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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

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SUMMARY

The practical design of a steel frame with semi-rigid joints requires the idealization of the non-linear behaviour of the beam-to-column joints. As a matter of fact, it is quite difficult in practice to account separately for the non-linear response of sheared column web panels and that of connections subject to bending moments. Based on large parametric studies of braced and unbraced frames - performed by means of the non-linear finite element program FINELG - present paper gives guidelines on how to concentrate the joint deformability in single isolated springs acting at beam ends and on how to "bi-linearize" their non-linear response.

INTRODUCTION

For sake of economy, beam-to-column bolted joints without any column web stiffener become a common practice (joints between H or I sections). Such a joint has a non-linear behaviour : when the beam is subject to bending, the axes of the connected members do not rotate a same angle, what results in a relative rotation that is not proportional to the beam bending moment.

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.

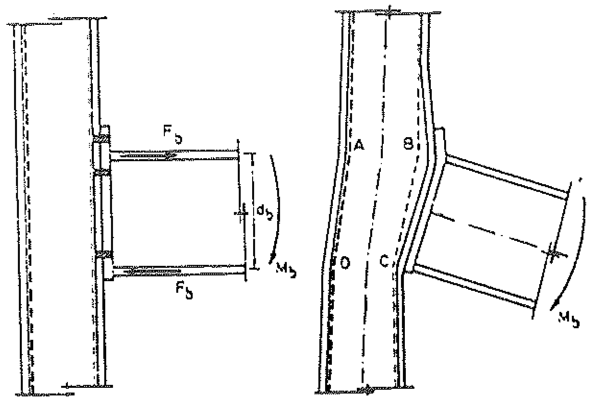


Figure 1 - Deformation of a strong axis joint

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 the 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) (Ref. 1).

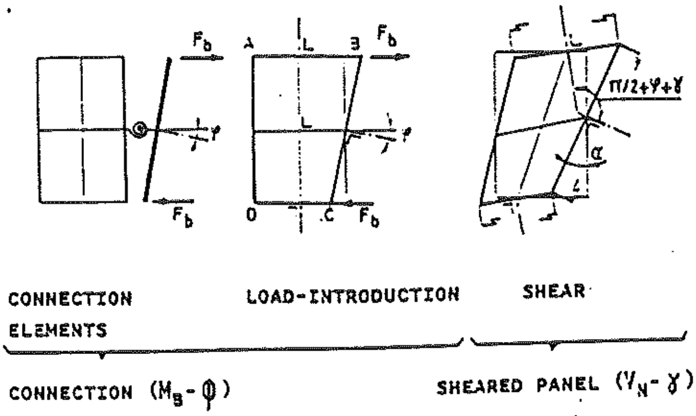


Figure 2 - Joint deformability components

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.

Furthermore, the use of non-linear curves for the characterization of the joint behaviour is quite incompatible with a practical design of steel frames. This means that the moment-rotation curve associated to the single flexural spring has to be idealized.

Based on an extended parametric study of braced and unbraced frames-performed by means of the non-linear finite element program FINELG - present paper gives guidelines on how to define the spring characteristics in an accurate and safe manner for design practice.

2. FINELG FINITE ELEMENT PROGRAM

FINELG is a materially and geometrically nonlinear finite element program which has been developed jointly at the University of Liège, Belgium, and at the Polytechnic Federal School of Lausanne, Switzerland. It is used to solve problems such as :

- step-by-step structural response up to and beyond collapse;
- linear and non-linear instability with calculation of critical loads and instability modes;
- calculation of eigen frequencies and eigen modes, possibly taking into account the current stress state.

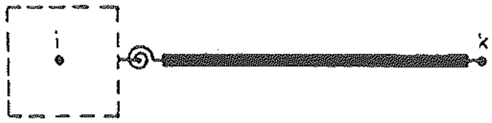
Its library is composed of spatial truss bar, plane beam, spatial beam, membrane plate, thin and thick shells, springs, linear constraints,...

FINELG program has been recently implemented to simulate accurately the non-linear behaviour of connections and sheared column web panels (Ref. 2).

The flexural behaviour of the connection and of the adjacent beam as well as that of the column web panel are gathered into a <u>single</u> "plane beam + connection + sheared panel" finite element. Any type of non-linear response may be associated to the behaviour of the beam, of the connection and of the web panel respectively. Aforementioned element can be used according one of the three different manners sketched in figure 3.

a. - "Beam" element

b. - "Beam + connection" element



c. - "Beam + connection + sheared web panel" element

Figure 3 - Use of the finite element

It exhibits several superiorities :

