Design Improvements for Patch Loading Resistance in Bridges During Launching

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SUMMARY

When dealing with bridges erected by launching, all cross sections, either stiffened or unstiffened, will be subjected to patch loading when passing over the launching rollers on the piers. In this case, an accurate assessment of patch loading resistance has great importance. First of all, large span bridges with deep steel girders with slender webs, especially if no longitudinal stiffeners are placed, are likely of experiencing transverse instabilities when subjected to patch loading. Secondly, improving patch loading resistance by increasing web thickness may produce a considerable increment in total steel weight and cost. Patch loading resistance may also be improved by placing longitudinal stiffeners of by reducing the spacing between transverse stiffeners, which has great influence on the overall design of the bridge. In the following paper, the use of triangle-shaped bottom steel cells and lateral concrete strips cast against the web is suggested as a way of improving patch loading resistance, seeking not only a quantitative improvement in resistance but a robust design, less sensitive to the uncertainty of the calculation methods. A calculation method, based on the doctoral thesis work of Lagerqvist [1] and the Eurocode-3 checks [2] shall also be presented. Results obtained with this method will be finally compared with a full non-linear F.E. calculation for an accurate assessment of the patch loading resistance obtained with the suggested design.

KEYWORDS

Patch loading, launched bridges, design improvements, eigenbuckling critical load, non-linear finite elements

1. Introduction

Incremental launching is one of the most useful construction methods for the erection of bridges. Launching is widely used with concrete bridges, but when dealing with steel bridges there are several specific aspects which are not so straightforward, nor are they properly dealt with in standard codes. The designer must therefore take his own decisions on many aspects of the project, such as tolerances, design criteria or control during execution.

Two main differences between the structural response of steel and concrete bridges during launching must be pointed out: firstly, the stiffness of a concrete deck is several times than that of the steel deck, specially if only the steel subsection is launched and top or bottom concrete slabs are cast once launching is concluded; secondly, the total weight of the deck is significantly smaller in steel bridges. Both of them introduce enormous advantages for launching steel bridges. Not only auxiliary equipments are reduced, but control requirements are also reduced due to the low sensitivity of the structural response (stresses and internal forces in the deck) to minor geometric deviations on execution tolerances.

However, there is one critical aspect which must be thoroughly taken into account when launching steel decks, which is the local introduction of the reaction from the bearings onto the webs during construction. Web transverse stiffeners are usually equally spaced along the deck, so it is obvious that local reaction loading will be eventually applied directly onto the non-stiffened web panel during launching. This may give rise to critical buckling of the web, thus concluding that the problem must be carefully studied in the design phase.

Specific regulations for this phenomenon are given in several codes such as BSI or Eurocode 3, part 1.5. However, from our point of view neither of them provide useful design criteria for the project, specially in such case in which the cross section structural form differs significantly from the classical double-T beam or steel box girders.

Patch loading, that is, local introduction of forces onto the stand-alone web without stiffeners, may therefore be critical for determining the thickness of the web plates. This has obviously great influence on the total weight of the deck and, consequently, on the total cost of the bridge. If this is the case, there are several design options available in order to improve the structural response against patch loading. The designer may then modify the geometric configuration of the bottom flange, which directly receives loading during launching, in order to achieve an overall improvement in the web stiffness against patch loading. Such design improvements are rarely considered in the code regulations, so the designer must rely on detailed non linear analysis using finite elements.

This paper shows some alternatives which the authors have studied for the design of launched steel bridges, some of which are presently under construction. It will focus on several alternatives such as the external triangle-shaped bottom cells or the longitudinal concrete beam-like reinforcements cast on site against the web. Both of them provide bending stiffness to the bottom flange which reduces vertical in-plane displacements of the web and, in turn, increase in-plane membrane stiffness of the web, consequently improving patch loading response. Moreover, this design seeks improving patch loading resistance by providing a stiff longitudinal beam-like member, able of distributing the concentrated load over a longer length of resisting web.

A calculation method, based on the doctoral thesis work of Lagerqvist and the Eurocode-3 checks shall also be presented. This method calculates patch loading resistance as the yield resistance reduced by a resistance function. The first is mainly dependant on the length of resisting web, whereas the second depends on the slenderness or, in other words, the buckling resistance of the web. In this way, the improvement in both resistant mechanisms provided by the suggested design can be clearly analysed. A finite element model shall be presented for obtaining the elastic critical load with which the slenderness of the panel is calculated. This same model will be later used for performing a full non-linear F.E. calculation, in order to accurately assess the patch loading resistance obtained with the simplified calculation method.

2. Patch loading resistance and calculations methods

2.1 Patch loading resistance mechanism

When a patch load is applied on the flange, it distributes through the bearing pad, the flange and either the web to flange fillet weld, in the case of a plate girder, or the root fillet, in the case of a rolled beam. In this way, the patch load acts as a knife load on the web, and produces, in the elastic range, a non uniform vertical stress distribution on the web. This stress distribution is generally replaced by one of constant amplitude over a length L_y, usually referred to as the length of resisting web (fig. 1). This length increases with increasing web slenderness and with the stiffness of the flange.

In the case of a stocky web, patch loading resistance will be governed by local yielding of the top part of the web, adjacent to the longitudinal fillet. Resistance will consequently tend towards the web yield resistance, equal to the yield stress applied to a web surface of length L_y and thickness t_w. Plate girders with slender webs, on the other hand, are not able of reaching yield resistance due to previous web instability. In very slender webs, instability consists of local bucking close to the load, which is usually referred to as "crippling". For webs with intermediate slenderness, instability affects the full depth of the web, producing cross section distortion. Despite this distinction between yielding, buckling and crippling, test results do not show a clear frontier between the resisting mechanisms but a gradual transition between them with increasing slenderness.

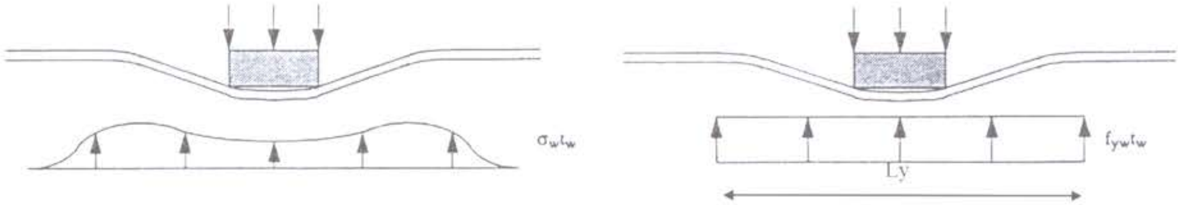


Fig. 1.- Length of resisting web (taken from [1])

2.2 Calculation methods

An exhaustive review of the different calculation methods for patch loading resistance, obtained from the numerous research studies carried out, can be found in [1]. Here we will present the most relevant ones for the purpose of our study.

