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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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IIIc

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 multi-bay 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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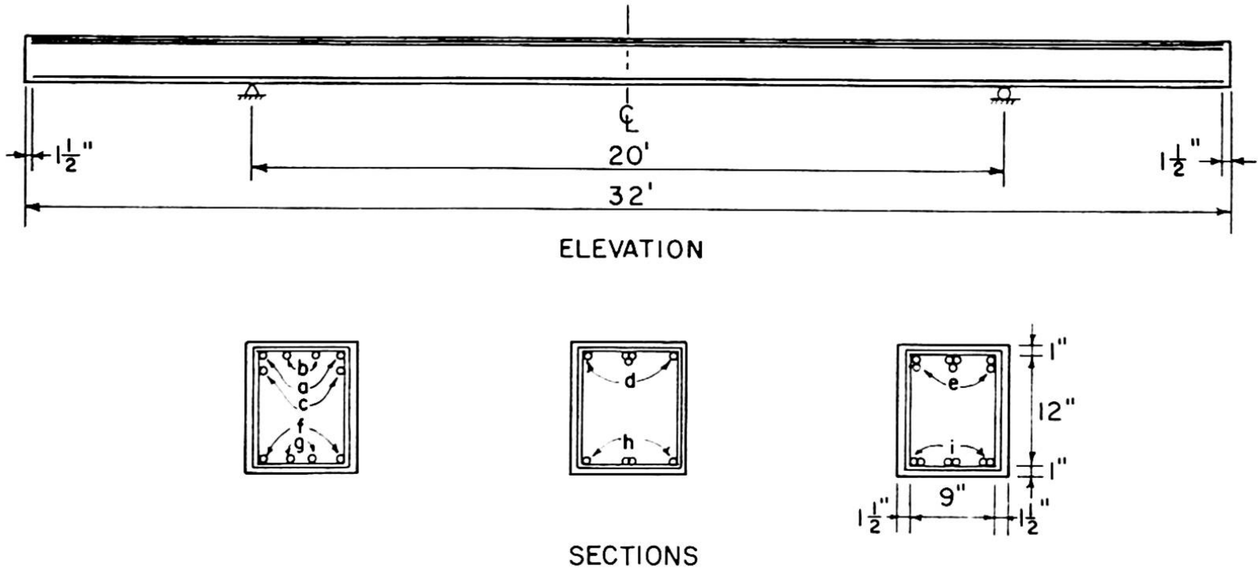


SPECIMEN TYPE and DESIGNATION[1]	TOP BAR[3] (Beginning 1½" from each end)						BOTTOM BAR[3] (Symmetric about ℄)					STIRRUPS (Beginning 3" from each end)						
	DESIGN[1]	a	b	c	d	e	DESIGN	f	g	h	i	DESIGN	SIZE	SPACES				
I AAA	A	2 #6 31'-9"	2 #6 9'-2½"	2 #6 7'-11½"	–	–	A	2 #6 31'-9"	2 #6 11'-8"	–	–	A	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.

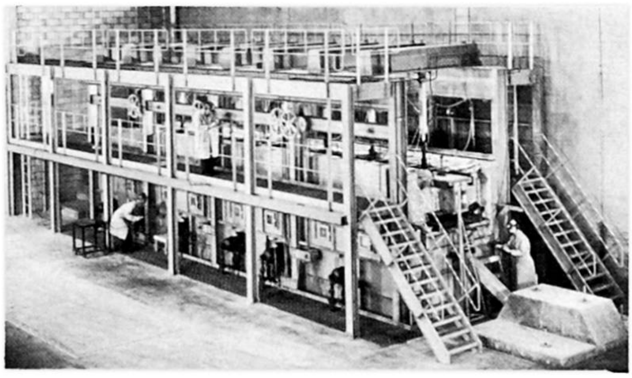
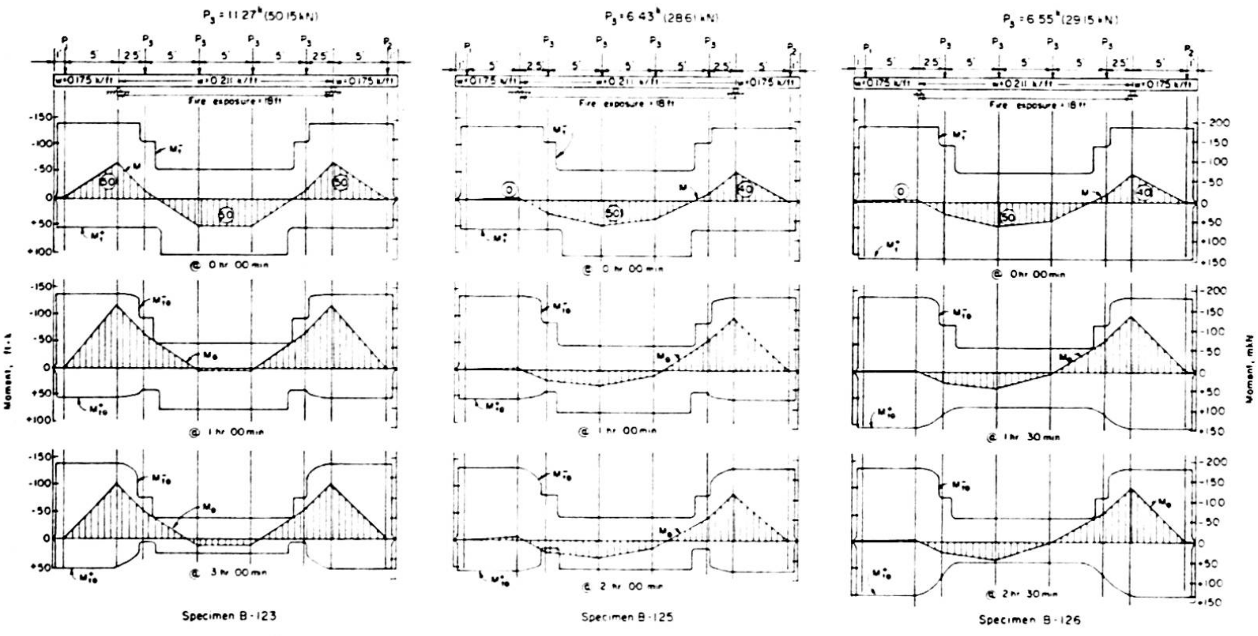


Fig. 2 - Beam Furnace

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

Specimen No.	Specimen Type	Moment Intensity, M/M_t at Start of Test: West Support	Midspan	East Support	Span Loads P_3	Cantilever Loads West, P_1: 0 Hr	Maximum	End of Test	Cantilever Loads East, P_2: 0 Hr	Maximum	End of Test	Avg. Temp. Bottom Reinforcement End of Test		Midspan Deflection End of Test	Test Duration
		%	%	%	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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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		âme			
	largeur	épaisseur	largeur		hauteur	
			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

