Université
de Liège

Mécanique des matériaux
Stabilité des constructions
Mécanique des structures

MSM - Institut du Génie Civil
Quai Banning, 6
B-4000 LIEGE - BELGIQUE
Tél. (0)41 - 52 01 80 - Ext. 238
Telefax 32-41-53 09 78

MATERIAL IDENTIFIED AS SIGNIFICANT FOR EUROCODE 3 AT THE UNIVERSITY OF LIEGE

J.P. JASPART

INTERNAL REPORT N°206

APRIL 1992

MATERIAL IDENTIFIED AS SIGNIFICANT

FOR EUROCODE 3

AT THE UNIVERSITY OF LIEGE

J.P. JASPART

INTERNAL REPORT N°206

APRIL 1992

TABLE OF CONTENTS

INTRODUCTION

The analysis and design of multi-storey frames with semi-rigid and partial strength joints raise some problems; in particular :

- the characterization of the joint response ;
- the frame analysis procedure appropriate for the determination of the redistribution of internal actions ;
- the evaluation of the resistance and of the stability of isolated elements, of sub-structures and of the whole frame.

It seems therefore well timed to point out not only the different aspects of the problems but also their interaction and their complementarity. The following flow chart has been developed with this aim in view. A number of brief observations are made as follows.

The characterization of a semi-rigid beam-to-column joint is neither easy nor obvious. In fact, the deformability of a steel or composite joint results from several sources but, for sake of simplicity, these may be grouped (Figure 1) in two main components (1) the column web panel subject to shear force V_n and (2) the connection(s) between the beam(s) and the column subject to bending moment M.
Both components may be considered separately, each acting at a specific location which corresponds to reality. Alternatively, they may be added in order to characterize the joint as a whole i.e. a concentration of the joint deformability. Whichever option is adopted any prediction of the M-ϕ relationship must be made only from knowledge of the mechanical and geometrical joint properties.
It is necessary to define the characteristic non-linear curves - two when the deformability sources are differentiated; only one when it is concentrated - or to calculate specific parameters such as initial stiffness, secant stiffness, design moment resistance, etc... according to the analysis which is planned.
The use of non-linear joint moment-rotation curves seems to be restricted to sophisticated numerical analysis methods which are often able to take also account of the two types of frame non-linearities (material and geometrical). The alternative procedure base on the evaluation of specific joint characteristics is required by hand (or pseudo-hand) design methods; it may also be associated with less sophisticated numerical analysis methods which are however sufficiently accurate for practice.

The use of non-linear programs has the advantage that they provide all the quantitative information which is necessary to form an opinion of the performance of the frame studied.
However, reliable results quite satisfactory for daily practice may be based on simplified joint modelling associated with first or second order analysis methods - depending on whether the frame is braced or not against lateral displacements. The designer has to choose between two approaches - elastic or plastic - according to his wishes or to the class of cross-sections used for the connected members.
Such a less important consideration in the selection occurs when the analyses are used to determine the displacements of the frame or to check the elastic resistance of the cross-sections under servive loads (SLS - serviceability limit-states), on one hand, and to evaluate the ultimate frame resistance at collapse (ULS - ultimate limit-states), on the other hand. The judgement of the Engineer to assess the ability of the

frame to satisfy the limit-states is the direct corollary of these calculations

This flow chart, which covers all the aspects of a semi-rigid design, has been used* as a basis for the common presentation of the work performed in Trento, Sheffield and Liège - ARBED-Luxemburg in the frame of the ECSC Contracts 7210-SA/413 - 819 and 507 and of other previous related projects carried out in these research centres.

For the sake of clarity, four charts have been prepared according to the type of frame (steel or composite) and to the type of loading (monotonic or cyclic), i.e.

i - Steel structures under monotonic loading ;
ii - Steel structures under cyclic loading ;
iii - Composite structures under monotonic loading ;
iv - Composite structures under cyclic loading.

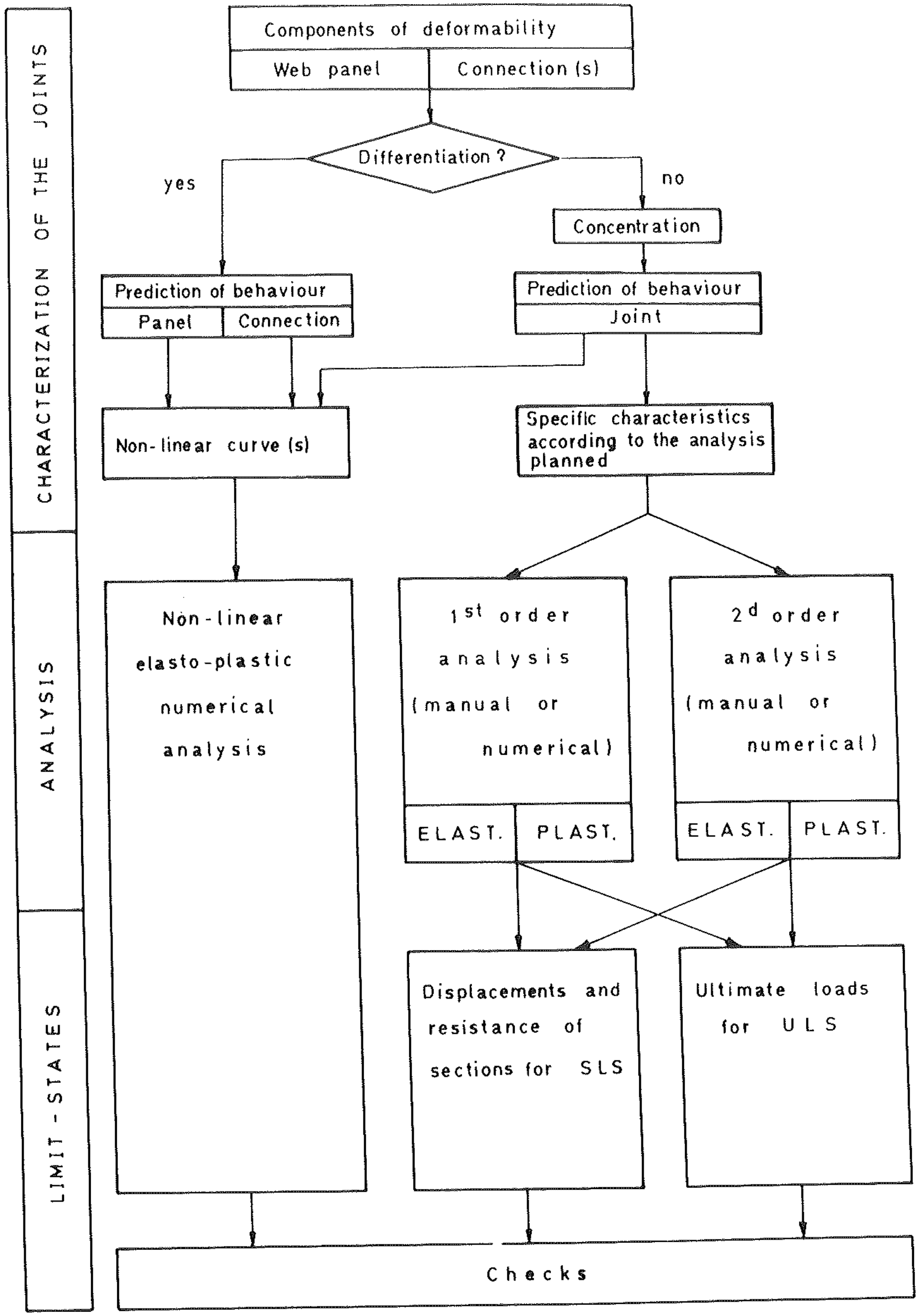
The aim of this work is threefold :

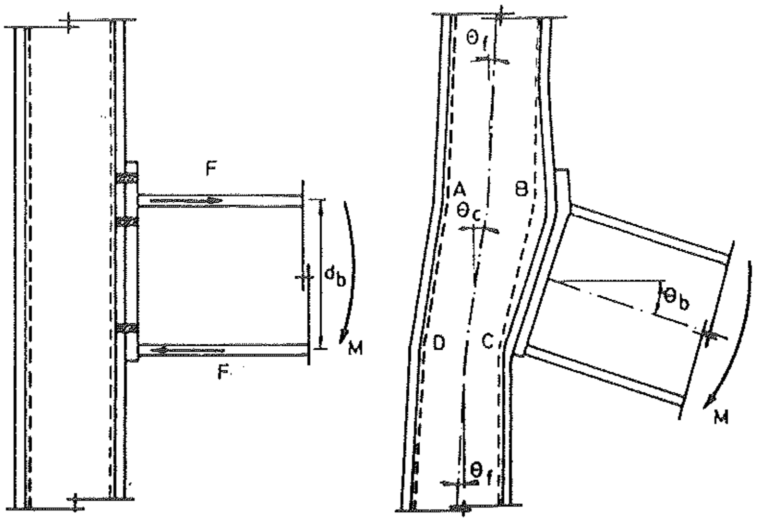
1)- By means of the flow charts to present a synthesis of the research work carried out in Trento, Sheffield and Liège-Esch and to show the complementarity of these researches;
2)- To point out the material identified as significant for Eurocode 3 and Eurocode 4.
3)- Together with the group's knowledge of other world wide investigations, highlight topics for further researches.

Each of these different points will be dealt with in the three main chapters of the final report of the ECSC Contract 7210-SA/829.

In order to make the material identified as significant for EC3 available in a short time to interesting people, the Liège contributions to this specific topic have been gathered in the present report.

* This work is carried out by italian (Trento-Siderservizi), british (Sheffield-BCSA), belgian (Liège) and Luxemburgian (Arbed) partners in the frame of the Contract 7210-SA/829 introduced by BCSA to ECSC.


CHARACTERIZATION OF THE JOINTS
ANALYSIS
LIMIT - STATES
Components of deformability
Web panel
Connection (s)
Differentiation ?
yes
no
Concentration
Prediction of behaviour
Panel
Connection
Prediction of behaviour
Joint
Non-linear curve (s)
Specific characteristics according to the analysis planned
Non-linear elasto-plastic numerical analysis
1st order analysis (manual or numerical)
2d order analysis (manual or numerical)
ELAST.
PLAST.
ELAST.
PLAST.
Displacements and resistance of sections for SLS
Ultimate loads for ULS
Checks



. connection deformability

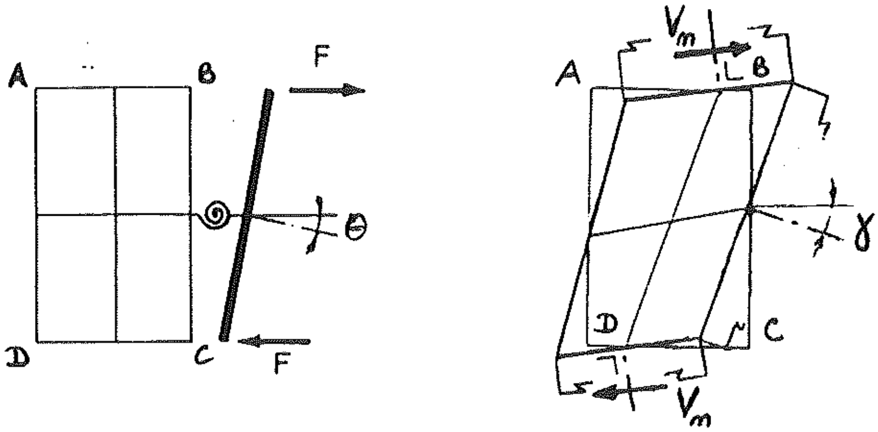
$$\theta = \theta_b - \theta_c$$

. shear deformability of column web panel

$$\gamma = \theta_c - \theta_f$$

. joint deformability

$$\phi = \theta_b - \theta_f$$



connection (M - θ)　　　　sheared panel (V_n - γ)

Figure 1 - Main components of joint deformability

CONCENTRATION OF THE JOINT DEFORMABILITY

In a strong axis beam-to-column joint, two main sources of deformability are identified (fig. 1) :

a) The deformation of the connection associated to the deformation of the connection elements (end plate, angles, bolts,...), to that of the column flange and to the load-introduction deformability of the column web ;
b) The shear deformation of the column web associated mostly to the common presence of forces F_b carried over by the beam(s) and acting on the column web at the level of the joint; these forces are statically equivalent to the beam moment M_b.

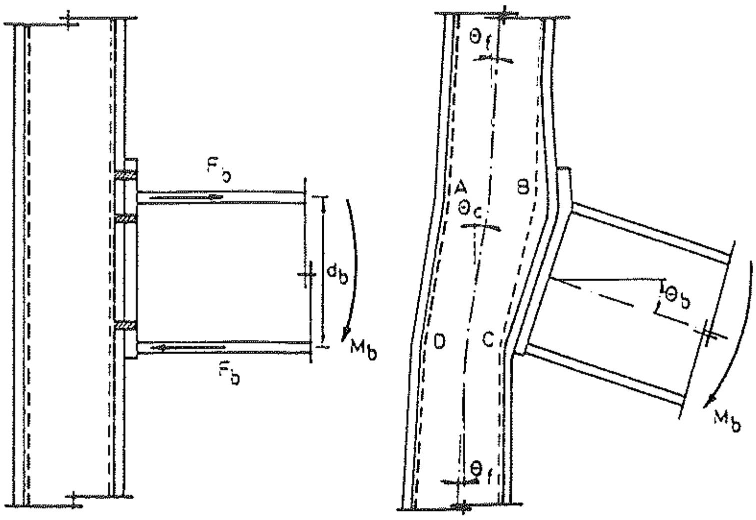


Figure 1 - Deformation of a strong axis joint

These components are illustrated in figure 2 for the particular case of a joint between a single beam and a column. The deformability of the connection elements is concentrated into a single flexural spring located at the end of the beam (fig. 2.a). The associated behaviour is expressed in the format of an M_b - ϕ curve.

The deformation of the ABCD column web panel is divided into :

- The load-introduction deformability which consists in the local deformation of the column web in both tension and compression zones of the joint (respectively a lengthening and a shortening) and which results in a relative rotation ϕ between the beam and column axes; this rotation concentrates mainly along edge BC (fig. 2.b) and provides also a deformability curve M_b - ϕ.

- The shear effect - due to shear force V_n - which results in a relative rotation γ between the beam and column axes (fig. 2.c); this rotation makes it possible to establish a second deformability curve V_n - γ.

It is important to stress that the deformability of the connection (connection elements + load-introduction) is only due to the forces carried over by flanges of the beam(s) (beam moment(s) M_b), while the shear in a column web panel is the result of the combined action of these equal but opposite forces and of the shear forces in the column at the level of the beam flanges (shear force V_n).

