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BENDING BEHAVIOUR OF DOUBLE-C THIN WALLED BEAMS

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SUMMARY

This paper is part of a general research project devoted to the study of the cold-formed thin walled sections, with the aim to provide useful data on the use of these profiles in seismic resistant structures. One of the main goal of this project is to define a consistent numerical model able to simulate the entire moment-rotation curve.

1. INTRODUCTION

As it is well known, the bending behaviour of thin walled structural elements is strongly affected by the local buckling of the compressed part. For this reason, in most papers on this subject we can find several interpretations of this phenomenon which are sometimes very sophisticated but mainly devoted to the definition of specific limit state conditions (yielding moment, ultimate moment, maximum rotation etc.) more than to the evaluation of the complete load-deformation (moment-curvature) history.

Aim of this paper is the study of the bending behaviour of the cold-formed thin walled sections, through the analysis of the parameters which affect the shape of moment-curvature diagrams, both directly influencing the physical meaning of the loading process and indirectly the numerical model chosen to interpret it.

In particular, we have investigated on the behaviour of the double-channel cold-formed sections. The influence of each parameter has been examined on sections specially selected in order to cover different values of b/t ratios both for flanges and webs.

2. SIMULATION PROCEDURE

The analysis of the influence of the selected parameters on the bending process has been numerically performed by means of a computer program built-up for this purpouse. Starting from the ideal moment-curvature relationship for a given section, made of elastic perfectly plastic material, the program allows the introduction of the different numerical models, and the related parameters, chosen to interpret the phenomenon.

The computer program works on the section subdivided in several small areas, to which it is possible to associate a residual deformation, a specific stress-strain relationship together with all informations needed to the knowledge of load history. During the deformation process unloadings may occur due to lack of simmetry generated by local buckling.

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3. INFLUENCE PARAMETERS

To investigate the behaviour of cold-formed thin walled sections, we have considered the main phenomena characterizing their behaviour which can be interpreted by means of numerical models.

In this way, starting from the simulation of the moment-curvature relation for a given section in the basic condition (elasto-plastic stress-strain relationship without local buckling) we have examined, first of all, the phenomenon of local buckling and the different numerical models able to give a suitable interpretation both in the elastic and inelastic range, as explained later on.

The second step considers the influence of geometrical imperfections which increase the local slenderness of the section. Different stress-strain relationships are also considered in order to interpret the changes in value of the elastic limit and of the ultimate stress due to hardening effects arising in consequence of bend forming.

4. GEOMETRICAL AND MECHANICAL DATA

With the aim to better interpret the influence of single parameters we have analyzed six double-channel sections with the following geometrical characteristics:

a) 300x200x50x2 mm	lipped channel	with	b_f/t_f=95	b_w/t_w=145
b) 300x200x80x5 mm	lipped channel	with	b_f/t_f=35.6	b_w/t_w=57.8
c) 200x100x2.5 mm	channel	with	b_f/t_f=38	b_w/t_w=76
d) 200x100x5 mm	channel	with	b_f/t_f=17.8	b_w/t_w=35.6
e) 200x40x3 mm	channel	with	b_f/t_f=10.7	b_w/t_w=61.3
f) 200x40x5 mm	channel	with	b_f/t_f=5.8	b_w/t_w=35.6

and with radius of curvature r=5 mm for section *a, c*
r=8 mm for section *e*
r=11 mm for section *b, d, f*

For an easier comparison, the results have been adimensionalized by the elastic limit moment and the corresponding curvature, obtained neglecting local buckling and interpreting the behaviour of the material by a bilinear stress-strain curve with elastic limit equal to 235 N/mm^2.

For each parameter considered the results have been reported in six adimensionalized moment-curvature diagrams. In each diagram we have emphasized the base curve of the section not affected by any parameter (curve 0), which maximum moment value is related to the shape factor of each section.

5. INFLUENCE OF COMPRESSED FLANGE INSTABILITY MODEL

According to the theory of stability in the elastic range, for a slab of thickness t uniformly axially loaded on the opposite sides of width b, the critical values for stress and strain are

$$\sigma_{cr} = \frac{k\,\pi^2 E}{12\,(1-\nu^2)\,(b/t)^2} \qquad (1')$$

$$\varepsilon_{cr} = \frac{k\,\pi^2}{12\,(1-\nu^2)\,(b/t)^2} \qquad (1'')$$

with k depending upon the restrain conditions. If the slab is subjected to a load greater then the critical one and the end sections remain plane, the normal stress in a plane parallel to the loaded one is no more constant, but varies reaching the maximum value at the edges.