The simplest formulae for estimating patch loading resistance considerer it proportional to the square of the web thickness. Such type of expression was proposed by Granholm or Bergfelt [3], who stated that patch loading resistance could be expressed as $F_R = 8{,}5 \cdot t_w^2$ or $F_R = 0{,}045\, E \cdot t_w^2$ with F_R in kN, E in GPa and t_w in mm. These formulae do not take into account factors which affect patch loading resistance such as web depth, distance between transverse stiffeners, presence of longitudinal stiffeners, flange stiffeners or length of load introduction.

The Swiss code recommendations are based on further researches performed by Bergfelt [4]. Patch loading resistance is calculated as the minimum of the yield resistance and the buckling resistance of the webs. The yield resistance is calculated as $F_y = [s_s + 5\,(t_f + r)\,] \cdot t_w \cdot f_{yw}$ which simply assumes that the length of resistant web equals the loaded length s_s plus a longitudinal distribution through the flange t_f and fillet weld r with a slope 1: 2,5. The buckling resistance of the web, on the other hand, is calculated as $F_c = 0{,}7\, t_w^2 \, \sqrt{E \cdot t_w} \cdot \alpha_1 \cdot \alpha_2 \cdot \alpha_3$. The correction factors α take into account the ratio of the inertia of the flange to the web thickness together with the interaction with the bending moments of the acting loads.

Lagerqvist [1] suggests a model which will be presented in more detail, since it is the basis for the calculations performed in this paper for evaluating the improvement in resistance obtained with the proposed design. This same model, with minor coefficient variations, is included in Eurocode 3 prEN-1993 part 1-5.

Patch loading resistance is expressed as the yield resistance F_y reduced by a resistance function $\chi(\lambda)$, which is a function of the slenderness of the panel.

$$F_R = F_Y \cdot \chi(\lambda) \tag{1}$$

The yield resistance of the web is its maximum resistance against patch loading, which could only be reached if it were stocky enough as to experience no crippling or buckling instabilities. It is given by:

$$F_Y = f_{yw} \cdot t_w \cdot l_y \tag{2}$$

The length l_y is calculated assuming a failure mode with plastic hinges in the loaded flange, equating the internal and external virtual work and minimizing F_y with respect to s_y (fig. 2).

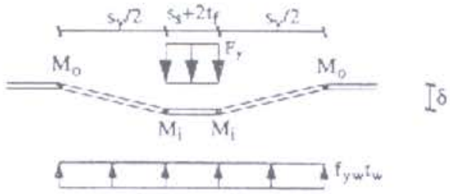


Fig. 2.- Four hinge collapse model for calculating length of resisting web (taken from [1])

External work is given by:

$$W_e = F_Y \cdot \delta - f_{yw} \cdot t_w \cdot \left(s_s + 2 \cdot t_f + \frac{s_y}{2} \right) \cdot \delta \tag{3}$$

Internal work is given by:

$$W_i = 4 \cdot \left(M_i + M_o \right) \cdot \frac{\delta}{s_y} \tag{4}$$

being M_i and M_0 the ultimate plastic moments of the inner and outer hinges, respectively.

Equating external and internal work gives:

$$F_Y = \frac{4}{s_y} \cdot \left(M_i + M_o \right) + f_{yw} \cdot t_w \cdot \left(s_s + 2 \cdot t_f + \frac{s_y}{2} \right) \tag{5}$$

The minimum of F_y with respect to s_y is given by:

$$\frac{\partial F_Y}{\partial s_y} = 0 \qquad \Rightarrow \qquad s_y = \sqrt{\frac{8 \cdot \left(M_i + M_o \right)}{f_{yw} \cdot t_w}} \tag{6}$$

finally giving a yield resistance:

$$F_Y = f_{yw} \cdot t_w \cdot \left(s_s + 2 \cdot t_f + 2 \cdot \sqrt{\frac{2 \cdot (M_i + M_o)}{f_{yw} \cdot t_w}} \right) \tag{7}$$

In the model suggested by Lagerqvist the ultimate plastic moment of the inner hinge is taken as that of the flange alone:

$$M_i = \frac{f_{yf} \cdot b_f \cdot t_f^2}{4} \tag{8}$$

whereas the ultimate plastic moment of the outer hinge is taken as that of a fictive T section with an effective web depth taken as $k \cdot h_w$. In this case, neglecting, on the safe side, several terms, M_0 can be worked out to be:

$$M_o \simeq \frac{f_{yf} \cdot b_f \cdot t_f^2}{4} + \frac{f_{yw} \cdot t_w}{2} \cdot k^2 \cdot h_w^2 \tag{9}$$

Substituting equations (8) and (9) in (7) and taking $k^2 = 0,02$, which results from a calibration of the model with experimental results, finally gives the following expression for yield resistance F_y according to Lagerqvist model:

$$F_Y = f_{yw} \cdot t_w \cdot \left(s_s + 2 \cdot t_f + 2 \cdot t_f \cdot \sqrt{\frac{f_{yf} \cdot b_f}{f_{yw} \cdot t_w} + 0,02 \left(\frac{h_w}{t_f} \right)^2} \right) \tag{10}$$

The resistance function, on the other hand, is a function of the slenderness parameter λ. The function suggested by Lagerqvist is:

$$\chi(\lambda) = 0,06 + \frac{0,47}{\lambda} \leq 1 \tag{11}$$

which is a decreasing function with increasingly slender panels, meaning that panels with increasing slenderness are more likely to buckle and consequently have patch loading resistance lower than the yield resistance.

The slenderness parameter is calculated as:

$$\lambda = \sqrt{\frac{F_y}{F_{cr}}} \tag{12}$$

where F_{cr} is the elastic buckling load of the panel. Lagerqvist calculates the elastic buckling load for several double T sections using F.E. analyses and suggests the following formula:

$$F_{cr} = k_f \cdot \frac{\pi^2 \cdot E}{12 \cdot (1 - \nu^2)} \cdot \frac{t_w^3}{h_w} \tag{13}$$

with the elastic buckling load coefficient k_f given by:

$$k_F = \left(1+\frac{s_s}{2\cdot h_w}\right)\cdot\left(5,3+1,9\cdot\left(\frac{h_w}{a}\right)^2+0,40\cdot\sqrt[4]{\beta}\right) \quad (14)$$

where the parameter β is the ratio between the torsional stiffness of the flange and the flexural plate stiffness of the web, that is:

$$\beta = \frac{b_f\cdot t_f^3}{h_w\cdot t_w^3} \quad (15)$$

Neglecting the influence of the length of load introduction s_s and taking an average value of 1 for the β parameter, equation (14) is simplified to:

$$k_f = 6+2\left(\frac{h_w}{a}\right)^2 \quad (16)$$

Eurocode 3, as previously stated, adopts this same calculation model, with the only differences of using a resistance function given by $\chi = 0,5/\lambda$ instead of equation (11) and limiting the maximum length of resisting web, or effective loaded length l_y, to the distance between transverse stiffeners. This condition may lead to anomalous results, with decreasing instead of increasing resistance, when closing up too much the spacing between transverse stiffeners. The limitation of EC-3 of $l_y \leq a$ will be later analysed in further detail.

