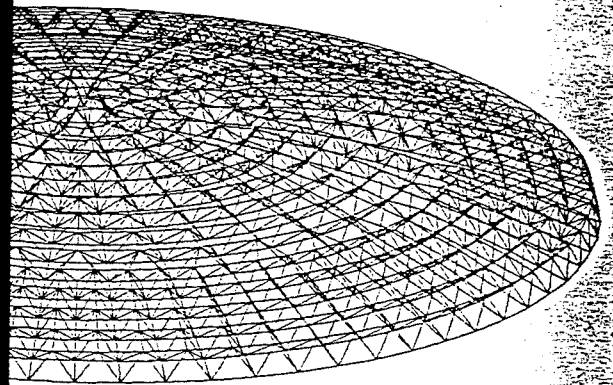**COPPE**

Graduate Center of Engineering - COPPE Federal University of Rio de Janeiro

Structural Stability Research Council - SSRC

5th International Colloquium on Structural Stability

Brazilian Session
SSRC IC/BRASIL '96
Rio de Janeiro
August 5 - 7, 1996

EEDINGS

bility Problems
Designing,
struction and
habilitation of
tal Structures"

C. Batista, Eduardo de M. Batista
hèle S. Pfeil

**Structural Stability
Research Council - SSRC**

**5th International
Colloquium on
Structural Stability**

Brazilian Session
SSRC IC/BRASIL'96
Rio de Janeiro
August 5 - 7, 1996

PROCEEDINGS

"Stability Problems in Designing, Construction and Rehabilitation of Metal Structures"

COPPE/UFRJ-Federal University of Rio de Janeiro
Structural Stability Research Center

5th International Colloquium on Structural Stability
Brazilian Session

Stability Problems in Designing, Construction and Rehabilitation of Metal Structures

Rio de Janeiro, Brazil, August 5-7, 1996

Edited by
    Batista, R.C., Ph.D., Professor COPPE/UFRJ
    Batista, E.de M., D.Sc., COPPE/UFRJ
    Pfeil, M. S., D.Sc., EE/UFRJ

Address: COPPE/UFRJ
    Programa de Engenharia Civil
    Caixa Postal 68506
    21945-970 Rio de Janeiro, RJ
    BRAZIL
Phone : (55)(21) 280-9993
            230-5315
Fax   : (55)(21) 280-9545

Papers have been accepted for printing without substantial corrections and on the authors responsability.

Cover: Reticulated space structure

This i:
on Structural
this series star
of these "Tra
Chicago, last ,
    We a:
University of
Brazilian Sess
This is the thi
various parts
Colloquium w
in Chicago, L
one, will be he
1997).
    The Br
in Designing,
contributions
Argentina, Au.
Romania, Rus
summed up to
papers were p
Connections, :
Failures, Dam.
Strength of Si
Analysis. Invi
Structures we:
Hopefully the:

FIRE RESISTANCE OF SIMPLE FRAMES ACCORDING TO EUROCODE 3.

Franssen, J. M., Schröder, L. & Dotreppe, J C.

University of Liege, Belgium.

Abstract--The recommendations presented in Eurocode 1-2 and Eurocode 3-2 which are relevant for the structural analysis of steel portal frames at elevated temperature are discussed. Comments are made on indirect actions, on the classification of the sections, on the adaptation factors for non uniform temperature distribution, on the resistance verification in members subject to bending and axial compression, and on the stability of the frame. It is proposed to use an elastic global analysis and to account for the effect of deformations by indirect methods, i.e. amplification of the sway moments. All phenomenons related to displacements, like second order effects or sway - non sway classification, should be evaluated at the critical temperature.

Keywords: Fire resistance, design, steel, frame, elevated temperature.

INTRODUCTION

One of the most commonly built structures may be the industrial one storey one bay steel portal frame. It is therefore necessary to have a simple method of analysis for the evaluation of the fire resistance of these frames, which does not rely on sophisticated non linear computer codes. This paper discusses the simple calculation model proposed in the eurocodes, highlights some of the problems and questions raised by the practical application of this method, and proposes some answers or solutions from the experience and from research studies made by the authors. This paper is limited to frames loaded and failing in their plane.

RECOMMENDATIONS OF EUROCODE 3 : PART 1.2.

