Zeitschrift: IABSE congress report = Rapport du congrès AIPC = IVBH Kongressbericht

Band: 10 (1976)

Artikel: Justification par le calcul du comportement au feu des structures métalliques

Autor: Barthelemy, B. / Brozzetti, J. / Kruppa, J.

DOI: https://doi.org/10.5169/seals-10525

Justification par le calcul du comportement au feu des structures métalliques

Feuerwiderstandsberechnung von Stahltragwerken

Fire Resistance Calculation of Steel Structures

B. BARTHELEMY J. BROZZETTI J. KRUPPA
C.T.I.C.M.
Paris, France

After having done in its research station a great number of fire tests sponsored by the Commission of European Committies (C.E.C.) and the European Conventional Steelwork Associations (C.E.C.M.), the French Technical Center for Steel Construction (C.T.I.C.M.) has developed a method of fire resistance calculation of steel structures.

This calculation requires the knowledge of two different temperatures, which are :

1° The critical temperature of the structure (or only a part of it) which is the steel temperature when the structure can no longer hold its function, i.e. when it collapses either by loss of strength or because of a too large deflection.

2° The steel temperature at the end of the time of stability required by official fire safety rules.

Thus, the knowledge of these two temperatures gives us an answer to the question : will the structure collapse or will it not collapse during the required time of stability ? Or, in other terms : is the critical temperature of the structure higher or lower than the steel temperature at the end of the stability required time ?

Let us explain quickly these calculations :

The critical temperature is mainly dependent upon the loading and type of structure (i.e. statically or non-statically determinate structure) and end restraint conditions. It is not the same for all kinds of structures, as many people believe : it can be 700°C as well as 300°C !

Practically, its determination is made from a flow-chart relating to the temperature the loading rate, i.e. the quotient of the normal loading by the ultimate strength of the structure which is obtained by a plastic calculation. This chart is nothing but the curve giving the decrease with temperature of yield stress of steel (fig. 1).

The calculation is a little bit more complicated when the member is thermally restraint by the rest of the structure which is not affected by fire. In this case, one must calculate the structure rigidity with respect to restraint member in order to determinate the supplementary stress induced in the member. Some charts have been drawn to make this calculation easier (fig. 2).

The heating-up behaviour of non-protected steel member is directly read on à chart drawn from a heating formula supposing a heat transfer through the steel uniform an instantaneous (fig. 3). If the member is protected by direct application of fire resistant material such as vermiculite, plaster, etc.., the protection is introduced in the calculation by a single coefficient function of temperature which makes the summary of all the thermal properties of the protection in the normal conditions of use of the product. This coefficient is determinated by an heating test of protecting members. If there is moisture in the product, the method is slightly different to take into account the water vaporisation level. We have considered that spalling does not occur and that moisture is beneficial for fire endurance.

This method is always simplificated by charts giving directly for each product the necessary thickness of protection when the critical temperature and stability time are known (fig. 4).

In summary, this method, though it is justified by complex theoretical calculations, is quite easy to use because many charts have been established. It can be used whatever kind of fire is considerated (natural fire or ISO curve), but it cannot yet solve some particular disposals such as external columns, composite structures or members shelted by suspended ceilings or partition walls. Researches are in progress in France on all these points.

This method of calculation is exposed in detail in a document named "Verification par le calcul du comportement au feu des structures en acier" [1].

References

[1] "Verification par le calcul du comportement au feu des structures en acier" Projet de recommandations établies par le CTICM, avril 1976

