APPLICATION OF THE COMPONENT METHOD TO STEEL JOINTS

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ABSTRACT: In Revised Eurocode 3 Annex J (hereafter: Annex J) the design of steel joints is based on the so-called component method. In Annex J design rules are provided to determine the strength, stiffness and deformation capacity of individual components (for instance bolts in tension, column web in compression etc.). The overall joint behaviour may be assessed by assembling the mechanical characteristics these individual components.

Annex J mainly focuses on major-axis joints between European hot rolled sections or welded sections with similar geometry in mild steels. For other forms of connection, Annex J doesn't provide design information for all components.

In the frame of COST C1 Working Group 2 (WG2), fundamental research has been carried out to new components, so as to cover for instance minor-axis joints, joints between slender sections and joints between higher strength steels. Also research has been carried out to other section types than I and H sections, for instance hollow core sections.

In this paper a review on the research activity to new components in the frame of COST C1 project is given. The paper shows that knowledge has been obtained about the mechanical behaviour of new components. The project results have been deeply discussed in a group of European scientists.

A next step should be to transfer the scientific knowledge obtained during the COST C1 project to simple and easy-to-apply design guidance for practitioners.

1. INTRODUCTION

The mechanical behaviour of steel joints in terms of strength, stiffness and rotation capacity is a complex phenomena. To determine this complex behaviour, the joint can be decomposed into different parts, the so-called components. A component forms an identity in a joint and may include more than just a bolt or steel plate. For instance, in beam-to-column joints, an end plate in bending forms a component, but this component can include an extended part and transfer loads through several bolt rows and a variety of welds.

The mechanical behaviour of these components is studied separately. When all components of a joint have been characterised in terms of strength, stiffness and deformation capacity, the mechanical behaviour of a joint can be determined by assembling the individual contributions of the components with help of mechanical models.

In Annex J [1, 2] design rules are given for a number of components. With help of these rules, the strength, stiffness and rotation capacity of variety of steel joints can be determined. Annex J mainly focuses on major-axis joints between European hot rolled sections or built-up sections with similar dimensions made of mild steels [3]. The list of components in Annex J and the design rules for these components do not cover other types of connection, for instance, minor-axis joints, joints between slender built-up sections and joints between I beams and tubular columns. The reason is that when Annex J was written, no backgrounds were available for these types of connection.

In the frame of the COST C1 Working Group 2, studies have been carried out on a number of components not covered in Annex J. In this paper, a review of these studies is given. These studies were carried out to obtain fundamental knowledge about the behaviour of new components. The resulting models sometimes have a high level of refinement in order to describe the mechanical behaviour of the components as accurate as possible. This is an important step in a process to obtain simple design models which are applicable in practice.

In the next paragraphs of this paper, a review is given of the research to components in:
- minor-axis joints;
- joints in higher steel grades;
- joints between slender sections.

The paper finishes with conclusions and recommendations for future actions.

2. MINOR-AXIS JOINTS

2.1 Introduction

In beam-to-column minor-axis joints the beam is directly connected to the web of an I-section column, causing bending about the minor-axis of the column section. Figure 1 shows some common types of minor-axis joints where the beam is connected to the column web without stiffeners. The connection can be welded or bolted using web cleats, flange cleats, flush end plates or extended end plates. Annex J provides design rules to evaluate the behaviour of connecting elements (end plates, cleats, bolts) but it does not cover the component "out-of-plan deformation of the column web". Different failure mechanisms of the column web have been analysed [5 - 7], and it should be emphasised that the same kind of mechanisms can be observed in the case of connections between a beam and a rectangular hollow section (RHS) column, see Figure 2.