The present text is a summary of the main recommendations found in Eurocode 1 - Part 2-2[1] on actions on structures and in Eurocode 3 : part 1.2[2] on structural steel design, which are related to the design of simple steel portal frames. Since 1990, when the first drafts of the fire Eurocodes have been presented to the public in Luxembourg, a lot of modifications have been introduced, leading to the situation that new drafts were frequently produced, sometimes twice a year. The present text relates to the draft dated February 1995 for Eurocode 1-2[1] and to the draft dated May 1995 for Eurocode 3-2. As these drafts have now the status of European Prestandards, it is hoped that the situation will be stabilised for a period of some years until all comments have been received to allow a final revision and the status of European Standards to be given to these documents.

A distinction is made in the eurocodes between:
* *principles*, i.e. general statements for which there is no alternative,
* *application rules*, generally recognised rules which satisfy the requirements of the principles. It is allowed to use alternative rules , provided it is shown that they accord with the relevant principles and have at least the same reliability.

Certain safety elements in these ENV have been given *boxed values*, i.e. indicative values. The authorities in each member country may substitute alternative definitive values for these safety elements for use in national application. In this paper, the boxed values have been used for these safety elements.

The *fire resistance* is defined as the ability to fulfil required functions for a specified fire exposure and for a specified period of time. Examples of required functions are load bearing function, and/or separating function. For steel members, only the load bearing function applies. There is no specific mention of deflection criteria, such as L/30, or of defection rate criteria, which mean that elements and structures may be tested or calculated until the ultimate state. A method is yet given in Eurocode 3-2[2] for applying deformation criteria to the load-bearing structure where the means of protection, or the design criteria for separating members, require such consideration. In this case a modified reduction factor is given which makes the decrease of the yield strength of steel with temperature more severe. This method is in fact an approximation because, instead of calculating the behaviour of an element with the "real" material properties and limiting the deformation owing to the presence of an insulating material, the element is calculated with modified material properties but without any control on the deformation

The *sta*
standard fir

Verifica
* in the ti
resistanc
* in the lc
value of
* in the te
value of
The last p
temperature

Fire is c
and simulta
not be cons

Indirect
considered
* may be r
* are acco
conserva
Indirect
safety requi
yet seems t
the structur

When ir
actions may
* either as
exposure
* or as a fi
from the
including

The stru
* global st
* analysis
* member

nd in Eurocode
: part 1.2^2 on
ple steel portal
des have been
ons have been
ently produced,
dated February
Eurocode $3-2^2$.
it is hoped that
il all comments
s c Iuropean

ative,
he requirements
ded it is shown
least the same

xed values, i.e.
may substitute
se in national
or these safety

functions for a
ples of required
tion. For steel
pec mention
a, which means
e ultimate state.
n criteria to the
sign criteria for
se a modified
strength of steel
approximation
with the "real"
presence of an
erial properties

The *standard fire resistance* is the ability to fulfil required functions for the standard fire exposure for a stated period of time.

Verification may be ;
- in the time domain: design value > required value, for the standard fire resistance,
- in the load domain: design value of the load bearing resistance > design value of the relevant effect of action, at the required fire resistance time,
- in the temperature domain: design value of material temperature < design value of the critical material temperature, at the fire resistance time.

The last possibility implies that the element can be characterised by one temperature, i.e. by a *uniform temperature*.

Fire is classified as an *accidental situation*, in the sense of Eurocode $1-1^3$, and simultaneous occurrence with other independent accidental situation need not be considered.

Indirect fire actions, i.e. thermal expansion causing forces, shall be considered apart from those cases where they;
- may be recognised a priori to be either negligible or favourable,
- are accounted for by conservative support and boundary conditions and/or conservatively specified fire safety requirements.

Indirect actions from adjacent members need not be considered when fire safety requirements refer to members. Rule 3.2.(2) in annex F of Eurocode $1-2^1$ yet seems to indicate that it should be the case also for the analysis of parts of the structure, and not only for the analysis of members.

When indirect fire actions need not be explicitly considered, effects of actions may be determined
- either as those existing at t = 0, applied as constant throughout the fire exposure,
- or as a fixed percentage of the design value of the relevant effects of actions from the fundamental combination at 20°C according to Eurocode $1-1^3$, including partial factors.