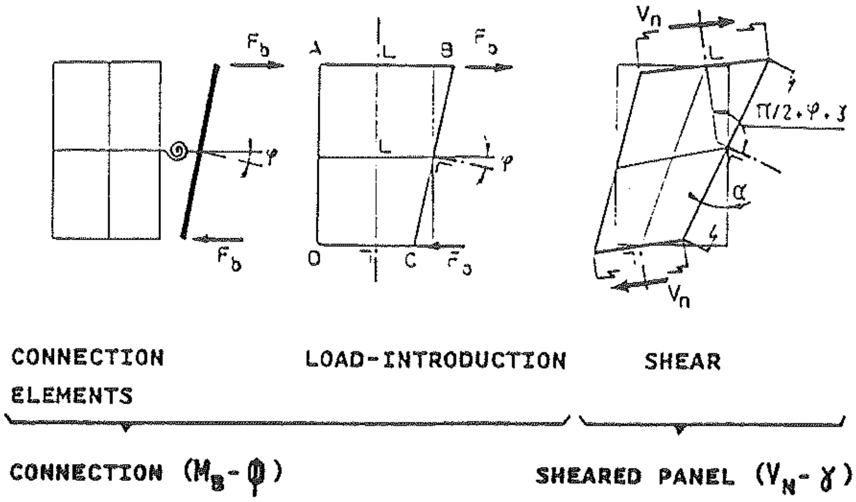
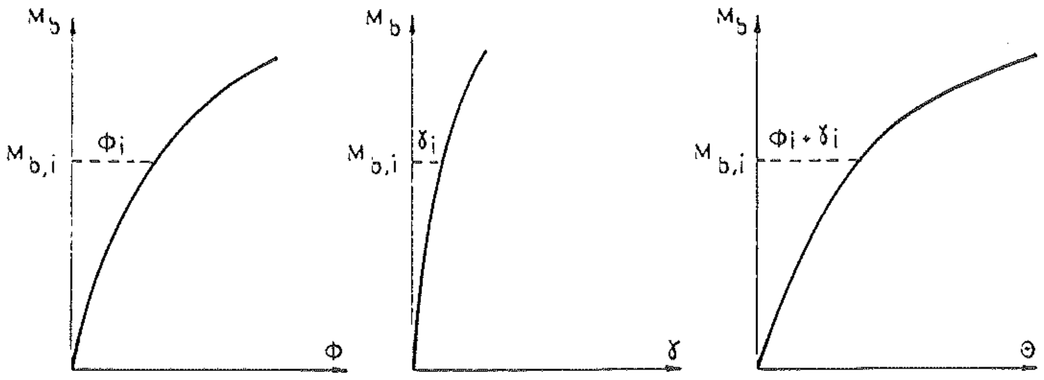


Figure 2 - Joint deformability components

The difference between the loading of the connection and that of the column web in a specified joint requires, at a theoretical point of view, that account be taken separately of both deformability sources when designing a building frame. However doing so is only practicable when the frame is analysed by means of a sophisticated computer program allowing for the separate modelling of both deformability sources. In all other cases, the actual behaviour of the joints must be simplified by concentrating the whole deformability into a single flexural spring acting at the beam end.

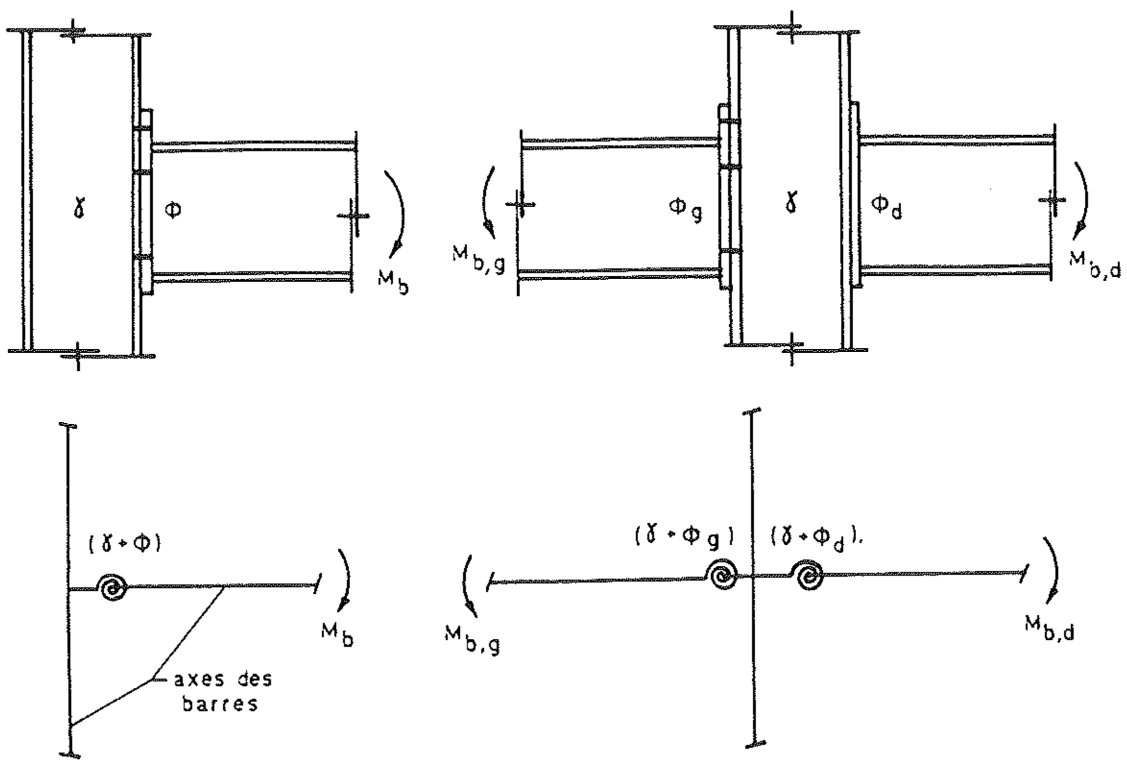
This way of doing is recommended in Annex J of EC3 without any explanation or justification.
A large study has been consequently performed at the University of Liège and at ARBED-Recherches in order to get an answer to the following question: does the concentration of the joint deformability into single flexural springs lead, or not, to a safe and accurate frame design ?
The conclusions of this study are detailed in [1, 2] and summarized here below.

The concentration of the joint deformability is schematized in figures 3 and 4.



a - Connection b - Sheared panel c - Spring

Figure 3 - Flexural characteristics of the spring



a - Exterior joint b - Interior joint

Figure 4 - Concentration of the joint deformability into flexural springs

Regarding the location of the springs, two possibilities exist (fig. 5):
- either at the beam-to-column physical interface (point A), or
- at the intersection of both beam and column axes (point B).

The optimum location of the spring as well as the allowance for summing up the joint deformability components cannot be demonstrated theoretically. The parametric study performed, which consists in the numerical simulation of the behaviour upt to collapse of braced and unbraced frames with semi-rigid joints, is consequently aimed at determining to which extent the actual and relatively complex behaviour of a joint (shear panel + 1 or 2 connections) may be represented, with a sufficient accuracy, by isolated springs fitted with appropriate characteristics.

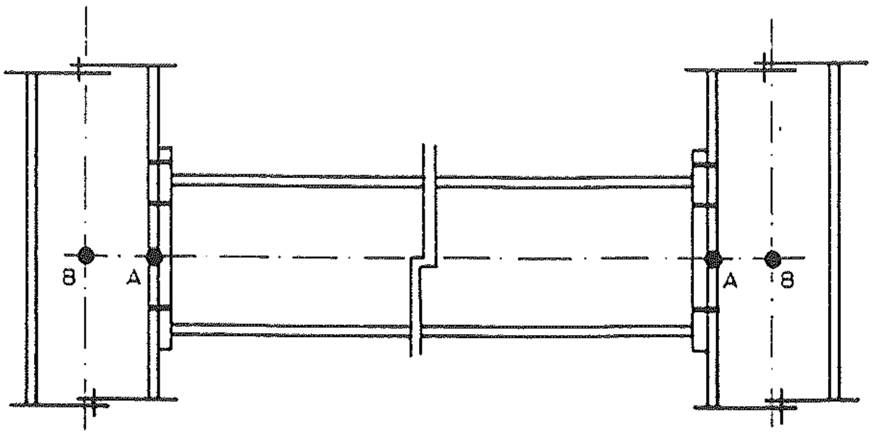


Figure 5 - Possible locations for the springs

In reality, it has been shown that the use of the simplified modelling represented in figure 6 (concentration of the joint deformability at the beam-to-column interface) leads to an accurate prediction of the actual response of braced and unbraced frames, except when the beam-to-column connections are almost fully rigid. In the latter case, the actual frame collapse load would be somewhat underestimated.

It is worthwhile stressing that this simplified modelling is fully representative of the actual joint behaviour when the column web panel is stiffened for shear. The joint deformability then consists in the sole connection deformability which is concentrated at the beam-to-column physical interface, in complete accordance with the proposed modelling.

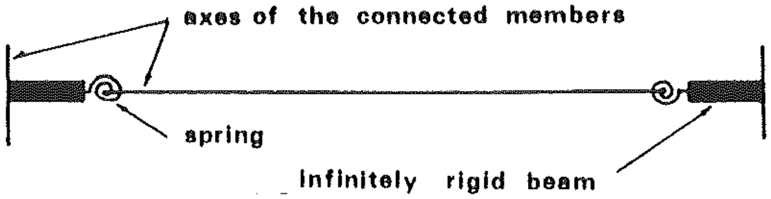


Figure 6 - Simplified modelling of the joint behaviour

This conclusion is in accord with findings of RIFAI [3], when making comparisons between simulated (analytical) predictions and actual (experimental) response of subassemblages.

In definitive: the modelling of the rotational joint characteristic may safely be made at the face of the column and should be an agregate of all joint deformabilities (sheared column web panel and connection).

[1] MAQUOI, R. and JASPART, J.P., "Parametrical study of the numerical modelling for semi-rigid joints", Proceedings of the Annual Technical Session and Meeting, SSRC, Chicago, U.S.A., April 15-17, 1991.

[2] MAQUOI, R. and JASPART, J.P., "Influence of the non-linear and bi-linear modellings of beam-to-column joints on the structural response of braced and unbraced steel building frames", Proceedings of the Fourth International Colloquium on Structural Stability, Mediterranean Session, Istanbul, Turkey, September 16-20, 1991, pp. 133-151.
To appear in the Journal of Constructional Steel Research.

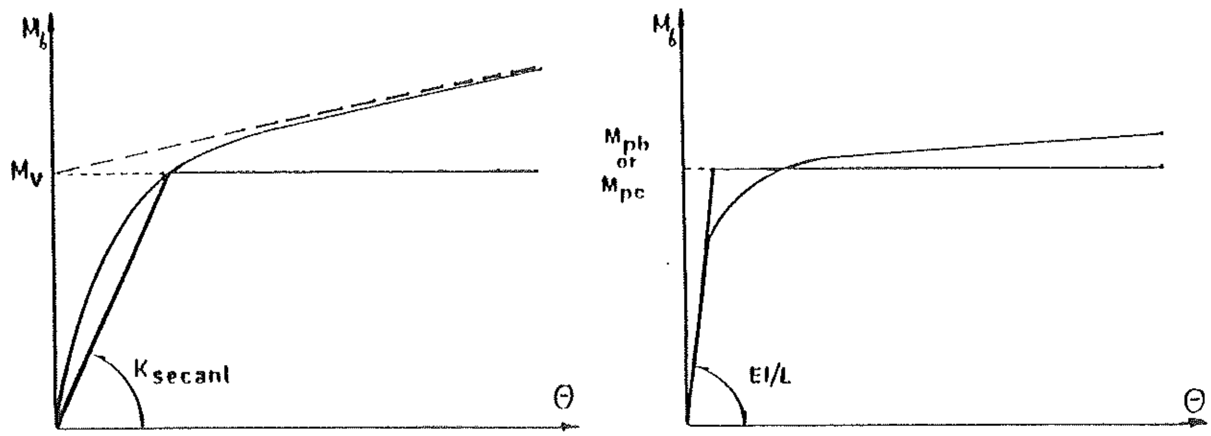
[3] RIFAI
(to be completed in Sheffield).

CONCENTRATION AND "BI-LINEARIZATION" OF THE JOINT RESPONSE

The nonlinear behaviour of the isolated flexural spring which characterizes the joint response (see part of this document entitled "Concentration of the joint deformability") cannot be taken into account in the design practice; the corresponding moment-rotation curve has consequently to be idealized. One of the most simple idealizations to which it may be referred is the elastic-perfectly plastic one (fig. 1.a). This modelling has the advantage to be quite similar to that used traditionally for beam and column sections subject to bending (fig. 1.b).

The moment corresponding to the yield plateau is the joint plastic capacity M_v (called design resistance in Eurocode 3). The constant stiffness which is usually recommended (for instance in Eurocode 3) is the secant stiffness (fig. 2).

BIJLAARD and ZOETEMEIJER assert in [1] that the use of this bi-linear idealization leads to a safe estimation of the frame resistance and of the frame stability. Their argumentation is however far from being satisfactory. The lack of theoretical justification for this concept is the starting point for the study recently performed at the University of Liège, briefly presented here below and in which the design of braced and unbraced frames is successively considered.



a - Joint b - Steel member

Figure 1 - "Bi-linearization" of a moment-rotation curve

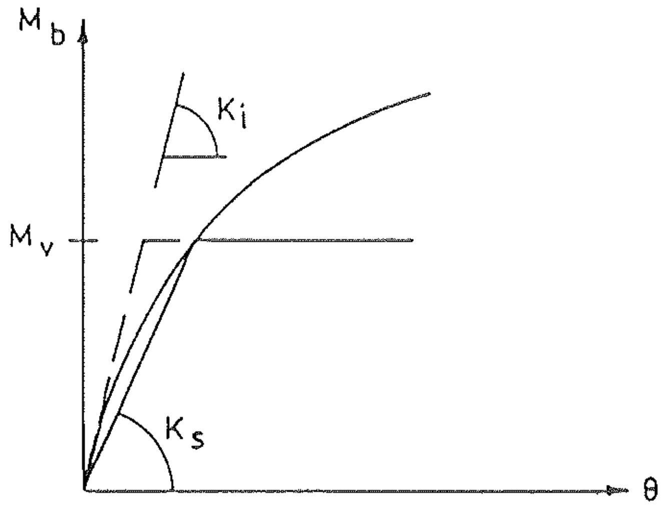


Figure 2 - Bi-linear idealization usually adopted for joints

Before describing the conclusions of this study, let us recall that, according to the philosophy of limit states, any structure must be designed so that it offers not only a specified safety against collapse under factored loads but also complies with durability conditions under service loads over its whole presumed life duration. In other words, the common use of the concentration and of the "bi-linearization" of the joint deformability will be considered as safe if it leads to :

- an overestimation of the transverse beam displacement under service load ;
- an underestimation of the ultimate strength of the frame.

Design of braced frames

The study performed in Liège seems to confirm the conclusions of BIJLAARD and ZOETEMEIJER. In particular, the very good agreement between the instability loads and between the beam displacements under service loads have to be pointed out.

