STUDY OF THE SHEAR DEFORMABILITY OF COLUMN WEB PANELS IN STRONG AXIS BEAM-TO-COLUMN JOINTS

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INTRODUCTION

For some years, experimental and/or theoretical research works have been devoted to the actual behaviour of beam-to-column joints in steel buildings and more especially to semi-rigid joints. This is the result of a search for simple and cheap connections with a view to a reduction of the labour cost, which grew much faster than the material cost. Thus for the sake of economy, bolted joints without any stiffener became a common practice. Unfortunately such joints have a non linear behaviour: when subject to an applied bending moment M, the axes of the connected members do not rotate a same angle, so that there is a relative rotation θ, which is not proportional to the applied bending moment M.

Both strength and stability of steel frames are affected by the semi-rigid behaviour of the joints [1]. Though several computer programmes [2], which allow for material and geometrical non linearities - including semi-rigid connections - are available, there is an urgent need of knowledge for the M-θ characteristics of the joints.

Present paper is aimed at presenting a mathematical approach that is likely to provide the M-θ curve associated to the shear rotation of the unstiffened column web panels. This model is demonstrated to give results in close agreement with numerical simulations and with experimental tests.

JOINT DEFORMABILITY COMPONENTS

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

a) <u>the deformation of the connection</u> associated to the deformation of the connection elements (end plate, angles, bolts,...), to the slip, to the column web in the tension and compression zones (respectively a lengthening and a shortening) ;

b) <u>the deformation of the column web under shear</u> 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 case of the end plate connection of figure 1 may be chosen to illustrate this. The rotation ϕ of the connection is mathematically defined by the difference of the two rotations θ_b and θ_c and includes the deformation of the end plate, of the bolts and of the column flange, the lengthening of the zone BC and the shortening of the zone AD of the web.

The rotation γ of the column web under shear is defined by the difference of the rotations θ_c and θ_f where θ_f represents the flexural rotation of the column.

It is important to know that the shear in the column web is the result of the combined action of the equal but opposite forces F_b in the beam flanges, which are statically equivalent to the beam moment, and of the shear forces in the column at the level of the beam flanges.

The difference between the loading of the connection and that of the column web in a same joint leads to take account separately of both deformability sources.

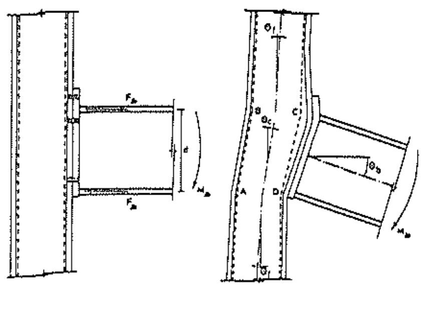


Figure 1 - Deformation of a strong axis joint with an end plate connection

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 [3].

This study is based on numerical simulations with the non linear FE-program FINELG [4] of the loading up to failure of welded beam-to-column joints. Material and geometrical non-linear effects are taken into account, although the latter is far less important than the former. 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).

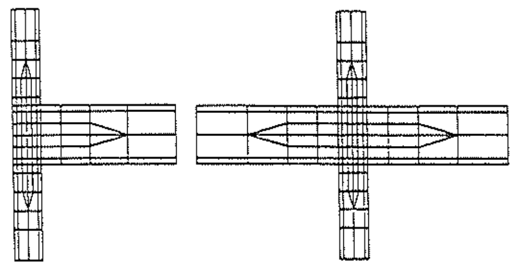


Figure 2 - Joint finite element meshes

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

Steel is supposed to follow a piecewise linear law shown on Fig. 3. The 2D elastoplastic state of stress is dealt with by using the incremental flow theory and the von MISES yield criterion. Parabolic patterns of rolling residual normal stresses in flanges and webs are taken according to the ECCS recommendations [5]. Welding imperfections are not considered. Complete data may be found in [6].

The good agreement between the numerical simulations and results of experimental tests on joints is shown in [6].

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 load-introduction deformability is the component of the connection deformability associated to the local deformation of the column web in the tension and compression zones (respectively a lengthening and a shortening).

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 behaviour of the sheared column web panels are presented here.

a) The shear stresses in the column web panels may be considered as uniformly distributed; this is due to the action of the column flanges.

b) The actual value of the shear force V_n may be obtained from the equilibrium equations of the web panel [7].