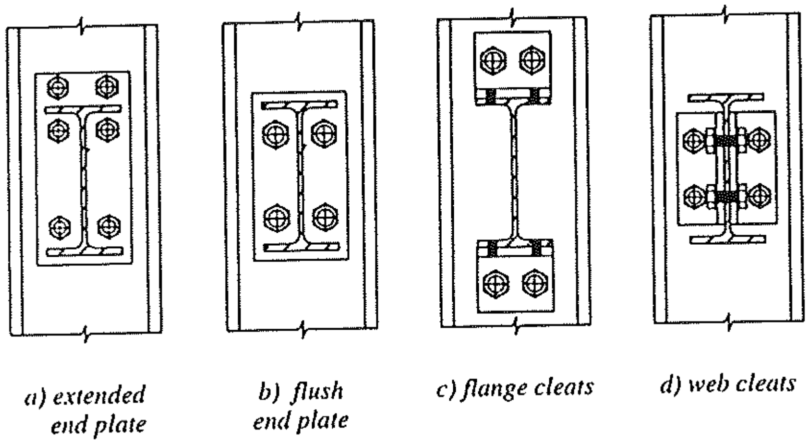

Figure 1: Beam-to-I section column minor-axis joints

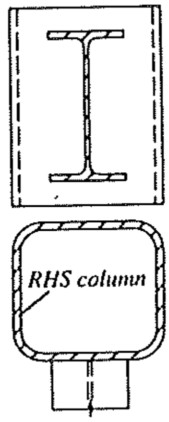

Figure 2: Beam-to-RHS column joint

For the evaluation of moment capacity and rotational stiffness, the moment transmitted by the beam to the column web is decomposed in a couple of forces F. It is assumed that these two forces are equal and that each force F acts on an area (compression or tension zones) defined in the plane of the column web as follows:

- *Loading case* 1: the force F acts on a rigid rectangle with the dimensions $b \times c$, see Figure 7, as in the case of a welded connection where these dimensions are defined by the perimeter of the welds around the beam flange;
- *Loading case* 2: the force F is transmitted to the column web by one or more rows of bolts, as in the tension zone of the joints represented in Figure 3 and Figure 8, where one single beam is connected to the column web. In this case, the definition of the loaded area depends on the distance between bolts and on the diameter of bolt heads (or nuts):

$$\begin{cases} b = b_0 + 0.9d_m \\ c = c_0 + 0.9d_m \end{cases} \tag{1}$$

where b_0 and c_0 are the distances between bolt centres as indicated in Figure 3, and d_m is the mean diameter of the bolt head (or nut), see Figure 4:

$$d_m = \frac{d_1 + d_2}{2} \tag{2}$$

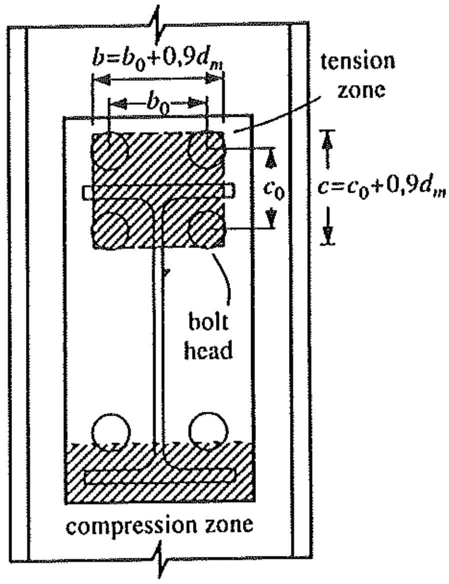

Figure 3: Single sided joint

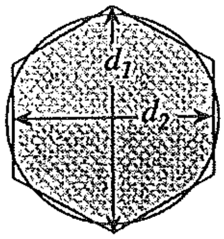

Figure 4: Bolt head (or nut). Definition of d_m

2.2 Strength of minor-axis joints

2.2.1 Assembly procedure for strength

The moment resistance of a minor-axis joint can be calculated based on the methods as proposed in Annex J. For instance, for an end plated minor-axis joint, the following components need to be taken into consideration:

- column web in bending and punching;
- bolts in tension;
- end plate in bending;
- beam web in tension;
- beam flange and web in compression.

For the determination of the strength properties of all components, design guidance is given in Annex J, except for the component column web in bending and punching. This paper pays attention to this specific component.

The design value of the moment resistance can be calculated as follows:

$$M_{j.Rd} = z \cdot F_{Rd} \tag{3}$$

where z is the lever arm in the joint;
F_{Rd} the resistance of the weakest component in the minor-axis joint

For a full description of assembly procedures, it is referred to [4]. In the next paragraph, attention will be paid to the strength of the component column web in bending and punching.

2.2.2 Strength of column web in bending and punching

In order to determine the strength of a column web in bending and punching, all admissible failure mechanisms of the column web should be analysed. These failure mechanisms are divided into two main groups:

- *Local mechanism:* the yield line pattern is localised in the compression zone or in the tension zone, see Figure 5. This mechanism is associated to the force $F_{Rd.local}$ given by equation (4).
- *Global mechanism:* the yield line pattern involves both compression and tension zones, see Figure 6. This mechanism is associated to the force $F_{Rd.global}$ given by equation (8).