In view to justify these results, let us consider successively the influence of the concentration and of the bi-linearization of the joint deformability curves on :

- the transverse displacement of the beams under servive loads;
- the plastic collapse load of the frames associated to the formation of a plastic mechanism in a beam;
- the ultimate load of the frames associated to the column instability.

This dissociation is helpful in view of a better understanding of the studied phenomena:

- the underestimation of the actual joint loading which results from the concentration (see [2]) and from the bi-linearization of the moment-rotation curves leads systematically to beam displacements under service loads higher than those obtained in the actual frame ;

- the fact that the strain-hardening is not accounted for in the bi-linear model (in the post-limit domain $M_b > M_v$ - fig. 2) is sufficient to explain the safe evaluation of the plastic collapse load (beam mechanism).

The theoretical justification of the use of the bi-linear idealization for the calculation of instability loads is in contrast more questionable. As a matter of fact, it has to be noted that the carrying capacity of columns is not only influenced by their loading at collapse, but also by the amount of flexural restraints at their ends. All these matters are discussed in [2]. It seems however not appropriate to report here on this discussion, the only merit of which is to highlight the influences, often divergent, of the concentration and of the "bi-linearization" on the column loading and on the degree of restraint at the column ends.

The lack of theoretical justification for the safe character of the concentration and of the bi-linearization for the evaluation of the column stability does not prevent us however from recommending it for practical applications. Indeed :

- the modification of the column loading appears to be not very detrimental at collapse;
- the numerical simulations performed [2] let believe to a compensation of the "safe" and "unsafe" restraints at the column ends.

Design of unbraced frames

When loaded, an unbraced frame presents a progressive increasing horizontal deflection, usually designated by the wording "sway". It is generally acknowledged that the sway check under service loads constitutes, much more often than the ultimate limit state, the commanding design criterion. Because of the governing role of the sway, it is quite justified to conduct the design of unbraced frames under service loads by using a geometrically non-linear elastic analysis, i.e. with account taken of the second order effects due to sway, and not to exceed the value M_v of the bending moment (fig. 2) in any connection. In this context, the influence of the secant stiffness on the frame behaviour under service loads appears as one of the most important points to investigate in a near future.

Numerical simulations [2] allow to point out the large overestimation (sometimes 50%) of the actual sway deflection under service loads resulting from the use of the secant stiffness. The fulfillment of the serviceability limit states requires, if it is referred to the secant stiffness, the strengthening of the beam and column sections, as well as that of the connections. The use of the secant stiffness appears consequently to be extremely safe in most of the cases and should not, in these conditions, be regarded as the solution for economical reasons.

Preliminary studies performed in Liège [2] indicate that it would be preferable to refer to a fictitious linear stiffness, called K_{sf} in figure 3, the value of which is intermediate between the initial stiffness K_i and the secant stiffness K_s. This stiffness K_{sf} depends on the type of connections, but also on the type of frames in which these connections are used (number of storeys, of bays, loading, lateral rigidity of the frame). Studies are presently in progress in Liège in order to propose simple procedures for the practical evaluation of this fictitious stiffness.

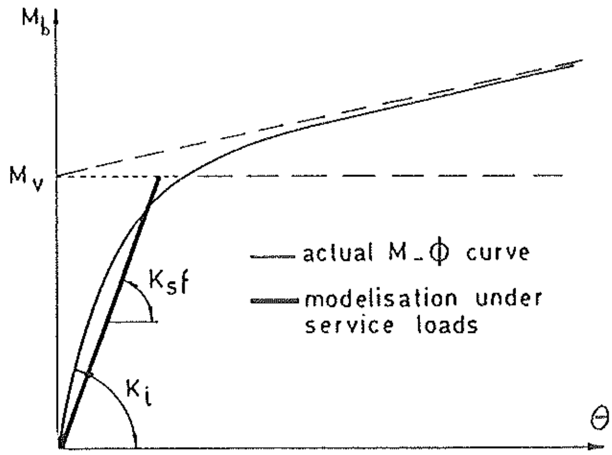


Figure 3 - Fictitious linear stiffness K_{sf}

[1] BIJLAARD, F.S.K., ZOETEMEIJER, P., "Influence of joint characteristics on the structural response of frames".
Proceedings of the International Conference "Steel Structures: Recent Advances and their Application to Design", Budva, Yougoslavia, Sept. 29 - Oct. 1, 1986, pp. 109-133.

[2] JASPART, J.P., "Etude de la semi-rigidité des noeuds poutre-colonne et de son influence sur la résistance et la stabilité des ossatures en acier". Ph. D. Thesis, M.S.M. Department, University of Liège, January 1991.

THEORETICAL PREDICTION OF THE JOINT RESPONSE FOR DIFFERENT CONNECTION TYPES

a. INTRODUCTION

The first step of a semi-rigid structural analysis consists in the characterization of the rotational response of beam-to-column joints or of some of their components (sheared column web panel or connection - see part of the document entitled "Concentration of the joint deformability")*.
Three main approaches may be followed by the designer to achieve this goal:

- the experimental one ;
- the numerical one ;
- the analytical one.

The testing of beam-to-column joints provides the best information concerning the resistance and the stiffness of the joints at each level of loading; however it is also the most expensive and time consuming way of investigation. This approach has consequently no real interest from a practical point of view.

The numerical approach consists in the prediction of the joint deformability curve by means of sophisticated non-linear programs which take account of all the mechanical and geometrical non-linearities. In addition to the use of so sophisticated tools - which, usually, are only available in research centres -, this technique requires a lot of hypothesis relative, for instance, to the modelling of the bolt behaviour, the friction and contact phenomena,...

In conclusion, the only practical way to predict the joint response is the development of analytical procedures, the use of which is based on the knowledge of the mechanical and geometrical properties of the joint components.

Much research work has been devoted, during the last years, to the development of such analytical procedures. Basically reference is made here to the analytical procedure which has been introduced in the Annex J of Eurocode 3 (Chapter 6) [1] for the prediction of the rotational response of beam-to-column joints with welded and end-plate connections.

One of the advantages of the EC3 procedure is its ability to provide a different prediction of the rotational response for the same joint according to the kind of analysis which is planned :

---

* In order to be in accordance with EC3 notations, it will be referred in this part of the document to M-ϕ curves, whatever the component of deformability considered.

- elastic joint representation for a linear analysis (figure 1);
- plastic joint representation for a plastic analysis (figure 2);
- non-linear joint representations for a non-linear computer analysis (figure 3).

Whatever the type of joint representation (figures 1 to 3), the maximum moment carried over by the joint is limited to M_{Rd}, the design resistance moment, which may be considered as the "pseudo-plastic moment" of the joint. Strain-hardening effects and possible membranar effects are consequently disregarded, what explains, in figures 2 to 3, the difference between the actual curve and the "yield plateau" of the modelled one.

A computer program called ENDPLATE has been developed at the University of Liège in view of an extensive comparison, with experimental results, of all the analytical procedures available for joints with end-plate connections. The EC3 approach has obviously been introduced in the program and the conformity with different official publications [2, 3] giving directions for use and worked examples relative to EC3 Annex J has been clearly established.

In this section, results of such comparisons with EC3 will be presented and discussed. Some specific formulae likely to be improved are identified and proposals in complete agreement with EC3 philosophy are presented accordingly.
A good knowledge of the EC3 Annex J and its background are of course very helpful fo fully understand what follows.

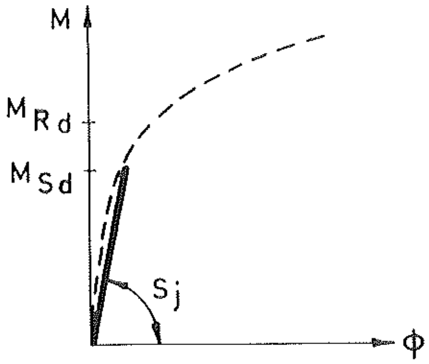
b. COMPARISONS WITH EXPERIMENTAL AND NUMERICAL RESULTS

The quality of the conclusions drawn from comparisons between analytical models and experimental results is largely dependent on the level of available information concerning the testing conditions (mechanical and geometrical properties of all the components, loading sequence, kind of moment and rotation measurements, testing arrangement,...). The comparisons presented in the annex 1 of this section refer to 12 fully documented tests, the results of which are available at Liège University.

These 12 tests on joints with end-plate bolted connections may be subdivised into two categories :

- Test on complete joints (figure 4)

  . Tests 01,04,07,010,013 and 014 between IPE beams and HEB (01,04,07,014) or IPE (013, 014) columns performed at the University of Liège [4].
  . Test T9 between an IPE beam and a HEA column performed at Delft University of Technology [5].



—— : actual M - φ curve
– – : linear representation
M_{Rd} : applied moment
S_j : rotational stiffness

Figure 1 - Linear representation of a M-φ curve

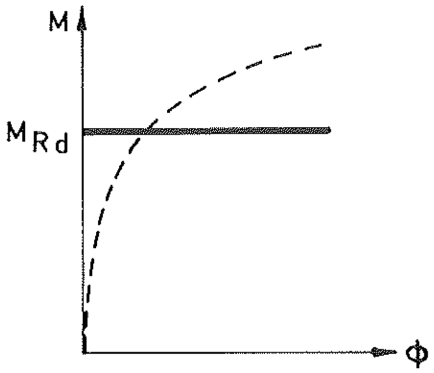


Figure 2 - Rigid-plastic representation of a M-φ curve

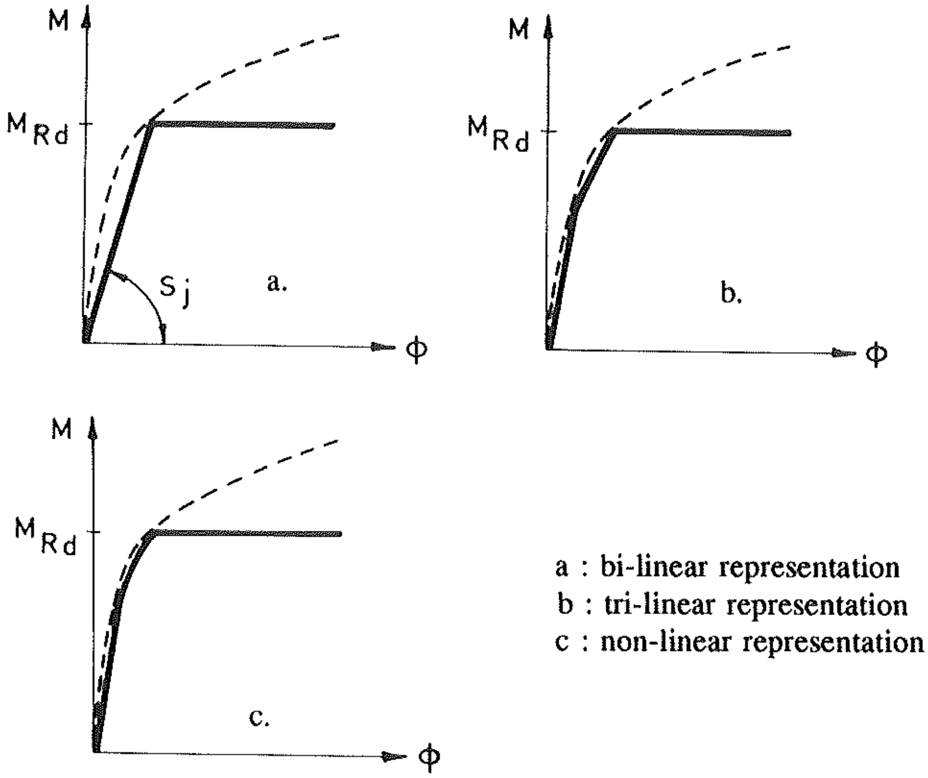


Figure 3 - Non-linear representations of a M-φ curve

- <u>Tests on isolated components</u> (figure 5)

  . Tests EP1-1, EP1-2, EP1-3, EP1-4 and EP1-5 on extended end-plates performed at Politecnico di Milano [6].

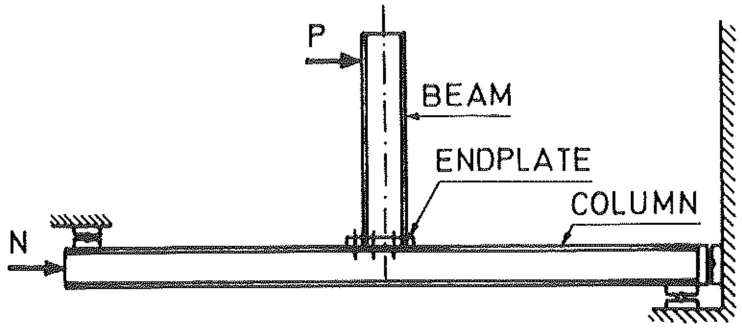


Figure 4 - Testing arrangement for complete joints ("T" joints)

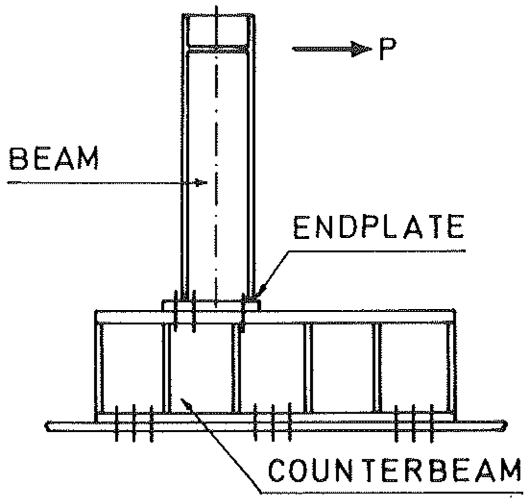


Figure 5 - Testing arrangement for isolated components (end-plates in this case)

For each test performed at Liège University (tests 01, 04, 07, 010, 013 and 014), the three following moment-rotation curves have been recorded :

- the joint moment -rotation curve ;
- the connection moment-rotation curve ;
- the column web panel moment-rotation curve.

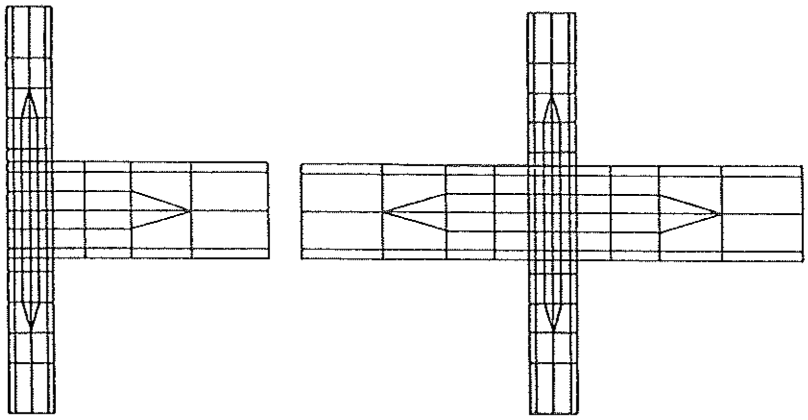
As explained in part of this document entitled "Concentration of the joint deformability", the connection deformability and the shear deformability of the column web panel constitute the two main components of deformability of a strong axis beam-to-column joint.

