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Théorie des Equivalences. Application à la thermoélasticité

Äquivalents-Lehre.
Anwendung an der Thermo-Elastizität

Theory of Equivalences.
Application to the Thermoelasticity

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1 - INTRODUCTION

Pour résoudre les problèmes dérivant d'un champ, la Théorie des Equivalences propose une approche générale qui consiste à substituer à l'étude du corps réel, celle d'un corps fictif plus accessible au calcul. Les caractéristiques de ce corps, appelé solide équivalent, sont déterminées en écrivant l'égalité des fonctionnelles définissant l'état physique des deux solides. Cette condition d'équivalence permet d'affirmer qu'ils auront le même comportement.

La Théorie des Equivalences a déjà permis de résoudre avec succès des problèmes d'élasticité (Réf. 1 et 5) et d'infiltration d'eau dans le sol (Réf. 3) en utilisant comme solide équivalent, respectivement des structures de poutres et treillis, et des réseaux orthogonaux de canaux. Ainsi nous nous proposons de résoudre les problèmes de thermo-élasticité en attribuant tour à tour au solide équivalent des caractéristiques thermiques et mécaniques. La conduction de chaleur est alors ramenée à un problème d'écoulement et la recherche des contraintes à un simple problème de contraintes thermiques dans une structure. Pour résoudre ce dernier problème, une méthode systématique est utilisée.

2 - DIFFUSION DE LA CHALEUR

Il est identique de résoudre l'équation de la chaleur (1) ou de trouver le champ thermique T qui minimise la fonctionnelle (2).

(1) $$k \nabla^2 T + Q = \rho c \dot{T}$$

(2) $$I(T) = \int \left[k_i \, (T_{,i})^2 - (2Q - \rho c \dot{T}) \, T \right] dV$$

Le solide étudié est remplacé par un grillage orthogonal de barres dont les caractéristiques thermiques et les sections sont calculées en écrivant l'égalité de leur fonctionnelle respective.

Si les deux solides occupent le même volume et s'ils sont soumis aux mêmes conditions sur le contour, il reste seulement à identifier les termes de la forme :

$$(3) \qquad \int k_i \, (T_{,i})^2 \, . \, dV$$

Il est nécessaire de supposer que le découpage du corps réel est suffisamment fin pour admettre que les gradients thermiques sont constants dans chaque maille du grillage. On peut alors déterminer (Réf. 4) les caractéristiques du solide équivalent.

Nous découpons la durée du phénomène en une suite d'intervalles telle que l'approximation suivante soit admissible :

$$(4) \qquad \dot{T} = \frac{T\,(t_k) - T\,(t_{k-i})}{\Delta t_k}$$

On écrit pour chaque noeud i que la quantité de chaleur amenée par toutes les barres (i,j) aboutissant en i est égale à la quantité de chaleur $Q_i(t)$ reçue par l'extérieur, augmentée de la quantité de chaleur nécessaire à l'échauffement de l'élément de volume ΔV_i pendant le temps Δt_k.

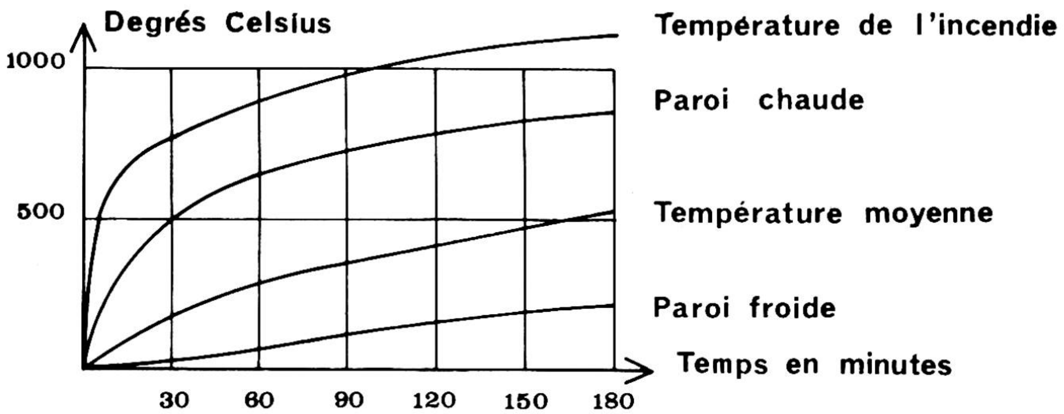
$$(5) \qquad \sum_{j \to i} \left(\frac{kS}{L}\right)_{ij} \left[T_j(t_k) - T_i(t_k)\right] = Q_i(t_k) + \rho c \, \frac{\Delta V_i}{\Delta t_k} \left[T_i(t_k) - T_i(t_{k-1})\right]$$

Exemple 1

Cherchons l'évolution des températures dans la section d'une poutre en Té soumise à un incendie en sousface. La poutre est initialement à 0°C. La température de l'incendie est donnée par la formule normalisée :

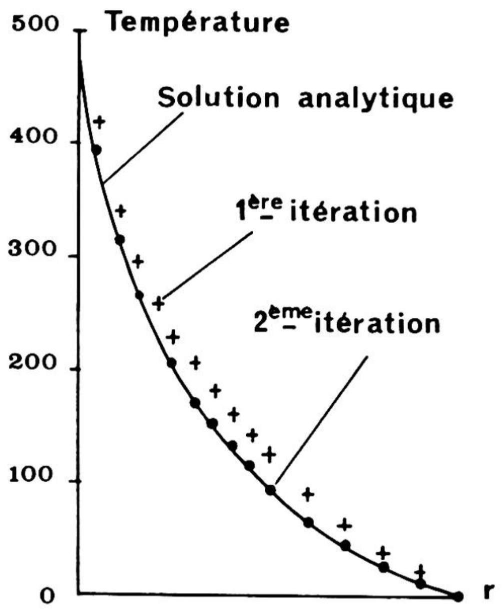
$$T = 345.\ \log\,(8t + 1)$$

H_1 et H_2 les coefficients de convection rayonnement des faces froide et chaude ont pour valeurs respectives 13 et 46 kcal/h.m^2.°C. Les caractéristiques du béton ont pour valeurs : ρ = 2400 kg/m^3, k = 1,4 kcal/h.m.°C et C = 0,22 kcal/kg.°C.



Exemple 2

On peut étudier facilement les solides à caractéristiques variables, il suffit de faire évoluer la structure équivalente sans avoir à modifier les programmes de résolution.



Considérons une enceinte cylindrique circulaire en béton dans laquelle nous imposons la température des deux faces T_i = 500°C, T_e = 0°C. Nous supposons que la conductivité thermique du béton varie avec la température suivant la formule suivante :

$$k(T) = 1,4 - 0,0012.T$$

3 - CALCUL DES STRUCTURES EN THERMOELASTICITE

Pour déterminer les contraintes d'origine thermique, nous proposons une méthode systématique de calcul fondée sur la notion d'équation intrinsèque. L'effet du champ thermique T peut être remplacé par l'action en chaque noeud i de sollicitations thermiques équivalentes T_i (Réf. 2 et 4).

4 - CONTRAINTES THERMIQUES DANS LES SOLIDES CONTINUS

Problèmes tridimensionnels

En thermoélasticité la densité d'énergie de déformation est :

$$(6) \quad U = \frac{1}{2}(e_{ij} - \alpha T \delta_{ij})\sigma_{ij} = \frac{\lambda}{2}(e_{ii})^2 + \mu(e_{ij}\, e_{ij}) - (3\lambda + 2\mu)\alpha T(e_{ii}) + \frac{3}{2}(3\lambda + 2\mu)\,(\alpha T)^2$$

Considérons le modèle équivalent parallélépipèdique (Réf. 1 et 3), une barre (i,j), de direction Ox, a pour énergie de déformation :

$$(7) \quad W_{ij} = \frac{1}{2}\,\rho_x(e_{xx} - \alpha T_{ij})^2 + 6\,\eta_x(e_{xy}^2 + e_{xz}^2) \quad \text{avec} \quad \rho_x = (ESL)_{ij} \quad \text{et} \quad \eta_x = (EI/L)_{ij}$$

La condition d'équivalence du modèle s'écrit :

$$(8) \qquad \sum_{i,j} W_{ij} = \int_V U dV$$

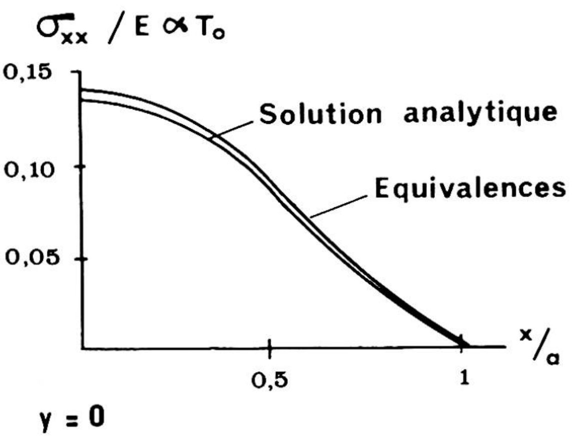
L'égalité des termes purement mécaniques nous permet de retrouver les valeurs des caractéristiques ρ et η (Réf. 1,4 et 5); si on admet que le champ thermique est uniforme dans le petit élément étudié, l'équivalence des termes thermiques est vérifiée. Cette hypothèse étant peu admissible, il est proposé (Réf. 4) des formules de calcul des températures des barres en fonction des températures des noeuds de manière à vérifier avec le plus de précision possible les conditions d'équivalence.

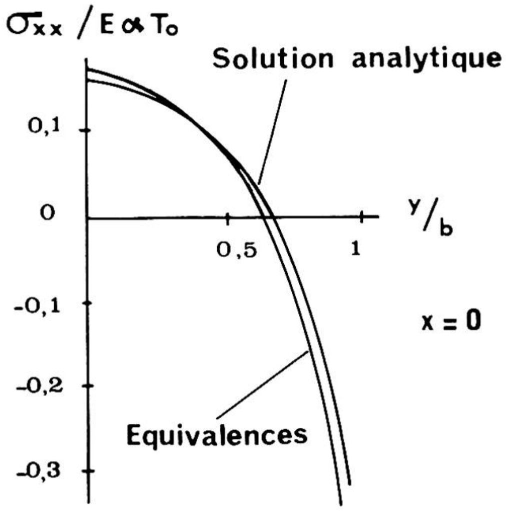
Problèmes plans

Le cas des contraintes planes se traite sans difficulté avec un modèle plan. Au contraire, dans les déformations planes, la troisième dimension a une contribution non nulle dans l'énergie de déformation thermoélastique et la résolution par modèle plan est impossible directement.

Exemple de contrainte plane

Considérons une plaque carrée, soumise au champ thermique $T = T_o\ (Y^2/b^2 - 1/3)$. Sur la structure équivalente sont appliquées les sollicitations thermiques équivalentes.





Etude des dalles

Les hypothèses habituelles sur la flexion des dalles ne sont admissibles que pour les champs thermiques variant seulement avec z ou linéaire en z. Le cas suivant a été principalement développé.

(9) $$T = T_o\ (x,y) + (z/h).\Delta T(x,y) \qquad -\frac{h}{2} \leqslant z \leqslant \frac{h}{2}$$ h épaisseur

L'énergie de déformation par unité de surface (Réf.4) est :

(10) $$U = U_o + \frac{D}{2}\Big[(w,xx)^2 + (w,yy)^2 + 2\,\nu\, w_{,xx}\, w_{,yy} + 2\ (1-\nu)(w_{,xy})^2 + 2\ (1+\nu)(w_{,xx} + w_{,yy})\ \alpha\,\frac{\Delta T}{h} + 2(1+\nu)\ (\alpha\,\frac{\Delta T}{h})^2 \Big]$$

où U_o est l'énergie d'une plaque de température $T_o(x,y)$, problème traité au paragraphe précédent.

Les solides équivalents sont des grillages de poutres. Si la poutre (i,j) est soumise au gradient thermique $\frac{\Delta T}{h}$, elle aura l'énergie de flexion suivante :

(11) $$W_{ij} = \frac{1}{2} EIL \left(w,_{xx} + \frac{\alpha \Delta T}{h}\right)^2$$

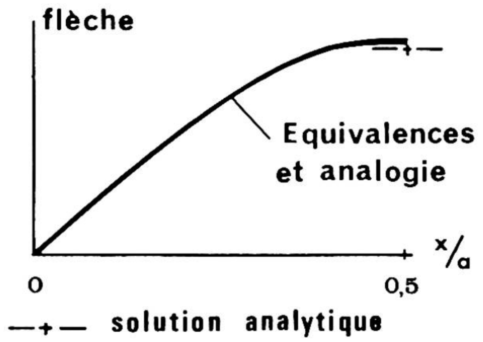
Il est possible de définir une analogie isotherme (Réf. 4) en appliquant à la dalle un chargement transversal réparti p^* et des sollicitations sur les bords dépendant des types d'appui.

(12) $$p^* = - \frac{1}{(1-\nu)} \nabla^2 M_T \quad \text{avec } M_T = \int_{-\frac{h}{2}}^{\frac{h}{2}} \alpha\, ETzdz = \alpha\,(1-\nu^2)\, D \frac{\Delta T}{h}$$

Exemple de dalle

Considérons une dalle carrée avec deux bords opposés encastrés et les deux autres simplement appuyés. Cette dalle est soumise au champ thermique :

$$T(x,y,z) = \frac{\Delta T_o}{h} \frac{4x}{a} \left(1 - \frac{x}{a}\right).z \qquad \text{d'où} \qquad p^* = \alpha\, D(1+\nu) \frac{8}{a^2} \frac{\Delta T_o}{h}$$



Il est remarquable d'observer que la Théorie des Equivalences et l'analogie isotherme donnent des résultats très proches bien que la première s'applique sous forme de moments aux noeuds et la seconde sous forme d'un chargement transversal.

5 - CONCLUSION

Nous avons pu remarquer que la Théorie des Equivalences donne des résultats meilleurs ou très proches de ceux obtenus avec une analogie isotherme appliquée à la structure équivalente. Il est possible d'en déduire que la discrétisation nécessaire pour appliquer ce type de théorie introduit une erreur plus importante que les imprécisions sur les sollicitations thermiques équivalentes.

Enfin le formalisme simple de la Théorie des Equivalences permet des extensions fructueuses, en particulier aux solides à caractéristiques variables.

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RESUME

La théorie des Equivalences a pour principe de substituer au solide étudié un corps fictif plus accessible au calcul. En utilisant un solide équivalent constitué de barres possèdant des caractéristiques thermiques et mécaniques, le problème de thermoélasticité est alors remplacé par un problème d'écoulement de chaleur suivi d'une recherche de contraintes thermiques dans une structure.

ZUSAMMENFASSUNG

Die Theorie der Aequivalenzen oder Gleichwertigkeiten ist ein Versuch das Studium eines festen Körpers durch das Studium eines anderen, fiktiven Körpers, der sich besser berechnen lässt, zu ersetzen. Bei der Anwendung eines gleichwertigen Körperinhaltes, bestehend aus Stäben mit thermischen und mechanischen Eigenschaften wird also das Problem der Thermoelastizität durch das Problem der Wärmeübertragung ersetzt und eine Untersuchung der Temperaturspannungen angeschlossen.

SUMMARY

The principle of the theory of equivalences is to replace the solid body by a fictious body which is easier to calculate. Using an equivalent solid made up of bars with thermal and mechanical properties, the problem of thermoelasticity is then replaced by a problem of heat followed by research on thermal stresses in a structure.

Détermination par la méthode des éléments finis des évolutions de température pour les structures soumises à l'incendie

Bestimmung des Temperaturverlaufes in brandgefährdeten Hochbauten mittels der Methode der Finiten Elemente

Temperature Transients Determination by the Finite Element Method on the Fire Response of Structures

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1. INTRODUCTION.

La pratique actuelle pour évaluer la résistance au feu des structures consiste le plus souvent en des techniques expérimentales où une éprouvette est soumise à un test en four. La représentativité d'un tel essai pour une structure complexe est une première difficulté à circonscrire, à laquelle s'ajoute le coût des installations et des mesures.

Cet article présente une alternative à la conduite d'essais : l'élaboration de modèles mathématiques d'éléments finis aptes à simuler les conditions de test. On y envisage le problème de base, à savoir celui de la transmission de chaleur dans les matériaux des divers types de structures en cause.

La souplesse de la méthode laisse en outre présager son extension aux conditions en service de structures réelles.

2. CODE DE CALCUL PAR ELEMENTS FINIS.

Un module faisant partie du code général de calcul par éléments finis SAMCEF [1] a été développé pour répondre à cet objectif. Son principe repose sur la Méthode des Eléments Finis couplée à une intégration temporelle pas à pas : cette approche est bien connue pour les situations linéaires [2,3], mais elle a reçu peu d'applications dans le domaine du transfert non-linéaire de chaleur [4-8] : celui-ci est caractérisé par des propriétés de matériau variant avec la température et des conditions aux limites qui, dans le cas présent, doivent être aptes à reproduire les situations d'incendie. Ces non-linéarités impliquent le plus souvent le recours à une technique incrémentale itérative [4-6].

Un des buts de l'article est de présenter une technique incrémentale basée sur le concept de matrice de conductivité "tangentielle" [7-8] qui permet d'éviter les itérations au cours d'un pas de temps et rend le code aussi économique que pour les problèmes linéaires. La gamme des éléments finis utilisables couvre toutes les formes structurales habituelles uni-, bi- et tridimensionnelles. Les lois de variation des caractéristiques avec la température (éventuellement différentes d'élément à élément) sont à la disposition de l'utilisateur sous forme de polynômes de la température (de degré deux au plus) ; le choix du pas de temps peut varier au cours du processus pas à pas et est guidé par les considérations de stabilité et de précision dégagées en analyse linéaire [7,8].

2.1. Rappel des équations et conditions aux limites du problème.

La distribution de température dans un corps solide V limité par une surface S et rapporté à un système de coordonnées cartésiennes x, y, z est régie par l'équation aux dérivées partielles non-linéaire :

$$\frac{\partial}{\partial x}\left[k_x(\Theta)\frac{\partial\Theta}{\partial x}\right] + \frac{\partial}{\partial y}\left[k_y(\Theta)\frac{\partial\Theta}{\partial y}\right] + \frac{\partial}{\partial z}\left[k_z(\Theta)\frac{\partial\Theta}{\partial z}\right] + Q = c(\Theta)\frac{\partial\Theta}{\partial t} \qquad (1)$$

assortie de la condition initiale

$$\Theta(x, y, z, o) = \Theta^{*} \qquad (2)$$

et des conditions aux limites du type

a) température imposée à la frontière $\Theta = \overline{\Theta}$ sur S_1 (3)

b) flux de chaleur connu ou fonction de Θ sur $S_2 + S_3 + S_4$:

$$n_x k_x(\Theta)\frac{\partial\Theta}{\partial x} + n_y k_y(\Theta)\frac{\partial\Theta}{\partial y} + n_z k_z(\Theta)\frac{\partial\Theta}{\partial z} = q_e + h(\Theta_e - \Theta) + \sigma_o \varepsilon_{ep}(T_e^4 - T^4) \qquad (4)$$

avec
- $T(x,y,z,t)$ température absolue instantanée ;
- $\Theta(x,y,z,t)$ écart de température par rapport à une référence uniforme T_o : $\Theta = T - T_o$;
- $k_x(x,y,z,\Theta)$, $k_y(x,y,z,\Theta)$, $k_z(x,y,z,\Theta)$ coefficients de conductivité principaux du matériau ;
- $c(x,y,z,\Theta)$ capacité calorifique du matériau par unité de volume ;
- $Q(x,y,z,t)$ distribution de sources de chaleur par unité de volume ;
- n_x, n_y, n_z cosinus directeurs de la normale extérieure au volume ;
- $q_e(x,y,z,t)$ distribution de sources de chaleur par unité de surface ;
- $h(x,y,z)$ coefficient d'échange par convection ;
- $\Theta_e(x,y,z,t)$ température de la source représentant l'incendie ;
- $T_e(x,y,z,t)$ température absolue de cette source ;
- σ_o constante de Boltzmann = $4{,}93.10^{-8}$ Kcal/m2.h.(°K)^4
- $\varepsilon_{ep}(x,y,z)$ facteur de rayonnement mutuel entre la source rayonnante et la paroi.

2.2. Formulation du problème par la Méthode des Eléments Finis.

La démarche initiale consiste à écrire une forme globale qui puisse traduire à l'échelle d'un petit domaine, appelé élément fini, le bilan calorifique local décrit par (1). En dehors de la conduction de chaleur en régime permanent, pour laquelle il existe un support variationnel vrai [8], cette globalisation est réalisée au mieux par l'application de la Méthode des Résidus Pondérés au système (1-4) : le cas particulier le plus fréquent de celle-ci est la Méthode de Galerkin, par laquelle on orthogonalise les résidus par rapport aux fonctions de base locales $m_k(x,y,z)$ choisies pour le champ de température :

$$\Theta(x,y,z,t) = \sum_{k=1}^{N} m_k(x,y,z) \cdot a_k(t) \qquad (5)$$

Ceci produit le système matriciel élémentaire [8] :

$$K_E(q_E) \cdot q_E(t) + C_E(q_E) \cdot \dot{q}_E(t) = g_E(t) \qquad (6)$$

qui, après assemblage des éléments finis pour reconstituer le domaine, livre le système différentiel non-linéaire structural :

$$K(q) \cdot q(t) + C(q) \cdot \dot{q}(t) = g(t) \tag{7}$$

où K_E, K sont les matrices de conductivité élémentaire ou structurale ;

C_E, C les matrices de capacité élémentaire ou structurale ;

q_E, q les vecteurs des températures locales élémentaires ou structurales;

$\dot{q}_E$, $\dot{q}$ les vecteurs des taux de variation des températures élémentaires ou structurales ;

g_E, g les vecteurs des flux thermiques nodaux élémentaires ou structuraux

2.3. Technique incrémentale de solution basée sur la notion de conductivité tangentielle.

La majorité des auteurs qui ont à résoudre le système non-linéaire (7) utilisent des techniques itératives à l'intérieur d'un pas de temps [4-6]. La technique préconisée ici est de linéariser le premier terme du système (7) autour de la solution $q(t)$ à l'instant t et de ne retenir que les termes du développement d'ordre un en Δq :

$$K(q+\Delta q) \cdot (q+\Delta q) \simeq \left[K(q) + \frac{dK}{dq}\Delta q\right] \cdot (q+\Delta q) \simeq K(q) \cdot q + \left[K(q) + \frac{dK}{dq} \cdot q\right] \cdot \Delta q$$

où le second terme du dernier membre fait apparaître la matrice (8)
de conductivité tangentielle structurale, qui s'écrit formellement

$$K^T(q) = K(q) + \frac{dK}{dq} \cdot q \tag{9}$$

L'interprétation graphique de la méthode dans le cas stationnaire à une dimension est présentée figure 1 : elle s'apparente à la méthode bien connue de Newton-Raphson pour la solution d'une équation algébrique non-linéaire.

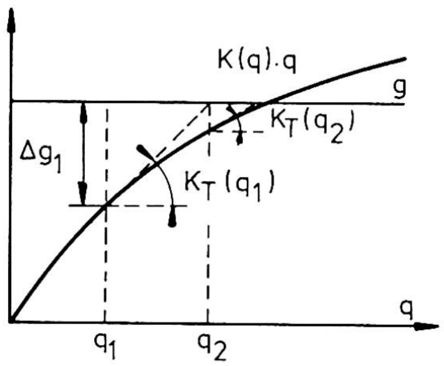

FIG. 1

La seconde étape dans la résolution du système différentiel (7) concerne l'intégration pas à pas proprement dite : pour ce faire, on choisit d'écrire (7) en un instant particulier de l'intervalle de temps

$$t^* = t + \phi \Delta t \qquad \phi \in [0,1] \tag{10}$$

pour lequel le taux de variation des températures est approché par le quotient

$$\dot{q}(t^*) = \frac{q(t+\Delta t) - q(t)}{\Delta t} \tag{11}$$

L'utilisation conjointe de (8) avec $\Delta q = \phi[q(t+\Delta t) - q(t)]$ et de (10-11) dans (7) produit dès lors le schéma pas à pas

$$q(t+\Delta t) - q(t) = \left\{\frac{1}{\Delta t} C[q(t)] + \phi K^T[q(t)]\right\}^{-1} \cdot \left\{g(t+\phi\Delta t) - K[q(t)] \cdot q(t)\right\} \tag{12}$$

qui livre le vecteur des températures en $t + \Delta t$ en fonction de sa valeur en t et moyennant la connaissance des flux thermiques nodaux à l'instant t^* ; on réalise souvent une linéarisation de ceux-ci, qui produit :

$$g(t+\phi\Delta t) = (1-\phi)\, g(t) + \phi\, g(t+\Delta t) \tag{13}$$

le paramètre continu ϕ introduit en (10) compte comme cas particuliers bien connus

$\phi = 0$ le schéma explicite d'Euler ;

$\phi = 0{,}5$ le schéma du trapèze ou de la différence centrée ;

$\phi = 1$ le schéma implicite pur.

Remarquons enfin que la technique (12) implique une matrice incrémentale non-symétrique du fait de la présence de (9), ce qui nécessite une organisation spéciale de l'algorithme de résolution.

3. MODELISATION DU PHENOMENE PHYSIQUE.

Le problème le plus difficile se situe au niveau de la détermination des échanges thermiques éprouvette-four, compte-tenu de l'environnement existant et des propriétés thermiques des matériaux en présence.

La densité de flux de chaleur (4) à laquelle le matériau est soumis peut se mettre sous la forme (cfr. (4))

$$q_e = h\,(\Theta_e - \Theta) + \sigma_o \varepsilon_{ep}\,(T_e^4 - T^4)$$

Les paramètres critiques à déterminer sont les émissivités relatives de la source et de la paroi de l'éprouvette. Le coefficient d'échange par convection a moins d'importance, car les températures atteintes lors de l'exposition au feu sont telles que l'échange radiatif est prépondérant.

En ce qui concerne les échanges par radiation, il est couramment admis [6] que, dans le cas du béton ou de l'acier, l'émissivité des parois vaut 0.8 ou 0.9. L'émissivité de l'environnement, par contre, est beaucoup plus incertaine : elle dépend de l'émissivité de la source rayonnante (flammes ou résistances) et est influencée par les autres surfaces d'échange. Le domaine des valeurs admissibles s'étend de 0,3 à 0,9, mais les comparaisons que nous avons effectuées ont montré que l'on obtient des résultats satisfaisants dans tous les cas en se limitant aux valeurs 0,5 et 0,6.

La modélisation du phénomène est aussi sensible à un autre facteur : il s'agit des propriétés thermiques de l'acier et du béton, et le fait que ces propriétés varient avec la température. Dans le cas du béton, les résultats expérimentaux présentent une dispersion considérable par suite de la nature même de ce matériau dont les propriétés varient avec les ingrédients, leur proportion, l'âge et les conditions d'environnement.

4. EXEMPLE NUMERIQUE.

En Belgique, une vaste recherche à caractère national a été entreprise dans le but d'améliorer les connaissances en matière d'incendie. Certains travaux expérimentaux se rapportent au comportement des matériaux de construction aux températures élevées.

Un de ces essais concerne une éprouvette de béton placée dans un petit four à résistances [12] . Il s'agit d'un prisme de béton de gravier de 18 x 18 x 30 cm placé au centre du four (figure 2). Des thermocouples sont noyés dans la masse à la fabrication, au niveau de l'axe de symétrie du plan médian : ils sont destinés à mesurer la courbe température-temps en ces points.

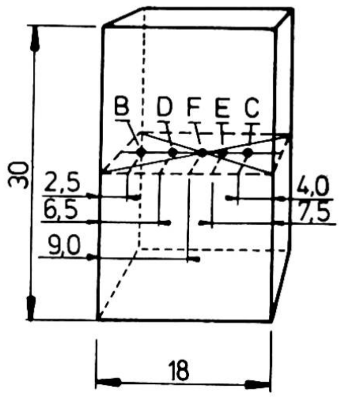

FIG. 2

On ramène l'étude de l'éprouvette à celle de sa section droite dans le plan médian (cfr. figure 3): cette manière de voir les choses est évidemment idéale, puisque l'écoulement n'est pas bidimensionnel. On peut cependant supposer que les influences des pertes de chaleur par les faces supérieure et inférieure ne se fait que peu sentir au niveau du plan médian.

L'échange convectif est régi par un coefficient d'échange h = 7,5 kgcal/h.m2.°C.. En effet, dans le cas de petits fours à résistances, l'environnement est relativement calme, et on peut prendre un coefficient d'échange modéré. Le facteur de rayonnement mutuel ε_{ep} est pris égal à 0.45.

D'une manière générale, la conductivité thermique du béton k_b décroît avec la température. Ce phénomène est représenté ici par la loi linéaire

$$k_b = 1.8 - 10^{-3}\ \theta$$

où k_b est exprimé en kgcal/h.m2.°C et θ en °C.

Le modèle adopté est doté d'une capacité calorifique invariante avec la température, soit c_b = 700 kgcal/m3.°C., ce qui ne prend pas en compte l'augmentation brusque de capacité qui se produit au voisinage de 100°C, par suite des réactions endothermiques dans le béton. Cette grande valeur de la capacité thermique renforce l'hypothèse d'un écoulement plan de chaleur.

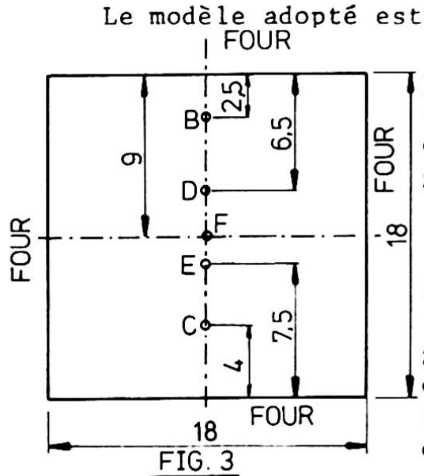

FIG. 3

La discrétisation en éléments finis est limitée à un quart de la section droite par raison de symétrie (figure 4). Elle comporte 64 éléments quadrangulaires plans pour 81 degrés de liberté, les champs de base étant linéaires. On a raffiné la discrétisation au voisinage de la paroi, par suite des gradients de température élevés en ces endroits.

Les résultats théoriques obtenus sont comparés avec ceux fournis par les thermocouples aux figures 5 et 6, pour les points C et E. On peut noter un excellent accord, vu les approximations du modèle. On a noté aussi une excellente concordance pour les points B et D, mais un peu moins bonne pour le point F.

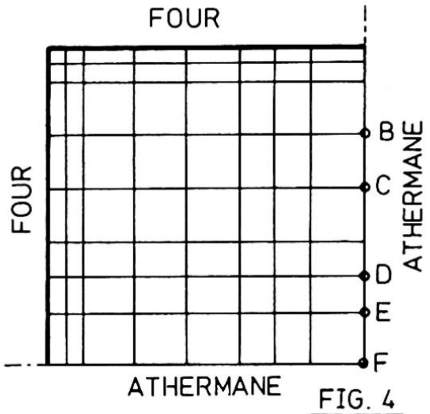

FIG. 4

En conclusion, on peut dire que la Méthode des Eléments Finis fournit un moyen puissant et efficace de modélisation de problèmes complexes et constitue une alternative attrayante à des essais en vraie grandeur. On notera à cet égard sa grande souplesse vis-à-vis d'éventuelles modifications dans les conditions d'environnement.

