Research activities under COST C1 at the Department MSM of the University of Liège[*]

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Keywords: Steel Connection, Semi-Rigid, Classification, Fictitious Stiffness, Stress Interaction

Abstract: *Present paper summarises the research activities under COST C1 project at the University of Liège. The different topics investigated in this research funded by the Walloon Region of Belgium are listed. Three of them are discussed with more details and first results are presented.*

1. Introduction.

Department MSM of the University of Liège is involved since two years in a three years COST C1 research project funded by the Walloon Region of Belgium. This project is a continuation, in Liège, of different past projects which were all dealing with the semi-rigid response of structural joints and building frames. Actually, through this COST C1 project, the possibility is given to the Department MSM to investigate some different specific topics which have been less investigated in the past, on which the scientific knowledge is therefore limited and which could prevent, in the future, the practical designers from the economical use of the semi-rigid concept.

The following research topics are investigated:

a. Extending of the existing prediction models for the mechanical properties of structural steel joints to other types of joints like composite ones or column bases (theoretical investigations, development of softwares and experimentation on column bases);
b. Stress interaction in the column web panels of major axis beam-to-column structural joints and influence of the normal stresses in the columns on the response of the column web panels (theoretical investigations and experimentation);
c. Development of predesign procedures for braced frames with semi-rigid joints (theoretical investigations);
d. Study of criteria allowing to evaluate the influence of the stiffness decrease of columns in compression on the collapse load of braced frames (theoretical investigations);
e. Definition of a fictitious constant joint stiffness in view of the analysis, under service loads, of unbraced frames with semi-rigid joints (theoretical investigations);
f. Experimentation and theoretical studies on weak-axis beam-to-column joints;
g. Classification of beam-to-column joints.

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Because of the limited number of pages, only points b., e. and g. will be discussed in the following chapters of the present paper. Information concerning the other topics can be provided to any interested person.

2. Influence of stress interactions in the column web panels of strong axis beam-to-column joints.

2.1. Objectives.

The interaction of stresses in the column web panel of strong axis beam-to-column joints resulting from the internal forces acting in the connected members leads to a decrease of the design resistance of the column webs subjected to transversal forces carried over from the beam(s) to the column through the connection(s) (load-introduction resistance).

These interactions are covered by specific reduction factors in the new revised Annex J of Eurocode 3 [1]. These ones are safe but lead in some cases to some significant underestimations of the design resistance. For information, the previous version of Annex J [2] was sometimes particularly unsafe.

The aim of this part of the COST C1 research project is to improve again the existing reduction factors by limiting their possible conservative character.

2.2. Research progress.

In a strong axis joint between H or I hot-rolled sections, the collapse of the column web panel can result from two different modes: shear yielding (figure 1) and local yielding under load-introduction (figure 2) or eventually web buckling for slender web.

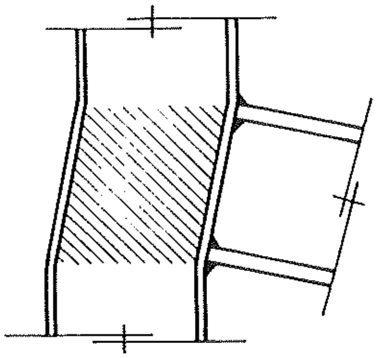

Figure 1 - Shear collapse

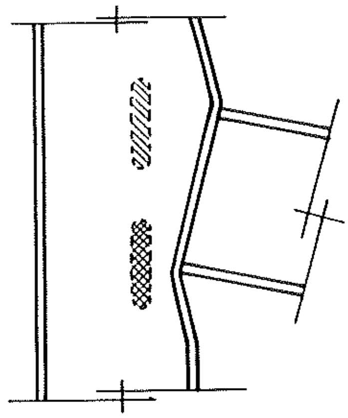

Figure 2 -Collapse under load-introduction

For a given joint, the collapse mode of the web panel depends on the loading; this is illustrated in figure 3 where the ratio between the left and right loads varies from 0 to 1(in the case of a web with low slenderness - so, not likely to buckle -).