3. Design Improvements and Calculation Method Proposed

3.1 Scope of the study

The brief review of the calculation methods presented in the previous section highlights the most significant parameters governing patch loading resistance, such as web thickness, spacing between transverse stiffeners, existence of longitudinal stiffeners, length of load introduction or torsion stiffness of the flange. In this paper, a design improvement for patch loading resistance is suggested, consisting of stiffening the loaded flange with a triangle-shaped bottom cell and a lateral concrete strip cast in place against the web. Casting a lateral concrete strip may be a worthy design for bridges with double composite action, in which the lateral concrete strip is cast together with the bottom flange concrete. This design has been used for the high speed viaduct across the river Las Piedras in Spain, whose cross section during launching is shown in fig. 3.

This design seeks improving patch loading resistance by providing a stiff longitudinal beam-like member, able of distributing the concentrated load over a longer length of resisting web. This can be easily visualized by means of equations (7) or (10), in which the length of resisting web increases with the stiffness of the loaded flange (ultimate plastic moment in equation (7) or flange thickness in equation (10)). The elastic critical load of the cross section must also improve above that of the double T section, since, on one hand, the web slenderness is reduced as the cell panel depth is not subjected to buckling and, secondly, torsion stiffness of the flange is greatly increased, improving resistance as seen in equation (14). Moreover, providing bending stiffness to the bottom flange must reduce vertical in-plane displacements of the web and, in turn, increase in-plane membrane stiffness of the web, consequently improving patch loading response.

From a conceptual point of view, this design may be considered as a robust mechanism in contrast with the uncertainty in assessing buckling resistance of the web. This fact is indeed taken into account in the calculation method of BSI-5400 appendix D [5], which does not affect the contribution to resistance of the flange by the resistance function reduction factor.

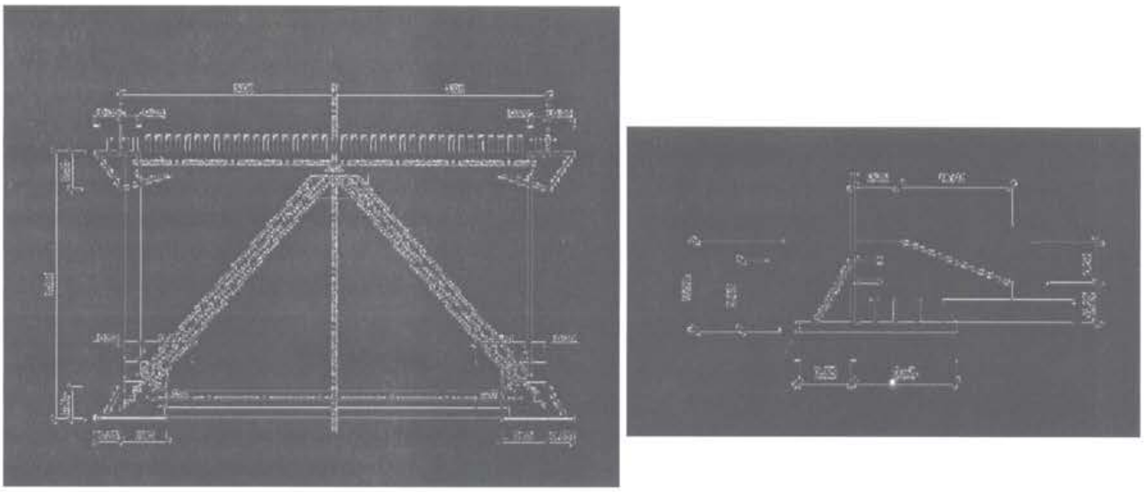


Fig. 3.- Cross section of the Las Piedras High Speed Viaduct.

This paper presents several calculations in order to evaluate the improvement in resistance achieved with the design suggested. To do so, several previous calculations varying some of the standard parameters (web thickness, spacing between stiffeners and length of loaded length) are performed, in order to calibrate the calculation model used and to visualize the trend in resistance improvements, for later comparison with the suggested design.

3.2 Calculation method

The calculation method used in the study is simply adapted from the design method proposed by Lagerqvist. The procedure is explained hereafter:

- Patch loading resistance is calculated as the yield resistance affected by a resistance function: $F_R = F_Y \cdot \chi(\lambda)$

- The yield resistance is calculated with a 4 hinge model which yields a length of resisting web according to equation (7). In the case of including the bottom cell and lateral concrete strip, the ultimate plastic moments of the hinges are calculated as those of the steel or composite cross sections shown in fig 3, easily obtained in our case with a specific computer analysis program. In the case of the composite cross section, both the positive and negative bending ultimate moments are calculated, since they differ when concrete cracking is taken into account.

- The resistance function χ is assumed to the same as the one calibrated by Lagerqvist using the double T tests, no matter the presence of the bottom cell or concrete strip. Therefore, the resistance function is given by equation (11).

- The slenderness parameter $\lambda=\sqrt{\frac{F_Y}{F_{cr}}}$ is calculated with an elastic critical load obtained from an eigenbuckling calculation using a F.E. model, including all the design parameters involved: spacing between stiffeners, length of load introduction, bottom cell or concrete strip.

  The most outstanding features of the F.E. model used are show in the following figure.

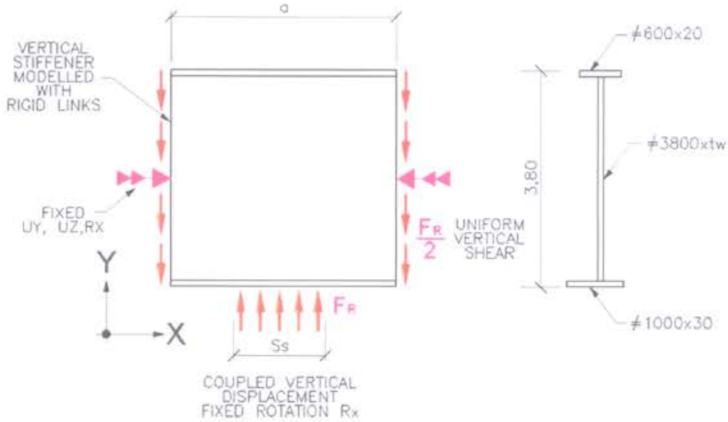


Fig. 4.- F.E. Modelling details

There are several aspects which may be criticized of the calculation model suggested. First of all, assuming a four hinge collapse model with hinges on the full extension of the bottom cell or concrete strip is a direct extrapolation of the collapse model with hinges in the flange which has been observed in the experimental tests, whereas these is no empirical evidence that this same collapse model takes place when the stiffness of the loaded flange is considerably increased with a cell or a concrete strip. Secondly, we have assumed that the resistance function is the same obtained with the empirical results a double T beams, with only the stiffness contribution of the flange. Despite these objections, both assumptions are reasonable, so that the calculation model can be considered fully adequate for carrying out a series of influence analyses on the different parameters that govern patch loading resistance. Moreover, the overall accuracy of the method shall be tested with several full non-linear F.E. calculations which reproduce, as accurate as possible, the actual patch loading resistance of the girder.

