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STRESS INTERACTION IN COLUMN WEBS

1. INTRODUCTION

Department M.S.M. of University of Liège is dealing for several years with 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 begun in order to investigate some different specific topics of the semi-rigid concept. The present paper concerns one of these topics: « the influence of stress interactions in the column web panel of strong axis beam-to-column joints ».

The interaction of the different stresses acting in the web panel of a strong axis joint, resulting from the internal forces in the connected members, leads to a decrease of its design resistance. One of the aims of the COST research project is to improve the existing reduction factors by limiting their possible conservative character.

2. DESIGN RESISTANCE AND JOINT DEFORMATIONS

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 (figure 1) or local yielding under the tension or compression forces carried over from the beam to the column by the connection (load-introduction - figure 2). For slender webs, a third mode, the web buckling or web crippling, can also be observed.

For a given joint, the collapse mode of the web panel depends on the loading; this is illustrated in figure 3 where the ratio η between the left and right loads varies from 0 to 1; figure 3 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 force which can lead to a shear collapse (see figure 1), whilst a ratio close from *one* means that the joint is symmetrically loaded. In this case, the collapse could only result from load-introduction yielding (see figure 2) or web buckling or crippling.

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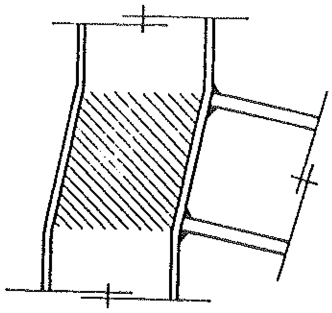

figure 1 - Shear collapse

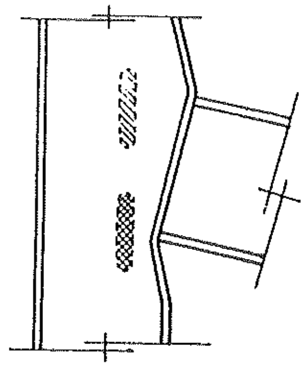

figure 2 - Collapse under load-introduction

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;
- transversal stresses σ_i due to load-introduction (local effect).

JASPART has shown (Jaspart, 1991) that 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 also decrease the load-introduction resistance.

Except for the last one, these effects were not taken into account by the rules of the old version of Annex J of Eurocode 3 (Eurocode 3, 1992), whom unsafe character is represented by the hachured zone of figure 3, compared to the JASPART model.

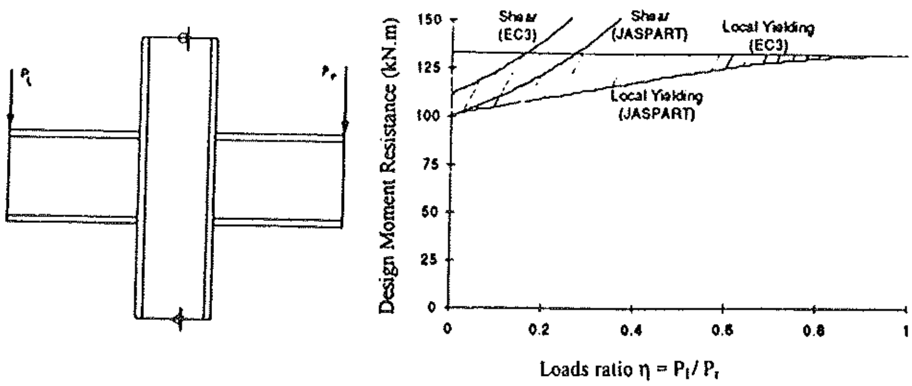

figure 3 - Variation of web panel resistance according to the loading

Annex J of Eurocode 3, dealing with joint resistance problem, has been recently completely revised; it's now able to cover a large set of connections such as welds, bolted endplates, cleats... In particular, 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. The influence of longitudinal stresses σ_n has been simply taken into account with a reduction factor equal to 0,9 (equation 1).

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

$A_{v,c}$ is the shear area of the profile, f_{ywc} the yield limit and γ_{m0} the partial safety factor.