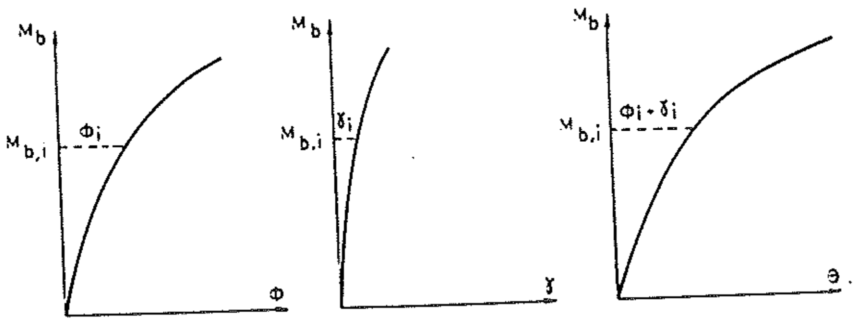
- in contrast to many other approaches (Ref. 3, 4, 5), it fulfills the equilibrium equations of the web panel;
- it makes possible an accurate and realistic picture of the actual macroscopic behaviour of the column web panel;
- it does not need a more refined discretization than that just required when rigid joints;
- the number of equations which have to be solved at each analysis iteration is not increased due to the non-linear semi-rigidity of the joints.

3. PARAMETRIC STUDIES

For daily practice, it cannot be expected to account separately for both the flexural behaviour of the connection and the sheared behaviour of the column web panel. Therefore the deformability of both connection and column web panel has to be concentrated into a flexural spring located at the beam end (fig. 4 and 5). Furthermore, how such springs affect the frame response can only be reflected through appropriate design methods provided that their flexural behaviour be idealized (Ref. 2). And it must be noted that, among the different recommended idealizations (in Eurocode 3 for instance (Ref. 6)), only the bi-linear one appears to be of interest and really useful for a simplified design (fig. 6.a.).

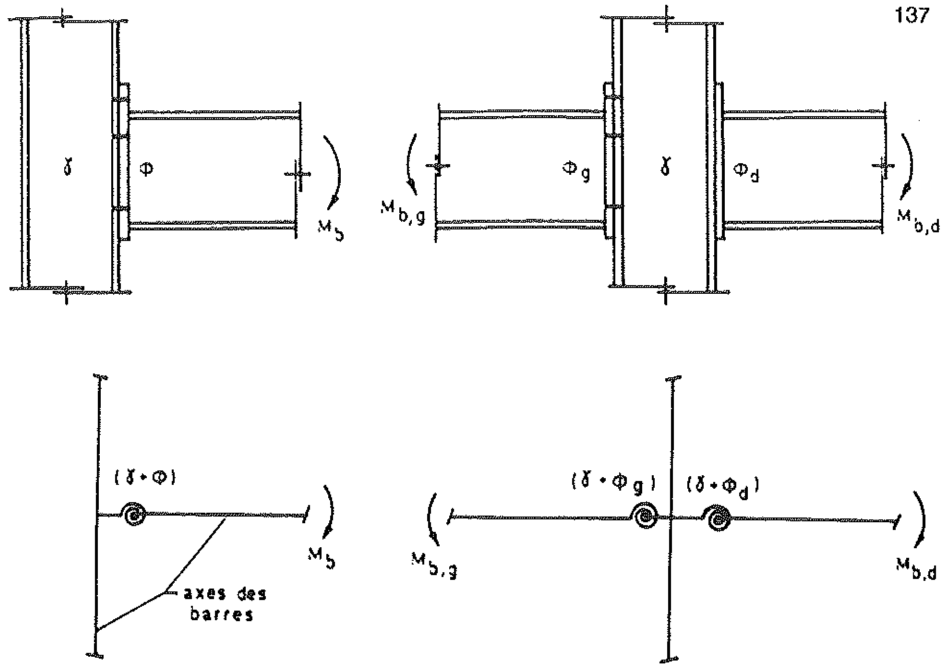
In these conditions, a question occurs inevitably : do the concentration of the joint deformability into single flexural springs and the "bi-linearization" of their corresponding moment-rotation curves lead, or not, to a safe and accurate frame design?

The two parametric studies described in this paper are aimed at providing an answer to this specific problem.



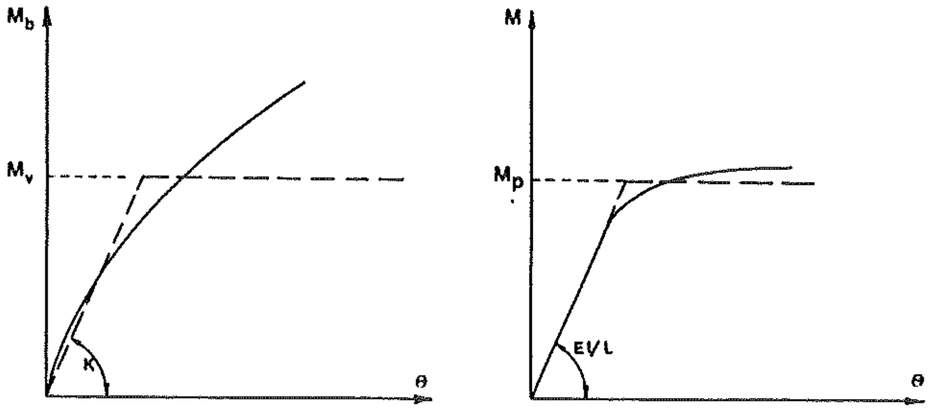
a - Connection b - Sheared panel c - Spring

Figure 4 - Flexural characteristics of the spring



a - Exterior joint b - Interior joint

Figure 5 - Concentration of the joint deformability into flexural springs



a - Joint b - Steel member

Figure 6 - "Bi-linearization" of a moment-rotation curve

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3.1. <u>Concentration of the joint deformability</u> The concentration of the joint deformability is schematized in figures 4 and 5.