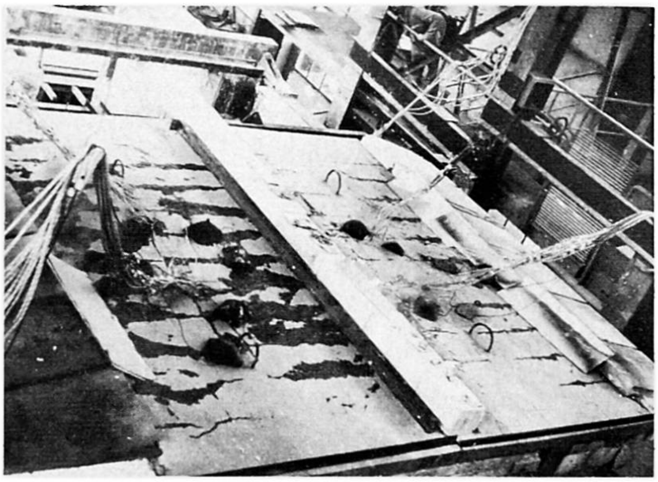
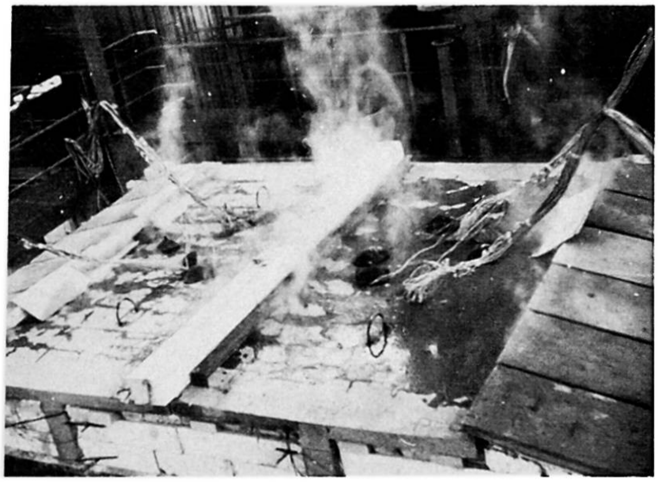
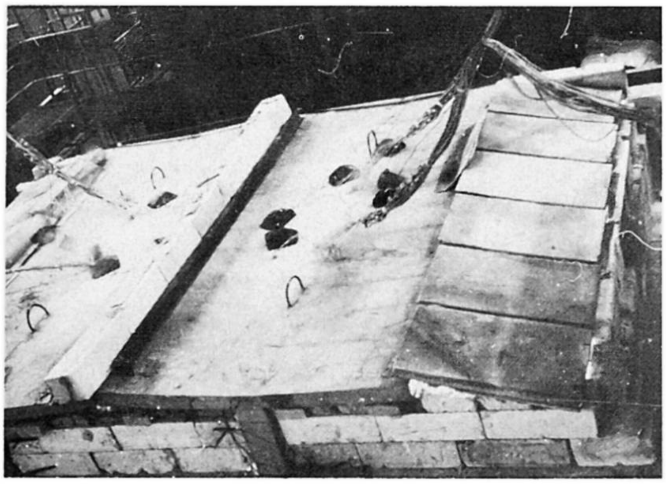


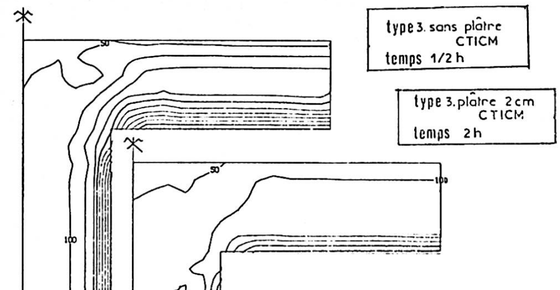
Fig. 1. Vue 18 minutes après l'allumage, la poutre faite avec le béton à plus forte maniabilité "rend" beaucoup plus d'eau.

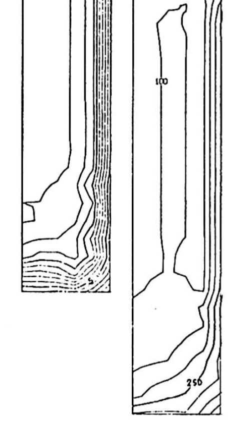
Fig. 2. Vue 21 minutes après l'allumage, la vapeur commence à apparaître.

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.

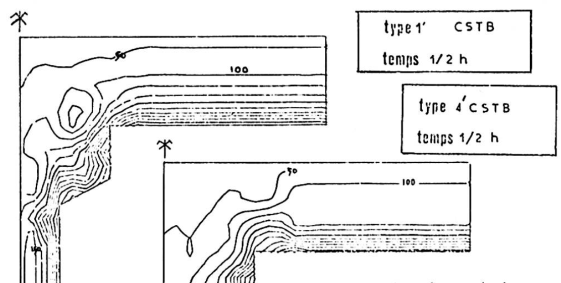
Compte tenu de l'importance du programme, celui-ci a été partagé entre les deux laboratoires du C.S.T.B. et du C.T.I.C.M. où les poutres ont été essayées, soit isolément, soit par lot de deux : les figures 1 à 3 donnent la vue de dessus des poutres de type 3 sans protection plâtre, en cours d'essai.

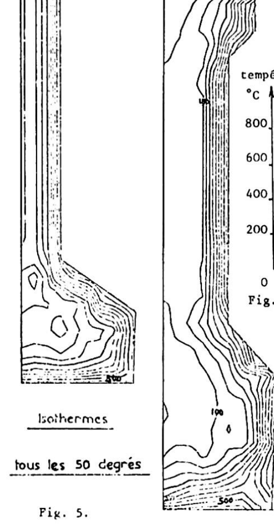
2.3. EXPLOITATION DES MESURES EFFECTUEES.

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 ;

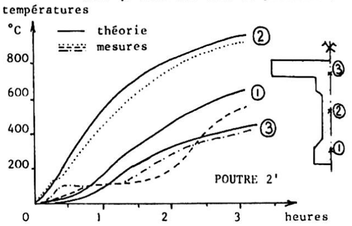


Fig. 6.

