Fire-exposed Hyperstatic Concrete Structures

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YNGVE ANDERBERG

FIRE - EXPOSED HYPERSTATIC CONCRETE STRUCTURES

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Fire-exposed hyperstatic concrete structures

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National Swedish Building Research

Brandpåverkade statiskt obestämda betongkonstruktioner

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Byggforskningen Sammanfattningar

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Uppsatsen, som är skriven på engelska, ger en första delredovisning av ett grundläggande studium av beteende och verkningssätt för den mot rotation tvåsidigt fast inspända och mot längsförskjutning helt fria armerade betongplattstrimlan vid ensidig brandpåverkan. En väsentlig utvidgning jämfört med hittills publicerade försök är att studier genomförts med differentierade brandförlopp samt med en uppföljning av hela brandförloppet inklusive avsvalningsfasen.

Tidigare publicerade undersökningar av, för brand utsatta, armerade betongkonstruktioner har mestadels varit begränsade till bärverk av statiskt bestämd typ. Karakteristiskt för undersökningarna har genomgående varit en odifferentierad brandpåverkan med det internationellt standardiserade brandförloppet. Undersökningarna har dessutom begränsats till en renodlad uppvärmning med ett utelämnade av den på en uppvärmningsfas följande avsvalningen. Vidare har målsättningen oftast varit att bestämma konstruktionernas brandmotståndstid. Emellertid har man på senare år också börjat intressera sig för ett mera ingående studium av det konstruktiva verkningssättet och bärförmågan hos brandpåverkade konstruktioner och därvid även för statiskt obestämda, armerade betongbärverk.

För en mera ingående analys av det komplexa verkningssättet hos brandpåverkade, statiskt obestämda, betongkonstruktioner krävs en serie av så långt möjligt renodlade studier av alla väsentliga influenser. Den redovisade undersökningen kan ses som ett bidrag till ett sådant mera systematiskt studium, som sedan något år pågår vid Institutionen för byggnadsstatik, LTH. Undersökningen omfattar ett studium av en i sammanhanget extremt renodlad konstruktion, nämligen den tvåsidigt mot rotation fast inspända och mot längsförskjutning helt fria, armerade betongplattstrimlan under ensidig brandpåverkan.

Inspänningen mot rotation är provningstekniskt realiserad genom kontrollerbara yttre punktböjmoment, så att inspänningsmomentets successiva förändring på ett mätbart sätt kan följas under hela brandförloppet. En väsentlig utvidgning jämfört med hittills publicerade försök är de genomförda studierna av differentierade brandförlopps inverkan på beteende och bärförmåga samt en uppföljning av hela brandförloppet inklusive avsvalningsfasen.

Väsentliga parametrar i en undersökning av skisserad art utgör brandförloppskarakteristika, plattjocklek, betongsammansättning, armeringskvalitet, armeringsprocent, täckskiktstjocklek och provobjektets ålder. Avgörande för brandförloppskarakteristika är därvid faktorer som termiska egenskaper för omgivande konstruktion, brandbelastningen q, brandrummets storlek, geometri och ventilationsegenskaper karakteriserade genom en öppningsfaktor $A\sqrt{H}/A_t$. A betecknar sammanlagd öppningsyta, H ett vägt medelvärde av öppningshöjd och A_t brandrummets totala omslutningsyta, inklusive öppningar.

Försöksuppläggning

Den redovisade undersökningen har genomförts för obelastade respektive med två symmetriska punktlaster belastade, mot vinkeländring tvåsidigt fast inspända, betongplattstrimlor (FIG. 2). FIG. 1 och 3 visar försöksutrustningens principiella utformning och funktion. Här ses bland annat den för undersökningen använda gasoluppvärmda ugnen samt inspänningsanordningen över respektive stöd. Med den använda försöksutrust-

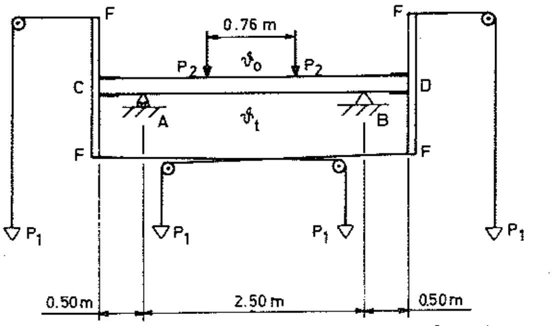

FIG. 1 Försöksutrustningens principiella utformning och funktion.