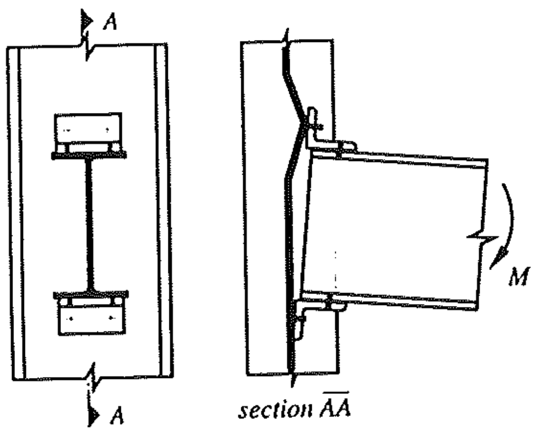

Figure 5: Local mechanism

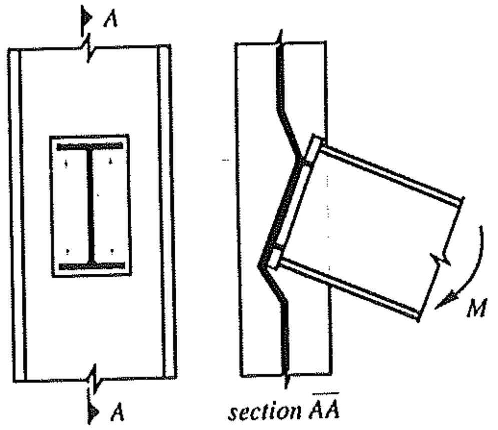

Figure 6: Global mechanism

In the design model it is assumed that prying action between end plate or the angle cleat doesn't occur. This assumption is conflicting with the assumptions made in Annex J. There it is assumed that prying develops in case yielding occurs in either end plate or cleat. This point is still under investigation. In most practical cases, the end plate or the cleat is stiffer and stronger then the column web, and it is reasonable to assume that no prying develops between the components.

Local failure

Different local mechanisms of the column web are considered. The local failure mechanism is the mechanism associated to the smallest plastic force:

$$F_{local.Rd} = min\left(F_{punch.Rd}\ ; F_{comb.Rd}\right) \tag{4}$$

where $F_{punch.Rd}$ is the resistance to punching shear;
$F_{comb.Rd}$ is the resistance to combined punching, shear and bending,
given by equations (5) and (6) respectively.

The resistance to punching depends on the loading case. For the *loading case* 1 the punching perimeter is $2(b+c)$. For the *loading case* 2 the punching perimeter of the column web depends on the diameter of the bolt heads (or nuts) and on the number n of bolts in the tension zone. The resistance to punching is then

$$F_{punch.Rd} = \begin{cases} 2(b+c)\, t_{wc}\, f_y / (\sqrt{3} \cdot \gamma_{MO}) & \text{for the loading case 1} \\ n\, \pi\, d_m\, t_{wc}\, f_y / (\sqrt{3} \cdot \gamma_{MO}) & \text{for the loading case 2} \end{cases} \tag{5}$$

where t_{wc} is the thickness of the column web and
f_y is the yield strength of the column web;
γ_{M0} is the partial safety factor of steel.

Combined flexural and punching shear mechanisms manifest itself by flexural yield lines (thick lines in Figure 7) and punching shear yield lines (dotted lines in Figure 7). Mechanisms with only flexural yields lines, see Figure 8, are particular cases for which x=0 in equation (6). For the *loading case* 2 (bolted connections) the dimensions $b \times c$ of an equivalent rectangle are defined by equations (1).

$$F_{comb.Rd} = k\, t_{wc}^2 f_y \left[\frac{\pi \sqrt{L(a+x)}\ + 2c}{a+x} + \frac{1.5\, c\, x + x^2}{\sqrt{3}\, t_{wc}\ (a+x)} \right] / \gamma_{MO}, \tag{6}$$

where: $$k = \begin{cases} 1 & if (b+c)/L > 0.5 \\ 0.7 + 0.6(b+c)/L & if (b+c)/L \leq 0.5 \end{cases} \tag{7}$$

$$a = L - b$$

and $$x = \begin{cases} 0 & if\ b \leq b_m \\ -a + \sqrt{a^2 - 1.5\, ac + \frac{\sqrt{3}\, t_{wc}}{2} \left[\pi \sqrt{L\left(a + x_0\right)} + 4c \right]} & if\ b > b_m \end{cases}$$

in which $x_0 = L\left[\left(\frac{t_{wc}}{L}\right)^{\frac{2}{3}} + 0.23\frac{c}{L}\left(\frac{t_{wc}}{L}\right)^{\frac{1}{3}}\right]\left(\frac{b-b_m}{L-b_m}\right)$

and $b_m = L\left[1 - 0.82\frac{t_{wc}^2}{c^2}\left(1+\sqrt{1+2.8\frac{c^2}{t_{wc}L}}\right)^2\right]$, but $b_m \geq 0$