4. RESULTS OF THE STUDY

4.1 Influence of web thickness

Web thickness is the principal parameter governing patch loading resistance. In order to analyse the influence of varying thickness and the improvement obtained in resistance, several analyses are performed with the following parameters:

- Web thickness varying in the range of 15 to 35 mm.
- Distance between transverse stiffeners a= 4 m.
- Length of load introduction Ss = 1,5 m.
- Double T cross section, with only top and bottom flange contribution to stiffness.

The elastic critical load is calculated with a F.E. model, with the same modelling details already indicated. The first buckling mode for one of the calculations is shown in fig. 5, which is representative of the first buckling shape obtained for all web thicknesses.

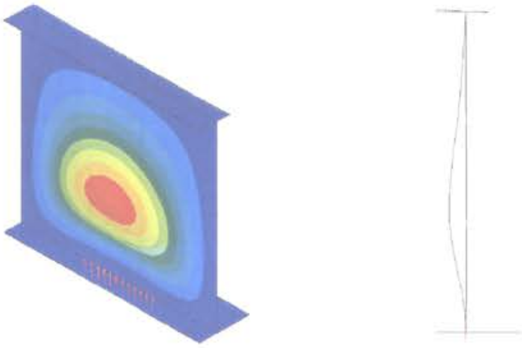
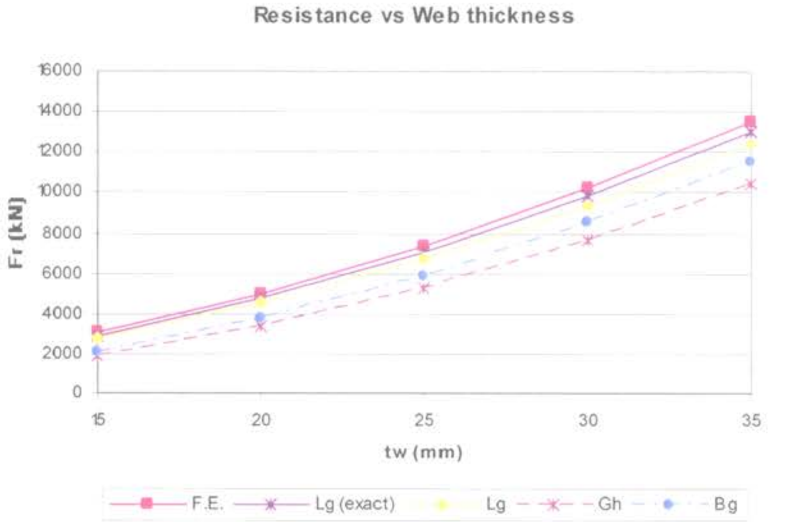
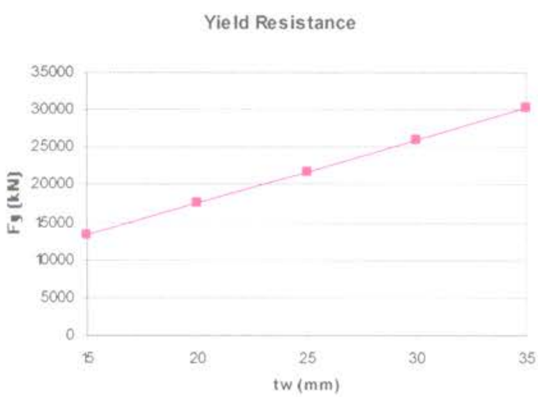


Fig. 5.- First elastic buckling mode shape

In the following graphs total patch loading resistance is plotted against web thickness. Together with the results of the F.E. model, total resistance calculated with the Lagerqvist model is also included, calculated both with the full and simplified expressions for the buckling coefficient k_f. The resistance calculated with the simplified expressions proposed by Bergfelt and Gronholm are also plotted for the sake of comparing results. Graphs of yield resistance and resistance function are also included, in order to visualize the contribution of each factor to total resistance.





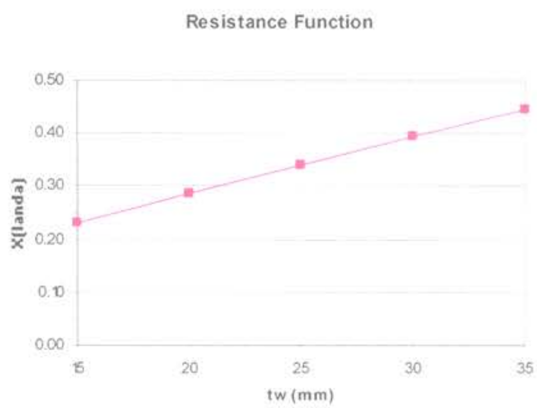


Fig. 6.- Results of patch loading resistance with varying web thickness

We observe that increase in patch loading resistance is quadratic with web thickness, due to a linear increase in yield resistance and a fairly linear increase in resistance function. The increase in resistance function is a consequence of the decreasing slenderness of the panel, which becomes less likely of experiencing transverse instabilities. It is also worth noticing that the prediction of resistance directly proportional to the square of the web thickness, despite their simplicity, yield a fairly accurate estimation of patch loading resistance, at least in the case of double T beams. The results obtained with the Lagerqvist model differ from the F.E. results in the prediction of the elastic critical load, which affects the slenderness parameter and consequently the resistance function. A better accuracy is obtained if the full expression for the buckling coefficient k_f is used but, nonetheless, even the simplified expression yields fairly accurate results. In the following graph the buckling coefficients are plotted against the web thickness, including the two proposals by Lagerqvist in expressions (14) and (16), together with the coefficient obtained from the elastic critical loads with the F.E. calculations.