This ratio is the one between the two bending moments applied on each side of the column. When it is close to 0, the web panel is subjected to high shear force which could lead to a shear collapse. On the other side, a ratio close to 1 means that the joint is symmetrically loaded. The collapse can just result from local yielding under load-introduction (figure 3).

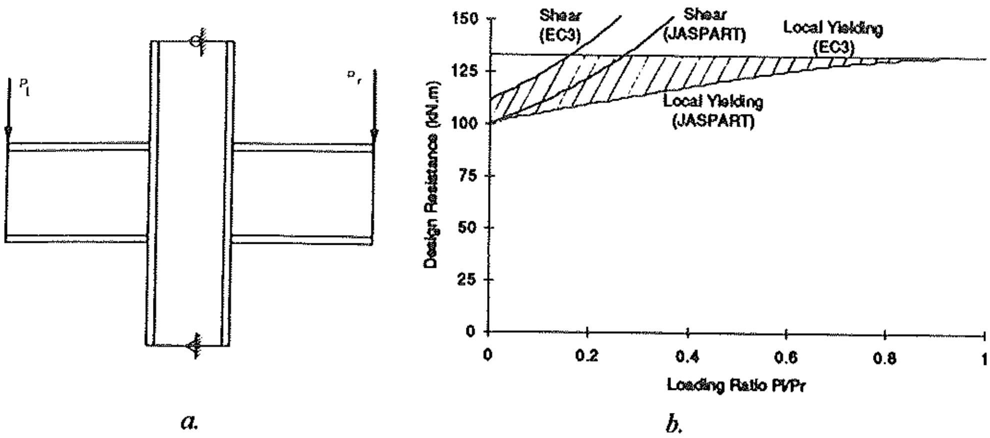

Figure 3 - Variation of web panel resistance according to the loading.

JASPART has shown in his Ph.D. thesis [3] that the shear stresses in the web panel decrease the local load-introduction resistance. In the same way, he asserts that the shear resistance is also decreased by the longitudinal stresses in the column resulting from the normal bending moment. The hachured zone of figure 3 .b. represents the unsafe character of the design rules which were proposed in the old version of Eurocode 3 Annex J [2] compared to the JASPART's model.

As a consequence, the relevant parts of Annex J [1] have been recently revised. Now, it takes into account:

- the variation of the load-introduction resistance with the loading ratio (see figure 4);
- the reduction of the shear resistance (figure 4);
- for the slender webs, the possible reduction of the design resistance due to the web buckling.

On the other hand, a large set of numerical simulations with shell elements have been performed with the non linear finite element program FINELG [4] developed at the Department MSM of the University of Liège. Some of the results of these simulations are shown in figure 4. They show that the design rules introduced in the new revised Annex J are safe but somewhat too conservative in some cases.

The present part of the COST C1 research project is then aimed at improving the Annex J rules by developing a new load-introduction model allowing to follow in a less conservative way the actual behaviour of the web.

A preliminary model has been developed, the application of which allows to predict the shape of the "numerical simulations" curves shown in figure 4.