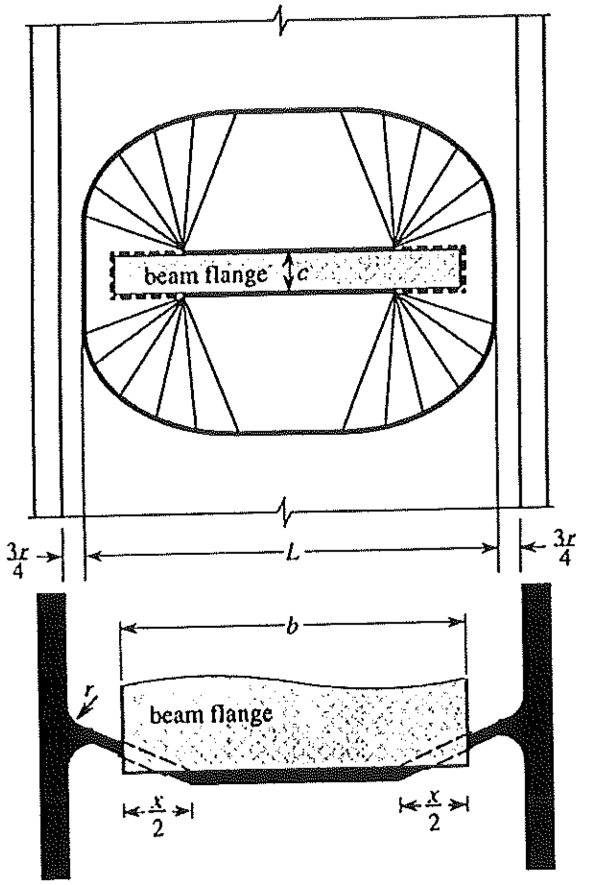

Figure 7: Combined flexural and punching shear failure

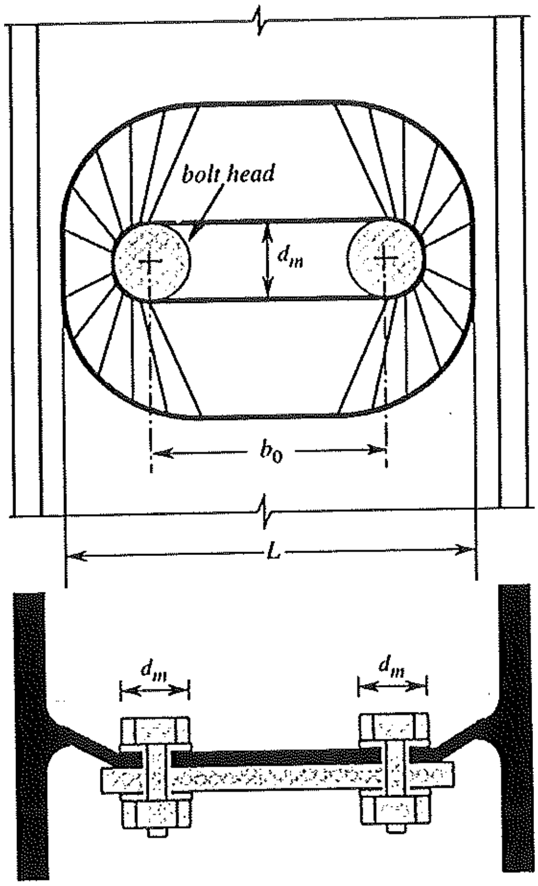

Figure 8: Flexural mechanism in the tension zone of a bolted connection

Global failure

The global failure force, for either flexural mechanisms or combined flexural and punching mechanisms, may be evaluated as:

$$F_{global.Rd} = \frac{F_{comb.Rd}}{2} + \frac{t_{wc}^2 f_y}{4}\left(\frac{2b}{z} + \pi + 2\rho\right) / \gamma_{M0} \tag{8}$$

where $F_{comb.Rd}$ is given by equation (6) and

$$\rho = \begin{cases} 1 & if \quad \dfrac{z}{L-b} \leq 1 \\ \dfrac{z}{L-b} & if \quad 1 \leq \dfrac{z}{L-b} \leq 10 \end{cases} \tag{9}$$

Global failure mechanisms involve both compression and tension zones, see Figure 6. If the dimensions $b \times c$ of the compression zone are different from those of the tension zone, e.g. Figure 3, equation (8) should be applied separately for each zone, leading to two different forces, and the actual failure force will be an intermediate value. However, when the two zones are equal, or assumed to be equal, equation (8) will be used only once [7].
The theoretical predictions have been successfully compared with experimental results [8] and with a large number of numerical simulations [9, 10].