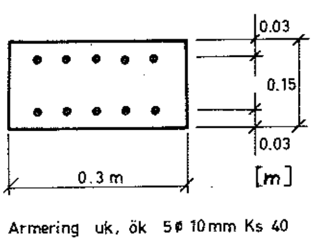

FIG. 2 Tvärsnitt av plattstrimla.

Nyckelord:

betongplattstrimla (armerad, statiskt obestämd, brandpåverkad), bärförmåga, böjmomentupptagande förmåga, resthållfasthet

brandpåverkan, brandförloppskaraktеristika, brandbelastning, öppningsfaktor, brandvaraktighet

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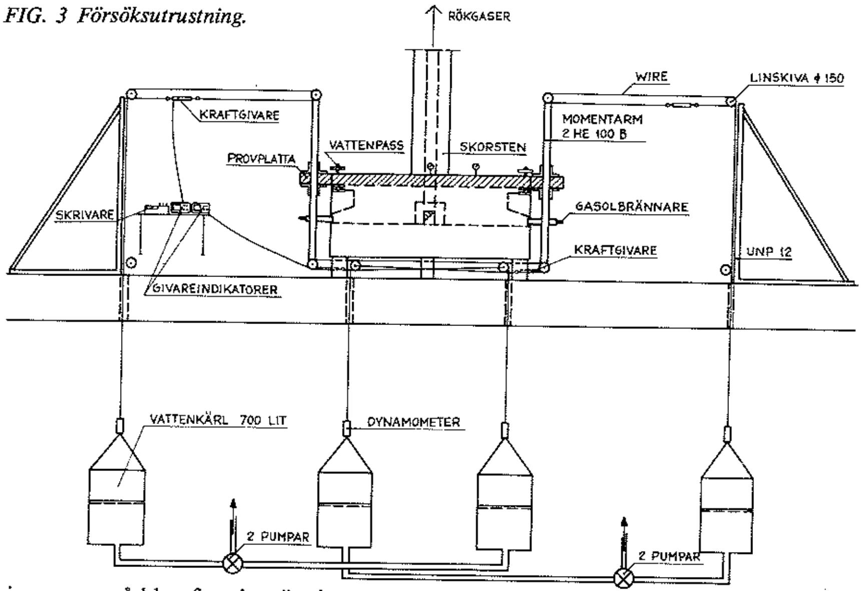
FIG. 3 Försöksutrustning.



ningen uppnåddes fast inspänning mot rotation utan något hinder mot axiell längdändring. För kontroll av oförändrad fast inspänning mot rotation av plattstrimlorna användes två vattenpass över vartdera upplaget. Krafterna P_1 påfördes genom vattenpåfyllning av anslutna vattenkärl samt reglerades med fyra manöverkranar, en för varje vattenkärl. Vattenkärlen kunde vid behov dessutom tömmas med hjälp av fyra pumpar.

Servogor-skrivare registrerade kontinuerligt krafterna P_1, med hjälp av givarindikatorer som var kopplade till elektriska lastceller inmonterade i wirarna.

För kontinuerlig bestämning och registrering av temperaturen i ugnen samt temperatur-tid-fältet i plattstrimlan användes termoelement som var anslutna till en 24-kanalsskrivare.

Deformationsmätningarna genomfördes manuellt med hjälp av fyra indikatorklockor, två st monterade i mittsnittet och en i vartdera fjärdedelssnittet.

De parametrar som i första hand har varierats i försöken är brandförloppskarakteristika (brandvaraktighet, brandbelastning och öppningsfaktor), provobjektets ålder och betongsammansättning (karakteriserad av vattencementtal och cementlimsmängd). Försöksprogrammet omfattar 5 försöksserier med följande karakteristika.

Försöksserie A1 och A2

Obelastade plattstrimlor (20 försök)

Brandförloppet varierades med koppling till varierande öppningsfaktor $A\sqrt{H}/A_t$ inom området 0,01–0.08 m $^{1/2}$ och brandbelastningen $q = 7.5 - 480$ Mcal/m² omslutningsyta (31–2010 MJ/m²). (1MJ = 10^6J).

