LOAD-INTRODUCTION RESISTANCE OF COLUMN WEBS IN STRONG AXIS BEAM-TO-COLUMN JOINTS

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Abstract

Present paper is aimed at presenting the results of the study performed at the University of Liège on the deformability and the resistance of column web panels in strong axis beam-to-column joints, when subjected to transverse loads carried over by the beam(s).
Formulae for the assessment of the ultimate resistance and stability (buckling and crippling) of the column webs are proposed; they are based on the conclusions of a parametrical study which is briefly described.
Present paper is restricted to the beam-to-column joints with H or I hot-rolled sections.

1. Joint deformability components

The two following sources of deformability of a strong axis beam-to-column joint have to be distinguished :

a) the deformation of the connection associated to the deformation of the connection elements (end plate, angles, bolts,...), to the slip, to the column flange deformation and to the local deformation of the column web in the tension and compression zones;
b) the deformation of the column web under shear associated mostly to the common presence of forces, equal and opposite, in tension and compression, carried over by the beam(s) and acting on the column web at the level of the joint.

The load-introduction deformability of a column web panel is defined as the component of the connection deformability relative to the local deformation of the column web in the tension and compression zones of the joint (respectively a lengthening and a shortening).

Figure 1 illustrates schematically these definitions in the particular case of a joint between one beam and one column. The deformation of the ABCD column web panel (figure 1.a.) has to be divided into two parts :
- the transversal effect of the beam flange forces F_b (statically equivalent to the beam moment M_b) results in a relative rotation ϕ between the beam and the column axes (figure 1.b.); this rotation provides a first deformability curve M_b-ϕ;
- the shear effect due to the shear force V_n results in a relative rotation γ between the beam and the column axes (figure 1.c.); this rotation makes it possible to establish a second deformability curve V_n-γ.

2. Numerical investigations

An important parametric study has been realized recently at the Polytechnic Federal School of Lausanne and at the University of Liège. All the results and all the conclusions of this study may be found in [1].

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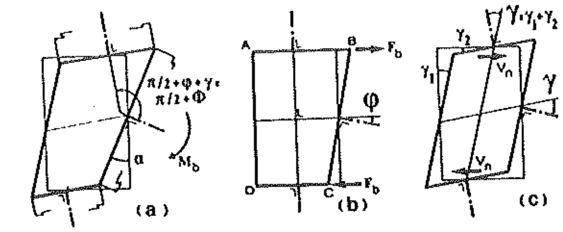


Figure 1 - Global deformation of the column web (a) decomposed into the load-introduction effect (b) and the shear effect (c).

This study is based on numerical simulations with the non-linear FE-program FINELG [2] of the loading up to failure of welded beam-to-column joints. Material and geometrical non-linear effects are taken into account. The specimens of the chosen joints are analysed in three dimensions by using "shell" finite elements to model the webs and flanges of the profiles and "beam" finite elements to model stiffeners. The adopted finite element meshes are shown on figure 2, respectively for a "T" joint (one column, one beam) and a "cross" joint (one column, two beams). Complete data may be found in [1].

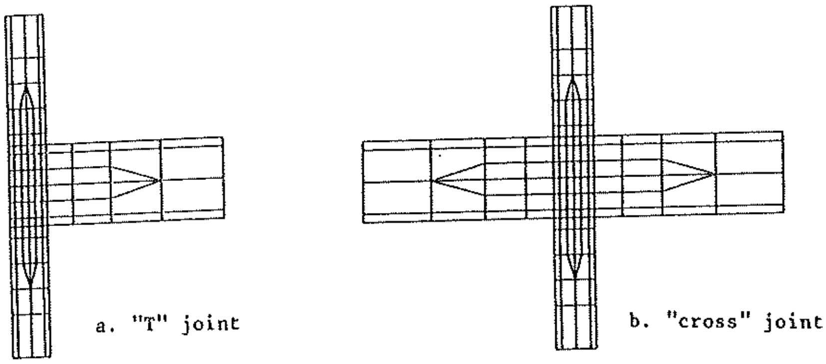


Figure 2 - Types of beam-to-column joints studied numerically.