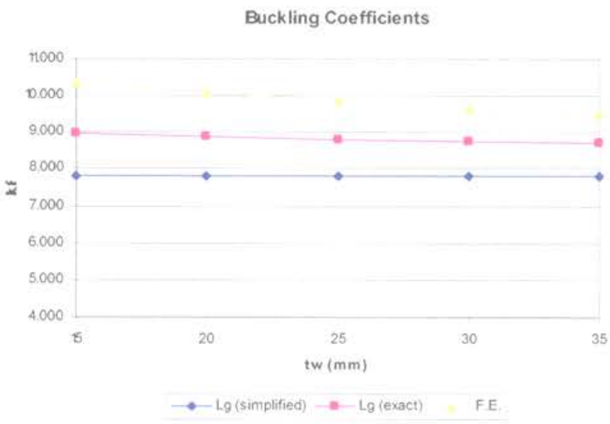


Fig. 7.- Comparison between buckling coefficients

4.2 Influence of distance between transverse stiffeners.

The second parameter studied is the influence of the distance between transverse stiffeners. The analyses performed involve the following parameters:

- Web thickness varying in the range of 15 to 35 mm.
- For each web thickness resistance is obtained for three spacing between transverse stiffeners: a= 2, 4 and 8 m.
- Length of load introduction s_s = 1,5 m.
- Double T cross section, with only top and bottom flange contribution to stiffness.

First of all, results are compared between the F.E. model and the Lagerqvist (similar to the EC-3) (fig 8), with a buckling coefficient calculated with the simplified expression(16). Apart from the increase in resistance due to increase in web thickness, an increase in resistance is observed as the distance between transverse stiffeners is reduced. The most outstanding aspect is that for a small spacing, less than the depth of the panel, the resistance calculated with the Lagerqvist model underestimates resistance calculated using the elastic critical load obtained with a F.E. calculation.

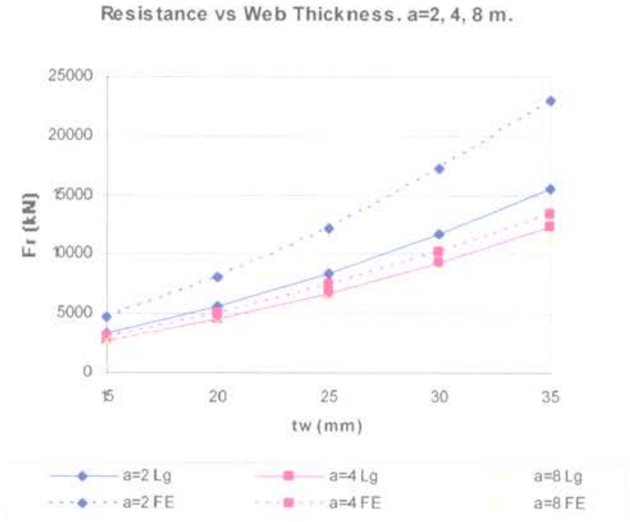


Fig. 8.- Patch loading resistance with varying vertical stiffener spacing

In fig 9 the increase in patch loading resistance is plotted against the distance between transverse stiffeners *a*. Together with total resistance, yield resistance and resistance function are also plotted. Yield resistance is independent of the distance between transverse stiffeners (except for the comments included further on) so the increase in patch loading resistance is due to a similar increase in resistance function. The most significant increase is obtained with very closely spaced transverse stiffeners, denoting greater capacity of restraining lateral instability on the loaded length.

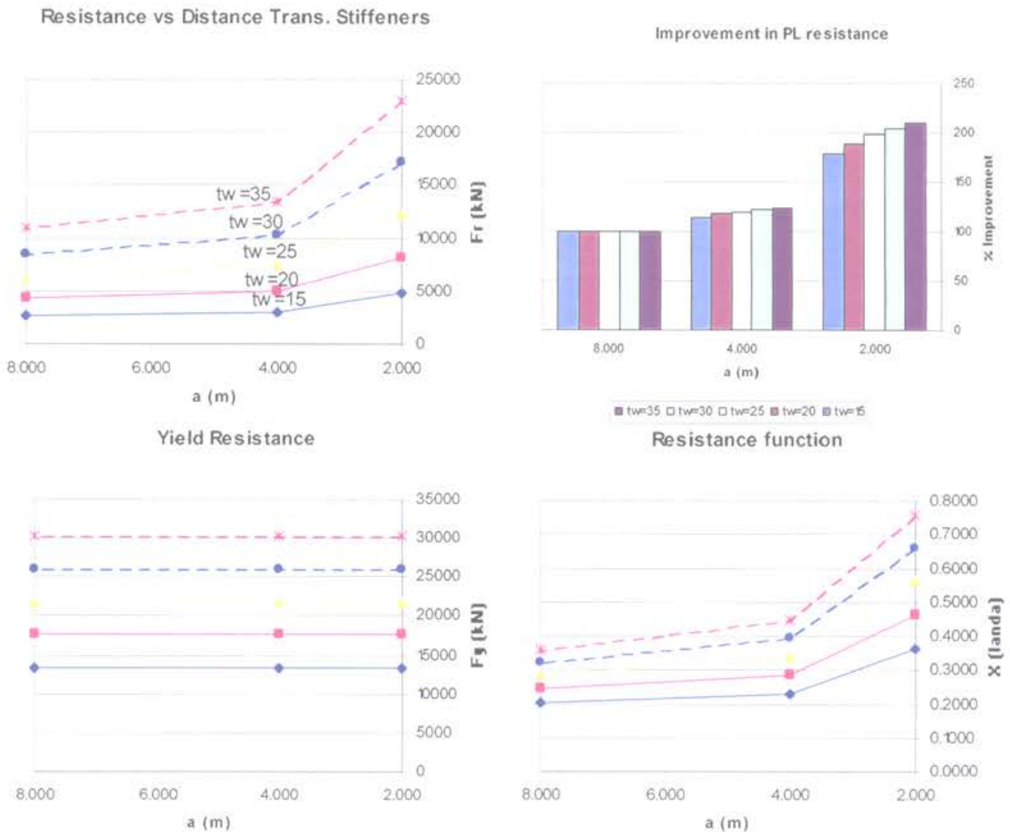


Fig. 9.- Results with varying vertical stiffener spacing

The ratio of improvement in total patch loading resistance is shown as a bar chart in fig 9 Improvement is calculated, for each web thickness, as the ratio of patch loading resistance for a given spacing to the resistance with maximum transverse stiffeners spacing a= 8 m. Once again, a significant improvement is observed by reducing distance to a = 2 m.