FIGURES

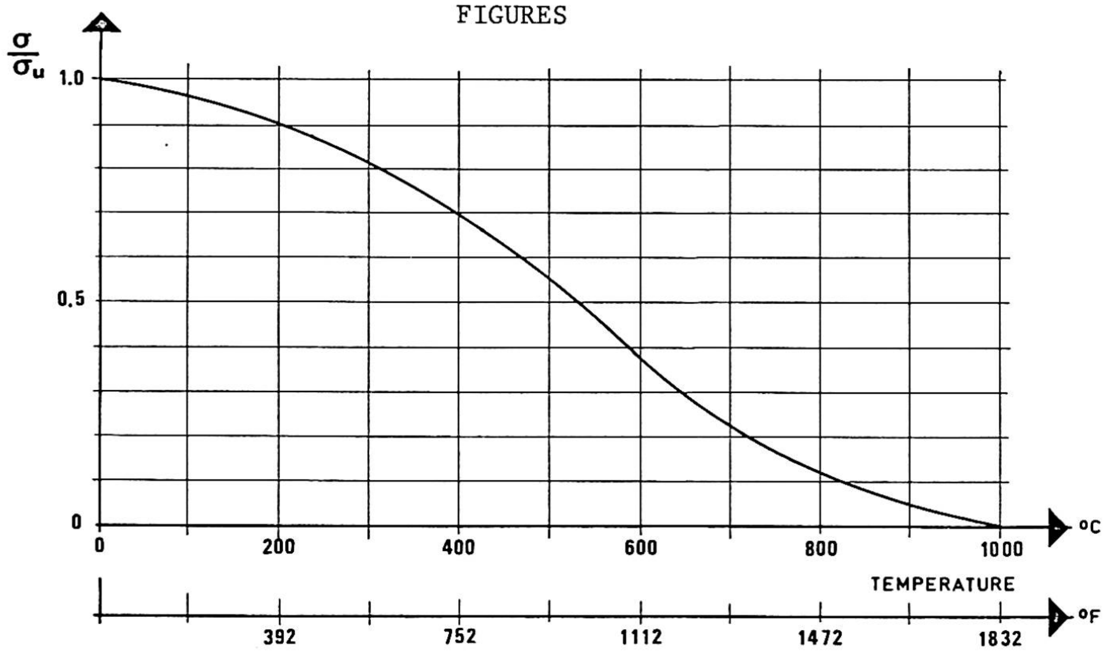


Figure 1 : Chart for calculation of critical temperature.
σ : normal loading
σ_u : ultimate strength