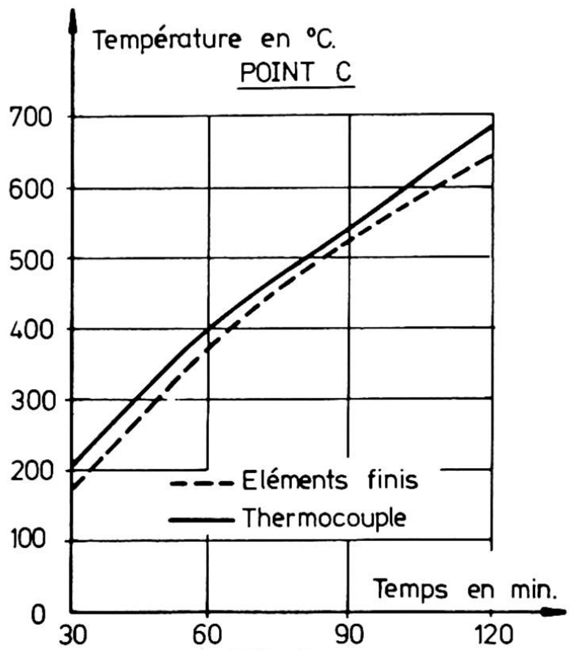

FIG. 5

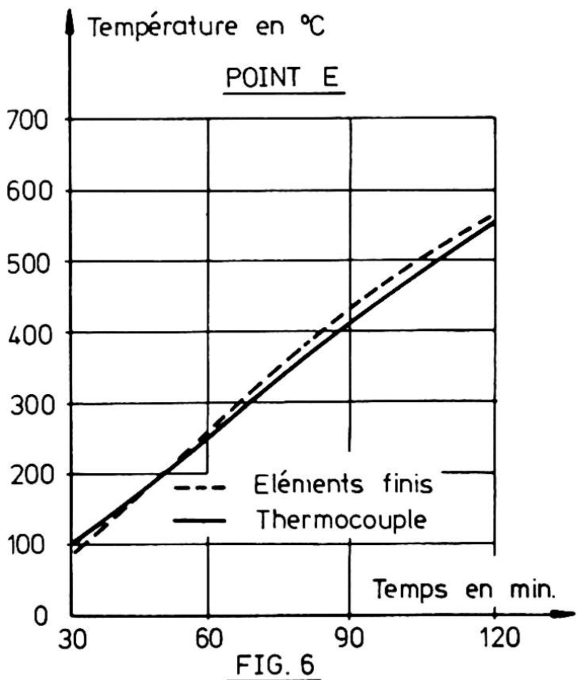

FIG. 6

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RESUME - On présente un code de calcul, qui permet d'évaluer les évolutions de la température dans les structures soumises à l'incendie. Ce code est basé sur la Méthode des Eléments Finis, et son originalité principale consiste à résoudre le problème incrémental sans itérations en se basant sur la notion de conductivité tangentielle. Les résultats fournis par le code sont comparés à des résultats expérimentaux recueillis lors d'essais en four, et la concordance obtenue est excellente.

ZUSAMMENFASSUNG - Ein Rechenverfahren, das die Erfassung der Temperaturänderungen in einem Tragwerk unter Brandeinwirkung ermöglicht, wird vorgeschlagen. Die Haupteigenschaften dieser auf dem Verfahren der Finiten Elemente ruhenden Methode besteht in der Lösung des Inkrementalproblems ohne Iteration durch das Konzept der tangentialen Leitfähigkeit. Die nach diesem Verfahren erhaltenen Ergebnisse wurden jenen von Brandkammerversuchen gegenübergestellt, wobei sich eine sehr gute Uebereinstimmung ergab.

SUMMARY - A computation code is presented which enables to compute the temperature distribution in structures submitted to fire. This code is based on the Finite Element Method and its main originality in solving the incremental problem without iterations by using the concept of tangential conductivity. The results given by the code are compared with experimental results from fire tests and the agreement obtained is very good.

Application of a Limit State Concept to the Performance of a Structure under Fire Conditions

Application du concept de l'état limite aux réactions d'une structure en feu

Anwendung des Konzepts der Grenzzustände auf das Verhalten eines brandbelasteten Bauwerkes

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A RATIONAL PHILOSOPHY

The concept of structural fire protection as used currently was developed over half a century ago on the basis of fire experience and intuitive knowledge, and during the course of time has been marginally modified particularly following public reaction to large-scale fires. Most building codes and regulations base their requirements on assumed fire load, divide buildings into different risk categories and make some allowance for the height or the size of the building on a rule of thumb basis. The relationship between the fire load and fire resistance is basically that derived by Ingberg[1] nearly 60 years ago and is shown in Table 1 and Figure 1.

Combustible content		Fire load density		Duration of exposure in standard test
lb/ft^2	kg/m^2	Btu/ft^2	MJ/m^2	h
10	50	80×10^3	900	1.0
15	70	120×10^3	1360	1.5
20	100	160×10^3	1820	2.0
30	150	240×10^3	2720	3.0
40	200	320×10^3	3630	4.5
50	250	380×10^3	4310	6.0
60	300	432×10^3	4410	7.0

Table 1. Equivalent severities of building fires

The United Kingdom authorities accepted this with some simplifications on the basis of a study published in 1946[2]. On this basis domestic and residential buildings qualify for a fire resistance of half to one hour and office buildings and shops one to two hours. High buildings have been taken to mean those beyond the reach of the fire brigade rescue ladders ($\geqslant$ 25 m) and considered to require some increase in their fire resistance to compensate for the difficulty of fire control.

With the interest in the fundamental aspects of fire protection in recent years, the need for examining the rationale of this approach has been suggested by the research workers, specifiers of safety levels and the design engineers[3]. From a structural point of view the relevant areas of interest are the prediction of the severity of a fire to be expected in a given building and its effects on the structure of the building in the fire zone as well as others remote from it. Consequently a rational fire protection approach should be based on the following:

(a) The probability of a fire

(b) The probable severity of the fire

(c) The response of the structure to the fire

(d) The re-use of the structure after the fire.

Of these the first needs a statistical approach to establish the number of buildings at risk, the frequency of fires, records of fire control and the assessment of damage[4]. Study of such data should provide a predictive capability on the risk attached to different types of occupancies. Other technical, economical and social considerations should allow judgments to be made on the acceptable level of risk in a given situation.

PROBABLE SEVERITY OF FIRE

Over the last ten to fifteen years a number of studies have been carried out, notably in Japan[5], Sweden[6] and the United Kingdom[7] on the post flashover behaviour of a fire. These studies have clearly illustrated that a number of factors, shown below, govern the severity of a fire, of which the amount of fuel is one.

A Fire load: Total quantity and distribution

B Ventilation: Amount and disposition

C Compartment boundaries: Size, shape and thermal characteristics

The quantity of the fire load and its nature represents the total heat potential and rate of availability, roughly represented by the relationship between the surface area and the mass. The amount of ventilation available exercises a critical influence on the burning behaviour of the fuel[8], with restricted ventilation the decomposition rate is proportional to the availability of the air supply up to an optimum point (Figure 2) after which increase in ventilation has little effect. In the first regime the fire severity can be regarded as ventilation controlled and in the second as fuel controlled ie the availability of the fuel or the relationship between its mass and surface area has a critical effect. Ventilation to the fire is available from the windows and can be related to the window size as the glazing is usually destroyed by the time flashover occurs.

The compartment characteristics influence the heat balance of the fire. Some of the heat is dissipated through the exposed surfaces and consequently the surface area and the conductivity of the boundaries may be critical. In large compartments the progress of the fire may be by stages and the whole compartment may not undergo flashover conditions at the same time.

The traditional method of expressing the severity of a fire has been to relate it to a period of exposure in the standard fire resistance tests which follow the standard temperature/time relationship such as that specified in ISO 834 : 1975[9] . A simplified expression to take account of different factors allows the expected temperature conditions to be related to the standard curve by an expression of the following type[7]:

$$t_f = k \frac{L}{\sqrt{A_W \cdot A_T}}$$

where t_f = equivalent fire resistance time, L = fire load, k = fuel factor for the fire load, A_W = window area A_T = compartment surface area.

Another approach defines the temperature/time relationship for each situation and therefore provides a family of curves, with partially standardized heating and cooling rates. A comparison between the three approaches is shown in Figure 3.

THE LIMIT STATE APPROACH

Both the Comité Européen de Beton(CEB)[10] and ISO [11] (International Organization for Standardization) have adopted a semi-probablistic approach to the design of structures so that the structure will not become unfit for its intended function during its useful life ie it will not reach a limit state. CEB explains that 'The initial idea of referring to a single failure criterion has been replaced by the comprehensive concept of limit states'. A practical effect of this approach has been to consider the characteristic strength of the structure and the characteristic loads to which it will be subjected and to replace the global or overall safety factor by partial safety factors, each appropriate to the limit state being considered. The two limit states specified in a recent British Code[12] are the ultimate limit state and the serviceability limit state, the latter being concerned with deflections and widths of cracks in concrete. The characteristic load (W_k) can be defined as the load which is not likely to be exceeded during the useful life of the structure and the characteristic strength (S_k) as the strength that is normally expected to be exceeded. To take account of the effects of fire two special limit states need to be considered, one concerns the maintenance of stability and corresponds to the ultimate limit state and could be termed the 'limit state of stability' in a fire and the other the maintenance of integrity of the space separating components of a structure and could be termed the 'limit state of integrity' in a fire. These limit states are diagrammatically shown in Figure 4 together with the factors which influence their occurrence.

LIMIT STATE OF STABILITY

Assuming that a fire is likely to occur in a building and reach the post flashover stage without control it would subject the structure to high temperature conditions which have the effect of reducing its characteristic strength. If the probable severity of the fire is known or predictable, the design of the building should be such that the reduction in the characteristic strength is not sufficient to decrease it to the characteristic load level otherwise the structure will become unstable and collapse. Reduction in strength will be caused primarily by the heating of the materials used in its construction (eg steel and concrete); increased stresses and redistribution of stresses due to thermal movement and thermal restraint, deformation due to unequal heating, creep and physical rupture of some materials at high temperatures.

Some practical considerations may necessitate the imposition of additional requirements such as a limit on the deformation of floors and beams, prevention of progressive collapse, the need to retain a margin of residual strength after fire or the need to repair a building quickly particularly after a minor fire. These considerations will require the introduction of partial safety factors Figure 5 illustrates different factors which have to be considered in this connection.

The most important consideration from a structural point of view is the ability to estimate reduction in the characteristic strength and the onset of instability. The amount of reduction in the characteristic strength would depend upon the severity of fire, properties of the constructional materials at high temperatures and the design of the structure as shown in Figure 6.

The severity of fire specifies the exposure conditions and consequently the temperature regimes in various parts of the construction and at different depths in materials Data on material properties show the losses in physical properties which have been suffered and the consequent reduction in the strength of the structure. The design of the structure allows an analysis to be carried out to find the time at which the loss in strength approaches the critical limit state. The non-steady heating regime leads to a progressive reduction in strength which for simple cases can be fairly simply illustrated as in Figure 7, where two beams or floors are shown with the normal and the limit state moment distribution curves. The time taken for the ultimate moment capacity to be lowered to the same level as the applied or the design moment is the time to reach the limit state of stability.

For this analysis appropriate partial safety factors need to be established as shown in the example below.

If the characteristic load on the structure is assumed to be

$$W_k = W_o + k_1 \sigma_w$$

and its characteristic strength as

$$S_k = S_o - k_2 \sigma_s$$

where W_o and S_o are the mean load and the mean strength respectively k_1 and k_2 are the probability factors for load and for strength and σ_w and σ_s are the standard deviations

$$\text{The global safety factor } \lambda = \frac{S_o - k_2 \sigma_s}{W_o + k_1 \sigma_w} \qquad \ldots\ldots \quad (1)$$

The exposure of the structure to a fire for time 't' will result in the strength being reduced to S_t, then

$$S_t = \gamma_t (S_o - k_2 \sigma_s)$$

γ_t being the reduction factor due to heating At the limit stage of stability

$\lambda = 1$ and therefore

$$\gamma_t (S_o - k_2 \sigma_s) = (W_o + k_1 \sigma_w)$$

ie the strength reduction factor has the same value as the global safety factor. Consequently the structure is on the verge of collapse at time t. In many practical situations it is desirable, and in some cases essential, to prevent this happening and an additional factor γ_a is used to amend the value of the characteristic load. The value of γ_a will vary between 1.0 and 1.5 depending upon the additional needs and following are some examples of the way in which its value could be adjusted:

limiting deflection criterion	=	1.1
residual strength criterion	=	1.2
tall structures	=	1.25
repairability criterion	=	1.3

LIMIT STATE OF INTEGRITY

This limit state is only applicable to those elements of construction which have a separating function to perform ie walls and floors. Even if these retain their structural stability it is still possible for fire penetration to occur in two ways. Excessive transfer of heat through the construction can raise the temperature of the face remote from the fire to a point at which combustible materials in contact are likely to become ignited. The other way is by the passage of hot gases and flames through gaps, openings, cracks or orifices. Factors which influence integrity failure are shown in Fig 8 below.

Heat transfer under the non-steady heating conditions is determined by the thermal diffusivity ($\alpha = k/\rho c$) of the barrier which is influenced by the thermal constants, the moisture content and the existence of air gaps. For materials such as concrete data are available to estimate the contribution made by a known quantity of moisture to delay the transfer of heat. Whilst it is possible to calculate heat transfer under the unsteady state by approximate methods, data are lacking on the precise thermal properties of materials at relatively high temperatures.

Flame barrier limit state is purely a mechanical feature of the construction and generally is not critical with monolithic constructions, masonry work, precast concrete blocks or panels 100 mm or more in thickness or constructions with a protective coating of plaster, asbestos or mineral fibres. Most problems due to the formation of gaps or openings are experienced with fabricated constructions where dry joints occur and particularly where combustible materials are involved. The solution lies in providing allowance for the expansion of metallic components, absence of through openings, staggering the joints and using sealing materials of an inert type.

The higher pressure on the fire side causes hot gases to flow through the gaps and orifices, the rate of flow depends upon the square root of the pressure difference, the area of the gap and the flow characteristics. Flames will find it difficult to pass through gaps of less than 5 mm width but hot gases, smoke and other products of combustion can penetrate in large quantities. These may not create a fire situation on the other side but are more than likely to lead to an unbearable atmosphere for the occupants. Safety considerations for the occupants demand that the quantity of gases so penetrated should be minimal.

CONCLUSION

Fire safety principles for high rise buildings should follow a rational approach proposed in this paper as a part of which the structural behaviour can be analysed using a limit state concept. This needs to be developed more fully into a set of relationships which form an adjunct to the normal analysis techniques.

ACKNOWLEDGEMENT

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SUMMARY

Structural fire protection in buildings should follow a rational philosophy and take account of the probability as well as probable severity of a fire. In analysing structurel behaviour the limit state concepts can be applied with a limit state of stability as a universal requirement and a limit state of integrity for separating structures. Partial safety factors need to be determined to deal with limits on deformation, retention of a specified residual strength, re-use after a fire and extra safety for tall structures.

RESUME

On devrait suivre une ligne de conduite logique en ce qui concerne les mesures de protection des structures des bâtiments contre le feu et l'on devrait tenir compte de la probabilité du feu autant que de son importance. Dans l'analyse des réactions d'une structure, le concept de l'état limite peut être mis en pratique en prenant l'état limite de stabilité comme une nécessité d'ordre général et en prenant un état limite d'intégrité pour la séparation entre les structures. Il faut déterminer les facteurs de sécurité partielle pour traiter les limites de déformation, une résistance post incendie donnée, une réutilisation des bâtiments après incendie et pour trouver des mesures de sécurité supplémentaires pour les maisons hautes.

ZUSAMMENFASSUNG

Der bauliche Brandschutz in Gebäuden sollte logischen Ueberlegungen folgen, und sowohl die Wahrscheinlichkeit als auch die wahrscheinliche Schwere eines Brandes in Betracht ziehen. Bei der Analyse des baulichen Verhaltens ist es möglich, das Konzept der Grenzzustände anzuwenden, mit einem Grenzzustand der Standsicherheit als allgemeine Anforderung und einem Grenzzustand der Unversehrtheit für räumlich getrennte Baukonstruktionen. Nötig ist die Festlegung partieller Sicherheitsfaktoren für Fälle von Verformungsgrenzen, von der Beibehaltung einer bestimmten Restfestigkeit, von Wiederverwendung nach einem Brand und von Reservesicherheit für hohe Bauwerke.

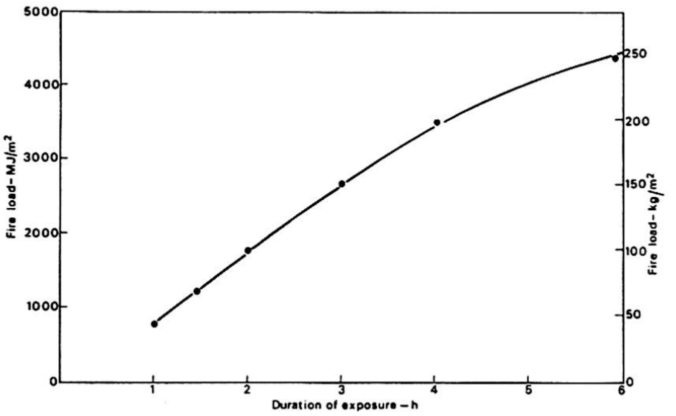

Figure 1 Inberg's relationship for fire severity

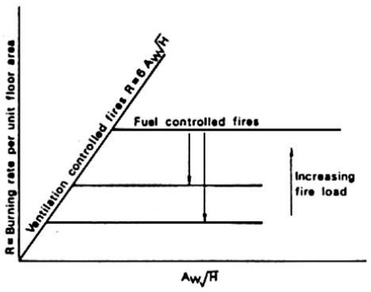

Figure 2 Burning rates of fully developed fires

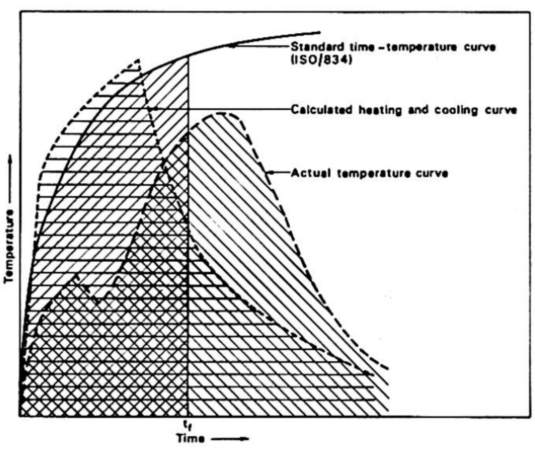

Figure 3 Different methods of expressing fire severity

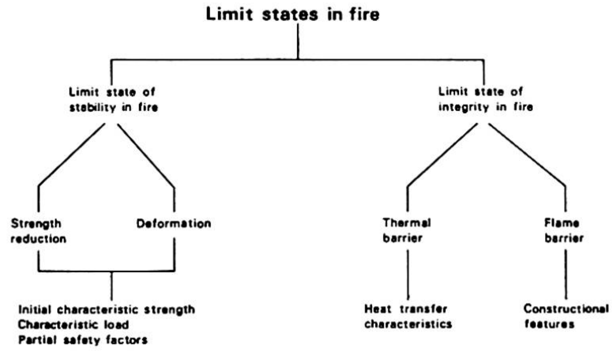

Figure 4 Limit states in fire for structures

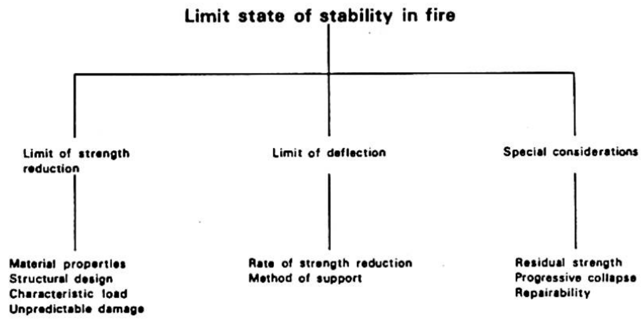

Figure 5 Main factors for limit state of stability in fire

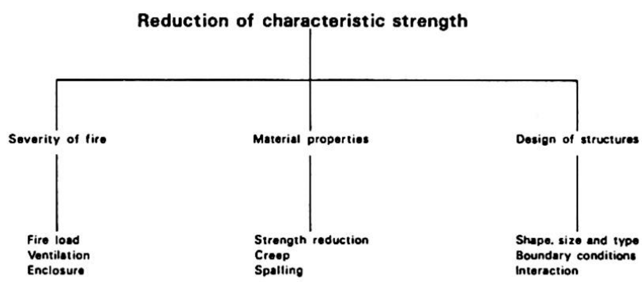

Figure 6 Factors affecting reduction of characteristic strength

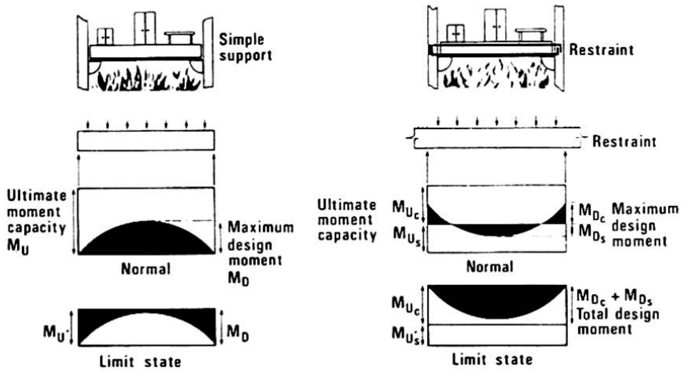

Figure 7 Effect of end restraint of a beam or slab

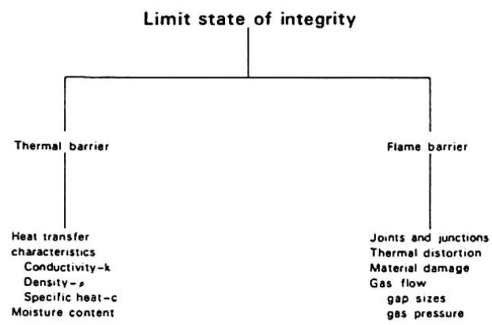

Figure 8 Factors affecting the integrity limit state

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A Differentiated Approach to Structural Fire Engineering Design

Une méthode différenciée pour la détermination de la sécurité au feu des éléments de structure

Ein differenziertes Verfahren für die brandtechnische Dimensionierung von Baukonstruktionen

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A development of analytical design procedures, based on differentiated functional requirements, within different fields of the overall fire safety concept is an important task of the future fire research. Such procedures, successively replacing the present, internationally prevalent, schematic design methods, are necessary for getting an improved economy and for enabling more well-defined fire safety analyses. A derivation of such analytical design systems is also in agreement with the present trend of development of the building codes and regulations in many countries towards an increased extent of functionally based requirements and performance criteria.

For fire exposed load-bearing structures and partitions, an essential step in the direction of the described development was taken in the Swedish Standard Specifications of 1967 by introducing different alternatives of structural fire engineering design, leading to a different degree of accuracy and a different amount of engineering design work. This differentiated view is underlined further in the new edition of the standard specifications, in force from 1976.

A differentiated fire engineering design of load-bearing structures, as approved in the Swedish Standard Specifications, comprises a thorough determination of [1, 2, 3, 4, 5, 6, 7]
(a) the fire load characteristics,
(b) the gastemperature-time curve of the fire compartment as a function of the fire load density, the ventilation characteristics of the fire compartment, and the thermal properties of the structures enclosing the fire compartment,
(c) the temperature-time fields, and
(d) the structural behaviour and minimum load-bearing capacity of the fire exposed structure for a complete process of fire development.

The components of the design system as well as the appurtenant functional requirements are summarized in Fig. 1 for interior load-bearing structures. The survey covers the general case of application with additional requirement on reserviceability of the structure after a fire exposure.

As concerns the fire exposure characteristics, the Swedish Standard Specifications generally permit a structural fire engineering design on the basis of a gastemperature-time curve, calculated in each individual case from the heat and mass balance equations of the fire compartment with regard taken to the combustion characteristics of the fire load, the ventilation of the fire compartment, and the thermal properties of the enclosing structures of the fire compartment.

As a provisional solution, the structural fire engineering design may be based on differentiated gastemperature-time curves of the complete process of fire development, specified in the code. These fire exposure curves, exemplified in Fig. 2, are approximate curves, generally determined on the assumption of ventilation controlled compartment fires [8, 9, 10]. One principle reason for choosing this assumption as a general basis in this connection is dictated by the great difficulty in finding representative values of the free surface area and the

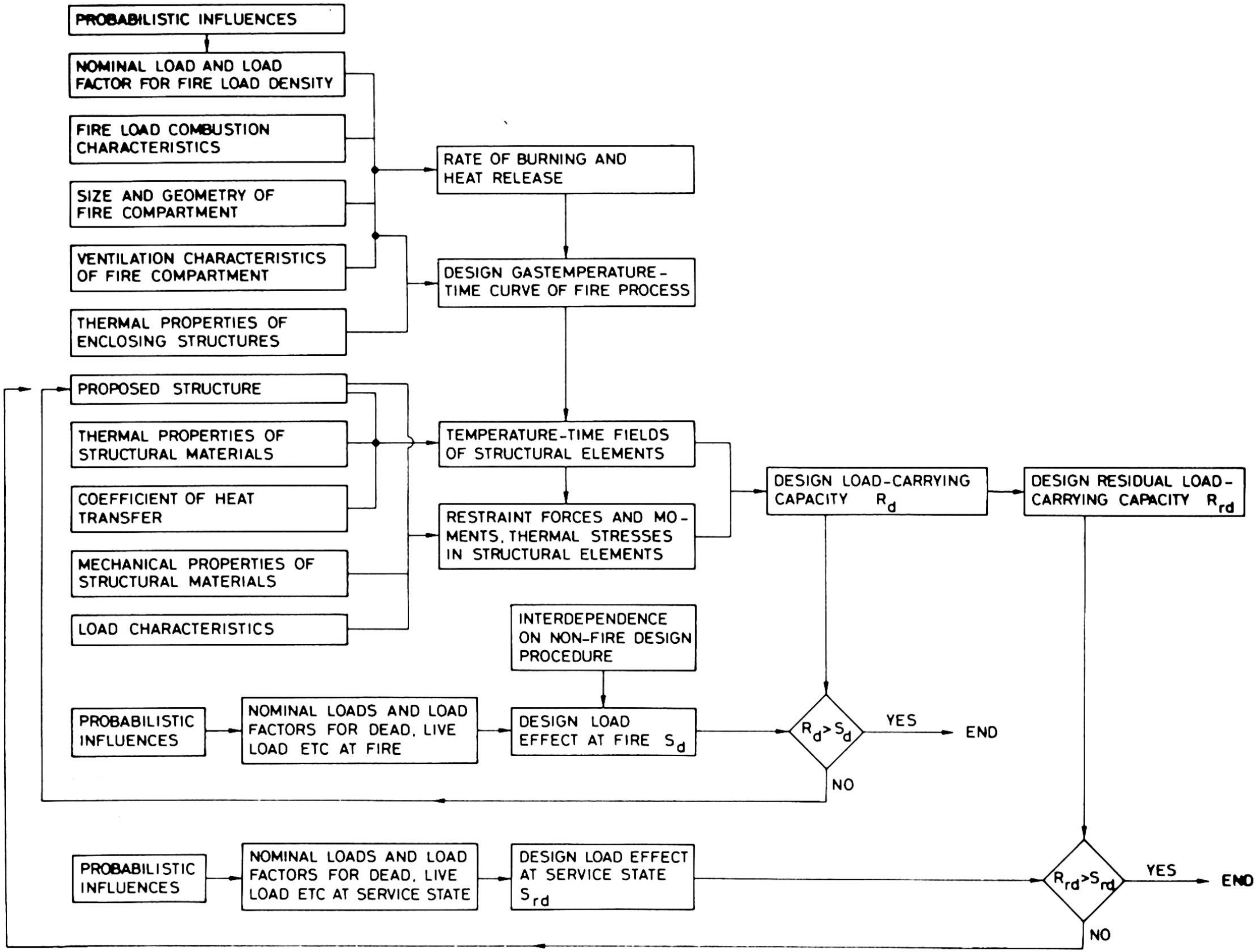


Fig. 1. Procedure of a differentiated fire engineering design of load-bearing structures with additional requirement on re-serviceability after fire

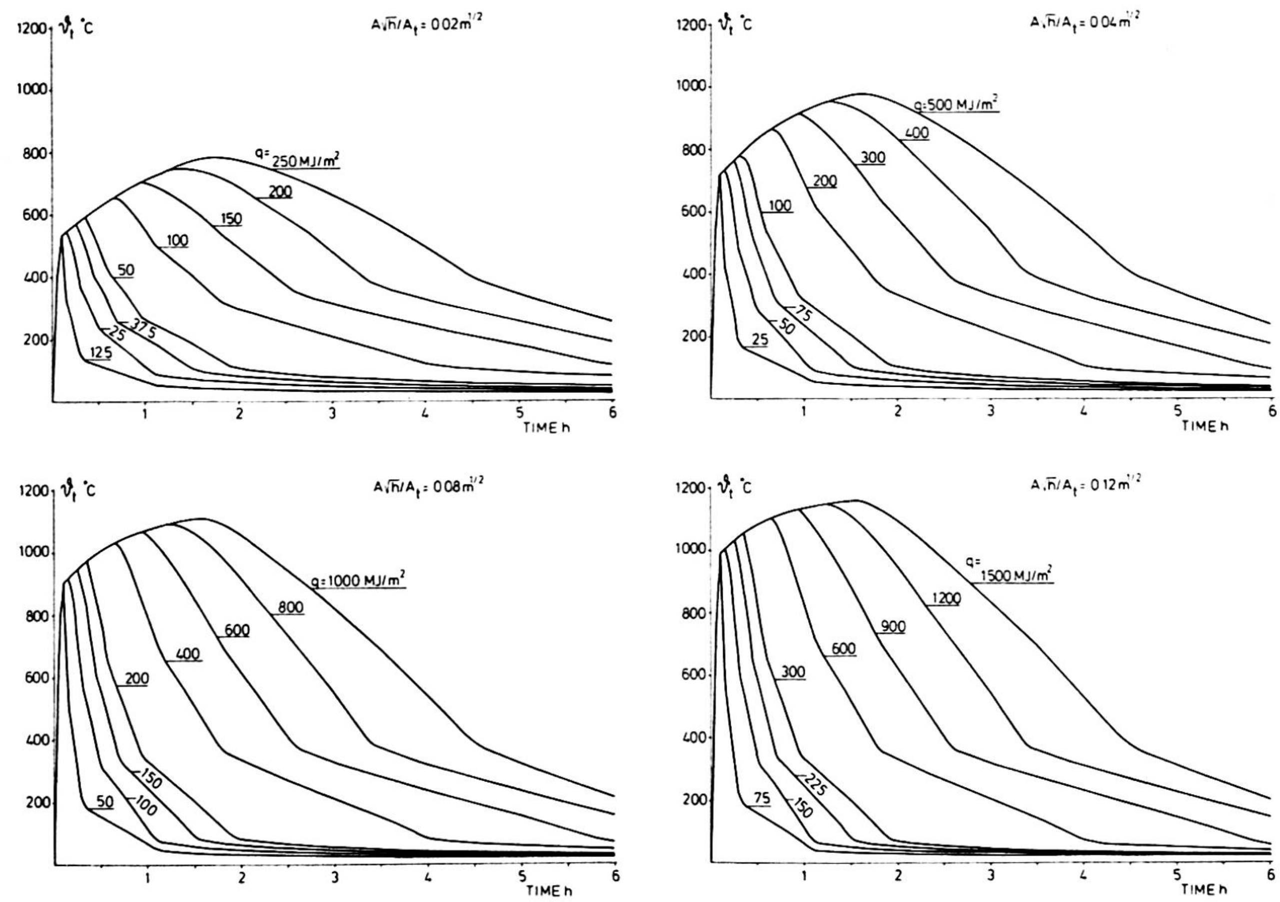


Fig. 2. Gastemperature-time curves ϑ_t-t of the complete process of fire development for different values of the fire load density q and the opening factor $A\sqrt{h}/A_t$. Fire compartment, type A

porosity properties of real fire loads of furniture, textiles, and other interior decorations, which are essential quantities for a combustion description of a fuel bed controlled fire but of minor importance for the development of ventilation controlled fires. Another principle reason is related to the fact that the gastemperature-time curves themselves do not constitute the primary interest of the problem in this connection but an intermediate part of a determination of the decisive quantity, viz. the minimum load-bearing capacity of the structure during a complete fire process. For fuel bed controlled fires, the assumption of ventilation control leads to a structural fire engineering design which will be on the safe side in practically every case, giving an overestimation of the maximum gastemperature and a simultaneous, partly balancing, underestimation of the fire duration. For the minimum load-bearing capacity, the gastemperature-time curves specified in the code are giving reasonably correct results, which has been verified in [3, 4, 10].

The fire exposure curves, specified in the code, apply to a compartment with surrounding structures of a material with a thermal conductivity $\lambda = 0.81\ W \cdot m^{-1} \cdot {}^oC^{-1}$ and a heat capacity $\rho c_p = 1.67\ MJ \cdot m^{-3} \cdot {}^oC^{-1}$ - fire compartment, type A. Entrance parameters for the curves are the fire load density q ($MJ \cdot m^{-2}$), and the ventilation characteristics of the fire compartment, expressed by the opening factor $A\sqrt{h}/A_t$ ($m^{1/2}$). A = the total area of the window and door openings (m^2), h = the mean value of the heights of window and door openings, weighed with respect to each individual opening area (m), and A_t = the total interior area of the surface bounding the compartment, opening areas included (m^2). The fire load density q is defined according to the formula

$$q = \frac{1}{A_t} \Sigma\, m_\nu H_\nu \qquad (MJ \cdot m^{-2})$$

where m_ν = the total weight (kg), and H_ν = the effective heat value ($MJ \cdot kg^{-1}$) for each individual combustible material ν of the fire compartment.