Regarding the location of the spring, two possibilities exist (fig. 7) :
- either at the beam-to-column physical interface (points A), or
- at the intersection of both beam and column axes (points 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, which consists in the numerical simulation of the behaviour up 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.

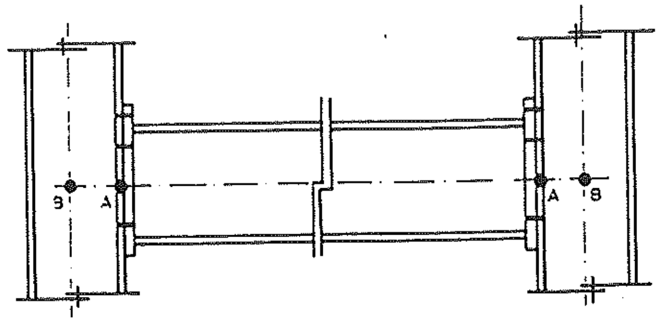
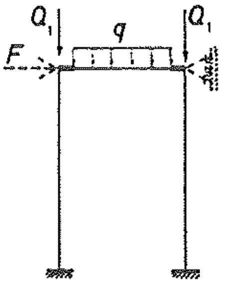
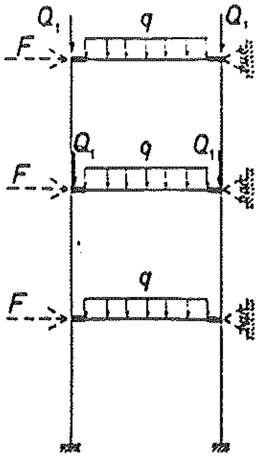


Figure 7 - Possible locations for the springs

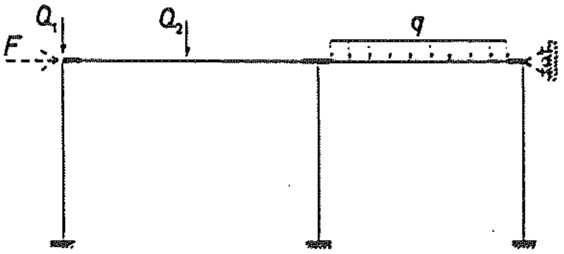
The main characteristics of the frames used for the parametric study are presented in Table 1. Geometry and loading pattern are reported in figure 8. Two types of connections commonly used in practice are considered : flange cleated and end-plate connections. Their non-linear deformability characteristics are given in (Ref. 2). All the connections of a specified frame are presumed identical. The residual stresses, the elasto-plastic behaviour of steel and the initial deformed shape of the frames are accounted for in the numerical simulations.



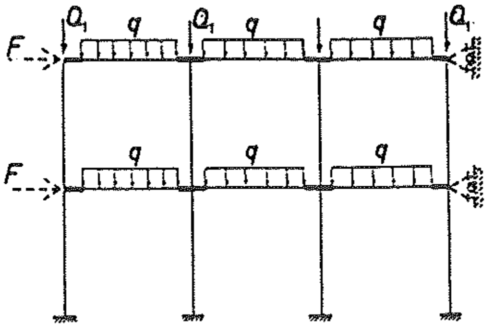


a - Type A frame

b - Type C frame



c - Type B frame



d - Type D frame

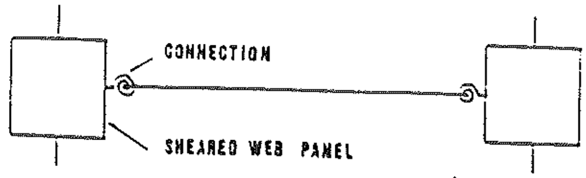
Figure 8 - Different types of braced and unbraced frames