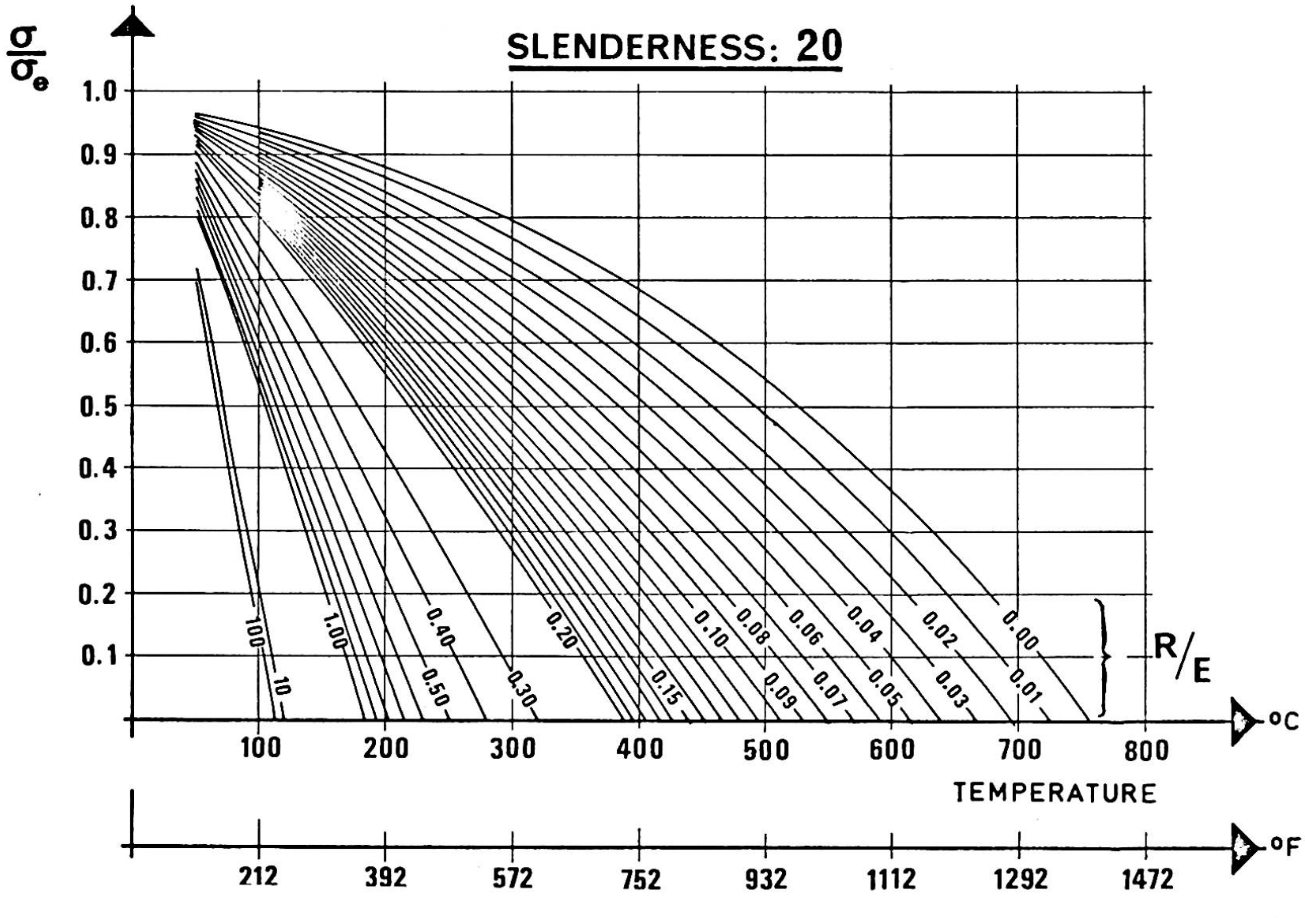


Figure 2 : Example of chart for calculation of critical temperature of a thermally restraint member.
σ_e : yield stress
R/E : structure rigidity with respect to restraint member