The structural analysis may be carried out using one of the following:
- global structural analysis,
- analysis of portions of the structure,
- member analysis.

For verifying standard fire resistance requirements, a member analysis is sufficient.

For member analysis, indirect fire actions are not considered, except those resulting from thermal gradients across the cross-section.

Indirect fire actions within sub-assembly are considered, but no time-dependent interaction with other parts of the structure.

Indirect fire actions are considered throughout the structure, in case of global structural analysis.

For members and sub-assemblies, reactions and external forces and moments at boundaries applicable at t = 0 may be assumed to remain unchanged throughout the fire exposure. As an alternative, these forces may be assumed to have a fixed percentage of their design value for the normal temperature design.

Simple calculation models allow to verify that the design effect of actions for the fire design situation are not larger than the corresponding design resistance, for a defined period of fire or until a defined temperature of the structure. Effect of actions and resistance are normal, bending and shear forces, individually or in combination.

The classification of sections for the members of a frame are made as for normal temperature design;
- for Class 3 or Class 4, without any change;
- for Class 1 or 2, using a modified value of the strain coefficient defined in section 1.6.4. of Eurocode 3-1[4] :

$$\varepsilon(\Theta) = K_\varepsilon(\Theta)\sqrt{\frac{235}{f_y}} \qquad (1)$$

where

$$K_\varepsilon(\Theta) = \sqrt{\frac{E(\Theta)}{E}\Big/\frac{f_y(\Theta)}{f_y}} \qquad (2)$$

and Θ is the temperature of the section, supposed to be uniform in this paper.

When a reduced yield strength is used for satisfying deformation criteria, a member may be classified as for normal temperature design.

The des
and axial c
direct trans
Eurocode 3

$$\frac{N}{\frac{\chi}{1.2}\,A\,f_y(\Theta)}$$

where N, N
$\quad\chi$
$\quad\chi$
$\quad A$
$\quad k$

$\quad k$
$\quad W$
$\quad W$
$\quad\chi_{LT}$

Lateral-tors
the non-dim
exceed 0.4 ,

Apart from
of the secti
temperature
instability co
1. Buckling
   axis of bu
2. The non-

$\quad\overline{\lambda}(\Theta)$ =
and
$\quad\overline{\lambda}_{LT}(\Theta)$

where

er analysis is

except those

)ut no time-

e, in case of

forces and
d t⌐ ⌐emain
orce⌐ ⌐nay be
⌐ the normal

⌐ct of actions
nding design
rature of the
shear forces,

made as for

nt defined in

(1)

(2)

form in this

⌐n criteria, a

The design buckling resistance of a member subject to combined bending and axial compression should be verified by satisfying equation 3 which is a direct transposition of the same equation established for normal temperature in Eurocode 3-1[4].

$$\frac{N}{\frac{\chi}{1.2} A f_y(\Theta)} + \frac{k M_y}{\frac{\chi_{LT}}{1.2} W_y f_y(\Theta)} + \frac{k_z M_z}{W_z f_y(\Theta)} \le 1 \tag{3}$$

where N, M_y, M_z are the actions in case of fire,

$\chi = \chi_{min}$ if lateral-torsional buckling is not a potential failure mode,

$\chi = \chi_z$ if lateral-torsional buckling is a potential failure mode,

A is the area of the cross-section,

$k = k_y$ if lateral-torsional buckling is not a potential failure mode,

$k = k_{LT}$ if lateral-torsional buckling is a potential failure mode,

$W = W_{pl}$ for Class 1 and Class 2 cross-sections,

$W = W_{el}$ for Class 3 cross-sections,

$\chi_{LT} = 1.2$ if lateral-torsional buckling is not a potential failure mode.

Lateral-torsional buckling is admitted not to be a potential failure mode when the non-dimensional slenderness for the elevated temperature $\overline{\lambda}_{LT}(\Theta)$ does not exceed 0.4 , see equation 5.

