INFLUENCE OF STRUCTURAL FRAME BEHAVIOUR ON JOINT DESIGN

Stephane GUISSE (1)

Jean-Pierre JASPART (2)

Abstract

Joints between H or I hot-rolled sections are subjected to internal forces induced by the connected members. The interaction between these forces can lead to a substantial reduction of the joint design resistance. Annex J of Eurocode 3 has been recently revised and, on the basis of theoretical models developed in Liege, the design rules for web panels have been modified to take those effects into account. Those new rules have been recently compared in Liege with new numerical simulations and experimental tests.

The aim of this paper is first to present briefly the modifications introduced in the Annex J of Eurocode 3 and related to the interaction of internal forces, and then to show the main conclusions that can actually be drawn from the comparisons with new numerical simulations and experimental tests.

1. INTRODUCTION

The department M.S.M. of the University of Liege is studying since several years at the semi-rigid response of structural joints and building frames. Since two years, a three years COST project funded by the Walloon Region of Belgium has started; it is aimed at investigating some different specific topics, related to the semi-rigid concept. The present paper concerns one of them.

It is well known that the joint behaviour has a major influence on the frame response. On the opposite side, the interaction between the forces acting in the connected members influences, generally in a negative way, the joint design resistance.

---

(1) Assistant, Department MSM, University of Liège, Quai Banning 6, 4000 LIEGE, BELGIUM

(2) Research Associate, Dr, Idem.

Annex J of Eurocode 3 (CEN / TC 250, 1994) has been recently revised to take those effects into account by means of some different reduction factors. One of the aims of the COST research project is to check again and possibly improve the expression of these last ones.

2. EFFECTS OF INTERNAL FORCES ON JOINT DESIGN RESISTANCE

2.1. WEB PANEL DESIGN RESISTANCE

In a strong axis joint between H or I hot-rolled sections, the collapse of the column web panel can result from two different modes: shear yielding or local yielding under the tension or compression forces carried over from the beam to the column by the connection (also called *load-introduction collapse*). For slender webs, a third mode (web buckling or web crippling) can also be observed.

For a given joint, the collapse mode of the web panel depends on its external loading; this is illustrated in figure 1 where the ratio η between the left and right loads, P_l and P_r, varies from 0 to 1. Figure 1 corresponds to a joint with a web of low slenderness, so, not likely to buckle.

The ratio η is the one between the two bending moments induced by the beams on each side of the column. When it is close to *zero*, the web panel is subjected to high shear forces what leads to a shear collapse. A ratio close from *one* means that the joint is symmetrically loaded; in this case, the collapse can only result from load-introduction yielding (web buckling or crippling is also possible for more slender webs)

In the joint web panel, three kinds of stresses are acting together:

- shear stresses τ;
- longitudinal stresses σ_n due to normal force and bending moment in the column;
- transverse stresses σ_i due to load-introduction (local effect).

The interactions between these stresses have different effects on the joint resistance:

- longitudinal stresses σ_n decrease the shear resistance;
- shear stresses τ decrease the load-introduction resistance;
- longitudinal stresses σ_n may decrease the load-introduction resistance (compression zone).

If the last kind of interactions is known for several years, the two first ones were not taken into account by the rules of the old version of Eurocode 3 Annex J (Eurocode 3, 1992). They have been pointed out by JASPART (Jaspart, 1991). The unsafe character of the previous Annex J, compared to the JASPART model, is represented by the hachured zone of figure 1.