Försöksserie B och C:

Obelastade plattstrimlor (18 försök)

Betongsammansättningen karakteriserad av vatten-cementtalet respektive cementlimsmängden samt provobjektets ålder varierades.

$A\sqrt{H}/A_t = 0.04$ m$^{1/2}$

$q = 120$ Mcal/m² (502 MJ/m²)

Vatten-cementtal 0.55, 0.63 och 0.77

Cementlimsmängd 257, 277 och 296 l/m³

Försöksserie D:

Belastade plattstrimlor (12 försök)

Belastningsnivåer och brandförlopp varierades med koppling till varierande öppningsfaktor. Fyra belastningsnivåer provades (se FIG. 5).

$A\sqrt{H}/A_t = 0.01-0.08$ m$^{1/2}$

$q = 60$ Mcal/m² (251 MJ/m²)

Resultat

I undersökningen redovisas för plattstrimlan med utformning enligt FIG. 1 och 2 sammanfattande och exemplifierande resultat för temperatur-tid-fält i ugn och plattstrimla, tvångsböjmomenttillstånd, nedböjningsförlopp, bärförmåga under brandförloppet samt restbärförmåga efter avsvalning till rumstemperatur.

En exemplifiering av väsentliga resultat visar FIG. 4 och 5. FIG. 4 belyser inspänningsböjmomentets tidsvariation för obelastade plattstrimlor från försöksserie A2 vid olika brandförlopp, karakteriserade av konstant brandbelastning och varierande öppningsfaktor. Tillhörande tidkurvor för ugnstemperaturen finns också inritade. Inspänningsböjmomentets tidsvariation för plattstrimlor belastade vid 5 olika nivåer med tillhörande temperatur-tid-kurva visas i FIG. 5. Flytböjmomentets uppnående för olika belastningar noteras. Maximala mittnedböjningar för transversellt obelastade plattstrimlor har varierat från 2 till 5 mm, medan motsvarande nedböjningar för belastade plattstrimlor varit väsentligt större (upp till 15 mm). För obelastade plattstrimlor karakteriseras restdeformationstillståndet efter avsvalning av en uppböjning med tillhörande positiva restböjmoment. I figurerna är även resttillståndet angivet.

Nedsättning i bärförmåga och restböjstyvhet efter avsvalning för de med positivt moment belastade plattstrimlorna blev 10–15 % respektive 0 %. Vid belastning med negativt moment erhölls 20–25 % respektive 50–70 % reduktion. Bärförmågan under brandförloppet blev inte i något försök uttömd.

Vid en analys av bärförmågan under brandförloppet framgår att en plattstrimla teoretiskt kan uppnå brottstadiet även under avsvalningsfasen. Detta illustreras i uppsatsen, där den principiella variationen av aktuellt stöd- och fältmoment samt kapacitetsmoment (böjmomentupptagande förmåga) vid ett tänkt brandförlopp har analyserats.

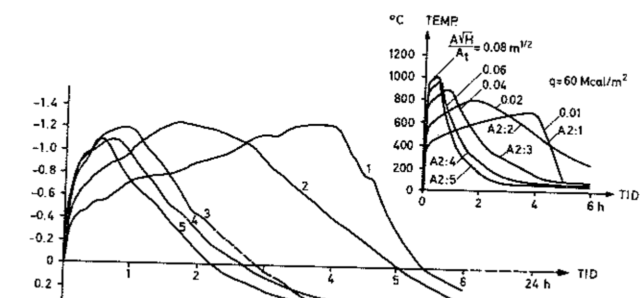


FIG. 4 Inverkan av brandförloppskarakteristika på inspänningsböjmomentet för obelastade plattstrimlor ur försöksserie A2, som karakteriseras av konstant brandbelastning q = 60 Mcal/m² (251 MJ/m²) omslutningsyta och varierande öppningsfaktor 0.01–0.08 m$^{1/2}$. Tillhörande temperatur-tidkurvor visas också.