In the design procedure, a transfer can be done between fire compartments of different thermal properties of the surrounding structures according to simple rules, based on fictitious values of the opening factor and the fire load density [3, 4, 5, 7]. By introducing such a transfer system, design diagrams and tables - facilitating a practical application - can be limited to one type of fire compartment, viz. type A.

A differentiated design according to the described procedure can be carried through in practice today in a comparatively general extent for fire exposed steel structures. The practical application then is facilitated by the availability of a manual [4], comprising a comprehensive design basis in the form of tables and diagrams which directly are giving the maximum steel temperature for a differentiated, complete fire process and the corresponding load-bearing capacity. The manual has been approved for a general practical use in Sweden by the National Board of Physical Planning and Building.

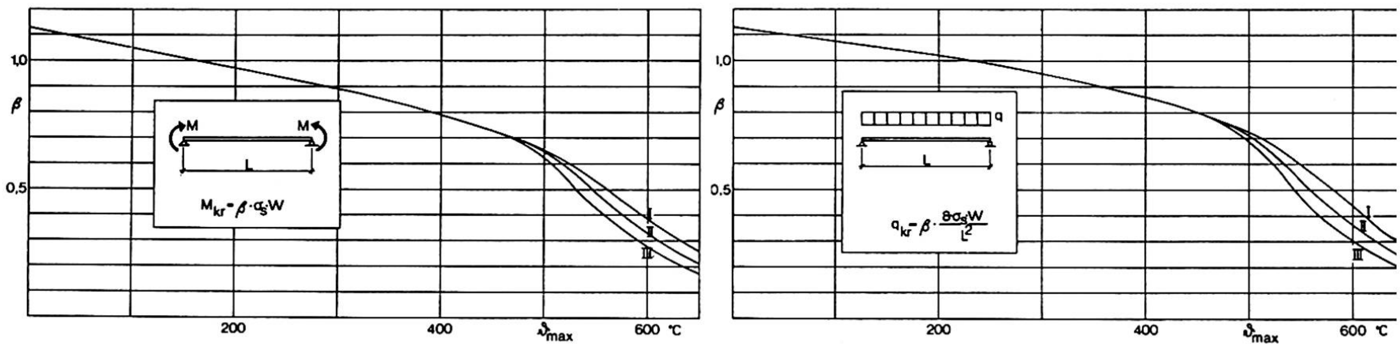


Fig. 3. Load-bearing capacity (M_{kr}, q_{kr}) for two types of loading at a simply supported steel beam of constant I cross section. Curves I, II, and III correspond to a rate of heating of 100, 20, and $4^oC \cdot min^{-1}$, respectively, and a rate of subsequent cooling = 1/3 of rate of heating.
ϑ_{max} = maximum steel temperature, σ_s = yield point stress at ordinary room temperature, and W = elastic modulus of cross section

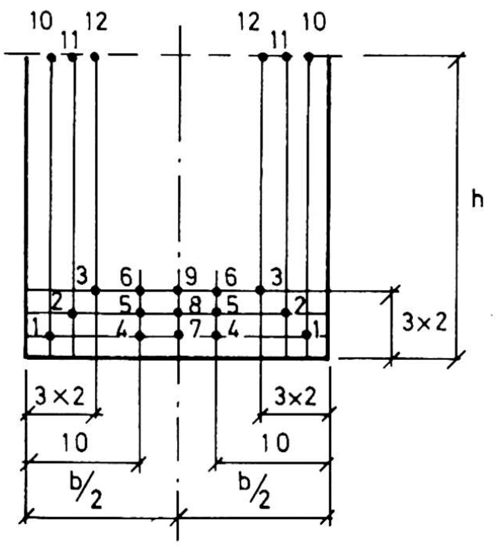
In comparison with steel structures, fire exposed reinforced and prestressed concrete structures generally are characterized by an essentially more complicated thermal and mechanical behaviour. In consequence, the basis of a differentiated structural fire engineering analysis and design is considerably more incomplete for concrete structures - cf., for instance, [5, 6, 7, 11], in which summary reports are given on the present state of knowledge. Completing the manual on fire exposed steel structures [4], another manual is in course of preparation - to be edited by the National Board of Physical Planning and Building - with the purpose to facilitate the practical application of the differentiated design procedure also to other types of load-bearing structures - reinforced and prestressed concrete structures, aluminium structures, and wooden structures. A design guidance for fire exposed partitions of various materials is included, too.

Fragmentary examples of the design basis quoted are given in Fig. 3 [4], Table 1 [4], and Table 2 [7].

Table 1. Maximum steel temperature ϑ_{max} for a fire exposed, insulated steel structure at varying fictitious fire load density q_f ($MJ \cdot m^{-2}$), and structural parameter $A_i\lambda_i/V_s d_i$ ($W \cdot m^{-3} \cdot {}^oC^{-1}$). Fictitious opening factor $(A\sqrt{h}/A_t)_f = 0.04\ m^{1/2}$. A_i = interior jacket surface area of insulation per unit length (m), d_i = thickness of insulation (m), λ_i = thermal conductivity of insulating material ($W \cdot m^{-1} \cdot {}^oC^{-1}$), and V_s = volume of steel structure per unit length (m^2)

q_f	$A_i\lambda_i/(V_s d_i)$												
	50	100	200	400	600	1000	1500	2000	3000	4000	6000	8000	10000
25	25	35	50	70	85	115	140	170	210	245	290	330	365
50	35	50	75	115	150	200	245	290	350	395	450	505	540
75	45	65	100	155	200	260	325	380	450	500	565	615	650
100	50	80	125	190	245	320	395	450	525	575	640	685	715
200	85	135	210	310	385	490	575	635	710	755	800	825	835
300	115	180	275	410	500	615	700	755	815	845	875	890	895
400	140	225	345	505	605	720	800	845	890				
500	170	270	415	585	685	790	860	895					

Table 2. Maximum temperature ϑ_{max} during a complete process of fire development in different points of a rectangular concrete beam, fire exposed from below on three surfaces, at varying values of the fictitious fire load density q_f($MJ \cdot m^{-2}$), and the cross-sectional width b (m). Fictitious opening factor $(A\sqrt{h}/A_t)_f = 0.04\ m^{1/2}$. The temperature values are computed for a cross-sectional height h = 0.2 m but are applicable with sufficient accuracy also to other values of h > 0.2 m



q_f	b/2	1	2	3	4	5	6	7	8	9	10	11	12
50	0.04	345	260					300	260	240	225	215	
	0.06	335	185	170				230	185	170	210	140	140
	0.08	335	180	135				210	145	135	205	115	105
	0.10	335	180	125	210	125	105	210	125	105	205	110	95
	0.125	335	180	120	210	125	100	205	110	95	205	110	85
	0.15	335	180	120	205	120	95	205	105	90	205	105	85
	0.20	335	180	120	205	115	95	205	105	85	205	105	85
	0.30	335	180	120	205	110	95	205	105	85	205	105	85
100	0.04	500	425					465	425	400	380	370	
	0.06	480	325	295				370	315	295	315	245	245
	0.08	480	300	235				325	255	230	305	185	180
	0.10	480	295	210	315	215	190	315	215	190	305	180	140
	0.125	480	295	200	310	200	170	305	180	150	305	175	125
	0.15	480	295	200	310	190	155	305	175	135	305	175	125
	0.20	480	295	200	310	185	150	305	175	120	305	175	120
	0.30	480	295	200	305	180	150	305	175	120	305	175	120
200	0.04	690	610					655	610	585	570	555	
	0.06	650	495	460				545	485	460	460	400	395
	0.08	645	450	375				480	400	370	440	310	300
	0.10	645	435	335	455	345	315	455	345	315	435	280	235
	0.125	645	435	315	450	325	270	435	295	245	435	275	200
	0.15	645	435	315	445	305	245	435	280	210	435	270	195
	0.20	645	430	315	440	300	240	435	270	190	435	270	190
	0.30	645	430	315	440	295	235	435	265	190	435	270	190
300	0.04	795	740					775	740	720	705	690	
	0.06	755	625	585				670	610	585	580	520	520
	0.08	740	570	490				600	515	485	535	415	400
	0.10	740	550	440	565	455	415	565	455	415	525	370	320
	0.125	740	545	415	550	425	370	535	390	335	525	355	275
	0.15	740	540	410	535	400	330	525	365	290	520	350	265
	0.20	740	540	410	535	390	320	520	350	260	520	350	260
	0.30	740	540	410	530	385	315	520	345	250	520	345	255

The summarily presented, differentiated design procedure is to be seen as an attempt to build up a logical system for a structural fire engineering design, based on functional requirements. The system is well devoted to stimulate the architects and structural engineers to solve the fire engineering problems in a qualified way over a design procedure which is equivalent to the non-fire, structural design, conventionally applied. The design system is not homogeneous, as regards the present basis of knowledge for the different design steps, which could be put forward as a criticism of the system. However, such a remark is not essential. Instead, this fact should be used as an important information on how to systematize a future research for enabling a successive improvement of the design system

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SUMMARY

On the basis of the general functional requirements, a differentiated, analytical procedure is presented for a fire engineering design of load-bearing structures. Examples are given. The method is approved for a general practical use in Sweden by the National Board of Physical Planning and Building.

RESUME

Sur la base des fonctions générales des constructions une méthode analytique différenciée est présentée pour la détermination de la sécurité au feu des éléments de structure. Des exemples sont donnés. La méthode est approuvée par les autorités pour l'utilisation pratique en Suède.

ZUSAMMENFASSUNG

Es wird ein differenziertes Verfahren für die brandtechnische Dimensionierung von Baukonstruktionen beschrieben, das sich auf direkte Funktionsforderungen stützt. Das Verfahren ist von den Behörden für eine generelle, praktische Anwendung in Schweden zugelassen.

III b

Calcul et conception des structures métalliques ou mixtes en vue de leur résistance à l'incendie

Bemessung von Stahl und Verbundbauwerken gegen Brandeinwirkungen

Design of Steel and Composite Structures for Fire Resistance

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Creep Buckling of Steel Column at Elevated Temperatures

Flambage par fluage d'un poteau en acier aux températures élevées

Kriechknicken von Stahlstützen bei hohen Temperaturen

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1. INTRODUCTION

The buckling strength of a steel column may be considerably reduced due to exposure to elevated temperatures during a fire. This reduction is now taken into account by use of a chart where the buckling stress for the steel material is plotted versus the slenderness ratio for each temperature considered [1]. Such curves have been obtained by introducing in conventional room-temperature buckling formulas the mechanical properties determined from standard material tests at the various temperatures. There is a large variation between curves for temperatures exceeding 500°C published by different authors. The reason is probably that the creep rate of ordinary structural carbon steels increases rapidly at this temperature. The material tests rather arbitrarily include creep during the time taken to increase the load to ultimate failure. During a fire the column is usually subjected to constant load during the whole heating period implying a larger creep deformation. Furthermore, creep buckling has a non-linear course, rendering the present design procedure an unconservative approximation.

In order to establish the basis of a more reliable method, including the time parameter, for determining the collapse load of a column in a fire, a study has been made of a hinged steel column of I-section with an initial deflection, subjected to elevated temperatures, mainly 600°C but also 550 and 650°C. Material creep tests were carried out at 600°C, the results being extrapolated to other temperatures by use of the Dorn-theory. Creep constants were determined and introduced into a computer programme providing the creep life at given constant stress and temperature. By performing a large number of such calculations at different stress levels a diagram was obtained giving the buckling stress versus the slenderness ratio for various times of exposure to the temperature considered. The computer programme was also modified to allow a realistic variation of temperature history and computations were run to determine the critical stresses corresponding to maximum temperatures of 600 and 650°C.

2. CREEP LAW AND CREEP TESTS

Standard creep tests are performed on material coupons at constant load and temperature, giving a relationship between strain ε and time t.

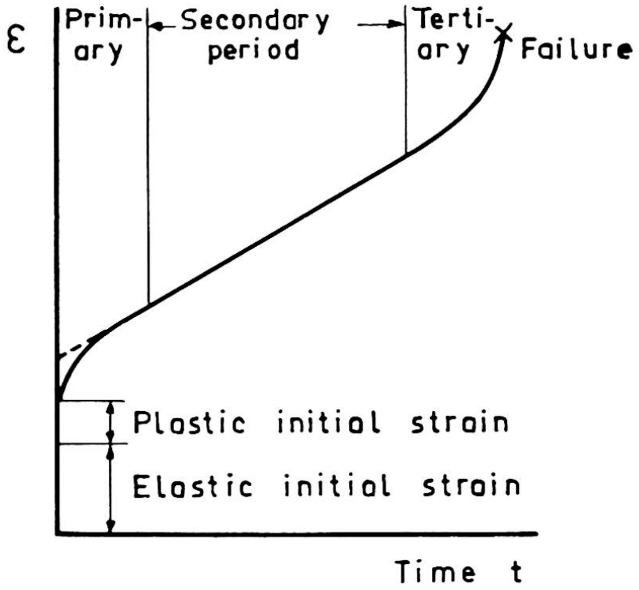


Fig 1

Creep curve for metal at constant load and constant temperature

A creep curve typical for metals at elevated temperatures, Fig 1, includes three phases of which the secondary creep is dominating. In modern metal creep research the creep law of Norton-Odqvist is normally used

$$d\varepsilon/dt = \dot{\varepsilon} = k\sigma^n \tag{1}$$

where σ is the constant stress and k and n are creep constants belonging to the temperature applied. It was found by Dorn that the creep rate $\dot{\varepsilon}$ may be determined for other elevated temperatures by introducing a temperature compensated time parameter

$$\theta = \int_0^t \exp(-\Delta H/RT)dt \tag{2}$$

where ΔH and R are constants and T the temperature in $^\circ$K. Harmathy [2] carried out several creep tests and established a generalized creep curve, based on Dorn's theory, for ASTM A36 steel valid for temperatures of 400-700°C. Results of creep tests within the same temperature range have also been published by Thor[3].

To obtain creep data for calculations of critical times to buckling, creep tests were run in tension at 600°C with four constant stress levels σ = 30, 40, 50 and 60 MPa. The material coupons were made of a carbon steel with yield strength 300 MPa and ultimate strength 460 MPa, i.e. rather similar to A36. The creep rates determined gave the creep constants

$k = 1.88 \times 10^{-11}$ $n = 4.9$

For other temperatures the Dorn theory was used to obtain creep rates, introducing $(\Delta H/R) = 39000\ ^\circ$K as found by Harmathy for A36. This gives a constant value of n for all temperatures, while

$$k = 1.88 \times 10^{-11} \exp(44.7 - 39000/T) \tag{3}$$

3. THEORY OF CREEP BUCKLING

In a column having an initial maximum deviation w_o from a straight line and subjected to an axial load P with an excentricity a, Fig 2,

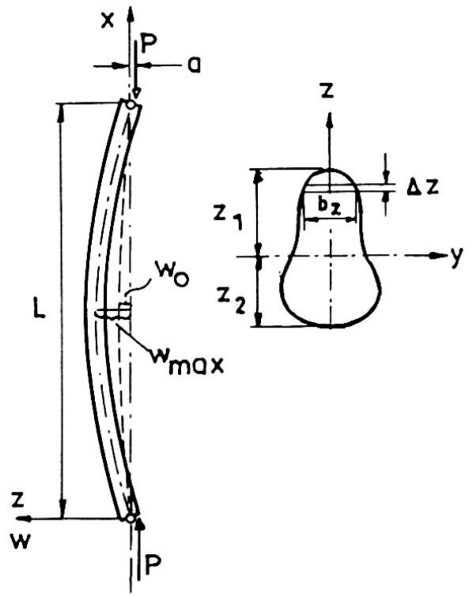


Fig 2 Initially curved column hinged in both ends, with cross-section of single symmetry and eccentric axial load P

a bending moment will occur increasing the deflection to w_{max}. If the load is much smaller than the short-time buckling load, and no bending out of the xz-plane can take place, the increase is rather small but if the load is kept constant and creep sets in, the deflection increases with time. The creep strain rate at a distance z from the CG-axis of a section, Fig 2, may be written

$$\dot{\epsilon}_x = \dot{\sigma}_x/E_o + k\sigma_x^{\;n} \qquad (4)$$

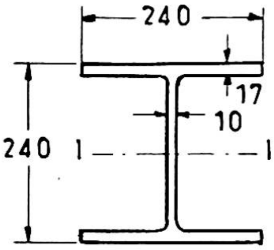
where σ_x is the compression stress which is continuously growing with increasing deflection. E_o is a modulus taking elastic and plastic deformation, and possibly also primary creep, into account. The second term represents the secondary creep. A constant n considerably larger than one will obviously cause a fast acceleration of the deflection w_{max} with growing stress.

Creep buckling theories and approximate solutions of the creep buckling life t_k for metal struts were first published by Hoff and Hult. Closed solutions for more general cases present considerable mathematical difficulties. Samuelson [4] developed a computer programme for a hinged column of singly symmetrical constant section subjected to constant load and temperature. The cross section was divided into thin layers of thickness Δz and width b_z, while the length L was split up into elements Δx and time into intervals Δt. This programme was used for evaluating the critical time for different loads on a column with varying slenderness ratios, and also modified to allow a variation of the temperature between time intervals.

4. DISCUSSION OF COMPUTED CREEP LIVES

The numerical analysis was carried out for a column section HE240B, Fig 3, assuming no excentricity, but an initial deviation according to Dutheil

$$w_o = 4.8 \times 10^{-5}\, L^2/d = 4.8 \times 10^{-5} L^2/0.12 = 4 \times 10^{-4}\, L^2$$



$A = 1.06 \times 10^{-2}\ m^2$

$I_1 = 1.13 \times 10^{-4}\ m^4$

$i_1 = 0.103\ m$

$d = H/2 = 0.12\ m$

Fig 3

Column section HE 240B used in computations

The modulus of elasticity E_o of Eq(4) was determinad from a formula proposed by Thor [3].

$$E_o = 325000 - 404\ \vartheta_s \quad \text{MPa} \tag{5}$$

The creep buckling was defined as the moment when w_{max} exceeded twice the height of the section in the buckling direction giving a very high creep rate. Critical creep times at 600°C were determined for columns of four different lengths L = 3, 4.5, 6 and 9 m, yielding slenderness ratios $\lambda = L/i_1$ = 30, 45, 60 and 90. A number of different mean stresses were treated for each column length. The results of the computations are presented in Fig 4, where the creep buckling time is plotted versus the compression stress of the column for each slenderness ratio. Creep lives were also obtained for the steel temperatures ϑ_s = 550 and 650°C, assuming in both cases λ = 45, while σ was 70 and 35 MPa respectively. These results are entered into Fig 4 as isolated

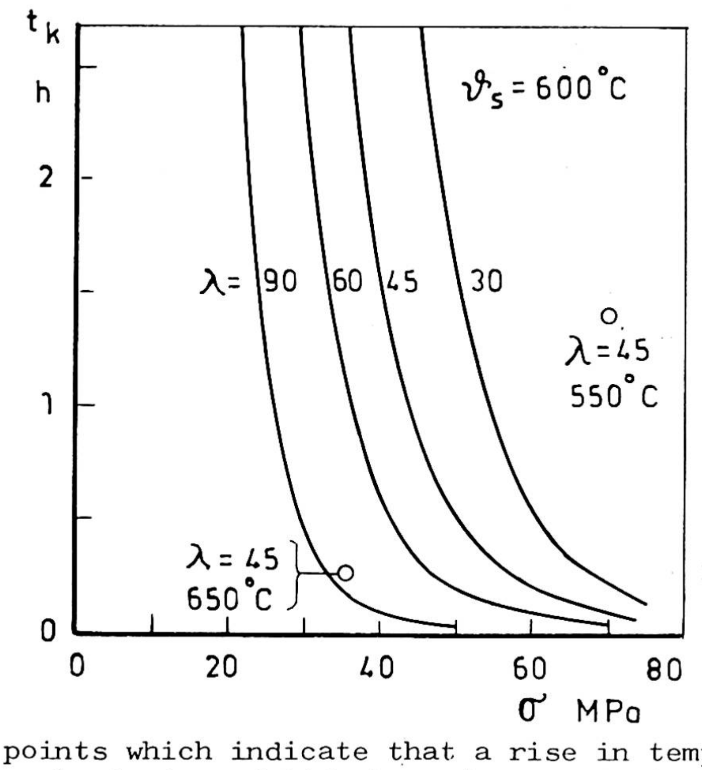


Fig 4

Creep buckling time versus compression stress for various slenderness ratios at 600°C

points which indicate that a rise in temperature of 50°C corresponds to a shortening of life by a factor of 10, or a decrease in stress by about 40 per cent.

The curves of Fig 4 are replotted in Fig 5, giving the buckling stress versus the slenderness ratio for exposures to 600°C from 0.2 to 2 h. The buckling curves presented by Kawagoe-Saito [1a] and Sfintesco [1b] are introduced for comparison. While the former is extremely conservative, corresponding to several hours of heat exposure, the latter seems to be

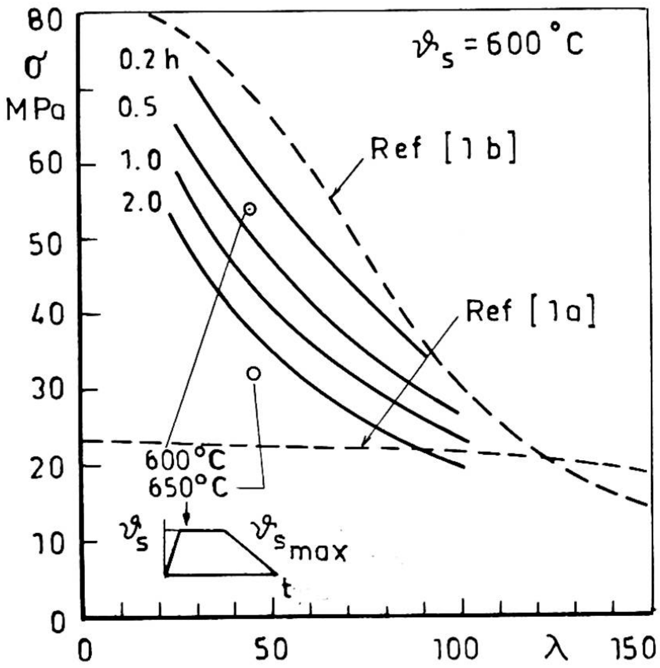


Fig 5

Buckling stress versus slenderness ratio for various times of exposure to $600^\circ C$

unsafe even for a few minutes of $600^\circ C$.

In a fire the temperature of the steel structure is normally gradually rised from room temperature to a maximum determined e.g. by the fire load, after which the cooling starts. A temperature-time history according to Fig 6 was introduced in the computer programme assuming $\vartheta_{s\ max} = 600$

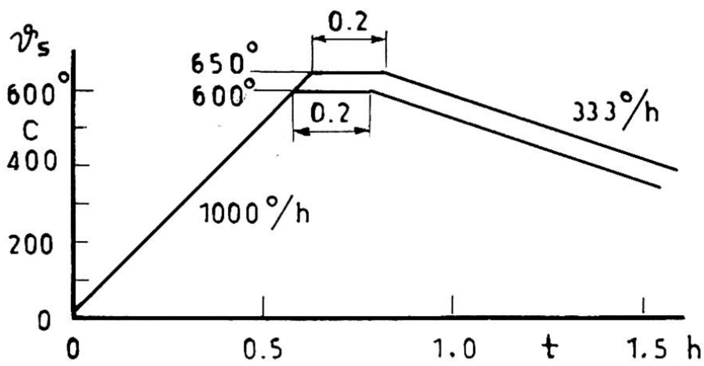


Fig 6

Temperature-time history introduced into computer programme

and $650^\circ C$. Using the same column section as before, the slenderness ratio $\lambda = 45$, the stress was varied to allow interpolation of the value just causing collapse during a temperature cycle. These stresses are also plotted in the diagram, Fig 5.

Although it may be objected that still a number of factors remain to be considered in a realistic analysis of the behaviour of a steel column during fire, the results of the calculations clearly show that an analysis of creep buckling is worth-while.

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SUMMARY

The creep buckling life of a steel column is determined by feeding data from standard creep tests into a computer programme. It is shown that the effect of creep on the buckling strength is very important at temperatures around 600 °C.

RESUME

L'évolution du flambage par fluage d'un poteau en acier est déterminé au moyen d'un calcul numérique, dans lequel on introduit les résultats des essais de fluage standard. Il est montré que l'influence du fluage est très important aux températures de 600 °C environ.

ZUSAMMENFASSUNG

Die Belastungsdauer einer Stahlsäule bis zum Kriechknicken wird durch ein numerisches Programm bestimmt, wobei man Dehnungsmessungen von Standardkriechversuchen benutzt. Es wird gezeigt, dass dem Kriechen bei Temperaturen um 600 °C grosse Bedeutung zukommt.

Fire Resistance of Steel Structures in Neighbouring Fire

Résistance au feu des structures métalliques à proximité d'un incendie

Feuerwiderstand von Stahlkonstruktionen in benachbarten Brandstätten

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1. Thermal effects on bridge members.

It is not so usual to take so large thermal increment as to be produced by fire into considerations in design of bridges. But the anticipated high temperatures and long duration of fire in large scale oil plants forced us to do some studies on their effects against structural members of continuous truss bridges.

In our cases on preliminary design for Meiko Big Bridge and Bannosu Bridge, the effects of pressure by heated air enclosed in members with box type section, the effects of temperature differences between plates in a member, and the effects of temperature differences between members in three span continuous truss girder bridges were studied.

1-1. The effect of internal pressure.

The sections of the members were idealized as right square with width B, and the plate thicknesses are denoted as hf in cover plates and hw inweb plates. These are assumed to be made air-tight with diaphragms at the panel points, the both ends of these members. Then, the increment of inside pressure "P" must be

$$P = \frac{Po}{To}\ t$$

where Po is standard pressure of the air, "To" is the absolute value of normal temperature, and t is average temperature increment of the member.

Expected maximum stress σ_1 in transversal direction of the member can be estimated as

$$\sigma_1 = \frac{P}{2}\left(\frac{B}{h}\right)\left(1+\frac{B}{h}\right) \doteq \frac{P}{2}\left(\frac{B}{h}\right)^2$$

where h is smaller one of hf and hw, and the maximum stress will appear at inside of corners. If B equals to 900 mm, and h is 12mm,

$$\sigma_1 = 9.75\ t \quad (kg/cm^2).$$

1-2. The effects of temperature difference between plates in a member.

The temperature difference between plates will causes transversal and longitudinal stresses in a member.

Transversal maximum bending stress σ_2 will be estimated easily neglecting small amount of in-plane stress as

$$\sigma_2 = \frac{3E\alpha\Delta t}{2(1-\nu^2)\ \{1+3(Hf/hw)^3\}\quad(B/hf)},$$

where Δt denotes the temperature difference, E is Young's modulus, ν is thermal elongation coefficient, and α is Poisson's ratio. This stress appears also at member corners, and if B equals to 900 mm, and hf 45mm, hw45mm,

$$\sigma_2 = 0.52\ \Delta t \quad (kg/cm^2).$$

It seems not so dangerous, but must be added to σ_1 at corners of shaded/side of the members.

Longitudinal thermal stress σ_3 may appear by constraints against bending of the member, and they must be affected by boundary conditions at panel points. But we may assume safely that the panel points do not rotate at all. Then the stress σ_3 will be given as

$$\sigma_3 = \frac{E}{2}\alpha\Delta t = 12.6\ \Delta t \quad (kg/cm^2).$$

1-3. Effects of temperature difference between truss members.

The temperatures of each member by the fire had to be estimated, at first, and the following approximated assumptions were adopted to this end:

a) The temperature of a member depends on the view factor of the surface of plate facing to the fire, and the relation can be represented approximately linear in the region of present temperature under consideration.

b) View factor itself varies inversly as square of distance from the member to the center of fire, and is proportion to cosine of the angle between direction to the fire and the normal line of the plate.

The view factors of members shaded by another members or deck of the bridge are so smaller than the ones facing the fire. The results of more or less troublesome calculation considering shading effects on Bannosu bridge showed us very lower temperature increments in members at shaded side of the structure (Fig. 1). In this case, upper chords of shaded side of the bridge have been affected largely by the wide deck slab. As the Meiko Big Bridge have been designed as double deck bridge, behavior of the lower chord members will be almost the same to the upper chords of Bannosu Bridge.

In any case, no trouble will happen if the structure is statiscally determinate. But the continuous bridges are indeterminate, and the constraints at supports will make some effects on their stresses. Theoretical calculation of member stresses σ_T knowing each temperature increments is not so difficult one, and the maximum effect have been found at a chord member near the one of center piers in our cases. The estimated maximum stress due to the temperature differences reached $13.7\ \Delta T$ kg/cm^2 in Meiko Big Bridge and $17.4\ \Delta T$ kg/cm^2 in Bannosu Bridge, where ΔT denotes temperature difference at the nearest section of the bridge to the fire.

1-4. Permissible temperature conditions.

The permissible limit of the fire effects must be considered taking into account the resultant stresses of members and the buckling strength of compressed ones. But for buckling strength, the problem will be not so difficult if it is permitted to use increased allowable unit stresses of standard specifications.

For the stress limits, although local yield will occur at comparatively early stage of the fire because of the resultant effects of bending and in-plane stresses of the plate, the limit of the stresses plate may be considered as the instance when the one of principal stress on surface of the plate reaches yield point σ_Y member axes.

Standing on such considerations, the permissible limit of the temperature conditions can be decided from following three inequalities:

$$\frac{t}{t_0} + \frac{\Delta t}{\Delta t_1} \leq 1.0 \quad \text{(for transversal stress)}, \qquad -\ (1)$$

$$\left| \frac{\sigma_T + \sigma_d}{\sigma_Y} \pm \frac{\Delta t}{\Delta t_2} \right| \leq 0 \quad \text{(for longitudinal stress)}, \qquad -\ (2)$$

$$|\sigma_T| < \beta \sigma_{ca} \quad \text{(for buckling if } \sigma_T \text{ is negative)}, \qquad -\ (3)$$

where

$$t_0 = \frac{577h^2 \sigma_Y}{B}$$

$$\Delta t_1 = \frac{\{1+3(hf/hw)^3\}\ (B/hf)\sigma_Y}{41.54\ (hf/hw)}$$

$$\Delta t_2 = \sigma_Y / 12.6$$

and β is a coefficient smaller than the safety factor, σ_d is dead load stress.

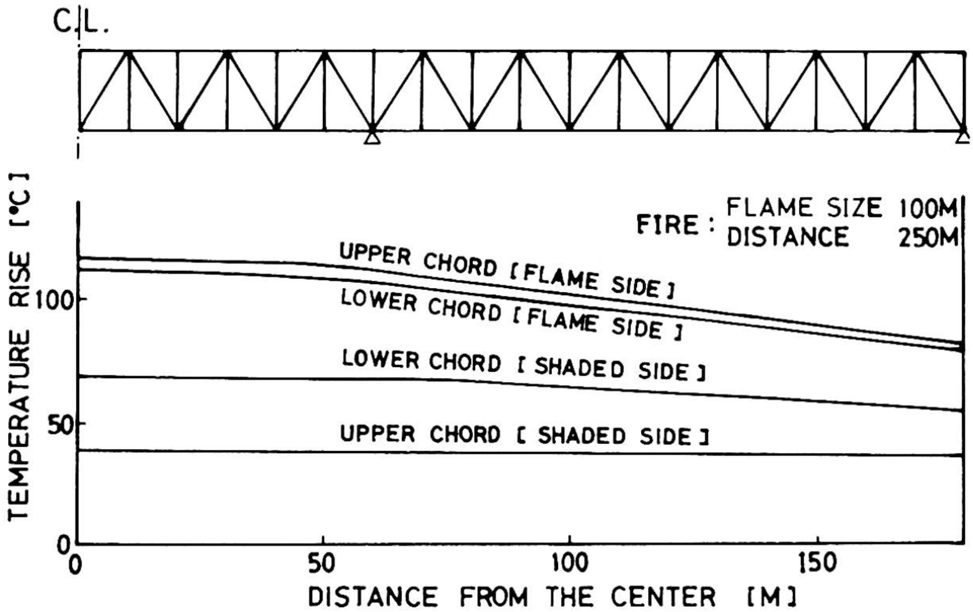


Fig. 1 Estimated temperature rises of cord members

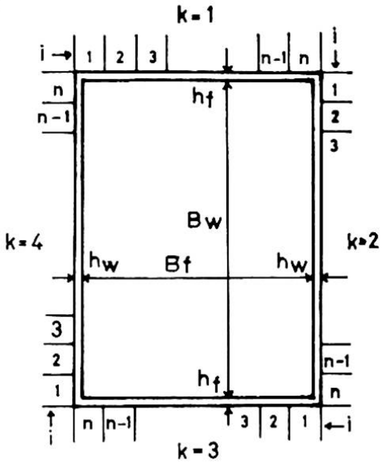


Fig. 2 Mathematical model of cross section of a member

2. Calculation of temperature of a bridge member.

In case of any fire occurring nearby a bridge, radiant heat from the fire should produce some temperature increase which has some harmful effects on bridge members as described above. The temperature increment depends on several conditions such as scale, severity and duration of the fire, distance from the bridge to the fire, and emissivity of surfaces of the member.