This phenomenon is usually interpreted considering the stress constant and equal to the maximum value for a reduced width, the so called "effective width". For a given $\sigma_{max}>\sigma_{cr}$ the effective width b_{eff} is provided imposing $\sigma=\sigma_{max}$ for a slab width b_{eff} (von Karman [1]) and then

$$\frac{b_{eff}}{t}=\sqrt{\frac{k\,\pi^2 E}{12\,(1-\nu^2)\,\sigma_{max}}} \quad (2')$$

$$\frac{b_{eff}}{t}=\sqrt{\frac{k\,\pi^2}{12\,(1-\nu^2)\,\varepsilon_{max}}} \quad (2'')$$

By the same hypothesis we can obtain the average value for stress and strain

$$\sigma_m=\sqrt{\sigma_{max}\times\sigma_{cr}} \quad (3')$$

$$\varepsilon_m=\sqrt{\varepsilon_{max}\times\varepsilon_{cr}} \quad (3'')$$

A different interpretation of the phenomenon is provided taking into account the whole section with a variable distribution for stress and strain approximatively given by

$$\sigma=\sigma_m+(\sigma_{max}-\sigma_m)\cos\frac{2\pi y}{b} \quad (4')$$

$$\varepsilon=\varepsilon_m+(\varepsilon_{max}-\varepsilon_m)\cos\frac{2\pi y}{b} \quad (4'')$$

This formulation is completely equivalent to the previous one in the elastic field, providing the same average values.

For deformation values greater then the elastic limit, we can find in literature many interpretations based on theoretical analysis and on experimental data. Among them we have examined the following models:

1) Critical value provided by (1') and post-critical behaviour interpreted by means of the effective width using (2'). Since both expressions are referred to stress, the buckling of the compressed part doesn't arise in the yielding field.
2) Critical value and post-critical behaviour based on the same concept previously related, but using the relations (1") e (2") in terms of strain instead of stress.
3) Critical value provided by (1") and post-critical behaviour analysed assuming the strain distribution given by (4") with the average value by (3").
4) Critical value of strain and post-critical behaviour interpreted by means of the effective width, using the following relation given by the authors on the base of formulation proposed by Kemp [2], Lay and Galambos [4]

$$\frac{b_{eff}}{t}=\frac{A}{\sqrt{\varepsilon+B}} \quad (5)$$

with $A=0.951\sqrt{\frac{e-1}{s-1}}$ $\quad B=\frac{e-s}{s-1}\varepsilon_y$ $\quad$ for $\varepsilon<\varepsilon_h$

$A=1.406$ $\quad B=0$ $\quad$ for $\varepsilon>\varepsilon_h$

and $e=\frac{E}{E_h}$ $\quad s=\frac{\varepsilon_h}{\varepsilon_y}$

E_h and ε_h are the values of tangent modulus and of strain at the beginning of the hardening range.

5) Critical value of strain and post-critical behaviour interpreted by means of the effective width, using the following relation given by the authors on the base of the experimental data reported in [6]

$$\frac{b_{eff}}{t} = \frac{0.49\sqrt{k}\,\varepsilon_y^{-5/18}}{(\varepsilon_{max} - 0.95\,\varepsilon_y)^{2/9}} \qquad (6)$$

In fig.1 the curves corresponding to the above mentioned models for flange buckling have been plotted. No local buckling of web is here considered. For model 1, 2, 3 and 5, k coefficient has been chosen equal to 4 for the lipped sections and 0.425 for unlipped ones. Number 0 indicates the curve obtained when local buckling is not at all considered.

We note that curves 0 and 1 are coincident for compact sections, for which yielding is achieved before the critical value. The other curves show a maximum moment plateau wider for compact sections than for the slender ones, followed by a lightly falling branch. The choice of buckling model in order to evaluate the maximum moment appears to be more relevant for the slender sections, while for the more compact ones the maximum moment is always almost equal to the plastic moment.

6. INFLUENCE OF RESTRAIN CONDITIONS OF FLANGE

Using the second buckling model of the previous section, we have analysed the influence of restrain condition of the uniformly compressed flange, in relation to the geometrical dimensions of web and flanges. We have considered 4 values of k coefficient for lipped sections and 2 values for unlipped ones, as illustrated in fig.2.

For compact sections the influence of the restrain conditions on the maximum moment value is roughly appreciable while in the decreasing branch the variation is more significant; the slender sections show an opposite behaviour.