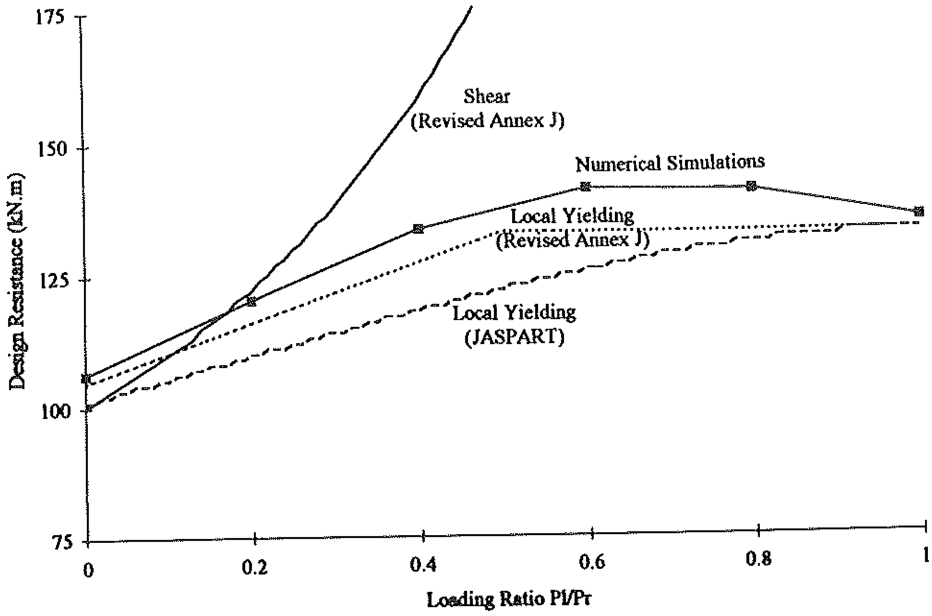

Figure 4 - Comparison between JASPART's model, revised Annex J model and numerical simulations.

An other point studied in this research is the effect of a normal force in the column on the load-introduction resistance of the web panels and on the design resistance of column flanges subjected to transverse tensile forces (in bolted connections). These two interactions are covered by specific rules in the new revised Annex J.

A great number of numerical simulations with different dimensions, steel grades and normal force values lead to the conclusion that the Eurocode 3 rules, thought as very simple, are accurate enough in a lot of cases. But, in a point of view of a researcher, further investigations are certainly necessary.

Experimental tests are aimed at highlighting all these interactions in progress. They will be probably finished in November 94. Further to these tests, the available models should be improved.

3. Determination of a fictitious linear elastic stiffness for beam-to-column joints in view of the analysis, under service loads, of unbraced frames with semi-rigid joints.

The aim of the present part of the research is to determine a fictitious elastic stiffness for beam-to-column joints in view of the elastic analysis, under service loads, of unbraced frames with semi-rigid joints.

For elastic frame analysis, an idealized joint response characterized by a linear elastic stiffness has to be substituted to the complex and non linear actual M-ϕ curve, but in such a way that the global frame response is not significantly affected compared to the actual one. In view of the determination of the fictitious "equivalent stiffness", five different structures (one storey-one bay; one storey-two bays; three storeys-one bay; two storeys-three bays; four storeys-three bays) have been numerically studied by means of the finite element program FINELG [4] and this, in the two following cases:

- *exact numerical simulation* (figure 5): real behaviour of joints with different non linear deformability curves for shear web panels and for connections in bending;

- *numerical simulation with idealized joint response* (figure 6): concentration of the connection and web panel deformability into flexural springs and linearization of the joint M-ϕ curve.

The elastic stiffness usually recommended is the secant stiffness corresponding to the design resistance M_{Rd}, which may be defined as equal to $K_i/3$ for welded joints and bolted joints with end plates. In the new revised Eurocode 3 Annex J [1], a value of $K_i/2$ is now recommended (figure 6).

The numerical simulations show that the use of $K_i/3$ is unsatisfactory; it leads to large and non economical overestimations of the frame displacements. The value of $K_i/2$ is better, generally safe, but can sometimes lead to underestimations of the displacements.

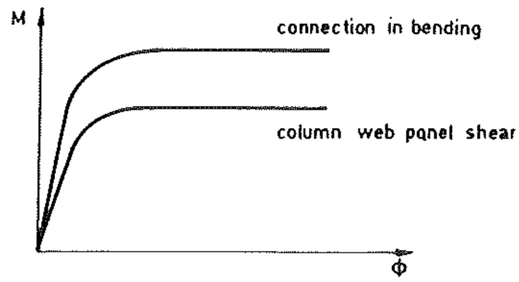

Figure 5 - Actual M-ϕ curves for connections in bending and column web panel in shear.