The second modification consists in a reduction of the design resistance 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 (equation 3).

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

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

η is the ratio between the concentrated tension or compression loads carried over by left and right beam connections, 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 proposal. The difference between the two approaches is illustrated in figure 4.

The effects 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, $k_{r,wc}$ (equation 4), that was already existing in the old version of Annex J. The reduction of the load capacity, in this case, results from buckling of the web panel under the effect of the two compression stresses that are acting together: σ_i and σ_n.

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

$\sigma_{n,ed}$ is the normal stress in the column web, due to longitudinal force and bending moment, at the root of the fillet or of the weld. The minimum value of $k_{r,wc}$ is 0,75; it means that the maximum reduction of the joint resistance due to the normal force is equal to 25 percent.

Finaly, the last modification introduced in Annex J is the extension of the design rules to slender webs by the limitation of the design resistance, given in equation 2, by the buckling resistance value (equation 5).

$$\left| \begin{array}{l} F_{wc,Rd} = \rho \dfrac{f_{ywc} \cdot t_{wc} \cdot b_{eff}}{\gamma_{m0}} \leq \dfrac{f_{ywc} \cdot t_{wc} \cdot b_{eff}}{\gamma_{m0}} \cdot \left[\dfrac{1}{\bar{\lambda}} \cdot (1 - \dfrac{0.22}{\bar{\lambda}}) \right] \\ \text{where } \bar{\lambda} = 0.93 \sqrt{\dfrac{b_{eff} \cdot d_c \cdot f_{ywc}}{E . t_{wc}^2}} \end{array} \right. \qquad (5)$$

d_c is the clear depth of the column web, E the Young modulus and the other parameters are given here above. The reduction factor in equation 5 is lower than 1 only if $\bar{\lambda}$ is greater then 0,673.

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

3. NUMERICAL SIMULATIONS

A large set of numerical simulations have been performed in Liège with the nonlinear finite element program FINELG (Finelg, 1994). This one is developped in Liège for several years. It is able to describe the behaviour of structures from beginning of loading to collapse, taking into account various phenomenone such as non linear mechanical properties, second order effects, residual stresses, initial imperfections...

In the present context, joints have been modelled with shell elements.

These numerical 3D simulations, based on the geometry of experimental tests performed a few years ago in INNSBRUCK (Klein, 1985), provide very interresting informations such as strains and stresses anywhere in the joint, or global behaviour curves which allow direct comparisons with the theoretical model of JASPART. (Jaspart, 1991).

More than hundred simulations 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;
- effect of the initial imperfection of the column web.

But the two main parameters of this parametrical study were the loading factors η (figure 3) and the ratio between the normal force in the column and its squash load. This second ratio is called β.

Figure 4 illustrates the evolution of the design resistance versus the loads 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 developped by JASPART. Despite these differences, the agreement between the models and the numerical simulations can be considered as good.