7. INFLUENCE OF THE SLENDERNESS OF COMPRESSED FIBRES

In assessing the relations (2), (5) and (6) we can interpret the slab as composed by a set of fibers for which a slenderness linearly varing with the web distance can be assumed. It is possible to generalize this model and to consider a non linear variation of this parameter [7]. We have so selected two more models, parabolic and circular, providing a new value of effective width, b'_{eff}, related to the one obtained by the linear variation model b_{eff}

– parabolic model: $b'_{eff} = \sqrt{b_{eff} \times b}$

– circular model: $b'_{eff} = \sqrt{b_{eff} \times (2\,b - b_{eff})}$

The results plotted in fig.3, assuming the second buckling model for the compressed flange, with k=4 for lipped sections and 0.425 for the others, show an appreciable influence of models specially in the decreasing branch for compact sections and also for the maximum moment value for the slender ones.

8. INFLUENCE OF WEB BUCKLING

The local buckling of web, which is subjected to a linear strain distribution, can be interpreted, referring to the compressed zone only, in the same way as for the compressed flange. The different strain distribution requires greater values of k coefficient, shown in fig.4. The curves emphasize the influence of web buckling, which strongly reduces the moment as far as curvature increases, more relevantly and with an earlier starting point for the sections having a more slender web (cases *a, c*). At the contrary, in case of compact webs (cases *d, f*) the overall behaviour is practically not influenced by web buckling.

9. INFLUENCE OF GEOMETRICAL IMPERFECTIONS

As it is well known, geometrical imperfections play an important role on buckling behaviour of cold-formed sections. Usually we take into account this problem by substituting the previous von Karman expression with a new relation giving a reduced value of effective width [5], b'_{eff}, which influences both the critical limit value and the post-critical behaviour. We have utilized the well known expression by Winter

$$b'_{eff} = b_{eff} \times (1 - 0.22 \times b_{eff})$$

It easy to note (fig.5) how this parameter influences only the range of the low curvatures, giving a reduction in maximum moment for the sections in which buckling occurs now in the elastic field.

10. INFLUENCE OF STRESS-STRAIN RELATIONSHIP OF MATERIAL

Stress-strain relationship of steel, previously interpreted by the usual elastic perfectly plastic model, can be modified in the low strain range to interpret the non linear behaviour due, for example, to mechanical imperfections; it can be also modified in the high strain range to take into account the hardening of material.

For the first one, we can substitute the bilinear expression, up to a strain $\varepsilon_2 > \varepsilon_y$, by the curve ($a$) given by

$$\sigma = \sigma_y \left[1 - \left(1 - \frac{\varepsilon}{\varepsilon_2} \right)^{\varepsilon_2/\varepsilon_y} \right] \qquad (7)$$

Hardening is taken into account starting from the strain $\varepsilon_h = 12\ \varepsilon_y$, by a linear relation with E_h equal to tangent modulus at ε_h (b), a linear relation with E_h equal to secant modulus (c) or a non linear relation similar to the (7) with an horizontal final tangent (d).

The use of curve a gives an appreciable influence, even if limited to the range of low curvature. On the other hand the hardening influences obviously the range of the high curvature and in relation to the value of the selected modulus.

11. CONCLUSIVE REMARKS

The present analysis has emphasized the role played by the different influence parameters which have been considered in the proposed model. The numerical model of cold-formed thin walled sections is getting to be calibrated on the base of the results of an experimental program now in progress. After calibration, it will allow to perform a complete parametric analysis on this kind of sections, to get information on their bending behaviour in the whole range of deformation.