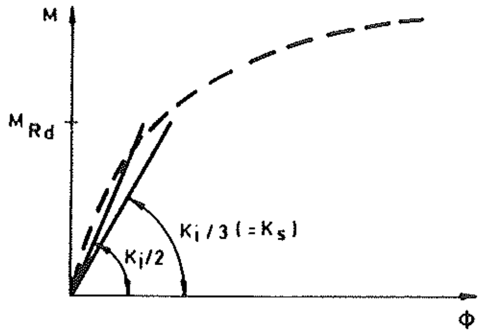

Figure 6 - Concentration and linearization of the M-ϕ curve.

In this project, an original approach for the determination of this fictitious stiffness has been proposed; it consists to estimate, by a simple procedure, the value of a mean rotation ϕ_g in the joint acting at the ends of the considered beam, to evaluate the associated bending moment by referring to the actual joint response and, lastly, to deduce the corresponding secant stiffness K_{sf} which will be used to characterize the joint behaviour when analysing the structure.

Step 1

Determination of the value of the rotation ϕ_g by similarity with the so-called "wind connection method"; the mean rotation in the joints at the beam extremities results from the beam deformability (the column being considered as rigid):

$$\phi_g = \frac{qL_b^3}{24\ EI_b} - \frac{ML_b}{2\ EI_b} \qquad \text{in case of distributed load q} \qquad (1)$$

$$\phi_g = \frac{PL_b^2}{16\ EI_b} - \frac{ML_b}{2\ EI_b} \qquad \text{in case of concentrated load P at mid-span} \qquad (2)$$

where I_b is the beam inertia;
L_b is the length of the beam;
M is the moment at beam end.

Step 2

Determination of the moment M which corresponds to the rotation ϕ_g by referring to the approximated tri-linear moment-rotation curve (see figure 7).

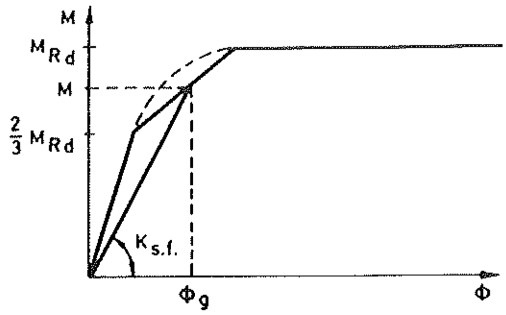

Figure 7 - Derivation of M and K_{sf}

Step 3

The fictitious secant stiffness K_s is such that $K_{sf} * \phi_g = M$ (figure 7). From this equation and from the equations 1 and 2, the value of the rotation ϕ_g is derived:

$$\phi_g = \frac{qL_b^2}{12(\frac{2\ EI_b}{L_b} + K_{s.f})} \qquad \text{distributed load} \qquad (3)$$

$$\phi_g = \frac{PL_b}{8(\frac{2\ EI_b}{L_b} + K_{s.f})} \qquad \text{concentrated load} \qquad (4)$$

By replacing M and ϕ_g by their expression in the straight line equation characterizing the elastic-plastic part of the tri-linearized actual M-ϕ curve:

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$$\frac{M-\frac{2}{3}M_{Rd}}{M_{Rd}-\frac{2}{3}M_{Rd}}=\frac{\phi_g-\phi_{el}}{\phi_{pl}-\phi_{el}} \quad \text{with} \quad \begin{cases}\phi_{el}=\dfrac{2M_{Rd}}{3K_i}\\ \phi_{pl}=\dfrac{3M_{Rd}}{K_i}\end{cases} \tag{5}$$

the values of $K_{s.f}$ are determined:

$$K_{s.f} = \frac{qL_b^{\,2}\ K_i + 96\ \dfrac{EI_b}{L_b}\ M_{Rd}}{7\,qL_b^{\,2}\ -\ 48\ M_{Rd}} \quad \text{distributed load} \tag{6}$$

$$K_{s.f} = \frac{PL_b\ K_i + 64\ \dfrac{EI_b}{L_b}\ M_{Rd}}{7\,PL_b\ -\ 32\ M_{Rd}} \quad \text{concentrated load} \tag{7}$$

An upper bound (K_i) and a lower bound (K_s) of $K_{s.f}$ values have obviously to be defined.

