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Calcul automatique de la résistance au feu des ossatures métalliques

Automatische Berechnung des Feuerwiderstandes von Stahlbauten

Automatic Computation of Fire Resistance of Steel Structures

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1. INTRODUCTION.

Dans cet article, on présente un code de calcul permettant d'évaluer, de manière automatique, la résistance au feu des ossatures métalliques protégées et non protégées.

Le comportement au feu des structures métalliques présente un certain nombre de caractéristiques qui permettent l'utilisation de techniques de calcul particulières, analogues à celles utilisées pour le calcul à la ruine des structures métalliques ordinaires. Le procédé s'apparente aux méthodes de l'analyse limite des ossatures en acier doux, basées sur le concept de rotule plastique.

Le calcul de l'évolution de la température dans les éléments se fait de manière simple en admettant que la température est uniforme sur la section droite.

2. CALCUL DE L'ELEVATION DE LA TEMPERATURE DANS LES PROFILS.

Dans le Rapport Préliminaire [5], on a indiqué la technique utilisée pour le calcul de l'évolution de la température dans les éléments soumis à l'incendie. En règle générale, il s'agit de résoudre l'équation de Fourier, assortie de conditions aux limites de différents types.

Dans le cas d'un profilé en acier, le calcul des températures peut être fortement simplifié, par suite des valeurs élevées de la conductivité thermique λ . L'hypothèse simplificatrice consiste à admettre que la température du profilé est uniforme. Cette hypothèse est plausible dans le cas des profilés ordinaires en double tê, par suite de la minceur de l'âme et des semelles et de la grande surface exposée au feu.

2.1. Cas d'un profilé non protégé.

Soit un élément de volume dV soumis à un flux de chaleur Q. Le bilan calorifique de cet élément s'écrit (cf. figure 1) :

$$Q\, dA = c\rho \frac{\partial\theta}{\partial t}\, dV \qquad (1)$$

où dA est la surface extérieure de l'élément.

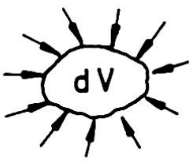

Figure 1

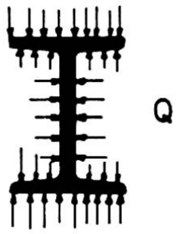

Figure 2

Le même bilan peut s'écrire en ce qui concerne un profilé métallique non protégé, puisqu'on suppose que les calories absorbées se répartissent instantanément de manière uniforme dans toute la masse de l'acier (figure 2). L'équation (1) s'écrit alors :

$$Q \frac{U}{F} = c\rho \frac{\partial\Theta}{\partial t} \qquad (2)$$

dans laquelle apparaît le facteur de massiveté $\frac{U}{F}$.

En passant aux différences finies et en explicitant la densité de flux de chaleur Q, on obtient une équation de résolution permettant de calculer pas à pas la courbe de montée en température du profil [2] [4].

2.2. Cas d'un profilé protégé.

Le calcul de la réponse thermique des structures protégées peut être effectué à différents degrés de raffinement.

La méthode la plus simple envisage un bilan thermique restreint ne concernant que l'acier, dans lequel U est la surface intérieure de l'isolant qui constitue, pour le profilé, la surface émettrice. La massiveté $\frac{U}{F}$ devient donc dépendante du type de protection.

Dans ce cas, il est nécessaire de faire un certain nombre d'hypothèses concernant la répartition de la température dans l'isolant pour calculer les échanges et les propriétés thermiques moyennes du matériau de protection. On peut alors résoudre l'équation obtenue par itérations comme précédemment.

De telles méthodes ne fournissent des résultats satisfaisants que dans le cas de produits secs et pour des épaisseurs de protection relativement faibles. Pour des problèmes où l'inertie thermique de l'isolant n'est pas négligeable, il faut s'orienter vers des méthodes plus raffinées.

Il existe certaines méthodes intermédiaires, comme celle qui consiste à écrire le bilan thermique de la manière suivante :

$$Q\,U = c_a\,\rho_a\,\frac{\partial\Theta_a}{\partial t}\,F + c_i\,\rho_i\,\frac{\partial\Theta_i}{\partial t}\,L_i e_i \qquad (3)$$

où a se rapporte à l'acier et i à l'isolant.
L_i est la longueur de la fibre moyenne de l'isolant.

Dans ce cas, on peut encore aboutir à une équation en différences finies explicites.

Il faut cependant noter que ces méthodes se heurtent généralement à la méconnaissance des valeurs suffisamment précises de la chaleur spécifique des isolants courants.

3. CALCUL AUTOMATIQUE DE LA RESISTANCE AU FEU DE L'OSSATURE.

Pour pouvoir utiliser les théories classiques de l'analyse limite pour le calcul de la résistance au feu du système, il est indispensable de conserver la notion de rotule plastique, donc d'adopter un diagramme contrainte-déformation élastique parfaitement plastique aux différentes températures (figure 3). La limite élastique R_e décroît alors avec la température suivant une loi connue.

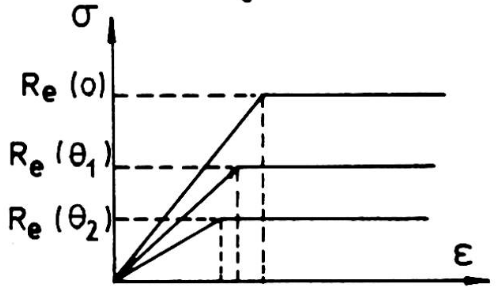

Figure 3

Il s'agit en réalité d'une approximation. Des résultats expérimentaux dus à Magnusson [7] ont en effet montré que le diagramme contrainte-déformation tend à devenir non linéaire aux hautes températures.

Une autre hypothèse concerne la répartition longitudinale de la température dans les profilés.

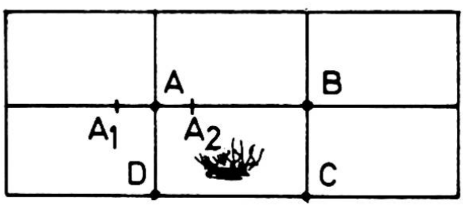

Figure 4

A la figure 4, on a représenté une maille incendiée dans une ossature (ABCD). Dans ce cas, on admet généralement que, pour chacun des éléments AB, BC, CD, DA se trouvant dans la zone incendiée, la température est uniforme d'un bout à l'autre. Aux extrémités A, B, C, D, la température varie brusquement lorsqu'on passe d'une zone incendiée à une zone non incendiée.

Les calculs que nous avons effectués montrent que la distance A_1 A_2 sur laquelle se produit le gradient thermique est généralement petite, ce qui justifie l'hypothèse admise.

Pour résoudre ce problème de manière automatique, il existe deux types de méthodes : les méthodes pas à pas et celles utilisant les techniques de la programmation mathématique. Nous avons donné la préférence aux méthodes pas à pas, qui paraissent mieux adaptées ici. En effet, le problème dépend de plusieurs paramètres ; pour le résoudre, il faut le décomposer en une succession de problèmes dépendant d'un seul paramètre.

La solution apportée est une solution du premier ordre, en ce sens que l'on n'a pas tenu compte des effets P.Δ. On a cependant prévu une vérification des colonnes au flambement.

La ruine détectée par le code de calcul correspond à l'apparition d'un des phénomènes suivants :

- formation d'un mécanisme d'ensemble ou local (mécanisme de poutre) ;
- instabilité d'une colonne comprimée et fléchie ;
- plastification par effort normal d'un élément massif.