Types of frames (figure 8)		Columns		Beam(s)		Nomenclature	Type of connections FC-flange cleated EP-end-plate	Loading (figure 8)				
		Type	Height between beam axes (m)	Type	Span between column axes (m)			q (kN/m)	P_1 (kN)	P_2 (kN)	F (kN)	Denomination
A	braced	HE160B	8,0	IPE200	5,0	A1	FC	8,242	397	-	-	AL.1
						A2	FC	10,303	0	-	-	AL.2
	unbraced	HE160B	6,0	IPE200	5,0	A3	FC	12,363	108	-	6	AL.3
						A4	EP	12,363	108	-	6	AL.3
B	braced	HE160B	7,0	IPE300	left beam: 10,0	B1	FC	8,863	450	28	-	BL.1
					right beam: 8,0	B2	FC	2,037	0	32	-	BL.2
	unbraced	HE160B	12,0	IPE300	12,0	B3	FC	4,050	36	24	3	BL.3
						B4	EP	4,050	36	24	3	BL.3
C	braced	HE140B	5,0	IPE270	6,0	C1	FC	14,305	120	-	-	CL.1
						C2	FC	18,392	0	-	-	CL.2
	unbraced	HE200B	4,0	IPE300	5,0	C3	FC	41,537	0	-	10	CL.3
						C4	EP	41,537	0	-	10	CL.3
D	braced	HE120B	5,0	IPE220	5,0	D1	FC	11,245	160	-	-	DL.1
						D2	FC	11,245	0	-	-	DL.2
	unbraced	HE160A	4,0	IPE300	5,0	D3	FC	41,178	0	-	5	DL.3
						D4	EP	41,178	0	-	5	DL.3

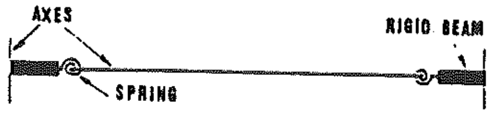
Table 1 - Data of the numerical simulations

Three numerical modellings of joints are examined :

a. the deformabilities of both connection and sheared web panel are represented separately - type M3 (because being the most sophisticated modelling, this case is used as reference).



b. the deformabilities of both connection and sheared web panel are concentrated into a single spring located at the beam-to-column physical interface - type M4.



c. the deformabilities of both connection and sheared web panel are concentrated into a single spring located at the intersection of beam and column axes - type M5.

Preliminary calculations (aimed at defining the constituents of the parametric study) have indicated that the deformability of the joints may be concentrated at the beam-to-column interface (points A in fig. 7), wherefrom model M4. In view to confirm these results, the numerical simulations relative to M3 and M4 modellings have been performed in the first instance.

The resulting curves are presented as follows :

- curves "load factor λ vs. mid-span vertical displacement Δ for the beam, the transverse displacement of which is the most important at each level of the frame loading" for braced and unbraced frames ;
- curves "load factor λ vs. horizontal displacement V at the top" for unbraced frames.

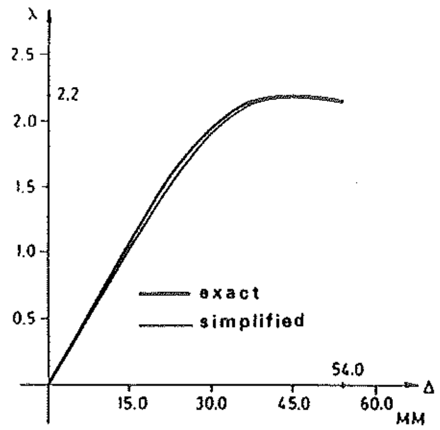
These curves allow to control the influence of the simplified numerical modelling M4 on :

- the displacement of the beams and of the frame under service loads(λ=1);
- the collapse load factor.

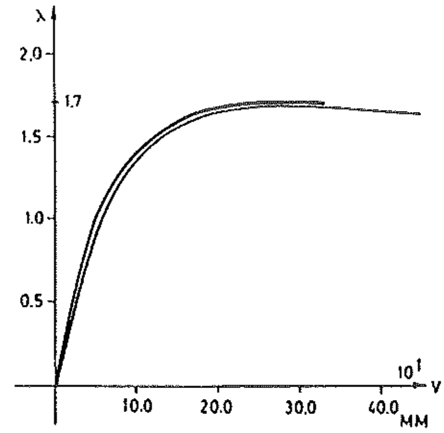
Two of these comparisons are plotted in figure 9. Others are reported in (Ref. 2).

unbraced	HE160A	4.0	IPE300	5,0	D3	FC	41,178	0	-	5	DL.3
					D4	--	41,178	0	-	5	DL.3

Table 1 - Data of the numerical simulations



a- Braced frame C
Loading CL.1



b- Unbraced frame C
End-plate connections

Figure 9 - Comparison between M3 and M4 modellings

Except for type B unbraced frame with end-plate connections, the agreement between the curves relative respectively to "exact" modelling M3 and simplified one M4 is almost perfect. The physical explanation of the underestimation of the collapse load factor resulting from the use of M4 modelling for the unbraced frame B (fig. 10) is detailed in (Ref. 2). This kind of discrepancy - anyway on the conservative side - is linked to the high stiffness and resistance of the connections in the zone M_b > 110 kNm (see fig. 11) compared to those of the corresponding sheared column web panel.