The two expressions of $K_{s.f}$ (6 et 7) can be written in the following format:

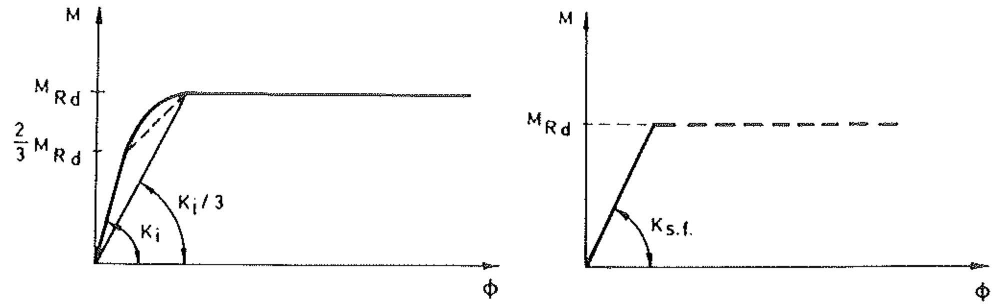
$$\boxed{K_{s.f} = \frac{K_i\ M_{enc}\ +\ 4\,R\ M_{Rd}}{7\ M_{enc}\ -\ 4\ M_{Rd}}} \tag{8}$$

where M_{enc} is the end moment at the fixed ends of a symmetrically loaded beam;
R is the beam stiffness ($2EI_b/L_b$).

To verify this result, the transversal displacement V under service loads of the five studied structures, obtained through two types of numerical simulations:

- numerical simulation with actual joint response (figure 8a);
- numerical simulation with linear joint response (figure 8b).

have been compared.



a- actual M-ϕ curves *b - linearized M-ϕ curves*

Figure 8 - Joint M-ϕ curves.

An example is given in figure 9 for the structure with three storeys and one bay:

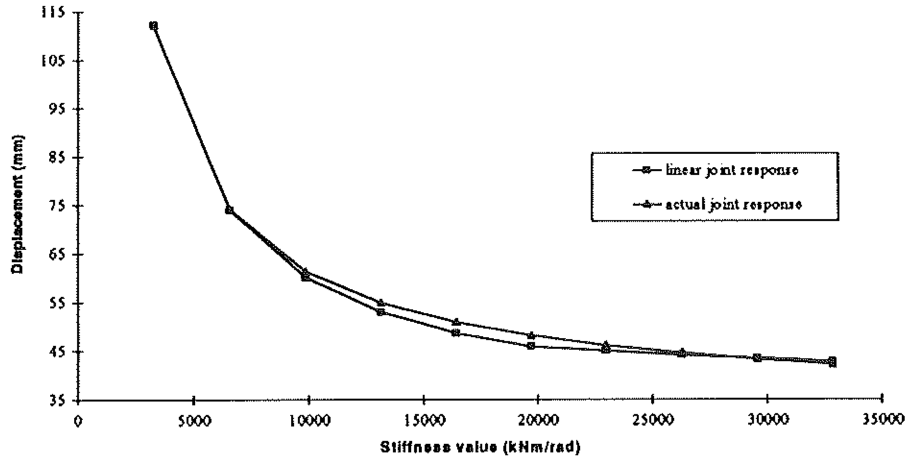


Figure 9 - Comparisons.

Comparisons have been performed, for all the five considered structures, with different connection detailing: extended end plates and flush end plates.

In figure 10, actual and calculated displacements are reported; it can be seen that the majority of the calculated displacements differ of less than 5 % from the actual ones. In view of practical application, the simple format of the proposed expression has also to be highlighted.