It is given by the following formula (figure 4) :

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

Some other researches refer to another formula :

$$V_n = \frac{M_{c1} + M_{c2}}{d_b} \qquad (2)$$

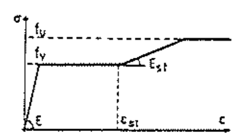


Figure 3 - Stress-strain curve (mild steel)

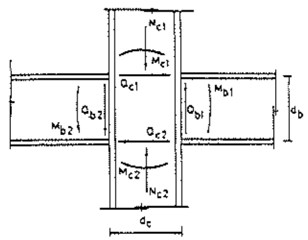
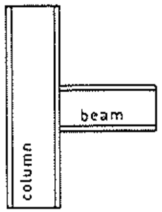


Figure 4 - Loading of an interior joint

The validity of the proposed formula (1) has been clearly demonstrated.

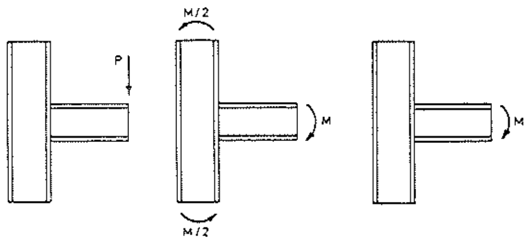
c) The V_n - γ curve for a given joint depends on the actual loading of the joint.

Let us assume that the two unstiffened welded nodes of figure 5 are subject to different types of loading (figure 6) and let us report, for each node, the characteristic V_n - γ curve in a common diagram (figures 7 and 8).



	Beam	Column
A	IPE 300	HE 160 B
B	HE 500 B	HE 300 B

Figure 5 - Definition of two welded joints ("T" arrangement)



Simple bending (FS) Pure bending (FP) Pure bending in the beam (MP)

Figure 6 - Different types of loading

The shear force V_n takes account, by means of formula (1), of the loading of the joints ; in consequence one could believe that the V_n - γ curves are identical for a given node. Actually only a similarity exists in the elastic range of the web panel behaviour and this demonstrates the validity of the proposed shear force definition (formula 1).

The difference between the V_n - γ curves in the non-elastic range of the web panel behaviour are not negligible.

The existing methods for the prediction of the shear deformability of web panels do no take the influence of the actual joint loading into account. Figures 7 and 8 show that this is questionable and has led to the elaboration of a new approach.

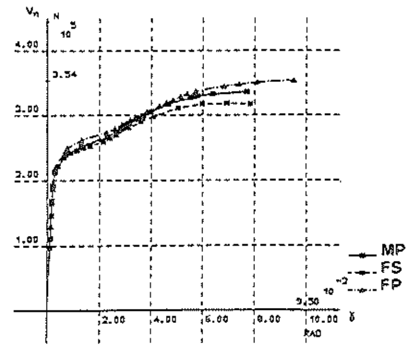


Figure 7 - Characteristic V_n - γ curves (joint A)

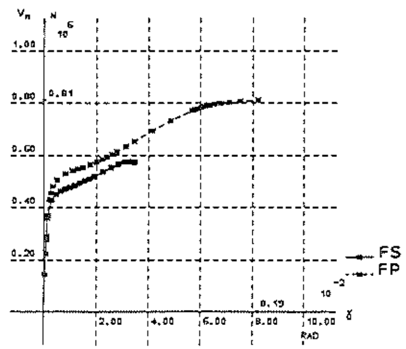


Figure 8 - Characteristic V_n - γ curves (joint B)

THEORETICAL DEVELOPMENTS

The theoretical developments presented in this paper are related to the study of the unstiffened column web panels. It will be referred to [3] for stiffened columns.

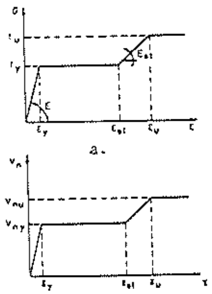
Let us consider a small column web element subject to shear stresses τ (figure 9.a) and whose material characteristic is elastic-perfectly plastic with strain-hardening (fig. 10.a). The shear deformability γ of this element (figure 9.b) versus the shear stress τ may be deduced.

a. b.

Figure 9 - Shear in a small web element

The shear stresses being uniformly distributed in the web panel, a first approximation (pure shear) of the searched V_n - γ curve may be easily obtained by multiplying the shear stress τ by the column web area (figure 10.b).

The theory of plasticity may be used in connection with the von MISES yield criterion to modify the characteristic values V_{ny} (γ_y), V_{nu}, γ_{st} and γ_u (figure 10.b) with a view to account for the actual node loading.



a.

b.