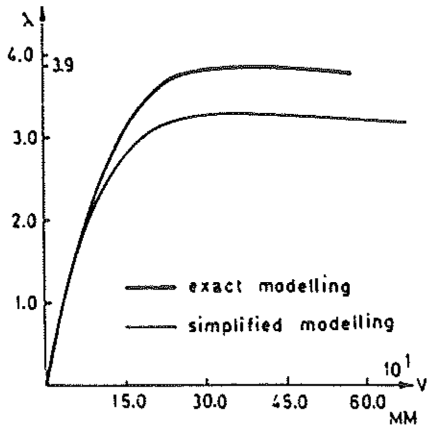


Figure 10 - Comparison relative to unbraced frame B with end-plate connections

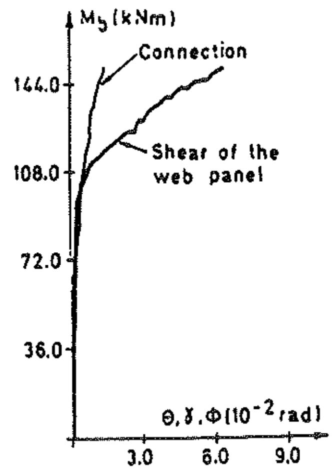


Figure 11 - Characteristics of the joint with end-plate connection relative to the unbraced frame B

In conclusion, it has been shown that the use of the simplified numerical modelling M4 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.

Numerical simulations using M5 simplified modelling are being presently in progress. The results, which should confirm the choice of M4 modelling in practice, will be presented at the end of this year in the final report of the research launched jointly by M.S.M. Department of the University of Liège and ARBED Recherches in the frame of an ECSC Research Contract (agreement N° 7210-SA/507).

Last, it is worthwhile stressing that the simplified modelling M4 is fully representative of the actual joint bevahiour 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 M4 modelling.

3.2. <u>"Bi-linearization" of the joint deformability</u> The nonlinear behaviour of the flexural spring which characterizes the joint response cannot be accounted for 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. 6.a.). This modelling has the advantage to be quite similar to that used traditionally for beam and column sections subject to bending (fig. 6.b.).

The moment corresponding to the yield plateau is the joint plastic capacity M_v (called design resistance in Eurocode 3 - Ref. 6). The constant stiffness which is usually recommended (for instance in Eurocode 3) is the secant stiffness (fig. 12). Simple methods for the evaluation of both values are suggested in (Ref. 2) for joints with end-plate and flange cleated connections.

BIJLAARD and ZOETEMEIJER assert in (Ref. 7) 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 presented here below and in which the design of braced and unbraced frames is successively considered.

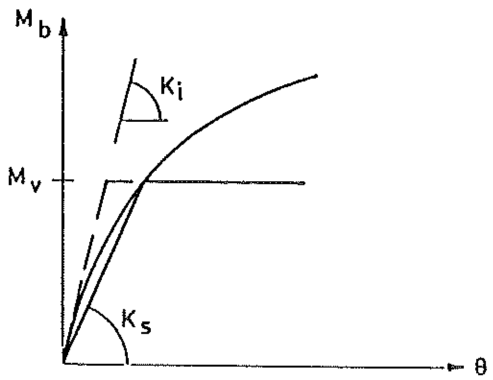
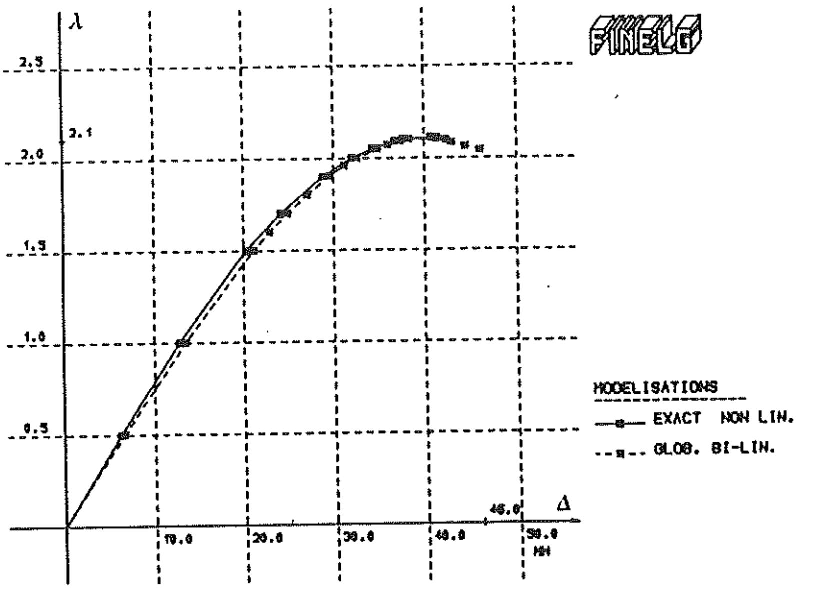