2.3 Rotational stiffness of minor-axis joints

2.3.1 Assembly procedure for stiffness

The rotational stiffness S_j of minor-axis joints may be calculated as follows, see [2] and [4]:

$$S_j = \frac{M}{\phi} = \frac{E \cdot z^2}{\mu \sum \frac{1}{k_i}} \tag{10}$$

where k_i are stiffness factors

z is the lever arm in the joint

μ is the stiffness ratio $\mu = S_{j.ini} / S_j$. An evaluation of μ for minor-axis joints can be found in refs. [9, 10]

This description of the stiffness assembly procedure for minor-axis joints is similar to that of major-axis joints [4]. This paper will further concentrate on the stiffness coefficients k_i.

Relevant stiffness factors k_i for the minor-axis joints are dependent on the type of joint for instance:

- column web in bending and punching, compression zone, see paragraph 2.3.2.
- column web in bending and punching, tension zone, see paragraph 2.3.2.
- bolts in tension
- end plate in bending (end plated joints only)
- angle cleat flange in bending (cleated joints only)

The stiffness coefficients of the latter three components are given in Annex J, see [2] and [4]. In this paper no further attention will be given to these components, although extension of Annex J to cover the stiffness coefficients for the case without prying may be necessary

2.3.2 Stiffness factors for column web in bending and punching

In the tension zone or in the compression zone of the column web in bending and punching, the stiffness factors, given as $F/(E\delta)$, where δ is the displacement perpendicular to the plane of the column web, may be expressed as:

$$k_i = \frac{t_{wc}^3}{L^2} \cdot 16 \frac{\alpha + (1-\beta)\tan\theta}{(1-\beta)^3 + \frac{10.4(c_1 - c_2\beta)}{u^2}} \cdot k_{rot} \tag{11}$$

In this expression c_1 = 1.5 and c_2 = 1.63 are coefficients that have been obtained from calibration of the physical model by numerical simulations [10], and:

$$\begin{array}{ll} u = L/t_{wc} & 10 \leq u \leq 50, \\ \beta = b/L & 0.08 \leq \beta \leq 0.75 \\ \alpha = c/L & 0.05 \leq \alpha \leq 0.2 \\ \theta = 35° - 10°\beta & \end{array}$$

In Equation (11) the factor k_{rot} is equal to 1 if the rotation of the column flanges is restrained, as in the case of Figure 9(a). In this figure the rotation of the column flanges is restrained by the major-axis beams. In the case of a minor-axis joint alone, see Figure 9(b), there is a drop in the stiffness factor, that may be taken into account by, see ref. [11],

$$k_{rot} = \begin{cases} 0.52 - 0.40\beta & \text{for HE sections greater than HEA 400,} \\ & \text{HEB 500, HEM 600, and for IPE sections} \\ 1 & \text{for HE sections smaller or equal to HEA 400,} \\ & \text{HEB 500, HEM 600.} \end{cases} \tag{12}$$

Figure 9: (a) Restrained and (b) unrestrained column flanges

Equation (11) for the stiffness has been derived for the case of a local mechanism, but it has been shown that it may be used, on the safe side, in the case a global mechanism. A comparison with laboratory tests shows that the model may be used to predict the stiffness of a minor-axis joint within the range of parameters considered.

2.4 Rotation capacity of minor-axis joints

A simplified evaluation of the post-plastic behaviour of such joints, which is necessary to check its rotation capacity can be found in refs. [9, 10].

3. JOINTS IN HIGHER STRENGTH STEELS

3.1 Introduction

In Eurocode 3 Revised Annex J, rules are given to evaluate the stiffness and resistance characteristics of the components in a joint. An assembly procedure is also described. The rules for the evaluation of the component properties have been validated through comparisons with experimental results. As the available experiments are not covering all the actual constructive situations (range of relative dimensions, steel grade, ...), some limitations have therefore been given so as to avoid the use of the EC3 rules outside these ranges of application.

Table 1 indicates, for each component listed in Annex J, its field of application.

Table 1: Components in Annex J and their field of application

Components	Limitation to S355 steel grade	Other limitation
Column web panel in shear	√	$d_c / t_{w.c} \leq 69\varepsilon$
Column web in compression	√	-
Beam flange and web in compression	√	-
Column flange in bending	√	-
Column web in tension	√	-
End plate in bending	√	-
Beam web in tension	√	-
Flange cleat in bending	√	-
Bolts in tension	-	-
Bolts in shear	-	-
Bolts in bearing	√	-
Plate in tension and compression	√	-

From this table, it may clearly be seen that the major limitation is that relative to the steel grade. In this paragraph, the possible extending of the EC3 Annex J rules to higher steel grades up to S460 is discussed.

The limitation of EC3 Annex J to S355 steel grades is a general statement for each constitutive component.