Apart from the factor 1.2 introduced in equation 3 and from the classification of the sections, a new modification is introduced which takes the effect of temperature into account. This modification is on the evaluation of the instability coefficients, χ and χ_{LT}.
1. Buckling curve c is used, irrespective of the type of cross-section or the axis of buckling.
2. The non-dimensional slenderness is evaluated as:

$$\overline{\lambda}(\Theta) = K_\lambda(\Theta) \overline{\lambda} , \tag{4}$$
and
$$\overline{\lambda}_{LT}(\Theta) = K_\lambda(\Theta) \overline{\lambda}_{LT} , \tag{5}$$

where $K_\lambda(\Theta) = \frac{1}{K_\epsilon(\Theta)} = \sqrt{\frac{f_y(\Theta)}{f_y} \bigg/ \frac{E(\Theta)}{E}}$. $\tag{6}$

COMMENTS AND QUESTIONS ON THE RECOMMENDATIONS

There seems to be a contradiction in the member analysis between the fact that bending moments are supposed to remain unchanged during the fire, while the effects of thermal gradients in the cross-section need be considered. In hyperstatic members, those gradients are known to cause a variation of the bending moments. As the non variability of the bending moments is stated as a principle, whereas the sentence on the gradients across the section is in an application rule, we will consider the moments as constant.

The sections which are classified as Class 3 or Class 4 at 20°C keep the same class at elevated temperatures. Sections which are classified as Class 1 or Class 2 at 20°C must be reclassified at the elevated temperature taking into account equation 1. Some sections may then pass from class 2 at 20°C to class 3 at the critical temperature.

The paragraphs related to beams give resistance formula which are based either on the real non uniform temperature distribution in the beam, or on the maximum temperature. In the latter case, two adaptation factors are given:

κ_1 for non-uniform temperature across the cross-section, equal to 0.7 for beams exposed on 3 sides, with a composite or concrete slab on side four, equal to 1 in all other cases;

κ_2 for non-uniform temperature along the beam, equal to 0.85 at the supports of a statically indeterminate beam, equal to 1 in all other cases.

Nothing is said about the adaptation factors in the paragraph on members subject to bending and axial compression, which is the case for all members in a portal frame. Because the non dimensional slenderness of each element must be evaluated at elevated temperature, and because only one temperature can be introduced in equation 4 and 5, equation 3 must therefore be applied with one single temperature, the maximum temperature, as explicitly stated in the code. Is it then allowed to apply the adaptation factors as for a beam?

It is our opinion that neither κ_1 nor κ_2 should be introduced in equation 3 because this equation is for the verification of stability and:

1. it is not certain at all that it is favourable for the stability to reduce the temperature in one flange of the section. It is possible that in some cases the thermal bowing created by the thermal gradient may have an unfavourable effect on the resistance to buckling;

2. the reduction of temperature at the support may slightly reduce the buckling length, but the reduction is too localised to have a significant effect on the overall stability of the member, and furthermore it affects the zones where the lateral deflections are very small.

This way, w
members, ign
figure 1, as w
beams, point E

One of the
for the bucklin
First of all,
Let us conside
critical and be
buckling resist
tends to the pl
bending mome
reflected by th
giving a conve
base column,
happen that th
whereas the res
column. We th
to the same int
elevation of ter
be done for th
situation, poin
sections with b

TIONS

the fact that
re, while the
nsidered. In
ation of the
s stated as a
ion is in an

°C keep the
is C' is 1 or
e tak..ng into
0°C to class

:h are based
n, or on the
e given:
l to 0.7 for
on side four,

the supports

on members
members in a
ient must be
iture can be
ed with one
in the code.

eq...tion 3

· reduce the
ne cases the
infavourable

the buckling
:ffect on the
zones where

This way, we have a consistent verification process for the stability of members, ignoring adaptation factors in compression members, point A on figure 1, as well as in members subject to bending and axial compression or in beams, point B on figure 1.
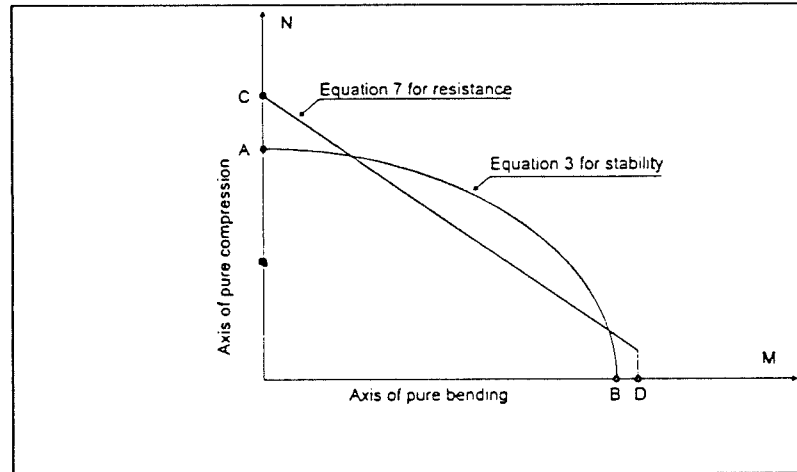
Figure 1 : stability and section formulae.