The three experimental curves are compared with the EC3 model in annex 1, except for tests 013 and 014 where the shear deformability of the column web panel at joint collapse is very limited.

In addition to these comparisons for bolted joints, it is also referred in annex 1 to welded joints. Numerous numerical simulations of cruciform and "T" joints with end-plate connections (figure 6) have been performed jointly at the University of Liège and at the Polytechnic Federal School of Lausanne [7, 8] and the validity of this tool for the prediction of the non-linear behaviour till collapse of welded joints has been demonstrated.

The deformability of such welded joints reduces to two components :

- the shear deformability of the column web panel (for "T" joints only);
- the load-introduction deformability of the column web resulting from the compression and tension deformability of the column web (for transversally unstiffened joints only).



a. "T" joint　　　　b. Cruciform joint symmetrically loaded

Figure 6 - Beam-to-column welded joints : 3-D finite element meshes

The comparison of the EC3 model and the moment-rotation curves relative to these two sources of deformability is of particular interest; therefore some results have been reported in annex 1.

As already mentioned hereabove, the joint modelling depends on the prospective method of frame analysis: the full non-linear analysis requires a non-linear modelling of the M-ϕ curves and, as explained in the part of the document dealing with the "Concentration and bi-linearization of the joint response", it is convenient to refer to a bi-linear modelling similar to that reported in figure 3.a. when using a hand or pseudo-hand analysis procedure. This explains why both kinds of modellings have been compared with experimental and numerical test results in annex 1

c. APPLICATION OF THE LIEGE PREDICTION MODEL TO THE EXPERIMENTAL AND NUMERICAL RESULTS

An original model for the characterization of the rotational non-linear response of beam-to-column joints with welded or end-plate connections has been recently

developed in Liège [8]. One of its main features is its ability to predict the behaviour until collapse either of the joints or of one of their components separately (it takes into account the propagation of strain-hardening and allows the determination of the actual collapse mode).
This model as well as its simplified "bi-linear" version have been also applied to the tests reported in annex 1.

d. CONCLUSIONS OF THE COMPARISONS

The two following tables allow to express an opinion concerning the ability of EC3 and Liège models to predict accurately (figure 3) :

- the stiffness S_j and ;
- the design moment resistance M_{Rd} ;

of the joints with end-plate and welded connections.
Table 1 relates to the semi-rigid behaviour of the connections; it gives the following informations for each test :

- the M-ϕ curves to which it is referred ;
- the type of collapse associated to M_{Rd} ;
- the quality of the prediction of M_{Rd} ;
- the quality of the prediction of S_j.

Table 2, quite similar to table 1, is relevant for sheared column web panels.

d.1. Prediction of the connection behaviour by means of the EC3 model

■ STIFFNESS

- The EC3 model tends to overestimate the load-introduction stiffness (tests NR4 on welded joints) and underestimate the stiffness of the plate elements (tests EP1-1 and EP1-2 for end-plates, test T9 for column flange). The underestimation of the end-plate and column flange stiffness has already been pointed out by different authors.
- Both effects seem usually to compensate for complete bolted joints (tests 01, 04, 07, 010, 013, 014).

■ DESIGN MOMENT RESISTANCE

- The EC3 model underestimates the design resistance of the end-plate connection when the collapse of the connection is associated to the complete yielding of the column flange or to that of the inner (or outer) part of the end-plate (see tests 013, T9, EP1-1, EP1-2, EP1-3 and EP1-4).
  Similar conclusions have been drawn by MOORE from tests on flush end-plate connections [11].
  For tests 014 and EP1-5 respectively, the resistance associated to the collapse by bolt fracture and yielding of the column flange (test 014) or of the extended part of the end-plate (EP1-5) is just higher than the resistance associated to the complete yielding of the column flange or end-plate, what justifies the good prediction of M_{Rd} obtained by means of the EC3 model.

Connection deformability curves		Design resistance M_{Rd}					Stiffness S_j	
		Figure number in annex 1	EC3		Liège		EC3	Liège
Tests			Type of collapse mode associated to M_{Rd}	Prediction	Type of collapse mode associated to M_{Rd}	Prediction		
end-plate	01 Liège	3.c and 3.d	Bolt fracture with column flange yielding	Good	Bolt fracture with column flange yielding	Good	Good	Good
	04 Liège	4.c and 4.d	Bolt fracture with column flange yielding	Good	Bolt fracture with column flange yielding	Good	Good	Good
	07 Liège	5.c and 5.d	Bolt fracture with column flange yielding	Good	Bolt fracture with column flange yielding	Good	Good	Good
	010 Liège	6.c and 6.d	Complete column flange yielding	Seems good	Compression zone of the column web	Seeems good	Good	Good
	013 Liège	7.c and 7.d	Complete column flange yielding	Underesti-mated	Bolt fracture with column flange yielding	Good	Good	Good
	014 Liège	8.c and 8.d	Complete column flange yielding	Good	Bolt fracture with column flange yielding	Good	Good	Good
	T.9 Delft	9.a and 9.b	Complete column flange yielding	Underesti-mated	Complete column flange yielding	Good	Underesti-mated	Good

end-plate	EP1-1 Trento	10.a and 10.b	End-plate - Complete yielding (extd. part) - Bolt fracture with yielding of the inner part	Underesti-mated	End-plate - Complete yielding (Extd. part) - Bolt fracture with yielding of the inner part	Slightly underes-timated	Underesti-mated	Good
	EP1-2 Trento	11.a and 11.b	idem EP1-1	Underesti-mated	idem EP1-1	Slightly underes-timated	Underesti-mated	Good
	EP1-3 Trento	12	idem EP1-1	Underesti-mated	idem EP1-1	Good	-	-
	EP1-4 Trento	12	idem EP1-1	Underesti-mated	Bolt fracture with end-plate yielding in the extended and inner parts	Good	-	-
	EP1-5 Trento	12	idem EP1-1	Good	idem EPI-4	Good	-	-
welded	NR4 "T" Innsbruck-Liège	13.c and 13.d	Load-introduction in column web	Overesti-mated	Load-introduction in column web	Good	Overesti-mated	Good
	NR4 "+" Innsbruck-Liège	14.a and 14.b	Load-introduction in column web	Good	Load-introduction in column web	Good	Overesti-mated	Good

Table 1 - Comparisons of connection deformability curves with EC3 and Liège models

Column web panel deformability curves Tests		Design resistance M_{Rd}					Stiffness S_j	
		Figure number in Annex 1	EC3		Liège		EC3	Liège
			Type of collapse mode assoc. to M_{Rd}	Prediction	Type of collapse mode assoc. to M_{Rd}	Prediction		
end-plate	01 Liège	3.e and 3.f	Sheared colum web panel	Underesti-mated	Sheared column web panel	Good	Good	Good
	04 Liège	4.e and 4.f	idem 01	Underesti-mated	idem 01	Good	Good	Good
	07 Liège	5.e and 5.f	idem 01	Underesti-mated	idem 01	Good	Good	Good
	010 Liège	6.e and 6.f	idem 01	Underesti-mated	idem 01	Good	Good	Good
welded	NR4 "T" Innsbruck-Liège	13.e and 13.f	idem 01	Underesti-mated	idem 01	Good	Good	Good
	transversally stiffened NR4 "T" Innsbruck-Liège	15.a and 15.b	idem 01	Good	idem 01	Good	Overesti-mated	Good

Table 2 - Comparisons of connection deformability curves with EC3 and Liège models

- The load-introduction resistance of column webs is highly overestimated by EC3 annex J (see "T" test NR4). This conclusion is confirmed by a lot of other examples reported in [7, 8].

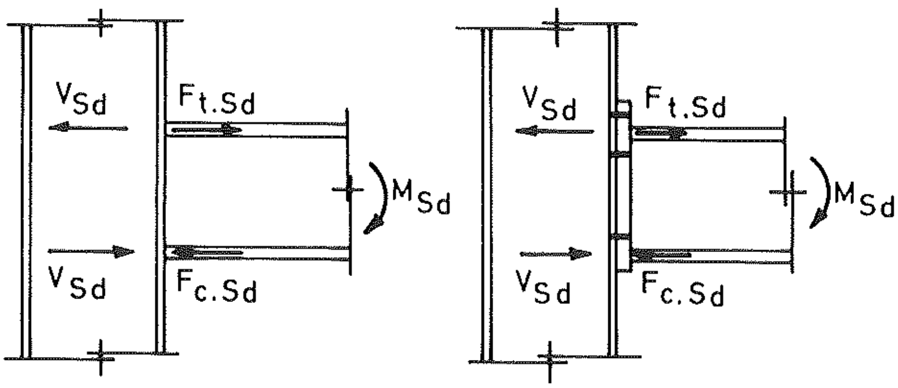
d.2. Prediction of the sheared column web panel behaviour by means of the EC3 model

■ STIFFNESS

- The stiffness of the sheared column web appears to be well predicted by EC3 model.

■ DESIGN MOMENT RESISTANCE

The shear resistance of the column web is underestimated by the EC3 formula, especially for transversally stiffened joints (the good prediction of M_{Rd} obtained by means of the EC3 model for the stiffened "T" joint NR4 results from special loading conditions and is justified herebelow).
It is important to specify that the design resistance of the panel has been obtained, as it is made in references [2] and [3], by assuming that the shear force V_{Sd} in the web panel is equal to the force F_{Sd} acting in the beam flange (figure 7) for "T" joints, irrespective of the shear in the column.



V_{Sd} = shear force in the column web panel
F_{Sd} = forces in the beam flanges statically equivalent to M_{Sd}

a. Joint with welded connection b. Joint with end-plate connection

Figure 7 - Shear force V_{Sd} in the panel

By generalizing this definition to the inner joint represented in figure 8, the following expression is obtained :

$$V_{Sd} = \frac{M_{b1} + M_{b2}}{d_b} \qquad (1)$$

In fact, formula (1) provides only a rough approximate of the true shear force given by :

$$V_{Sd} = \frac{M_{b1} + M_{b2}}{d_b} - \frac{V_{c1} + V_{c2}}{2} \qquad (2)$$

as it results from equilibrium equations [7].

When reference is made to the correct definition of V_{Sd} (formula 2), the EC3 model is found to give (see figure 9 for "T" test NR4 for instance) values of the design resistance which overestimate the experimental results.

For unstiffened "T" joint NR4 (figure 15 in annex 1), the panel of which is subject to pure shear (the actual value of V_{Sd} is equal to $(M_{b1} + M_{b2})/d_b$ in this case), this overestimation is compensated by the additional strength of the panel resulting from the "frame effect"; this "frame effect" is linked to the shear resistance of the frame constituted by the column flanges and the stiffeners.

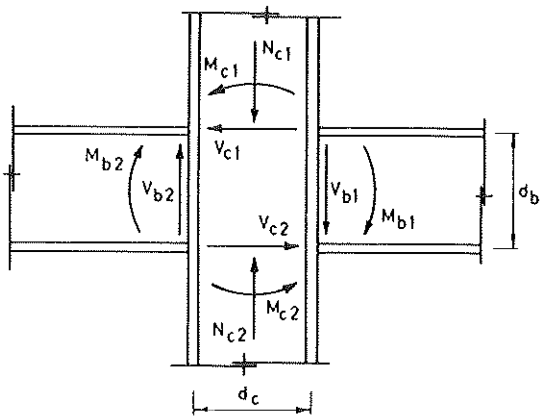


Figure 8 - Loading of an inner joint

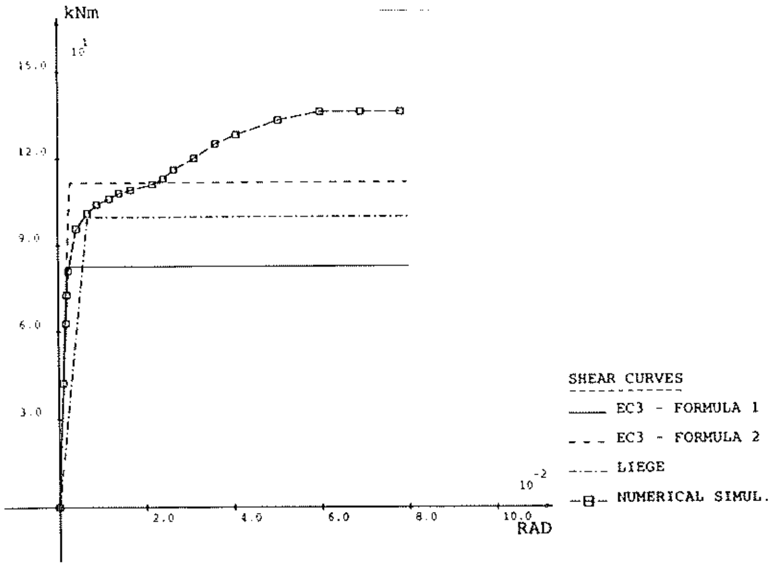


Figure 9 - "T" test NR4: bi-linear characterization of the column web Influence of the V_{Sd} definition.

In conclusion, it is preferable, when applying the EC3 model, to evaluate the shear force V_{Sd} by means of formula (1). Because such a procedure is safe but often too conservative, an amended approach could be contemplated as worthwhile.

d.3. Comparisons with the Liège model

The application of the Liège model to the experimental tests and to the results of the numerical simulations reported in tables 1 and 2 and in annex 1 shows the accuracy of the formulae proposed in [8] for the estimation of :
- the rotational stiffness S_j ;
- the design moment resistance M_{Rd}

of joints with welded and end-plate connections except for tests EP1-1 and EP1-2. For these two tests, the Liège model is still somewhat too conservative; as explained in annex 2, such a situation - linked to the fact that the bolts are not well proportioned to the stiffness of the plates - will probably be met only exceptionnally because not economical at first sight.

The Liège model has consequently been used as a basis for the proposal of the amendments to the EC3 Annex J presented in the next section.

e. PROPOSALS FOR REFINEMENT OF ANNEX J.

e.1. Design moment resistance

The comparisons performed in the previous section allow to highlight the formulae of EC3 Annex J which provide possible unsafe or too safe estimations of the corresponding collapse load. These formulae are related to the three following collapse node :

- Tension resistances of the column flange and of the end-plate which are given in terms of equivalent T-stubs.

Mode 1 : Complete flange or end-plate yielding

$$F_{t.Rd} = \frac{4\ M_{pl.Rd}}{m} \tag{3}$$

Mode 2 : Bolt fracture with flange or end-plate yielding

$$F_{t.Rd} = \frac{2\ M_{pl.Rd} + n\Sigma B_{t.Rd}}{m + n} \tag{4}$$

Mode 3 : Bolt fracture

$$F_{t.Rd} = \Sigma B_{t.Rd} \tag{5}$$

The reader is begged to refer to Annex J for the notations.
Comparisons have pointed out the too safe character of the formula relative to mode 1 "complete flange and end-plate yielding".