APPENDIX - REFERENCES

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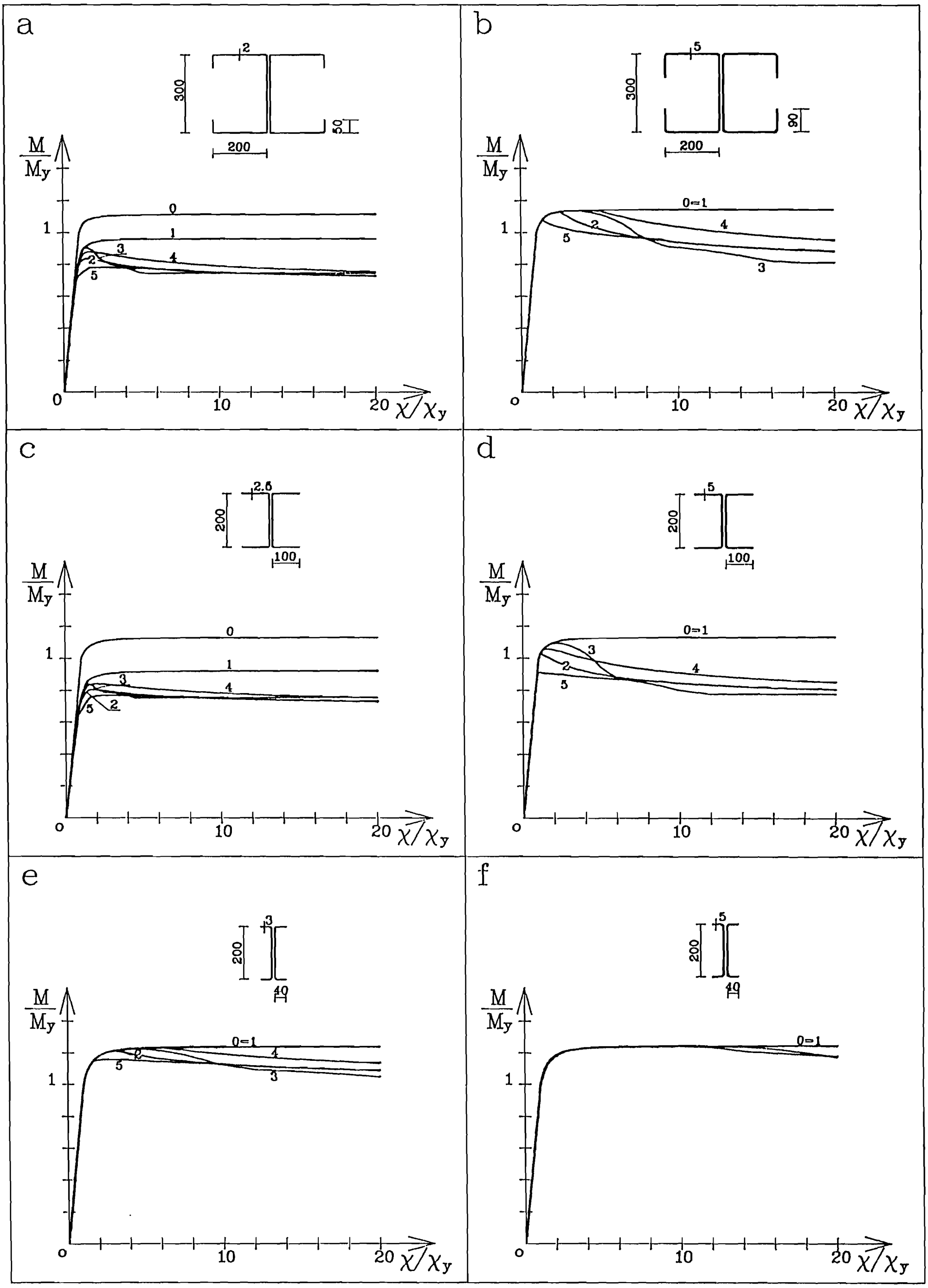