First of all an equation was introduced for estimating unsteady-state temperatures on different steel members having box type section. Then the equation was solved to obtain the temperature of a certain member to be used for a main truss girder of a bridge. The calculation was made by use of an electric computer for various values of the view factors for radiative heat exchange between the fire and the surfaces of the member. To examine the effects of paints applied on the surface on the temperature increase, we considered two kinds of paints, an ordinary paint with larger emissivity and an alumina paint with smaller one.

2-1. Equation for calculating the temperature.

Fig. 2 shows a cross section of a steel member as a mathematical model for deriving the equation which enables to calculate the unsteady-temperature distribution of the bridge member exposed to radiant heat from a fire. The equation was obtained from a heat balance on a small rectangular zone (k,i) in Fig. 2, by taking account of the following four heat exchanges;

(1) radiation between the external surface and the surroundings involving the fire
(2) radiation between any internal surfaces
(3) natural convection on the external and internal surfaces
(4) heat conduction in the solids of the member

The equation is written in finite-difference form as shown in Eg. (4), and the solution is accomplished by finite-time step advancement.

$$T(k,i,N+1) = T(k,i,N) + \frac{\Delta\tau}{c\rho h(k)}\Big[\sigma\varepsilon_s\ \{FF(k)T_F(N)^4 - T(k,i,N)^4 - [1 - FF(k)]T_0^4 - (\varepsilon_i/\varepsilon_s)\sum_{\eta=1}^{4}\sum_{j=1}^{n} F_{k\eta}(i.j)\ [T(k,i,N)^4 - T(\eta,j,N)^4] - H_0(k,i,N)\ [T(k,i,N) - T_0] - H_I(k,i,N)\ [T(k,i,N) - \sum_{k=1}^{4}\sum_{i=1}^{n} T(k,i,)/4n] + \frac{\lambda n^2}{B(k)}\Big\{\frac{T(k,i-1,N) - T(k,i,N)}{\alpha(i-1)} + \frac{T(k,i+1,N) - T(k,i,N)}{\alpha(i+1)}\Big\}\Big]$$

---(4)

where $\alpha(i-1) = \alpha(i+1) = B(k)/h(k)$, when $2 \leqq i \leqq n-1$;

$\alpha(i-1) = \alpha$, when $i = 1$; and $\alpha(i+1) = \alpha$, when $i = n$;

where $\alpha = (Bf + Bw)/(hf + hw)$

$B(k) = Bf$ and $h(k) = hf$, when $k = 1$ or 3;

$B(k) = Bw$ and $h(k) = hw$, when $k = 2$ or 4

$T(k,i,N)$: absolute temperature at the center of a zone (k,i) of a member at the time $\tau = N\Delta\tau$, where $\Delta\tau$ denotes a time interval

T_F, T_0 : equivalent radiative temperature of a fire and temperature of the air in absolute value respectively

$\lambda, c, \rho,$: thermal conductivity, specific heat and density of

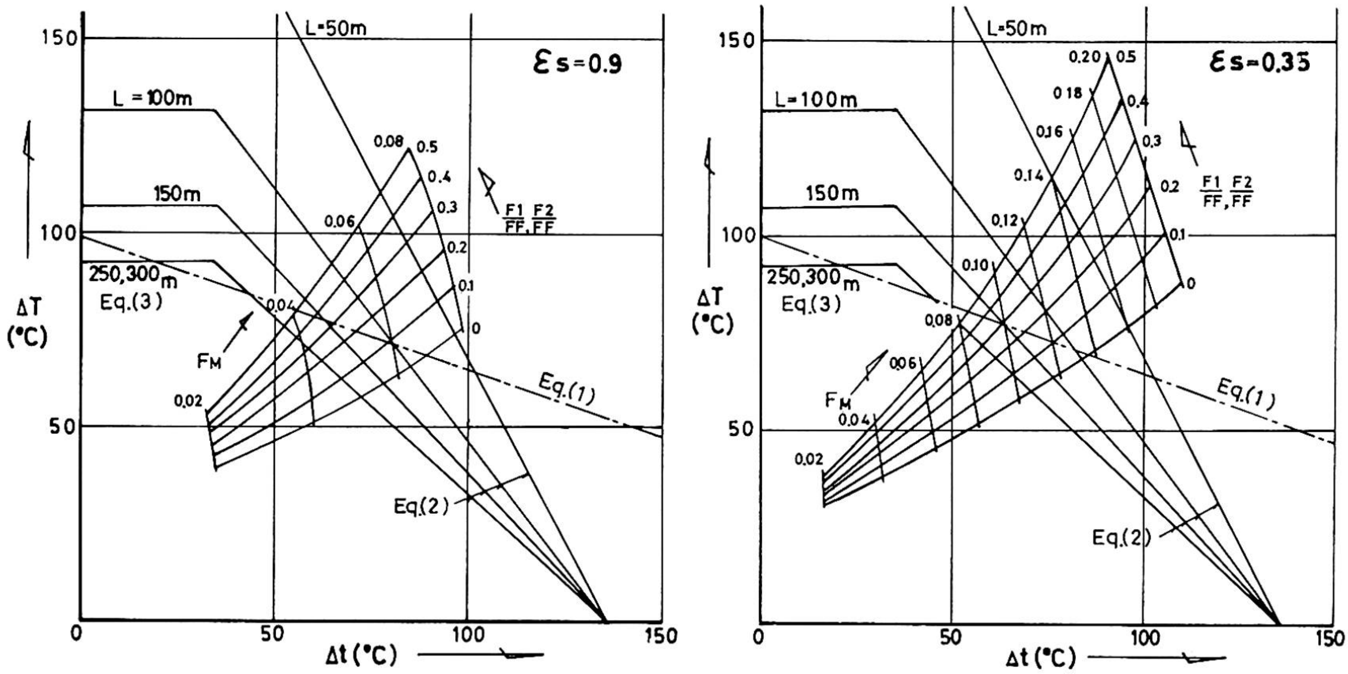
		steel respectively
σ	:	Stefan-Boltzmann constant
ε_s, ε_i	:	emissivity of external and internal surface of member respectively
FF(k)	:	view factor between internal surfaces of zone (k,i) and any zone
H_O, H_I	:	heat transfer coefficients by natural convection at an external and an internal surface of the zone respectively

2-2 Assumptions for calculation

Calculations were made on the following assumptions;

(1) cross section of the member: Bf = Bw = 900, hf = 22, hw = 25 (mm)
(2) number of zones (k,i): k = 4, n = 3, therefore 12 in total
(3) emissivity of the internal surfaces: $\varepsilon_i = 0.9$
(4) emissivity of the external surfaces: $\varepsilon_s = 0.9$ and ε_s 0.35 ($\varepsilon_s = 0.9$ for ordinary paint, $\varepsilon_s = 0.35$ for alumina paint)
(5) temperatures T_F, T_O: $T_F = 1000$, $T_O = 298$ (°K)
(6) duration of the fire: three hours
(7) view factors between the fire and three external surfaces of the member FM, F1 and F2: FM is for the surface receiving the largest amount of the radiative heat among the three surfaces. F1 and F2 are for the two surfaces normal to the surface concerned with FM, where F2 ≤ F1 ≤ FM.
FM: 21 steps in range from 0.02 to 0.92
F1: 6 steps, 0, 10, ..., 50 percent of FM
F2: 3 steps, 0, 50, 100 percent of F1

Therefore the number of cases for calculation amounts to 672 in total.



(a) for an ordinary paint (emissivity $\varepsilon_s = 0.9$)

(b) for an alumina paint (emissivity $\varepsilon_s = 0.35$)

Fig. 3 Relation between temperature differences (ΔT and Δt) and view factors (FM, F1 and F2) for the case of F1 = F2 = 0.5FM

2-3 Results of calculation

Through these calculations, many available data have been obtained to know various thermal conditions on the member under different situations of fire. But two examples of several diagrams synthesized from the calculated results only are shown in Fig. 3 for two cases of the emissivity $\varepsilon_s = 0.9$ and $\varepsilon_s = 0.35$ in each of which F1 = F2 = 0.5FM. The figure shows the relation between the temperature differences (ΔT and Δt) and the view factors (FM, F1 and F2) in range concerned with permissible temperature limits represented by inequalities (1), (2) and (3), where ΔT and Δt denote respectively the temperature difference between two truss members and one between two plates of a member.

3. Geometrical requirement for ensuring bridge against fire

As shown in Fig. 3, we have clarified the relation between the permissible thermal conditions and the view factors. They are given from the diameter "ϕ" and the height "H" of a fire, the distance "L" from the center of the fire to the surface of the bridge member and so on.

Thus we have obtained finally the following geometrical conditions represented by ϕ/L required for ensuring a bridge against fire, assuming to be H = 1.5 ϕ.

Emissivity of external surface of a member	$\frac{F1}{FM}$ (= $\frac{F2}{FM}$)	ϕ / L
0.9 (for ordinary paint)	0	$\leq$ 0.33
	0.5	$\leq$ 0.29
0.35 (for alumina paint)	0	$\leq$ 0.46
	0.5	$\leq$ 0.40

SUMMARY

A problem how to ensure elevated bridges against fires of oil plants situated near the bridges has been submitted recently in Japan. Thermal effects on steel members of bridges are discussed and some permissible thermal requirements for the members are derived. An equation for calculating temperatures of a member having a box section is introduced and solved for various conditions of radiation. A method for estimating necessary clearance between bridge and an oil plant according to probable scale of the fire is suggested.

RESUME

Le problème de la protection d'un pont contre l'incendie d'une grande raffinerie près du pont s'est posé récemment au Japon. La discussion de l'effet thermique sur les éléments métalliques du pont a permis de tirer quelques règles de résistance thermique des éléments. Une équation pour calculer la variation de température d'un élément en caisson a été développée, tenant compte de différentes conditions de radiation. Une méthode pour estimer l'éloignement nécessaire entre le pont et la raffinerie en fonction de l'importance de l'incendie a été proposée.

ZUSAMMENFASSUNG

In jüngster Zeit befasste man sich in Japan mit der Frage, wie Brücken vor Bränden in nahegelegenen Erdölraffinerien geschützt werden können. Man untersuchte die Wärmeeinflüsse auf Brückenglieder und gewann daraus einige zulässige Widerstandsbedingungen. Es wurde eine Gleichung zur Errechnung der Temperaturen für Stäbe mit Hohlquerschnitt abgeleitet und für verschiedene Strahlungsbedingungen gelöst, und eine Berechnungsmethode für die erforderliche Entfernung der Brücke von der Erdölraffinerie in Funktion der Ausdehnung des Brandes vorgeschlagen.

Probabilistic Analysis of Fire Exposed Steel Structures

Détermination probabilistique de la sécurité au feu des éléments de structure métallique

Wahrscheinlichkeits-theoretische Auswertung der Brandsicherheit von Stahlbauteilen

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A large amount of work is presently in progress regarding the optimum level, in an economic sense, of the over-all fire protection of buildings. Structural damages can be prevented or limited by many measures, such as compartmentation, installation of detectors and sprinklers, reducing the attendance time of the fire brigade etc. Among those steps taken to reduce the fire damage, the oldest and most evident one is to increase the fire endurance of the individual structural member. For a high-rise building, the fire endurance must reach the level where the structural integrity of the building is maintained even during the most severe fire possible. For economic reasons, though, the fire endurance cannot be unlimitedly high. Some element of risk, however small, has to be accepted. Evidently, there is a need for a reliability analysis that makes it possible to identify this risk of structural collapse by fire and compare with the risks due to other kinds of catastrophic events.

This need has been accentuated by the different design rationales or systems put forward during the last few years. Particularly interesting in this connection is the differentiated Swedish method, see /1/, /2/. The special attention derives partly from the fact that for the first time the new developments have been transformed into a ready-to-use design manual. The manual permits, with the aid of charts, diagrams and tables, the practising engineer to make a rational design of fire-exposed steel structures. The method is based on the load factor concept, and as in any other design procedure, the choice of nominal loads (fire load density, live and dead load) and load factors will determine the final safety level.

The safety analysis of fire-exposed structures must begin with the procedure critical in every reliability evaluation; the assessment of underlying uncertainties.

Following the general outline of Fig. 1 in /1/, a general systematized scheme may be set up for the identification and evaluation of the various sources and kinds of uncertainty possible for a fire-exposed building component. Lack of space prohibits any attempt to account for the detailed process of data acquisition and evaluation, reference is made to /3/, where all particulars may be found. Here it can only be stated that, with Fig. 1 in /1/ as a functional basis and with the basic data variables selected (type of structural element, type of occupancy), the different uncertainty sources in the design procedure are identified and dissembled in such a way that available information from laboratory tests can be utilized in a manner as profitable as possible. The derivation of the total or system variance (R) in the load-carrying capacity R is divided into two main stages:

- variability $Var(T_{max})$ in maximal steel temperature T_{max} for a given design fire compartment
- variability in strength theory and material properties for known value of T_{max}.

Consecutively $Var(T_{max})$ is decomposed into three parts:

- equation error in the theory of compartment fires and heat transfer from fire process to structural component,
- variability in insulation material characteristics,
- possible difference between T_{max} obtained in laboratory test and in a real service condition.

In step number two, uncertainty in R for a given maximum steel temperature is, in the same way, broken down into three parts:

- variability in material strength,
- prediction error in strength theory,
- difference between laboratory test and a real life fire exposure.

These uncertainty terms must be superimposed upon the basic variability due to the stochastic character of fire load density. Mean and variance of load effect S are evaluated using results from publications covering the non-fire loading case.

To get appliable and efficient final safety measures, the reliability calculations are illustrated for the structural component, where the strength and deformation theories predicting the member performance under fire exposure seem most complete: an insulated simply supported steel beam of I-cross section as a part of a floor or roof assembly. The chosen statistics of dead and live load and fire load density are representative for office buildings.

The component variances are quantified, whenever possibel comparing the design theory with experiments. System variance is evaluated in two ways: by Monte Carlo simulation and by use of a truncated Taylor series expansion. Employing the Monte Carlo procedure, the mean and variance of R and S have been computed for different values of ventilation factor of fire compartment, insulation parameter κ and ratio D_n/L_n, where D_n = nominal dead and L_n = nominal live load used in the normal temperature design. The second moment reliability as a function of these design parameters is evaluated by the Cornell and Esteva-Rosenblueth safety index formulations /

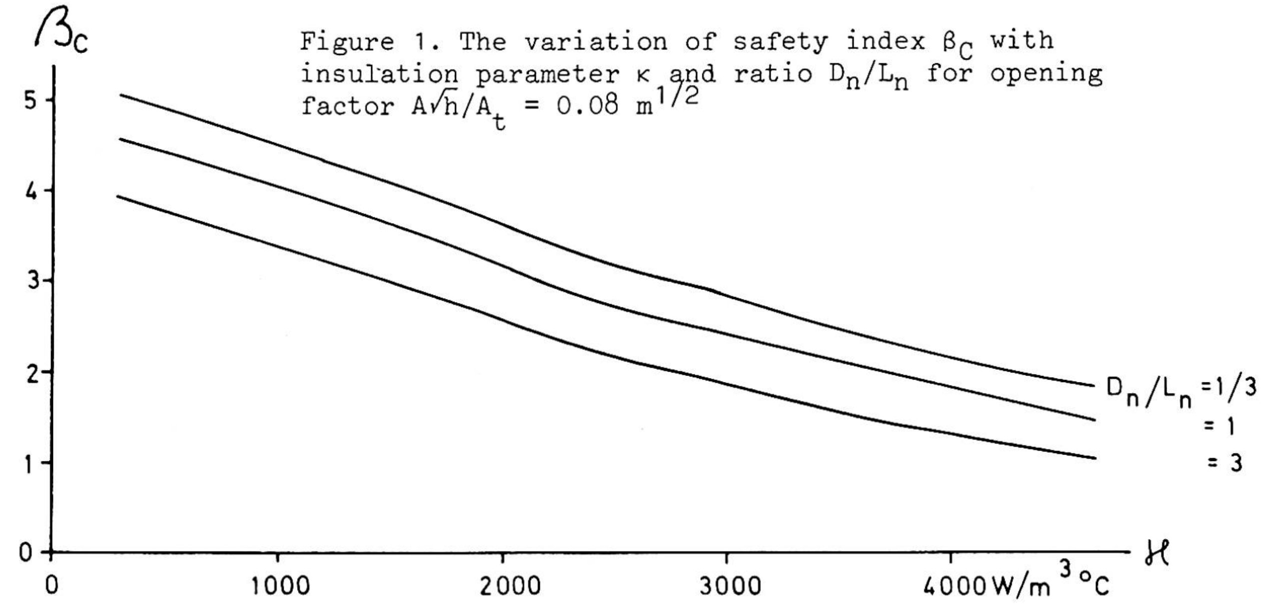

Figure 1. The variation of safety index β_C with insulation parameter κ and ratio D_n/L_n for opening factor $A\sqrt{h}/A_t$ = 0.08 $m^{1/2}$

Fig. 1 gives the safety index β_C,

$$\beta_C = \frac{\bar{R} - \bar{S}}{\sqrt{\sigma_R^2 + \sigma_S^2}} \tag{1}$$

for an insulated, fire-exposed steel member as a function of κ. The insulation parameter κ is defined by

$$\kappa = \frac{A_i \cdot \lambda_i}{V_s \cdot d_i} \quad (W/m^3\ {}^oC) \tag{2}$$

where

A_i = fire exposed area of steel element (m^2/m)

V_s = volume of steel (m^3/m)

d_i = insulation thickness (m)

λ_i = thermal conductivity of insulation material (W/m ^oC)

Continuing the summary of /3/, the accuracy of the distribution-free second moment theories to uniquely define the reliability is touched upon, and the variation in safety-index value with varying uncertainty measures characterizing the insulation and the degree of complete combustion is examplified.

The Taylor series expansion method is compared with the Monte Carlo method and demonstrated to give surprisingly good agreement. The mathematical structure of the partial derivatives method makes it natural to use it as a basis for a closer investigation of how the total uncertainty in e.g. load-carrying capacity R varies with the uncertainties arising from different sources. Such information is necessary in a systematic study of how to economically optimize the avoidance of a structural failure.

Table 1 gives an example of such a decomposition. Of special interest is the variability inherent in the largely empirical design gastemperature-time curves, see /1/, /2/. The variance of these curves was measured by comparing design maximum steel temperatures with the corresponding experimental values for 97 natural fire-exposed insulated steel columns. The comparison was made for well-known thermal characteristics of the insulation material, but includes scatter due to the approximate heat transfer theory used in computing steel temperature values. From Table 1 it may be deduced that the uncertainties deriving from ventilation-controlled gas-temperature-time curves is of minor importance for the final safety index value.

The following section turns to the problem of comparing the reliability levels of the traditional and the new, differentiated design method. It is demonstrated how the flexibility of the new method results in drastically improved consistency for the failure probability P_f.

At the same time it is shown that the temporary nominal loads and load factors given by the manual /2/ do not result in reliability levels that are independent of the ratio D_n/L_n. Using the linearization factor defined by Lind, see /4/, it is examplified how statistically more consistent load factors easily may be derived. Finally it is pointed out how mathematical programming algorithms may be employed to obtain load factors or partial safety factors that for a broader range of design parameters minimizes the difference between the demanded, preselected and the actual reliability level.

These load factor evaluation studies underline a fundamental fact. In sharp contrast to the standard design procedure, the design model of Figure 1 in /1/ has the capability of being systematically and rationally improved as knowledge increases.

Summing up, this pilot study has demonstrated that a safety analysis, using probabilistic methods, of fire exposed structural steel components is today well within the bounds of possibility. The implication is that one of the main components in the over-all fire-safety problem for the first time has been rationally assessed, thus opening the way for an integrated system approach with a reliability optimization as final objective.

Table 1. Decomposition of the total variance of load-carrying capacity into a sum of component variances for an insulated steel beam designed according to the differentiated Swedish model

Variability in load-carrying capacity R due to	per cent of total variance
stochastic character of fire load density	36
uncertainty in insulation material properties	10
uncertainty in theory transforming fire load density into maximum steel temperature (theory of compartment fires and theory of heat transfer burning environment - structural steel component)	10
difference between laboratory test and an actual complete process of fire	2
uncertainty in yield strength of steel at room temperature	12
uncertainty in the deformation analysis giving the design capacity	11
difference between the impact of fire on R in laboratory test and under service conditions	19

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SUMMARY - A first attempt has been made to assess the reliability of fire-exposed steel structural member, using the available tools of modern safety analysis.

RESUME - C'est une première tentative pour évaluer la probabilité de rupture d'une construction en acier exposée au feu, en appliquant les moyens disponibles de l'analyse de sécurité moderne.

ZUSAMMENFASSUNG - An diesem ersten Versuch wird gezeigt, dass eine wahrscheinlichkeitstheoretische Auswertung der Brandsicherheit von Stahlbauteilen entwickelt werden kann.

The Analysis, Design and Remedial Repairs for a Fire Damaged Two-Way Roof Truss Structure

Calcul, projet et réparations d'une charpente métallique endommagée par le feu

Berechnung, Entwurf und Überholungsarbeiten an einer brandgeschädigten Dachkonstruktion

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1. Introduction

The structural problems presented by fire damage to a structure are numerous in that many effects not normally considered in the design of structures must be taken into account. These problems are especially acute with steel structures where the tensile strength and yield strength of the material decreases drastically at temperatures above 370°C. This temperature is easily reached in a building fire of short duration.

Several publications (1,2) and textbooks are available to assist the structural engineer in the consideration of thermal effects on various grades of steel at high temperatures. However, these references are primarily concerned with the ability of steel to withstand continuous sustained high temperatures and not to assess the performance of steel under continuous high temperatures for relatively short periods of time.

A major consideration which the structural engineer must take into account is when steel is subjected to high temperature, it expands, reducing the modulus of elasticity of the steel. As a result of expansion, additional forces are applied to adjacent restraint points located in cooler parts of the structure. This additional force can result in increases in stress or stress reversals in adjacent areas of the building.

To determine the full effect of a fire on structural steel, one must have a fairly good idea of what happens to the steel during such an exposure. Complicating the problem of determining the effects are numerous uncertainties such as:

a. Temperature attained by the steel is hard to determine and can only be estimated.

b. Time of exposure at a given temperature is unknown.

c. Heating is uneven.

d. Cooling rates vary and are often subjected to sudden quenching through contact with water as the fire is extinguished.

e. Steel is usually under load and restrained from normal expansion.

f. Microstructural changes in material properties are often uneven throughout a particular member.

2. Structural System

It is the intent herein to describe one approach to the analysis, design, and remedial repairs to a 106 meter by 106 meter roof structure due to a sudden, intense fire of short duration in the roof area. The structure is the physical education, athletic and convocation center at Middle Tennessee State University in Murfreesboro, Tennessee. The architects for the project are Taylor & Crabtree of Nashville, and the structural engineers for the roof structure are Stanley D. Lindsey and Associates, Ltd., of

Nashville. The roof framework is a symmetrical two-way truss system supported on four columns as shown in Figure 1. The structural system is considered a space grid to obtain the local distribution on each truss. The four main support trusses spanning between the four columns serve to distribute the load equally to the support columns.

The roof structure was analyzed as a two-way grid system under transverse loading, with the member moments and shears being applied to the joints of the corresponding trusses. Due to the symmetry of the roof framework, only one-eighth of the total grid was analyzed using standard matrix methods of analysis. Numerous grid loading situations were considered to determine the maximum stress within each individual truss member. The resulting truss elevations are shown in Figure 2. Volunteer Structures, Inc. of Nashville fabricated the steel to form individual sections, some 8.84 meters and others 15.24 meters in length, and all 3.96 meters deep. The individual sections were joined at the site to form one large square. Extensive use of U. S. Steel's EX TEN 50 high strength steel was made throughout the structure. A490 high strength bolts were used for the main truss connections.

3. Structural Fire Damage

During the construction stage of the project (after all steel trusses, bar joists, and roof decking were in place and with the majority of the dead load present), a flash fire broke out on a scaffolding platform adjacent to and just below one of the mechanical rooms. This was just to the side and at midspan of one of the main support trusses. While the fire was under control within thirty minutes, the heat in the roof reached a minimum temperature of 540°C causing a major reduction in the strength of the steel and expansion of several of the truss members. As a result of these changes, several members deformed, thus weakening the structure and causing it to be on the verge of collapse. Immediate action was necessary.

Upon receiving proper authorization to save the structure, temporary shoring and bracing were placed in the area of greatest damage. Before installation of the shoring tower could be completed, the roof structure gave a loud "crack," and the main truss dropped 5 to 7.5 centimeters, as later verified by measurement. The structure remained standing; however, a considerable increase in deflections was apparent. As soon as the main shoring towers were in place under the main support truss, the truss was jacked back up 2.54-3.8 centimeters in an effort to eliminate the large deflections and to relieve stresses in the truss. The problem then became one of trying to assess the extent of the structural damage by deciding which members were no longer effective; the extent of the stress redistribution; and, ultimately the structural soundness of the roof once the full live load was placed on the structure.

An inspection of the damaged area revealed the following physical changes:

a. The top chord of the main support truss had major flange buckling and lateral deformations.

b. Virtually all bar joists and bridging were damaged beyond repair.

c. The top chords of several adjacent trusses had warped stems.

d. Several diagonals composed of double angles had buckled.

e. Virtually all the miscellaneous support steel for the mechanical equipment was deformed.

The above changes plus the large deflected positions of the trusses in the area resulted in a structural system substantially different from the original design.

Based on microstructural studies of A36 steel from the area of the fire excessive grain growth did not occur. Hardness measurements on damaged material indicated that the mechanical properties were still in the acceptable range, and the A490 bolts appeared to be undamaged and should not have to be replaced. The exact temperature reached was not known; however, cooling curves of material which had been partially melted indicated the temperature reached at least 540°C and the maximum temperature was probably below 650°C or of very short duration. The problem was one of trying to analyze and correct the structure as best one could due to large deformations present.

4. Structural Repairs

An extensive analytical investigation into the structural problems presented by the damage to the roof structure from the fire was undertaken. A structural model was formulated which predicted reasonably well the behavior of the structure as defined by inspection and displacement measurements. This model was based on the original design model with the panel that buckled (top chord of truss 3A) being zero effective. By modeling the structure this way, while not an exact solution, the analysis yielded a set of design parameters which were an upper bound for existing and future member loads and thus assured that all areas of stress redistribution were adequately anticipated.

Once the structure model was developed, modifications to this model were made to determine action necessary to correct the damaged zone. While many different modifications were considered, only three approaches seemed feasible. These approaches were:

a. The possibility of reshoring and jacking the entire structure back to its original elevation and replacing those members, joists, bridging, etc. which were damaged by the fire.

b. The possibility of reshoring and jacking a portion of the structure around the damaged zone to its original elevation. Once this was done, these members, joists, etc., which were damaged by the fire could be replaced.

c. The possibility of reinforcing the structure in its current condition (i.e., at some intermediate elevation and braced by cables and shores as mentioned). Those members and connections which received more than their design load with the addition of live load would be reinforced, and the joists. bridging, etc., which were damaged by the fire would be replaced.

Our investigation showed that of the three different approaches, only the first and third approaches were feasible. These approaches, hereafter referred to as Option 1 and 2 respectively, are discussed herein. The second approach was not feasible due to the limiting capacity of commercial shores available to lift only a portion of the truss structure back to its original elevation and the serious stress reversal that would occur in adjacent truss members making reinforcing practically impossible.

The first option was that of reshoring and jacking the truss structure back to its original elevation and replacing those members, joists, etc., that were damaged by the fire. This procedure required the same shoring arrangements that were defined for the construction stage of the project. Once the truss was in its original elevation, the damaged members, joists, and bridging would have to be replaced. The buckled portion of the top chord truss 3A would have to be replaced while the top chord of truss 2 at mid-span of truss 6 and truss 7 would have to be reinforced. Also, it was decided that all high strength bolts in the top chord connections at the intersection of trusses 6 and 7 and truss 3 should be replaced individually once the roof structure is back to its zero elevation.

The apparent advantage of this option was that it returned the structural system to its original design before the fire with the exception of the reinforcement of the top chord of truss 2 and the two splices required to insert a new top chord section of truss 3. The deflections were then very close to the original design deflections. The apparent disadvantages were that the entire structure must be reshored back to its zero elevation and thus restrict work underneath the roof structure. This in turn could result in a delay in the project plan plus increase the labor involved in reshoring and jacking. Also, it should be noted that while the shoring pattern to raise the truss was defined, the jacking procedure was not well defined due to the new unsymmetrical deflection pattern. For this reason, the jacking procedure could result in stress reversals causing some tension members to buckle and as such must be monitored closely.

The second option was one of reinforcing the structure in its current state for live load and replacing the secondary members, joists, bridging, etc., which were damaged in the fire. Also, overstressed connections in both the top and bottom chord planes would need to be reinforced.

Since all members and connections must be reinforced to within the allowable stress for total design load, additional steel must be added to the truss in the overstressed areas. Likewise, the connections must be reinforced to carry the additional increase in force due to both the increase in dead load as well as the stress redistribution of the damaged structure. Once these corrections are made to the damaged structure, the system would be structurally sound. The only noticeable difference, in that the truss superstructure will be covered up, is that of an increase in deflection on the exterior facia under design loads.

The advantage of this option is that it results in a new structural system which is structurally safe without the addition of new shores. As such, work could continue underneath the roof structure. The disadvantages are that a large increase in pounds of steel would result in the damaged area in that 13 top chord members, 10 bottom chord members, 44 diagonal members, and 12 connections must be reinforced. This procedure results in the addition of approximately 25,373 kilograms of reinforcing members (plates, angles, and structural ties) plus the labor involved in this many corrections. Also, a non-symmetrical displacement pattern results.

Our investigation indicated that the two options to the correction of the damage to the truss structure as presented above were feasible and would result in a structural system which was structurally safe.

It was the concensus of all concerned that Option 2 would be the better of the two options. The corrections were made and the facility is now operational. The building has been subjected to approximately its full design load without unrealistic increases in deflections as predicted by the analytical model and based on later long term measurements.

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1. Fire-Resistant Steel-Frame Construction, 2nd Edition, American Iron and Steel Institute.
2. Manual of Steel Construction, 7th Edition, American Institute of Steel Construction.

SUMMARY

This paper describes the analysis, design and remedial work to a two-way steel roof truss damaged by a sudden, intense, flash fire of short duration in the main support area. As a result of the fire several members deformed, thus weakening the strucutre and causing the roof to be on the verge of collapse. The changes in the structural geometry due to permanent deformations, the resulting re-analysis of the roof frame, and the repairs required to return the structure to as close to the original design as possible are presented.

RESUME

Cet article décrit le calcul, le projet et les réparations d'une charpente métallique subitement endommagée par un feu violent et de courte durée dans la principale région d'appui. Le feu a déformé plusieurs membres et affaiblit l'ouvrage, de sorte que le toît s'est trouvé sur le point d'effondrement. L'étude traite les changements de géométrie de la structure dus aux déformations permanentes, l'analyse résultant de la charpente et les réparations à effectuer pour que l'ouvrage corresponde à nouveau, aussi fidèlement que possible, à sa conception d'origine.

ZUSAMMENFASSUNG

Diese Arbeit beschreibt die Berechnung, Entwurf und die nötigen Ueberholungsarbeiten an einer brandgeschädigten, dachtragenden Trägerkonstruktion. Die wichtigsten Tragelemente wurden durch kurzzeitige sehr intensive Feuereinwirkung geschädigt, so dass einige Tragelemente verformt wurden und die Gefahr des Einsturzes bestand. Die aus der Verformung resultierenden strukturellen Aenderungen wurden in die Neuberechnung aufgenommen; die nötigen Ausbesserungsarbeiten, um die Konstruktion der alten soweit als möglich anzugleichen, werden näher beschrieben.