For some of them, however, its application is quite questionable:

<u>Column web panel in shear</u> : The limitation of the panel slenderness to $d_c / t_{w.c} \leq 69\varepsilon$

is there to ensure that no shear buckling occurs. As $\varepsilon = \sqrt{235 / f_{ywc}}$ (with f_{ywc} = yield stress of the column web in N/mm²), the steel grade is already taken into consideration there and no other limitation is required; in other words, the formula given in Annex J for the plastic shear resistance of the panel is valid whatever is the steel grade, as long as $d_c / t_{w,c} \leq 69\varepsilon$. The limitation to S355 can therefore be removed.

Beam flange and web in compression :

The strength evaluation is based on the evaluation of the design resistance M_{cRd} of the beam section. $M_{c.Rd}$ is dependent on the class of the beam profile and, as for the column web in shear, this one is influenced by ε. A priori, no limitation of the steel grades seems to be needed for this component, but the validation of the related design formula for strength would anyway be welcome.

Bolts in bearing
Plate in tension and compression
is Beam web in tension
Column web in tension

No instability is likely to occur in these components, even for the plate in compression, where the risk of instability prevented through the use of appropriate bolt pitches. Their strength is linked to plasticity and a limitation to steel grade lower then S355 does not appear as quite justified.

End plate in bending
Column flange in bending
Flange cleat in bending

These components are idealized as T-stubs subjected to tension forces. At design collapse, the bolts fail in tension, a plastic mechanism develops in the T-stub flange or a mixed failure involving bolt fracture and plasticity in the T-stub flange occurs.
Once again, the relative values of the yield stresses (bolts/plate) are taken into consideration in the strength calculations and no limitation of the yield stress for the connected plate (end plate, column flange or cleat) has to be considered for these components.

Column web in compression

The only component for which an extending of the design rules to HSS seems quite questionable is the column web in compression where the buckling and crippling resistances (not the crushing one) are highly dependent on the web slenderness and the longitudinal stresses in the column web resulting from bending moments and normal forces.

The column web in compression is discussed here below.

3.2 Strength of a column web in compression and extension to higher strength steels

In a major-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 crushing 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 10 where the ratio η between the left and right loads, P_l and P_r, varies from 0 to 1. Figure 10 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 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 considered for several years, the two first ones were not taken into account in the old version of Eurocode 3 Annex J. They have been pointed out by Jaspart [14]. The unsafe character of the previous Annex J, compared to the Jaspart model, is represented by the hachured zone of
Figure 10.

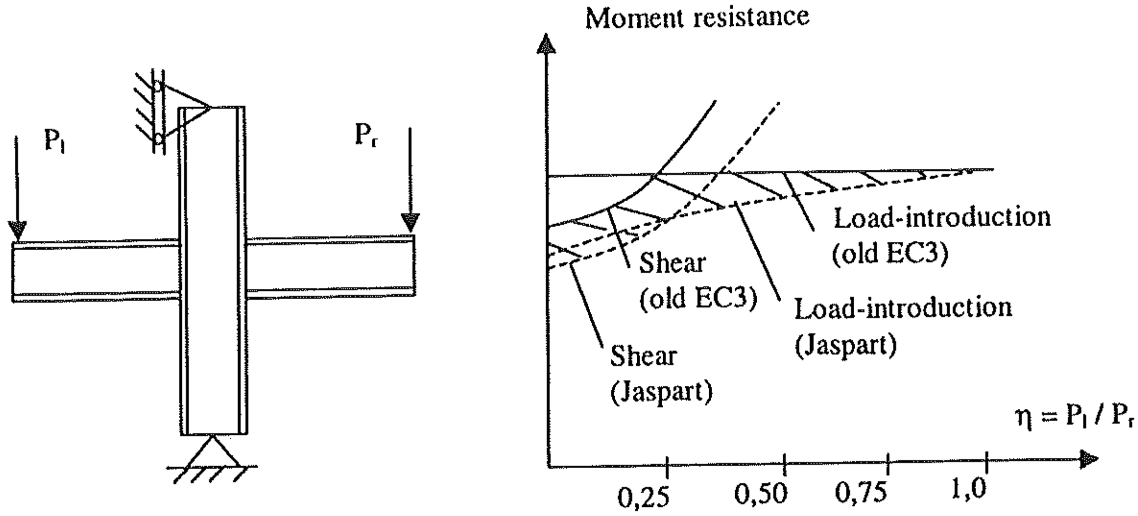

Figure 10: Variation of web panel resistance according to the external loading

In the revised annex J, the rules concerning the web panel design resistance have been modified on the basis of the proposals made in [12].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 :

$$V_{wc,Rd} = 0{,}9\frac{A_{v,c}\cdot f_{ywc}}{\gamma_{mo}\cdot\sqrt{3}} \tag{13}$$