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

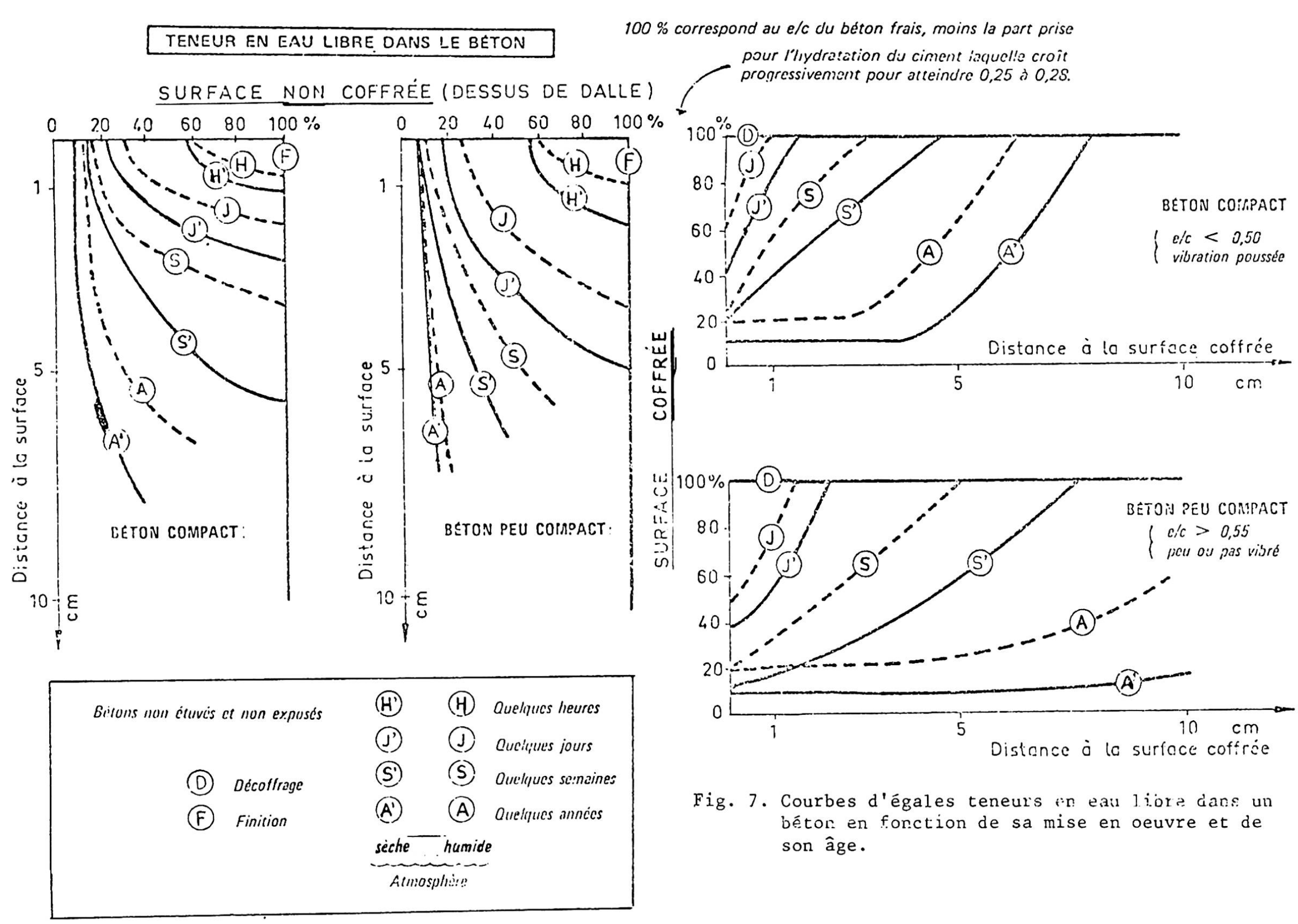


Fig. 7. Courbes d'égales teneurs en eau libre dans un béton en fonction de sa mise en oeuvre et de son âge.

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.

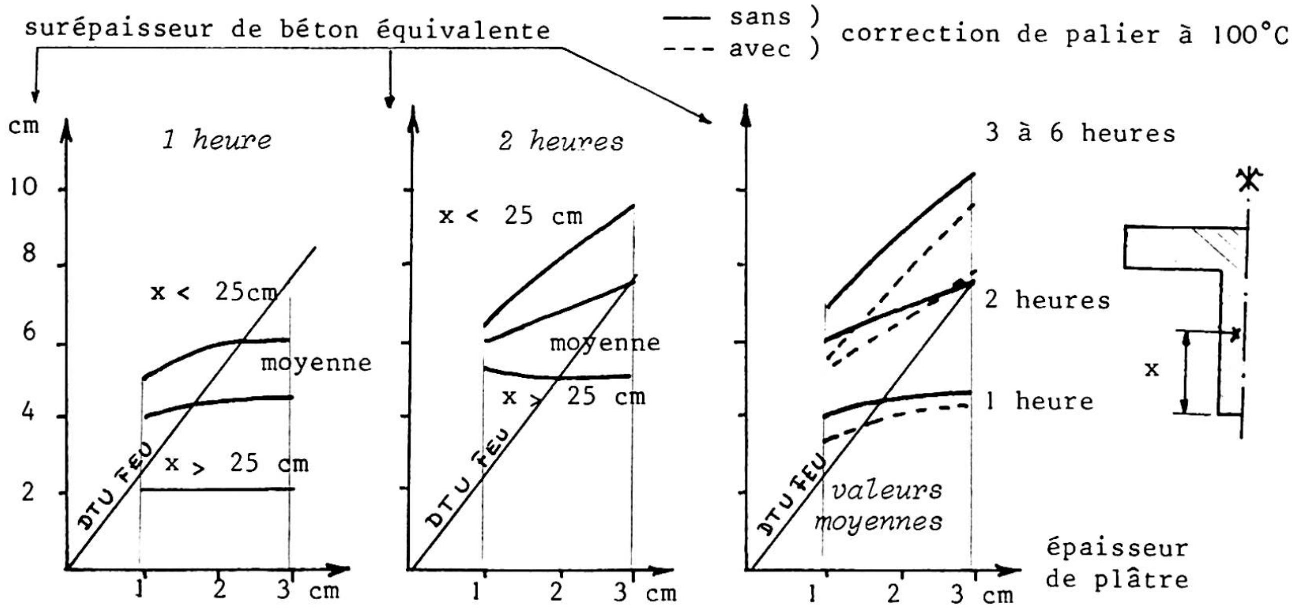


Fig. 8.

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.

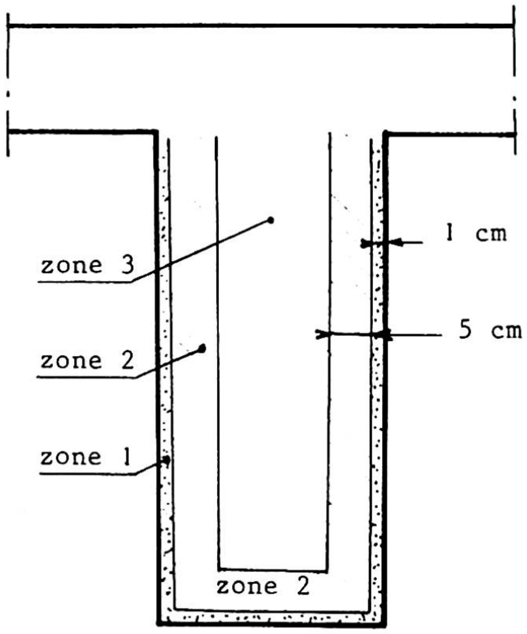


Fig. 9.

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.