En ce sens, le programme est entièrement automatique : il indique la formation successive des rotules plastiques, l'instant où ces rotules se forment et la durée de résistance au feu, correspondant à la formation d'un des mécanismes de ruine précités.

4. EXEMPLE D'APPLICATION.

Les considérations précédentes sont illustrées par le calcul de la résistance au feu d'un building de 3 étages. Les caractéristiques de l'ossature sont données à la figure 5.

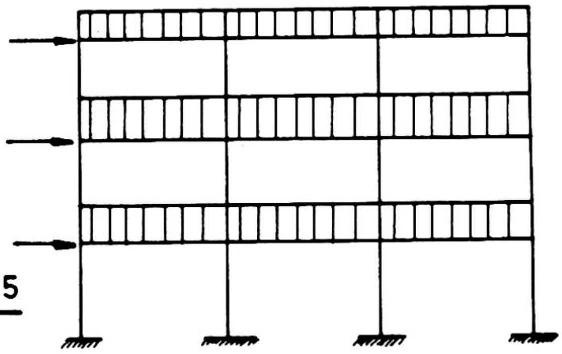

Figure 5

Protection : box - casing vermitecta 2cm

Charges :	vert.	hor.
toit	4 T/m	1 T
étages	5,8 T/m	2T

Deux cas différents d'incendies sont examinés. Dans le premier (cfr. figure 6), l'incendie se développe dans la cellule centrale inférieure. La rupture se produit par instabilité de la colonne indiquée après 1 h 23' d'incendie, alors que la température dans l'acier est de 475°. Dans le deuxième cas (cfr. figure 7), un premier foyer se déclare, puis 30 minutes plus tard, l'incendie se propage dans la cellule supérieure. Comme les éléments concernés sont beaucoup moins sollicités, la ruine n'apparaît qu'après 2 h 25', alors que la température dans l'acier est de 730°C.

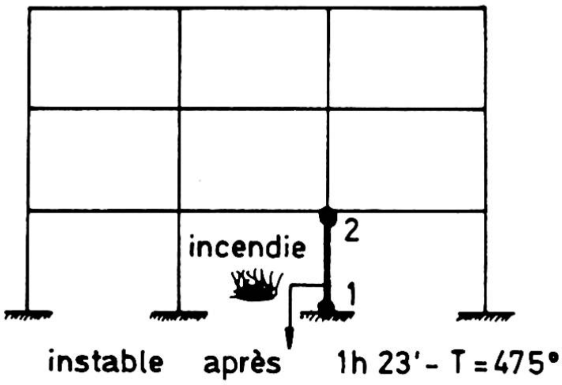

Figure 6

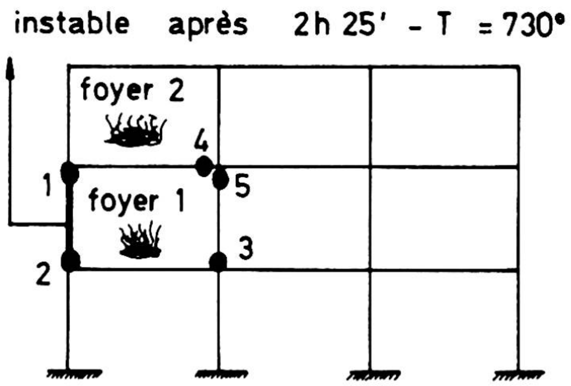

Figure 7

5. CONCLUSIONS.

Cet article fait apparaître l'intérêt des méthodes de simulation pour le calcul de la résistance au feu des structures. L'exemple précédent montre qu'on ne peut pas assimiler la résistance d'un élément essayé isolément à celle qu'il présente dans une ossature complexe. Il met en évidence l'importance de certains facteurs, comme le taux de travail des éléments et la localisation de l'incendie.

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RESUME

On présente une méthode théorique pour le calcul automatique de la résistance au feu des ossatures métalliques protégées et non protégées. Le procédé utilisé est apparenté aux méthodes de l'analyse limite des ossatures en acier doux, basées sur le concept de rotule plastique. Un exemple d'application fait apparaître que le choix d'une température critique unique n'est pas réaliste.

ZUSAMMENFASSUNG

Man stellt ein theoretisches Verfahren vor, um den Feuerwiderstand geschützter und ungeschützter Stahlbauten automatisch zu berechnen. Dieses Verfahren ist mit dem Traglastverfahren verwandt (plastische Gelenke). Ein Beispiel zeigt, dass die Wahl einer einzigen kritischen Temperatur unrealistisch ist.

SUMMARY

A theoretical method is presented for the automatic computation of the fire resistance of protected and unprotected steel structures. The procedure used is rather similar to the method of the limit analysis of steel structures, based on the concept of plastic hinge. An example shows that it is not realistic to adopt only one critical temperature.

Justification par le calcul du comportement au feu des structures métalliques

Feuerwiderstandsberechnung von Stahltragwerken

Fire Resistance Calculation of Steel Structures

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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 Conven tional 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

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FIGURES

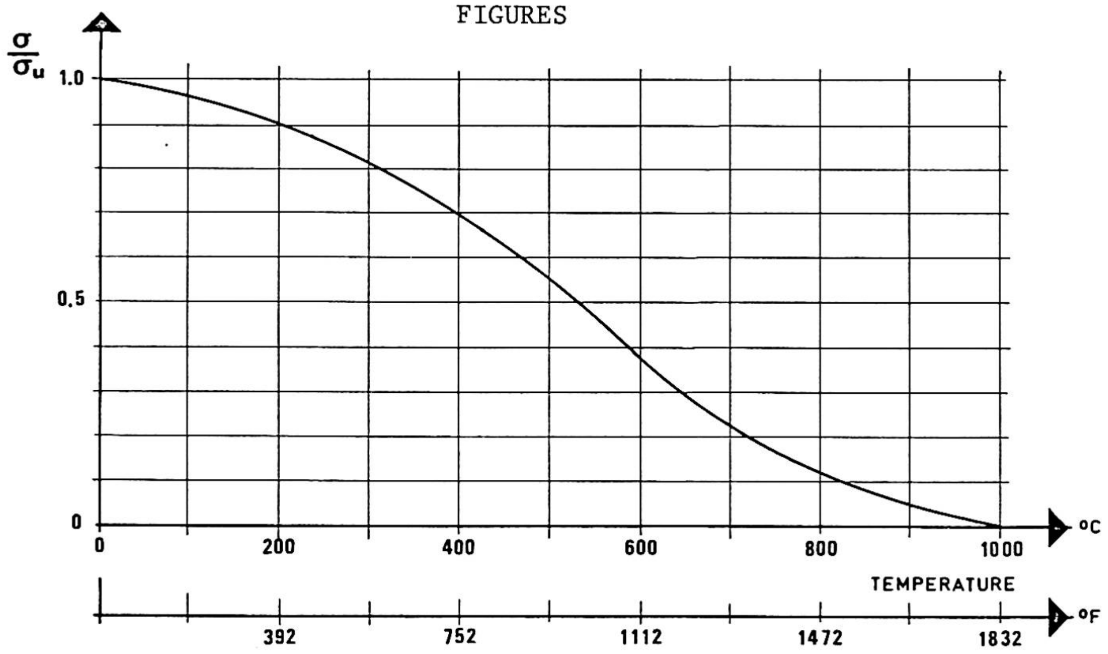


Figure 1 : Chart for calculation of critical temperature.
σ : normal loading
σ_u : ultimate strength