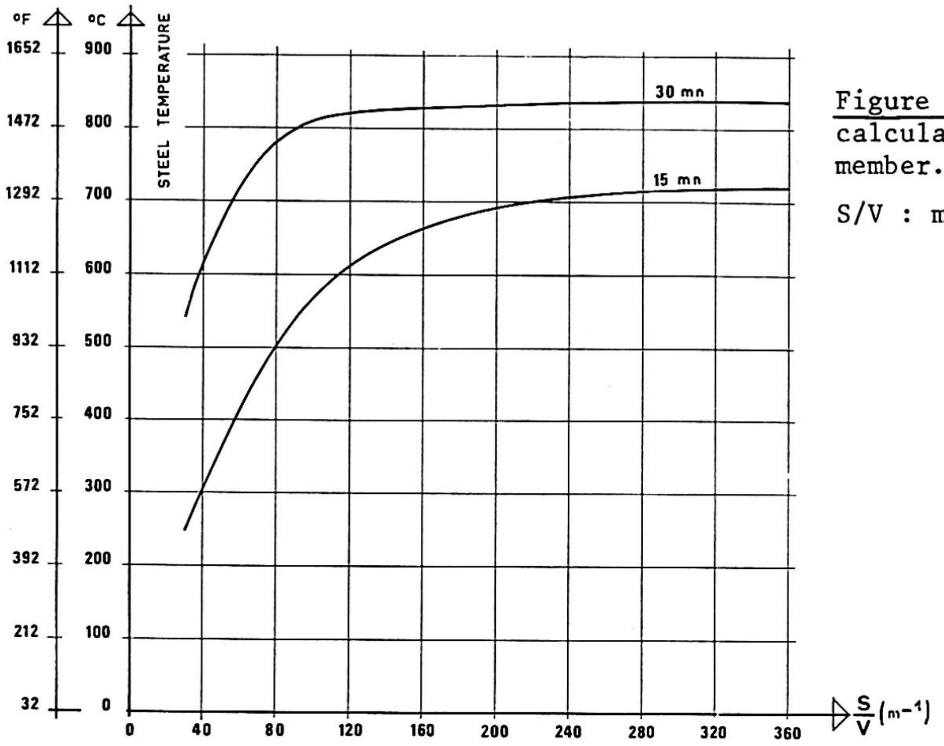
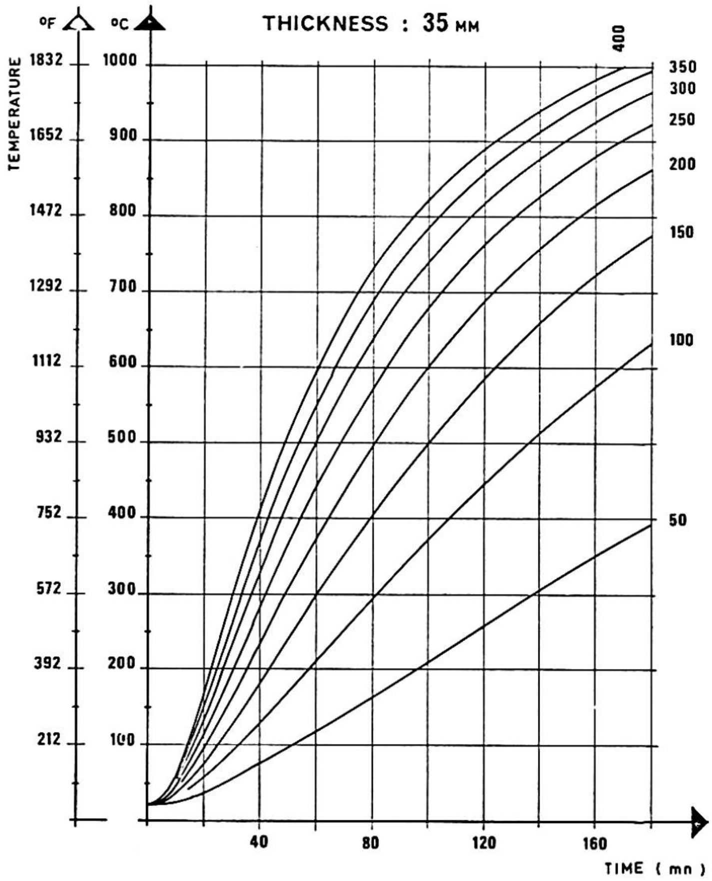


Figure 3 : Chart for heating calculation of non-protected member.

S/V : massivity of member

Figure 4 : Example of chart for calculation of thickness of a given product of protection.



Theoretical and Experimental Analysis of Steel Structures at Elevated Temperatures

Analyse théorique et expérimentale des constructions métalliques soumises à des températures élevées

Theoretische und experimentelle Untersuchung von Stahlkonstruktionen bei hohen Temperaturen

J. WITTEVEEN
Professor

L. TWILT
Research Engineer

F.S.K. BIJLAARD
Research Student

Institute TNO for Building Materials and Building Structures
Delft, Netherlands

1. INTRODUCTION

In principle the reduction in bearing capacity of steel structures under fire action can be determined on the basis of the mechanical properties at elevated temperatures. Some authors [1, 2] solve the problem by using data derived from standard material tests at various temperatures, applying conventional time independent methods of analysis. This procedure has the adventage of being ready for use to designers, but the material tests rather arbitrarely include the creep behaviour, which becomes manifest at temperatures over 400°C. Other authors like Thor [3] and Eggwertz [4] use data derived from conventional creep tests, which are fed into computer programmes,providing the time dependent creep behaviour of structural elements. This method is more reliable but is rather complicated and so far only solutions have been obtained for simple structural elements like beams [3] and columns [4]. Extending this method to more complicated structures like frames encounters practical difficulties. So far, both methods could only be checked by fragmentary experimental data.