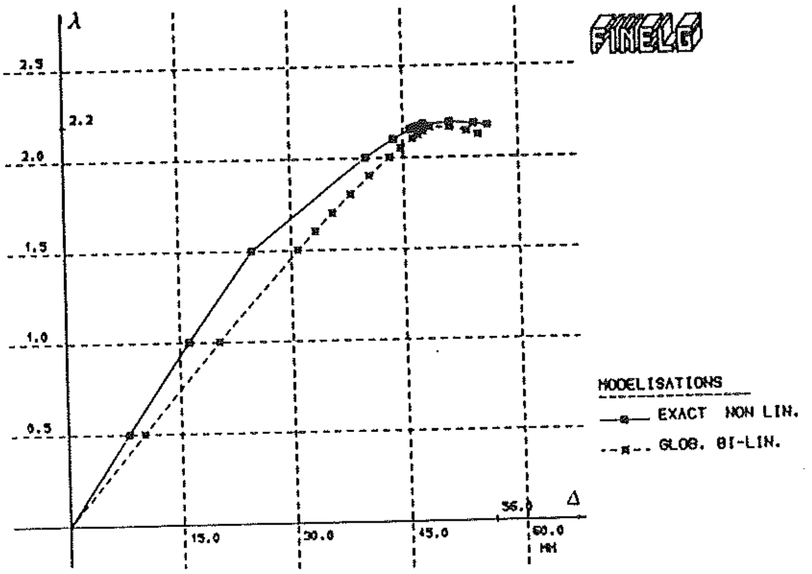
Figure 12 - Bi-linear idealization usually adopted for joints

3.2.1. Design of braced frames   Figures 13.a. to 13.f. present the λ-Δ response till collapse (see 3.1.) for six braced frames (AL.1, AL.2, BL.1, BL.2, CL.1 and DL.1 in table 1) obtained by means of the FINELG program by assuming successively that :

- the nonlinear behaviour of the connections and of the sheared column web panels is modelled separately (exact simulation);
- the deformability of the connections and of the column web panels is concentrated into isolated flexural springs acting at the beam end (beam-to-beam interface) and characterized by bi-linear laws similar to that described in figure 12 (secant stiffness K_s and plastic capacity M_v).



a - Frame AL.1 (collapse by column instability)



b - Frame BL.1 (collapse by column instability)

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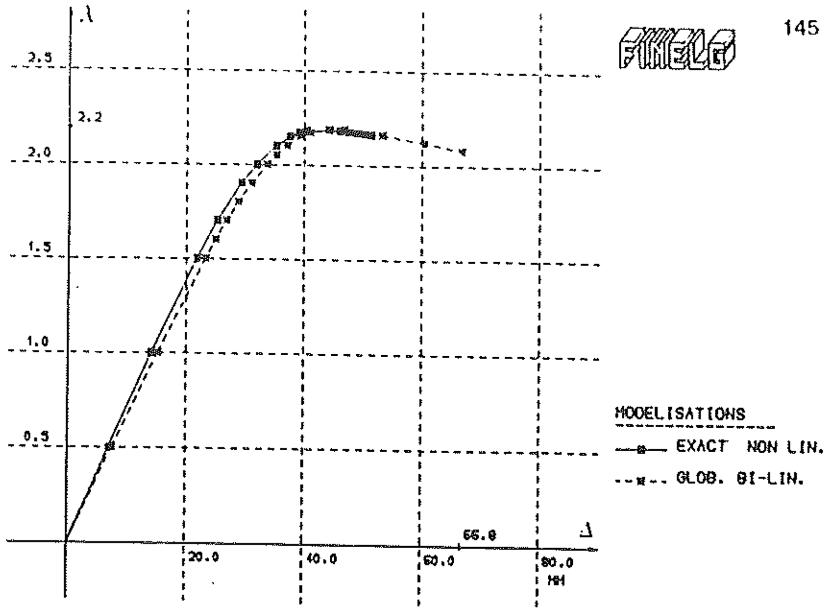
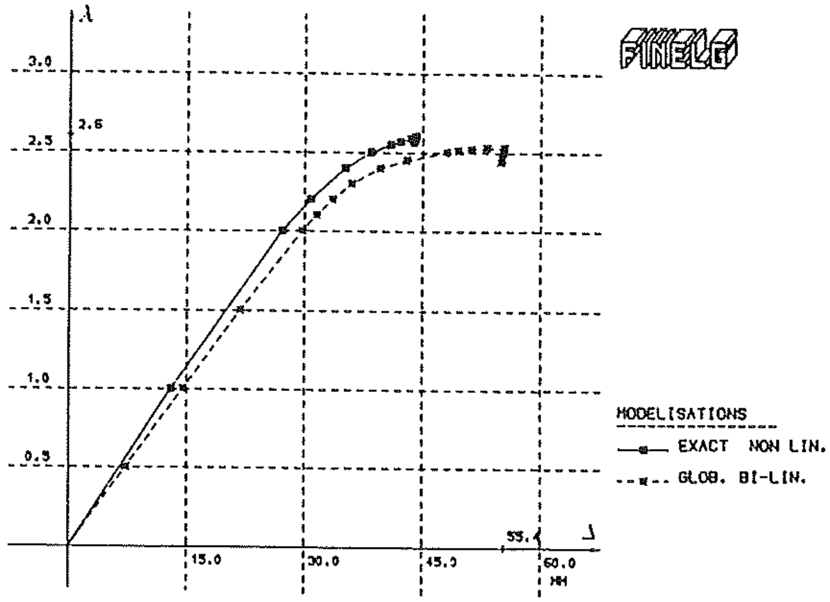
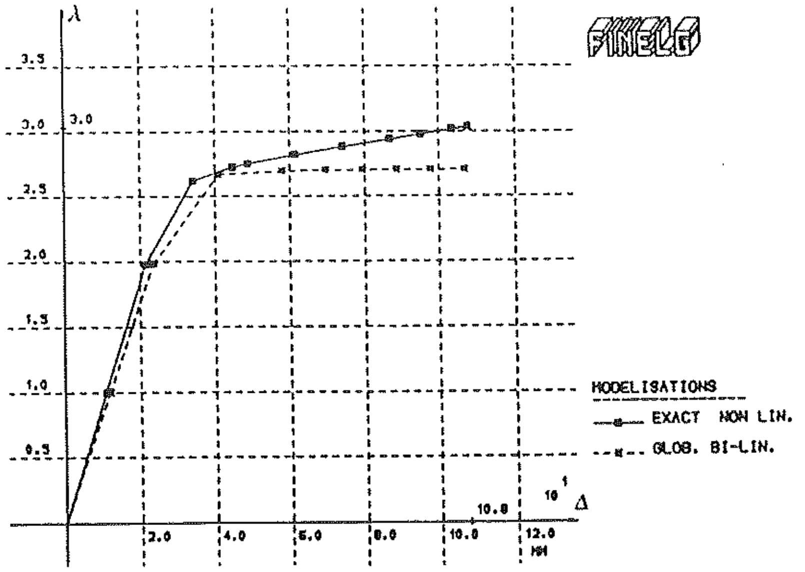