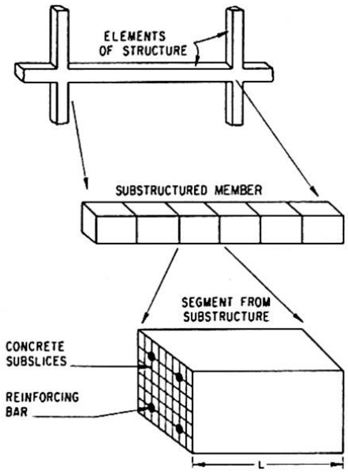

FIG. 1 GEOMETRIC IDEALIZATION

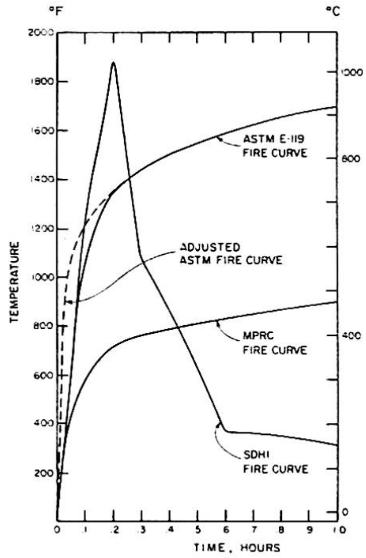

FIG. 5 FIRE CURVES

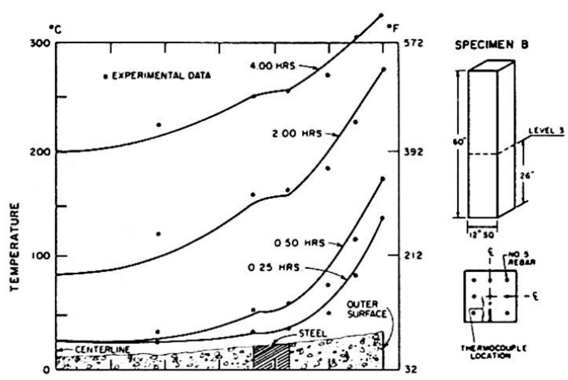

FIG. 2 COMPARISON OF EXPERIMENTAL DATA TO CALCULATED TEMPERATURES

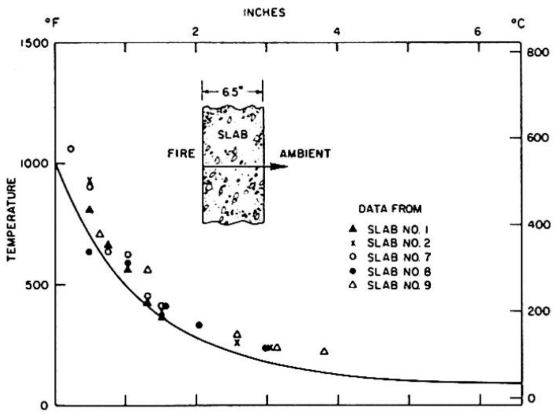

FIG. 6 COMPARISON OF EXPERIMENTAL DATA TO CALCULATED TEMPERATURES - 30 MINUTE EXPOSURE

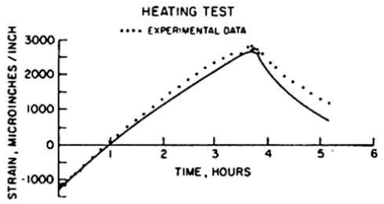

FIG. 3 COMPARISON OF EXPERIMENTAL DATA TO CALCULATED STRAINS

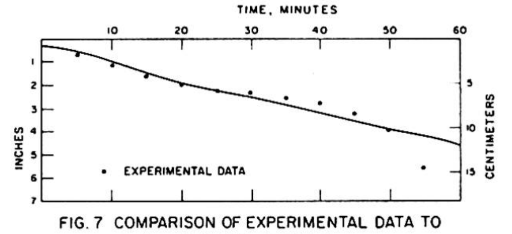

FIG. 7 COMPARISON OF EXPERIMENTAL DATA TO CALCULATED DEFLECTIONS

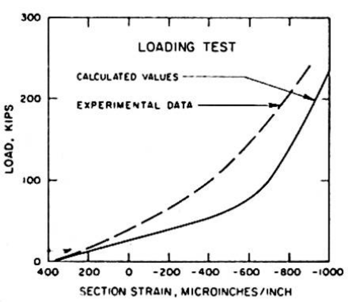

FIG. 4 COMPARISON OF EXPERIMENTAL DATA TO CALCULATED STRAINS

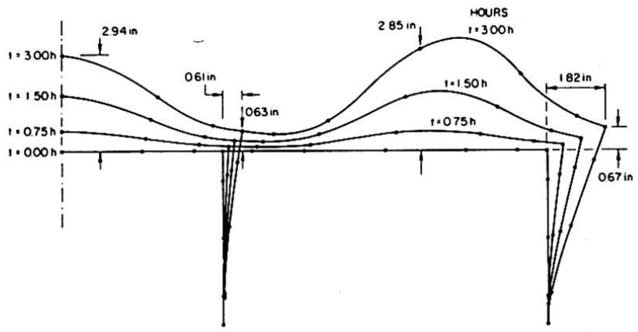

FIG. 8 FRAME DEFORMATIONS

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:

$$\sum \Delta P_e - P_i = 0 \quad \text{...(6)}$$

$$\sum \Delta M_e - M_i = 0 \quad \text{...(7)}$$

$$U_i = \sum (\Delta U_e) \quad \text{...(8)}$$

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:

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

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 :

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

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):

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

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

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

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):

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

where $E_S = 2.1 \times 10^6$ Kg/cm^2 :Young's modulus

f_T = 5000 - 5T'Kg/cm^2) :yield stress

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

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

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

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 considerablly. 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.