In order to arrive at consistent and practical solutions some years ago in the Netherlands work was started combining a theoretical approach with an, in this field, new experimental technique of small scale model tests on beams, columns and frames. For the models, steel bars were used with rectangular cross-sections, with dimensions varying between 5 and 20 mm. The length of the columns and the span of beams varied between 100 and 400 mm. This makes possible a uniform temperature distribution accross and along the members, an assumption which also applies to the theoretical analysis. This contribution summarizes the results obtained so far; more detailed information is published in [5, 6].

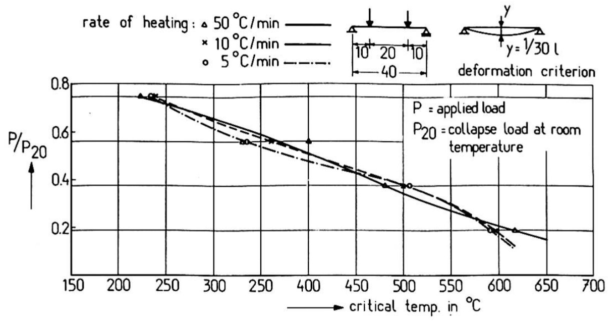
2. DEFORMATION CHARACTERISTICS OF STEEL AT ELEVATED TEMPERATURES

In case of fire in general a structural member will be under constant load and subjected to a temperature increase as a function of time. Depending on the severety of the fire and the insulation of the structural member, the rate of heating can vary. The influence of the rate of heating on the deformation behaviour was studied by analyzing the behaviour of beams and columns on model scale, with a constant load and heated with different heating rates. Linear heating rates were chosen of 50°C per minute (approximately corresponding to an unprotected steel member); 10°C per minute (normally protected steel member) and 5°C per minute (heavily protected steel member). In figure 1 the results of beams with a span of 400 mm and a cross-section of 6x6 mm^2 are summarized. On the vertical axis the applied load is plotted, as a fraction of the collapse load at room temperature, determined experimentally. On the horizontal axis the critical temperature is plotted, being the temperature at which the deflection becomes 1/30th of the span. It follows from the tests that the heating rate does not influence the deformation behaviour in a significant way. For columns similar results were obtained [5]. The results imply that the collapse temperature of steel elements can be considered as time independent, and are consequently not influenced by the heating history. This conclusion makes a theoretical approach possible, which is identical to well known methods at room temperature. Instead of one stress-strain diagram a family of diagrams for different temperatures should than be used. In those stress-strain diagrams creep can be considered as incorporated in the relationships.

To find these relationships non-conventional warm-creep tests were carried out with a loading- and heating procedure similar to the tests on beams described before. A standard cylindrical testpiece was subjected to a constant load and an increasing temperature. In figure 2 a typical result is given. It can be seen that at a certain temperature the elongation increases at constant temperature. This phenomenon is called thermal activated flow. After a certain elongation strainhardening occurs. With Harmathy's theory, slightly modified, the influence of the rate of heating could be determined theoretically. As can be seen the influence of the rate of heating is quite modest. From the warm-creep tests the stress-strain relationships can be derived with a simple transformation. It is emphasised that this transformation is only justified for "practical" heating rates (i.e. between 5 and 50°C per minute and temperatures not over, say 600°C). In addition also stress-strain relationships were obtained by analyzing the small scale beam tests. It appeared that the results obtained by these tests were in reasonable agreement with those obtained by warm-creep tests. In figure 3 a family of stress-strain relationships is presented. The phenomenon of thermal activated flow is visable in this figure by the gap between the relationships applying to 200°C and 300°C. In the subsequent discussion we will consider the family of stress-strain diagrams at elevated temperatures as a starting point for the theoretical analysis of beams, columns and frames.