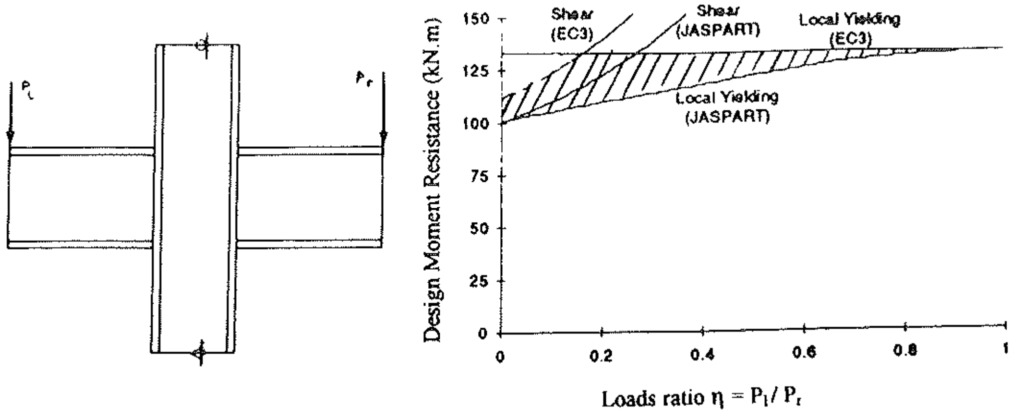

figure 1 - Variation of web panel resistance according to the external loading

Annex J has been recently completely revised: the rules concerning the web panel design resistance have been modified according to JASPART's proposals (Jaspart, 1991). The first modification concerns the shear resistance, $V_{wc,Rd}$. The influence of the longitudinal stresses σ_n has been simply taken into account with a constant reduction factor equal to 0,9:

$$V_{wc,Rd} = 0{,}9 \frac{A_{v,c} \cdot f_{ywc}}{\gamma_{m0} \cdot \sqrt{3}} \tag{1}$$

$A_{v,c}$ is the shear area of the profile, f_{ywc} the yield stress and γ_{m0} the partial safety factor. The second modification consists in a reduction of the design resistance $F_{wc,Rd}$ of the column web in tension or compression (equation 2), due to the possible presence of shear stresses, by means of a reduction factor ρ:

$$F_{wc,Rd} = \rho \frac{f_{ywc} \cdot t_{wc} \cdot b_{eff}}{\gamma_{m0}} \tag{2}$$

$$\text{with } \rho = \begin{cases} \rho_1 = \dfrac{1}{\sqrt{1 + 1{,}3 \left(\dfrac{b_{eff} \cdot t_{wc}}{A_{v,c}} \right)^2}} & \text{if } \eta = 0 \\ \rho_1 + (1 - \rho_1).2.\eta & \text{if } 0 < \eta < 0.5 \\ 1 & \text{if } 0.5 \le \eta \le 1 \end{cases} \tag{3}$$

t_{wc} is the web thickness and b_{eff} is the effective yielding length. This last parameter depends on the connection details. Equation (3), represented by two lines, is a simplification of the initial JASPART's proposal. The difference between the two approaches is illustrated in figure 2.

The effect of the longitudinal stresses σ_n on the resistance of the column web in compression is taken into account by means of an other reduction factor, k_{wc} (equation 4), that was already existing in the old Annex J (Zoetemeijer, 1975) :

$$k_{wc} = 1{,}25 - 0{,}5\frac{\sigma_{n,Ed}}{f_{ywc}} \leq 1 \tag{4}$$

$\sigma_{n,Ed}$ is the normal stress in the column web, at the root of the fillet or of the weld, due to longitudinal force and bending moment. The minimum value of k_{wc} is 0,75 (when $\sigma_{n,Ed}$ is equal to f_{ywc}). k_{wc} covers the possible buckling of the web panel under the combined action of the σ_i and σ_n compression stresses.

Finally, the last modification introduced in Annex J concerning web panel is the extension of the design rules to slender webs ($\bar{\lambda} > 0.673$) by limiting the design resistance, given in equation 2 to the buckling resistance value of the web:

$$F_{wc,Rd} = \rho\frac{f_{ywc} \cdot t_{wc} \cdot b_{eff}}{\gamma_{m0}} \qquad \text{if } \bar{\lambda} \leq 0.673$$

$$F_{wc,Rd} = \rho\frac{f_{ywc} \cdot t_{wc} \cdot b_{eff}}{\gamma_{m0}} \cdot \left[\frac{1}{\bar{\lambda}} \cdot (1 - \frac{0.22}{\bar{\lambda}})\right] \qquad \text{if } \bar{\lambda} > 0.673 \tag{5}$$

$$\text{with } \bar{\lambda} = 0.93\sqrt{\frac{b_{eff} \cdot d_c \cdot f_{ywc}}{E \cdot t_{wc}^2}}$$

d_c is the clear depth of the column web, E the Young modulus and the other parameters are given here above.

Equations 1 to 5 are discussed in the present paper on the basis of comparisons with numerical simulations (section 3) and experimental tests (section 5).