- Shear resistance of a diagonally unstiffened column web panel

$$V_{Rd} = f_{yc}\, A_V / \sqrt{3} \tag{6}$$

As clearly shown in section d.2, the too safe or unsafe character of this formula is depending on the definition of the shear force V_{Sd} to which the panel is subject (formulae 1 and 2). From now onwards, it will be systematically referred in this document to the exact definition of V_{Sd} (formula 2), what means that the shear resistance given by EC3 Annex J will be considered as unsafe (see figure 9).

- The design resistance of an unstiffened column web subject to a transverse tensile force :

$$F_{t.Rd} = f_{yc}\, t_{wc}\, b_{eff} \tag{7}$$

or to a transverse compression force :

$$F_{c.Rd} = f_{yc}\, t_{wc}\, [1.25 - 0.5\sigma_n / f_{yc}]\, b_{eff} \tag{8.a}$$

but :

$$F_{c.Rd} \leq f_{yc}\, t_{wc}\, b_{eff} \tag{8.b}$$

These formulae overestimate systematically the actual resistance of the web except when they are applied to inner joints symmetrically loaded (no shear force V_{Sd} in the column web).

Studies performed at Liège University in the seven last years have allowed to explain physically these differences and have lead to proposals of amendments to existing EC3 formulae. The aim of these modifications is to improve the accuracy of the model, to annul its possible unsafe character, but also to keep its practical applicability.

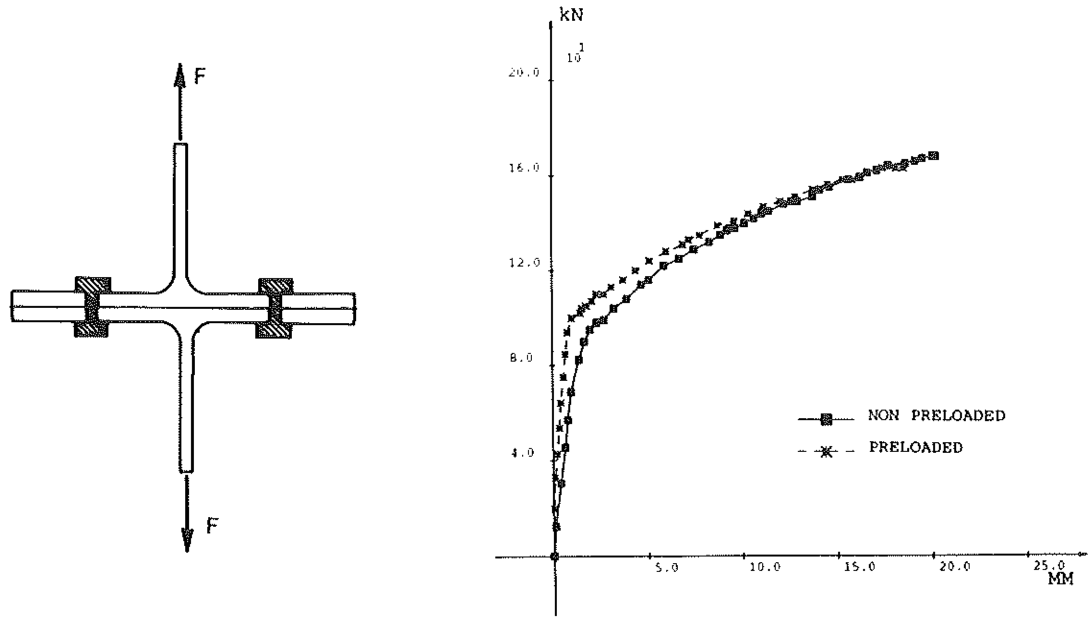
Each of the formulae listed here above are successively envisaged in the following section; reference is made to existing papers or reports for background explanations and justifications and tables summarize proposed "modified EC3" formulae.

■ TENSION RESISTANCE OF THE COLUMN FLANGE AND OF THE END-PLATE

In references [8] and [9], it is clearly stated that the safe character of "EC3 Mode 1" formula (3) results from the hypothesis according to which the forces in the bolts are always idealized as point loads (it is never explicitely accounted for the actual sizes of bolts and washers, on the one hand, and on the degree of bolt preloading, on the other hand).

Physically speaking, the influence of the actual size of bolts and washers on the plastic collapse of a "T-stub" subject to tensile forces is well accepted; other attempts to take this factor into account have been made in the past by different authors. The influence of the bolt prestressing, on the other hand, appears as somewhat new but however as obvious if it is referred to the theoretical model described in [8] and [9] or to the results of two tests recently performed in Liège on quite similar sub-connections constituted of two "T-stubs" connected respectively by means of preloaded and non-preloaded bolts (fig. 10.a).
The related deformability curves point out (figure 10.b):

- the influence of the bolt preloading on the initial stiffness and on the <u>design resistance</u> of the connection and ;
- the absence of influence of this factor on the ultimate state due to the complete loss of prestressing between the connected flanges at that load level.



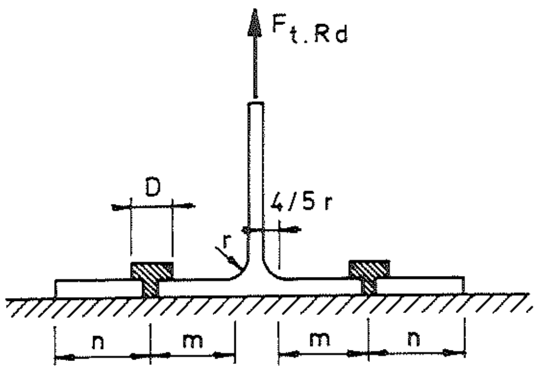
a - Tested sub-connection   b. Related deformability curves

Figure 10 - Influence of the bolt preloading

Refinements have been brought [8, 9] to the T-stub model of EC3 with the result that the amended model provides a higher resistance for collapse mode 1 (formula 3) - see amendments in table 3 - without altering at all the accuracy regarding both collapse modes 2 (formula 4) and 3 (formula 5).
Reference [9] constitutes the annex 2 of the present document.

<table>
<tr><th colspan="2">PLASTIC CAPACITY OF THE COLUMN FLANGE AND OF THE END-PLATE (MODE 1)</th></tr>
<tr><th>EUROCODE 3</th><th>NEW PROPOSAL</th></tr>
<tr><td>

$$F_{t.Rd} = \frac{4\ M_{pl.Rd}}{m}$$



</td><td>

$$F_{t.Rd} = \frac{[8n' - 2(1-K^*)e]M_{pl.Rd} + 1.2n'eK^*\Sigma S}{2mn' - e(1-K^*)(m+n')}$$

with . n' = min (n; 1.25 m)
. e = 0.25 D
where D = the diameter of the bolt head, nut or washer according to which is the greater
. S = initial preloading load per bolt
. $K^* = 0$
The formula is valid under the reservation that :

$$\frac{2F_{t.Rd}\ n' + 4\ M_{pl.Rd}}{2n' - e} \geq 1.2\Sigma S$$

If the condition is not fulfilled, $F_{t.Rd}$ is obtained by means of the same formula where $K^* = 1/1.2$

</td></tr>
</table>

Table 3

■ SHEAR RESISTANCE OF THE COLUMN WEB PANEL

Formula (6) is provided in EC3 Annex J for the assessment of the design resistance of a diagonally unstiffened column web panel subject to shear.
This formula is recommended independently of the presence or not of transverse stiffeners welded on the column web.

This formulation differs from that developed in Liège by an unsafer definition of the maximum shear stress. As a matter of fact, different stresses interact in a column web panel (figure 11):
- shear stresses τ ;
- load-introduction stresses σ_i ;
- longitudinal stresses in the column σ_n

and it has been stated in [7] and [8] that :
- the local distribution of σ_i stresses does not affect significantly the <u>global</u> behaviour of the column web panel ;
- the reduction of the maximum shear stresses at plastic collapse due to σ_n stresses may be estimated by means of the von MISES criterion (see annex 3 [10]).

For sake of simplicity, it is however suggested here not to account for this stress interaction - what could complicate the use of the formula and is not really necessary because of the relatively limited influence of this factor - but to compensate it by a fictitious reduction of the shear web area of the column (see annex 3).
This lead to the proposal summarized in table 4 where the transversally stiffened and unstiffened panels are clearly distinguished - the "frame effect" associated to the shear resistance of the frame constituted by the column flanges and the transverse stiffeners is far from being negligeable-.

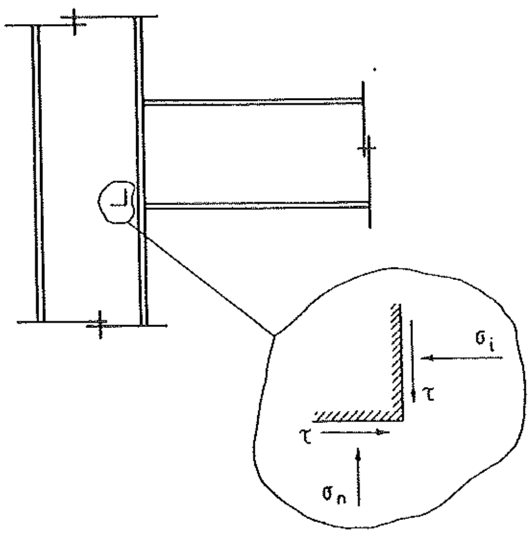
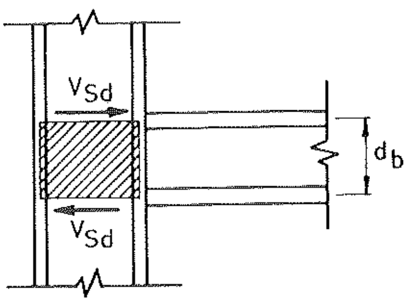
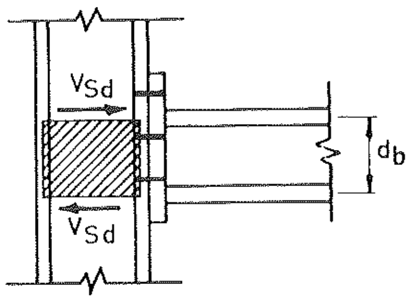


Figure 11 - Stresses interacting in a column web panel.

DESIGN RESISTANCE OF A SHEARED COLUMN WEB PANEL





EUROCODE 3

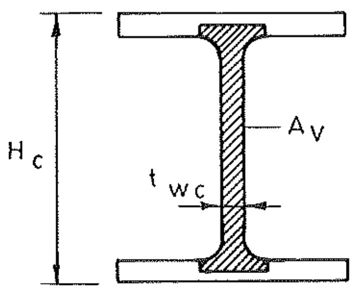
<u>Web panels transversally stiffened or not</u>

$$V_{Rd} = A_v \cdot f_{yc} / \sqrt{3}$$

with :

A_v = shear area of the column web which may be approximated by :

$A_v = 1{,}04\ H_c\ t_{wc}$



NEW PROPOSAL

<u>Unstiffened web panels</u>

$$V_{Rd} = 0.9\ A_v \cdot f_{yc} / \sqrt{3}$$

with :

A_v = shear area of the column web panels given in table 5 and which may be approximated by :

$A_v = \eta\ H_c\ t_{wc}$

où : η = 1,2 for IPE profiles
= 1,5-0,4H_c (H_c in mm) for HEA profiles
= 1,45-0,4H_c (H_c in mm) for HEB profiles

<u>Transversally stiffened web panels</u>

$$V^r_{Rd} = V_{Rd} + V_{cf}$$

avec :

$V_{cf} = 4M_{pf}/d$

M_{pf} = plastic moment of the column flange given in table 5.

Table 4

SECTIONS	Ash (cm^2)	If (cm^4)	Zf=Mpf/fy (cm^3)	L=1/λ (mm)
IPE 80	3.58	0.12	0.44	16.43
IPE 100	5.08	0.24	0.71	20.36
IPE 120	6.31	0.31	0.93	22.69
IPE 140	7.64	0.41	1.19	24.99
IPE 160	9.66	0.69	1.67	28.91
IPE 180	11.25	0.85	2.05	31.15
IPE 200	14.00	1.50	2.88	36.12
IPE 220	15.88	1.83	3.49	38.51
IPE 240	19.14	3.00	4.76	43.82
IPE 270	22.14	3.45	5.50	46.28
IPE 300	25.68	4.03	6.43	48.73
IPE 330	30.81	6.28	8.52	54.90
IPE 360	35.14	7.79	10.39	58.40
IPE 400	42.69	11.50	13.29	64.87
IPE 450	50.85	13.86	15.71	68.76
IPE 500	59.87	17.16	18.96	73.17
IPE 550	72.34	24.87	24.16	80.46
IPE 600	83.78	31.17	29.40	85.48
IPE 750*137	92.90	18.40	23.75	81.37
IPE 750*147	105.41	19.29	24.37	79.55
IPE 750*161	110.98	25.96	30.58	84.74
IPE 750*173	116.44	34.20	37.55	89.81
IPE 750*185	121.12	42.69	44.12	94.12
IPE 750*196	127.27	51.84	50.75	97.69
IPE 750*210	131.52	66.63	60.71	103.35
IPE 750*222	139.76	77.29	67.39	105.65

SECTIONS	Ash (cm^2)	If (cm^4)	Zf=Mpf/fy (cm^3)	L=1/λ (mm)
HE 100 A	7.56	1.29	2.58	27.60
HE 120 A	8.46	1.40	2.90	30.15
HE 140 A	10.12	1.72	3.60	32.78
HE 160 A	13.21	2.90	5.02	37.66
HE 180 A	14.47	3.33	5.90	40.56
HE 200 A	18.08	5.19	7.81	45.49
HE 220 A	20.67	6.48	9.73	48.70
HE 240 A	25.18	10.12	13.14	54.54
HE 260 A	28.76	13.89	16.14	60.18
HE 280 A	31.74	15.57	18.14	62.50
HE 300 A	37.28	22.39	23.24	68.42
HE 320 A	41.13	26.78	27.24	71.94
HE 340 A	44.95	30.28	30.20	74.60
HE 360 A	48.96	34.11	33.32	77.25
HE 400 A	57.33	40.85	38.46	81.57
HE 450 A	65.78	49.97	45.38	87.94
HE 500 A	74.72	60.61	52.92	94.21
HE 550 A	83.72	66.94	57.10	98.42
HE 600 A	93.21	73.74	61.43	102.47
HE 650 A	103.19	81.01	65.93	106.40
HE 700 A	116.97	89.70	70.94	109.55
HE 800 A	138.83	109.08	79.26	118.33
HE 900 A	163.33	128.45	89.66	125.40
HE 1000 A	184.56	138.99	95.10	130.77