3. RESULTS ON BEAMS

In figure 4 a typical result is given of a simple supported beam loaded with two point loads. As can be seen the results of the calculations are in good agreement with the tests. In this case the complete deformation process as a function of the temperature is given. In practice in many cases only the collapse temperature under a given load is of interest. A much less elaborative procedure can than be used by applying simple plastic theory. In that case only the yield stress at a given temperature derived from the stress-strain relationships has to be used. In figure 5 results are shown of tests on beams with variable end restraints. The beams are framed into columns of which buckling was prevented. By varying the plastic moments of the beams and the columns, the restraint conditions can be varied. It can be seen that the theory is in good agreement with the test results.



temperature at which a maximum midspan deflection is reached as function of the load and the rate of heating (FeE 24)

fig.1

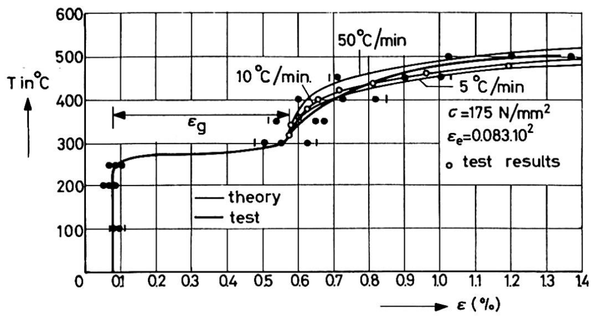
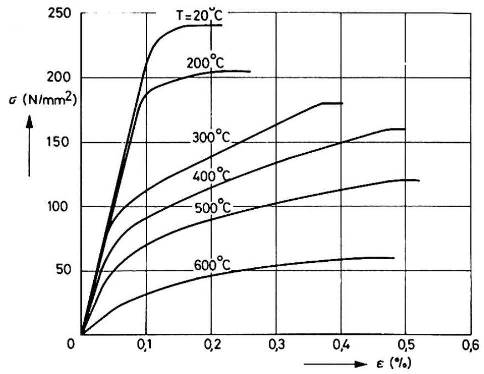


fig.2 T-ε relations for different heating rates



stress-strain relationship of FeE 24 at elevated temperatures

fig.3

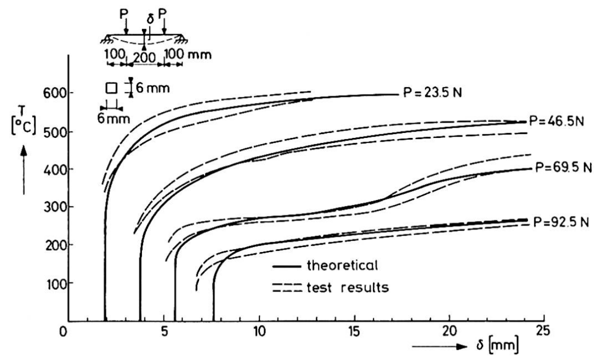


fig.4 temperature-deflection curves of beam at certain loadlevels.

4. RESULTS ON INDIVIDUAL COLUMNS

In figure 6 computated buckling curves are presented together with experimental results from small scale models. The computations are based on the assumption of an initial out-of straightness of the columns. The value of the out-of straightness is chosen in such a way that at room temperature the computated value of the load bearing capacity coincides with experimental values. In the same figure the buckling curve based on the Dutch design code is given.It is pointed out that the result established for $600^{\circ}C$ coincides with a creep buckling analyses by Eggwertz [4] of a column with a slenderness ratio of 45 subjected to a temperature-time history with a maximum of $600^{\circ}C$ (allowance is made for difference in yield strength for the respective materials).