2.2. COLUMN FLANGE DESIGN RESISTANCE IN BOLTED JOINTS.

In bolted joints (with flush or extended endplates, flange cleats), the column flange is subjected to transverse forces. The design resistance of this component, given in Annex J of Eurocode 3 - it was already in the old Annex J -, is reduced because of possible high longitudinal stresses $\sigma_{com,Ed}$ (> 180 MPa) in the column flange by means of a reduction factor k_{fc} (Zoetemeijer, 1975) equal to :

$$k_{fc} = \frac{2 \cdot f_{y,fc} - 180 - \sigma_{com,Ed}}{2 \cdot f_{y,fc} - 360} \leq 1 \tag{6}$$

$f_{y,fc}$ is the yield stress of the column flange.

3. NUMERICAL SIMULATIONS

A large set of numerical simulations have been performed in Liege with the non linear finite element program FINELG (Finelg, 1994). This software is developed in Liege since several years. It is able to simulate the behaviour of structures until collapse and to take into account various phenomena such as non linear mechanical properties, second order effects, residual stresses, initial imperfections...

The welded joints have been modelled with shell elements (figure 2). These numerical 3D simulations, based on the geometry of experimental tests performed some years ago in INNSBRUCK (Klein, 1985), provide very interesting informations such as strains and stresses anywhere in the joint, or global behaviour curves which allow direct comparisons with the theoretical models of JASPART and Eurocode 3 revised Annex J.

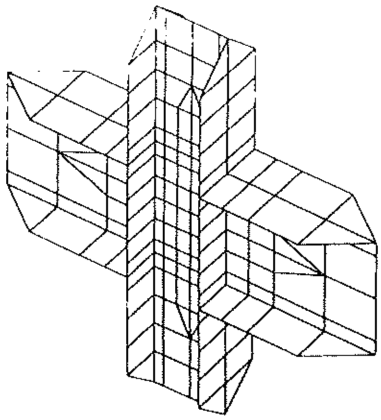

figure 2 - Discretisation of welded joints.

More than hundred simulations of welded joints have been performed in order to investigate the effect of the following parameters: dimensions of the joint (three different geometries have been considered), loading, steel grade, effect of strain-hardening and effect of the initial imperfection of the column web.

But the two main parameters of this parametrical study were the loading factors η (figure 1) and the ratio β between the normal force in the column and its squash load.

Figure 3 illustrates the evolution of the joint design resistance versus the loading ratio η. The graph has an extremum at about η=0.7 and its shape is different from both the theoretical prediction of the new Annex J, and the analytical model developed by JASPART. Despite these differences, the agreement between the models and the numerical simulations can be considered as good.

The effect of a normal force in the column on the joint behaviour (represented by the value of β, see above) has also been considered for some joint configurations, either symmetrically loaded (η=1) or just loaded on one side (η=0). Generally, the agreement between equation (4), though as very simple, and the numerical simulations is good. However, some joint configurations lead to a higher difference between the numerical results and the theoretical rules. In particular, the steel grade seems to have a great influence on the normal force effect: figure 4 shows that more the steel resistance is high, more the Eurocode rule is accurate.

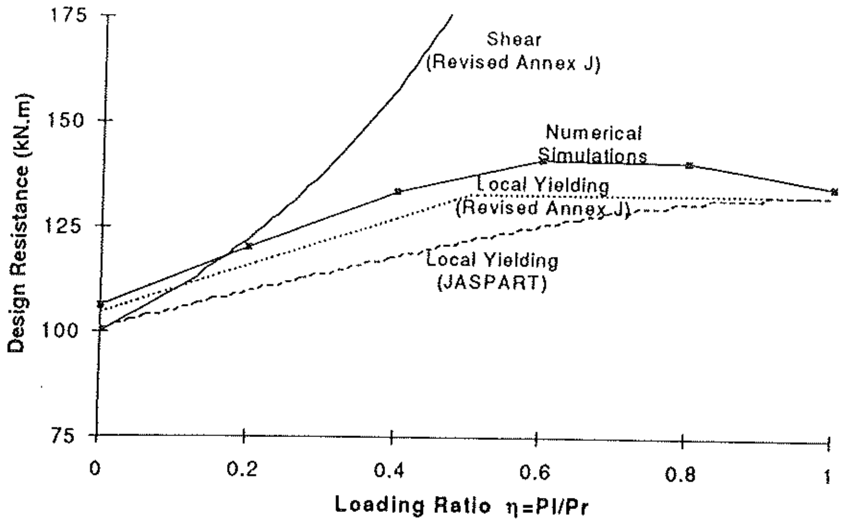

figure 3 - Variation of the joint resistance with the loading ratio η.