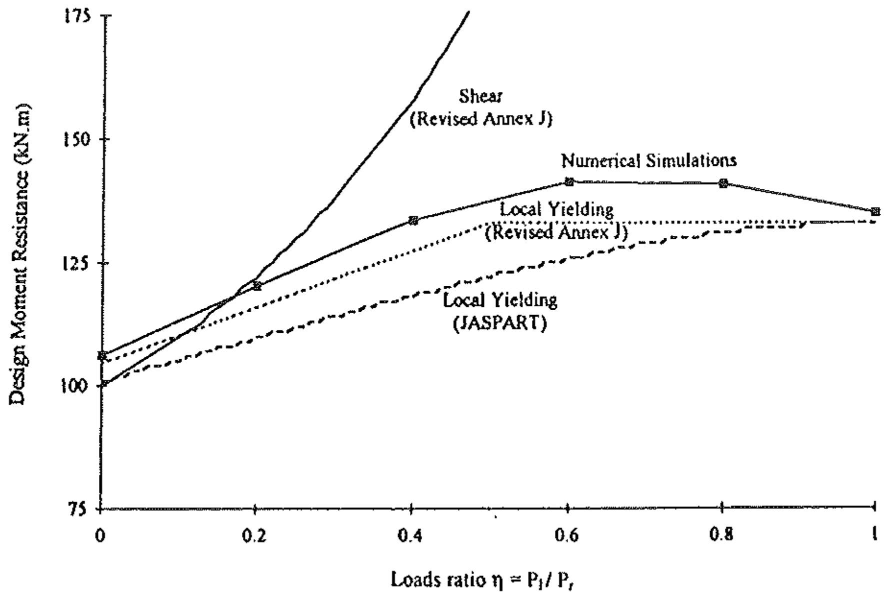

figure 4 - Variation of the joint resistance with the loads ratio η.

The effect of 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 very good.

However, some joint configurations lead to a higher difference between the two approaches. In particular, the steel grade seems to have a great influence on the normal force effect: figure 5 shows that more the steel resistance is high, more the Eurocode rule is accurate. This could be simply explained: the load capacity reduction given by equation 4 results from buckling problems that concern especially the higher steel grades.

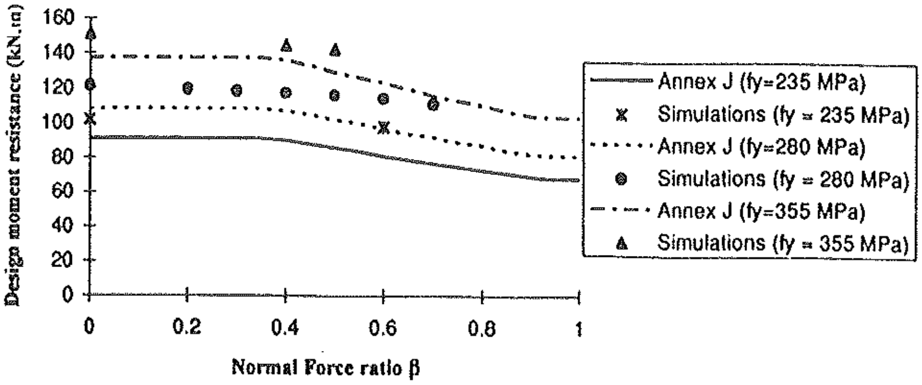

figure 5 - Variation of the joint resistance with the normal force in the column and the steel grade.

4. NEW THEORETICAL MODEL

In order to understand the shape of the curve got by the numerical simulations in figure 4, a new theoretical model has been developped, inspired by ROBERTS and JOHANSSON theories (Roberts, 1991). It is based on the following assumptions:

- the column flange connected to the beam is considered as a rigid-plastic *beam* liying on a rigid-plastic support: the column web. The resistance of this one depends on the level of shear stresses in the web;
- due to the possible buckling, tension and compression zones have different behaviours;
- the web panel in the compression zone is modelled by a rectangular column whom buckling load can be derived from the ECCS buckling curves;
- the resistance of the plastic hinges in the column flange is calculated with or without the collaboration of a part of the column web, depending on the strain state in the region of the hinge .The effective height considered for the web is also linked to the level of the shear stresses in the joint.

The new model is able to reproduce precisely the results of numerical simulations. It can explain why the slope of the numerical curve given in figure 4 is first characterized by a positive slope and then by a negative one. It has however to be refined by some further theoretical investigations.

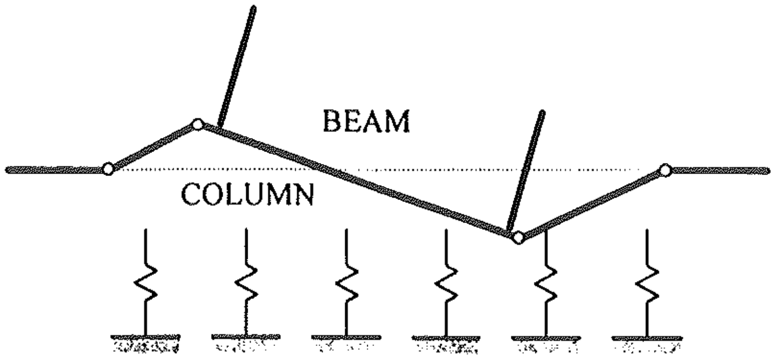

figure 6 - New theoretical model, applied to a welded joint.