Regarding the independency of yield resistance to the distance between transverse stiffeners, this is true as long as the length of resisting web is not greater than the distance between stiffeners. If, on the other hand, l_y is greater than a, then the outer plastic hinges are formed precisely at the stiffeners, thus making $l_y = a$. EC-3 in part prEN 1993-1-5 limits the maximum value of l_y to the distance between stiffeners, in which case absurd results may be obtained as stiffeners are closed together, since resistance may decrease due to limitation in the length of resisting web. In our opinion, if l_y is indeed limited to a, patch loading resistance must be computed adding the contribution of the transverse stiffeners:

$$F_R = F_Y(l_y \leq a) \cdot \chi(\lambda) + \Delta F_{stiffeners} \tag{17}$$

As far as we know, there is neither experimental research nor proposed formulae for assessing such contribution of the stiffeners to patch loading resistance. From our point of view, their contribution could be, at least, as much as the stand alone web, which suggests that the limitation of EC-3 should be revised. One possible approach could be calculating resistance without limiting l_y to a as a way of taking into account, in a somehow indirect and on-the-safe-side way, the contribution of the stiffeners as an "increase" in the length of resisting web above the distance between transverse stiffeners.

4.3 Influence of the length of load introduction

Given a distance between transverse stiffeners, in this section we will analyse the variation in resistance when the length of load introduction is modified. To do so, the parameters used in the analyses are:

- Spacing between transverse stiffeners: a= 2, 4 and 8 m.
- For each spacing, the ratio of the length of load introduction to the transverse stiffeners spacing (s_s/a) is varied in the range 0,25 to 1.
- For each analysis web thickness is varied in the range of 15 to 35 mm.
- The cross section is a double T, when the only contribution of the flanges to patch loading resistance.

Fig 10 shows total patch loading resistance plotted against the ratio s_s/a for the case of a=4 m, together with the graphs of the yield resistance and the resistance function. The ratio of improvement in resistance is shown as a bar chart. The rest of the graphs for the two other transverse stiffener spacing a=2 and a=8 follow the some pattern as the case presented, so they are not shown for the sake of conciseness. It can be observed that total resistance increases when increasing loaded length due to a fairly linear increase in yield resistance, whereas resistance function decreases with increasing loaded length, indicating that panels are more likely to buckle as the loaded length increases. When the loaded length is increased up to the distance between transverse stiffeners, rather than true patch loading we would be dealing with what Roberts and Chong defined as "distributed patch loading" [6], but we have nevertheless included those results for comparison purposes only. It can be noticed that doubling the length of load introduction from a ratio of 0,25 to 0,5 provides only an increase of 25% in resistance.

4.4 Influence of the bottom cell and lateral concrete strip.

Up to this point we have analysed the improvement in patch loading resistance obtained by acting on the "traditional" parameters such as the web thickness, spacing of transverse stiffeners and length of load introduction. These are obviously perfectly valid ways of improving resistance but they have, nonetheless, several disadvantages. Increasing web thickness, though very effective for raising patch loading resistance, has a great influence on total steel weight, which becomes determined by the launching phase. For very close transverse stiffeners, for which the improvement in resistance is most remarkable, we encounter the uncertainty of the limitation in effective web length imposed by

Eurocode 3. Finally, increasing the length of load introduction produces moderate improvement in total resistance, taking into account that the length of launching rollers is limited in practice by the space available on the top of the piers, which practically sets the maximum length in the range of 1,5 to 2 m.

As an alternative way of improving patch loading resistance, this study suggests the design of a bottom cell and a lateral concrete strip cast "in place", as previously sketched in fig. 3. This design seeks improving significantly the stiffness of the loaded flange, which is truly transformed into a beam capable of spanning the transverse stiffeners, leading to an increase in the length of resisting web. Moreover, this design should provide a very stiff and certain load carrying mechanism via the bottom cell and concrete strip, avoiding the uncertainty in resistance due to web buckling.

For the studied cases, the cell plate thickness is varied between 15 and 35 mm, whereas the remaining parameters are:

- Distance between transverse stiffeners *a*=4 and *a*= 8 m.
- Constant web thickness t_w= 25 mm.
- Constant length of load introduction s_s= 1,5 m. The load is applied on the F.E. model as a constant pressures over a surface of 1,5 x 0,5 m^2.