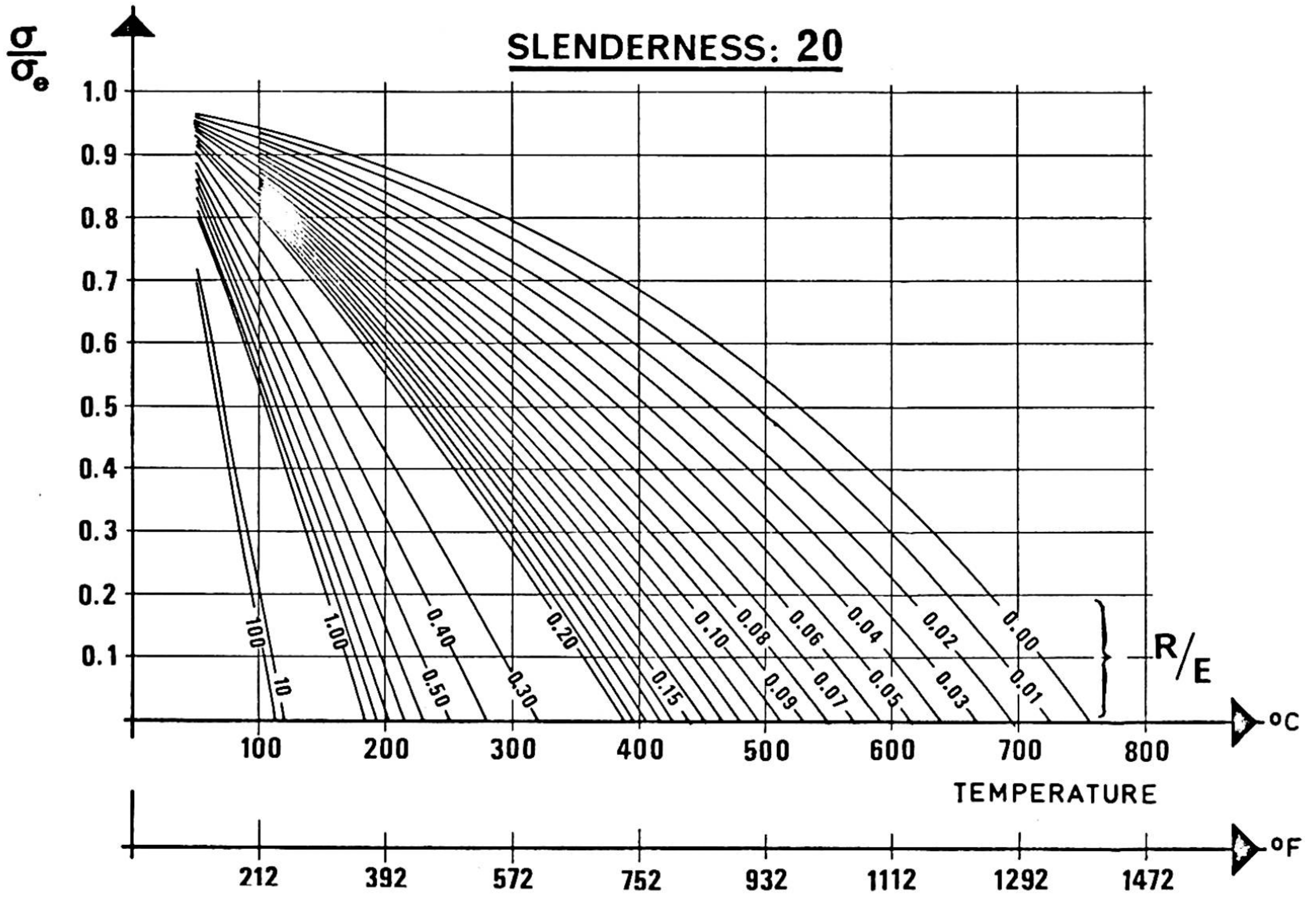


Figure 2 : Example of chart for calculation of critical temperature of a thermally restraint member.
σ_e : yield stress
R/E : structure rigidity with respect to restraint member

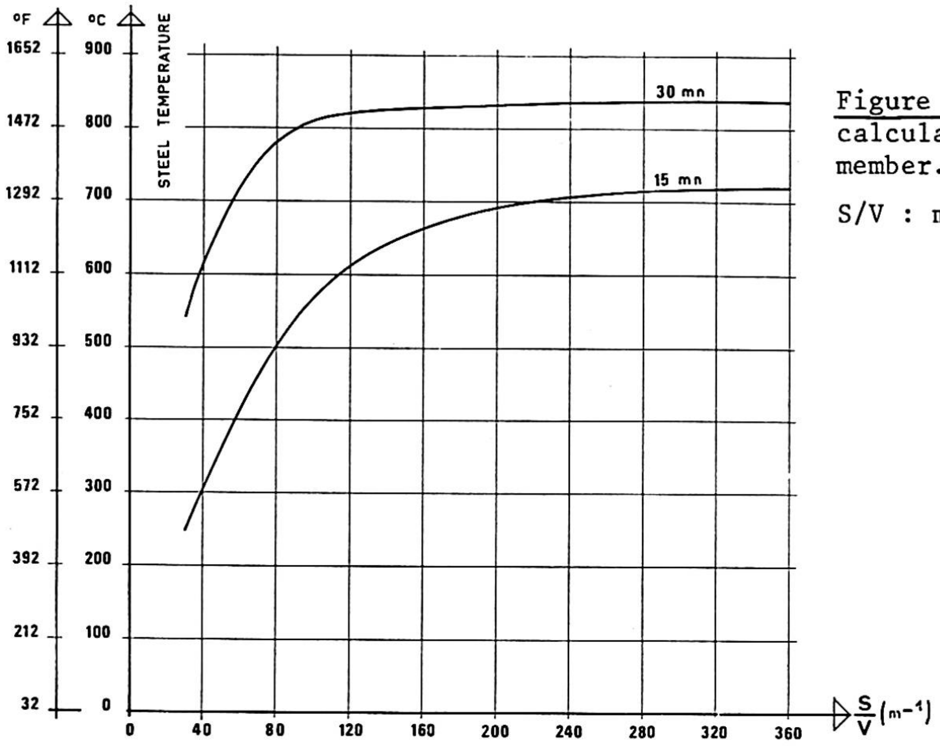
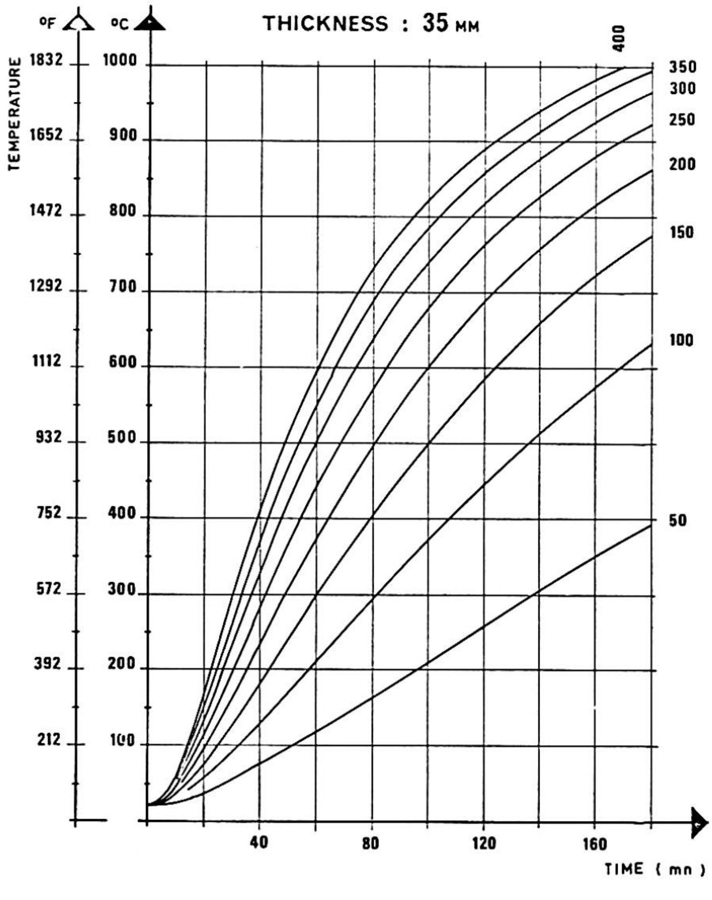


Figure 3 : Chart for heating calculation of non-protected member.