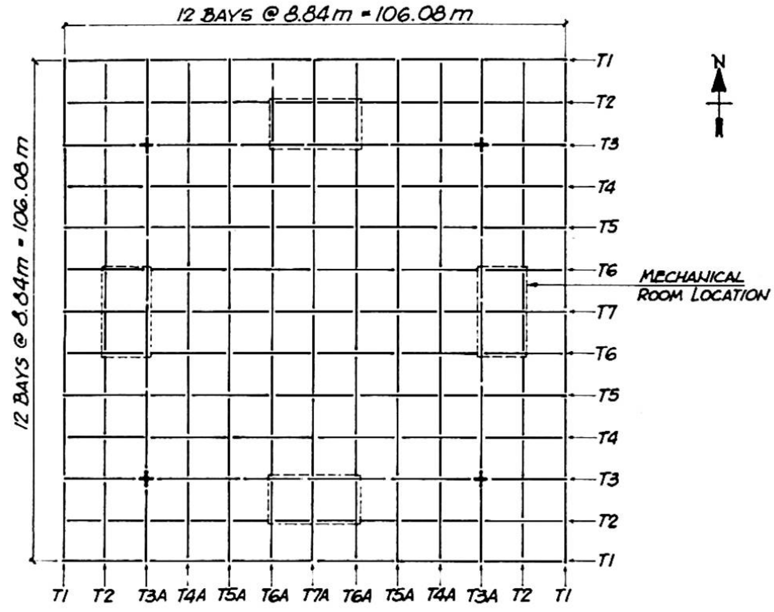
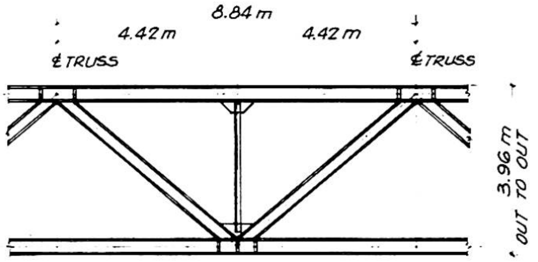
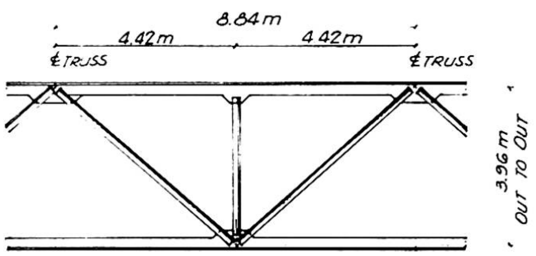


FIGURE 1. ROOF FRAMING PLAN



a) MAIN SUPPORT TRUSS



b) SECONDARY SUPPORT TRUSS

FIGURE 2. TRUSS ELEVATIONS

III c

Calcul et conception des structures en béton armé ou précontraint en vue de leur résistance à l'incendie

Bemessung von Stahlbeton- und Spannbetonbauwerken gegen Brandeinwirkungen

Design of reinforced and prestressed concrete Structures for Fire Resistance

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Fire Endurance of Continuous Reinforced Concrete Beams

Endurance au feu de poutres continues en béton armé

Feuerwiderstand durchlaufender Stahlbetonträger

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1. SYNOPSIS

Results of fire tests of eleven full-scale rectangular reinforced concrete beams are presented. The specimens represented simple spans and interior and exterior spans of multibay structures. Test results indicate that full redistribution of moments occurs when statically indeterminate structures are exposed to fire. This redistribution substantially increases fire endurance.

2. DESCRIPTION OF SPECIMENS

Specimen Design - Test beams 12-in. (305 mm) wide by 14-in. (356 mm) high and 32-ft (9.76 m) long were tested. Figure 1 shows the important features of the five designs used in the test program.

Ten beams were made with normal weight concrete containing a carbonate aggregate. One specimen was made with sanded lightweight aggregate concrete.

Thermocouples were attached to most of the top and bottom reinforcing bars at midspan and at four other locations between midspan and the support.

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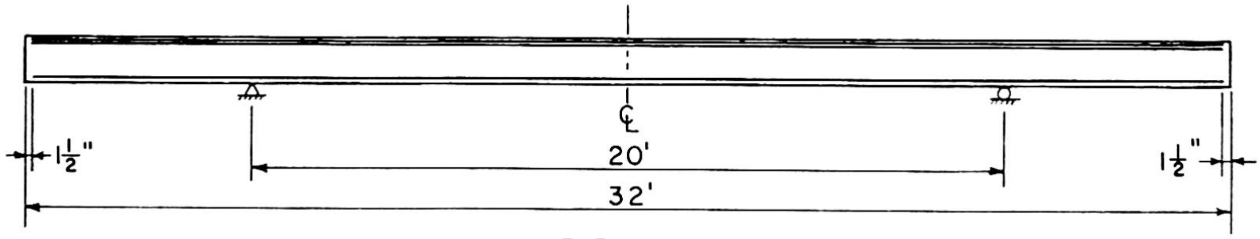
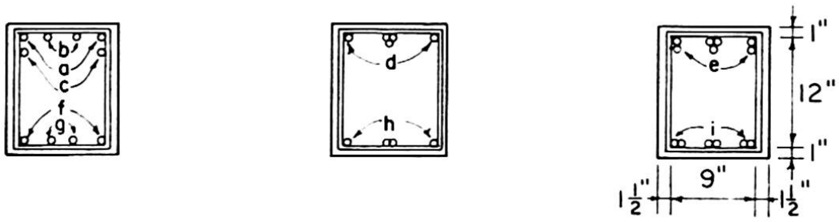


ELEVATION



SECTIONS

SPECIMEN TYPE and DESIGNATION[1]	TOP BAR[3] (Beginning 1½" from each end)						BOTTOM BAR[3] (Symmetric about ℄)					STIRRUPS (Beginning 3" from each end)						
	DESIGN[1]	a	b	c	d	e	DESIGN	f	g	h	i	DESIGN	SIZE	SPACES				
I AAA	A	2 #6 31'-9"	2 #6 9'-2½"	2 #6 7'-11½"	–	–	A	2 #6 31'-9"	2 #6 11'-8"	–	–	A	3	19 @ 6" = 9'-6"	5 @ 12" = 5'-0"			
II ABA	A	2 #6 31'-9"	2 #6 9'-2½"	2 #6 7'-11½"	–	–	B	2 #6 31'-9"	2 #6 31'-9"	–	–	A	3	19 @ 6" = 9'-6"	5 @ 12" = 5'-0"			
III AAB	A	2 #6 31'-9"	2 #6 9'-2½"	2 #6 7'-11½"	–	–	A	2 #6 31'-9"	2 #6 11'-8"	–	–	B	3 & 4	10 @ 4½"[2] = 3'-9"	19 @ 2½"[2] = 3'-11½"	7 @ 3" = 1'-9"	7 @ 6" = 3'-6"	2 @ 12" = 2'-0"
IV ABC	A	2 #6 31'-9"	2 #6 9'-2½"	2 #6 7'-11½"	–	–	B	2 #6 31'-9"	2 #6 31'-9"	–	–	C	3	7 @ 6" = 3'-6"	18 @ 3" = 4'-6"	3 @ 6" = 1'-6"	6 @ 12" = 6'-0"	
V BCD	B	–	–	–	5 #8 31'-6"	2 #8 13'-0"	C	–	–	4 #6 31'-6"	2 #6 10'-11"	D	3 & 4	10 @ 4½"[2] = 3'-9"	16 @ 3"[2] = 4'-0"	5 @ 4" = 1'-8"	6 @ 6" = 3'-0"	3 @ 12" = 3'-0"

1 Roman numeral designates type, first letter refers to top steel, second to bottom steel and third to stirrups
2 # 4 bar stirrups, all other stirrups # 3
3 All bars Grade 60 = 60 ksi or 4,218 kg/cm²

NOTE 1 in = 2.54 cm; 1 ft = .305 m.

Fig. 1 - Specimen Design Details

Fire Testing Procedure and Data - Fire tests were conducted in the Portland Cement Association beam furnace, Fig. 2, using procedures described elsewhere.(1) Each specimen was mounted in the furnace with a 20-ft (6.1 m) span between 6-ft (1.83 m) cantilevers, as shown in Fig.3. Specimens were supported on steel roller bearings to provide free rotation and longitudinal expansion.

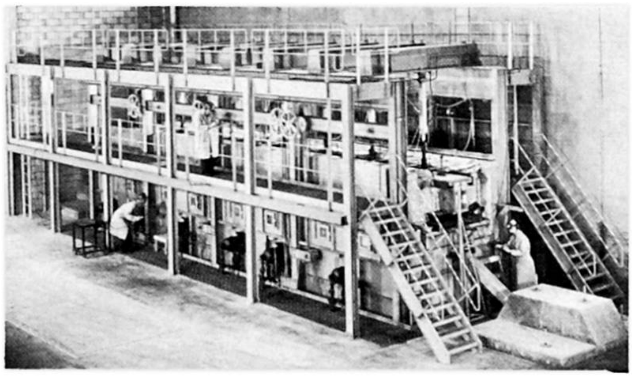
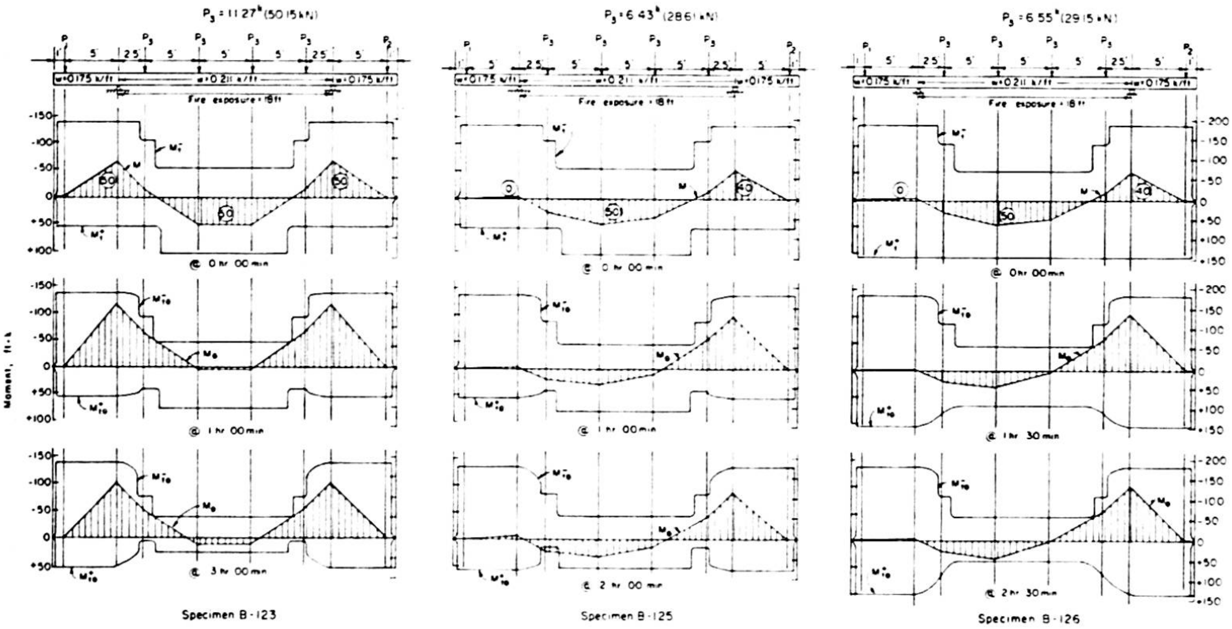


Fig. 2 - Beam Furnace

Four equally spaced hydraulic rams applied loads, P_3 shown in Fig. 3, to the interior span. Cantilever loads, P_1, P_2, were applied through hydraulic rams positioned 1-ft (0.305 m) from each end of the beam.



NOTE 1 ft = .305 m , 1 kip/ft = 14.59 kN/m
Circled numbers in the moment diagrams for 0hr 00min represent the ratio of applied moment to theoretical moment capacity at the start of test

Fig. 3 - Loading and Moment Diagrams Before and During Tests

Each specimen was loaded to develop applied moments at mid-span and over the supports equal to a predetermined percentage of the calculated ultimate moment. Strain hardening was not considered in these calculations. The magnitudes of the applied moments as percentages of the calculated capacities are shown in Table 1.

TABLE 1 - TEST DATA

Speci-men No.	Speci-men Type	Moment Intensity, M/M_t at Start of Test			Span Loads	Cantilever Loads						Avg. Temp. Bottom Re-inforcement End of Test		Midspan Deflection End of Test	Test Duration
						West, P_1			East, P_2						
		West Support	Midspan	East Support	P_3	0 Hr	Maximum	End of Test	0 Hr	Maximum	End of Test				
		%	%	%	Kips	Kips			Kips			F	C	in.	Hr:Min.
B-123	I	50	50	50	11.2	12.9	23.0	20.7	13.6	21.3	22.0	1315	712	6.3	3:30
B-124	I	0	50	0	4.5	0	0	0	0	0	0	952	511	6.4	1:20
B-125	I	0	50	40	6.4	0	0	0	10.4	20.1	18.1	1123	606	3.5	2:00
B-126	II	0	50	40	6.6	0	0	0	10.6	21.4	21.4	1213	655	5.4	2:33
B-127	III	55	50	40	10.5	14.9	23.1	19.1	10.2	22.7	20.0	1360	737	6.1	4:03
B-128	III	40	40	40	8.8	10.4	19.7	17.4	10.8	19.5	16.7	1450	787	4.0	4:31
B-129	III	50	50	50	11.2	13.2	21.6	20.0	13.3	23.3	20.8	1315	712	4.8	3:36
B-130	III	60	60	60	13.5	16.1	24.6	24.6	16.1	24.2	24.4	1293	700	5.3	3:03
B-131	IV	0	50	40	6.4	0	0	0	11.2	21.6	21.2	1280	693	7.0	3:01
B-132	V	0	60	60	11.3	0	0	0	33.0	53.1	46.5	1088	586	4.5	2:03
B-136[1]	I	55	55	55	13.4	16.2	27.7	24.1	16.0	27.3	22.9	818	436	3.0	1:30

[1]Lightweight aggregate concrete; all other specimens were of normal weight concrete.

Note: 1 in. = 25.4 mm; 1 kip = 4.45 kN.

With the exception of Specimen B-124, all beams were loaded to simulate continuous beam action. This was accomplished by varying loads P_1 and P_2 to maintain a constant elevation at the free end of one or both cantilevers. The cantilever loads generally increased sharply during the first 15 minutes of the fire test, reached a maximum value at 30 to 45 minutes, and then remained about the same for the rest of the test.

Furnace atmosphere temperatures were programmed to follow the time-temperature relationship specified in ASTM Designation: E119.(2) Reinforcing bar temperatures were monitored throughout each test.

3. ANALYSIS OF TEST RESULTS

General - Figure 3 shows applied moments and moment capacities at the beginning of and during three of the fire tests. The applied moments were calculated from the measured applied loads. The moment capacity, M_t, at the beginning of each test was calculated using the measured strengths of the reinforcement and concrete. Moment capacity during the test was calculated using the strength-temperature relationships for hot-rolled steel(3) and for concrete.(4)

Simple Support - One test, B-124, was loaded to simulate a simple support condition. During the test, no cantilever loads were applied, and no attempt was made to keep the ends at constant elevation.

The fire endurance of 1 hr 20 min. was reached when the moment capacity was reduced to the applied moment. The behavior of simply supported members is covered in another publication.(5)

Interior Spans - Six specimens were loaded to simulate continuity at both ends. Specimens B-123 and B-129 were loaded to induce moments at midspan and over each support equal to 50% of the calculated moment capacities. Both tests were terminated when it appeared that the flexural capacity was about to be reached. The test was stopped at 3 hr 30 min. for B-123 and 3 hr 36 min. for B-129.

Specimens B-128, B-129, and B-130 were loaded to moment intensities of 40, 50 and 60%, respectively, of calculated capacities over the supports and at midspan. Observed fire endurances were 4 hr 31 min., 3 hr 36 min., and 3 hr 03 min. Fire endurance decreased as the applied loading increased.

Specimen B-127 was loaded so that the midspan applied moment was 50% of the calculated capacity and the moments over the supports were 55% and 40%, respectively. The resulting fire endurance was 4 hr 03 min.

Specimen B-136 failed in shear. From Table 1 and Fig. 1, data show that B-136 was more vulnerable to shear failure than were the others.

Initial loading of B-136 was greater than that of other specimens with similar shear reinforcement. In addition, it was made of lightweight aggregate concrete.

The crack that precipitated the shear failure was located between two stirrups spaced 12-in. (305 mm)on center.Calculations indicated that shear reinforcement was required. Although the area provided was adequate, the spacing was nearly twice that permitted by ACI 318-71.(6)

End Spans - Four specimens, B-125, B-126, B-131, and B-132, were loaded to simulate end spans of continuous beams. No provisions were made to restrict rotation or movement at one end. At the other end, the cantilever was maintained at a constant elevation to provide continuity.

The flexural capacity of Specimen B-125 in the region near the bottom cut-off bars was reached at 2 hr 00 min.

Specimens B-126 and B-131 were similar in design except for shear reinforcement. Fire endurances of 2 hr 33 min. and 3 hr 01 min. were observed for these specimens.

Specimen B-132 was of a significantly different design. The top reinforcement consisted of bundled No. 8 bars. During the test, the top bars yielded over the support due to thermal deformation of the beam. This provided full redistribution of moment. The redistribution was not limited by cut-off bars.

4. CONCLUSIONS

1. For simply supported concrete beams exposed to fire, the flexural end point is reached when the positive moment capacity is reduced to a value that equals the applied moment. The positive moment capacity can be accurately calculated by taking into account the heat-reduced strengths of steel and concrete.

2. Continuous concrete flexural members undergo a redistribution of moments during fire exposure. Negative moments at supports increase causing a reduction in positive moments. Such redistribution occurred early during the fire tests reported here. In all cases, full redistribution was obtained.

3. Redistribution of shear was observed in several of the specimens tested. A failure attributed to redistribution of shear was observed in one specimen. However, the shear reinforcement for this beam was inadequate even at normal temperatures.

4. From the data obtained, it appears possible to develop design procedures for calculating fire endurance of continuous concrete flexural members.

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5. Gustaferro, A. H., and Selvaggio, S. L., "Fire Endurance of Simply Supported Prestressed Concrete Slabs," JOURNAL OF THE PRESTRESSED CONCRETE INSTITUTE, Vol. 12, No. 1, Feb. 1967, pp. 37-52; PCA REsearch Bulletin 212.

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SUMMARY

The redistribution of moments that occurred in a fire test of continuous flexural members resulted in an increase in moments over the supports and a decrease in moment at midspan. This redistribution of moment increases the fire endurance of concrete structural members. The results of tests of eleven concrete beams are reported in this paper.

RESUME

La redistribution des moments qui s'est effectuée durant des essais à l'incendie sur membres fléchissants continus a résulté en l'augmentation des moments aux appuis et en la diminution des moments au milieu de la portée. Cette redistribution des moments augmente l'endurance au feu d'éléments en béton. Les résultats d'essais sur onze poutres en béton sont présentés.

ZUSAMMENFASSUNG

Bei Versuchen an brandbeanspruchten durchlaufenden Stahlbetonträgern ergab sich eine neue Verteilung der Biegemomente. Die Stützmomente wurden grösser, die Feldmomente dagegen kleiner. Die neue Momentenverteilung erhöht den Widerstand der Betonträger. Die Versuchsergebnisse für 11 Betonträger werden in diesem Aufsatz beschrieben.

IIIc

Du comportement au feu de poutres en béton

Brandverhalten von Betonträgern

On Fire Behaviour of Concrete Beams

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1. CONCEPTION D'UNE STRUCTURE EN BETON FACE AU FEU.

1.1. CONTEXTE ACTUEL.

Il est très difficile de pouvoir apprécier le comportement au feu d'une structure car la sanction de la tenue effective au feu n'est pas réalisable sur les ouvrages réels, sauf à provoquer un incendie dont les conséquences sont sans commune mesure avec le but recherché.

A défaut, un certain nombre de précautions sont prises concernant, par exemple, la protection de la structure, ou l'étude sur une partie de celle-ci de son comportement lors d'essais dans des fours.

En France notamment, jusqu'à la fin de l'année 1975, la seule justification admise légalement |1| était l'essai dans un four chauffé selon un programme respectant la courbe de température définie par l'ISO :

(1) $$T - T_o = 345 \log_{10} (8 t + 1)$$

T : température en degrés Celsius au voisinage de l'échantillon
T_o : température initiale
t : temps en minutes

d'un ou plusieurs éléments de la construction concernée.

Compte tenu du nombre restreint de laboratoires possédant des fours équipés à cet effet (dimensions, capacités de chauffe et de contrôle importantes), seule une proportion extrêmement faible des constructions existantes a bénéficié de ce genre d'essais.

Encore devons-nous préciser que, même dans ce cas, l'élément pris isolément est rarement représentatif de son homologue situé dans le contexte de la construction réelle, les conditions de liaisons hyperstatiques ne pouvant être respectées au fur et à mesure que la température s'élève.

En plus, pour le béton, les essais sont en général effectués sur des éléments ayant à peine plus de trois mois d'âge alors que ce matériau continue à évoluer avec le temps de manière considérable, notamment en ce qui concerne *sa*

Présentation de travaux effectués par un groupe d'ingénieurs français réunis en Comité d'Etude Spécialisé (M. COIN, Animateur. MM. ARNAULT, BRUNET, ELICHE, BOUTIN, GAMOT, MARZAUX, MARTIN, LE DUFF, LOPEZ, GRUBER, MATHEZ et ADAM).

teneur en eau libre, laquelle joue un rôle très important dans la tenue au feu.

C'est pourquoi les représentants, aussi bien des administrations concernées que des constructeurs, ont estimé qu'il convenait de repenser les modes de conception de la sécurité face à l'incendie et ont établi une *METHODE DE PREVISION PAR LE CALCUL DU COMPORTEMENT AU FEU DES STRUCTURES EN BETON*, dit plus simplement "D.T.U. FEU", dont le texte initial établi en 1972 a fait l'objet d'une refonte en octobre 1974 et d'additifs en mai 1975. |2|

Cette orientation diffère notablement de la plupart des positions prises actuellement |3, 4, 5| qui consistent à définir, en se basant sur des essais, des dimensions hors tout des pièces, des enrobages et des dispositions constructives concernant les armatures en fonction des matériaux utilisés, et à tenir compte des protections éventuelles.

1.2. RECOURS AU CALCUL.

Afin d'introduire l'action du feu dans les calculs, il convient de partir d'un certain nombre d'hypothèses et de s'assurer de leur fiabilité.

Certaines de celles-ci ne peuvent être qu'arbitraires, faute de pouvoir standardiser les incendies qui dépendent :

- du potentiel calorifique des matériels et matériaux existant dans les locaux concernés,
- des conditions de ventilation,
- des locaux environnants,
- de la structure et de sa géométrie ;

ainsi, nous avons admis comme hypothèse de base la courbe(1) de montée en température en surface des éléments calculés. En fonction du flux de chaleur défini de la sorte, il est possible de déterminer la distribution des températures dans les éléments en utilisant l'équation de Fourier qui, pour des problèmes plans, peut être facilement transformée en équation aux différences finies.

Dès lors, la méthode permet d'apprécier :

. les températures atteintes sur la face non exposée d'un élément, et en conséquence d'en connaître le comportement en tant qu'isolation thermique *(notion de coupe-feu)*,

. les températures atteintes dans la masse même des éléments et, en fonction des coefficients de dilatation thermique des matériaux, les effets complémentaires provoqués dans la structure. En tenant compte des connaissances actuelles concernant l'incidence de la température sur les diverses caractéristiques mécaniques des matériaux (contrainte nominale de rupture, allongement, module d'élasticité...), on peut également à chaque instant calculer les conditions de rupture d'une section droite quelconque.

La comparaison, pour tout ou partie de la structure, de la charge de rupture qui en découle avec la charge de service permet de déterminer le moment à partir duquel *la stabilité au feu n'est plus assurée*.

1.3. JUSTIFICATION DU CALCUL.

Parmi les méthodes de justification possibles d'après le document D.T.U. FEU, plusieurs solutions existent :

- Soit le respect de *règles simples* très analogues à celles que l'on retrouve dans les Recommandations FIP-CEB |4| qui sont essentiellement des dispositions constructives complémentaires du calcul à froid où les dimensions des pièces ou des enrobages sont imposées en fonction de la durée envisagée pour la stabilité au feu. Soulignons toutefois que, pour des données géométriques égales, le D.T.U. FEU est plus pessimiste quant à la durée de tenue au feu que les Recommandations FIP-CEB.

- Soit *des calculs à rupture* à partir des résultats de température trouvés.

- Soit *des calculs de température et de rupture*.

Dans ces deux derniers cas, il a fallu vérifier la bonne représentativité des méthodes proposées, ce qui était aisé pour certains éléments tels que :

- les dalles homogènes,
- les poteaux,

pour lesquels de nombreux essais sont relatés avec détails dans la littérature spécialisée, mais bien plus difficile dès que les éléments concernés sont composés de plusieurs couches ou comportent des profils complexes, tels que les poutres à talon ou les poutres en T.

- Soit enfin *par des essais*.

2. PROGRAMME DE RECHERCHE.

C'est donc pour contrôler le bien fondé des méthodes proposées qu'un programme d'étude a été décidé, lequel a porté au cours d'une première phase, aujourd'hui achevée, sur la distribution des températures dans des poutres rectangulaires en T et à talon, puis se poursuit actuellement avec :

- l'étude des bicouches (dalles constituées d'une prédalle de 5 cm préfabriquée et d'une couche de 6 à 10 cm de béton coulé en place),
- l'étude de l'éclatement des dalles chauffées sur une seule face et soumises à divers gradients de contrainte,
- l'étude de la redistribution des contraintes au cours de la formation de rotules sur les appuis des poutres continues,
- ainsi que l'étude du rôle de l'eau libre.

Ces diverses études, aujourd'hui entamées, feront l'objet de publications ultérieures et seule l'étude de la distribution des températures dans les poutres est ici abordée. |6|

2.1. PROFILS ETUDIES.

24 poutres ont été étudiées représentant 12 cas différents :

- 5 poutres en T rectangulaires de dimensions (en cm) :

Type	aile		âme	
	largeur	épaisseur	largeur	hauteur
1	100	6	12	30
2	150	8	15	40
3	200	10	20	50
4	250	12	25	60
5	300	14	30	70

- 3 poutres du type 3 précédent protégées par une couche de plâtre spécial contenant notamment de la vermiculite, de 1, 2 et 3 cm ;

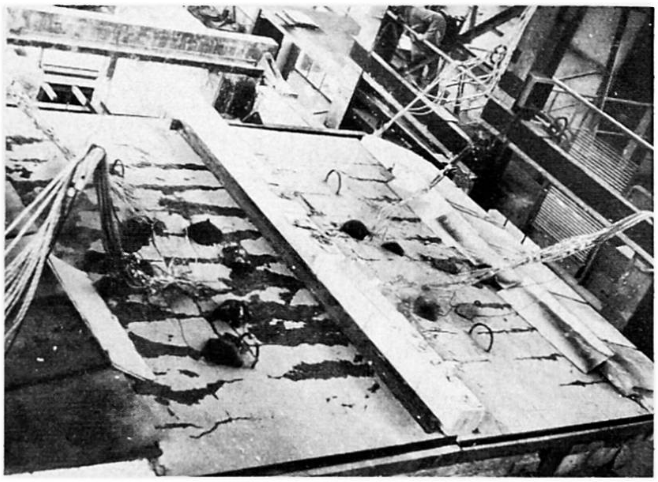
- 4 poutres en T à talon de dimensions (en cm) :

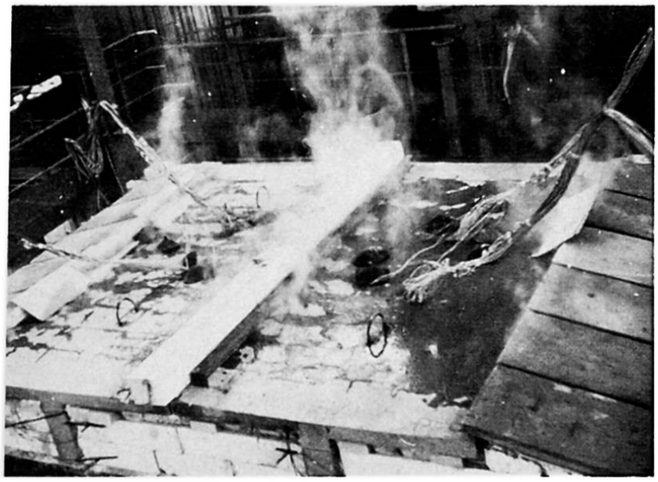
Type	aile		â m e			
			largeur		hauteur	
	largeur	épaisseur	talon	partie mince	totale	talon
1'	150	10	25	9	60	8
2'	150	10	25	9	60	15
3'	150	10	26	15	60	8
4'	150	10	25	15	60	15

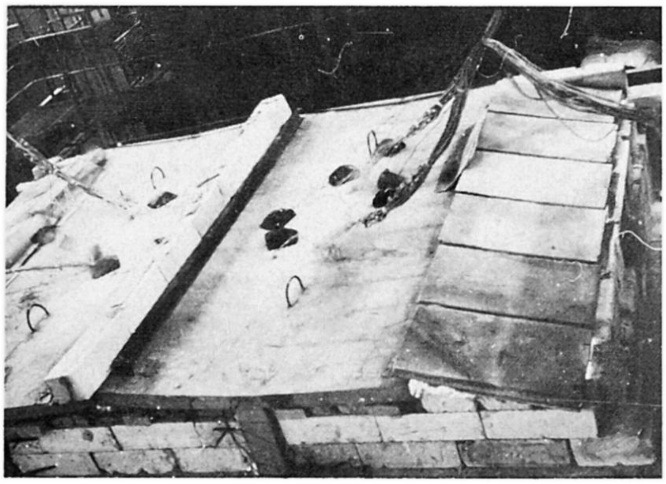
Fig. 1. Vue 18 minutes après l'allumage, la poutre faite avec le béton à plus forte maniabilité "rend" beaucoup plus d'eau.

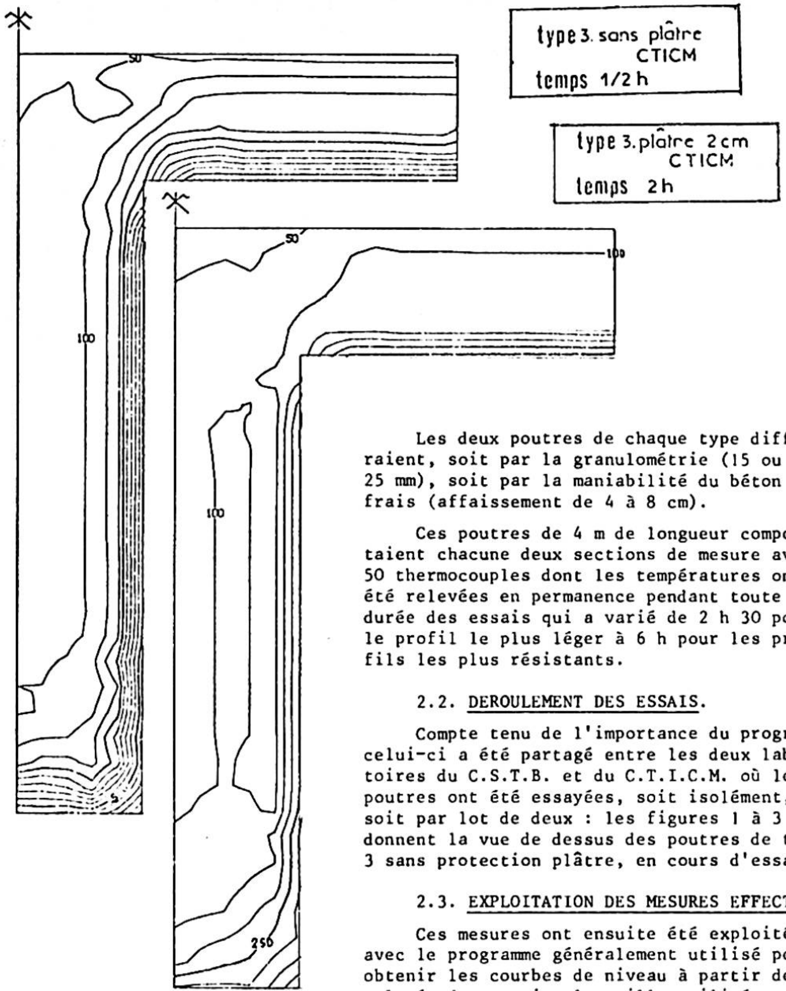
Fig. 2. Vue 21 minutes après l'allumage, la vapeur commence à apparaître.

Fig. 3. Vue 2 heures après l'allumage : le dessus des poutres est sec, il n'y a pratiquement plus dégagement de vapeur.









Isothermes tous les 50 degrés

ISOTHERMES
RESULTATS D'ESSAIS

Fig. 4.

Les deux poutres de chaque type différaient, soit par la granulométrie (15 ou 25 mm), soit par la maniabilité du béton frais (affaissement de 4 à 8 cm).