fig.1 - Influence of compressed flange instability model

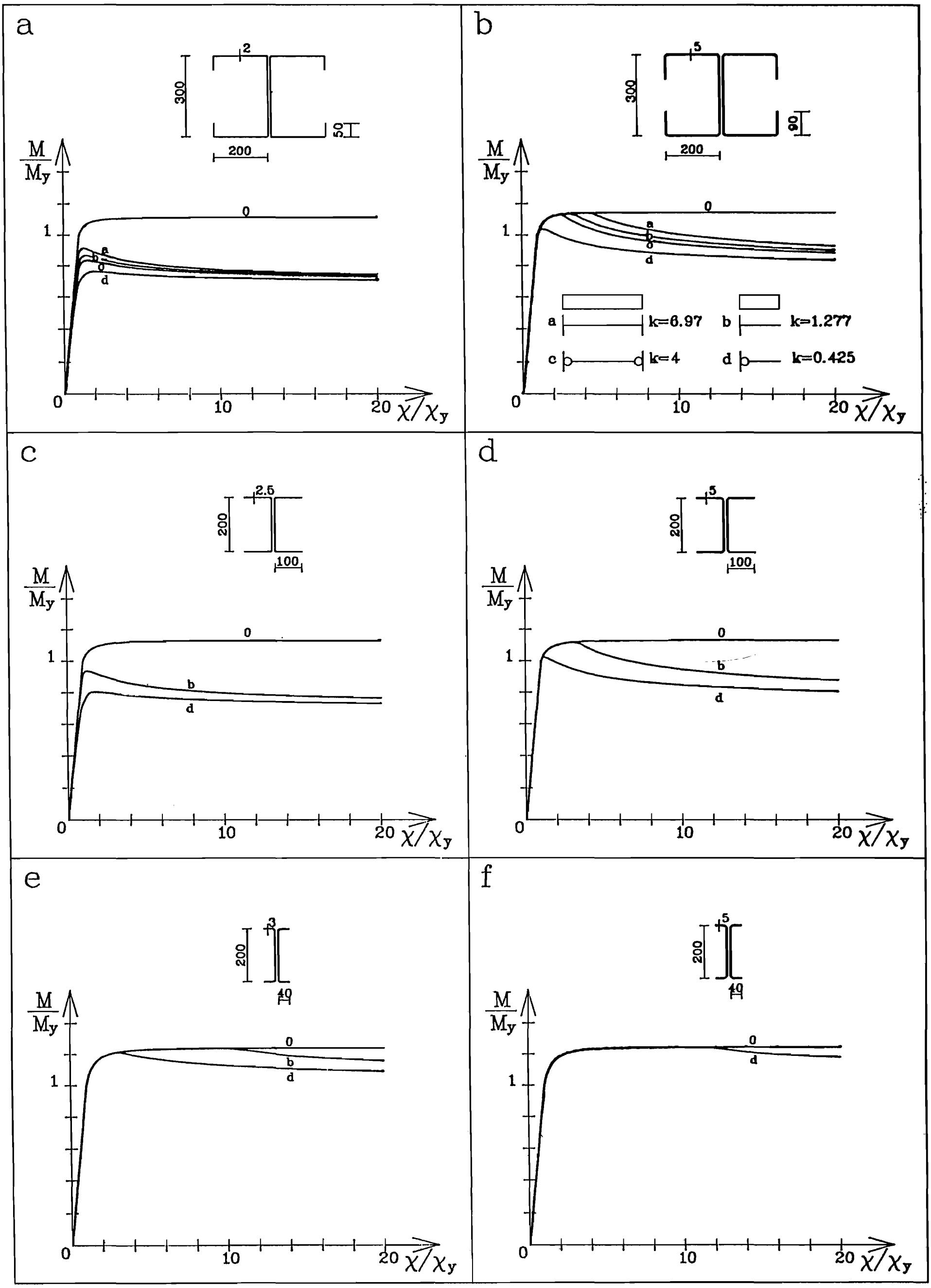

fig.2 - Influence of restrain