The numerical simulations allow to study the propagation of the plasticity in the profiles and to observe the exact failure modes.

The good agreement between the numerical simulations and results of experimental tests on joints is shown in [3].

The moment-rotation curves characterizing the shear deformability and the load-introduction deformability of the column web panel have been reported for every simulation.

The following parameters have been taken into account in the parametric study of the joints :

a) the type of the beam(s);
b) the type of the column;
c) the loading of the joint;
d) the initial out-of-flatness of the column web;
e) the presence or not of transverse stiffeners on the column web.

Only the conclusions relative to the load-introduction behaviour of the unstiffened column web panels are presented here.

a) The M_b-ϕ curve for a given joint depends on the actual loading of the joint.
Let us subject three similar unstiffened welded joints ("T" arrangement) to different types of loading and let us report the characteristic M_b-ϕ curves in a common diagram (figure 3). A similarity exists only in the elastic range of the web panel behaviour.
The differences between the M_b-ϕ curves in the nonlinear range of behaviour cannot be neglected.
In reality an unstiffened column web panel experiences three types of stresses in its most stressed zone (figure 4) :
- the shear stresses τ;
- the normal stresses σ_n resulting from the compression force and the bending moment in the column;
- the normal stresses σ_l resulting from the introduction of beam loads into the column web.

The load-introduction behaviour of a web panel shall obviously be affected, except in the elastic range, by the relative importance of each of these stresses according to the type of joint loading.

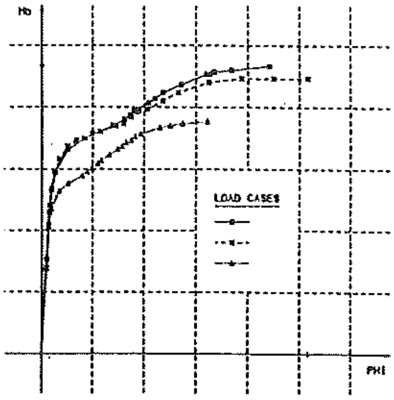


Figure 3 - Characteristic M_b - ϕ curves

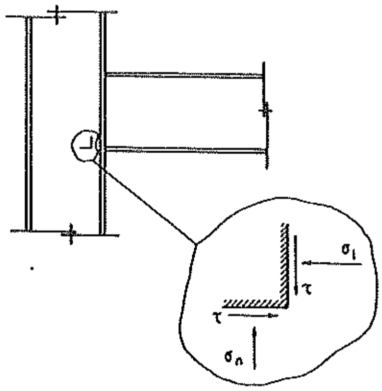
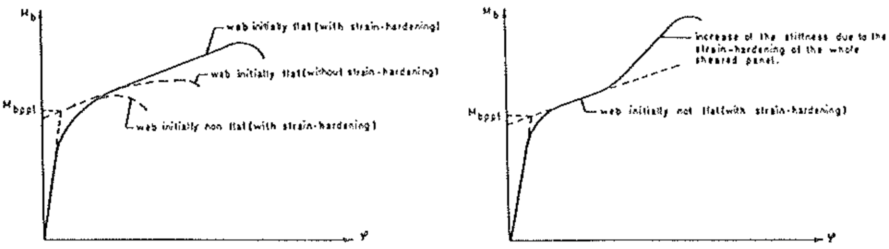


Figure 4 - Different types of stresses in a web panel

b) It is not allowed to define the plastic capacity of a web subject to transverse loads as this may be done for sheared column web panels : the propagation of the plasticity in the column web transversally loaded does not end indeed in the apparition of an horizontal yield plateau - when the strain-hardening is omitted in the numerical simulation -, but rather in the development, till the attaintment of the ultimate buckling load, of a zone characterized by a progressive increase of the resistance and the deformability of the web (figure 5.a.).
It will be consequently referred to the so-called pseudo-plastic moment, M_{bppl}, that is associated to a limit state of the column web panel due to the effect of transverse loads (figure 5.a.). This characteristic load level may obviously be similarly defined for the M_b-ϕ curves relative to "T" joints (see figure 5.b.).