One of the main question arises from the fact that equation 3 accounts only for the buckling resistance of the members.

First of all, the resistance of cross-sections is not covered by this equation. Let us consider equation 3 in the case where lateral-torsional buckling is not critical and bending is around the strong axis. The equation tends to the buckling resistance in case of pure compression, point A on figure 1, and it tends to the plastic moment in case of pure bending, point B on figure 1. The bending moment is far from constant in the elements of a frame and this is reflected by the factor k_y in equation 3. k_y can have a value smaller than 1 giving a convex shape to the interaction curve, especially in the case of a fixed base column, where the moment distribution is bitriangular. It could then happen that the stability of the member is verified according to equation 3 whereas the resistance of the section is not verified, especially at the ends of the column. We therefore propose to add this verification of the section according to the same interaction formula as in Eurocode 3-1[4] but taking into account the elevation of temperature and the two adaptation factors, κ_1 and κ_2, as it must be done for the verification of the resistance in beams, i.e. for pure bending situation, point D on figure 1. The equation is then, for standard rolled H sections with bending around the strong axis :

$$\frac{N}{A f_y(\Theta)} + \frac{0.90 M_y}{W_y f_y(\Theta)} \leq \frac{1}{\kappa_1 \kappa_2}$$

$$\text{with} \quad \frac{M_y}{W_y f_y(\Theta)} \leq \frac{1}{\kappa_1 \kappa_2} \tag{7}$$

As the real 3D temperature distribution in the beam to column connections of steel portal frames has not been investigated in detail, we propose not to use the factor κ_2 at this location. On the other hand, the base of a fixed column is normally in contact with the concrete foundation that acts as a heat sink and cools the foot of the column. The factor κ_2 should normally be used at this location.

The other point is that the frame stability is not covered by this equation 3. Nothing is said in Eurocode 3-2[2] about the way to calculate the effects of actions in the frame. Must an elastic global analysis be performed or is a plastic global analysis allowed? Are the effects of imperfections taken into account using first order or second order theory? Is the classification between sway and non sway frames established at 20°C still valid at elevated temperatures? In case of an elastic global analysis where the second order effects are included indirectly, which one of the following alternatives should be used : amplified sway moments or sway mode buckling lengths?

One paper is probably not enough to answer all those questions. Research has been made at the University of Liege on this topic, first on sway frames[6], then on non-sway frames[7], and some partial results have been published for the case of sway frames[8]. Yet, as Eurocode 3-2[2] has been changed so often, the conclusions of the previous research studies must be re-evaluated. A new research program has been started in Belgium to evaluate Eurocode 3-2[2] and to develop a practical way to apply it which, in case of simple portal frames, leads to an acceptable safety level. The aim of these works is to propose a simple way to determine the effects of actions in a simple frame which is in accordance with Eurocode 3-1[4] while providing a satisfactory safety level. A parametrical study is done on a wide variety of frames. For each of them, a member analysis of the different elements of the frame is made by the simple calculation model and the critical temperature is compared with the one obtained from an advanced calculation model, i.e. the non linear thermo-mechanical finite element code SAFIR[5] developed in our department.

PROPOSED METHOD OF ANALYSIS

Here is a description of the method of analysis which has been adopted to calculate the effects of actions in the frame at 20°C. These actions are then supposed to be constant when the fire develops. At the present development

stage of rese
when compar

The meth
20°C which h
that compute
popular than
ambient temp
simple frame
fire, when d
histories. The
restrictions w
steel, whereas
Eurocode 3-1

It is prop
theory, i.e. ei
in braced and
the desire wa
analysis.