conditions of flange

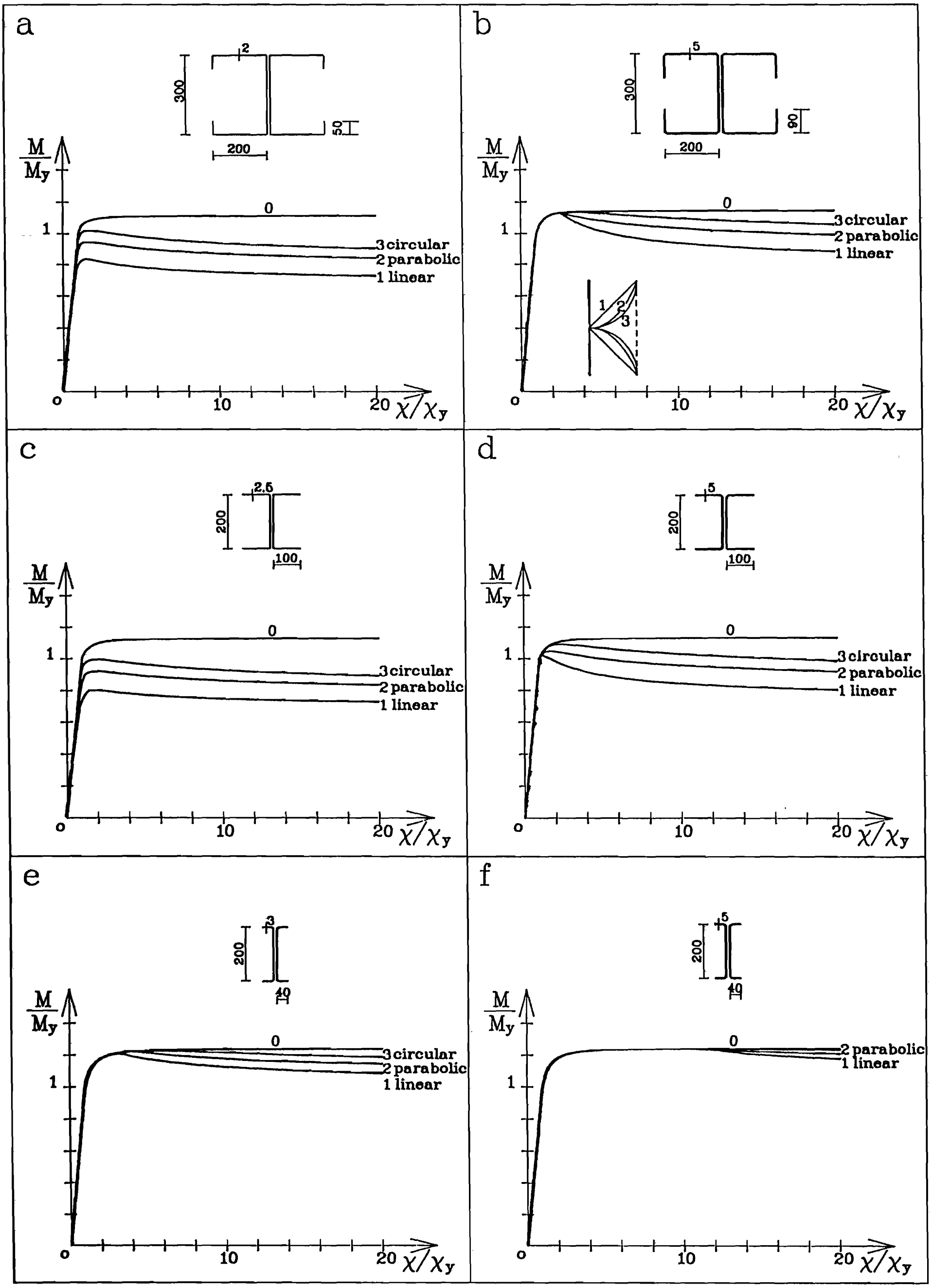

fig.3 - Influence of the slenderness of compressed fibers