$A_{v,c}$ is the shear area of the profile, f_{ywc} the yield stress and γ_{mo} 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, see equation (14), due to the possible presence of shear stresses, by means of a reduction factor ρ :

$$F_{wc,Rd} = \rho\frac{f_{ywc}\cdot t_{wc}\cdot b_{eff}}{\gamma_{mo}} \tag{14}$$

with :

$$\rho = \begin{cases} \rho_1 = \dfrac{1}{\sqrt{1+1.3(\frac{b_{eff}\cdot t_{wc}}{A_{v,c}})^2}} & if\ \eta = 0 \\ \rho_1 + (1-\rho_1).2.\eta & if\ 0<\eta<0.5 \\ 1 & if\ 0.5\le\eta\le 1 \end{cases} \tag{15}$$

t_{wc} is the web thickness and b_{eff} is the effective yielding length. This last parameter depends on the connection details. Equation (15), represented by two lines, is a simplification of the initial Jaspart's proposal.

The effect of the longitudinal stresses σ_n on the resistance of the column web in compression is taken into account by means of another reduction factor, k_{wc} , that was already existing in the old Annex J [13]:

$$k_{wc} = 1.25 - 0.5\frac{\sigma_{n.Ed}}{f_{ywc}} \leq 1 \tag{16}$$

$\sigma_{n,Ed}$ is the normal stress in the column web, at the root of the fillet or of the weld, due to axial 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 is the extension of the design rules to slender webs ($\bar{\lambda} > 0.673$) by limiting the design resistance given in equation (14) to the buckling resistance value of the web :

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

$$= \rho\frac{f_{ywc}.t_{wc}.b_{eff}}{\gamma_{mo}}.\left[\frac{1}{\bar{\lambda}}.(1-\frac{0.22}{\bar{\lambda}})\right] \qquad \text{if } \bar{\lambda} > 0.673 \tag{17}$$

with :

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

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

Equation (17), in itself, takes the risk of premature buckling of the web resulting from the use of HSS into consideration through the $\bar{\lambda}$ slenderness coefficient, but the validity of the formula has not been demonstrated.

In [14], the possible extending of equation (17) to steels up to S460 is discussed on the basis of comparisons with numerical simulations and experimental tests and the validity of the formula is demonstrated.

3.3 Stiffness of a column web in higher strength steels

The initial stiffness of the component column web in compression in independent on the steel grade, so the Annex J design rules can be directly applied to higher strength grades [2].

3.4 Deformation capacity of a column web compression in higher strength steels

Due to buckling phenomena, the deformation capacity of a column web in compression in higher strength steels may be limited, which may result in limited rotation capacity of a joint. This complex problem is still under investigation.

4. JOINTS BETWEEN SLENDER SECTIONS

4.1 Introduction

The behaviour of the component "unstiffened column web panel in shear" normally has an eminent influence on the strength and stiffness behaviour of single sided beam-to-column joint configurations or double sided joint configurations with unbalanced loading.

Annex J [2] provides design guidance for the determination of strength, stiffness and rotation capacity of the column web panel in shear. The use of these Annex J rules is restricted to column web panels with a slenderness smaller than $d_c / t_{wc} \leq 69 \sqrt{235/f_y}$, where d_c is the free depth of the web panel, t_{wc} is the column web thickness and f_y is equal to the yield strength of the column web material in N/mm^2, see Table 1. It is assumed that if this requirement is fulfilled, no shear buckling in the column web will occur.

This paper focuses on beam-to-column joints where slender web panels are stiffened in the tension and compression, but unstiffened in shear.

4.2 Strength of joints between slender sections

4.2.1 Assembly procedure for strength

The moment resistance of slender beam-to-column joint can be calculated based on the methods as proposed in Annex J. In the case of joints between slender sections, the same procedure may be adopted, as long as the normal force in the beam is low, see [2]. For more details on this point, it is referred to [4].

4.2.2 Strength of slender column web in shear

When shear buckling in a column web panel doesn't occur, the distribution of shear forces over the column web is rather uniform. Based on this, the resistance formula, as given in Annex J writes, see [2]:

$$V_{Rd} = \frac{0{,}9\, A_v f_y}{\sqrt{3}\, \gamma_{M0}} \tag{18}$$

where: A_v is the shear area of the column web cross-section

f_y is the yield stress of the column web;

Additional to this shear resistance, in case of column web stiffeners in tension and compression welded over the full depth of the column web, an additional resistance V_m^* due to the frame effect of the stiffeners and the column flanges may be taken into account, see [2].