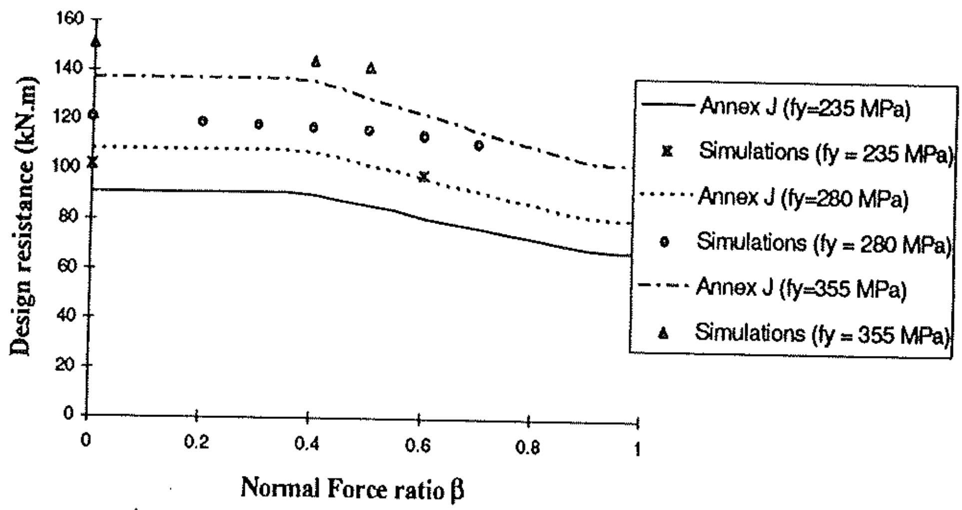

figure 4 - Example of the variation of the joint resistance with the normal force in the column and the steel grade. Cruciform joint (η*=1).*

4. NEW THEORETICAL MODEL

In order to understand the shape of the curve got by the numerical simulations in figure 3, a new theoretical model has been developed, inspired by ROBERTS and JOHANSSON theories (Roberts, 1991): the column flange connected to the beam is considered as a rigid-plastic beam lying on a rigid-plastic support (the column web). The new model reproduces precisely the results of the numerical simulations. The comparison of the model with the experimental results is actually in progress. It will be presented in a next paper.

5. EXPERIMENTAL TESTS

Recently, 24 experimental tests on joints have been performed at the University of Liege. 16 of them were welded joints while the others were bolted ones, with flush endplates. The results of tests on bolted joints are not reported in this paper.

Two different welded joint configurations (same beams, same columns) have been tested experimentally. The first one is dedicated to the study of the shear stresses effect. Its generic name is « WS » (as Welded and Shear). It is represented in figure 5. The second is aimed at testing the effect of the normal force in the column; it is called WN (as Normal force). For the first one, the only parameter is the ratio η between the forces applied on the left and right beams.

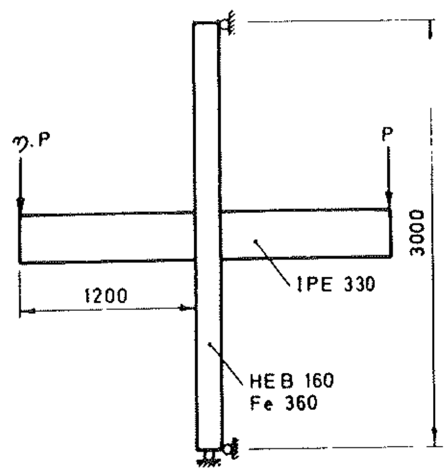

figure 5 - Example of test configuration.

Experimental tests provide a lot of different informations. In particular, figure 6 shows the complete M-φ curves for six of the eight WS tests. The joint design resistance is different from the ultimate one: the first one is calculated by neglecting the strain-hardening in the joint. If the ultimate resistance can be identified as « the top » of the M-φ curve, the design one is much more difficult to determine precisely. The procedure used in this paper is derived from the stiffness model of the revised Annex J (see related paper in these proceedings): the design resistance is identified as the intersection point between the actual M-φ curve and a straight line characterised by a slope equal to the third of the initial stiffness of the curves. This last value is almost theoretical values have been calculated on the basis of the actual yield stresses, of dimensions measured in laboratory and of a partial safety factor equal to 1.

While the theoretical predictions can be considered as very good for small and intermediate values of η, they seem to be too safe for the symmetrically loaded joints. Such a difference between tests and Annex J rules have never been observed before; some investigations are actually in progress to explain this difference.