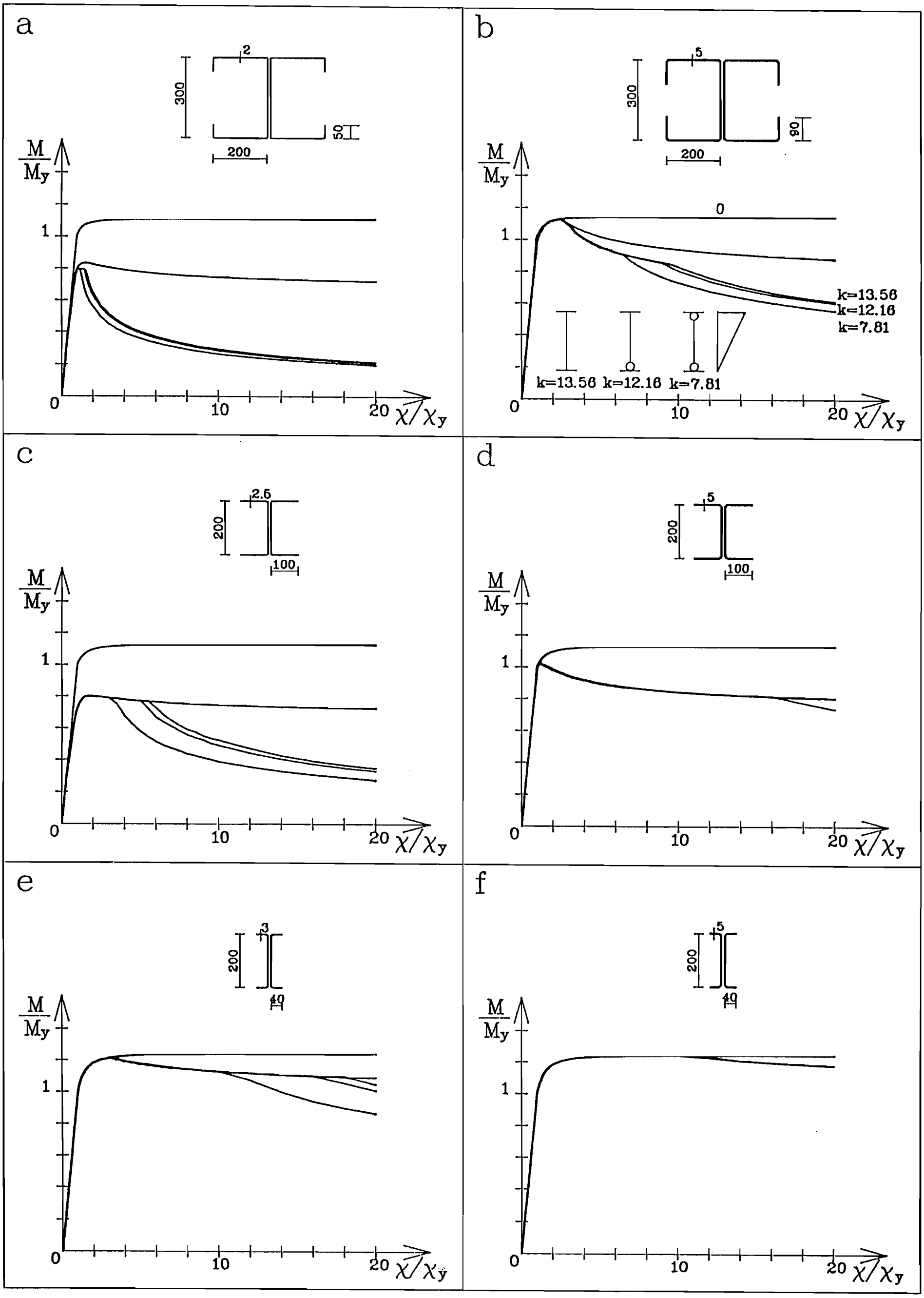

fig.4 - Influence of web buckling

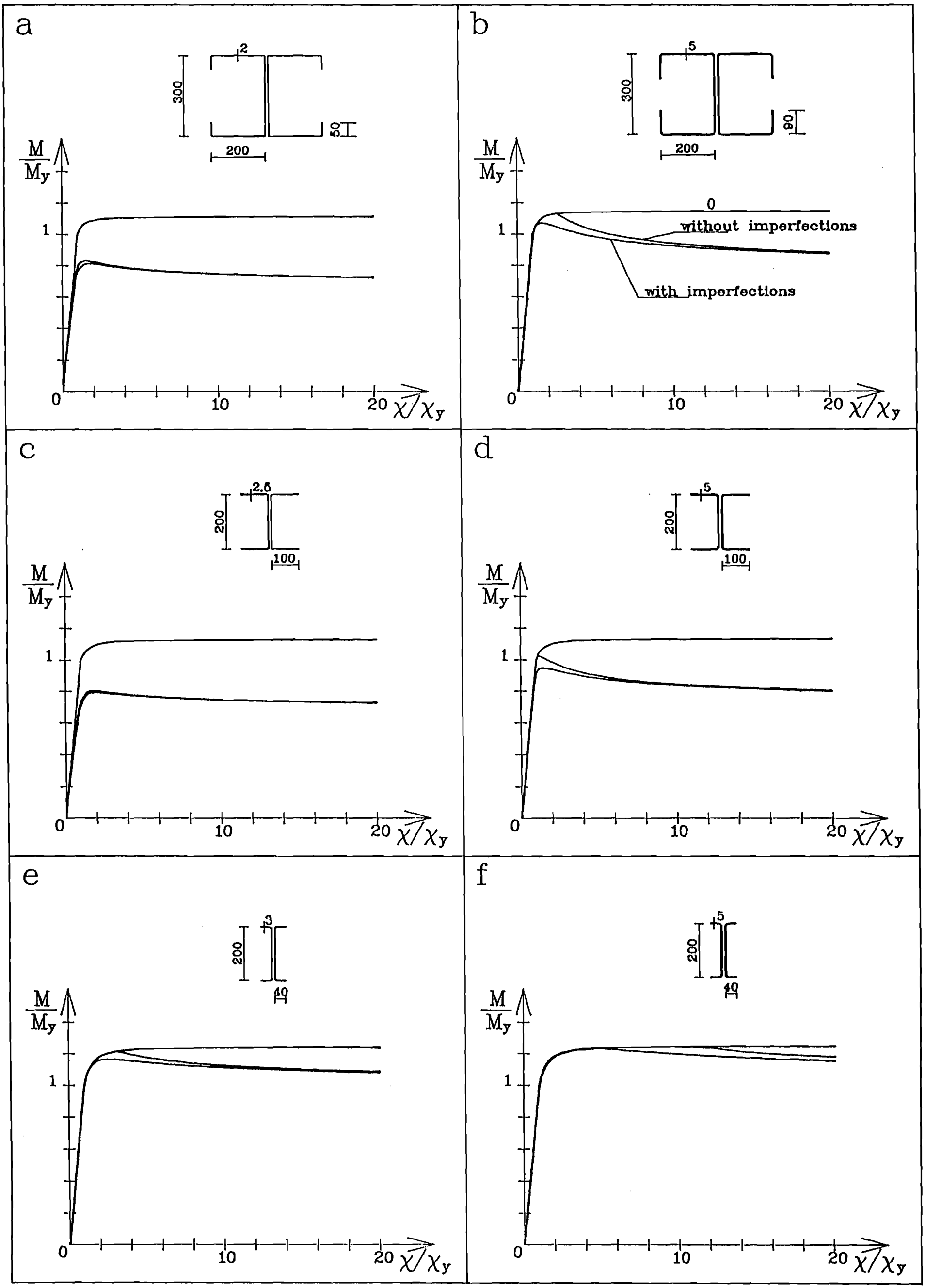