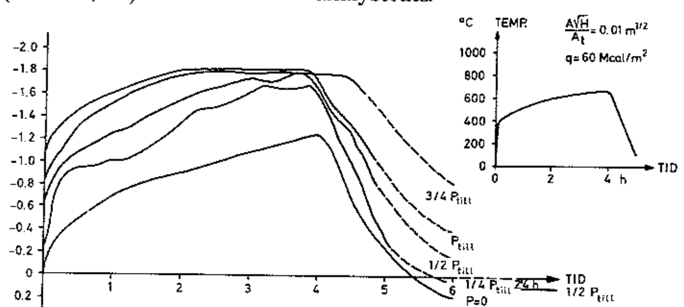


FIG. 5 Inspänningsböjmomentets tidsvariation för belastade plattstrimlor vid olika belastningsnivåer (försöksserie D). Temperatur-tidkurvan visas också. P_{till} = tillåten last enligt Svenska Betongbestämmelserna.

Fire-exposed hyperstatic concrete structures

Yngve Anderberg

National Swedish Building Research Summaries

D1:1973

This paper presents a first and partial account of the fundamental studies regarding the structural behaviour of a reinforced concrete strip exposed to fire on one side and completely fixed against rotation at both ends while the longitudinal movement is allowed to take place freely. In the present investigation substantial progress compared with the existing published experiments may be noted partly by a study of the subject based on a differentiated fire process and partly by considering the whole fire process including the cooling phase.

Published investigations of fire-exposed, reinforced concrete structures have, to a great extent, been limited to statically determinate structures. The existing investigations have predominantly been characterized by treating the subject in an undifferentiated way through an internationally standardized fire process. The experiments have further been limited to pure heating ignoring the effect of cooling which follows a heating phase. Furthermore, the objectives of the investigations have as a rule been limited to determining the duration of fire resistance. However, in recent years there has been a growing interest in more thorough study of the structural behaviour and load-bearing capacity of fire-exposed structures and consequently also in statically undeterminate reinforced concrete structure.

For a more detailed analysis of the complex structural behaviour of statically indeterminate concrete structures subject to fire it is required to investigate the subject on the basis of a series of fundamental studies which take all the significant factors into account. The investigation described in the present paper is intended as a contribution towards such a systematic study, which has been going on for some years at the Division of Structural Mechanics and Concrete Constructions, Lund Institute of Technology. In the present case the experiment comprises a study of an extremely pure type of structure concerned with the subject, namely a reinforced concrete strip exposed to fire on one side and completely fixed against rotation at both ends while the longitudinal movement is allowed to take place freely. Technically the end rotation is prevented through two controllable external concentrated bending moments so that the change in bending, restraint moments can be followed quantitatively during the whole fire process. In the investigation substantial progress compared with the existing published experiments may be noted partly by a study of the subject based on a differentiated fire process and partly by considering the whole fire process including the cooling phase.

Significant parameters studied in an investigation of this kind consist of characteristics of the fire process, plate thickness, concrete composition, reinforcement quality, proportion of reinforcement, thickness in concrete cover and age of the test specimen. The decisive factors governing the characteristics of fire process are for instance such factors as the size of fire compartment, shape and ventilation properties characterized by an opening factor given by the formula $A\sqrt{H}/A_t$, thermal properties of the surrounding constructions and the fire load q.

Test procedure and equipment

The experiment described is performed both for nil-loaded and loaded reinforced concrete strips. The vertical loading consists of two concentrated loads and the strips are fixed against rotation at both ends (FIG. 2). FIGS. 1 and 3 illustrate the principal form and function of the testing facilities. Here, among other things, the gasoil-heated furnace used in the experiment and the arrangement for end restraint at each support are seen. The end restraint against rotation without preventing the axial change of length was thus attained with the illustrated testing facilities. In order to

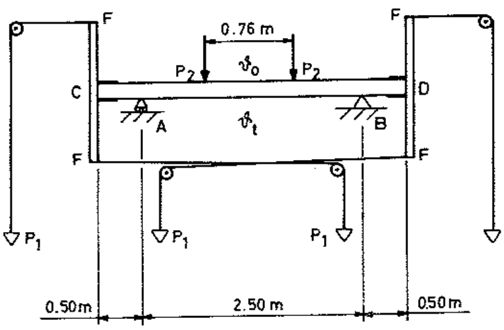


FIG. 1 Principal form of test arrangement.

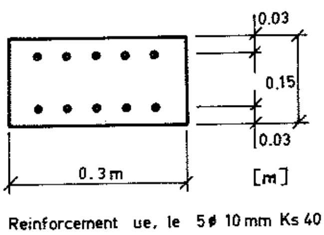


FIG. 2 Fire test specimen.