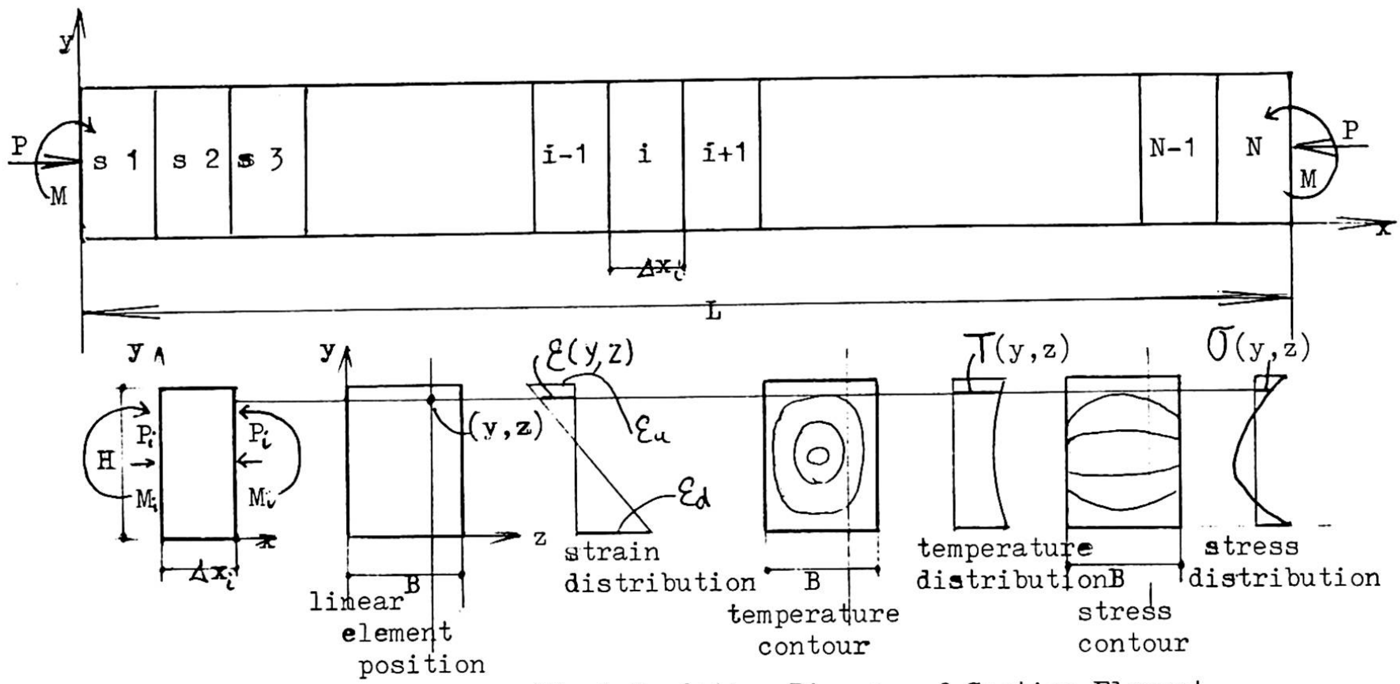

Fig.1-Condition Diagram of Section Element

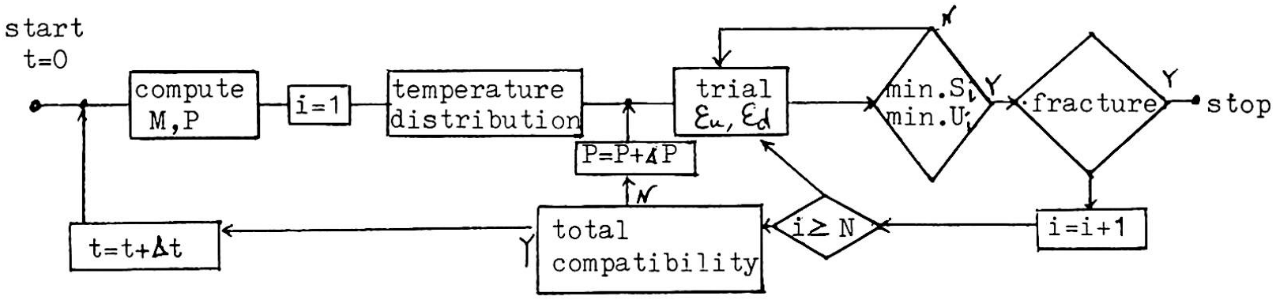

Fig.2-Flow Chart of Numerical Example

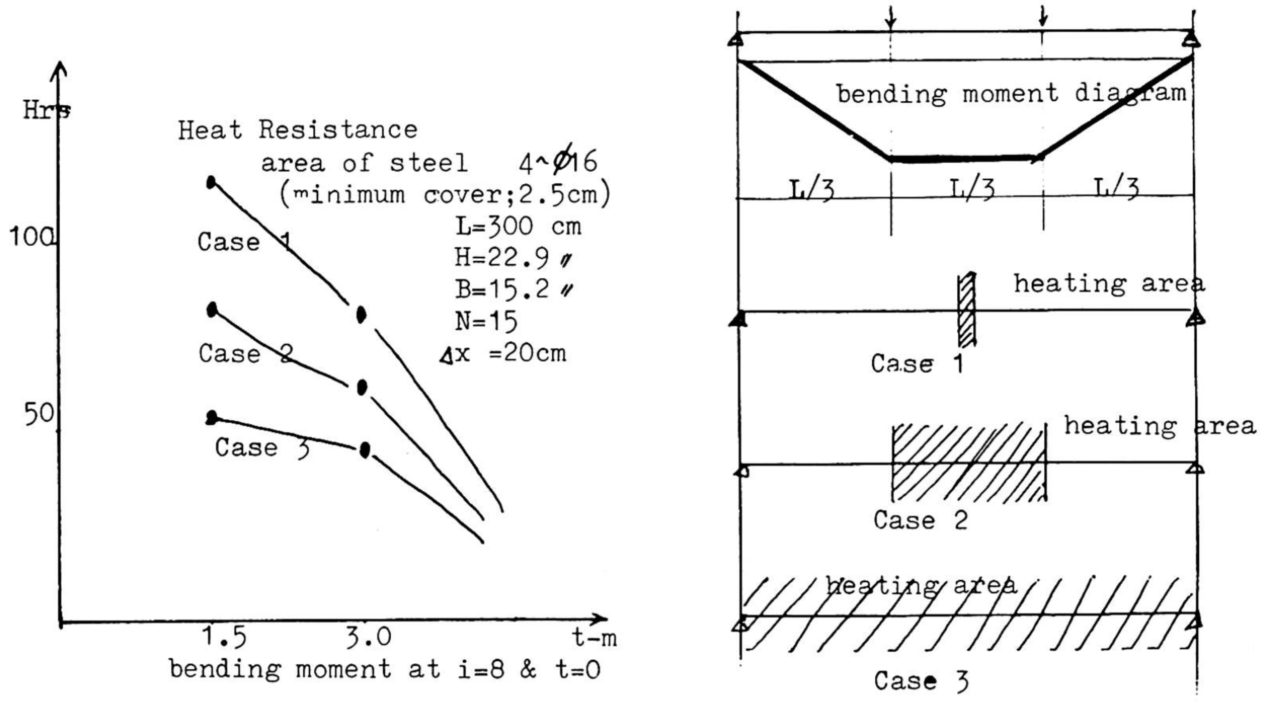

Fig.3-Case Study on Heat Resistance

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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.