5. EXPERIMENTAL TESTS

Recently, 24 experimental tests on joints have been performed at the University of Liège. 16 of them concern welded joints, the others are bolted ones. The objective was to improve the new design rules of Annex J of Eurocode 3 (see equations 1 to 5). Two different welded joints configurations (same beams, same columns) tested experimentally are represented in figure 7. The first one is dedicated to the study of the shear stresses effect. Its generic name is « WS » (as Welded and Shear). The second is aimed to test the effect of the normal force in the column and 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.

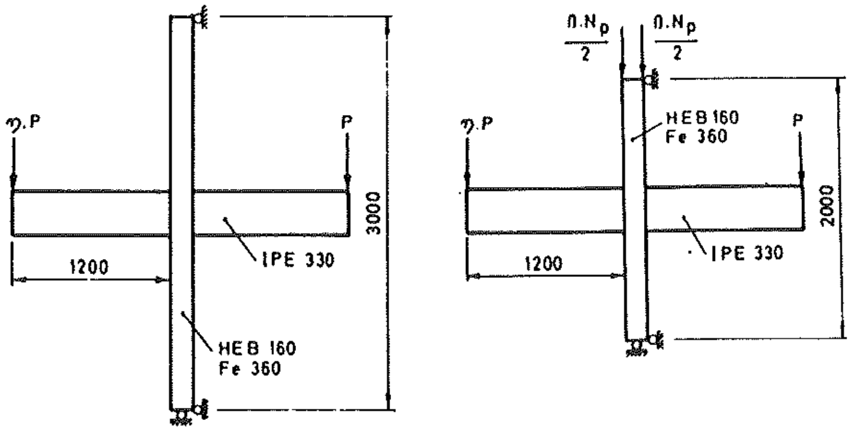

figure 7 - Tests configurations.

Figure 8 is a comparison between the prediction of Eurocode 3 and the experimental design resistance for the WS tests. The shape of the variation of the design resistance with the loads ratio is almost the same than the one observed for the numerical simulations (figure 4). The theoretical values have been calculated on the basis of the yield limit and dimensions measured in laboratory and with a partial safety factor equal to 1.

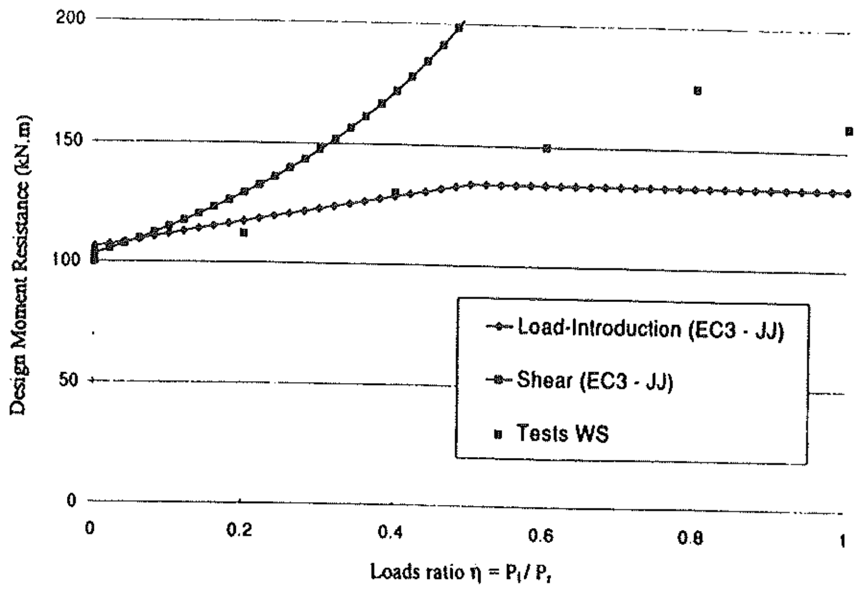

figure 8 - Comparison between WS tests results and Annex J of Eurocode 3.