Key words:

concrete plate strip (reinforced, statically undeterminate, fire-exposed), load-bearing capacity, bending moment capacity, residual strength

fire exposure, characteristics of fire process, fire load, opening factor, fire duration

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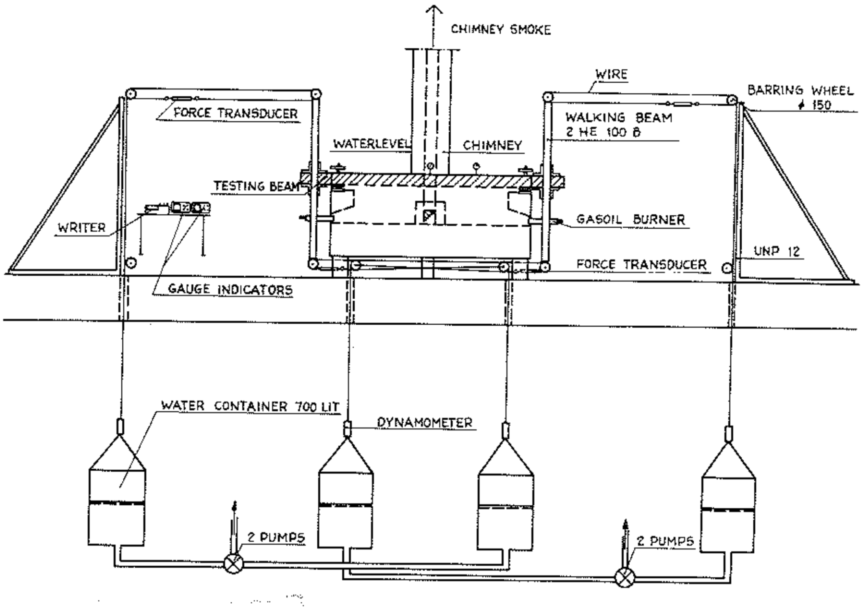

assure that no end rotation took place in the plate strips, two water levels were placed on each support. The forces P_1 were applied by filling the water containers with water through connected tubes. The filling could be adjusted by four taps located nearly the furnace, one for each water container. Emptying water tanks is achieved by four pumps with switches located beside the taps.

Servogor printers recorded continuously the forces P_1 through gauge-indicators, which were connected to electrical loading cells, attached to the cables.

In order to continuously determine and register the temperature in the furnace and the temperature-time field of the strips, thermocouples were used. These were connected to a 24-channelled printer.

The deformation measurements were performed manually with four gauge-indicators, two of them mounted at mid-section and 1/4-section respectively.

The parameters which have primarily been varied during the experiment are the characteristics of fire process (fire duration, fire load and opening factor), the age of the test specimen and concrete composition (characterized by water-cement-ratio and cement-paste quantity). The test program embraces 5 test series with following characteristics.

Test series A1 and A2:
Nil-loaded plate strips (20 tests)

The fire process is varied by changing the opening factor $A\sqrt{H}/A_t$ within the range 0.01–0.08 $m^{1/2}$ and fire load $q = 7.5 - 480$ Mcal/m² enclosing area.

Test series B and C:
Nil-loaded plate strips (18 tests)

Concrete composition characterized by water-cement-ratio and cement-paste quantity respectively and age of the specimen is varied.
$A\sqrt{H}/A_t = 0.04$ $m^{1/2}$.
$q = 120$ Mcal/m² enclosing area.
Water-cement-ratio 0.55, 0.63 and 0.77.
Cement-paste quantity 257, 277 and 296 l/m³.

Test series D:
Loaded plate strips (12 tests)

Vertical loading levels and fire process are varied by changing the opening factor. The loading levels 1/4, 1/2, 3/4 and 1/1 of P_{all} are used where P_{all} is the allowable load according to Swedish Concrete Standards. $q = 60$ Mcal/m² enclosing area.
$A\sqrt{H}/A_t = 0.01–0.08$ $m^{1/2}$.

Results

In the investigation the strips are presented with a detailed construction as in FIGS. 1 and 2 in which the results of the temperature-time field in the gasoil-heated furnace and the strips, restrained bending moment, deflection process, load-bearing capacity during the fire process and the residual bending stiffness and residual strength after cooling down to room-temperature are summarized and illustrated.