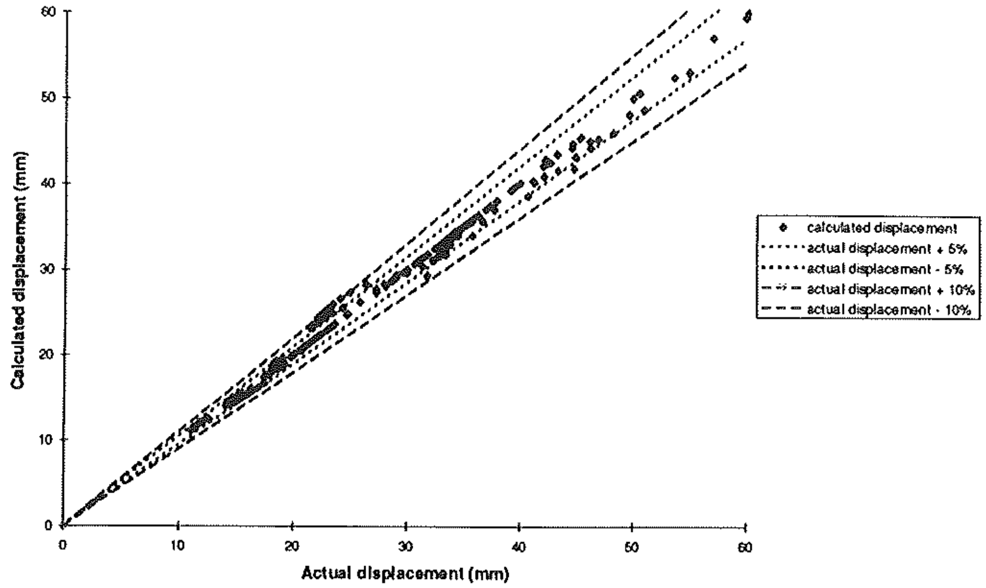


Figure 10 - Comparisons between actual and calculated frame transverse displacements under service loads.

4. Classification system for beam-to-column joints.

4.1. Generals.

Most of engineers, researches and designers do nowadays agree with the fact that the actual response of almost all the beam-to-column structural joints is non-linear.
The concepts of perfectly rigid or pinned joints is a pure theoretical view of mind (see figure 11) which is nevertheless well useful to simplify the calculation of actual frame structures.

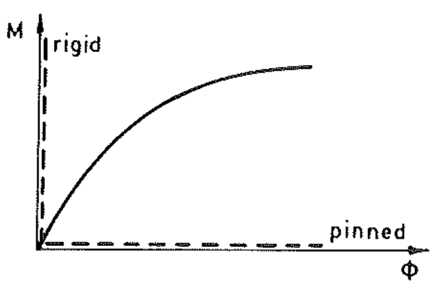


Figure 11 - Beam-to column connection behaviour

In practice, some real beam-to-column joints can yet be considered as pinned if their behaviour is such that the bending moment they can carry over is so law that it does not significantly influence the general behaviour of the structure. In the same way, some actual structural connections can be considered as perfectly rigid if the relative rotation between the connected beam and column is small enough not to significantly influence the general behaviour of the frame.

So it could be of great help to have a classification system at one's disposal in order to see whether an actual beam-to-column connection can reasonably be considered as pinned or rigid or whether the joint semi-rigidity has to be taken into account in the frame design procedure.

Several classification systems have already been proposed :
- beam reference length [5];
- classification adopted in Eurocode 3 [2] based on an Euler instability criterion [6];
- T.N.O.'s proposal [6];
- Innsbruck's proposal [7].

4.2. Purpose of the work.

As none of these classification systems is fully satisfactory, it has been decided to dedicate a part of the COST C1 Project to the study of new classification criteria. It was felt that the problem had to be taken back from the beginning to determine the parameters that effectively govern the different phenomena.

The aim of the research is thus to find two new classification boundaries; these boundaries have to be defined in terms of rotation stiffness (rigid, semi-rigid and pinned) and moment capacity (full strength, partial strength and pinned) (see figure 12).