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

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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].

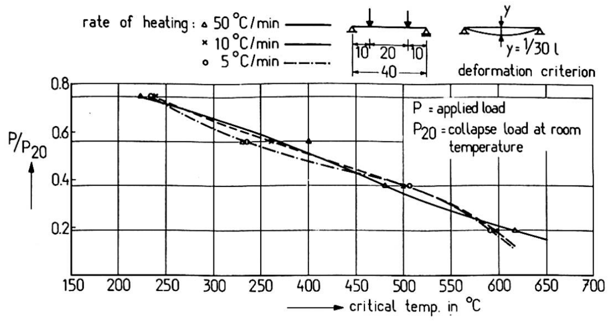
2. DEFORMATION CHARACTERISTICS OF STEEL AT ELEVATED TEMPERATURES

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

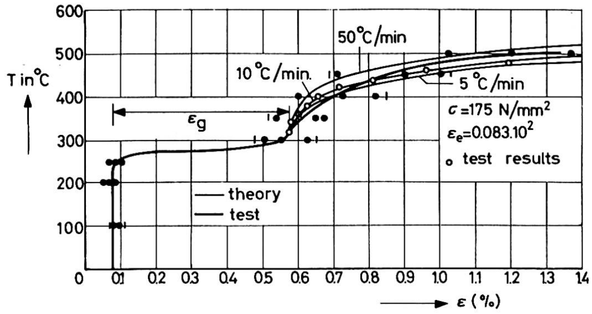
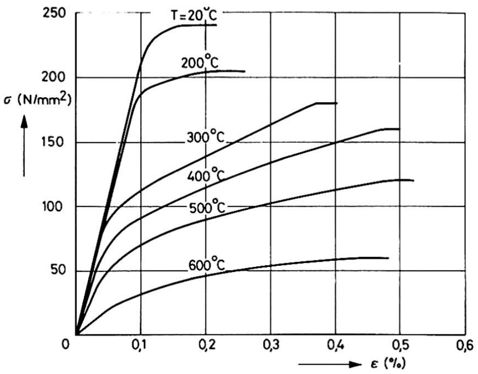


fig.2 T– ε relations for different heating rates



stress-strain relationship of FeE 24 at elevated temperatures

fig.3

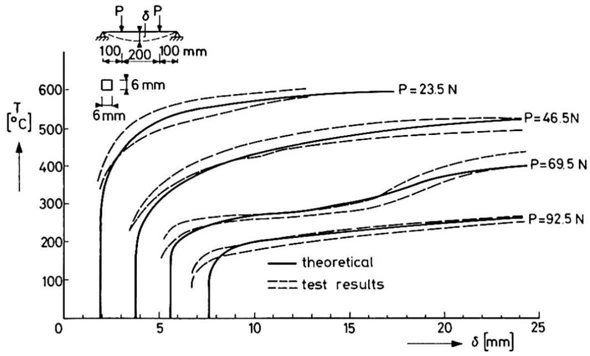


fig.4 temperature- deflection curves of beam at certain loadlevels.

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°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°C (allowance is made for difference in yield strength for the respective materials).

5. RESULTS ON BRACED AND UNBRACED FRAMES

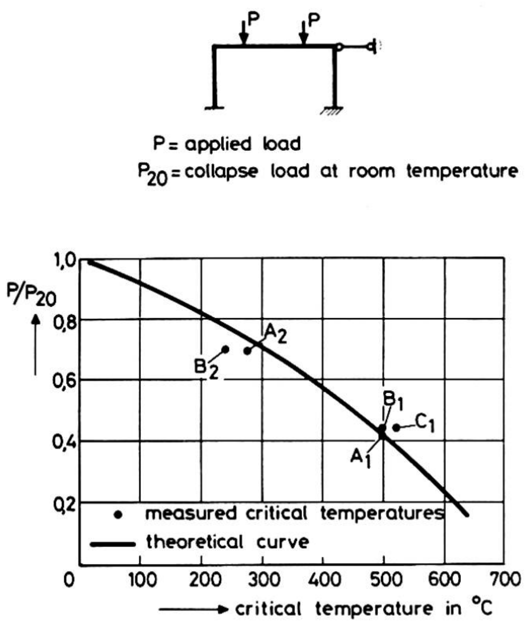
The analysis can take place in the same way as applicable to structures at room temperature, and needs no extensive discussion. As can be seen from figure 3 the deformation behaviour is non-linear, and as a consequence for the structural analysis, computer techniques must be used. Recently, however, 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.

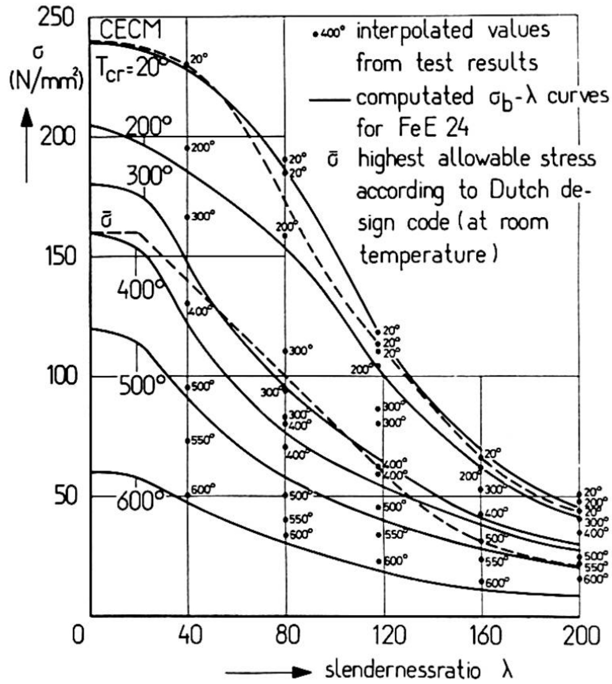
6. CONCLUSIONS

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



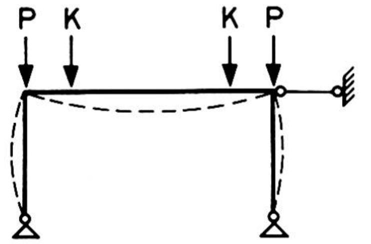
comparison between calculated and measured critical temperatures of beams with different end restraint (instability prevented)

fig. 5



relationship between critical stress and slenderness-ratio at different temperatures

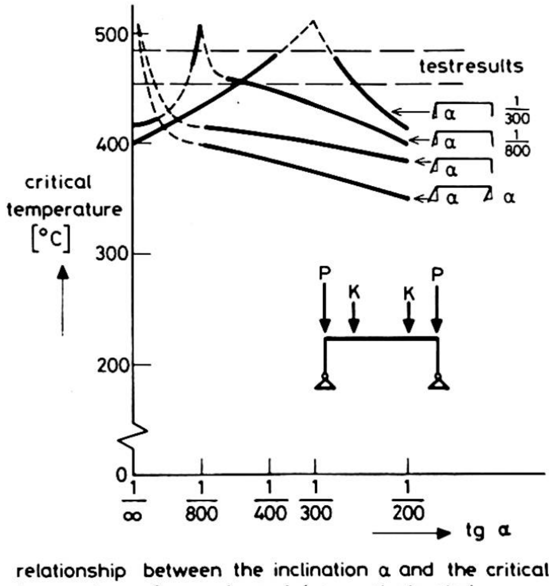
fig. 6



	$\frac{P}{P_{20}}$	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.