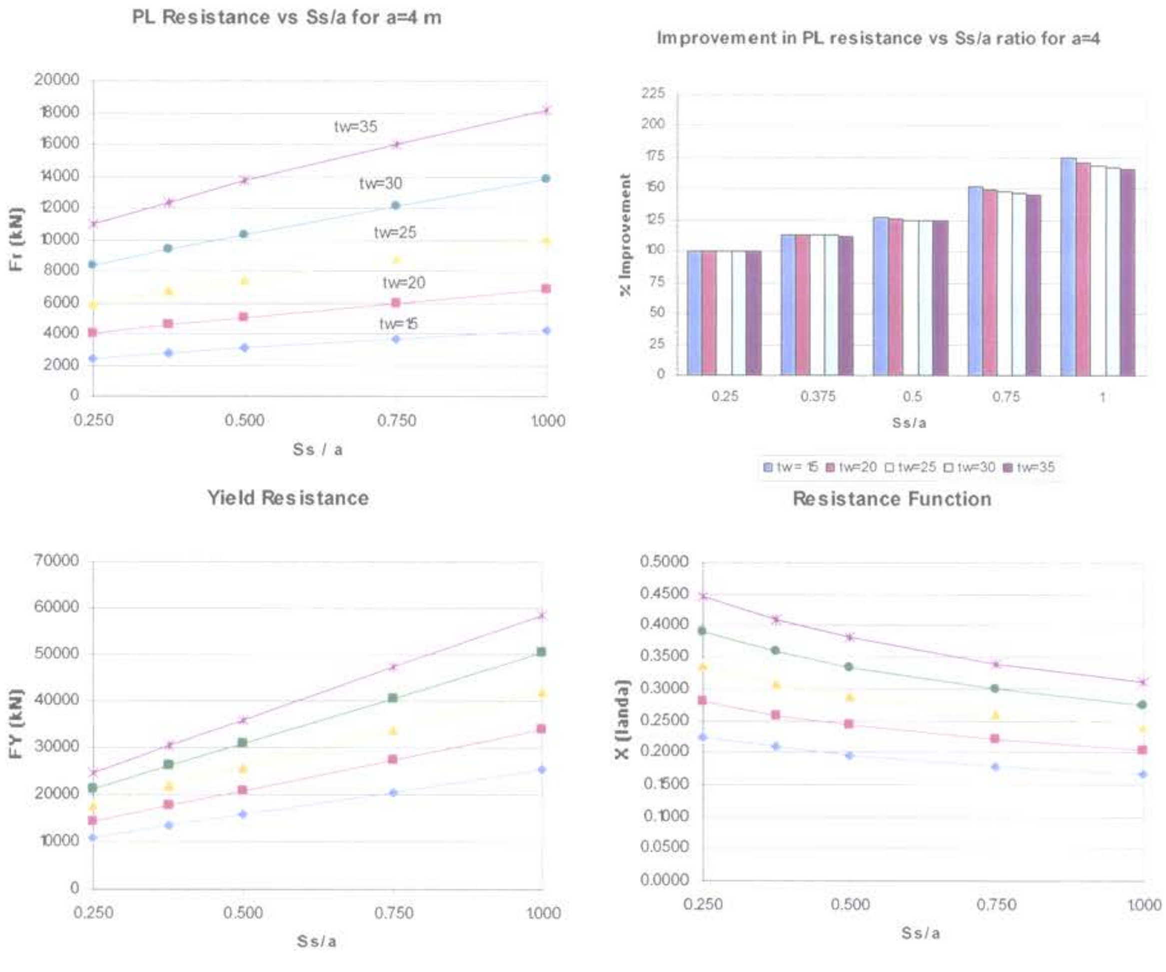


Fig. 10.- Results with varying length of load introduction

Fig 11 shows the buckling mode shapes obtained with the F.E. model with only bottom cell and with bottom cell plus concrete strip, both for transverse stiffeners spaced 4 and 8 m. The deformed shapes shown are representative of all the calculations performed.

In fig 12 total patch loading resistance, yield resistance and resistance function are plotted against the ultimate plastic moment of the bottom beam cross section, calculated as described in section 3.2. This parameter is chosen for the X-axis as it provides a consistent scale valid for the analyses either with or without concrete strip. The ratio of improvement in patch loading resistance, on the other hand, is shown as bar chart for each analysis performed. Observing the graph for patch loading resistance, a significant improvement in resistance is observed when placing a bottom cell, which greatly increases the stiffness of the loaded flange as seen in the ultimate plastic moments obtained. The lateral concrete strip provides further increase in flange stiffness, yielding an enhancement in total resistance. The improvement in resistance is clearly observed in the bar chart, which shows an average improvement around 50% for bottom cell only and around 70% if concrete strip is also used, both for the transverse stiffener spacing of 4 and 8 m.

The increase in stiffness of the loaded flange gives a fairly linear increase in yield resistance, when plotted against the ultimate plastic moment of the cell and concrete strip, as the stiffness provided enables the loaded flange to distribute load along a longer length of resisting web. A slight improvement in resistance function is also observed, as the depth of the web likely of buckling is reduced by the depth of the cell or concrete strip, consequently reducing the slenderness of the panel.