SECTIONS	Ash (cm^2)	If (cm^4)	Zf=Mpf/fy (cm^3)	L=1/λ (mm)
HE 100 B	9.04	2.05	3.72	29.56
HE 120 B	10.96	2.77	4.99	33.50
HE 140 B	13.08	3.69	6.55	37.33
HE 160 B	17.59	6.22	9.32	42.41
HE 180 B	20.24	7.85	11.60	46.08
HE 200 B	24.83	11.92	15.41	51.62
HE 220 B	27.92	14.53	18.55	55.23
HE 240 B	33.23	20.83	23.66	60.80
HE 260 B	37.59	27.10	28.15	66.18
HE 280 B	41.09	30.12	31.30	68.86
HE 300 B	47.43	40.85	38.46	74.55
HE 320 B	51.77	47.74	43.66	78.18
HE 340 B	56.09	53.06	47.44	80.97
HE 360 B	60.60	58.79	51.38	83.71
HE 400 B	69.98	68.62	57.75	88.23
HE 450 B	79.66	81.91	66.27	94.72
HE 500 B	89.82	97.03	75.42	101.08
HE 550 B	100.07	105.83	80.41	105.44
HE 600 B	110.81	115.16	85.56	109.63
HE 650 B	122.04	125.05	90.88	113.67
HE 700 B	137.10	136.62	96.74	116.95
HE 800 B	161.75	161.87	106.48	125.67
HE 900 B	188.75	187.26	118.52	132.88
HE 1000 B	212.49	200.94	124.79	138.43

SECTIONS	Ash (cm^2)	If (cm^4)	Zf=Mpf/fy (cm^3)	L=1/λ (mm)
HE 100 M	18.04	11.54	13.59	38.31
HE 120 M	21.15	14.71	17.09	43.20
HE 140 M	24.46	18.48	21.08	47.83
HE 160 M	30.81	26.30	27.24	52.87
HE 180 M	34.65	31.77	32.38	57.18
HE 200 M	41.03	42.81	40.09	62.54
HE 220 M	45.31	50.40	46.51	66.68
HE 240 M	60.07	101.77	77.10	78.04
HE 260 M	66.89	121.98	87.89	83.23
HE 280 M	72.03	133.56	96.05	86.74
HE 300 M	90.53	234.13	143.95	98.14
HE 320 M	94.85	248.32	150.11	101.57
HE 340 M	98.63	248.32	150.11	103.54
HE 360 M	102.41	247.77	149.71	105.35
HE 400 M	110.18	247.22	149.31	108.84
HE 450 M	119.84	247.22	149.31	112.82
HE 500 M	129.50	246.67	148.91	116.35
HE 550 M	139.58	246.67	148.91	119.77
HE 600 M	149.66	246.12	148.50	122.86
HE 650 M	159.74	246.12	148.50	125.79
HE 700 M	169.82	245.56	148.10	128.45
HE 800 M	194.27	264.29	152.71	135.72
HE 900 M	214.43	263.73	152.31	140.24
HE 1000 M	235.01	263.73	152.31	144.50

■ DESIGN RESISTANCE OF A COLUMN WEB PANEL SUBJECT TO LOAD-INTRODUCTION

Formulae 7 and 8 are recommended in EC3 Annex J for the assessment of the design resistance of an unstiffened column web subject respectively to a transverse compression and tensile force.

These formulae differ from those integrated in the Liège model - they are presented in références [7, 8] and in annex 3 [10] - by an unsafer definition of the maximum compressive or tensile stress in the web; this is linked to the fact that the actual interaction between σ_i and τ stresses (see figure 11) has not been accounted for in the EC3 rules.

The application of the EC3 and Liège formulae to the fully welded joints studied numerically and to the joints with extended end-plate connections tested in laboratory (see annex 1) lead to the conclusion that, contrarily to what has been shown for the shear resistance, the design resistance of a web subject to transverse loads is highly dependent on the values of the shear stresses in the web panel, and consequently to the actual joint loading. This influence, which may lead to substantial decreases of the maximum compressive or tensile stress has to be accounted for, even in a simplified computation of the design resistance of the web transversally loaded.

The Liège approach identifies itself obviously to the EC3 one for cruciform joints symmetrically loaded for which the web panel is not subject to shear stresses.

In conclusion, it is proposed to assess the design resistance of a column web by means of the formulae listed in tables 6 and 7.

e.2. Rotational stiffness

The formulation of the rotational stiffness in the Liège model differs appreciably from that suggested in EC3 Annex J. The present section could obviously consist in a presentation of the Liège model, as it is done in [8]. It seems however more appropriate, as the EC3 model provides a relatively good prediction of the stiffness, to use the experience drawn from the development of the Liège model, to slightly amend and modify the EC3 model.
This task should be really facilitated by the work of RYAN and OUDRY who, in the recent paper [12], clearly and simply show the basis on which the EC3 model lies.

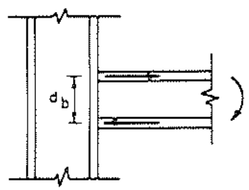
Such a work is actually in progress in Liège; it should lead to the proposal of new refined "k" coefficients (see EC3 Annex J) in a very near future.

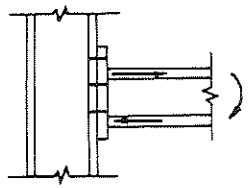
f. <u>EXTENDING OF EC3 ANNEX J TO OTHER TYPES OF CONNECTIONS</u>

An analytical model for the prediction of the rotational response till collapse of joints with flange cleated connections has also been developed in Liège [8]. Its validity has been demonstrated by means of numerous comparisons with results of experimental tests carried out in Liège, Sheffield, Trento and Hamburg [8].

As for the end-plate and welded connections, the knowledge got from this study is likely to be used, in view of the extending of the EC3 Annex J, to the prediction of the rotational behaviour of joints with flange cleated connections.

DESIGN RESISTANCE OF A COLUMN WEB SUBJECT TO A TRANSVERSE COMPRESSION FORCE

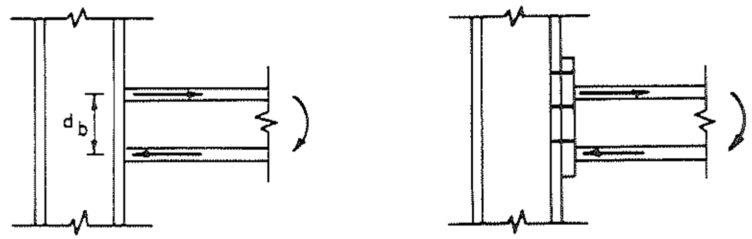




EUROCODE 3	NEW PROPOSAL
$F_{c.Rd} = f_{yc}\, t_{wc}\, [1.25-0.5\ \sigma_n\ /f_{yc}]\ b_{eff}$ but $F_{c.Rd} \le f_{yc}\, t_{wc}\, b_{eff}$	$F_{c.Rd} = \dfrac{f_{yc}}{\sqrt{1+\mu\beta^2}}\ t_{wc}\ [1.25-0.5\sigma_n\ /f_{yc}]\ b_{eff}$ but $F_{c.Rd} \le \dfrac{f_{yc}}{\sqrt{1+\mu\beta^2}}\ t_{wc}\ b_{eff}$ with : . β = ratio between the shear force V_{Sd} and the force in the beam flange $F_{c.Sd}$ at collapse (< 1) Its evaluation may be safely based on the use of formula (1). . $\mu = 3\ k^2\ t_{wc}^2\ /A_v^2$ where A_v = shear area of the column web (see tables 4 and 5) t_{wc}= column web thickness $k = L + 2L^2\ (1+L\ /d_b)/d_b + d_b/6$ where L is a geometrical characteristic of the column given in table 5 Values of μ are given in tables 8, 9 and 10 for common beam-column combinations.

Table 6

DESIGN RESISTANCE OF A COLUMN WEB SUBJECT TO A TRANSVERSE TENSION FORCE



EUROCODE 3

$$F_{c.Rd} = f_{yc}\, t_{wc}\, b_{eff}$$

NEW PROPOSAL

$$F_{c.Rd} = \frac{f_{yc}}{\sqrt{1 + \mu\beta^2}}\, t_{wc}\, b_{eff}$$

with:

- β = ratio between the shear force V_{Sd} and the force in the beam flange $F_{c.Rd}$ at collapse (<1)
  Its evaluation may be safely vased on the use of formula(1).
- $\mu = 3\, k^2\, t_{wc}^2 / A_v^2$

  where A_v shear area of the colum web (see tables 4 and 5)
  t_{wc} column web thickness

  $$k = L + 2L^2\,(1 + L/d_b)/d_b + d_b/6$$

  where L is a geometrical characteristic of the column given in table 5

Values of μ are given in tables 8, 9 and 10 for common beam-column combinations.

Table 7

	POUTRE IPE																	
HEA	80	100	120	140	160	180	200	220	240	270	300	330	360	400	450	500	550	600
100	0.606	0.541	0.525	0.531	0.552	0.582	0.618	*****	*****	*****	*****	*****	*****	*****	*****	*****	*****	*****
120	0.617	0.533	0.504	0.501	0.512	0.532	0.560	0.592	0.629	*****	*****	*****	*****	*****	*****	*****	*****	*****
140	0.663	0.555	0.512	0.499	0.502	0.516	0.536	0.562	0.593	0.646	*****	*****	*****	*****	*****	*****	*****	*****
160	0.710	0.564	0.499	0.470	0.460	0.462	0.472	0.486	0.505	0.540	0.582	0.627	*****	*****	*****	*****	*****	*****
180	0.753	0.581	0.503	0.465	0.449	0.445	0.449	0.459	0.473	0.500	0.534	0.572	0.613	0.675	*****	*****	*****	*****
200	0.837	0.618	0.516	0.464	0.437	0.424	0.421	0.423	0.431	0.448	0.471	0.499	0.529	0.576	0.641	0.712	*****	*****
220	0.947	0.681	0.557	0.492	0.456	0.437	0.429	0.428	0.432	0.445	0.464	0.487	0.513	0.554	0.612	0.676	0.746	0.820
240	1.114	0.768	0.606	0.520	0.470	0.441	0.425	0.417	0.415	0.420	0.431	0.447	0.466	0.496	0.541	0.590	0.645	0.704
260	1.249	0.830	0.636	0.531	0.470	0.433	0.410	0.397	0.390	0.388	0.392	0.402	0.415	0.436	0.469	0.507	0.550	0.595
280	1.355	0.889	0.673	0.557	0.488	0.446	0.420	0.404	0.395	0.390	0.393	0.400	0.411	0.431	0.461	0.496	0.536	0.578
300	1.603	1.018	0.750	0.606	0.521	0.468	0.433	0.411	0.397	0.386	0.383	0.386	0.392	0.406	0.429	0.457	0.489	0.524
320	1.819	1.135	0.824	0.657	0.558	0.497	0.456	0.429	0.412	0.396	0.390	0.390	0.395	0.406	0.426	0.451	0.480	0.512
340	1.979	1.220	0.876	0.692	0.583	0.515	0.470	0.440	0.420	0.401	0.393	0.391	0.394	0.403	0.420	0.443	0.470	0.499
360	2.146	1.307	0.929	0.728	0.608	0.534	0.484	0.451	0.428	0.406	0.395	0.392	0.393	0.400	0.415	0.435	0.460	0.487
400	2.398	1.435	1.004	0.776	0.640	0.556	0.499	0.461	0.434	0.408	0.393	0.387	0.385	0.389	0.400	0.417	0.438	0.462
450	2.776	1.622	1.111	0.842	0.684	0.584	0.518	0.473	0.440	0.408	0.389	0.378	0.373	0.372	0.379	0.390	0.406	0.425
500	3.199	1.830	1.230	0.917	0.733	0.618	0.541	0.488	0.450	0.412	0.387	0.373	0.365	0.360	0.362	0.370	0.382	0.397
550	3.378	1.907	1.267	0.934	0.740	0.619	0.538	0.482	0.441	0.400	0.374	0.357	0.347	0.341	0.340	0.345	0.355	0.367
600	3.554	1.983	1.304	0.952	0.748	0.620	0.535	0.476	0.434	0.390	0.362	0.344	0.332	0.324	0.321	0.324	0.331	0.341
650	3.729	2.059	1.340	0.970	0.756	0.622	0.533	0.472	0.428	0.381	0.352	0.332	0.320	0.310	0.305	0.306	0.311	0.319
700	3.841	2.103	1.359	0.977	0.757	0.620	0.528	0.465	0.420	0.372	0.341	0.321	0.308	0.297	0.291	0.290	0.294	0.301
800	4.218	2.262	1.434	1.014	0.772	0.623	0.524	0.456	0.407	0.355	0.321	0.299	0.283	0.269	0.260	0.257	0.258	0.261
900	4.594	2.427	1.518	1.059	0.797	0.637	0.530	0.457	0.404	0.349	0.312	0.288	0.271	0.255	0.244	0.239	0.237	0.239
1000	4.699	2.457	1.521	1.052	0.786	0.622	0.515	0.441	0.388	0.332	0.295	0.270	0.252	0.236	0.224	0.218	0.215	0.215