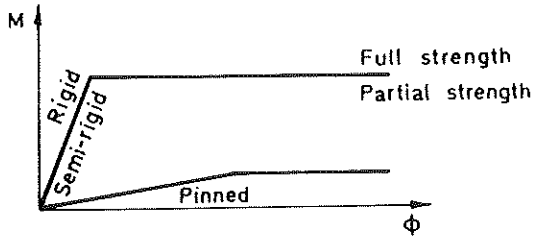


Figure 12 - Schematic joint classification

Only the stiffness classification is discussed in this paper.

The problem of classification is very complex and many factors must be taken into consideration:
- the geometrical and mechanical properties of the joints ;
- the connected elements (beam and column) ;
- the global structural geometry (braced or unbraced frames constituted of one or several bays and storeys).

Different classification criteria have also to be considered according to the envisaged limit states and a particular attention has to be paid to the serviceability limit state when deriving the stiffness classification boundaries.

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4.3. What has already been done ?

Different classification stiffness boundaries - between rigid and semi-rigid - have been established in the case of a simple portal frame - braced or unbraced - with rigid or pinned column bases (see figure 13). The comparison of these boundaries allows to determine, in each case, the most determinant ones.

Braced portal frames

Unbraced portal frames

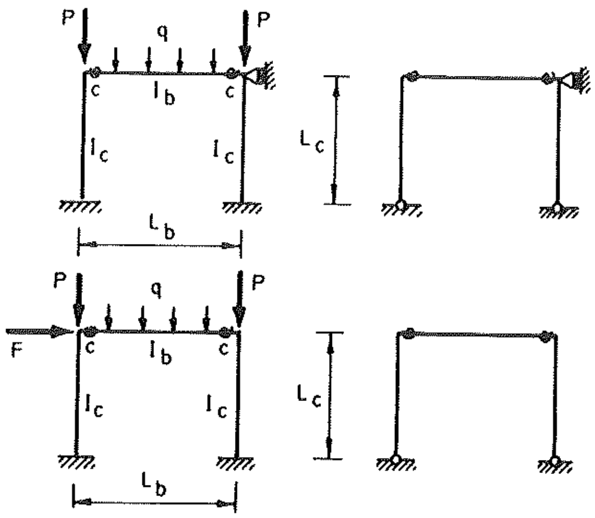

Figure 13 - Studied simple portal frames

The boundaries are based on classification criteria defined as ratios β either between two loads or between two displacements, one calculated for the structure with semi-rigid beam-to-column connections $(\bar{c})$, the other one for the same structure with rigid connections $(\bar{c} = \infty)$.

The parameter $\bar{c}$ is defined in formula (12).

The considered criteria are:

- Ultimate load criteria

$$\beta_u = \frac{F_u(\bar{c})}{F_u(\bar{c} = \infty)} \qquad (9)$$

The ultimate load is determined by the Merchant-Rankine formula in the case of an unbraced structure.

- Deformation criteria

$$\beta_f = \frac{f(\bar{c} = \infty)}{f(\bar{c})} \qquad (10)$$

with f = second order elastic lateral displacement under service load in the case of an unbraced frame ;
= elastic mid-span beam transversal displacement in the case of a brace frame.

The joints may be considered as rigid if there influence on the structural frame response is limited to 5% for resistance criteria (β_u = 0,95) or 10% for deformability criteria (β_f = 0,9).

All those criteria can be represented by a curve in the following non-dimensional axis:

$$\rho = \frac{R_b}{R_c} \text{ in abscissa ;} \tag{11}$$

$$\bar{c} = \frac{c}{R_c} \text{ in ordinate} \tag{12}$$

where c is the flexibility of the semi-rigid beam-to-column connection ;

$$R_b = \frac{EI_b}{L_b}\text{, the beam-rigidity} \tag{13}$$

$$R_c = \frac{EI_c}{L_c}\text{, the column rigidity.} \tag{14}$$

These curves represent in fact the boundary between the rigid and semi-rigid domains.

In the case of unbraced frames, the most determinant criterion is the one concerning the lateral displacement even if the β values for deformations are less severe ($\beta_f = 0{,}90$) than those relative to ultimate loads ($\beta_u = 0{,}95$) (see figure 14).