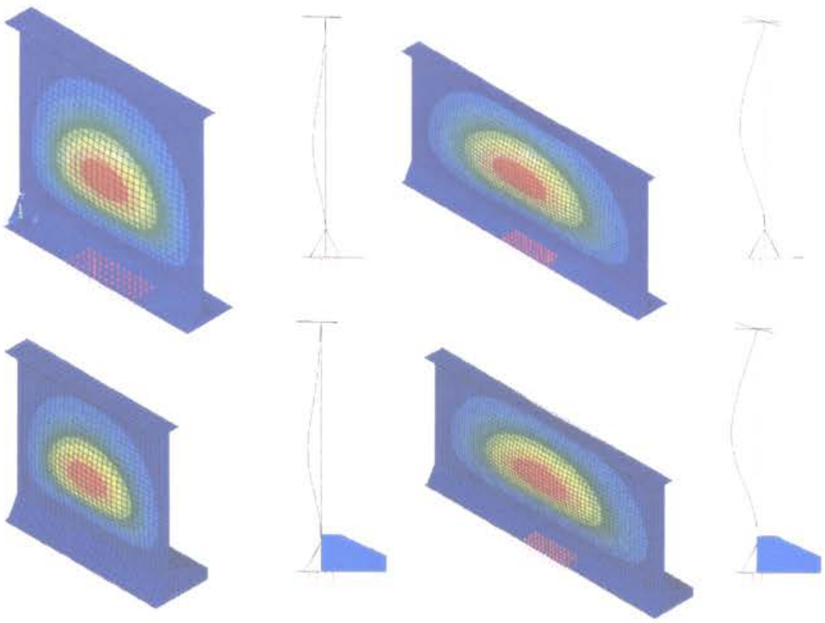


Fig. 11.- Buckling mode shapes with cell and cell plus concrete strip

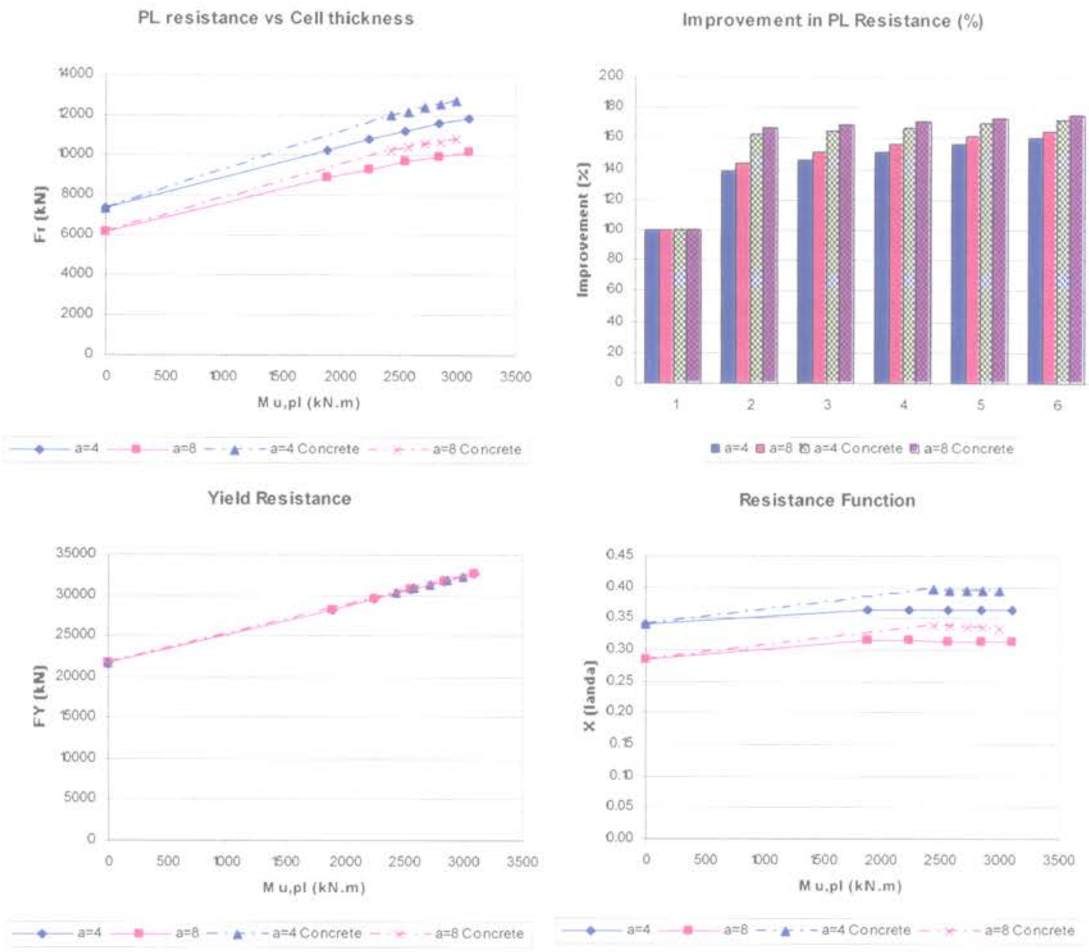


Fig. 12.-Results with bottom cell and concrete strip

Having analysed the improvement in patch loading resistance obtained varying the different parameters involved, which has been compared with the improvement obtained when a bottom cell and concrete strip are considered, one final analysis is performed with a full non-linear F.E. calculation including both geometric nonlinearities and elastoplastic behaviour of the steel. This analysis is mainly intended to present the capabilities of a full non linear analysis and to have at least a rough estimation of the overall accuracy of the calculation method for patch loading resistance proposed in this paper.

The non-linear F.E. calculation is performed on a girder with a web 25 mm. thick and bottom cell with 15 mm. thick plates, with a spacing of 8 m. between transverse stiffeners. The steel constitutive law is modelled as a perfect elastoplastic material, with a design yield strength plateau at 323 MPa. The initial imperfections are obtained from a previous eigenbuckling analysis, scaled from the first buckling mode with a maximum amplitude of 2.5 cm., which is about 1/200 of the web depth, as recommended in EC-3 part 1.5 when dealing with imperfections in F.E. analysis. Results from the non-linear calculation are shown in fig 13. The patch loading resistance obtained with the non-linear F.E. calculation is F_R=12.700 kN, whereas the calculated resistance with the suggested simplified design model is 8.800 kN., which may be regarded as a reasonable accuracy. The integration of vertical forces on the web along its full length, obtained from the distribution of vertical stresses shown in fig 13, yields a force of 9.000 kN. Therefore, the remaining force up to the total resistance of 12.000 kN is directly transferred to the vertical stiffeners as shear force by the bottom cell and flange as a beam in longitudinal bending. Moreover, the distribution of vertical stress on the web clearly indicates a concentration of vertical stresses along a length of about 4.5 m., which may be taken as the effective length or length of resisting web, similar to the calculated length of 3.5 m. using the simplified four hinge collapse model. Consequently, results from the non-linear calculation seem to indicate that the simplified design model yields a good prediction of the resistance provided by the web alone, taking into account the increase in effective length of the web due to the stiffness of the bottom cell member, whereas it does not include the contribution of the stiff bottom member in transferring load directly to the vertical stiffeners as a beam in longitudinal bending.

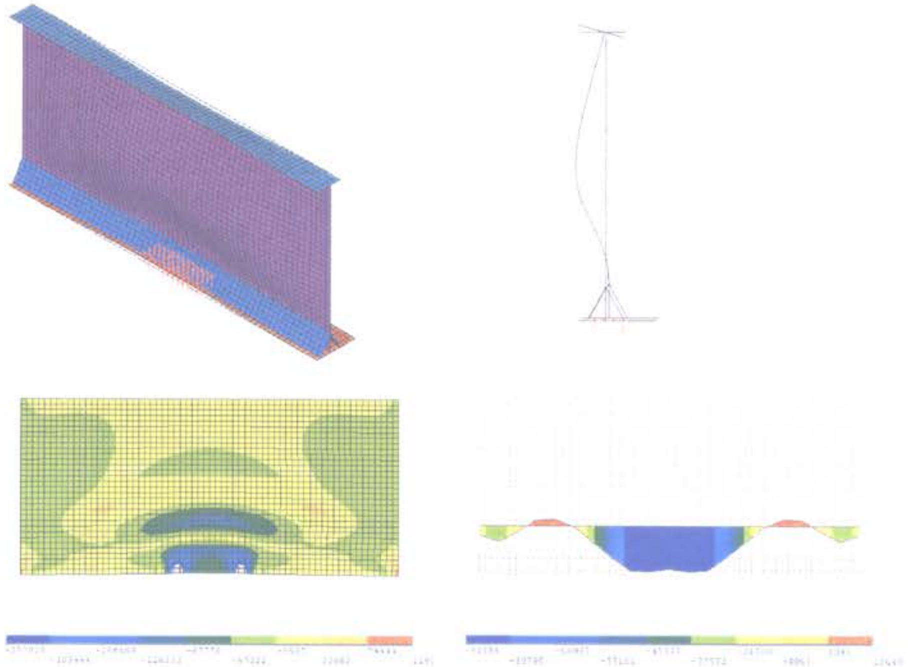


Fig. 13.- Results of non-linear F.E. calculation

5. CONCLUSIONS

This paper has presented a design model for calculating the patch loading resistance of girders, based on the method suggested by Lagerqvist and adopted by Eurocode 3, in which the elastic critical load of the girder for the calculation of the slenderness parameter is obtained with a simple F.E. model instead of the expressions included in the aforementioned methods. This gives a general approach for the calculation of patch loading resistance for any type of girder design, no matter if it includes variable web thickness, longitudinal stiffeners or other elements not considered in the simplified expressions of the elastic critical coefficient.

The calculation model has been used to estimate the improvement in patch loading resistance obtained by varying several parameters such as the web thickness, spacing between vertical stiffeners and length of load introduction. In the second place, the increase in resistance obtained when including a bottom cell and a lateral concrete strip has been analysed, concluding that these can be regarded as a valuable design improvement for patch loading resistance, achieving a stiff and robust load carrying member which increases resistance significantly.

The overall results obtained from the proposed model are checked with a full non-linear geometric and material F.E. analysis of one of the studied cases. The modelling details have been described in order to define a valid modelling procedure which yields reasonably accurate results. The resistance calculated with the simplified model is in good coincidence with the one obtained with the non-linear calculation, underestimated as the contribution of the bottom cell and concrete strip as a load carrying member spanning between stiffeners is no directly considered. In this way, further investigations may performed with full non-linear F.E. calculations in order to define a different resistance function which includes such contribution of the bottom stiff member, which can be also extended to determining the contribution of vertical stiffeners when the length of resisting web exceeds the spacing between stiffeners.

6. NOTATIONS

t_w thickness of web

h_w depth of web

t_f thickness of flange

b_f width of flange

a width of web panel between vertical stiffeners

s_s length of load introduction

l_y total length of web responding to the applied load

7. REFERENCES

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