IIIc

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

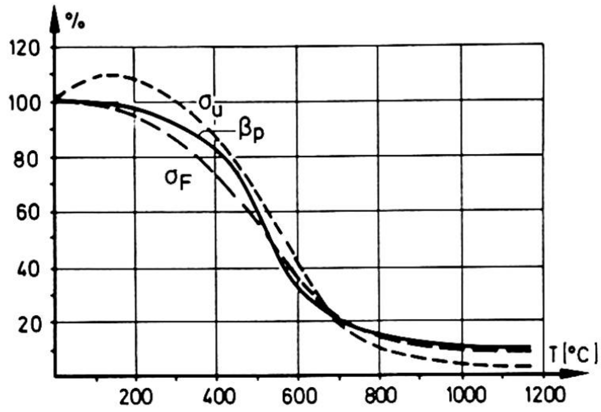


Abb.1: Veränderung der Festigkeitseigenschaften

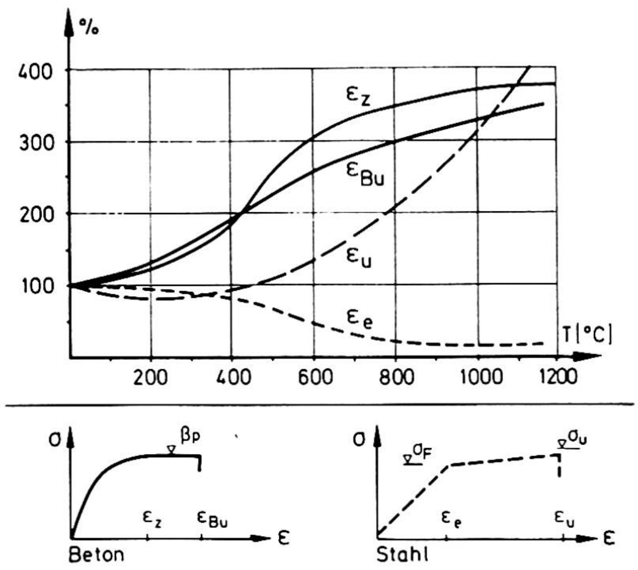


Abb.2: Veränderung der Verformungseigenschaften

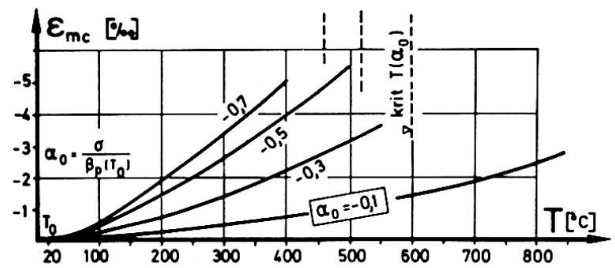


Abb.3: Kriech-bzw. Relaxationsanteile ε_{mc}

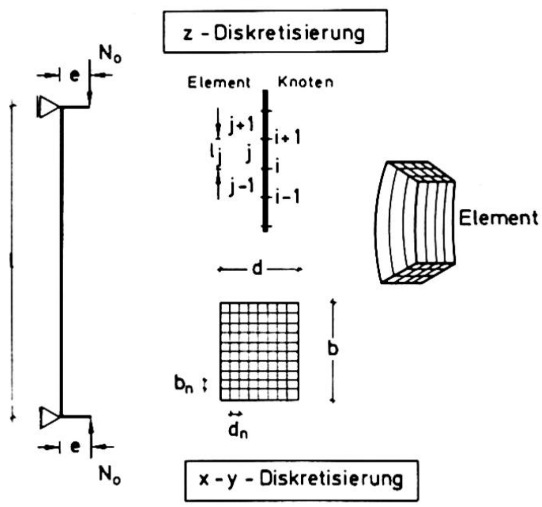


Abb.4: Diskretisierungsprinzip

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

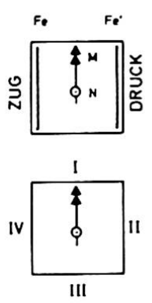
2.1 Stabförmige Bauteile

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



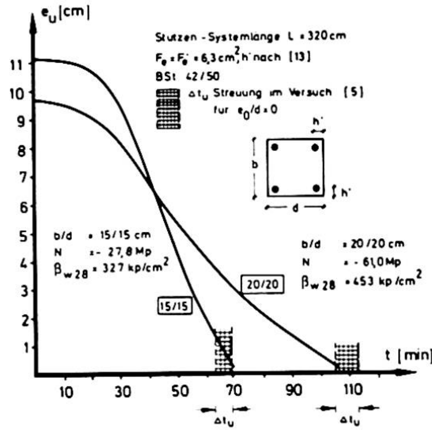


Abb.6: Versuchsnachrechnung von Stahlbetonstützen

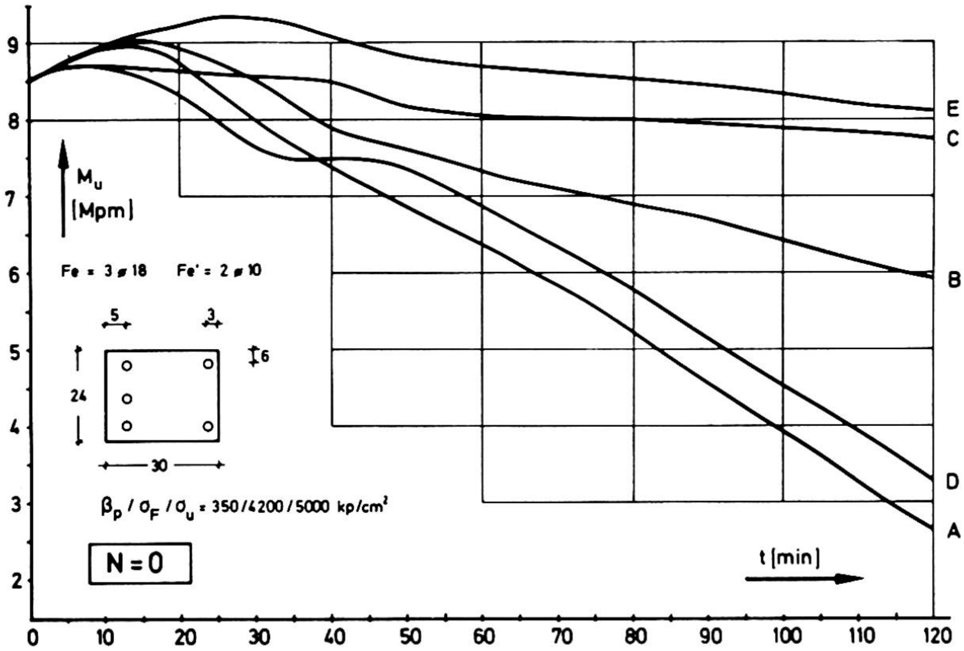


Abb. 5: Bruchmomentenverläufe eines Stahlbetonbalkens bei unterschiedlichen thermischen Randbedingungen

Brandes bleiben, ist für die Praxis ein $M_u(t)$-Diagramm besser geeignet. Entwickelt man diese Beziehung für $N_o \cong 0$, 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.