Following
are modified t
member, prov
Class 1 or Cla
critical tempe

The initial
account in th
horizontal for
does not exc
temperature,
neglected in
imperfections
made.

Another p
It has been
Eurocode 3-1
bracing syster
frame a react
smaller than 3
a fixed nodes
a frame calc
flexibility of t
depend on th
the investigat

ı connections of
pose not to use
fixed column is
a heat sink and
be   ˀd at this

this equation 3.
ᵃ the effects of
d or is a plastic
ᵃn into account
ween sway and
mperatures? In
ts are included
sed : amplified

ions. Research
sway frames[6] ,
ıblished for the
d so often, the
uated. A new
ᵢde 3-2[2] and to
.l frames, leads
ᵢposᵃ a simple
in   ᴵordance
A parametrical
ember analysis
:ulation model
ined from an
:hanical finite

ᵢn adopted to
ions are then
development

---

stage of research, this method proves to provide an acceptable safety level when compared to the analysis by a general calculation model.

The method of analysis to calculate the effects of actions in the frame at 20°C which has been adopted is the elastic global analysis. The first reason is that computer programs allowing an elastic global analysis are by far more popular than computer programs for the global plastic analysis. Of course, at ambient temperature, it is possible to make the global plastic analysis of a simple frame without any computer at all. This is not necessarily true in case of fire, when different members may experience different temperature-time histories. The second reason is that the plastic analysis is submitted to restrictions with regard to rotation capacity of the sections and ductility of steel, whereas the elastic global analysis may be used in all cases according to Eurocode 3-1[4].

It is proposed to determine internal forces and moments using first order theory, i.e. either neglecting the influence of the deformations of the structure, in braced and non-sway frames, or taking it into account indirectly. Here also, the desire was to be able to use the most commonly available methods of analysis.

Following the first-order elastic analysis, the calculated bending moments are modified by redistributing up to 15% of the peak calculated moment in any member, provided that all the members in which the moments are reduced have Class 1 or Class 2 cross-sections, this evaluation of the class being made at the critical temperature, see equation 1 and 2.

The initial sway imperfection with a sway angle of 1/200 is taken into account in the global analysis of the frames by means of the equivalent horizontal force[3]. It must be verified that the compression load in any member does not exceed 25% of the eulerian buckling load, evaluated at the critical temperature, to justify the fact that the member imperfections have been neglected in the global analysis. If this is not the case, then members imperfections should be introduced and second order global analysis should be made.

Another point concerns the efficiency of the bracing system in case of fire. It has been found that the criterium proposed for normal temperature in Eurocode 3-1[4] is not applicable at elevated temperature. If the flexibility of the bracing system is defined as the horizontal displacement which induces in the frame a reaction of 1 kN, then all bracing systems which have a flexibility smaller than 3 cm/kN are fully effective and allow the frame to be considered as a fixed nodes frame. Figure 2 shows the evolution of the critical temperature of a frame calculated by the general calculation method as a function of the flexibility of the bracing system[7]. It is surprising to find a value which does not depend on the geometry of the frame, but this criterion proved to work in all the investigated cases. For some frames, a higher flexibility can be accepted.
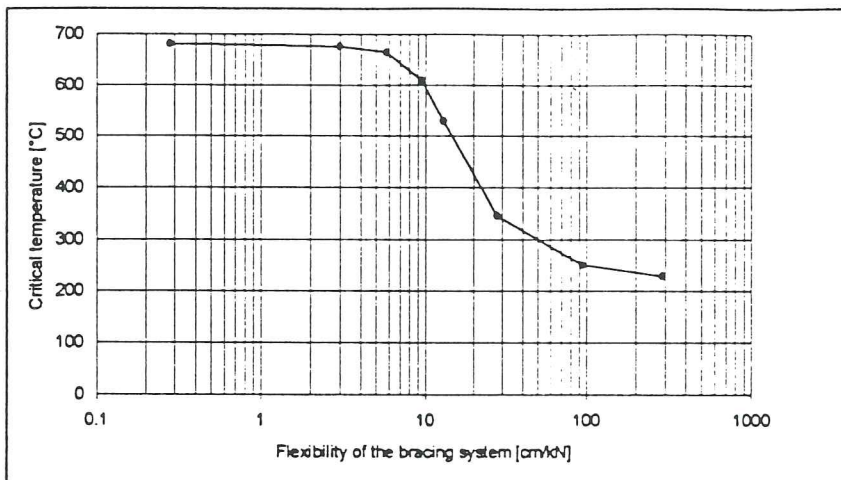
Fig. 2 : evolution of the critical temperature with the flexibility of the bracing system.