Table 8 - μ values for joints with IPE beams and HEA columns

	POUTRE IPE																	
HEB	80	100	120	140	160	180	200	220	240	270	300	330	360	400	450	500	550	600
100	0.737	0.641	0.609	0.608	0.624	0.651	0.686	*****	*****	*****	*****	*****	*****	*****	*****	*****	*****	*****
120	0.844	0.700	0.642	0.622	0.623	0.638	0.662	0.692	0.727	*****	*****	*****	*****	*****	*****	*****	*****	*****
140	0.959	0.764	0.678	0.641	0.628	0.632	0.645	0.666	0.693	0.742	*****	*****	*****	*****	*****	*****	*****	*****
160	1.052	0.798	0.681	0.623	0.595	0.585	0.587	0.596	0.611	0.643	0.682	0.727	*****	*****	*****	*****	*****	*****
180	1.195	0.878	0.730	0.654	0.614	0.595	0.589	0.592	0.601	0.624	0.655	0.693	0.734	0.798	*****	*****	*****	*****
200	1.342	0.944	0.758	0.660	0.604	0.573	0.557	0.551	0.552	0.563	0.582	0.607	0.637	0.683	0.749	0.823	*****	*****
220	1.523	1.046	0.822	0.703	0.634	0.594	0.571	0.559	0.555	0.560	0.574	0.594	0.619	0.658	0.716	0.781	0.852	0.929
240	1.731	1.147	0.876	0.730	0.644	0.592	0.560	0.541	0.531	0.527	0.532	0.544	0.561	0.590	0.634	0.684	0.740	0.801
260	1.902	1.223	0.910	0.742	0.642	0.581	0.541	0.516	0.500	0.489	0.488	0.494	0.504	0.524	0.557	0.595	0.639	0.686
280	2.067	1.310	0.963	0.777	0.667	0.598	0.554	0.525	0.506	0.491	0.487	0.490	0.498	0.515	0.544	0.579	0.619	0.663
300	2.377	1.466	1.053	0.832	0.701	0.619	0.565	0.529	0.505	0.483	0.472	0.470	0.474	0.485	0.506	0.533	0.565	0.601
320	2.673	1.622	1.149	0.897	0.748	0.654	0.592	0.550	0.521	0.494	0.480	0.475	0.475	0.483	0.500	0.524	0.553	0.586
340	2.886	1.732	1.214	0.940	0.777	0.675	0.607	0.561	0.529	0.498	0.481	0.473	0.472	0.477	0.492	0.513	0.539	0.569
360	3.105	1.843	1.280	0.982	0.807	0.696	0.622	0.572	0.537	0.502	0.482	0.472	0.469	0.472	0.484	0.502	0.526	0.553
400	3.430	2.002	1.370	1.038	0.842	0.719	0.637	0.581	0.541	0.501	0.477	0.463	0.457	0.456	0.464	0.478	0.497	0.521
450	3.926	2.242	1.505	1.120	0.895	0.754	0.659	0.594	0.548	0.500	0.470	0.452	0.442	0.436	0.438	0.447	0.462	0.480
500	4.468	2.503	1.651	1.210	0.953	0.793	0.685	0.611	0.558	0.503	0.468	0.445	0.432	0.422	0.419	0.424	0.434	0.447
550	4.691	2.596	1.694	1.229	0.959	0.791	0.679	0.602	0.546	0.488	0.451	0.426	0.411	0.398	0.393	0.395	0.402	0.413
600	4.908	2.687	1.736	1.248	0.966	0.791	0.674	0.594	0.536	0.475	0.436	0.410	0.392	0.378	0.371	0.370	0.375	0.384
650	5.121	2.776	1.777	1.268	0.974	0.792	0.671	0.587	0.527	0.464	0.423	0.395	0.377	0.361	0.351	0.349	0.352	0.359
700	5.251	2.825	1.796	1.273	0.972	0.786	0.662	0.578	0.516	0.452	0.410	0.381	0.362	0.345	0.334	0.331	0.332	0.337
800	5.662	2.989	1.868	1.303	0.981	0.783	0.652	0.562	0.497	0.429	0.384	0.353	0.332	0.313	0.299	0.292	0.291	0.293
900	6.111	3.182	1.963	1.353	1.007	0.796	0.656	0.561	0.492	0.420	0.372	0.339	0.316	0.295	0.279	0.271	0.267	0.267
1000	6.229	3.212	1.964	1.342	0.991	0.777	0.637	0.541	0.472	0.399	0.351	0.318	0.295	0.273	0.256	0.247	0.242	0.241

Table 9 - μ values for joints with IPE beams and HEB columns

	POUTRE IPE																	
HEM	80	100	120	140	160	180	200	220	240	270	300	330	360	400	450	500	550	600
100	1.610	1.270	1.119	1.050	1.024	1.025	1.043	*****	*****	*****	*****	*****	*****	*****	*****	*****	*****	*****
120	1.893	1.426	1.210	1.102	1.048	1.028	1.027	1.041	1.065	*****	*****	*****	*****	*****	*****	*****	*****	*****
140	2.185	1.583	1.302	1.155	1.075	1.035	1.018	1.018	1.029	1.063	*****	*****	*****	*****	*****	*****	*****	*****
160	2.305	1.608	1.282	1.108	1.009	0.953	0.923	0.910	0.908	0.923	0.952	0.990	*****	*****	*****	*****	*****	*****
180	2.634	1.784	1.389	1.176	1.052	0.979	0.936	0.912	0.902	0.904	0.922	0.950	0.986	1.045	*****	*****	*****	*****
200	2.858	1.874	1.419	1.174	1.029	0.941	0.886	0.852	0.833	0.822	0.827	0.843	0.866	0.908	0.971	1.045	*****	*****
220	3.244	2.079	1.544	1.256	1.086	0.980	0.912	0.869	0.842	0.822	0.819	0.827	0.844	0.877	0.930	0.994	1.066	1.145
240	4.826	2.931	2.077	1.622	1.353	1.184	1.072	0.997	0.945	0.895	0.870	0.861	0.862	0.876	0.908	0.951	1.004	1.063
260	5.151	3.063	2.131	1.638	1.346	1.163	1.041	0.958	0.900	0.842	0.810	0.794	0.789	0.794	0.815	0.847	0.887	0.934
280	5.635	3.307	2.274	1.730	1.409	1.207	1.073	0.981	0.916	0.850	0.812	0.791	0.782	0.782	0.798	0.824	0.859	0.901
300	8.046	4.547	3.023	2.231	1.768	1.479	1.286	1.153	1.057	0.958	0.896	0.856	0.833	0.818	0.817	0.829	0.852	0.882
320	8.593	4.807	3.167	2.318	1.824	1.516	1.309	1.167	1.065	0.959	0.891	0.848	0.821	0.801	0.795	0.804	0.822	0.848
340	8.693	4.836	3.170	2.310	1.810	1.498	1.290	1.147	1.043	0.935	0.866	0.822	0.794	0.772	0.764	0.770	0.786	0.809
360	8.743	4.840	3.159	2.293	1.790	1.476	1.267	1.123	1.019	0.911	0.841	0.796	0.767	0.744	0.734	0.738	0.751	0.772
400	8.806	4.831	3.127	2.252	1.746	1.431	1.222	1.077	0.973	0.864	0.793	0.747	0.716	0.691	0.678	0.678	0.688	0.704
450	8.827	4.794	3.075	2.197	1.691	1.376	1.167	1.024	0.920	0.811	0.740	0.692	0.661	0.634	0.618	0.615	0.620	0.632
500	8.760	4.719	3.004	2.131	1.629	1.319	1.113	0.971	0.868	0.761	0.690	0.643	0.611	0.583	0.565	0.560	0.563	0.571
550	8.671	4.635	2.930	2.065	1.570	1.264	1.061	0.921	0.821	0.715	0.645	0.598	0.566	0.538	0.519	0.511	0.512	0.518
600	8.531	4.531	2.847	1.995	1.509	1.209	1.010	0.874	0.776	0.672	0.604	0.558	0.526	0.497	0.478	0.469	0.468	0.472
650	8.398	4.433	2.770	1.932	1.454	1.160	0.965	0.832	0.736	0.635	0.568	0.523	0.491	0.463	0.442	0.433	0.430	0.433
7[illegible]	8.233	4.323	2.688	1.866	1.399	1.112	0.922	0.792	0.698	0.600	0.535	0.490	0.460	0.431	0.411	0.400	0.397	0.398
800	8.253	4.276	2.626	1.802	1.336	1.051	0.864	0.736	0.644	0.547	0.483	0.439	0.408	0.379	0.357	0.345	0.340	0.339
900	7.972	4.098	2.499	1.703	1.255	0.982	0.802	0.680	0.592	0.500	0.438	0.396	0.367	0.339	0.318	0.305	0.299	0.297
1000	7.709	3.936	2.384	1.615	1.183	0.921	0.749	0.632	0.548	0.459	0.401	0.361	0.333	0.306	0.285	0.272	0.266	0.263

Table 10 - μ values for joints between IPE beams and HEM columns

REFERENCES

[1] EUROCODE 3 - DESIGN OF STEEL STRUCTURES, Part 1: General Rules and Rules for Buildings, Commission of the European Communities, November 1991.

[2] STEENHUIS, M. and BIJLAARD, F.S.K., "Example calculation of mechanical properties of a beam-to-column connection according to Eurocode 3", TNO Building and Construction Research, Report B-91-0405, April 1991.

[3] BIJLAARD, F.S.K., "Appendix 1 : Worked example on connection properties", Appendix 1 of the forthcoming ECCS Document on Analysis and Design of Steel Frames with Semi-Rigid Joints, Technical Committee 8 - Structural Stability -, Technical Working Group 8.1/8.2 Skeletal Structures.

[4] JANSS, J., JASPART, J.P. and MAQUOI, R., "Experimental study of the non-linear behaviour of beam-to-column bolted joints", Proceedings of a State-of-the-Art Workshop on Connections and the Behaviour, Strength and Design of Steel Structures, Cachan, France, 25 - 27 May 1987, Elsevier Applied Science Publishers, February 1988, pp. 26 - 32.

[5] ZOETEMEIJER, P., "A design method for the tension side of statically loaded, bolted beam-to-column connections". Heron, Delft University, Vol. 20, N° 1, 1974.

[6] ZANON, P. and ZANDONINI, R., "Experimental analysis of end-plate connections", Proceedings of a State-of-the-Art Workshop on Connections and the Behaviour, Strength and Design of Steel Structures, Cachan, France, 25-27 May 1987. Elsevier Applied Science Publishers, February 1988, pp. 41 - 51.

[7] ATAMAZ SIBAI, W. and JASPART, J.P., "Etude du comportement jusqu'à la ruine de noeuds complètement soudés". Internal report IREM N° 89/7, Polytechnic Federal School of Lausanne and M.S.M. N° 194, University of Liège, October 1989.

[8] JASPART, J.P., "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 M.S.M., University of Liège, 1991.

[9] JASPART, J.P. and MAQUOI, R., "Plastic capacity of end-plate and flange cleated connections - Prediction and design rules", Proceedings of the Second International Workshop on Connections in Steel Structures: Behaviour, Strength and Design, Pittsburgh, U.S.A., 10-12 April 1991.

[10] JASPART, J.P., "Shear and load-introduction deformability and strength of column web panels in strong axis beam-to-column joints - EC3 formulae : discussion and proposals for improvement", Internal Report N° 202, Department MSM, University of Liège, April 1990.

[11] MOORE, D.B., "The design of end-plate connections". R.H. WOOD Memorial Conference, Garston, Watford, 1988, pp. 75 - 107.

[12] OUDRY, G. and RYAN, I., "A propos de la courbe moment-rotation des assemblages boulonnés poutre-poteau selon l'Eurocode 3", Construction Métallique, n°3, 1991, pp. 3 - 13.

ANNEX 2

Refined evaluation of the plastic capacity of the
tensile zone in end-plate connections

ANNEX 3

Design resistance of column web panels
subject to shear and load-introduction

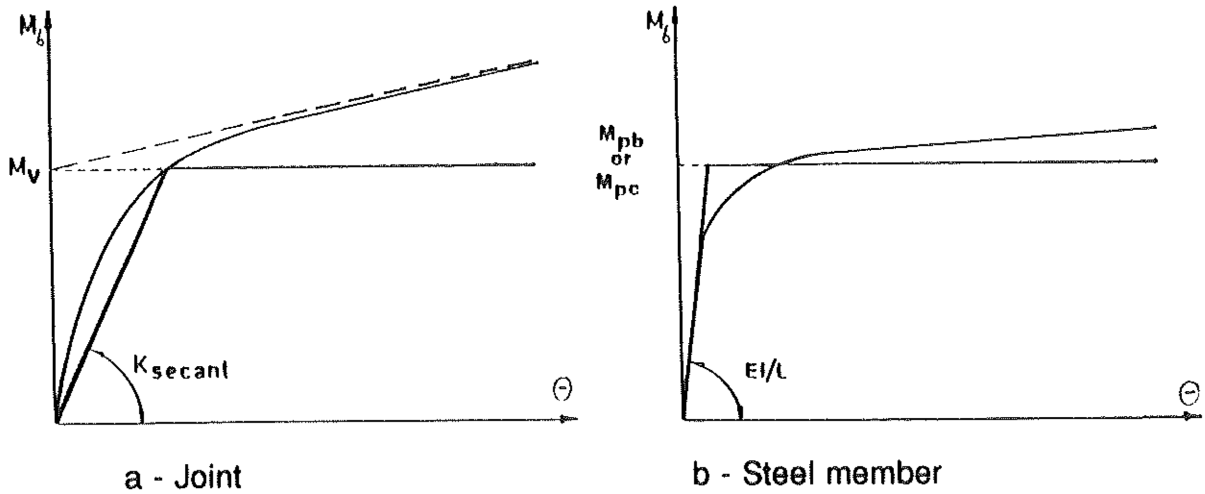
GUIDELINES FOR THE DESIGN OF STEEL BRACED FRAMES WITH SEMI-RIGID JOINTS.

The recent development of numerical programs for structural analysis capable of integrating all the material and geometrical non-linearities, and particularly the actual behaviour of connections, now enables the response of the frames to be simulated until collapse. However, this does not prevent us from the necessity of proposing to designers simple but nevertheless accurate design methods more appropriate to daily practice.
The present paper, devoted to the study of braced frames, falls within this field.

The non-linear behaviour of joints as well as the separate response of connections and of sheared column web panels can obviously not be taken into consideration in practical design. The deformability sources have consequently to be concentrated at the beam-to-column interface (see part of the document entitled "Concentration of the joint deformability") and the corresponding M_b-θ curves have to be idealized (see part of the document entitled "Concentration and "bi-linearization" of the joint response"). The maximum bending moment M_v assumed to be carried over by the joints is represented in Fig. 1.a. This moment is physically linked to an ultimate limiting state that generally corresponds to the yielding of a connection part.

The connection constant stiffness considered in the stability calculation of frames is the secant stiffness (Fig. 1.a).

This idealization of the M_b-θ curves is quite similar to that usually considered for beam and column sections (Figure 1.b).



a - Joint b - Steel member

Figure 1 - Modelization of moment-rotation curves

Hand-calculation methods for the assessment of the pseudo-plastic moment and of the secant stiffness are available for the usual connections (see part of the document entitled "Theoretical prediction of the joint response for different connection types"). This bi-linear representation of the semi-rigid and partial-strength connection behaviour constitutes a safe approximation for the stability calculation of steel frames (see part of the document entitled "Concentration and bi-linearization of the joint response").

The two usual philosophies - elastic and plastic - may be followed for the design of braced frames with semi-rigid joints.

a. ELASTIC DESIGN OF BRACED FRAMES

a.1. Design principles

The elastic design of a braced frame requires a first-order elastic linear analysis in order to determine the internal forces. The extension to the analysis of frames with semi-rigid joints of classical elastic linear methods such as the slope-deflection method and the moment-distribution method was introduced in 1942 by JOHNSTON and MOUNT [1].

The design in itself is achieved according to a 'weak-column-strong-beam' criterion, [2, 3] which consists in designing beams and connections in such a way that their collapse never precedes that of the columns. The stability check of the whole frame is then reduced to the individual check of columns by means of the usual interaction formulae for in-plane or space-loaded columns.

The buckling length of an isolated column, useful for its stability check, may be chosen equal to the column height, commonly termed 'system length'. Studies in Liège [4] have clearly shown that this choice does not lead in all cases to a safe estimation of the buckling collapse load, contrary to what is usually supposed. In reality, columns form part of the frame and a more accurate estimation of their carrying capacity is obtained by considering a buckling length, termed 'effective length', generally smaller but sometimes greater than the system length. This "effective length" results from the presence of end restraints due to the rest of the structure and particularly to the surrounding beams and connections (whose elastic behaviour until frame collapse provides restraints with a constant character) and to the surrounding columns.