On an other hand, the design resistances of WS8 and WS10 tests are almost equal to the ultimate ones. This can be explained by the fact that the column web buckles as soon as it has been crushed.

For WN tests, the length of the column is smaller to avoid any buckling problem, due to the high compression force. The tests are realised in two steps: first, the normal force is progressively applied to the column until the nominal value; after, the loads on the beams are increased until the joint collapse while the normal force in the column is kept constant.

Figure 8 shows a comparison between the results of the WN tests and the Annex J rules (equation 4). Four of the 8 tests, called WN S, were symmetrically loaded whilst the other ones were completely unbalanced (η=0). For one of these four last tests, some experimental problems have been observed; so, it is not reported here.

The number contained in the test name is here relative to ten times the nominal ratio β, between the normal force in the column and its squash load. For example, the test WN5 S is a test characterised by a ratio β of almost 0,5 and symmetrically loaded.

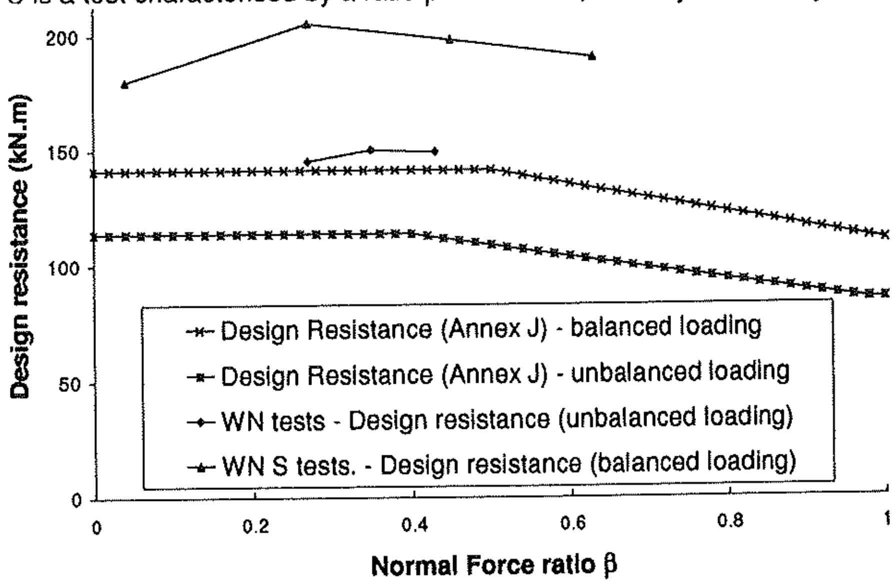

figure 8 - Comparison between results of WN tests and Eurocode 3 Annex J.

The normal force in the column seems to have no effect on the joint resistance despite the substantial value of the normal force applied. This one is limited by the

the same for each test (see figure 6). The number contained in the test name is equal to ten times the ratio η between the left and right forces. For example, WS2 is a test characterised by a ratio η of 0,2.

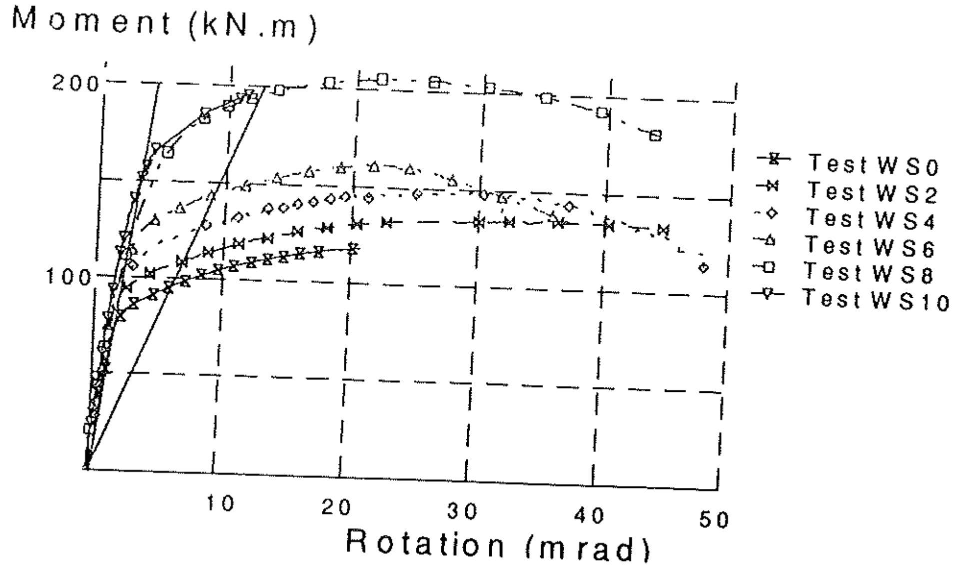

figure 6 - Results of WS tests (M-φ curves).