5. RESULTS ON BRACED AND UNBRACED FRAMES

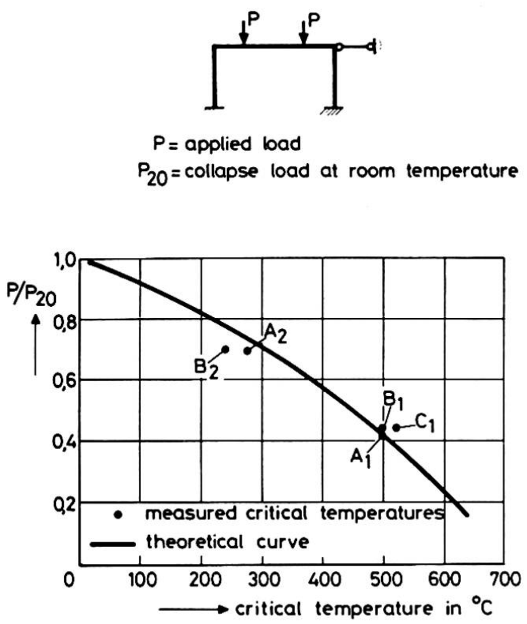
The analysis can take place in the same way as applicable to structures at room temperature, and needs no extensive discussion. As can be seen from figure 3 the deformation behaviour is non-linear, and as a consequence for the structural analysis, computer techniques must be used. Recently, however, computer programmes are available in which physical non-linearity as well as geometrical non-linearity can be taken into account. In connection with physical non-linearity there is one complication to be discussed here. The unloading characteristics of steel at elevated temperatures are not well established yet. For the time being, an elastic unloading behaviour is assumed, of which the path of unloading coincides with the path of loading. This assumption in general leads to lower bound solutions.

In figure 7 the results of theoretical and experimental analyses are summarized for braced frames. By variation of the cross-sectional dimensions two types of frames were obtained with different effective slenderness ratio's. Two different load levels P were applied, defined as a fraction of the collapse load P_{20} at room temperature. In addition loads K have been applied, chosen in such a way that the linear elastic moment at the top of the column equals one half of the moment producing the yield stress in the outmost fibres of the column. It follows from figure 7 that the theoretical predicted collapse temperatures are slightly lower than those resulting from the experiments. This trend is not surprising considering the simplifying assumption of elastic unloading characteristics of steel at elevated temperatures.

Analyzing the stability of unbraced frames it becomes apparent that the initial inclination of the frame under vertical load is an important factor. This is illustrated in figure 8 where a typical result is given of an unbraced frame with the same loading type as for the braced frames. It follows that the theoretically derived collapse temperature is substantially influenced by the mode and magnitude of the inclination. As can be expected maximum values are found if the mode of inclination is symmetric. It can be seen that the test results are conservative compared with the theoretical values. In [6] results are described of unbraced frames with additional horizontal loads.

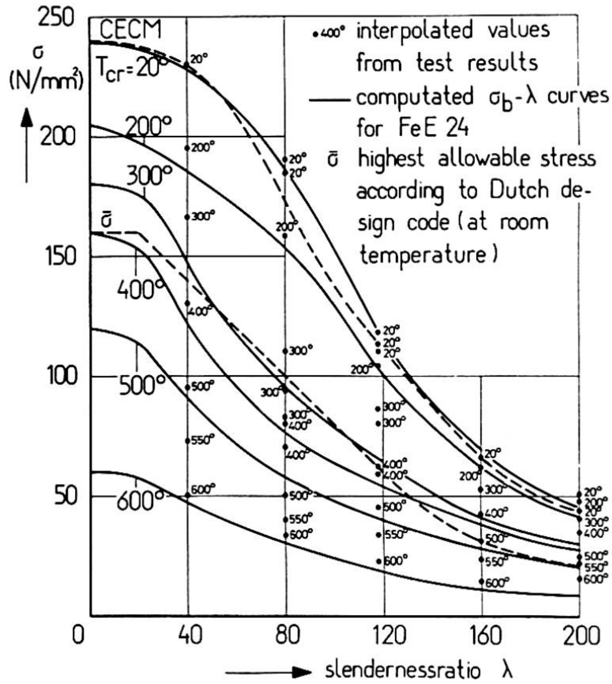
6. CONCLUSIONS

Summarizing it can be stated that the presented method, describing the behaviour of beams, columns and frames at elevated temperatures seems reliable. A basic feature is that the deformation characteristics of steel at elevated temperatures can be considered as time independent. Further activities are going on to impliment the method for real structures and make it ready for use



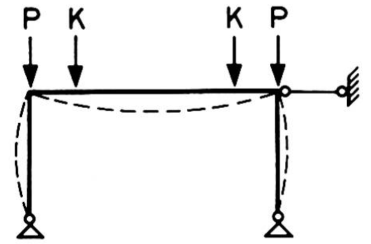
comparison between calculated and measured critical temperatures of beams with different end restraint (instability prevented)

fig. 5



relationship between critical stress and slenderness-ratio at different temperatures