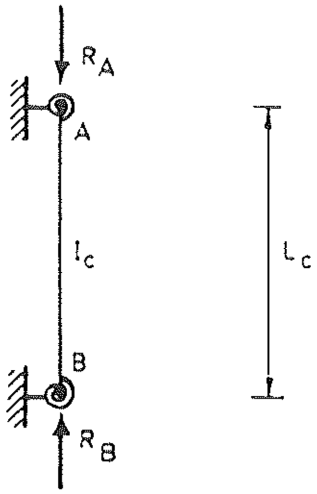


Figure 2 - Isolated column

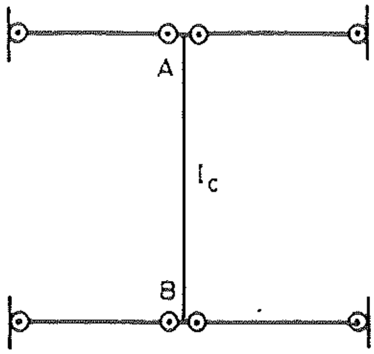


Figure 3 - Substructure defined by BJORHOVDE

a.2. Buckling length of linearly end-restrained columns

The formulae for the stability ckeck of bent and compressed columns apply to columns assumed isolated. Their application to actual columns in braced frames needs the definition of an equivalent isolated and restrained column (fig. 2). The effect of restraints is revealed by the presence, at the column ends, of flexural springs, the rigidity of which is defined in such a way that it equals the effect of that of the rest of the structure.

The determination of the effective buckling length of actual columns will result from the study of corresponding isolated and restrained columns. The main problem obviously lies in the evaluation of the flexural characteristic of the springs.

BJORHOVDE [5] limits the influence of the structure on the column studied to the beams (and the corresponding joints) ending at the extremity considered (Fig. 3). He proposes the following expression for the stiffness of the equivalent flexural spring at each column extremity :

$$\sum_i \frac{2EI_{g,i}}{L_{g,i}} \left\{ \frac{1}{1 + \frac{2EI_{g,i}}{C_{g,i}L_{g,i}}} \right\} \qquad (1)$$

where : E = YOUNG modulus ;
$I_{g,i}$ = stiffness of the beam i ending at the extremity considered;
$L_{g,i}$ = length of the beam i ending at the extremity considered ;
$C_{g,i}$ = secant stiffness of connection between beam i and column.

This equation assumes that the beams of the substructure are bent in single curvature with equal and opposite end rotations. It may be easily modified according to the actual beam-end conditions.

The practical assessment of the effective buckling length of isolated and linearly end-restrained columns may be achieved by means of simplified formulae resulting from a study of elastic linear stability or from the use of buckling curves for end-restrained columns.

A survey of the main existing approaches, as well as an original buckling-length-evalu ation method for columns with different restraints at their ends, is proposed in [4, 6].

Figure 4.b presents, for the lower column of the small structure represented in figure 4.a [column subject to the load $\lambda(P_1 + P_2)$], a comparison of the first order elastic buckling load multipliers obtained respectively by means of :

- the non-linear finite element program FINELG (λ_{crf});
- the EULER formula ($\pi^2 EI/l_{ef}^2$) with the effective length l_{ef} resulting from the BJORHOVDE's approach (λ_{cr});

versus the load P_1 in the upper column.
The critical multiplier obtained by means of FINELG is considered as the exact one.

When P_1 is equal to zero, the upper column acts as a beam in bending and fully restraints the lower column for buckling. The BJORHOVDE's approach is safe in this case (the 'positive' restraint offered by the upper column is neglected).

When P_1 increases, this flexural restraint decreases progressively and vanishes for $P_1 \approx 420$ kN ($P_2 = 800$ kN). For higher values of P_1, the restraint brought by the upper column becomes 'negative'; the BJORHOVDE's approach is then unsafe.

This example point out the necessity of extending the concept of substructure (figure 3) to the upper and lower columns (figure 5).

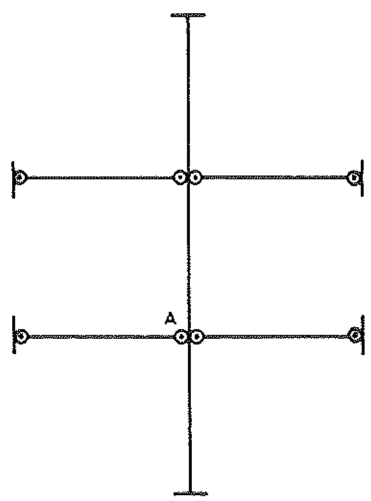
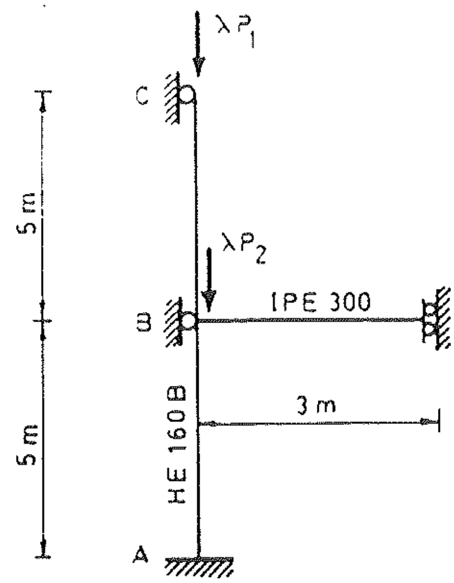
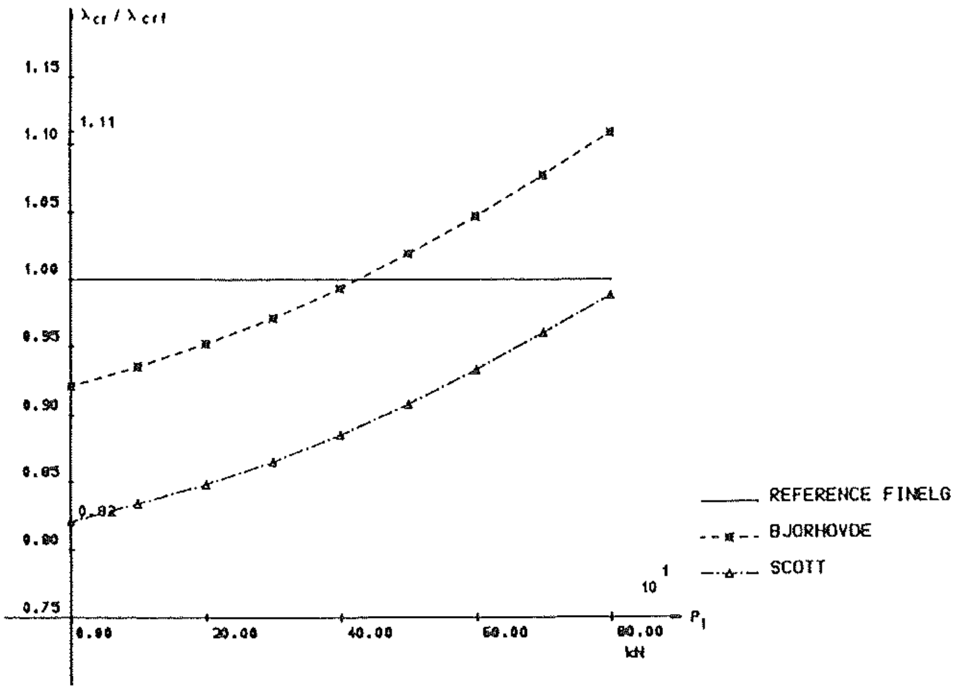


Figure 5 - Extended substructure.



a - Structure



b - Comparison of the two assessments of λ_{cr}

Figure 4 - Comparison between FINELG and BJORHOVDE's approach

BASILL SCOTT's approach for the evaluation of the column end-restraints is based on the study of this extended substructure. This method has been considered in EC3 Annex E; its application to the structure of figure 4.a. demonstrates its generally safe (sometimes really too safe) character (fig. 4.b).

New formulae also based on the study of the extended substructure have beenproposed recently in Liège in order to assess in a more accurate way the value of the restraints at the ends of the isolated column presented in figure 2; they allow to take the value of the normal compression forces in the upper and lower columns into account.
Their application to the submentioned example (figure 6) highlights their accuracy.

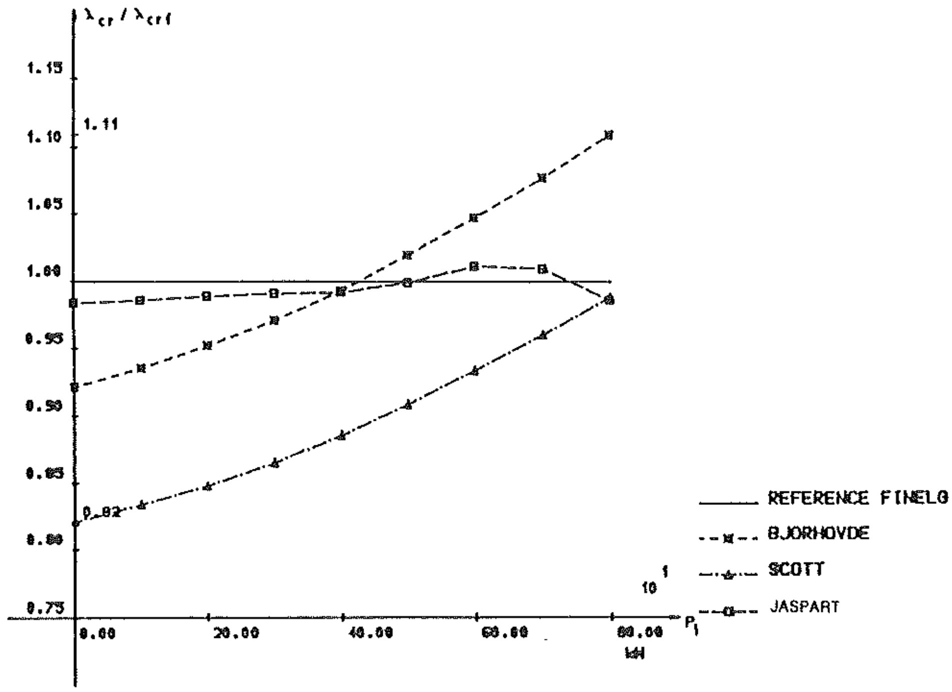


Figure 6 - Assessment of λ_{cr} based on the extended substructure.

Such conclusions should have some interest in view of the possible improvement of EC3 Annex E which refers basically to principles similar to those expressed by SCOTT.

a.3. Second-order effects

SNIJDER, BIJLAARD and STARK [7] have highlighted the possible importance of second-order effects on the behaviour of braced frames. Indeed, the compression axial forces acting in the columns produce a decrease of their flexural stiffness; this so-called 'ε effect' has an influence on the bending-moment diagram of the whole frame and may cause the premature collapse of beams and/or joints, which results, for the columns, in a reduction in the amount of restraint at their ends and in a modification of their loading. According to BIJLAARD and SNIJDER, the influence of these second-order effects could be neglected when :

- the beam-span-to-column-height ratio is larger than 1.0 ;
- the moment capacity of the beam is larger than that of the column.

However, studies performed in Liège [8, 9] have not allowed these conclusions to be confirmed.

As long as more reliable criteria are not available, the following steps are suggested [4] :

. to design the frame according to the principles expressed above by referring to the first-order elastic linear analysis of the whole frame ;
. then to check that the second-order moments in the frame at collapse do not exceed the plastic moment M_{pb} of the section in the beams and the design resistance M_v in the joints. It must be noted that the second-order elastic linear analysis of a braced frame may be achieved in a simple, accurate and non-iterative way by means of the modified slope-deflection method developed by VANDEPITTE [10].

b. PLASTIC DESIGN OF BRACED FRAMES

The plastic design is achieved according to a 'strong-column-weak-beam' criterion [2, 3] in which the frame collapse is associated with the formation of beam plastic mechanisms. The check of the column is performed, in a similar way to that described here, in the structure submitted to collapse loads, a part of which remains elastic [4].

The problem of the rotation capacity and of the required minimum stiffness of connections for a plastic design is dealt with, among other things, by STARK and BIJLAARD [11].

c. CONCLUDING REMARK

The calculation in Liège of a large number of different structures through the non-linear finite element program FINELG and the design methods described has enabled the degree of accuracy of the proposed methods to be shown.

[1] JOHNSTON, B. and MOUNT, E.H., Analysis of building frames with semi-rigid connections. American Society of Civil Engineers, Transactions, 107 (1942), 993-1019.

[2] VINNAKOTA, S., Design of columns in planar frames. A few comments, presented at the National Conference on Tall Buildings, held at New-Delhi, 1973.

[3] VINNAKOTA, S., Design of columns as part of frames - some remarks. Paper submitted to TG3 : SSRC, 1983.

[4] JASPART, J.P., 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, M.S.M. Department, University of Liège, January 1991.

[5] BJORHOVDE, R., Effect of end restraint on column strength - Practical Applications, Engineering Jl. A.I.S.C., to be published.

[6] MAQUOI, R. and JASPART, J.P., Contribution to the design of braced frames with semi-rigid connections. In Proceedings of the Fourth International Colloquium on Stability of Metal Structures, North American Session, New-York City, 17-19 April 1989.

[7] SNIJDER, H.H., BIJLAARD, F.S.K. and STARK, J.W.B., Use of the elastic effective length for stability checks of columns and consequences for checks on beams in braced frames. In Proceedings of the Michael R. Horne Conference, ed. L.J. Morris, Granada, London, 1983.

[8] ELSEN, Ph., Méthodes manuelles de prédimensionnement des ossatures métalliques contreventées à assemblages semi-rigides, Diploma Work, M.S.M. Department, University of Liège, Belgium, 1989.

[9] AMOUR, A., Contribution à l'étude de la capacité portante des colonnes de structures contreventées à noeuds semi-rigides. Maîtrise en Sciences Appliquées, Service M.S.M., Université de Liège, Belgium, 1988.

[10] VANDEPITTE, D., Non-iterative analysis of frames including the P-Δ effect. Journal of Constructional Steel Research, 2 (1982), 3-10.

[11] STARK, J.W.B. and BIJLAARD, F.S.K., Design rules for beam-to-column connections in Europe. Monograph on beam-to-column connections. Ch. 17. Institute TNO for Building Structures and Building Materials. Delft, Netherlands, 1984.

(References from Sheffield could certainly be added in this list - in particular for the work related to the column buckling length - pages 2 and 4).

Université
de Liège

Mécanique des matériaux
Stabilité des constructions
Mécanique des structures

MSM - Institut du Génie Civil
Quai Banning, 6
B-4000 LIEGE - BELGIQUE
Tél. (0)41 - 52 01 80 - Ext. 238
Telefax 32-41-53 09 78

ANNEXES