Ces poutres de 4 m de longueur comportaient chacune deux sections de mesure avec 50 thermocouples dont les températures ont été relevées en permanence pendant toute la durée des essais qui a varié de 2 h 30 pour le profil le plus léger à 6 h pour les profils les plus résistants.

2.2. DEROULEMENT DES ESSAIS.

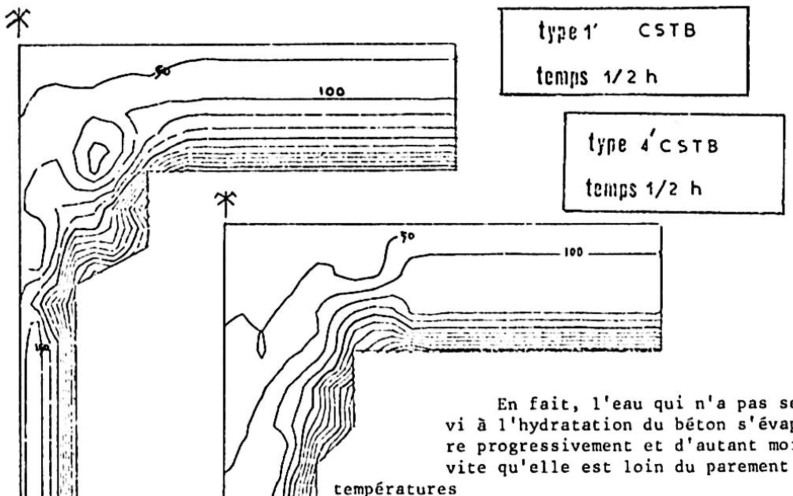
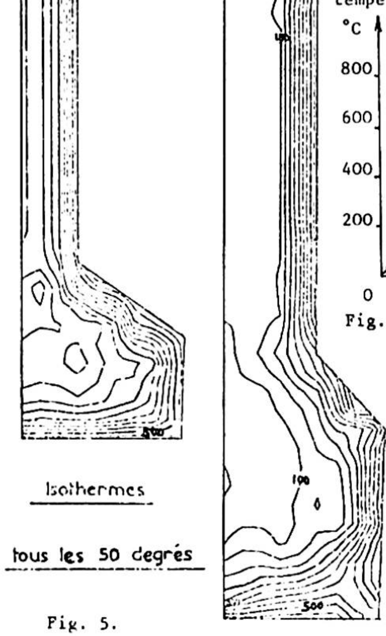
Compte tenu de l'importance du programme, celui-ci a été partagé entre les deux laboratoires du C.S.T.B. et du C.T.I.C.M. où les poutres ont été essayées, soit isolément, soit par lot de deux : les figures 1 à 3 donnent la vue de dessus des poutres de type 3 sans protection plâtre, en cours d'essai.

2.3. EXPLOITATION DES MESURES EFFECTUEES.

Ces mesures ont ensuite été exploitées avec le programme généralement utilisé pour obtenir les courbes de niveau à partir des relevés de terrain, la maille utilisée a varié selon les poutres de 12,5 à 25 mm.

Nous donnons sur les figures 4 et 5 quelques résultats ainsi obtenus; on notera sur la figure 4 la très large plage correspondant à l'isotherme 100°C : la vaporisation de l'eau et son éventuelle recondensation jouent un rôle très important dans le comportement du béton.

En un point donné de la section, en fonction du temps, nous retrouvons des paliers que nous donnons sur la figure 6 pour la poutre de type 2' ; il en est de même pour les poutres rectangulaires.

Isothermes

tous les 50 degrés

Fig. 5.

ISOTHERMES
RESULTATS D'ESSAIS

En fait, l'eau qui n'a pas servi à l'hydratation du béton s'évapore progressivement et d'autant moins vite qu'elle est loin du parement ;

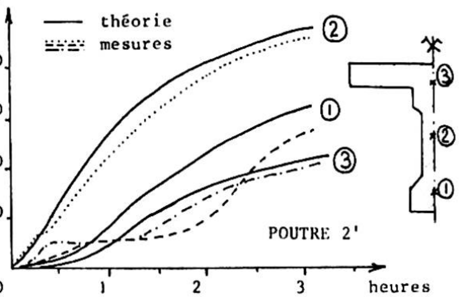


Fig. 6.

nous avons à cet effet représenté sur la figure 7 de la page suivante les teneurs en eau libre susceptibles d'être rencontrées dans le béton d'après les recherches faites en matière de retrait sur ce matériau.

Le palier noté à 100°C est d'autant plus important qu'il y a d'eau libre en un point, puis, après vaporisation de l'eau, la température s'élève à nouveau en suivant les lois de la thermique ; la vapeur ainsi formée, avant de s'échapper (figure 2), peut se condenser dans une zone froide dont elle élève prématurément la température à 100°C (courbe 3 expérimentale de la figure 6). Ce phénomène

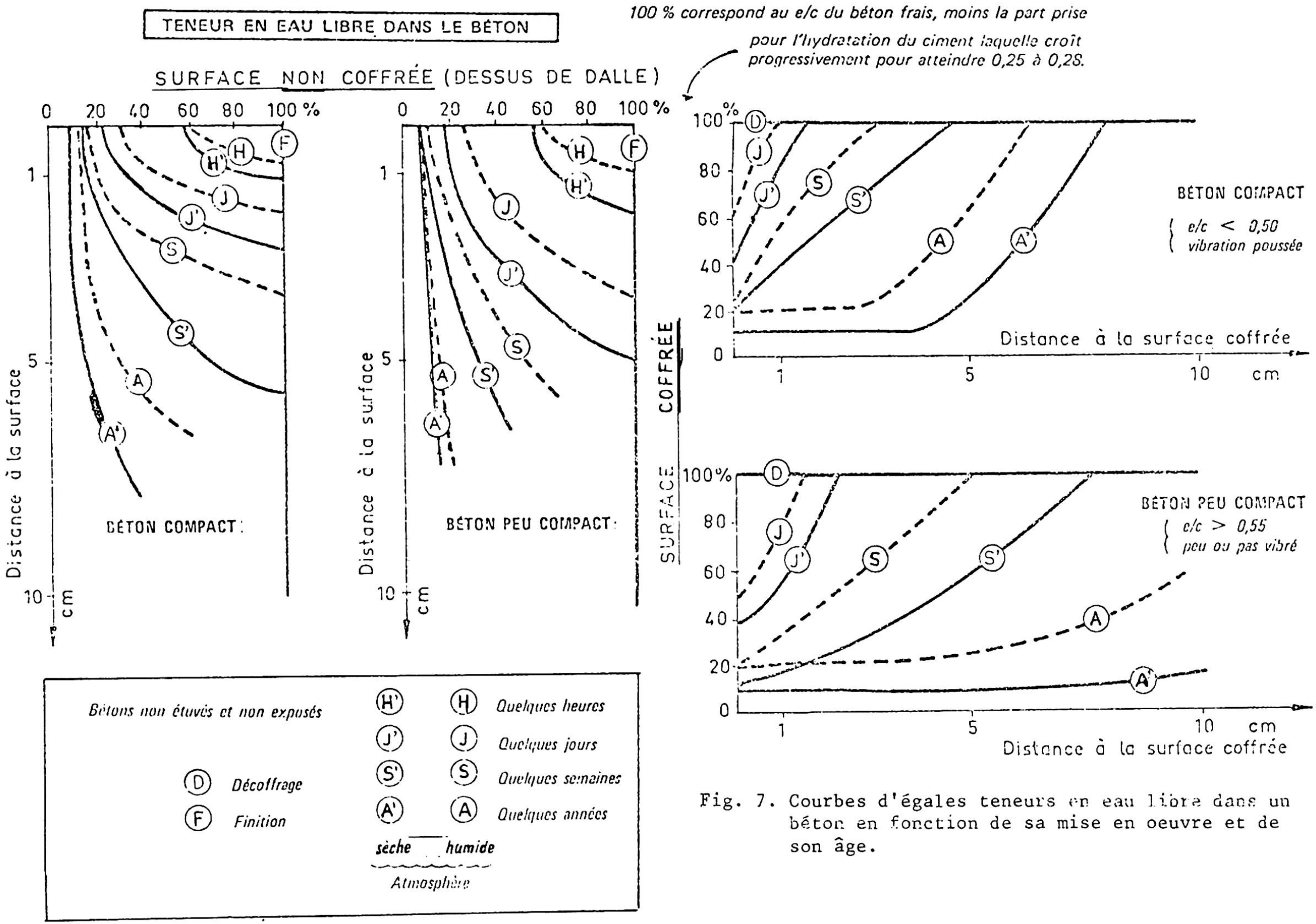


Fig. 7. Courbes d'égales teneurs en eau libre dans un béton en fonction de sa mise en oeuvre et de son âge.

est sans incidence sur le comportement du béton autre que les dilatations correspondantes.

En ce qui concerne les *poutres protégées par du plâtre*, les essais ont montré qu'il est très difficile d'énoncer une loi simple, mais que la proposition adoptée dans le D.T.U. FEU (équivalence de 2,5 cm de béton par centimètre de plâtre) est largement dans le sens de la sécurité; en effet on constate une équivalence plus forte dans les zones très exposées (x < 25 cm sur la figure 8), ces valeurs étant légèrement réduites lorsque l'on fait la correction correspondant au palier constaté à 100°C.

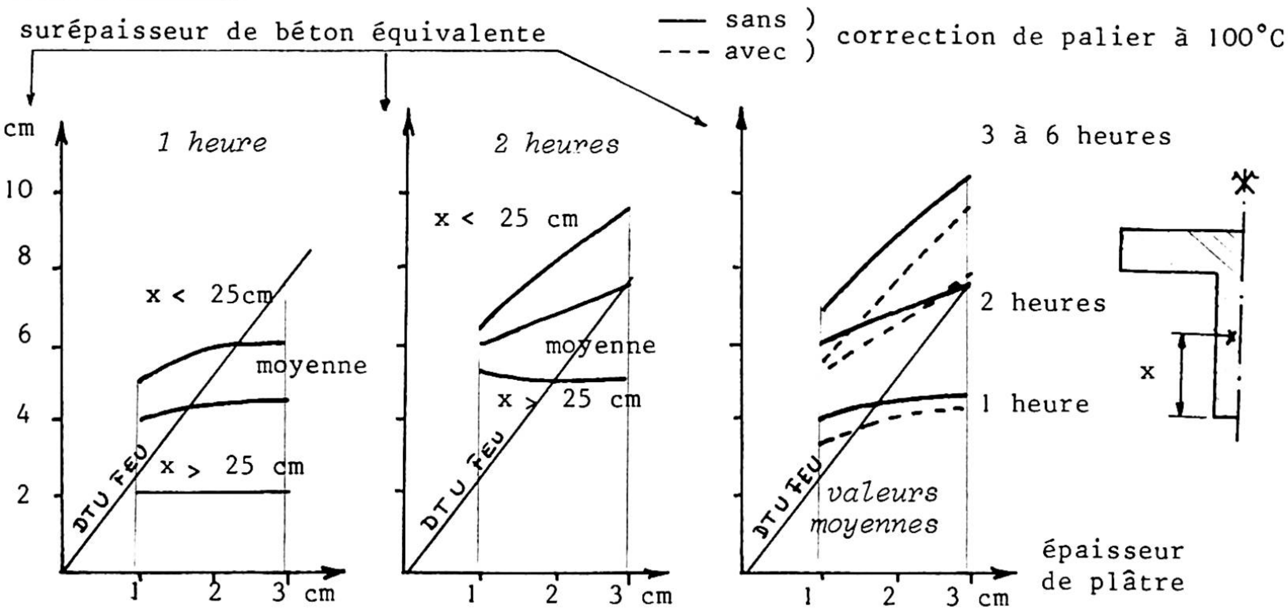


Fig. 8.

3. CONCLUSION.

L'étude de la concordance de la méthode de calcul proposée dans le D.T.U. FEU avec l'expérience montre que l'on peut considérer trois zones dans les poutres non protégées (figure 9) :

- *la zone 1* constituée par une bande périphérique de 1cm où les températures calculées sont légèrement inférieures aux valeurs mesurées;
- *la zone 2* formée d'une bande de 5 cm environ où se trouvent en général les armatures et pour laquelle la concordance des températures mesurées et calculées est parfaite, et toujours dans le sens de la sécurité;
- *la zone 3* au centre où les températures mesurées sont nettement inférieures aux valeurs de calcul en raison du décalage provoqué par la présence de l'eau libre.

Pour tenir compte de ces faits, M. COIN a proposé un programme |6| applicable à la méthode aux différences finies dans laquelle on adopte pour chaque maille une teneur en eau libre pouvant varier de 0 à 150 litres/m3, qu'il est aisé de définir d'après la figure 7.

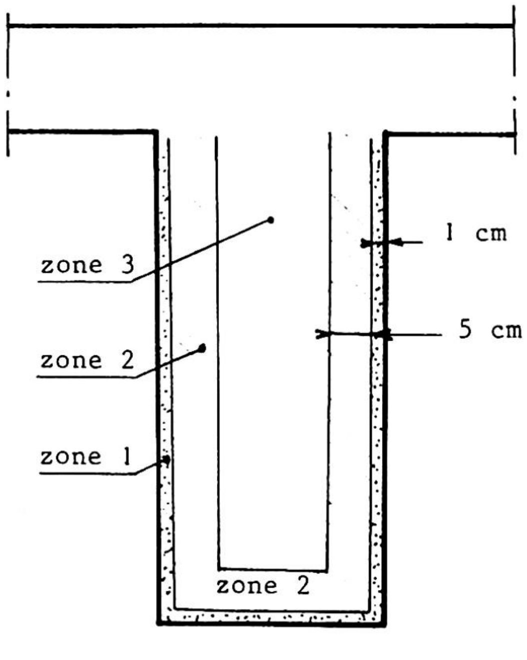


Fig. 9.

L'application de ce programme donne une excellente concordance de la théorie et de l'expérience pour les *zones 2 et 3*, seul le comportement de la peau sur 1 cm d'épaisseur reste difficile à déterminer mais a peu d'incidence sur la stabilité de la structure.

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RESUME - La communication, après avoir rappelé le contexte dans lequel se situe traditionnellement l'étude des structures vis-à-vis de l'incendie, et notamment la justification du bon comportement d'un ouvrage à partir de celui d'un ou de plusieurs de ses éléments soumis à une exposition plus ou moins longue à la chaleur dans un four, indique la tendance actuelle qui consiste à faire ces mêmes justifications d'après le calcul de la structure. Pour justifier ces calculs, un important programme d'essais a porté depuis 1973 sur 24 grandes poutres dont l'exploitation des mesures fait ressortir la bonne concordance des calculs et des mesures ainsi que la nécessité de prendre en considération en chaque point des sections étudiées la teneur en eau libre du béton.

ZUSAMMENFASSUNG - Um das Brandverhalten von Bauwerken besser zu verstehen, wurde in Frankreich eine neue Methode entwickelt. Seit 1973 wurde ein breites Versuchsprogramm auf 24 grossen Trägern durchgeführt. Die Resultate gaben eine Uebereinstimmung der Berechnungen mit den Messungen, sowie die Notwendigkeit, in jedem Punkt der untersuchten Profile den Freiwassergehalt des Betons in Betracht zu ziehen.

SUMMARY - A method has been developed in France, in order to investigate thoroughly the fire behaviour of concrete beams. An important testing program was carried out from 1973 on 24 large beams. Results show a good correspondance between calculations and measures, as well as the necessity to consider the water contents of the concrete in every point of any profile considered.

Response of Reinforced Concrete Frames to Fire

Comportement au feu des structures en béton armé

Brandverhalten von Bauteilen aus Stahlbeton

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INTRODUCTION

Structural design for fire resistance must provide structural integrity for the level of safety desired in a particular building. Providing such structural integrity requires that geometric space characteristics, building materials, contents, and occupancy, as well as different levels of fire intensity, spread, and damage be considered. For rational design, four categories of fire and corresponding levels of tolerable damage may be identified.

Category		1	2	3	4
Fire	Intensity	Low	Low	High	High
	Duration	Short	Long	Short	Long
Structural Response	Damage Level	Nonstructural damage only.	Some structural damage. No collapse.		Specified endurance - - hours

In order to determine probable damage levels, thermal and structural response to the critical fire environment expected in a particular building must be evaluated by calculating time variations in temperature distribution within the structural elements of the building, as well as deformation and stresses in these elements, and initiation and extent of degradation (cracking, crushing, yielding, or rupture) for different types of fire. Evaluation of structural response should also account for different conditions of restraint by the building system. This is essential for economical and safe design, as such information is needed for selecting trade-offs between various means of fire protection vs. additional structural integrity, and for realistically assessing in-place performance.

Fire endurance ratings based on observed behavior of structural elements under standard test conditions cannot provide the information necessary for a rational design for fire safety. For example, if no collapse for type 2 or 3 fire is ensured, then a lower endurance rating than the present requirement might be acceptable for some structures, leading to a more economical design. Therefore, analytical predictions of thermal and structural responses are needed for an optimum design decision.

Determination of thermal and structural response is possible provided that space characteristics, fire environment, structural system, and material behavior when exposed to a fire environment are suitably modeled. In this paper, the methods and validity of analytical predictions of behavior of reinforced and prestressed concrete elements in fire environments are discussed in relation to observed behavior and to current methods for rating the fire endurance of such structural elements.

ANALYTICAL MODELS

In modeling the fire response of structures, heat flow analysis was separated from structural analysis and two computer programs, FIRES-T and FIRES-RC, were written for solving the separate problems. The details of analytical modeling and numerical methods used in solving the problems have been described elsewhere [1-3]. A brief review and some additional comments on modeling fire environments are included here.

Thermal Analysis - For heat flow analysis, a finite element method [2] coupled with time step integration is used. The problem is solved by satisfying the heat balance equation and a known boundary condition at all nodes. The exposed surface boundary condition is modeled either as a prescribed temperature history at the surface or as a heat flux based on convective and radiative transfer mechanisms from an external heat source. For simplicity, this heat flux, q, is expressed as a sum of convective and radiative terms, q_C and q_R, respectively.

$$q = q_C + q_R = A(T_f - T_s)^N + V\sigma(a\varepsilon_f\theta_f^4 - \varepsilon_s\theta_s^4)$$

where: $T_f = F(t)$ is the time-dependent single-valued temperature of the fire, T_s is the average surface temperature of a small element associated with a particular node, A is the convection coefficient, N is the convection power factor, V is the radiation view factor, a is the surface absorption factor, ε_f and ε_s are emissivities of the fire and the surface, respectively, σ is the Stefan Boltzmann constant, and θ stands for absolute temperature.

This model of the boundary condition is based on the assumption that the heat source can be represented by a turbulent, well-mixed gas having, at any time t, a single value of temperature T_f, and a single value of emissivity ε_f. This model can be viewed as a pseudo-fire in which the effects of temperature gradients, gas flow, fire, load distribution, and enclosure wall radiation characteristics are represented by T_f and ε_f. The boundary condition is further simplified by assuming A, N, a, ε_f, and ε_s as constants throughout the fire duration. In some cases, view factors have been varied for different surfaces of the exposed element, although a value of 1.0 has been used in most cases. Exposure to nonfire conditions on the boundary, such as ambient atmospheric exposure, can be modeled as exposure to another 'pseudo-fire' with appropriate T_f and ε_f.

An iterative procedure is used within each time step to deal with the temperature dependence of material properties and nonlinear thermal boundary conditions. The problem is then linearized about the current temperature distribution within a given iteration. A two-dimensional problem is solved, assuming no heat flow along the long axes of frame members. Member cross-sections can have any shape and may be composed of several materials (concrete, steel, insulation); it is assumed that there is no contact resistance to heat transmission at the interface between these materials. Changes in geometric characteristics associated with structural distress, such as spalling, can be accommodated in solving the heat flow problem, provided that the time of occurrence and extent are defined. When such behavior is indicated in the structural response, the two solutions - heat flow and structural analysis - must be coupled and additional iterative cycles will be required to obtain a solution.

Structural Analysis - A nonlinear direct stiffness formulation coupled with time step integration is used for structural analysis [3]. Within a given time step, an iterative approach is used to find a deformed shape which results in equilibrium between the forces associated with external loads and internal stresses and degradation. The material behavior models for concrete and steel account for dimensional changes caused by temperature differentials, changes in mechanical properties of the material with changes in temperature, degradation of the section through cracking and/or crushing, and increased rates of shrinkage and creep with an increase in temperature. Nonlinear stress-strain laws are used to model the behavior of concrete and steel; these laws are capable of accounting for inelastic deformations associated with unloading. Based on this formulation, a computer program, FIRES-RC, has been developed which is directly coupled to the thermal analysis, FIRES-T.

Geometric discretization of the frame and its elements is shown in Fig. 1. The members are substructured into segments and the cross-sections are further subdivided into subslices by appropriately choosing a finite element mesh. Steel and concrete subslices are treated as uniaxially loaded prisms, so that only uniaxial stress states are considered. Wherever possible, advantage is taken of conditions of symmetry.

VERIFICATION OF ANALYTICAL MODELS

The validity of the simplifications made in the analytical models described above can be judged by comparing analytical results with experimental data.

University of California, Berkeley, Studies [4] - The specimen used in the UCB study was a 12 in. (0.3 m) square prism, 60 in. (1.5 m) long, reinforced with eight No. 5 (15.9 mm diameter) reinforcing steel bars. The specimen was instrumented with thermocouples on both steel and concrete, and with strain gages attached to the steel reinforcing bars. The unloaded specimen was subjected to several cycles of controlled heating in a radiant oven producing approximately uniform surface temperature. An upper limit of 600°F (316°C.) was selected for testing the specimen because reliable measurements of strain above this temperature are difficult to obtain. After heating tests of the unloaded specimen were completed, the specimen and oven were moved into a testing machine and three groups of tests were performed in which the specimen was subjected to: (1) heating, (2) axial compression loading and unloading without heating, and (3) heating under constant compressive load.

Analytical predictions of temperature distribution for a typical cycle are compared with experimental data in Fig. 2 where the influence of varying conductivity on calculated values is shown. The predicted concrete temperatures differed from the observed values, partly due to the approximation of thermal diffusivity values used in the analysis, and partly due to the assumption of constant diffusivity throughout the section. The outer 1-inch layer of concrete, which had undergone higher temperature exposure and moisture loss than the interior, is likely to have had a lower diffusivity than the interior, possibly accounting for the difference between observed and predicted temperature values. Nevertheless, the difference between computed and observed values is not great, and the analytical model for thermal response was therefore considered satisfactory.

Predicted and measured deformations of the prism, subjected to a constant load and a heating and cooling period, are compared in Fig. 3. Good agreement is observed in this case. Deformations during loading and unloading cyclic tests without heating are shown in Fig. 4. The low initial stiffness of the prism and subsequent stiffening with increased compressive load reflect the initially cracked state of the interior concrete portion (a consequence of prior heating), followed by closing of the cracks when a compressive load of about 100 kips was reached. The agreement between predicted and observed values under unheated conditions is somewhat less accurate, attributable to some deficiencies in modeling material properties such as nonlinear stress-strain and fracture behavior of concrete in tension under normal and elevated temperatures, nonlinear characteristics of the unloading portion of the compressive stress-strain relationship of concrete at different temperatures, and high-temperature creep in steel and concrete. Nevertheless, the predicted structural behavior of a reinforced concrete prism loaded in compression and subjected to heating cycles exhibited close agreement with measured values (Figs. 3 and 4).

Portland Cement Association Laboratories, Skokie, Illinois, Studies [5, 6] - Two types of prestressed concrete specimens were used in the PCA studies. One group of specimens [5] consisted of slab strips, uniformly loaded over a 12-ft. (3.7 m) simply supported span. In these specimens, aggregate type, concrete cover thickness, size of prestressing strand, and load intensity were varied. In the second group of specimens [6], I-beam specimens with six aggregates were tested using two load intensity levels. Results of the I-beam tests were compared with computed values and generally showed agreement as good as the slab data. The comparison is omitted here due to length limitations on this paper.

The slabs were tested in the PCA floor furnace, and the furnace heating was controlled to meet the standard ASTM E119 time-temperature requirements. The fire temperatures measured by the individual furnace thermocouples showed only small variations from the average, and the average value agreed closely with the standard time-temperature curve. However, during the initial phase of rapid heating, the gas (fire) temperature may differ significantly from the values recorded by shielded, slow response thermocouples. To obtain good agreement between measured and calculated thermal response, it is essential to use a pseudo-fire model reflecting actual conditions as closely as possible. Measurements of temperatures in a wall furnace carried out by Babrauskas [7] using fast response thermocouples have been used to establish a corrected ASTM E119 pseudo-fire time-temperature curve to be used in predicting thermal response during a standard test conducted in accordance with ASTM requirements. The corrected temperature (Fig. 5) is about 500°F (278°C.) higher at 1 minute, 350°F (194°C.) at 3 minutes, 150°F (83°C.) at 6 minutes, and 40°F (22°C.) higher at 12 minutes. No correction is required for times in excess of 24 minutes. These corrections, albeit approximate, provide a much better basis for predicting thermal response during the first 0.5 hour of a standard test.

In modeling thermal boundary conditions for the slabs, it was assumed that the pseudo-fire could be represented by the modified temperature history for a source of $\varepsilon_f = 0.5$. The ambient air was modeled as a pseudo-fire having a constant temperature of 68°F (20°C.) with $\varepsilon_f = \varepsilon_s = a = 1.0$. View factors for all horizontal external surfaces of slabs were taken as 1.0. The vertical sides of the slab strips were insulated so that the horizontal heat flow laterally and longitudinally could be neglected.

Comparisons of calculated and observed temperatures and deflections [8] for a typical prestressed slab (Figs. 6 and 7) indicate good agreement.

CASE STUDY [9]

Encouraged by the reasonably good agreement between analytical predictions and laboratory results, an attempt was made to study the behavior of the sixth story of the Military Personnel Records Center in St. Louis, in which the roof collapsed during a 30-hour fire on July 12-13, 1973 [10,11]. The complex history of the fire, the complex structural system of the building, the lack of accurate records of fire spread, intensity, and structural response, make detailed study of behavior very difficult. Nevertheless, correlation of observed behavior and analytical predictions demonstrated good agreement and provided explanations for observed failures which could not otherwise be explained.

Several observations can be made from the results of the MPRC case study:

1. The deflected shape of the roof after 3 hours of fire exposure (Fig. 5) is shown in Fig. 8. Extrapolating the calculated horizontal displacement of 1.2 in. (30 mm) for one bay, the maximum E-W displacement at the corners of the roof would be about 40 in. (1 m) each. The displacements measured after complete cooling were about 20 in. (0.5 m) each. Considering that some recovery must have taken place during cooling, but that complete recovery could not be achieved because of permanent damage in the slab and the supporting columns, the estimated deflection of 40 in. (1 m) during the fire appears to be reasonable.

2. The relative depression of the slab at the column support is contrary to the normal deflected shape and is primarily due to the very rigid restraint of the slab by the column capitals. The zones of relative depression of the roof slab could be observed on aerial photographs as small ponds of collected water. Locations of these ponds with respect to the structural frame could be established from the photos and correlated well with the locations of calculated depressions over the columns.

3. Calculated bending moments in the roof slab showed a sign reversal in the center region of the bay so that steel reinforcement was required under fire exposure in the top of the slab, while for service load conditions, top steel was provided in the end-quarters only. Absence of top steel in the middle portion of the bay would indicate the likelihood of failure in the vicinity of the top steel cut-off. This was fully supported by observations, as large portions of the roof slab seemed to have ripped along the lines where the top steel was discontinued.

4. Calculated moments and shears in the exterior columns under fire exposure increased dramatically. The maximum moment increased more than two-fold as compared to maximum moments under service conditions, and the shear in the column increased three-fold. Moments and shears also increased in the interior columns, but the increases were less pronounced. Calculated moments reached the ultimate, but did not exceed it significantly. A few moment failures were observed in columns, but in most cases, the columns failed in shear. Calculations showed that internal cracking of concrete reduced the effective (uncracked) area to about 18 percent of gross area and thus reduced shear capacity greatly. The amount of lateral ties in the columns was nominal and thus did not contribute to shear resistance. While the shear capacity of the uncracked columns would have been sufficient to resist the increase in shear forces, the extensive degradation of the interior core reduced the shear capacity to such an extent that on the basis of calculation, shear failures were estimated at about 2-1/4 hours. Shear failures observed after the fire support the general prediction of a dominant shear failure mechanism in the columns.

RESPONSE OF COLUMNS TO FIRE [12]

A pilot study to explore the effects of fire characteristics and of structural restraint on the response of reinforced concrete columns in a multistory frame building was carried out. To provide a realistic basis for the study, a typical 12-story reinforced concrete building was selected, and responses of the basement and 11th floor columns were determined analytically. The fire environments were characterized by two time-temperature curves, assigning two emissivities for each. Column behavior during the 1-hr. fire exposure was studied using the computer programs FIRES-T and FIRES-RC.

Axial restraint stiffness was modeled by springs at the upper and lower ends of each column; the spring constants were calculated using linear elastic behavior of the surrounding structure and were assumed to be constant throughout the fire.

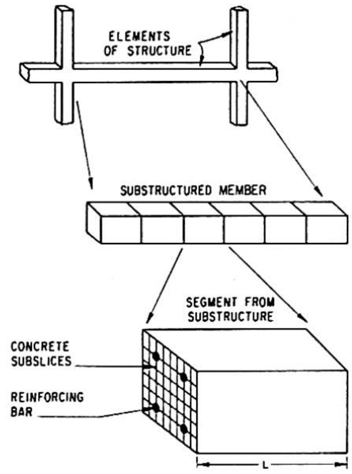

FIG. 1 GEOMETRIC IDEALIZATION

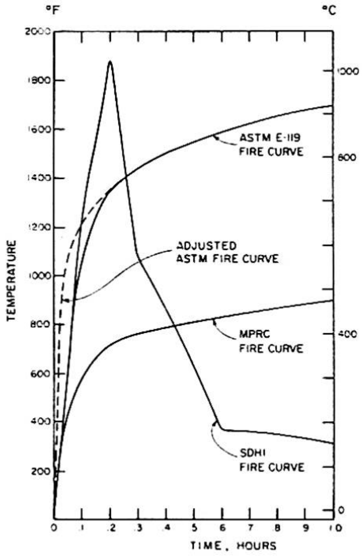

FIG. 5 FIRE CURVES

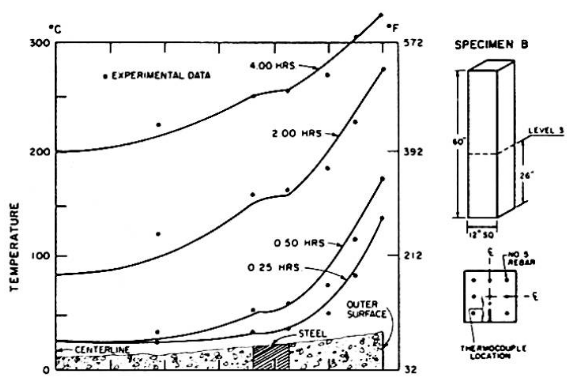

FIG. 2 COMPARISON OF EXPERIMENTAL DATA TO CALCULATED TEMPERATURES

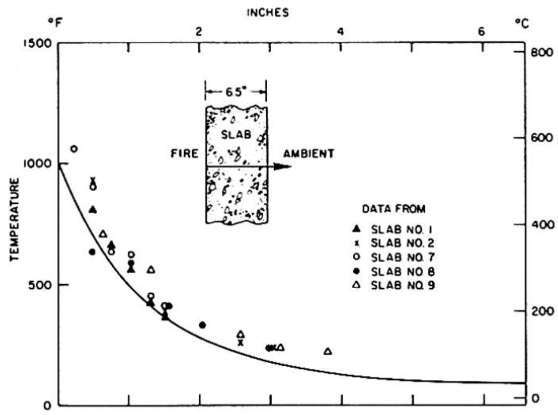

FIG. 6 COMPARISON OF EXPERIMENTAL DATA TO CALCULATED TEMPERATURES - 30 MINUTE EXPOSURE

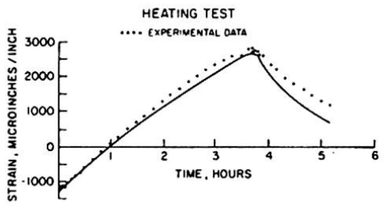

FIG. 3 COMPARISON OF EXPERIMENTAL DATA TO CALCULATED STRAINS

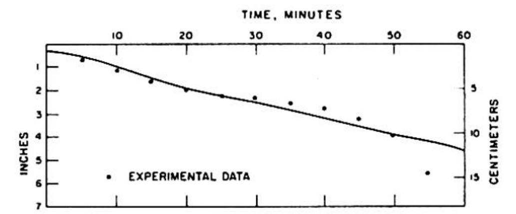

FIG. 7 COMPARISON OF EXPERIMENTAL DATA TO CALCULATED DEFLECTIONS

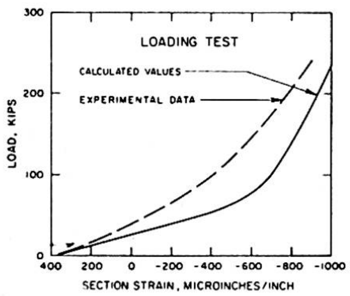

FIG. 4 COMPARISON OF EXPERIMENTAL DATA TO CALCULATED STRAINS

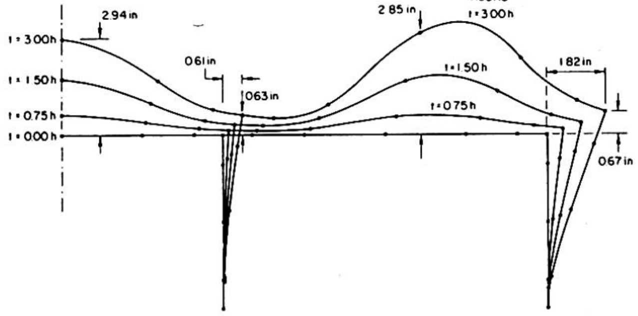

FIG. 8 FRAME DEFORMATIONS

Characteristics of the columns and thermal response in terms of maximum steel temperatures after 1-hr. fire exposure are summarized in the table below.