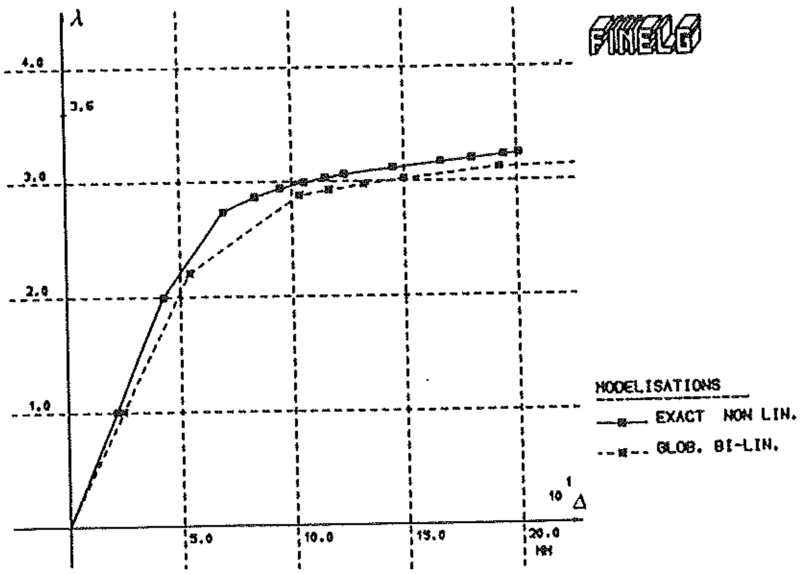
c - Frame CL.1 (collapse by column instability)



d - Frame DL.1 (collapse by column instability)



e - Frame AL.2 (plastic mechanism in the beam)



f - Frame BL.2 (plastic mechanism in a beam).

Figure 13

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 loads ($\lambda = 1.0$);
- an underestimation of the ultimate strength of the frame.

The compliance with these conditions for the six frames considered in figure 13 seems to confirm the conclusions of BIJLAARD and ZOETEMEIJER. In particular, the very good agreement between the instability loads (figures 13.a. to 13.d) 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 service loads;
- the plastic collapse load of the frames associated to the formation of a platic 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 Ref. 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. 12) is sufficient to explain the safe evaluation of the plastic collapse load (beam mechanism) at figures 13.c. and 13.f.

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. The diagrams in figure 14 allow, for instance, to point out these phenomena for the frame BL.1. All these matters are discussed in (Ref. 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 rather detrimental at collapse;
- the numerical simulations performed (Ref. 2) let believe to a compensation of the "safe" and "unsafe" restraints at the column ends.