The classification between sway and non sway frames can be made according to the same criterion as in part 1-1, see equation 8, provided that the following frame flexibility factor is evaluated at the critical temperature.

$$\Gamma(\Theta) = \frac{\delta V}{h H} \leq 0.10 \qquad (8)$$

where Γ is the flexibility factor of the frame,
     δ is the horizontal displacement at the top of the frame;
     V   is the total vertical reaction;
     h   is the frame height;
     H   is the total horizontal reaction.

Figure 3 is a flow-chart which can be followed to decide which method is to be applied, see also ref.[9] . On this graph, the different solutions are, from the simplest to the most sophisticated:
1. First order analysis, fixed nodes, non-sway mode buckling length;
2. First order analysis, non fixed nodes, non-sway mode buckling length;
3. First order analysis, non fixed nodes, non-sway mode buckling length, sway moments amplified by $1/(1-\Gamma(\Theta))$;
4. First order analysis, non fixed nodes, sway mode buckling length, sway moments amplified by 1.2 in beams and beam-to-columns connections;
5. Second order analysis.
The black circles are points where the designer can choose the way, either go to the more sophisticated method or try to go to the simplest, if permitted.
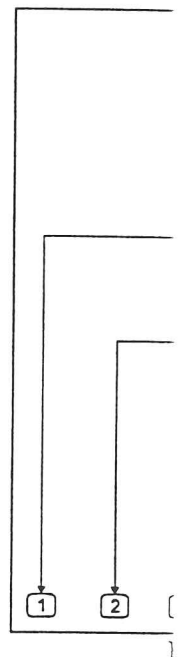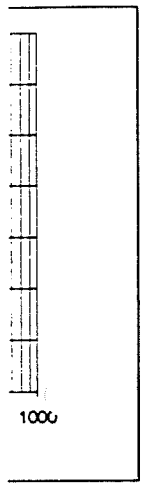
The points ...
criterium which l...

buckling length ...

procedure 4, swa...
in beams and conr...
The second meth...
depend on the cri...

ITERATIONS

There are a lot of ...
must be evaluatec ...
would like to cor ...
with the present ...
problem in the te ...
the temperature t ...
steel. This study r ...
by deformations c ...
the stiffness of st ...
might be possible ...
is to calculate the ...
its resistance at ...

1000

lity of the

an be made
ided that the
.ture.

(8)

e;

:the ' 's to be
re, ₁₁om the

h;
ength;
length, sway
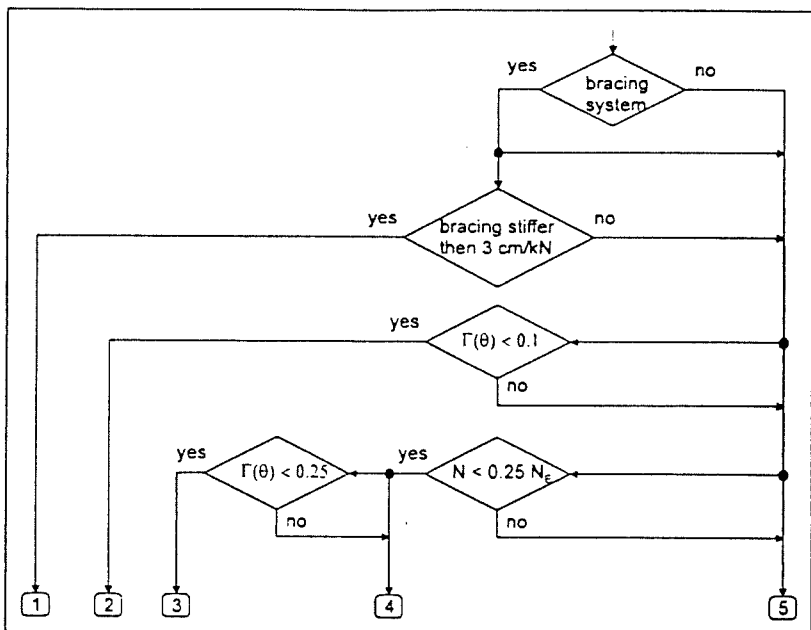
length, sway
ctions;

ay, either go
mitted.