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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.

Creep Buckling of a Steel Column in a Temperature-Time History Simulating a Fire

Flambage par fluage d'un poteau en acier selon un diagramme température-temps simulant un incendie

Kriechknicken von Stahlstützen in einem Temperatur-Zeit Verlauf, der einen Brandfall simuliert

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1. INTRODUCTION

At temperatures exceeding $500^{\circ}C$ ordinary carbon steels show a high creep rate even at low tensile or compressive stresses. In a paper published in the Preliminary Report of the 10th Congress of IABSE [1] the author presented results of creep buckling calculations for a steel column of section HE 240B with a yield strength of 300 MPa. A computer programme prepared at the Aeronautical Research Institute of Sweden [2] was used to obtain the creep buckling life of columns of various lengths subjected to a number of different mean compression stresses assuming a constant temperature of $600^{\circ}C$. A diagram was given where the buckling stress was plotted versus the slenderness ratio λ for times of exposure to this temperature ranging from 0.2 to 2.0 h. The diagram showed quite clearly that time is an important parameter which must be considered when analysing the buckling strength of a steel column at elevated temperatures.

The temperature during a fire varies, however, with a rise from room temperature to a maximum, determined by the fire load, after which the cooling starts, usually at a much lower rate than the heating. The computer programme mentioned was therefore modified to allow variations in temperature. A few calculations with such temperature histories assuming maximum temperatures 600° and $650^{\circ}C$ respectively were carried out and presented already in Ref [1]. This investigation has been continued. By repeated life calculations, the compression stress was determined which just resulted in creep buckling collapse at the end of a temperature-time history with a maximum of $600^{\circ}C$. This was done for the slenderness ratios λ = 30, 45, 60 and 90.

2. CALCULATIONS

The modified version of the creep buckling computer programme is able to take into account a linear increase in temperature from an initial value to a maximum ϑ_{smax}, then a constant period and finally a linear decrease in temperature. The creep law of Norton-Odqvist is utilized

$$\dot{\varepsilon} = k\sigma^n$$

where the exponent n = 4.9 is assumed to be constant for all temperatures considered, while the creep coefficient k varies with the temperature $T^{o}K$

$$k = 1.88 \times 10^{-11} \exp(44.7 - 39000/T)$$

Young's modulus, including elastic and plastic deformation, as well as primary creep, is determined by the relation

$$E_o = 325000 - 404\ \vartheta_s \text{ MPa}$$

The numerical calculations were performed for a temperature-time history shown in Fig 1 with heating and cooling rates $1000^{o}C/h$ and $333^{o}C/h$ respectively, and further $\vartheta_{smax} = 600^{o}C$ lasting for a period of 0.2 h. The radius of gyration in the buckling direction of the chosen column section HE 240B is i = 0.10 m. The assumed lengths were 3.0, 4.5, 6.0 and 9.0 m.

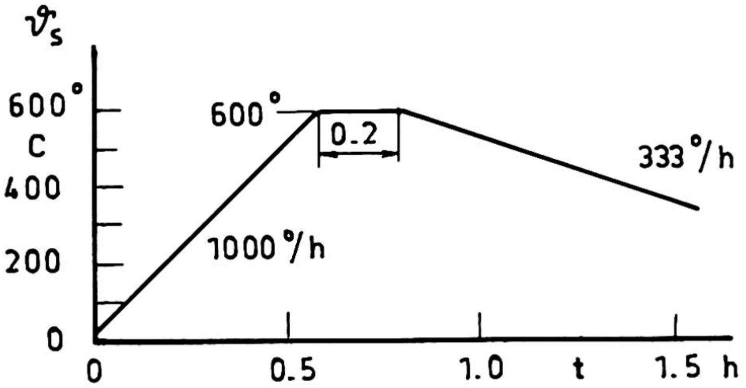

Fig 1 Temperature- time history introduced into computer programme

3. RESULTS AND DISCUSSION

After a number of trials with different loads on the column section it was found that for the slenderness ratios λ = 30, 45, 60 and 90 the mean stresses σ = 60, 51, 41 and 28 MPa respectively would result in creep collapse just at the moment of the temperature-time history during the cooling phase where creep had practically ceased. The results have been plotted in Fig 2 together with the creep buckling curves obtained earlier [1] for constant temperature exposure times. It was discovered that the value determined before with λ = 45, for a history with the maximum temperature $600^{o}C$, was somewhat too high due to an unconservative interpolation. The temperature-time history of Fig 1 seems to have about the same effect as if the column had been exposed to a constant temperature of $600^{o}C$ during 0.5 h. This result is not surprising since the temperature in Fig 1 exceeds $500^{o}C$, where the rapid creep starts, during 0.6 h. Further investigations are likely to show the feasibility of using constant temperature buckling curves to determine the critical stress of a column subjected to a fire temperature history.

The buckling analysis presented in Fig 2, although considering a realistic fire history of the mean temperature of the steel column, still neglects other complications met in practice such as temperature gradients and residual stresses within the column, as well as restraining forces from other structural elements. Thus, it is not claimed that Fig 2 may be used directly for design purposes. It seems to represent a substantial improvement, however, in comparison with the two dotted curves given by the general reporters, Theme III, in the Introductory Report of the Congress [3]. The analysis clearly shows that creep has to be taken into account at fire temperatures exceeding 500 °C, where time is consequently an important parameter. This conclusion was supported in the Prepared Discussion contribution by Fukumura [4].

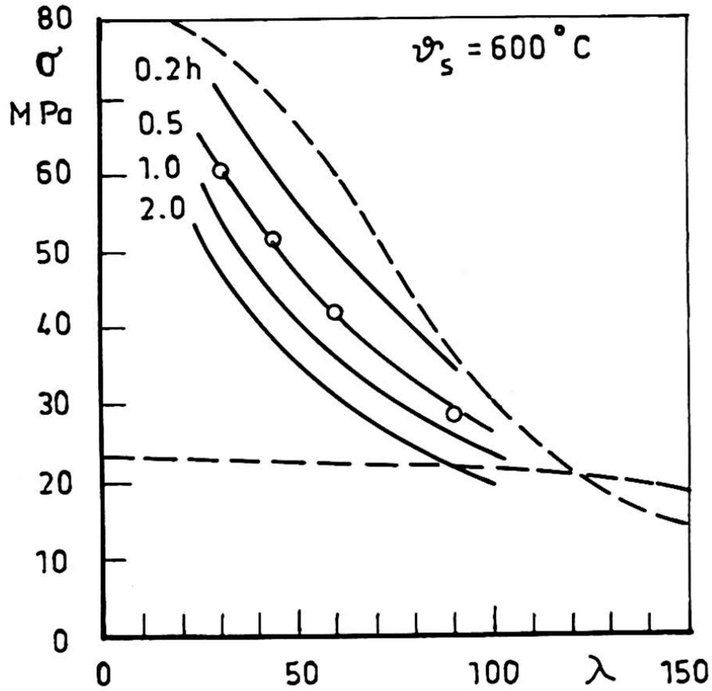

Fig 2 Buckling stress versus slenderness ratio for various times of exposure to 600 °C and for temperature-time history with maximum temperature 600 °C

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SUMMARY

The creep buckling stress was determined by computer analysis for steel columns subjected to a realistic fire temperature history with a maximum of 600 °C. The results were compared to earlier calculations with constant temperature of 600 °C. The influence of creep was still found to be important.

