to present a new bridge launching method in a clear way, optimizing the use of the design elements in the structure itself. This system is technically and economically competitive compared to existing procedures. Auxiliary means are not necessary and the material is efficiently used, where needed. All this is done in a safe and sustainable way whilst protecting the environmental conditions.

On the one hand, main structural variables are identified and numerical simulation by FEM developed. To do that, several numerical models are made and studied within a wide range of cases. On the other hand, the most important variables are optimized by means of sensitivity analysis and design of experiments (DOE).

Summarizing, the main conclusions are the following:

- A new efficient bridge launching method is presented, for assembling steel bridges of up to 150 m. span, without using auxiliary or temporary means that are usually expensive and non-reusable.

- The triangular cell along the lower flange is a very important stiffener that contributes to the patch loading resistance. Von Mises stress on the web decreases about 30% when 0.025 m thick plates are used.

- Common plate thickness values are obtained. Critical segment webs and triangular cell are 0.035 m thick; bottom plate is 0.02 m thick. However, the double-deck (or upper deck) maximum thickness is only 0.02 m. Therefore the lateral span does not have to be over-sized using the new construction system.

- Serviceability limit state considerations are taken into account. The maximum von Mises stress on the critical element is 25% lower than the yield strength.

- Once the equipment used to assemble the new launching system has been paid off, a saving of about 3% can be achieved, based on the case study presented in this paper.

Finally, authors suggest future research lines about the detailed definition of the joint between the lower deck and the upper segment by means of HSFG bolts and the system used to disconnect them. Also, other different bridge cross sections can be studied and optimized, such as inclined webs, two-girder beams, or variable depth decks.

5 Acknowledgements

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