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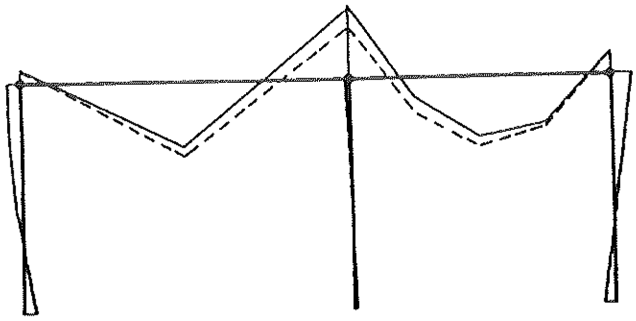
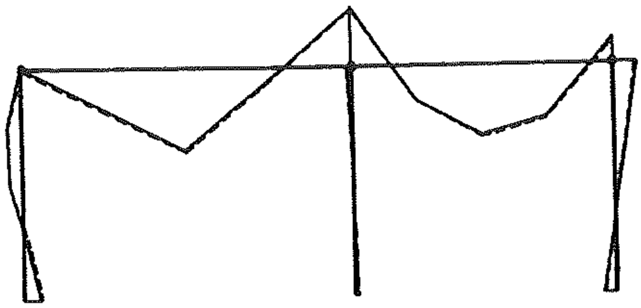
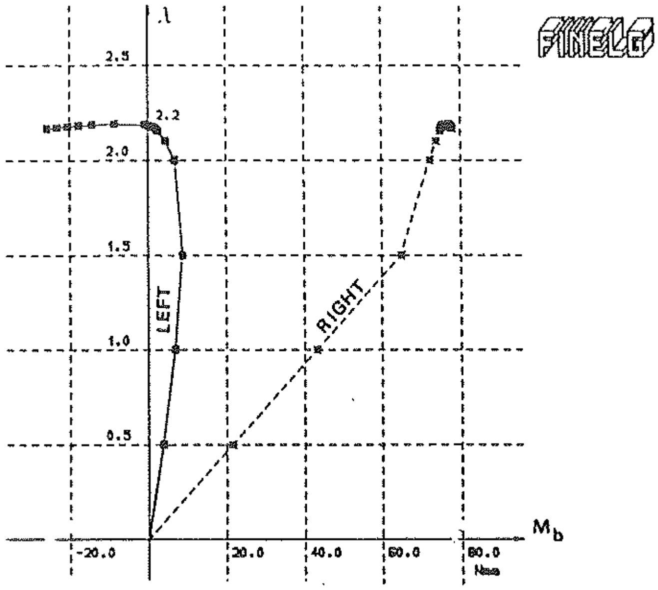


a - Diagram of bending moments under service loads ($\lambda = 1.0$)



b - Diagram of bending moments at collapse

(λ = 1.0)



c - Actual evolution of the bending moment at the ends for the left beam - Evolution of the beam restraints

Figure 14 - Influence of the concentration and of the bi-linearization on the behaviour of frame BL.1

3.2.2. <u>Design of unbraced frames</u> When loaded, an unbraced frame presents a progressive increasing horizontal deflection, usually designated by the wording "sway". It is generally acknowledged (Ref. 8) 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. 12) 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 (Ref. 2) performed by means of the FINELG program 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 recommended for economical reasons.

Preliminary studies performed in Liège (Ref. 2) indicate that it would be preferable to refer to a fictitious linear stiffness, called K_{sf} in figure 15, 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.

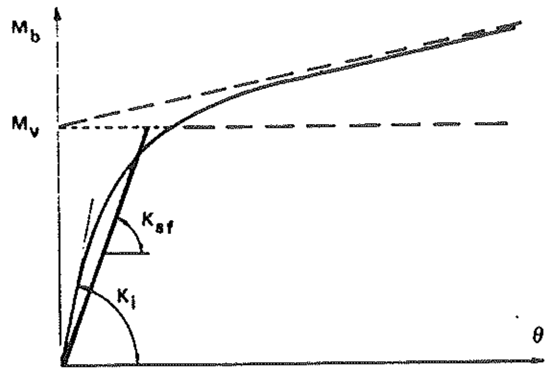


Figure 15

4. CONCLUSIONS

When the design of a building frame is not performed by means of a computer program allowing separately for the deformability of the connections and that of the sheared column web panels, the joint response has to be modelled in the most simple way as possible.

In the first part of this paper, it is shown how it is accurate to concentrate the joint deformability in single flexural springs acting at the interface between the beams and the columns. The characteristic of the springs is simply obtained by summing up the deformability of the connection and that of the sheared column web panel. This conclusion, based on a large parametric study, is valid for 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.

In view of a hand design, the non-linear characteristic of the joints has to be idealized. The bi-linear elastic-perfectly plastic idealization has the advantage to be quite similar to that commonly used for beam and column sections; in practice, it is used to refer to the secant stiffness of the joint for the elastic part and to the plastic moment (design resistance) for the yield plateau.

The parametric study, presented in the second part of this paper and devoted to braced frames :
- confirms the safe and sufficiently accurate frame response under service load and at plastic collapse;
- points out the safe estimation of the ultimate load for the frames, the failure of which is due to column instability;

resulting from the use of this model.

Lastly, the extremely safe character of the bi-linear model based on the secant stiffness for unbraced frames is shown. The substitution of the secant stiffness by a fictitious linear stiffness intermediate between the initial and secant stiffnesses is recommended;

work is presently in progress to define practically this new joint characteristic.

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