RESUME

La contrainte de flambage par fluage de poteaux en acier a été déterminée par des calculs numériques selon un diagramme température-temps simulant un incendie avec un maximum de 600 °C. Les résultats sont comparés aux calculs déjà faits pour une température constante de 600 °C. On constate que l'influence du fluage reste importante.

ZUSAMMENFASSUNG

Die Kriechknickspannung wurde für Stahlstützen durch ein numerisches Programm für einen Temperatur-Zeit Verlauf mit maximum 600 °C bestimmt, der einen Brandfall simuliert. Das Resultat wird mit früheren Berechnungen mit einer konstanten Temperatur von 600 °C verglichen. Der Einfluss des Kriechens ist von Bedeutung.

Inelastic Behaviour of Protected Steel Columns in Fire

Comportement inélastique des poteaux protégés contre l'incendie

Nichtelastisches Verhalten von feuergeschützten Stahlstützen

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1. Introduction

The fire resistance of structural members has been determined in the past by subjecting isolated members to standard fire tests. More recently, calculations based on heat transfer and the structural properties of materials at high temperature have been used to replace the costly and time-consuming standard fire tests. In the past the cost of testing precluded considerations of assuming more realistic situations corresponding to columns in continuous structures subjected to fires representing actual conditions. These more realistic situations can now be studied by computer calculation.

In this work three aspects for protected steel columns are studied: (1) the stress distribution in a column during fire, (2) the interaction of an interior column exposed to fire with the surrounding building structures, and (3) the fire resistance of columns which take into account creep deformation behavior.

This work is a continuation of Ref.1 and the method of analysis regarding determination of stress, strain and deformation of the column are contained therein.

2. Calculation of Temperature Rise

A computer program was written to calculate the temperatures with in a column when the temperature outside four faces was raised in the manner specified for fire tests in JIS A 1304.

Most columns used in buildings have sections less than 15 cm thick. In such cases detailed calculation of temperature distribution in the steel cross section is unnecessary, and one-dimentional model of the heated column can be usefully employed[2]. The model consists of a steel plate having the same cross-sectional and surface areas per unit height as the four sides of the heated column, with the edges and unexposed side perfectly insulated. This model permits use of a one-dimentional numerical procedure.

In calculating the temperatures, furthermore, it was assumed that fire protection panels became divided into a dry zone and a wet zone, within each of which the thermal properties were taken to be constant.

It was also assumed that vaporization occured at atmospheric pressure at the interface between these zones, and that therefore interface moved away from the heated face as drying penetrated into the protection. Typical results, for the temperatures in H cross-section steel columns with 3 or 5 cm thick protection, are shown in Figures 1 and 2.

3. Stresses and Strains in a Column during Fire

To analyse the strain and stress in a column section, the following assumptions were made:

(1) Plane sections remain plane. This assumption is approximately correct for long prismatic members in continuous construction.

(2) The free expansion of steel due to temperature change, ε_T, is assumed as follows (see Fig.3) :

$$\varepsilon_T = 5.04 \times 10^{-9} T^2 + 1.13 \times 10^{-5} T \quad \cdots\cdots\cdots\cdots \quad (1)$$

where T is in degrees Centigrade.

(3) The relationship between steel stress σ, and strain ε, in compression, is assumed as follows (see Fig.4) :

$$\varepsilon\, E_0(T) = C(T)\sigma - [\, 1 - C(T)]\, \sigma_y(T)\ \ln[\, 1 - \sigma/\sigma_y(T)] \quad \cdots\cdots (2)$$

where

$$E_0(T) = 2.15\times10^6 [0.745 + 0.137\ \ln[-0.01T + 6.3]\ ] \quad (kg/cm^2)$$

$$\sigma_y(T) = 5.0\times10^3 [-4.49\times10^{-9} T^3 + 3.57\times10^{-6} T^2 - 1.41\times10^{-3} T + 1.027] \quad (kg/cm^2)$$

$$C(T) = -4.0\times10^{-4} T + 1.0$$

In this expression $\sigma_y(T)$ is the steel yield point (SM 58), $E_0(T)$ is the initial tangent modulus and C(T) is a parameter that affects the form of the stress-strain curve as shown in Figure 4,in which T is expressed in degrees Centigrade. Besides these data,the material description must include behavior during unloading and in tension. This is arranged by making the following two assumptions: (1) Behavior in tension is the same as that in compression, and (2) Behavior during unloading from (or reloading to) a previously obtained value of a compressive stress given by equation 2 is linear $d\sigma/d\varepsilon = E_0(T)$, the initial tangent modulus.

(4) To evaluate the creep deformation, it is assumed that the creep strain for SM 58 is related to time t, absolute temperature T', and current stress σ in the form such that (see Fig.5)

$$\varepsilon_c = [\ 10^{c - a/2.3T'}] \times [\ [2.37\sigma/5.0]^{b/2.3T' + \alpha}]\ t^n/n \quad \cdots (3)$$

where $c = 20.53$ $b = 1.9\times10^4$ $\alpha = -7.25$ $n = 0.35$ $a = 4.5\times10^4$

t: minutes σ : kg/mm^2 ε_c : %

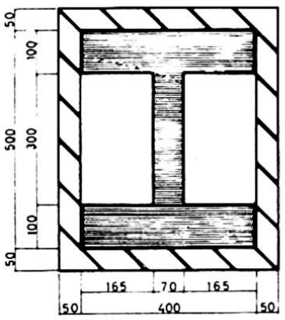
The strain-hardening creep low is applied for the calculation of the nonstationary creep deformation.

(5) The column is divided into blocks with 1/20 or 1/40 column length in the axial direction and each block is subdivided further into 20 elements in the cross sectional direction. For each element the strain, stress and material properties are assumed uniform.

(6) Initial stresses and strains in the cross-section before the fire, are determined, assuming that the axis of the column has initially the form of a sine curve which takes into account the various imperfections in a column.

(7) It is assumed that there are no cases in which a steel column subjected to compression will buckle torsionally or locally and the effect of shearing force on the deformation can be neglected. Buckling of the column will occur in the plane of minimum flexural rigidity.