c) The propagation of the plasticity in a web subject to transverse loads is not affected by the presence of σ_n stresses in the web insofar as their maximum value does not exceed a relatively high limit which should have to be explicitely determined.



a. "cross" joint   b. "T" joint

Figure 5 - Definition of the pseudo-plastic capacity of a web subject to transverse loads.

This conclusion seems to confirm the result of an experimental study carried out in the Netherlands [4] and which tends to show that the influence, on the "plastic capacity" of the web, of σ_n stresses not exceeding 50 % of the column web yield stress is not significant.
ZOETEMEIJER [4] proposes, for larger values of σ_n ($\sigma_n > 0.5\ f_y$), to reduce the "plastic capacity" by means of a factor e given by :

$$e = 1.25 - 0.5\ \frac{|\sigma_n|}{f_y} \qquad (1)$$

KATO considers for his own [5] that the attaintment of stresses σ_n greater than 0.5 f_y is not of practical interest because of the relative low values of the loads transmitted to the column in order to prevent instability.

d) The amplitude of the column web out-of-flatness influences only the shape of the M_b-ϕ curves of joints whose collapse is linked up to the buckling of the web; this initial out-of-flatness affects the value of the web ultimate buckling load in a significant way but modifies very slightly the deformability of the web as far as the collapse load is not reached (figure 5.a.)
e) The comparison (figure 6) of M_b-ϕ curves relative to a "T" joint (figure 2.a.) and to the corresponding (same column and same type of beam) "cross" joint (figure 2.b.) shows clearly the similarity of both web behaviours in the elastic range (it is not possible to compare the curves in the non elastic range on account of the different stresses interacting in the column webs).
This leads to the conclusion that the introduction of transverse loads in a column web constitutes, as far as the stability of the web is not concerned with, a local phenomena limited to the vicinity of the column flanges. The influence of the joint loading on the shape of the M-ϕ curves - as discussed in (a) - is seen to be very significant in this case.

3. Prediction of the web panel deformability curves

Mathematical models for the prediction of the characteristics V_n-γ and M_b-ϕ curves of stiffened and unstiffened column column web panels are proposed in [1]. They have been validated in [6] for joints with welded, extended end plate and flange cleated connections by means of comparisons with numerical and experimental results.

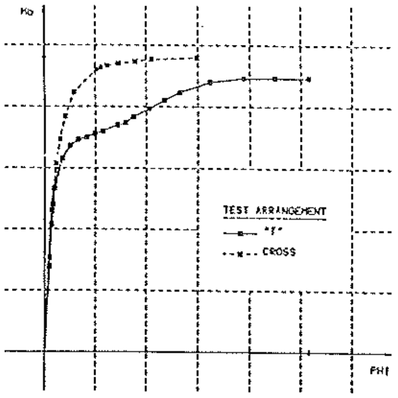


Figure 6 - Comparison between "T" and "cross" joint behaviour (M_b-ϕ curves)

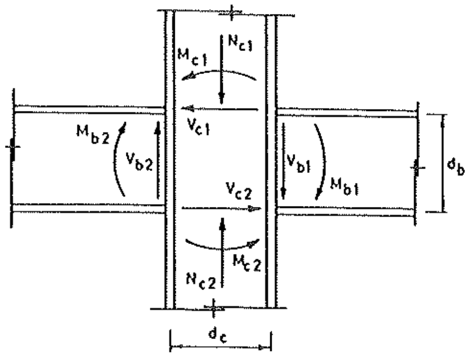


Figure 7 - Loading of an interior joint

4. Assessment of the ultimate strength for unstiffened web panels

The ultimate strength of a column web may be associated to one of the three following types of collapse :

- the shear collapse of the column web panel, V_{nbu}, to which corresponds a moment M_{nbu} in the beam by the following formula (figure 7) :

$$V_n = \frac{M_{b1} + M_{b2}}{d_b} - \frac{1}{2}(Q_{c1} + Q_{c2}) \tag{2}$$

- the excessive yielding of the web under transverse loads, M_{buy};
- the instability of the web under compression transverse loads, M_{bub}.