On the other hand, when shear buckling occurs, the associated resistance is called V_{cr}, the shear critical resistance. Beyond this value, the more or less uniform distribution of shear stresses over the column web is replaced by a diagonal tension field. In a last step, the failure under ultimate shear forces is preceded by the mechanism of the surrounding column web flanges and stiffeners.

The resistance of a slender column web panel in shear can now be calculated as follows [15, 16, 17], see Figure 11:

$$V_{Rd} = V_{cr} + V_{tf} + V_m \tag{19}$$

where: V_{cr} is critical shear resistance of the column web panel;

V_{tf} is the shear resistance associated with the diagonal tension field;

V_m is the shear resistance associated to the frame effect due to the yield line mechanism in the surrounding plates.

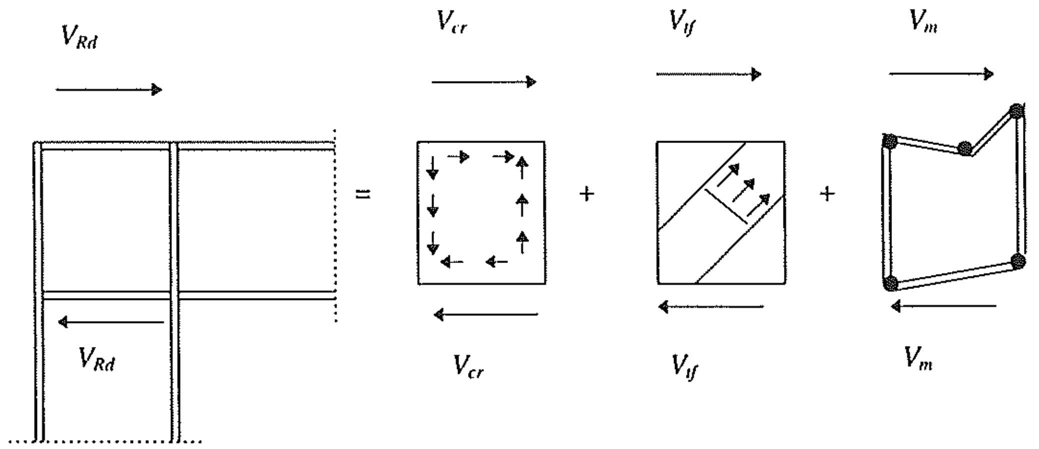

Figure 11: Shear resistance of a slender column web panel.

The sum of V_{cr}, V_{tf} and V_m may of course not exceed the sum of V_{Rd} as calculated with formula (18) and V_m^*.

In [16, 17] it is assumed that the column web is simple supported on the surrounding plates. This is a safe assumption. Numerical investigations indicate that the surrounding plates of the column web panel provides constraints to the web panel which may be taken into account in the design models [15].

In [16, 17] also guidance is provided on the calculation of the effective width of tension field in the column web panel. This model does not include parameters dependent on the type of joint, welded or bolted. Refinement may be possible [15].

4.3 Rotational stiffness of joints between slender sections

The assembly procedure for the stiffness properties of normal beam-to-column joints is given in [2], see also formula (10). What concerns the assembly procedure for joints between slender sections the following remarks should be made:

- the initial stiffness of the slender column web in shear can be determined based in the design rules of Annex J [2].
- the deformation of the column web in shear may have an influence on the stiffness reached at the level of the design moment resistance of the joint.

For more details on this last point, it is referred to [4].

So far, the derivation of the shear stiffness properties of slender web panels in the non-elastic range is still under investigation [15].

4.4 Rotation capacity of joints between slender sections

Because of buckling behaviour of slender column web panels in shear, the rotation capacity of the a joint between slender sections may be limited. This is still under investigation.

5. CONCLUSIONS

In the frame of the COST C1 project, studies have been carried out on new components of steel joints, for instance components for minor-axis joints, for joints in high strength steels and for joints between slender sections. In this paper, a review of this research is given. The result of these studies is that now fundamental knowledge is available about the mechanical behaviour of these new components. The knowledge has been discussed in an international platform of scientists, the COST C1 Working Group 2.

The fundamental knowledge obtained in the project has a high level of refinement. This is necessary because when the behaviour of the components is well understood in all aspects, the mechanical models may be simplified to design models for use by practitioners. Therefore, a next step should be to transfer the scientific knowledge obtained during the COST C1 project to simple and easy-to-apply design guidance for practitioners. For instance, this design guidance could be presented in the form of a design manual, containing design sheets, design tables and software where all the results of the COST C1 project are utilized.

Whenever design guidance about new components is available, steel construction industry can apply this to new forms of economic connection. Forms of connection which have been studied and understood in the frame of the present COST C1 project!

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