Figure 10 - Characteristic σ - ϵ and V_n - γ curves (first approximation)

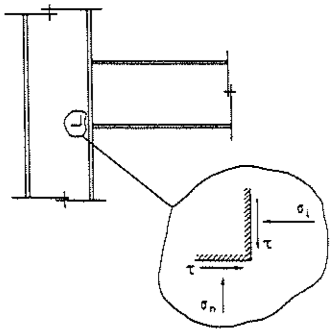


Figure 11 - Different types of stresses in a web panel

In its most stressed zone (figure 11) an unstiffened web panel is subject to three types of stresses :

- 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_i resulting from the introduction of beam loads in the column web.

The load introduction constitutes only a <u>local phenomena</u> which has no direct influence on the <u>global behaviour</u> of the web panel. The web panel deformability predicting model based on the modified value of V_{ny} (γ_y), V_{nu}, γ_{st} and γ_u (interaction between τ and σ_n stresses) is applied for instance in figure 12 to the node B submitted to pure bending (figures 5 and 6).

It may be seen that :

- the agreement between the values of the initial stiffness and of the strain-hardening stiffness given by the mathematical model and the numerical simulation is very good ;

- the mathematically predicted plastic load is slightly lower than the corresponding load obtained by means of the numerical simulation ; the small difference which represents the bending resistance of the column flange may be neglected in the case of joints with unstiffened columns.

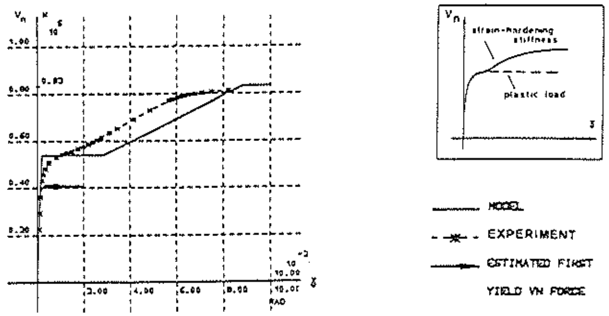


Figure 12 - Comparison between the numerical simulation of node B subject to pure bending and the model modified to account for the node loading. (Second approximation)

However, the mathematical model differs from the result of the numerical simulation for what regards the length of the yield plateau and the collapse load.

It must be referred to the interaction between the web panel shear and the load-introduction effect to explain these differences :

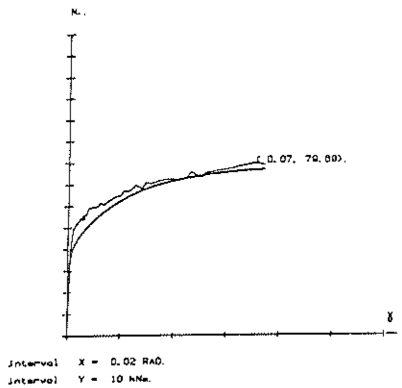
- the strain-hardening first appears locally in the most stressed zone of the column web (figure 11) where the three above-mentioned types of stresses interact and will depend on the importance of the σ_i stresses relatively to the two other types of stresses; the length of the yield plateau has been consequently empirically reduced ;

- the collapse load of the web panel is linked up either to the excessive shear or to the load-introduction resistance of the web (web crippling for instance), what depends on the relative importance of σ_i stresses too; formulae for the assessment of the resistance and the stability of column webs subject to transversal loads have then been developed.

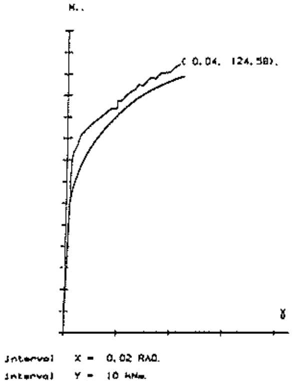
It is also easy to show the necessity of taking into account the interaction between shear and load-introduction by introducing the σ_i, σ_n and τ stresses into the von MISES yield criterion in order to determine the shear load corresponding to the beginning of yielding in the column web.

The agreement between the calculated elastic shear force and the result of the numerical simulation for the joint B (FP) is seen to be very good (figure 12).

The shape of the proposed multi-linear model is shown in figure 13. The mathematical formulation of its characteristic values is not given here because of lack of space but is presented in reference [3] which can be



a - Test 01



b - Test 010

Figure 16 - Comparison between experimental results and model (joints with end-plate connections).

SIMPLIFIED EVALUATION OF THE SHEAR PLASTIC CAPACITY

The analysis and the design of building frames by means of sophisticated non-linear programs requires an accurate prediction, similar to that described here above for the sheared column web panels, of all the deformability components of the beam-to-column joints, whereas only some simplified characteristics of the actual joint behaviour such as the secant or the initial stiffness and the plastic capacity are needed in the daily design practice.