Figure 1 SECTION



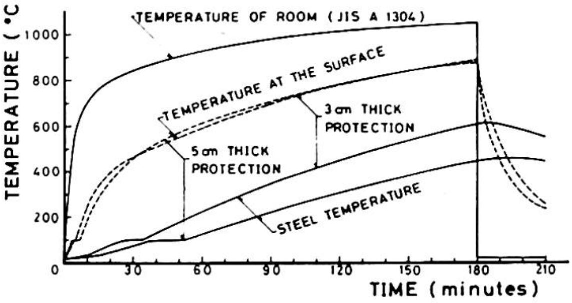
STEEL COLUMN
(SM 58)

Young's modulus	2.15×10^{6} Kg/cm^2
Strain corresponding to the yield stress	2.50×10^{-1} %
Yield stress	5.00×10^{3} Kg/cm^2 (20°C)

FIRE PROTECTION
(CONCRETE)

Thickness	3.0 or 5.0 cm
Percentage of moisture content	4.0 %
Specific gravity	2340 Kg/m^3

Figure 2 CALCULATED TEMPERATURE OF PROTECTED STEEL



FREE THERMAL STRAIN-TEMPERATURE CURVE
Figure 3

$\varepsilon_T = 5.04\cdot10^{-9}\theta^2 + 1.13\cdot10^{-5}\theta$

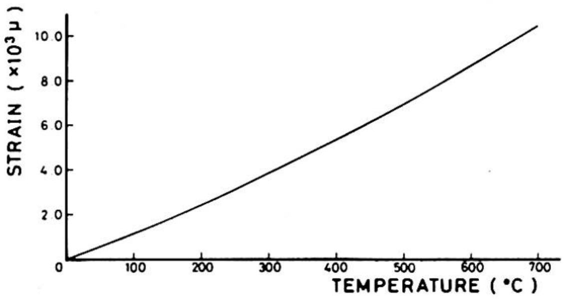


Figure 4 ASSUMED STRESS-STRAIN CURVE
(based on Harada and Furumura's Data)

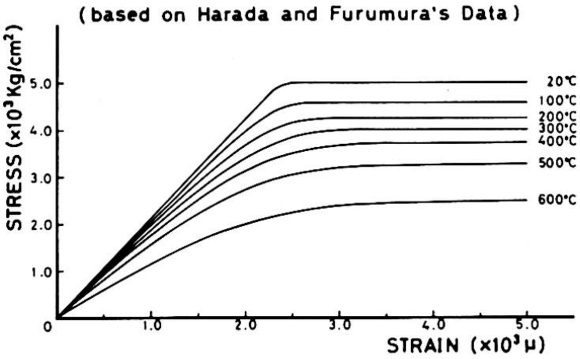
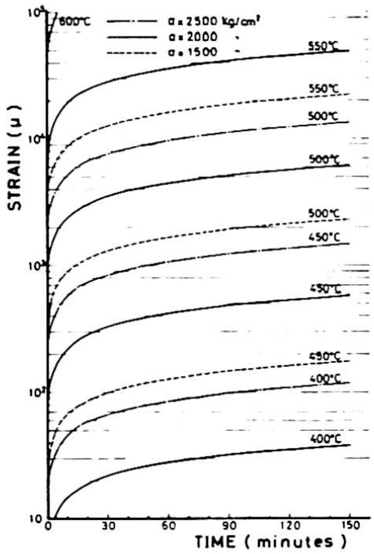
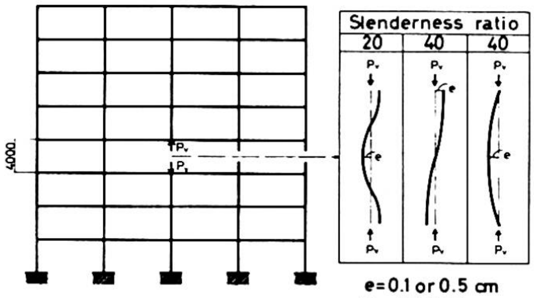


Figure 5 NONSTATIONARY CREEP

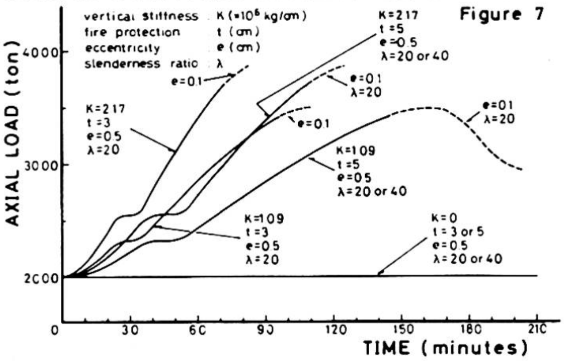


STRUCTURAL ARRANGEMENT OF BUILDING FRAME
Figure 6 AND HEATED COLUMN

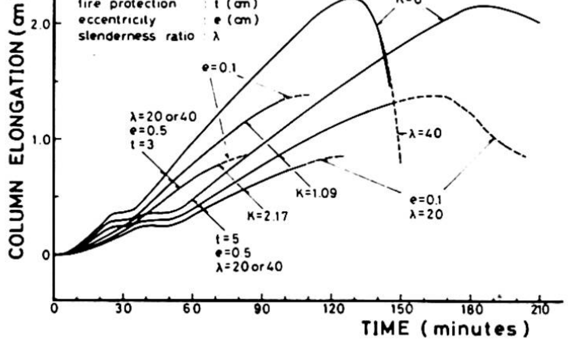


e=0.1 or 0.5 cm
Pv=2000 ton

COLUMN-STRUCTURE INTERACTION DURING FIRE
Figure 7



COLUMN-STRUCTURE INTERACTION DURING FIRE
Figure 8



Based on these assumptions, the distribution of strain and stress is calculated for each 5-minute interval during fire. The method of analysis is to determine, by trial and error, a linear strain distribution in the steel which provides equilibrium of the cross-section.

4. Interaction of a Single Column and Surrounding Structure

A column expands during a fire and, because of the resistance of the surrounding structure to this expansion, more than the initial dead plus live load is attracted to the column.

If every column in a floor were subjected to the same fire, all the columns would expand equally and additional load would not be attracted to any one column. If only one of the floor columns were exposed to fire, the expansion of the column would be resisted by the surrounding structure. The greater the number of slabs above the column, the greater the resistance, and the greater the load attracted to the column.

The correct solution to the interaction problem requires a trial and error determination of moments, curvatures and displacements along the column for each point in time.

5. Results of Calculation

It is clear that a combination of axial load and axial and flexural restraint would arise with the structural arrangement shown in Figure 6.

To calculate lateral deflection, two cases of the flexural restraint for H cross-section steel column are assumed: (1) The column has both ends built in, and (2) The column is built in at the lower end and is free to displace laterally at the upper end but is guided in such a manner that the tangent to the deflection curve remains vertical, and that is equivalent to the case of a column with hinged ends.

The former assumption is considerably in error early in a fire during elastic conditions, but appears to be approximately correct as the column approaches failure when it bends with large inelastic strains at the critical sections. The latter is considered to be corresponded to the case in which every column in a floor would be subjected to the same fire.

Figure 7 shows how the applied load (initial load= 2000 ton/a column) increases by the interaction effect. Figures 8 and 9 to 11 show the corresponding column lengthening and lateral deflection, respectively.

The interaction effect is shown clearly in Fig.7. The curve labelled "K=0", which corresponds to the assumption of no ineraction, serves as a basis for comparison. As the vertical stiffness of the surrounding structure increases from 0 to 1.09×10^6 kg/cm, which corresponds to 1/5 vertical stiffness of a column with 4 m length, the applied load more increases and the fire resistance is decreased, as seen by comparing column movements. As the vertical stiffness is increased further to 2.17×10^6 kg/cm, the fire resistance is more decreased. The fire resistance is also effected by the protection thickness.

Figures 12 to 14 show the variations of the strain, stress and tangential modulus in the extreme element on the convex side of the critical section. Figure 15 and 16 show the variations of the curvature distribution of the columns.