The ultimate moment M_{bu} in the beam corresponding to the collapse of the column web panel is equal to :

$$M_{bu} = \min(M_{nbu};\ M_{buy};\ M_{bub}) \tag{3}$$

Shear of the web panel

The ultimate carrying capacity of a sheared web panel is given by :

$$V_{nbu} = \tau_u^c \cdot A_{sh} \tag{4}$$

where A_{sh} represents the column sheared area. τ_u^c is the ultimate shear stress evaluated by means of the von MISES criterion allowing for the interaction between σ_n and τ stresses - it has been shown in [1] that the load-introduction (stresses σ_t) constitutes only a local phenomena which has no direct influence on the global shear behaviour of the web panel - but based on the attainment of the ultimate stress f_u in the column web. τ stresses are simply obtained by dividing the shear force V_n (formula 2) by the column sheared area A_{sh}.

Excessive yielding of the web

The ultimate resistance associated to the excessive yielding of a web subject to transverse loading is given by :

$$M_{buy} = \sigma^{c}_{iu} \cdot s_c \cdot l_p \cdot d_b \qquad (5)$$

σ^{c}_{iu}, the maximum permissible compression stress in the web, results from the consideration of the local interaction between σ_i and τ stresses by means of the von MISES criterion based, as for the shear resistance, on the attainment of the ultimate stress f_u in the web. Expression of σ_i stresses versus the beam moment are proposed in [6] for the column web of joints with welded, end plate and flange cleated connections. d_b is the distance between the compression and tensile forces acting on the web. l_p is the length of diffusion, given in [6], of the compression and tensile forces through the connection elements, the flange and the radii of fillet of the column. s_c is the column web thickness.
This ultimate resistance is associated to the collapse of the tensile zone or the compression zone according to which is the most determining.

Web instability

The instability of the web under compression affects either the whole depth of the column (web buckling) or the region located just under the beam flange (web crippling).
The associated instability load is given by :

$$M_{bub} = M_{bb} \not> M_{bppl} \qquad (6.a.)$$

with :

$$M_{bb} = \sqrt{M_{by}\, M_{bcr}} \qquad (6.b.)$$

M_{bppl} is the pseudo-plastic moment of the web evaluated by formula (5) in which σ^{c}_{iu} is simply replaced by σ^{c}_{iy} which is based on the attainment of the yield stress f_y in the web.
M_{by} is the elastic resistance of the transversally loaded web which corresponds to the onset of yielding in the compression or tensile zones of the web. It results from the study of a "beam" (column flange) on an "elastic foundation "(column web) [1,6]; only the interaction between σ_i and τ stresses must be considered.

The elastic linear instability load of the web, M_{bcr}, is expressed as :

$$M_{bcr} = (h_c - 2.t_c).s_c.d_b.k.\frac{\pi^2 . E}{12.(1-\nu^2)} . \left(\frac{s_c}{h_c - 2.t_c}\right)^2 \qquad (7)$$

h_c is the total depth of the column while t_c and s_c are respectively the flange and web thicknesses of the column.

The values to give to the k coefficient as well as the physical explanation of formula (7) will be found in the following sub-sections.