The agreement between tests and predictions can be considered as very good for unbalanced loadings (η close to 0) and a little too safe for the symetrically loaded joints. The greatest difference is relative to a value of η of 0,8 almost as for the numerical simulations.

The length of the column of the WN tests is smaller to avoid any buckling problem. The test is realized 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 collapse of the joint while the normal force in the column remains constant.

Figure 9 gives a comparison between the results of the WN tests and the theoretical recommendations (equation 4). Four of the 8 tests were symmetrically loaded (WN S - η=1) whilst the other ones were completely unbalanced (η=0).

The agreement between the prediction and the experimentation is once again good. 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 buckling of the column, as well for the tests than in the reality.

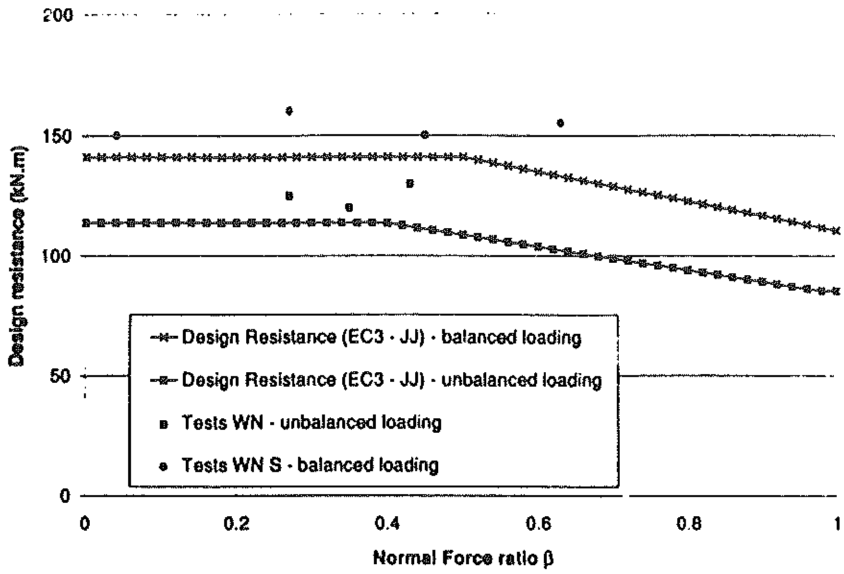

figure 9 - Comparison between WN tests results and Annex J of Eurocode 3.

Equation 4 can be considered close enough of the experimental results, and on the safety side. Some more developments are however necessary to investigate further the exact influence of some parameters such as the steel yield limit and the web slenderness.

6. CONCLUSIONS

Regarding either to the numerical simulations results or the experimental tests, the modifications of the design rules of annex J of Eurocode 3 appear to be really pertinent. The effect of stress interactions, pointed out by JASPART a few years ago, is confirmed by the two different approaches.

Furthers improvements are actually in progress.

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INTERAKCJA NAPRĘŻEŃ W ŚRODNIKACH SLUPÓW

Streszczenie

W połączeniach rygli ze słupami (przekroje H lub I) środnik słupa poddany jest działaniu dwukierunkowych naprężeń normalnych i naprężeniom ścinającym. Współzależności między tymi naprężeniami mogą prowadzić do zasadniczego zmniejszenia obliczeniowej nośności połączenia.

W poprawionym ostatnio Załączniku J do Eurocodu 3, zmodyfikowano między innymi zasady projektowania środników w taki sposób, aby brać pod uwagę wspomniane współzależności. Te nowe zasady porównano ostatnio w Liege z symulacjami numerycznymi i z wynikami doświadczalnymi.

Celem artykułu jest zaprezentowanie w skrócie modyfikacji wprowadzonych w Załączniku J do Eurocodu 3, dotyczących projektowanej nośności środnika, a następnie pokazanie zasdniczych wniosków, które mogą być wyciągnięte z porównań modyfikacji z symulacjami i z wynikami eksperymentalnymi.