From the numerical results, the column is not greatly affected by creep until the steel temperature reaches 450°C. After 450°C,

COLUMN-STRUCTURE INTERACTION DURING FIRE
Figure 9

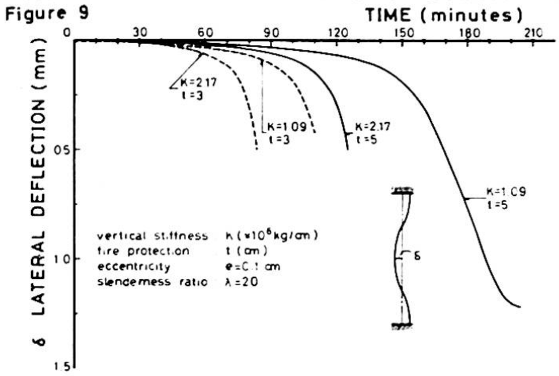
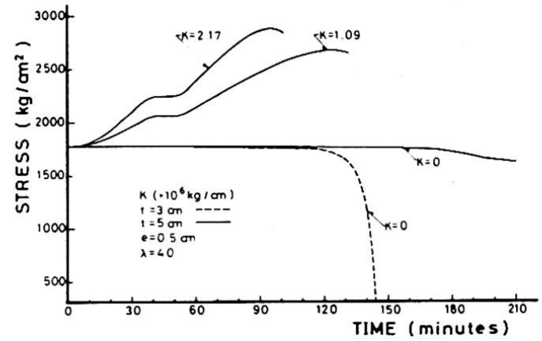


Figure 13 STRESS-TIME CURVES DURING FIRE
ELEMENT (1,20)



COLUMN-STRUCTURE INTERACTION DURING FIRE
Figure 10

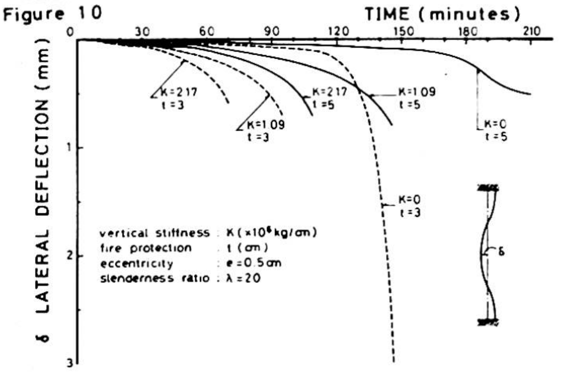
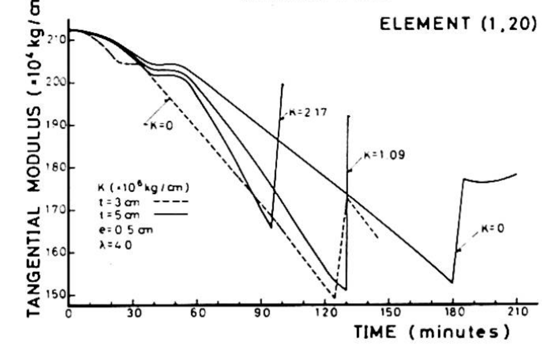
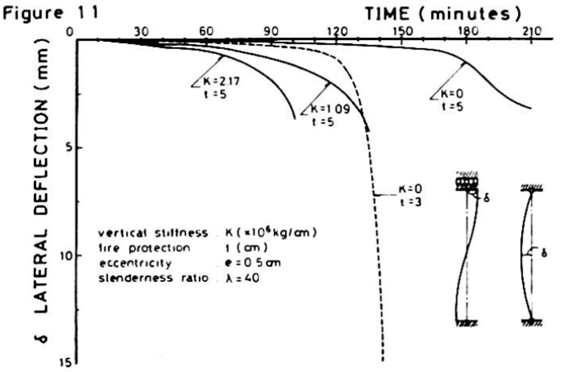


Figure 14 TANGENTIAL MODULUS-TIME CURVES DURING FIRE
ELEMENT (1,20)



COLUMN-STRUCTURE INTERACTION DURING FIRE
Figure 11



CURVATURE DIAGRAMS due to BENDING MOMENT
Figure 15

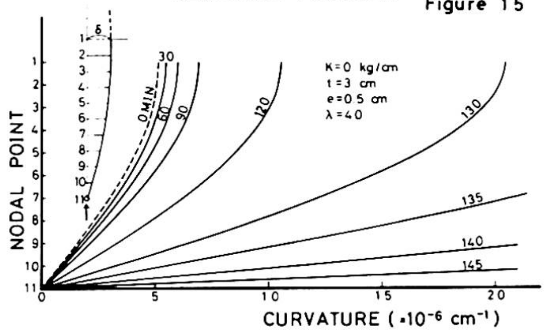
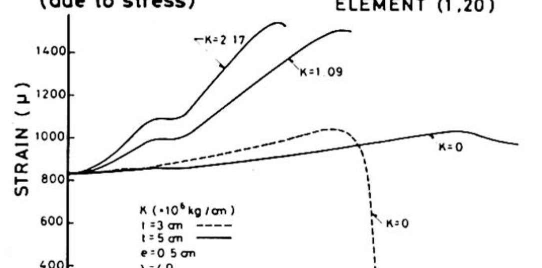
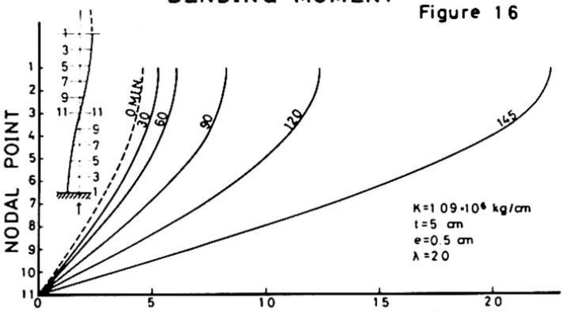


Figure 12 STRAIN-TIME CURVES DURING FIRE
(due to stress)
ELEMENT (1,20)



CURVATURE DIAGRAMS due to BENDING MOMENT
Figure 16



the creep rate increases rapidly and the column causes chord shortening, followed by lateral bending and relief of the applied load.

For very stiff vertical resistance the column buckles quite early, and the greater the initial deflection, the earlier the buckling of the column.

6. Conclusions

This study represents an initial attempt to evaluate in detail the behavior of a steel frame structure in a fire.

The creep deformation of columns in transient heating processes is of considerable interest to engineers working in the field of fire protection.

A numerical technique has been described in Ref.(1). The technique utilized a creep model was **originally suggested by England**[(3)] and Dougill[(4)] and expanded by the authers to suit certain practical requirements. The calculations that have been undertaken have been based on an idealized model of material behavior and it is doubtful whether the numerical values obtained have any significance in themselves. However, some aspects being important to understand modes of behavior of columns in fire have been found out and the information obtained from such detailed analysis is very useful in the future development of a simple and rational procedure for fire safety design of high rise steel structures.

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SUMMARY

The study shows the inelastic behaviour of protected steel columns under fire action. It is shown that the effect of creep on the buckling strength is very important at temperatures more than 450° C.

RESUME

L'étude s'occupe du comportement inélastique des poteaux métalliques protégés contre l'incendie. Il est montré que l'influence du fluage est très importante pour les températures supérieures à 450° C.

ZUSAMMENFASSUNG

Das nichtelastische Verhalten von geschützten Stahlstützen bei Brandbeanspruchung wird untersucht. Es wird gezeigt, dass dem Kriechen bei Temperaturen über 450° C grosse Bedeutung zukommt.