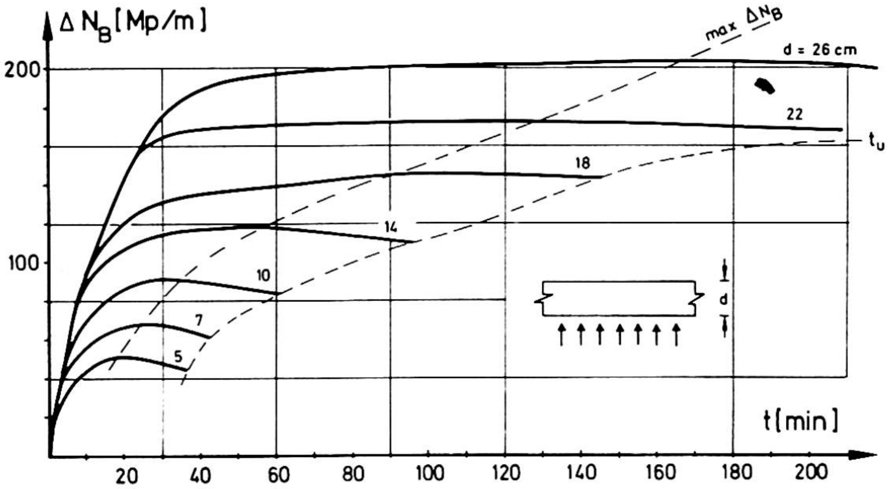


Abb.7:
Zwangskraftentwicklung in Stahlbetonplatten und -scheiben bei einseitiger lokaler Brandbeanspruchung

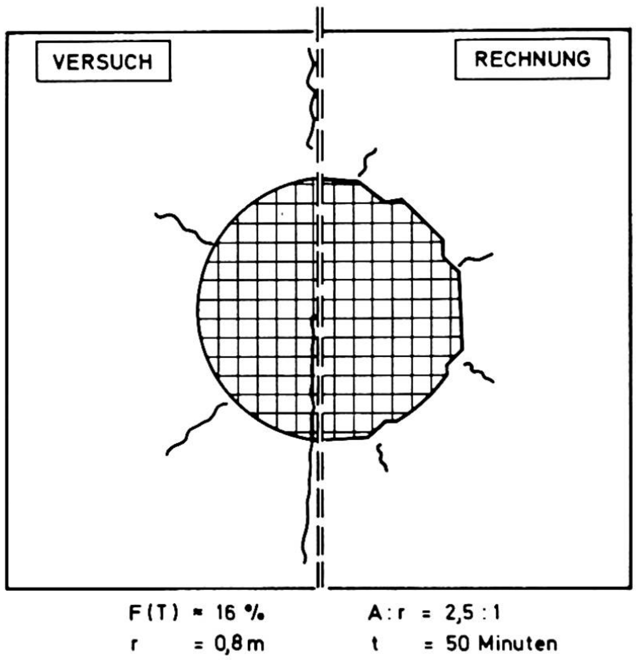


Abb.9:
Zwängungsrisse Vergleich zwischen Experiment und Rechnung

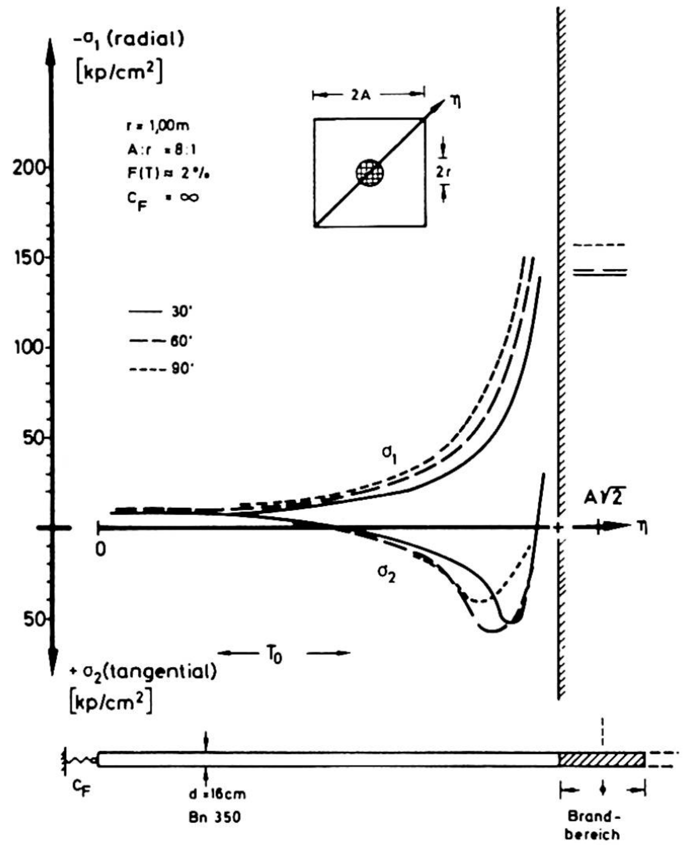


Abb.8:
Axiale Zwangsspannungen; Entwicklung und Verteilung bei lokaler Brandbeanspruchung

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

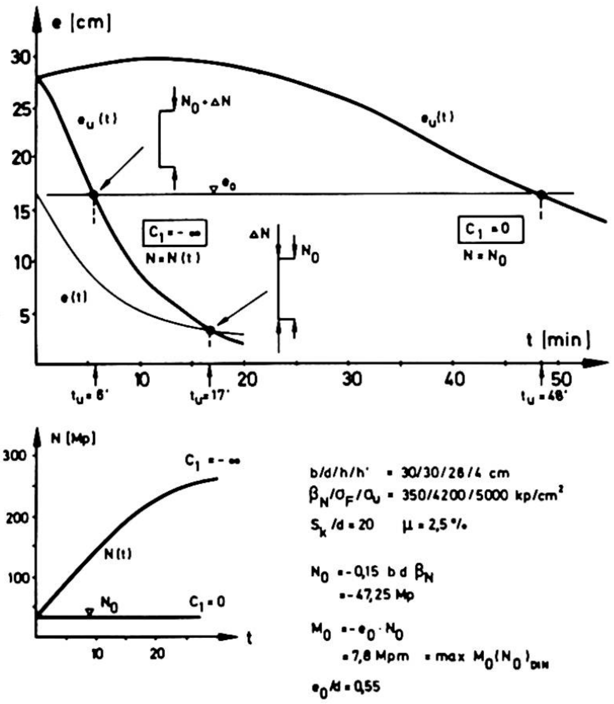
Interaktionen mit den umgebenden kalten Gebäudeteilen resultieren primär aus einer Verformungsbehinderung; sowohl Verdrehungs-, Durchbiegungs- als auch Dehnungsbehinderungen sind je nach System zu erwarten. Unter diesem Aspekt wurden u.a. das Verhalten von Durchlaufträgern, Rahmen und Stützen-Decken-Systemen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

4. Literatur

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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.

IIIc

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.