Column Location	Column Size in x in	Concrete Cover in.	Steel Reinf. Ratio	Initial Axial Load kips	Axial Restraint Stiffness kips/inch		Type of Fire & Emissivity				Maximum Steel Temperature °F (°C)			
							ASTM		SDHI Fig. 5		ASTM		SDHI	
					K_U	K_L	A9	A3	S9	S3	A9	A3	S9	S3
Basement	20 x 20	1.5	0.032	670	365	∞	0.9	0.3	0.9	0.3	761(405)	522(272)	401(205)	263(128)
11th Fl.	20 x 20	1.5	0.012	100	65	2000	0.9	0.3	0.9	0.3	637(336)	421(216)	368(187)	223(106)

Structural response in terms of relative deflections, steel and concrete load, and section degradation are summarized in the following table.

Column Location	Type of Fire	Deflection 1-hr % of initial [1]	Steel Load 1-hr % of initial	Concrete Load, 1-hr % of initial	Initial Time of Crushing hrs [2]	Crushed Area, 1-hr % of total	Initial Time of Cracking hrs	Cracked Area, 1-hr % of total	Initial Time of Yielding hrs [3]	Concrete Area, 1-hr % of total [4]	Flexural Stiffness 1-hr % of initial
Basement	A9	-397	435	42	0.25	34	0.45	50	0.50	16	30
	S9	-81	385	39	0.20	20	0.40	49	0.50	31	54
	A3	-250	353	54	0.55	20	0.70	54	0.75	26	25
	S3	+51	321	49	NC	1	0.65	19	NY	80	70
11-th Story	A9	-6413	-237	166	0.40	20	0.20	65	NY	15	21
	S9	-1875	+1033	18	0.20	19	0.15	74	NY	7	21
	A3	-5788	-2000	316	NC	3	0.35	80	NY	17	20
	S3	-275	+1128	12	NC	0	0.20	89	NY	11	32

1 minus sign indicates change from initial shortening to elongation or change in load from compression to tension.

2 crushing of entire 1 in. (2.54 cm.) thick peripheral layer; NC signifies that within 1 hr. there was no full crushing of the outer layer, although partial crushing (e.g. at corners) may have taken place.

3 NY signifies that no yielding of steel reinforcement within 1 hr. fire duration has taken place.

4 effective concrete area remaining after cracking and crushing.

Geometrically, the two columns (basement and 11th floor) differ only in the amount of steel reinforcement in each. The fire endurance rating obtained from a standard fire exposure test would be the same for both columns. Yet, as shown by the tables above, the thermal response differs greatly with type of fire, and the structural response differs greatly with both type of fire and amount of axial restraint.

CONCLUSIONS

The studies carried out to-date indicate that for reliable prediction of response, pseudo-fire characteristics should include emissivity in addition to realistic time-temperature models.

The structural response of reinforced concrete structures is sensitive to the following characteristics: variations in thermal coefficients of expansion, stress-strain relationships and creep in both tension and compression, inelastic deformation associated with unloading, and fracture (cracking, crushing, rupture).

In reinforced and prestressed concrete, cracking of interior concrete due to thermal gradients greatly reduces strength and stiffness of the element exposed to fire. This phenomenon is strongly influenced by pseudo-fire characteristics such as rate and duration of heating, peak temperature, and rate of cooling.

Four categories of fire with corresponding levels of tolerable damage have been suggested for a more rational design of structures for fire resistance. Standard tests of fire resistance do not provide sufficient information for such design. Information regarding loss of strength and stiffness, as well as the magnitude of fire-induced forces and deformations in structures for different fire conditions, must be considered in design.

ACKNOWLEDGEMENTS

The Fire Research Program at UCB has been supported by a National Science Foundation grant. The contributions of research staff and students to the work described in this paper [1-4, 7-9, 12] and the cooperation of the PCA and GSA in providing data used in this study are gratefully acknowledged.

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SUMMARY - Mathematical models developed for predicting thermal and structural response of reinforced and prestressed concrete frames in fire environments are substantiated by laboratory tests and case studies. Suggestions for a more rational design of structures for fire resistance are included.

RESUME - Des modèles mathématiques ont été étudiés afin de prédire le comportement thermique et structural de cadres en béton armé et précontraint dans un incendie. Ils ont été établis à partir d'essais en laboratoire et de cas réels d'incendie. Des propostions sont faites pour un dimensionnement plus rationnel des structures devant résister au feu.

ZUSAMMENFASSUNG - Mathematische Modelle wurden entwickelt, um das thermische und strukturelle Brandverhalten von Rahmen aus Stahl- und Spannbeton vorauszusagen. Sie wurden aufgrund von Laboruntersuchungen und wirklichen Brandfällen aufgestellt. Eine rationellere Berechnung der brandwiderstehenden Tragwerke wird vorgeschlagen.

IIIc

Structural Behaviour of Reinforced Concrete at Transient High Temperatures

Comportement structural de béton armé à de hautes températures passagères

Strukturelles Verhalten von Stahlbeton gegenüber vorübergehenden hohen Temperaturen

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1. Introduction

When a reinforced concrete is subjected to heating, all structural conditions can be time-dependent. The structural behaviour of composant materials is influenced by many parameters such as stress and temperatures. Concrete deformation, in particular, would involve many parameters. Therefore structural analysis on reinforced concrete in such condition can not but depend on rather crude approximations in order to avoid an entirely empirical approach which can lead to either uneconomical processes or dubious results.

Finite theory in which the continuity of quantity can be ignored has been used in a fire research defining the fire resistance by hours. This philosophy may be as well applicable to the definition of a relationship of a cross section element between a bending moment, an axial force and deformations in a linear structure (beams and columns) at given time---there can be only a finite number of cases in the relationship.

2. Sectional Theory

It is common in the structural analysis of reinforced concrete to take the cross section as the smallest element in linear structures. It may be convinient to divide the section into parts which may be subjected to different stress conditions according to their positions. Such a part can be represented by an imaginary linear element. Although these linear elements are distributed in three dimensions, two dimensional elements will be discussed, since the temperature change in the axial direction can be small compared with that in other directions. In the section these elements should comply with both the compatibility and the equilibrium. From the compatibility condition, the strain distribution should be continuous. From the equilibrium the resultants on the section should be zero with respect to certain point in it. The collection of all sections in a structure can give the behaviour of the whole structure. The behaviour of the linear element is based on the constitutive equations of material composing the element.

3. Behaviour of Linear Element

The knowledge on mechanical characteristics of the linear element should be obtained by the experiments with respect to composant materials. In a normal condition, this may be given as a stress-strain relationship. However, many parameters can be related to the transient deformation. Therefore the constitutive equation of the stress and that of the stored energy may take the following parameters in the function f( ) and g( ). respectively:

stress: $\sigma = f(\varepsilon, T, t, H)$ ....(1)

where ε = strain at time in question
T = temperature at time in question
t = time in question
H = historical term between the birth of material and the time in question

stored energy per unit volume: $E = g(\sigma, \varepsilon, T, t, H)$ ....(2)

Today these equations have not been fully cleared with actual structural materials, let alone these on concrete at transient high temperatures. Therefore these may have to be constructed combining experimental results previously offered by many different workers, in so much as the unique value of the stress and the stored energy can be given with all conditions defined between the material birth and the time in qestion.

4. Structural Conditions of Sectional Behaviour

The deformation of section elements, on which the thermal and the historical conditions can be difined, is due to the resultant of bending moment and normal force on the section of area A, when the stored energy is as follows, refering to Fig.1:

$\int_A dP_e = P_i$ ....(3)

$\int_A dM_e = M_i$ ....(4)

$\int_A dU_e = U_i$ ....(5)

where dP_e = normal force applied to the linear element
P_i = normal force resultant on the section i
dM_e = bending moment due to the linear element with respect to a given point
M_i = bending moment with respect to a given point
dU_e = energy stored in the linear element
U_i = energy stored in the section

Therefore we have three equations to give a strain distribution. A pattern of strain distribution on the section should comply with these three mechanical conditions in a given thermal condition. On top of it, three assumptions may be taken---(1) the temperature distribution can be independent from the stress distribution, (2) the strain distribution can be expressed in a finite number of parameters, (3) some parameters can be digitallized or---these parameters can be expressed in an integer number corresponding to a value of the parameter in their range. We are to find out the pattern of the strain distribution out of possible patterns. Since the available equations (1) and (2) would innevitably contain some vagueness, the rigorous mathematical solution for the strain distribution may not be practical. Under these assumptions, the equations (3), (4) and (5) can be written as follows:

$$\Sigma \Delta P_e - P_i = 0 \qquad \text{.............................(6)}$$

$$\Sigma \Delta M_e - M_i = 0 \qquad \text{.............................(7)}$$

$$U_i = \Sigma (\Delta U_e) \qquad \text{.............................(8)}$$

where U_i = stored energy in section i

5. Numerical Example

This is an example which has an uniform cross section and expansion restriction. Conditions of a section at time t are as shown in Fig.1. An arbitrary linear element at position (y,z) is subjected to the stress corresponding to its strain $\varepsilon_t(y.z)$ temperature $T_t(y,z)$ and its history $H_t(y,z)$. The following assumptions are taken:

(1) plane theory on the strain distribution,
(2) deformation of the section on y-z plane can be ignored,
(3) temperature is monotonically increasing,
(4) only the maximum stress in the past after heating began is taken into account as the historical stress of the history effects and
(5) time elapses in the interval of 0.15 minutes(not continuously).

5.1. Constitutive Equations

Concrete equation: from Malhotra's experimental results(Ref.1), the strength f' at T (difference between material temperature and constant room one in the centigrade unit) with t=0 (heating time in the unit of minutes) is:

$$f' = f_r (1 - T/1000)^2 \qquad \text{.............................(9)}$$

where f_r = strength at T=0 and t=0

From Furumura's work(Ref.2), the virgin stress σ may be expressed in a second degree equation with respect to the strain :

$$\sigma = \varphi(\varepsilon - \varepsilon_x)\left\{\varepsilon - (2\varepsilon_T - \varepsilon_x)\right\} \qquad \text{..................(10)}$$

where where $\varphi = -f'/(\varepsilon_T - \varepsilon_x)^2$

$\varepsilon_T = 0.002 + 0.007\left\{1 - \exp(-t/100)\right\}$:strain corresponding to the strength f' according to Rüsch(Ref.3).

$\varepsilon_x = -10^{-5}T$:thermal expansion

$\varepsilon_x \leq \varepsilon \leq \varepsilon_T$

When the stress in question is lower than the historical stress (corresponding to its strain):

$$\sigma = 2f'(\varepsilon - \varepsilon_H)/(\varepsilon_T - \varepsilon_x) + \sigma_H \qquad \text{..................(11)}$$

Also from Furumura's research work the stored energy in a unit volume may be as follows:

$$\Delta U_i = \sigma^2(\varepsilon_T - \varepsilon_x)/4f' \qquad \text{.............................(12)}$$

Steel equation: since its time effect due to relaxation is small compared with that of concrete, the equation may be simplified as follows: refering to Harada's results(Ref.4):

$$\sigma = \begin{cases} -f & \text{:when } E(\varepsilon - \varepsilon_x) < f_T \\ E(\varepsilon - \varepsilon_x) & \text{:when } -f_T \leq E_S(\varepsilon - \varepsilon_x) \leq f_T \\ f & \text{:when } E(\varepsilon - \varepsilon_x) > f_T \end{cases} \qquad \text{........(13)}$$

where $E_S = 2.1 \times 10^6$ Kg/cm² :Young's modulus
$f_T = 5000 - 5T$(Kg/cm²) :yield stress

Similarly as in the concrete equation, the stored energy may be expressed as:

$$\Delta U_i = \sigma^2/2E_S \qquad \text{.............................(14)}$$

Covering a diminutive area ΔA, a linear element will give rise the followings.

$$\Delta P_i = \sigma \Delta A, \quad \Delta M_i = y\, \sigma \Delta A, \quad U_i = \sum \Delta A . \Delta U_e . \Delta x_i$$

Temperature distribution at time t is that in the author's past work(Ref.5), in which the ambient temperature rise was given in the time interval according to the time temperature curve of IF code. A solution, corresponding to a given moment distribution and normal force, can be found by the trial-error method with two strain values at each section element, since the plane theory is employed and the two interim values of strain can give strain of all other linear elements in the same section. Equations (6) and (7) would have some discrepancy from zero at their rights even for the most plausible strains,

Thus,
$$\left\{ (\sum \Delta P_e) - P_i \right\} / P_i = \omega_1 \quad \dots\dots\dots\dots (15)$$
$$\left\{ (\sum \Delta M_e) - M_i \right\} / M_i = \omega_2 \quad \dots\dots\dots\dots (16)$$
$$S_i = \omega_1^2 + \omega_2^2 \quad \dots\dots\dots\dots (17)$$

where ω_1 and ω_2 are discrepancy from zero

A possible pair of two strains should make the least of S_i from the least square sum principle. At the same time the stored energy U_i should be minimum. However, the moments and the normal forces used in the above are the function of deformations of all section elements. Therefore, these also must be found by trial in order to comply with the restraining condition at the ends of the structural member--- the total compatibility should be kept. In this example, the distance between the two ends should be constant. Thus, the moment and normal force at each section may be computed by trials. The value of stress and strain at all linear elements should be stored for the next time step, once the solution is found.

Each section element is subjected to a different thermal condition. Thus the rate between the length of heated parts and the rest should be taken into account. The flow chart of this procedure is shown in Fig.2, the section analysis being applied between s-1 and s-N. Fig.3 shows the calculated results of heat resistance against the applied moment in three cases.

6. Conclusion

Even the simplest structural member can behave as the complicated machinery when heated. The results of the numerical example shows that the complete restriction against the expansion can reduce the heat resistance considerably. When the end rotation is restrained, the restriction can normally increase the resistance, because the whole structural system will be changed and the structural redundancy can support after the first fracture took place at the particuler section. Therefore the end effects should be divided into two: one against the axial expansion, the other against the rotation. However the assessment of resistance has to take the loading condition into account. Therefore the simple example could suggest that the most severe end effects may be caused by the complete expansion restraint, keeping rotation free, and that the higher the rate of the heated part in the structure, the shorter the heat resistance. Since it is difficult to obtain the experimental condition similar to that in the actual fire condition, particulaly on the full-scale specimens, the assessment of the behaviour and the resistance should be based on the data on both the temperature distribution change and the constitutive equations on materials.

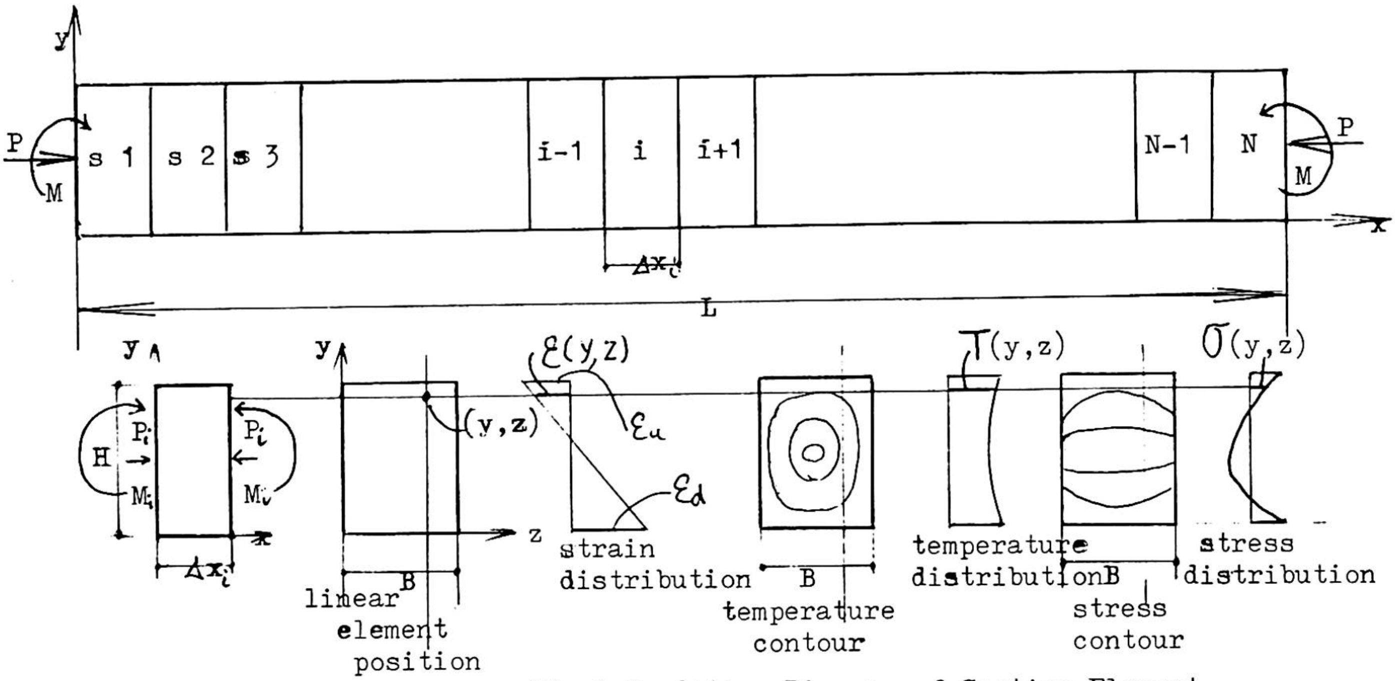

Fig.1-Condition Diagram of Section Element

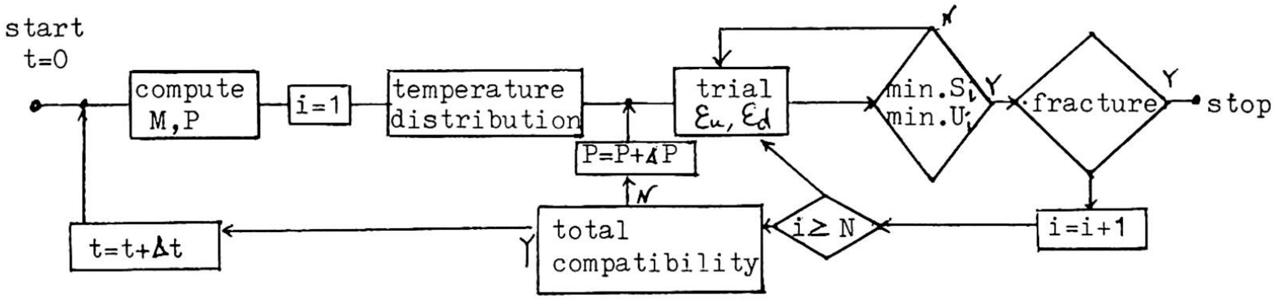

Fig.2-Flow Chart of Numerical Example

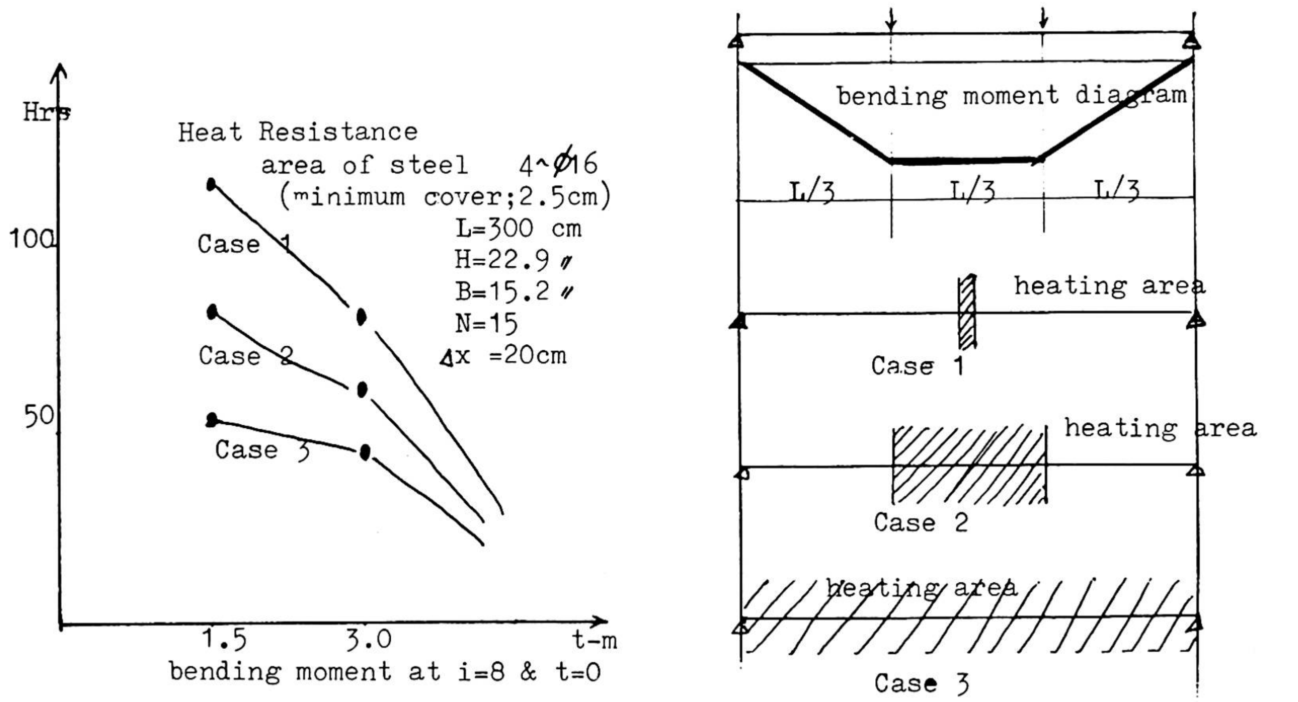

Fig.3-Case Study on Heat Resistance

Acknowledgement

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SUMMARY

The study centres on the mechanical response of a sectional element in beams and/or columns subjected to incessant changes such as temperature distribution, a bending moment and an axial force. If these changes can be ignored in the analysis, the procedure should give a result similar to the ones in ordinary structural analysis on the behaviour of section elements. For the normal analysis under a constant room temperature is a special case among the cases taking temperatures into account.

RESUME

L'étude se concentre sur les réactions mécaniques d'un élément de poutre et/ou de colonne sujet à des changements incessants tels que cas de charge, une distribution de température, un moment de flexion et une force axiale. Si ces changements peuvent être ignorés dans le calcul, la procédure devrait donner un résultat similaire à ceux d'un calcul ordinaire sur le comportement d'éléments. Car le calcul conventionnel sous température ambiante constante est un cas spécial dans les cas tenant compte des températures.

ZUSAMMENFASSUNG

Im Mittelpunkt der Untersuchung steht das mechanische Verhalten eines Elementes in Balken und/oder Stützen, die einem dauernden Wechsel der Temperaturverteilung, des Biegemomentes und der Normalkraft ausgesetzt sind. Wenn diese Aenderungen in der Berechnung ausser Acht gelassen werden können, sollte das Verfahren ein ähnliches Resultat ergeben wie bei der üblichen Strukturanalyse über das Verhalten von Elementen. Die normale Berechnung unter konstanter Raumtemperatur ist ein Sonderfall unter den in Betracht gezogenen Temperaturfällen.

Tragverhalten brandbeanspruchter Bauteile

Load Bearing Behaviour of Structural Members in Fire

Comportement des éléments en béton armé soumis au feu

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1. Grundlagen

Ausgang jeder realistischen Analyse zum Verhalten brandbeanspruchter Bauteile ist die Berücksichtigung der Temperaturabhängigkeit aller Werkstoffdaten. Darunter sind sowohl die thermischen Eigenschaften, wie z.B. Wärmeleitzahl und Temperaturausdehnungskoeffizient zu verstehen als auch sämtliche mechanischen Eigenschaften, wie Festigkeits-und Verformungskenngrößen. Die Abbildungen 1 und 2 zeigen für die letztgenannten Parameter den Verlauf der Rechenwertfunktionen, wie sie für die numerische Analyse benutzt wurden [2]. Charakteristisch für alle Werkstoffdaten ist ihr nichtlinearer Verlauf. Die herkömmliche Formulierung eines Werkstoffgesetzes der Art $\sigma = \sigma(\varepsilon)$ muß erweitert werden zur Formulierung $\sigma = \sigma(\varepsilon,T)$. Man erhält dann eine Schar von σ-ε-Beziehungen mit dem Scharparameter der Temperatur T [1].

Jüngste Untersuchungen zum Kriech-und Relaxationsverhalten von Beton unter hohen instationären Temperaturen [4] ermöglichen es, auch Zwängungsprobleme aus Bauwerksinteraktionen wirklichkeitsnah zu erfassen. In jenen Fällen ist es erforderlich, die spannungserzeugende Dehnung ε_σ, aufgefaßt als Summe aus thermischer (ε_{Th}) und lastabhängiger Dehnung (ε_p) um einen Kriech-bzw. Relaxationsanteil ε_{mc} (Abbildung 3) zu erweitern:

$$\varepsilon_\sigma = \varepsilon_{Th} + \varepsilon_p + \varepsilon_{mc} \qquad (1)$$

Da der Querschnitt durch einen Temperatur-Gradienten belastet ist, wird das näherungsweise gleichförmige Ausgangsmaterial in jedem Punkt anders beeinflußt. Zur Beschreibung und numerischen Erfassung dieses komplexen Verhaltens dient eine zweidimensionale Querschnittsdiskretisierung [2,3].

Die nachfolgend aufgezeigten Ergebnisse einer rechnergesteuerten Analyse des Brandverhaltens belasteter Stahlbetonbauteile soll exemplarisch die Leistungsfähigkeit des Verfahrens andeuten. Es wurde für die hier präsentierten Ergebnisse eine Temperaturbelastung entsprechend der ETK nach DIN 4102 zugrundegelegt und als Ausgangswerkstoff ein quarzitischer Normalbeton gewählt. Diese Annahmen stellen jedoch keine verfahrensbedingten Einschränkungen dar. Sofern

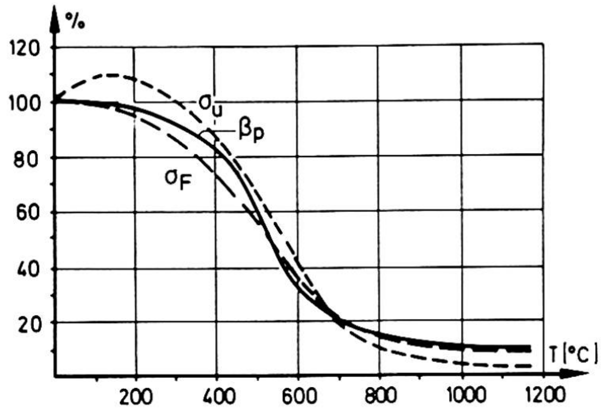


Abb.1: Veränderung der Festigkeitseigenschaften

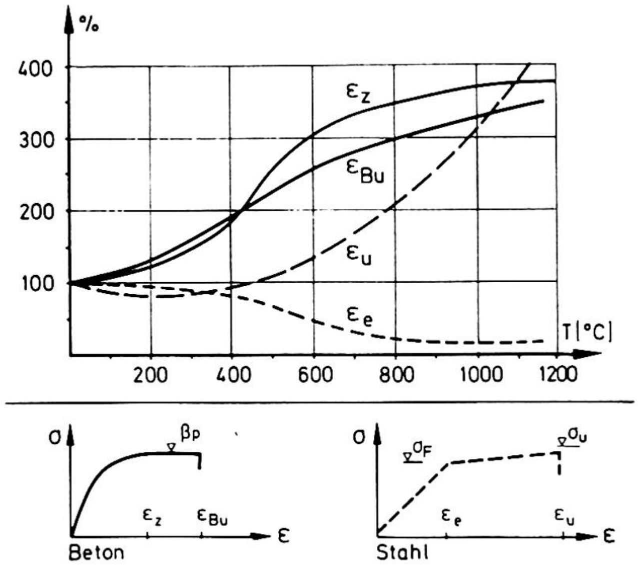


Abb.2: Veränderung der Verformungseigenschaften

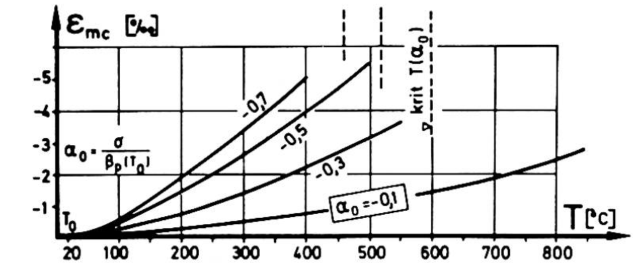


Abb.3: Kriech-bzw. Relaxationsanteile ε_{mc}

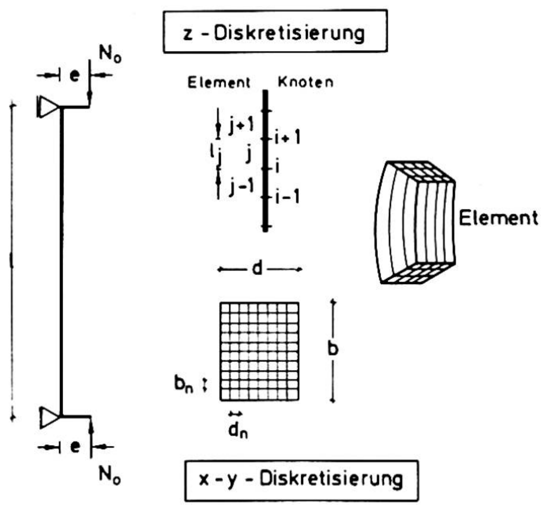


Abb.4: Diskretisierungsprinzip

nichts anderes angegeben ist, wurde für die gerechneten Beispiele ein Bn 350 (β_p = 350 kp/cm²) und BSt 42/50 (σ_F/σ_u = 4200/500 kp/cm²) angenommen.

2. Tragverhalten brandbeanspruchter Einzelbauteile

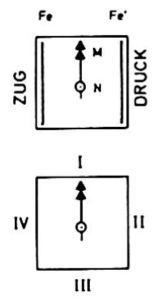
2.1 Stabförmige Bauteile

Bei stabförmigen Bauteilen ist neben der Belastungsart (Biegemoment M, Normalkraft N) der Einfluß der Schlankheit gesondert zu beachten, dies bedingt bei stabilitätsgefährdeten Stützen eine Erweiterung der zweidimensionalen Querschnittsdiskretisierung zu einer dreidimensionalen Systemdiskretisierung (Abb.4), da hier nicht mehr die reine Querschnittstragfähigkeit als Kriterium genügt, sondern zusätzlich der Einfluß aus Theorie II. Ordnung (geometrische Nichtlinearität) zu berücksichtigen ist.