The following formula for the assessment of the plastic capacity of a sheared column web panel is proposed in this respect in the annex J of Eurocode 3 [9] :

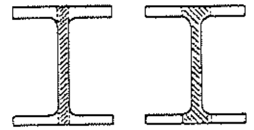
$$V_{ny} = H_c \cdot t_{cw} \cdot f_{ycw}/\sqrt{3} \qquad (3)$$

where :
- f_{ycw} is the yield stress of the column web ;
- H_c is the total height of the column ;
- t_{cw} is the column web thickness.

This formulation differs from that considered in the previous section (and which is given in [3]) by :

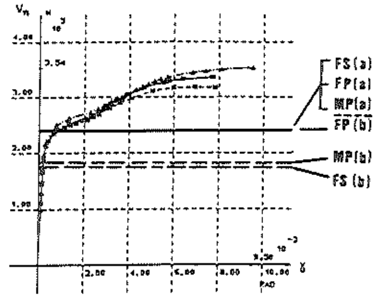
- a safer evaluation of the actual sheared web area (figure 17) ;
- an unsafer definition of the maximum shear stress (the actual stress interaction in the web is not accounted for).

Its application to the available numerical and experimental test results allows to validate it - the formula is safe but 10 or 20 % of safety may be sometimes obtained - as far as the shear force V_n is evaluated by means of the formula (1). The use of formula (2), which is recommended by different authors, leads usually (it depends on the joint loading) to too safe assessments of the shear plastic capacity of the column web (see figure 18 for instance for joint A).



a - EC3 b - Actual

Figure 17 - Definition of the column sheared area.



$$V_n = \frac{M_{c1} + M_{c2}}{d_h} \cdot \frac{1}{2}(Q_{b1} + Q_{b2}) \frac{d_c}{d_b} \qquad (a)$$

$$V_n = \frac{M_{c1} + M_{c2}}{d_b} \qquad (b)$$

Figure 18 - EC3 rule for plastic capacity.

CONCLUSIONS

Numerous numerical simulations of "T" and "cross" beam-to-column joints have been performed. The parameters investigated in this study were the type of beam(s) and of column, the loading of the joints, the transversal initial deformation of the column web and the presence or not of stiffeners on the column web.

This analysis has allowed :

1) to define accurately the shear force of the joint under any sollicitation ;
2) to show clearly the influence of the above mentioned parameters ;
3) to enlarge the knowledge on the physical behaviour of the joint up to failure.

Theoretical developments have led to a new approach for the mathematical prediction of the non linear behaviour of unstiffened column web panels. The model proposed has been validated for the column webs of joints with end-plate connections.

Last, the EC3 formula for the assessment of the shear plastic capacity of the panels is discussed.

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PLASTIC BUCKLING CAPACITY OF SQUARE SHEAR PLATES WITH CIRCULAR PERFORATIONS

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Abstract

The paper presents the results of a finite element buckling analysis of perforated shear plates with inelastic material behaviour. The problem typically applies to the design of working platforms and support caissons of offshore steel structures that are often designed with plate boxes or plate girders. The important shear walls or shear webs must often be perforated to allow utilities, etc., to pass through. The failure mode of these large perforated shear panels is typically plastic shear buckling. In the calculations nonlinearities in material properties and geometry were taken into account. Single unreinforced round holes were considered with variations of hole size and hole location.

Following the analytical study a series of experimental tests was conducted to compare the effect of hole location on the ultimate shear capacity of web plates. Comparative results are presented, accompanied by a benchmark test and finite element study on an unperforated plate.

Overview

A comprehensive introduction to the problem has been given by Martin (1985) in his M.A.Sc. thesis. A brief summary is presented in the following.

Shear plates or shear webs are flat rectangular plates, which are very common structural components in modern buildings and structures. As webs of plate girders, these flat plates are often subjected to a uniform shear stress. Often one or more holes are being cut out of the shear plates such as in girder webs, to allow utility pipes to pass through or to reduce the structural weight (Fig.1).

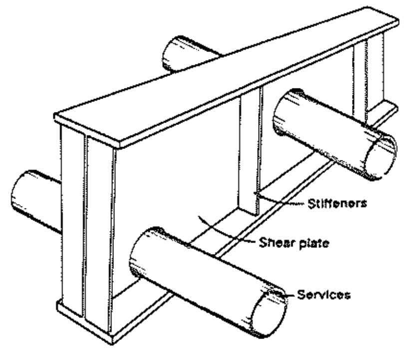

Fig. 1. Perforated shear plates