It is important to mention that the buckling strength M_{bb}, contrary to the pseudo-plastic moment M_{bppl}, is strongly dependent on the initial out-of-flatness of the web (see for instance figure 5.a.). The values of this imperfection have been chosen on base of rolling tolerances and those of the k coefficient, which will be defined in the next sub-sections, have been calibrated accordingly. Measurements of the actual values of the initial out-of-flatness are generally found lower than the tolerances; that results in a too safe theoretical approximation of the actual buckling load. Numerical simulations have shown that the variation of the buckling load may reach 25 - 30 % according to the value of the out-of-flatness.
This has not to be forgotten when comparisons between theory and experiments are performed.

4.1. Interior HE columns

For what concerns the interior columns (figure 2.b.) with HE sections, it has been shown [1] that the pseudo-plastic moment, M_{bppl}, constitutes a lower bound value for the web buckling load.
This may be easily explained by referring to figure 8.

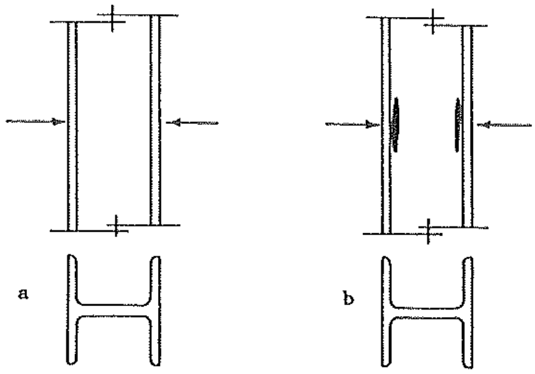


Figure 8 - Support conditions of the column web according to the load level

In the elastic range of the web behaviour (figure 8.a.), the column web may be considered as rigidly connected to the column flanges : its instability load is consequently high. The increase of the loading and the resultant yielding of the web just under the column flanges (figure 8.b.) lead to a modification of the support conditions for the web and consequently to a considerable decrease of the buckling load.
A collapse by instability in the elastic range behaviour has never been encountered, even for relatively slender column webs (HEA) and the numerical simulations have shown that the actual buckling load is always greater or equal to the pseudo-plastic moment M_{bppl} which then constitutes a lower bound value for the instability load.

The buckling strength M_{bb} given by formula (6.b.) is consequently based on the assumption that the web is pinned on the column flanges; it has to be compared to M_{bppl} in order to determine the actual buckling load of the web, M_{bub} :

$$M_{bub} = M_{bppl} \text{ if } M_{bppl} \geq M_{bb} \tag{8.a.}$$

$$M_{bb} \quad \text{if } M_{bppl} < M_{bb} \tag{8.b.}$$

The coefficient k which accounts in formula (7) for the type of loading - cross nodes symmetrically loaded - and for the web support conditions - ideal hinges - may be taken equal to 1.0.

4.2. Exterior HE columns

The fact that transverse loads are only applied to one side of the column web increases significantly the buckling strength M_{bb} of the web, whereas the pseudo-plastic moment M_{bppl} is independent of the node arrangement (cross or "T"). The k coefficient in formula (7) will consequently be chosen equal to 2.0 for "T" node.

The resulting formula (6) for the assessment of the web instability load, which has been discussed in the previous sub-section, may be applied in the case of exterior columns with HE sections.

4.3. Extension to IPE columns

Profiles with IPE sections being usually used as beams and not as columns, specific numerical simulations of joints with IPE columns have not been performed. Some test results (joints with end plate connections) are however available and the application of formula (6) has allowed to demonstrate its validity.

It must however be noted that the pseudo-plastic moment M_{bppl} corresponds rather to a web crippling than to a web buckling.

5. Comparison with results of numerical simulations and experimental tests

The here above proposed formulae have been validated in [6] through comparisons with numerous results of numerical simulations on welded joints and of experimental tests on joints with bolted connections.

6. Concluding remark

More information relative to the use of the formulae and also to the application of the mathematical model for the prediction of the characteristic deformability curves of joints may be found in the reports [1] and [6] which may be afforded to any interested people. An extensive comparison between the available numerical and experimental results and previously existing formulae for the assessment of the column web resistance and stability is included in [1].

7. References

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