Bei gedrungenen oder vorwiegend auf Biegung beanspruchten Bauteilen kann der Versagenszeitpunkt t_u unter Gebrauchslast N_o, M_o aus der Veränderung der aufnehmbaren Bruchschnittgrößen mittels eines M_u - N_u - t - Interaktions-Diagramms ermittelt werden [1]. Für Rahmenriegel, Unterzüge u.ä. Bauteile, die nur durch geringe Normalkräfte N belastet sind und dies auch während eines

TYP	Temp.- Randbeding. I	II	III	IV	Art der Erwärmung	Wirkung der therm. Krümmung
A	T(t)	T(t)	T(t)	T(t)	allseitig	keine Krümmung
B	T(t)	T(t)	T(t)	T_0	dreiseitig	gegen - sinnig zu M_0
C	T_0	T(t)	T_0	T_0	einseitig	
D	T(t)	T_0	T(t)	T(t)	dreiseitig	gleich - sinnig zu M_0
E	T_0	T_0	T_0	T(t)	einseitig	

Tabelle 1: Thermische Randbedingungen



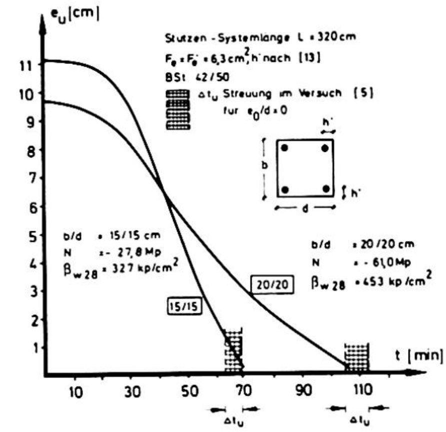


Abb.6:
Versuchsnachrechnung
von Stahlbetonstützen

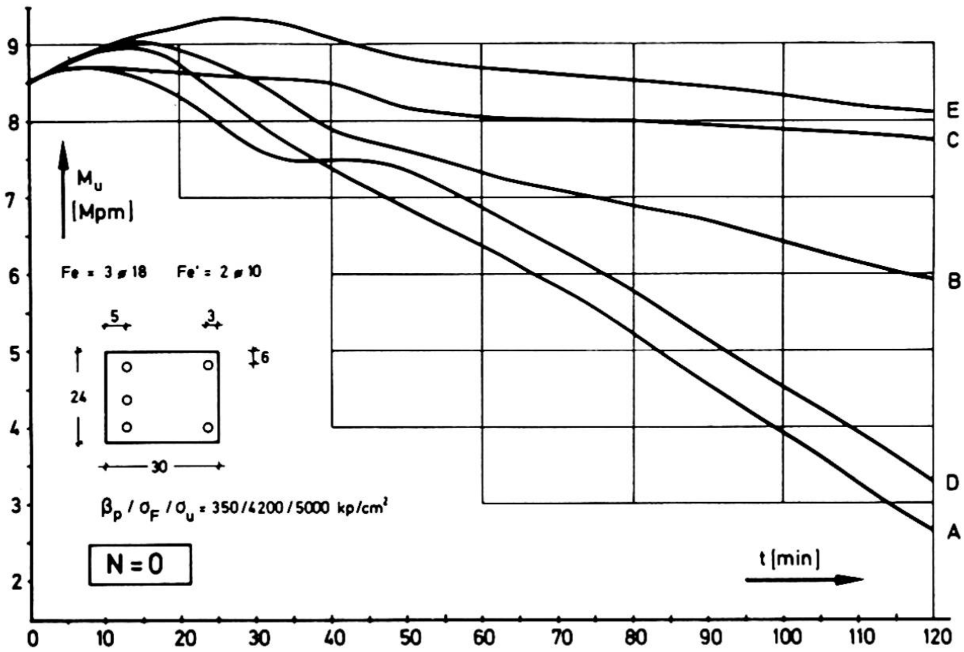


Abb. 5:
Bruchmomentenverläufe eines Stahlbetonbalkens bei unterschiedlichen thermischen Randbedingungen

Brandes bleiben, ist für die Praxis ein $M_u(t)$-Diagramm besser geeignet. Entwickelt man diese Beziehung für $N_o \simeq 0$, können damit i.d.R. die für jene Bauteile praktisch auftretenden N_o-Einflüsse ausreichend abgedeckt werden (Abb.5).Gleiches gilt teilweise für zusätzliche Zwängungen infolge Dehnungsbehinderung im Brandfall. Der Zeitpunkt des Querschnittsversagens kann über die Bedingung

$$t_u = t\ (M_u = M_o) \qquad (2)$$

ermittelt werden.

Sehr häufig wird in der Praxis eine partielle Brandbelastung der Art vorliegen, daß nicht alle Seiten thermisch beansprucht sind. Hierdurch wird eine erhöhte Feuerwiderstandsdauer erreicht, da die Querschnittsdurchwärmung mit all ihren Konsequenzen verzögert abläuft. Tabelle 1 gibt einen Überblick über die i.d.R. zu erwartenden praktischen Fälle (Typen A-E).

Bei Stützen ergibt sich der Versagenszeitpunkt t_u unter der Gebrauchslastkombination N_o und $M_o = N_o \cdot e_o$. Formuliert man den Versagenszeitpunkt als

$$t_u = t\ (e_u = e_o) \qquad (3)$$

so ergibt sich als Traglastcharakteristik ein $e_u(t)$-Verlauf [1]. Die in Abbildung 6 dargestellte Versuchsnachrechnung [5] zeigt eine gute Übereinstimmung zwischen Experiment und Rechnung. Trotz der planmäßig zentrischen Belastung erfolgte das Versagen als Stabilitätsbruch infolge unvermeidbarer Systemimperfektionen.

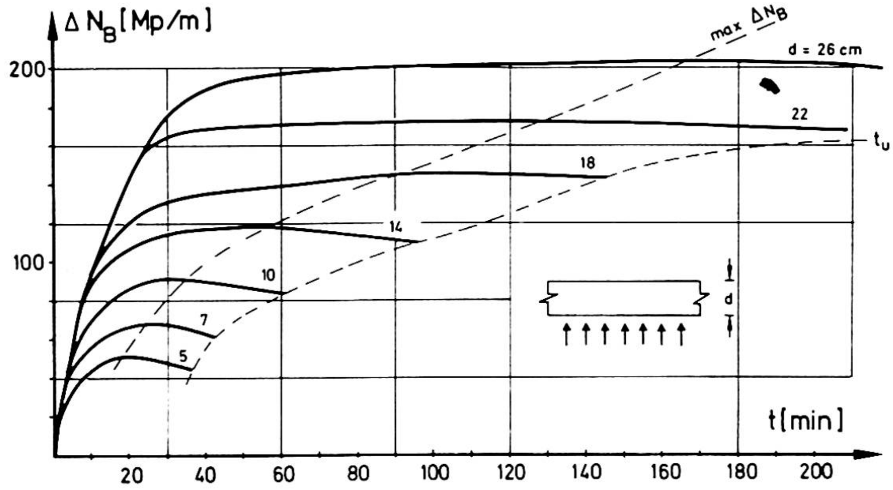


Abb.7:
Zwangskraftentwicklung in Stahlbetonplatten und -scheiben bei einseitiger lokaler Brandbeanspruchung

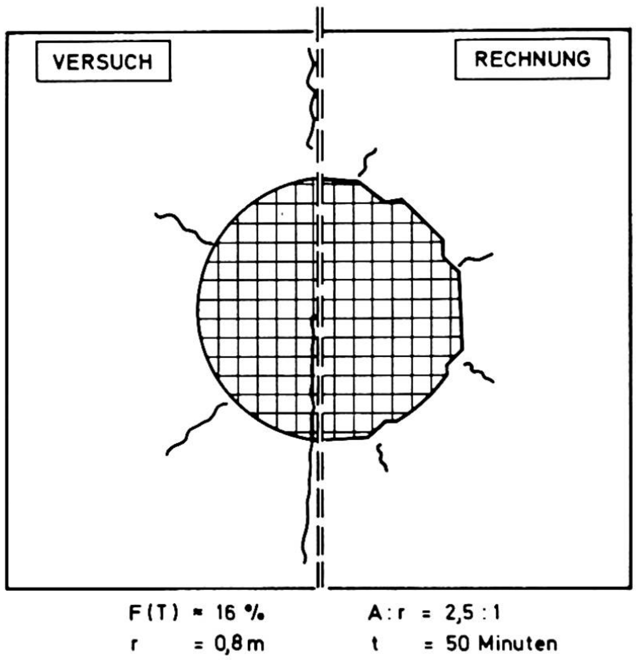


Abb.9:
Zwängungsrisse Vergleich zwischen Experiment und Rechnung

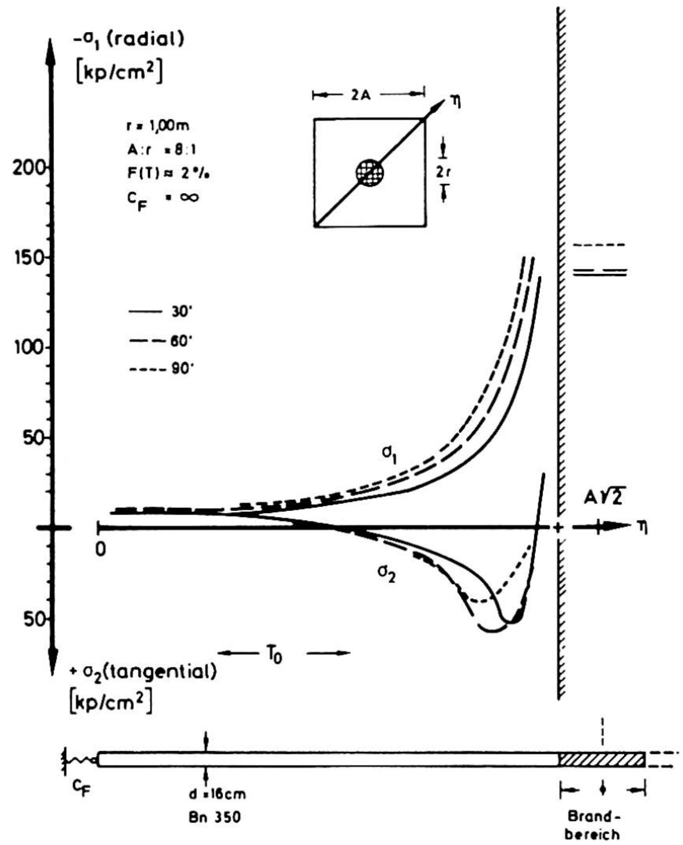


Abb.8:
Axiale Zwangsspannungen; Entwicklung und Verteilung bei lokaler Brandbeanspruchung

In gleicher Weise konnten zwischenzeitlich auch die von den Autoren durchgeführten ersten Versuche an planmäßig exzentrisch belasteten, brandbeanspruchten Stützen numerisch analysiert werden.

2.2 Flächentragwerke

Die folgenden Ausführungen betrachten den Sonderfall der partiellen einseitigen Brandbelastung, wie er z.B. bei lokalen Bränden innerhalb eines mehrfeldrigen Geschoßplattensystems i.d.R. erwartet werden kann. Die benachbarten kalten Bereiche behindern die thermische Dehnung und wecken axiale Zwangskräfte. Größe und Lage der Resultierenden ist dabei zeitabhängig. Hieraus resultiert eine zusätzliche Biegebeanspruchung, die je nach Lage des Temperaturmaximums gleichsinnig oder entgegengesetzt zur Biegemomentenbeanspruchung aus Gebrauchslast sein kann. Zusätzlich ergibt sich ein Scheibenspannungszustand infolge der inneren Zwängung. Größe und Verlauf der resultierenden Zwangskräfte sind dabei nicht nur zeitabhängig, sondern werden wegen des unterschiedlichen Durchwärmungsverhaltens sehr wesentlich von der Plattendicke beeinflußt (Abb.7). Hier muß u.U. mit einem lokalen Druckversagen infolge des Scheibenspannungszustandes gerechnet werden. Örtliche oberflächennahe Druckzerstörungen treten in jedem Fall schon nach kurzen Brandzeiten auf. Neben der Querschnittsschwächung ist die bereits erwähnte zusätzliche Exzentrizitätswirkung von besonderer Bedeutung. Für eine realistische Kalkulation hat sich hier der Hochtemperatur-Relaxationseinfluß entsprechend Gleichung (1), Abb.3, als besonders wichtig erwiesen.

Abbildung 8 zeigt den berechneten Hauptspannungsverlauf für ein spezielles Beispiel. Die Berechnung erfolgte mit Hilfe der Methode der Finiten-Elemente bei Berücksichtigung der physikalischen Nichtlinearität des Materials. Das berechnete Rißbild zeigt eine gute Übereinstimmung mit Versuchswerten (Abb.9). Interessant ist der begrenzte Zugbereich und das steile Umlenken der σ_2-Komponente aus dem Zug-in den Druckbereich bei Annäherung an den Brandbereich; experimentell spiegelt sich dies in dem örtlich begrenzten Rißbereich wieder [3].

3. Gebäudeinteraktion bei lokalen Bränden

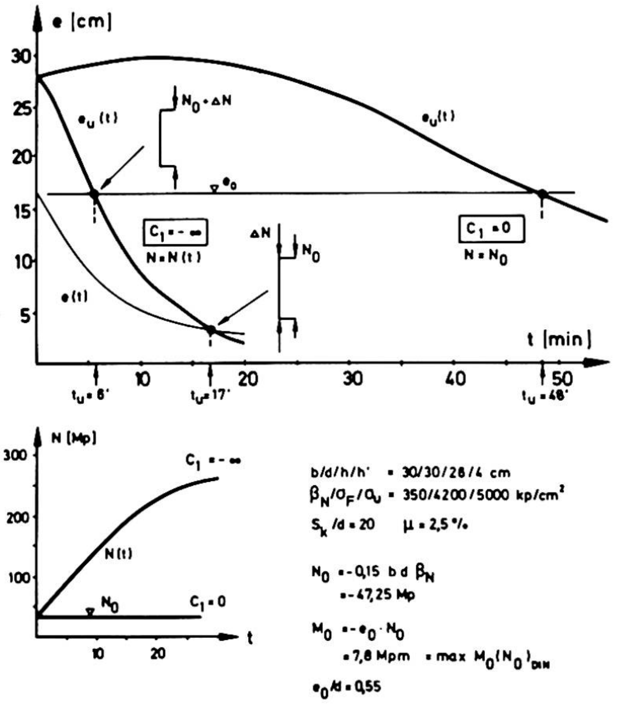
Interaktionen mit den umgebenden kalten Gebäudeteilen resultieren primär aus einer Verformungsbehinderung; sowohl Verdrehungs-, Durchbiegungs- als auch Dehnungsbehinderungen sind je nach System zu erwarten. Unter diesem Aspekt wurden u.a. das Verhalten von Durchlaufträgern, Rahmen und Stützen-Decken-Systemen untersucht [4] .

Im folgenden soll exemplarisch lediglich der gravierende Einfluß einer Längsdehnungsbehinderung auf Stützentraglasten aufgezeigt werden [4]. Die Berücksichtigung der Hochtemperatur-Relaxation (HTR) erlangt hier besondere Bedeutung [2].

Für ein Stützenbeispiel zeigt Abb.10 den Verlauf der beiden Grenz-Traglastcharakteristiken: allseitig erwärmte Stütze mit freier ($c_1=0$) und vollständig behinderter ($c_1 = -\infty$) thermischer Längsdehnung. Die schnelle Entwicklung hoher Zwangskräfte ΔN bewirkt einen raschen Abbau der gleichzeitig aufnehmbaren Momentenbelastung M_o:

$$N(t) = N_o + \Delta N(t)$$

$$M_o(t) = N_o \cdot e_o + \Delta N(t) \cdot e_\Delta(t)$$



Die Feuerwiderstandsdauer des unbehinderten Systems wird dabei um mindestens 60% reduziert. Der Versagenszeitpunkt wird bei Dehnungsbehinderung dabei nicht nur durch die Zwangskraft ΔN allein, sondern auch durch deren Exzentrizität sehr stark beeinflußt. Abbildung 10 zeigt die Auswirkungen für zwei Sonderfälle:

a) $t_u = 6'$ für $e(\Delta N) = e_o$

b) $t_u = 17'$ für $e(\Delta N) = 0$.

Hier muß allerdings bemerkt werden, daß Fall a) mit gleichbleibender Lastausmitte bei wirklichen Bränden wenig wahrscheinlich ist.

Abb.10:
Grenz-Traglastcharakteristiken einer schlanken Stahlbetonstütze und Versagenszeitpunkt-Beeinflussung infolge Zwangskraft-Exzentrizität

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ZUSAMMENFASSUNG

Das charakteristische Verhalten von Stahlbetonstab- und -flächentragwerken bei Brandbeanspruchung wird untersucht. Die numerischen Grundlagen beruhen auf einem Diskretisierungsprinzip, das die wirklichkeitsnahe Berücksichtigung der durchwärmungsbedingten Materialveränderungen erlaubt. Die Ergebnisse bilden die Grundlage einer praktischen Bemessungshilfe für bestimmte Feuerwiderstandseigenschaften.

SUMMARY

The study shows the characteristic behaviour of reinforced concrete members under fire action for columns and beams as well as for plates. The numerical basis is a discretisation principle which allows a realistic consideration of the temperature depending material behaviour. The results form the basis of practical design criteria for determining defined fire resistance.

RESUME

L'étude s'occupe du comportement caractéristique des éléments en béton armé soumis au feu. Il s'agit des colonnes, des poutres et des plaques. Les bases numériques se fondent sur un principe de discrétisation qui permet la considération réelle de la variation du matériau en fonction de la température. Les résultats forment la base d'un dimensionnement pratique correspondant à la réalité.

Experimental Study on Explosive Spalling of Lightweight Aggregate Concrete in Fire

Etude expérimentale de l'écrasement du béton d'agrégats légers dans un incendie

Experimentelle Untersuchung über explosionsartiges Ausplatzen von leichtem Beton in Brandfällen

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1. Introduction

The fire resistance of a structural concrete element is defined as a time, being the period for which an identical test specimen complies with prescribed requirements when subjected to specified conditions of heat and load, and an actual assessment is in accordance with a testing method, JIS (Japanese Industrial Standards) A 1304, in Japan.

The typical performance criteria in this testing method are temperature of steel reinforcement in the element, deformation, surface temperature of unexposed side etc., consequently, all designing of fire resistance is based on the criteria.

To discuss the fire resistance of structural concrete element, however, the phenomenon of explosive breaking off or spalling of concrete in fire cannot be excluded, which has been regarded considerably important especially in lightweight aggregate (expanded shale) concrete. The investigation into the cause of this phenomenon seems not enough so far to make a real performance evaluation of concrete element. In this context an experimental investigation was made under the simulated condition of fire pertaining to the expected three main factors on the spalling, property of aggregate, amount of free water in concrete and mechanical restraint.

2. Factors Regarded

2.1 Type of aggregate

It has been said that the property of aggregate at elevated temperature can be linked up directly with spalling of concrete.(1)

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4 types of aggregate were selected in this experiment on the basis of their property at elevated temperature, i.e. heat resistance, both for ordinary aggregates (N,N') and for lightweight ones (L,L'). N and L are aggregates having rather high heat resistance, N' and L' having lower one. Heat resistance of aggregate was defined as a ratio of number of damaged particles in the total by counting when exposed to an atmospheric condition at 800°C for 30 minutes in an electric oven (1). Heat resistance of selected aggregates is shown in Table 1 with other properties.

Table 1 Properties of Aggregates

Aggregate and its nomination \ Properties			Heat resistance	Specific gravity	Water absorption (24hr)	Water absorption (when used)	Fineness modulus
Coarse Aggregate	Ordinary	N	7.2 - 10.0	2.63 - 2.65	0.7 - 0.9	0.7 - 0.9	6.59 - 6.66
		N'	10.0 - 19.8	2.63	0.8 - 1.0	0.8 - 1.0	6.50 - 6.59
	Lightweight (Expanded Shale)	L	3.0 - 5.0	1.22 - 1.28	11.5 - 12.2	21.2 - 25.1	6.36
		L'	13.0 - 20.0	1.23 - 1.30	8.4 - 8.5	11.2 - 25.2	6.39 - 6.51
Fine Aggregate	Ordinary		-	2.55 - 2.57	1.8 - 2.1	1.8 - 2.1	2.80 - 3.34
	Lightweight (Expanded Shale)		-	1.53 - 1.59	13.9 - 17.1	11.2 - 19.0	2.77 - 2.92

2.2 Amount of free water

Increase of water vapour pressure can be naturally considered to give the influence of spalling (2), therefore, the amount of free water in concrete was controlled in the range of 60 to 180kg per cubic meter of concrete.

Specified level of free water in test panels (as in 3.1) could be obtained being cured for ten days, sealed by polyvinyl sheets after concreting in laboratory, for ensuring a uniformity of strength behaviour of all panels, and air dried at room temperature.

Panels necessary to be decreased the level of free water, they were dried at an atmospheric condition at 70 to 80°C until the specified level, following above curing conditions.

Amount of free water was determined by the direct measurement of weight change of concrete blocks, 30 x 30 x 6cm, made of the same batch and cured under the same condition as each panel, of which four sides were sealed for prevention of evaporation.

2.3 Restraint condition of test panel

The restraint condition (3) was divided into four levels according to diameter of reinforcing bars (deformed) in the ribbed part of test panel classified as D-10, D-13, D-19 and D-22, the number after each hyphen stands for nominal diameter in mm..

3. Test Panels and Fire Test

3.1 Test panels

Dimension of test panels were 1m x 1m x 6cm made of reinforced concrete surrounded by 12 x 12cm ribs, as shown in Fig. 1.

Concrete used was divided into 4 types (N, N', L, L') according to the types of aggregate, each having cement content 340 to 350kg/m^3 (ordinary portland cement), slump about 18cm and water cement ratio 50%.

3.2 Fire test

Fire tests were made by means of the furnace as shown in Fig. 2, which can get the Standard time/temperature curve (Fig. 3) as specified in JIS A 1304, Method of Fire Resistance Test for Structural Parts of Buildings, the period of heating was 60 min..

All the series of tests were conducted at Fire Test Laboratory of Building Research Institute.

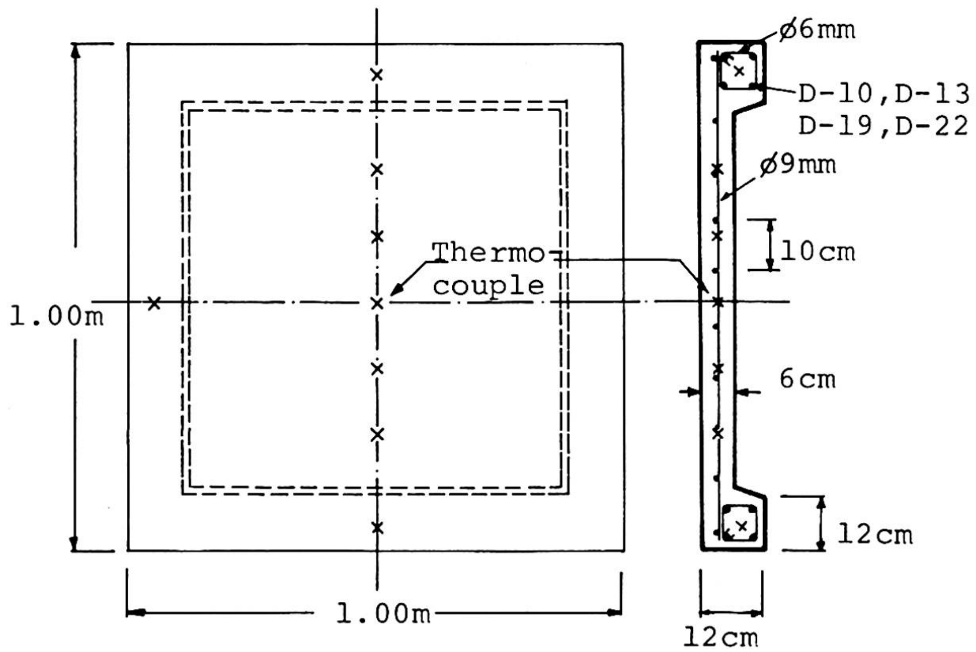

Fig. 1 Test panel

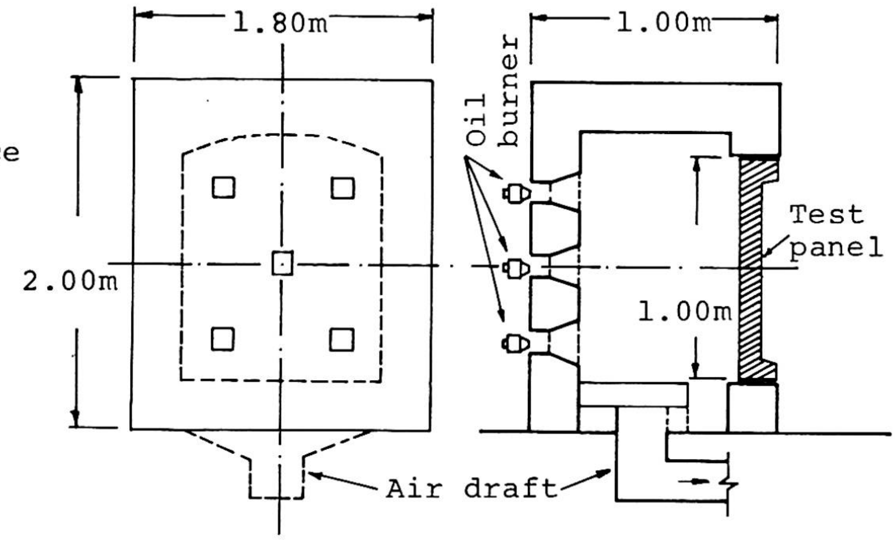

Fig. 2 Furnace for fire test

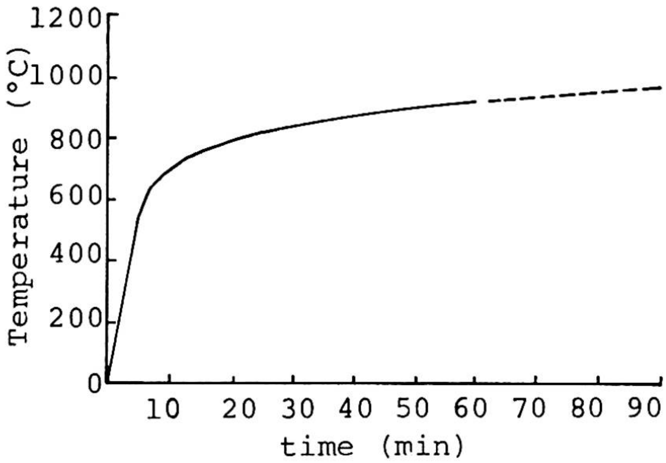

Fig. 3 Standard time/temperature curve (JIS A 1304)

4. Results and Discussion

Data obtained from the series of experiments spalling out of 52 panels are shown in Table 2, Fig. 4 and 5.

Evaluation criteria for spalling in this test are based on the mode of falling off by visual observation and area and volume of the part being spalled of panels.

As shown in these results it can be defined that the amount of free water is affected considerably, that is say, the spalling can not be observed in the most panels at a level of $140kg/m^3$. Spalling can be observed increasingly in the range of 120 to $130kg/m^3$ of free water, and excess spalling can be seen in more than $160kg/m^3$.

On the contrary it can be considered that there is no direct effect depending upon the type of aggregate, and influence of heat resistance of aggregate can be slightly recongnized in ordinary concrete however is not clear in lightweight aggregate concrete.

Regardless of the amount of spalling, phenomenon of falling off in lightweight concrete differed in its mode, likely small amount but successive falling off was observed compared with ordinary concrete.

View point of the influence of restraint condition, it is not so clear in this experiment as shown in Fig. 5.

Table 2 Test Results

		H.R. of Coarse Aggregate	Restraint	60	70	80	90	100	110	120	130	140	150	160	170	180
				Amount of Free Water												
Ordinary Concrete	N	7.2-10.0	D-10					○			○	⦶				
			D-13					○			○					
			D-19					○	○	⦶	⊕	⦶		◑		
			D-22									⊕				
	N'	10.0-19.8	D-10					○			○	●				
			D-13					○			○					
			D-19					○		○	◑	◑		◑		
			D-22									◑				
Light-weight Concrete	L	3.0- 5.0	D-10	○		○			○			○				●
			D-13	○		○			○							●
			D-19	○		○			○	○		○		●		●
			D-22									○				
	L'	13.0-20.0	D-10			○						⦶			●	
			D-13			○									●	
			D-19			○				○		⊕		⊕	●	
			D-22									◑		⊕		

○ No spalling
⦶ Slight spalling
⊕ Rather slight spalling
◑ Fairly heavy spalling
● Excessive spalling

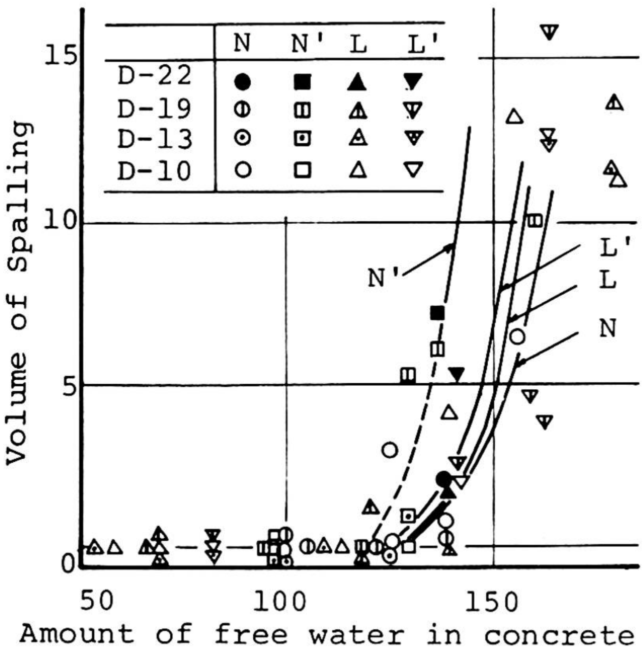

Fig. 4 Volume of spalling related to amount of free water for each type of concrete.

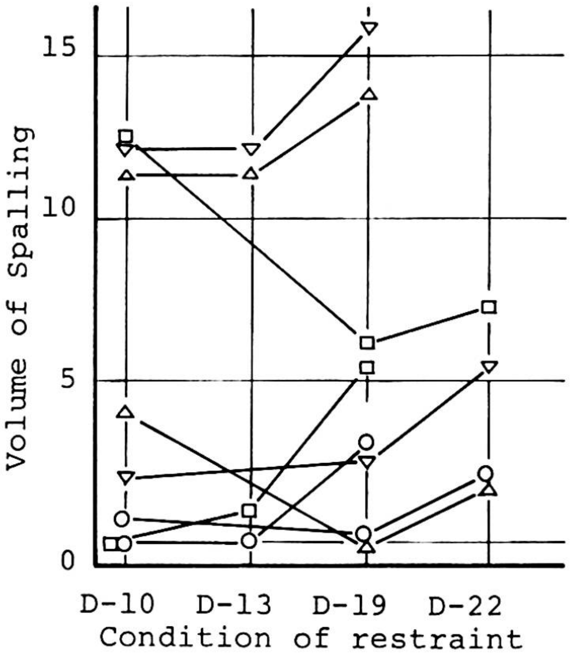

Fig. 5 Volume of spalling related to condition of restraint.

5. Conclusion

From all results obtained in this investigation and the limited data presented in this paper, the following statements can be made;

Explosive spalling point of view amount of free water plays a prominent part in the overall factors and the difference depending upon the type of aggregates does not effect very much.

And finally the explosive spalling can be avoided or minimized in both ordinary and lightweight aggregate concrete when the level of free water in concrete is decreased below the range of 120 to 130kg/m^3.

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SUMMARY

An experimental investigation of explosive spalling of light-weight aggregate concrete compared with ordinary concrete was made related to the property of aggregates, amount of free water in concrete and condition of restraint being expected three main factors of spalling. The results shows that free water plays a prominent part in overall factor, and that the spalling can be avoided when the amount of free water is decreased below the range of 120 to 130 kg per cubic meter of concrete.

RESUME

Une étude expérimentale de l'écrasement du béton d'agrégats dans un incendie a été faite en comparaison du béton ordinaire. On a fait varier la propriété des agrégats, la quantité d'eau libre à l'intérieur du béton et l'intensité de contrainte des éprouvettes. Les résultats montrent que la quantité d'eau libre joue le rôle le plus déterminant parmi les trois facteurs, et que l'écrasement peut être évité si la quantité d'eau libre est diminuée au-dessous de 120 à 130 kg/m3.

ZUSAMMENFASSUNG

Es wurde eine experimentelle Untersuchung über das explosive Ausplatzen von leichtem Beton, verglichen mit gewöhnlichem Beton unter Berücksichtigung der Aggregats-Eigenschaften, des Freiwassers im Beton und des Beanspruchungsgrades der Probestäbe als der drei zu erwartenden Einflussgrössen vorgenommen. Die Resultate zeigten die ausschlaggebende Rolle des Freiwassers; das Ausplatzen lässt sich vermeiden, wenn der Betrag an Freiwasser unterhalb des Bereichs von 120 bis 130 kg pro Kubikmeter Beton liegt.