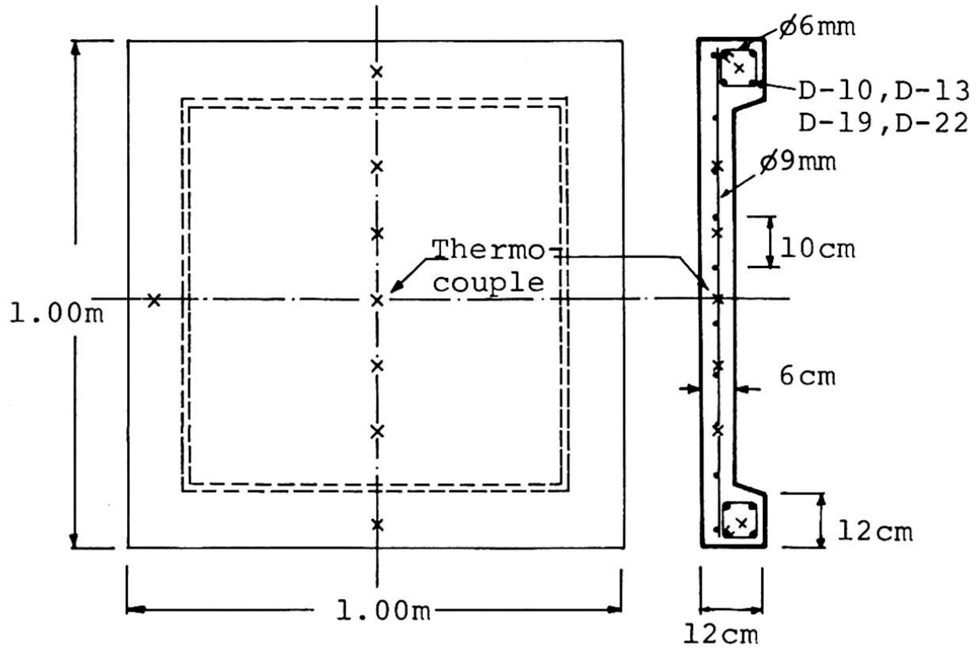


Fig. 1 Test panel

Fig. 2 Furnace for fire test

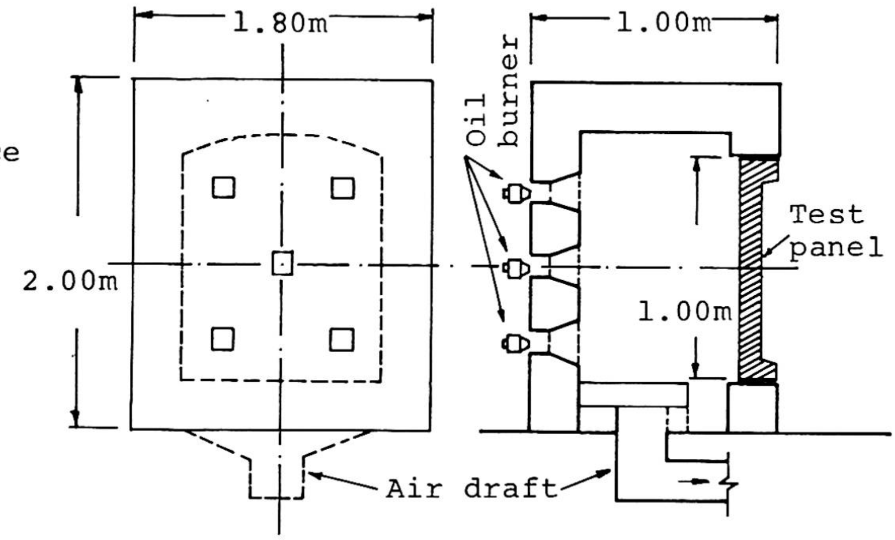
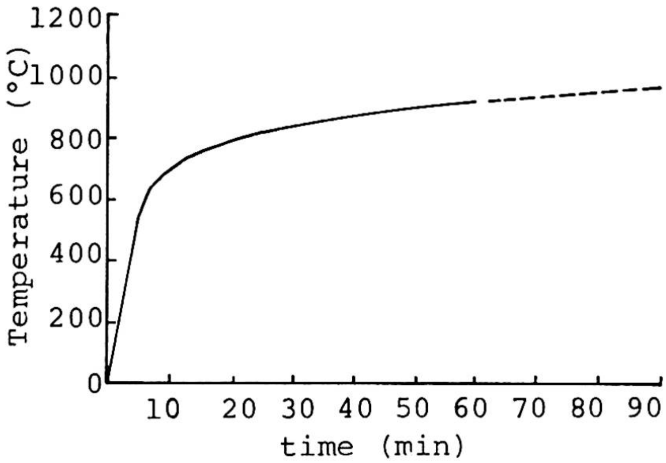


Fig. 3 Standard time/temperature curve (JIS A 1304)



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	Amount of Free Water 60		80		100		120		140		160		180
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

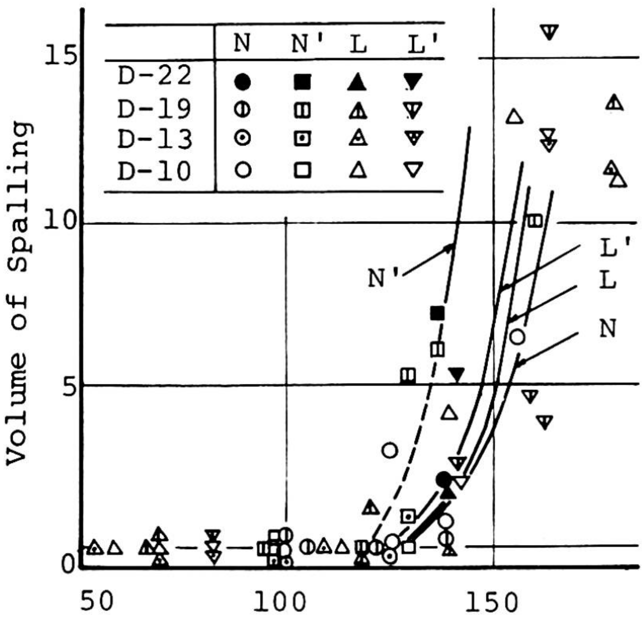


Fig. 4 Volume of spalling related to amount of free water for each type of concrete.

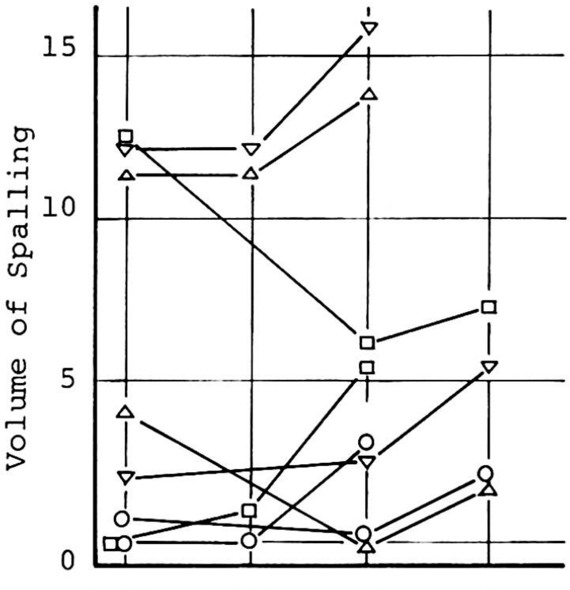


Fig. 5 Volume of spalling related to condition of restraint.

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.