These boundaries for unbraced structures have been confirmed by calculations performed by means of the non linear FEM software FINELG [4] on realistic simple portal frames with rigid or pinned column bases (see figure 14).

The next step in the study consists to see whether that deformation criterion built up for a simple portal frame could be extended to multi-bays, multi-storeys frames.

It seems that the problem can be solved, in the case of one-storey multi-bays structures, by referring to the so-called equivalent Grinter frame [8]. Two examples of one-storey two-bays frames - with rigid column bases - have been investigated with FINELG and it appears from the study that the same deformation criterion defined here above can be used provided the beam and column rigidity R_b and R_c be replaced by the corresponding ones R_c^{eq} *and* R_b^{eq} in the equivalent Grinter frame .

The results are presented in figure 15 where the boundary deformation criterion has been expressed in the new defined non dimensional axes:

$$\rho^{eq} = \frac{R_b^{eq}\left(\bar{c} = \infty\right)}{R_c^{eq}} \tag{15}$$

$$\bar{c}^{eq} = \frac{R_b^{eq}\left(\bar{c}\right)}{R_b^{eq}\left(\bar{c} = \infty\right)} \tag{16}$$

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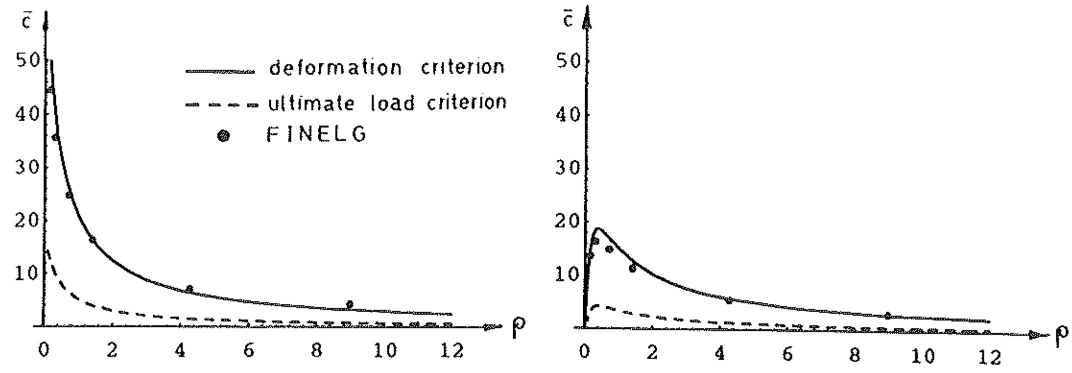

a - Portal frame with rigid column bases *b - Portal frame with pinned column bases*

Figure 14 - Classification criteria and comparison with FINELG.

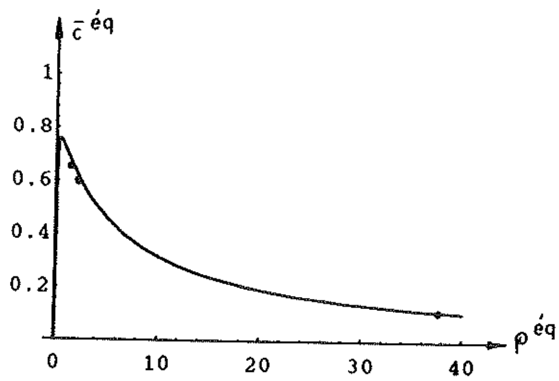

Figure 15 - Deformation classification criteria and FINELG results for multi-bays frames.

4.4. Future work.

The stiffness boundary for unbraced structures is in a good way of achievement as the governing parameters are beginning to be pointed out. Nevertheless, before establishing a classification system for beam-to-column connections, a lot of work is still to be done:

- stiffness boundary between rigid and semi-rigid domains for multi-storeys unbraced structures;
- stiffness boundary between rigid and semi-rigid fields for braced structures ;
- classification according to the resistance for braced or unbraced frames ;
- problem of boundary between semi-rigid and pinned fields.

Work on these different topics is in progress.

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