fig.5 - Influence of geometrical imperfections

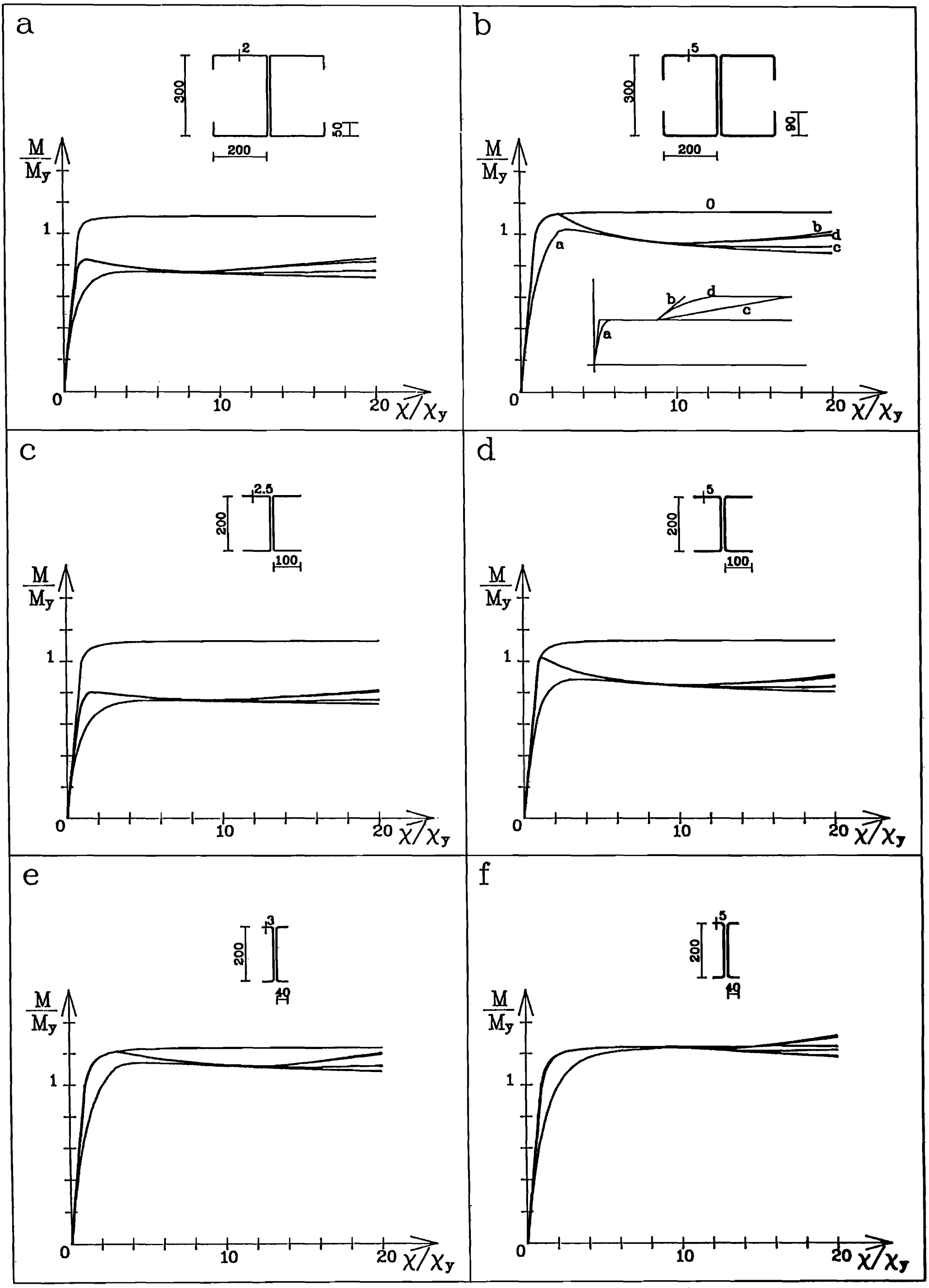

fig.6 - Influence of stress-strain relationships of material