Fig. 3 : flow-chart for the decision process.

The points which are now under investigation concern the decision criterium which leads to the choice between calculation procedure 3, non sway buckling length with sway moments amplified by $\dfrac{1}{1 - \Gamma(\Theta)}$, and calculation procedure 4; sway mode buckling length with sway moments amplified by 1.20 in beams and connections.

The second method has the advantage that the amplification factor does not depend on the critical temperature but it may be excessively severe.

ITERATIONS

There are a lot of discussions nowadays concerning the fact that the slenderness must be evaluated at the critical temperature, equation 4 and 5. Some people would like to come back to an evaluation of the slenderness at 20°C because, with the present recommendation, it is not possible to solve directly the problem in the temperature domain and 1 or 2 iterations have to be made on the temperature to solve equation 3. This is feared to deter people from using steel. This study reminds us that second order effects in sway frames are caused by deformations of the frame and that those deformations are directly linked to the stiffness of steel, which itself is strongly influenced by the temperature. It might be possible that, for this reason only, the only way to solve the problem is to calculate the frame at different temperatures and to verify its stability and its resistance at those temperatures. An iteration procedure is therefore

necessary, already at the level of the global analysis of actions, and the fact to evaluate the slenderness at the critical temperature does not add a real burden.

CONCLUSION

Eurocode 3-2[2] presents a simple calculation model which does not take explicitly into account the effects of thermal expansion. Some practical recommendations have been presented in this paper for the case of simple storey single bay rectangular portal frames, which allow an application of this simple method leading to an acceptable safety level when compared to the evaluation made using the general calculation model SAFIR developed at the University of Liege. This is true provided that, in addition to the verification of the buckling of the elements, the resistance of the sections is verified, as well as the stability of the sway frames. All quantities related to large displacements should be evaluated at the critical temperature.

REFERENCES

[1] *ENV 1991-2-2. Eurocode 1 - Basis of design and actions on structures - Part 2-2: Actions on structures - Actions on structures exposed to fire*, CEN, February 1995.

[2] *ENV 1993-1-2. Eurocode 3 - Design of steel structures - Part 1-2 General rules. Structural fire design* , CEN, May 1995.

[3] *ENV 1991-1. Eurocode 1: Basis of design and actions on structures - Part 1: Basis of design*, CEN, August 1994.

[4] *ENV 1993-1-1. Eurocode 3 - Design of steel structures - Part 1-1 General rules and rules for building*, CEN, April 1992.

[5] Franssen, J. M. et al, "A comparison between five structural fire codes applied to steel elements", Proc. of the 4th Int. Symp. on Fire Safety Science, Ottawa, IAFSS, T. Kashiwagi ed., 1125-1136, 1994.

[6] Heinen, P., "Le domaine de validité des méthodes simplifiées pour évaluer la résistance au feu des portiques en acier.", graduation work, Univ. of Liege, 1993.

[7] Schröder, L., "Applicabilité des modèles simples pour évaluer la résistance au feu des portiques contreventés en acier.", graduation work, Univ. of Liege, 1995.

[8] Heinen, P. and Franssen, J. M., "Application de l'Eurocode 3 - partie 1.1 : proposition de méthodes simplifiées pour le calcul de la résistance au feu de portiques simples non contreventés.", Construction Métallique, C.T.I.C.M., 3, 3-12, 1994.

[9] M. Braham and E. Lascrompes, "L'analyse élastique des ossatures au moyen de l'Eurocode 3", Construction Métallique, C.T.I.C.M., 4, 31-60, 1992.

STRENGTH
MEANS OF

Federico M. M

University of N

Abstract -- Thi:
of steelworks t(
rehabilitation o;
The recent dev
material both ir
common constr
also steel itself.
The flexibility
examples.

WHY STEEL'

It can be obser
to these activir
buildings, wher
of cases increas

The old ma
ravages of tim
functional rehal
concrete somet
preservation.

In these ope
structural and
execution and e
elements, which
optimum results