Figure 7 gives a comparison between the experimental design resistance for the WS tests, derived as described here above, and the prediction according Eurocode 3.

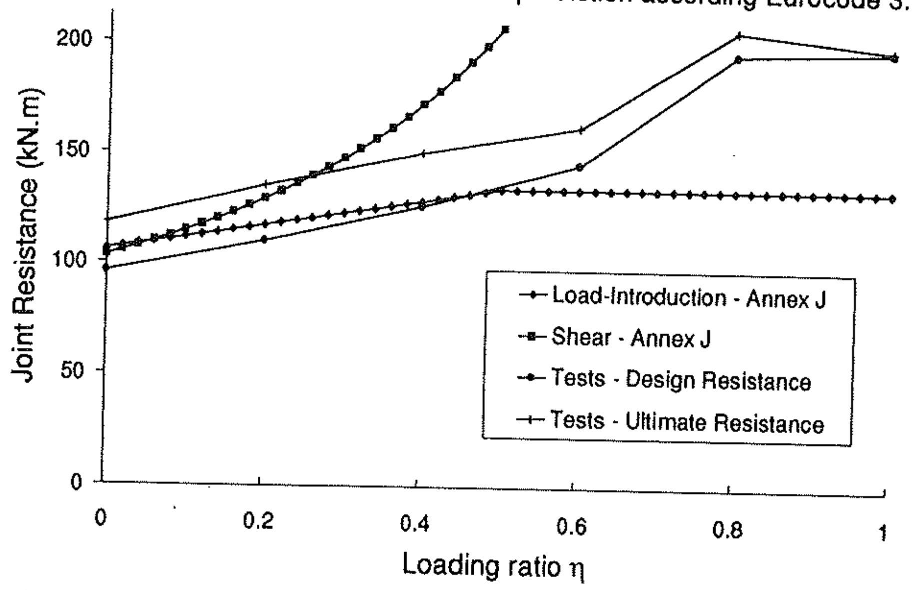

figure 7 - Comparison between WS tests results and Annex J of Eurocode 3.

The shape of the variation of the design resistance with the loading ratio is almost the same than the one observed for the numerical simulations (figure 3). The buckling of the column, as well for the tests . .n in the reality. The rules of the Eurocode 3 revised Annex J are conservative, especially for the symmetrically loaded joints. With regard to both the results of the numerical simulations (figure 4) and of the experimental results (figure 6), equation (4) can be considered as overconservative.

6. CONCLUSIONS

Considering either the results of numerical simulations or the experimental tests, the modifications of the design rules of annex J of Eurocode 3 appear to be really pertinent, tough as too safe in some circumstances.

Anyway, the effect of stress interactions, pointed out by JASPART a few years ago, is confirmed by the two different approaches.

Further improvements are actually in progress.

7. REFERENCES

CEN / TC 250, 1994, New Revised Annex J of Eurocode 3 : Part 1.1, CEN document N419 E.

EUROCODE 3, 1992, Design of Steel Structures, Part 1.1 : General Rules for Buildings, European Prestandard, ENV 1993-1-1, February 1992.

FINELG, 1994, Non Linear Finite Element Analysis Program, Version 6.2, User's Manual, Department MSM, University of Liège, BEG Design Office.

JASPART, 1991, Etude de la semi-rigidité des noeuds poutre-colonne et son influence sur la résistance et la stabilité des ossatures en acier, Ph.D. Thesis, Department MSM, University of Liège.

KLEIN, 1985, Das elastisch-plastische Last-Verformung verhalten Mb-θ steifenloser, geschweißter Knoten für die Berechnung von Stahlrahmen mit HEB-Stützen. Dr Dissertation, Univ. Innsbruck.

ROBERTS - ROCKEY, 1979, A mechanism solution for predicting the collapse loads of slender plate girder when subjected to in-plane patch loading. Proc. Instn Civ. Engrs Part 2, pp 155-175.

ZOETEMEIJER, 1975, Influence of normal-, bending- and shear stresses in the web of European rolled sections. Report N° 6-75-18, Stevin Laboratory, Delft University of Technology.