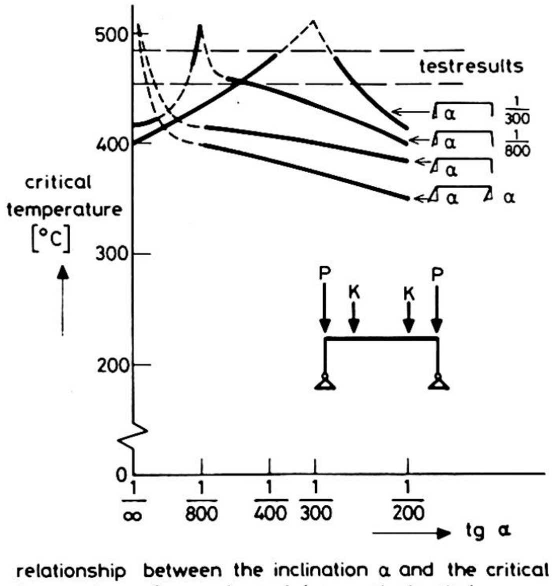
fig. 6



	$\frac{P}{P_{20}}$	T_{crit} tests	T_{crit} analysis
	0,4	525	500
	0,6	380	350
	0,4	590	515
	0,6	530	425

theoretical and test results of braced frames

fig. 7



relationship between the inclination α and the critical temperature of an unbraced frame at elevated temperatures ($P = 0{,}6\ P_{20}$)

fig. 8

to designers. The assumption of a uniform temperature distribution along and accross the members is not essential. An extension to non-uniform heating can easily be realized.

REFERENCES

1. Kawagoe, K. and H. Saito, "Thermal effects of fire in buildings", IABSE 10th Congress, Introductory Report, Zürich 1976.
2. Sfintesco, D., "Calcul et conception des structures métalliques ou mixtes en vue de leur résistance a l'incendie", IABSE 10th Congress, Introductory Report, Zürich 1975.
3. Thor, J., "Deformations and critical loads of steel beams under fire exposure conditions", National Swedish Building Research Document, 1973.
4. Eggwertz, S., "Creep buckling of steel columns at elevated temperatures", IABSE 10th Congress, Preliminary Report, Zürich 1976.
5. Witteveen, J. and L. Twilt, "Behaviour of steel columns under fire action", International Colloquium on Column Strength, Proceedings IABSE, Volume 23, Zürich 1975.
6. Witteveen, J. and L. Twilt, "The stability of braced and unbraced steel frames at elevated temperatures", Second International Colloquium on Stability of Steel Structures, Preliminary Report, 1977.

SUMMARY

In this study the deformation characteristics of steel at elevated temperatures are derived from creep tests as well as from small scale model tests on beams. As a result the load bearing capacity of steel structures under fire action can be determined with the well known methods used at room temperature. Both theoretical and experimental results of beams, columns and frames are summarized.

RESUME

Dans cette étude les caractéristiques de déformation de l'acier aux températures élevées sont déterminées par des essais de fluage et par des essais à échelle réduite sur des poutres. La résistance des constructions métalliques au feu peut ainsi être déterminée à l'aide des méthodes bien connues, utilisées aux températures ordinaires. Les résultats théoriques et expérimentaux sont présentés pour des poutres, poteaux et cadres.

ZUSAMMENFASSUNG

In dieser Untersuchung sind die Verformungseigenschaften von Stahl bei hohen Temperaturen aus Kriechversuchen und aus Modellversuchen in kleinem Massstab an Trägern ermittelt. Die Tragkraftfähigkeit von Stahlkonstruktionen unter einer Brandbeanspruchung kann mit den bekannten Verfahren bei normalen Temperaturen ermittelt werden. Theoretische und experimentelle Resultate für Träger, Stützen und Rahmen sind zusammengefasst.