FIGS. 4 and 5 illustrate the most essential results of the tests. The time-variation of the restrained bending moment of nil-loaded plate strips from test series A2 at different fire processes characterized by a constant fire load and varying opening factor are illustrated in FIG. 4. Corresponding time-curves of furnace temperature are also drawn. The time-variation of the bending, restraint moment of strips loaded at 5 different levels together with the corresponding time-curves of the furnace temperature are shown in FIG. 5. The value of the bending moment at yield stage is recorded at different loading levels.

The maximum deflections at the mid-section of the vertically nil-loaded plate strips have varied from 2 to 5 mm, whereas the corresponding deflections for vertically loaded strips have been considerably larger. After cooling, the residual deflection of the nil-loaded strips is characterized by an upward deflection with the corresponding positive residual bending moments. In the FIGS. the residual states are also shown. Reduction in bearing capacity and residual bending stiffness after cooling for strips loaded with positive moments amounted to 10–15% and 0% respectively. The reduction for strips loaded with negative moments was 20–25% and 50–70% respectively. The load-bearing capacity during fire exposure was not exhausted in any of the tests.

By analysis of the load-bearing capacity during the fire process it is observed that the plate strip cannot theoretically reach the fracture stage until during the cooling phase. This fact is illustrated in this paper, giving the basic variation of the actual moment at the support and in the span and the capacity moment (bending moment capacity) during an imaginary fire process.

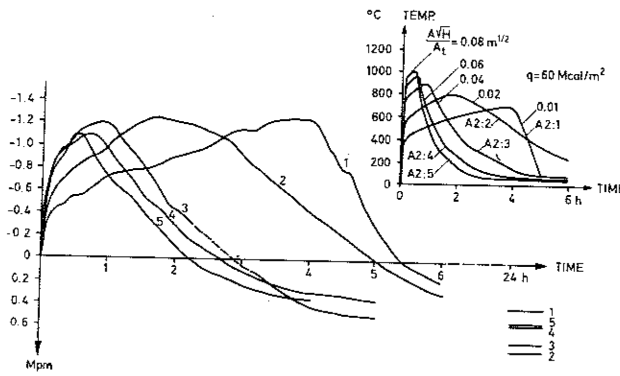

FIG. 4 The influence of fire process characteristics on bending, restraint moment shown by nil-loaded plate-strips in test-series A2, characterized by a constant fire load $q = 60$ Mcal/m² (251 MJ/m²) enclosing area with varying opening factors within the range 0.01 – 0.08 $m^{1/2}$. The corresponding temperature-time curves are also shown.

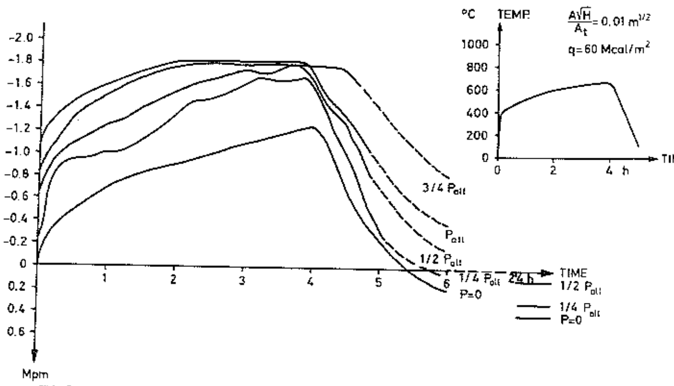

FIG. 5 Variation in bending, restraint moment with time of transversally loaded plate-strips at varying loading levels (test series D). The temperature-time curve is also shown. P_{all} = the allowable load according to Swedish Concrete Standards.

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FIRE-EXPOSED HYPERSTATIC CONCRETE STRUCTURES

A fundamental study of the structural behaviour and loadbearing capacity of statically undeterminate reinforced concrete plate strips exposed on one side to fires.

by Yngve Anderberg

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CONTENTS

NOTATION

Letters

A	Total opening area of a fire conpartment	m^2
A_t	Total bounding area of a fire conpartment including the openings	m^2
H	A weighted mean value of the vertical dimensions of the openings in a fire conpartment	m
$\frac{A\sqrt{H}}{A_t}$	Opening factor $m^{1/2}$	
P	Concentrated load Mp	
$\bar{P}$	Cantilever load Mp	
P_1	Concentrated load Mp	
$\bar{P}_1$	Concentrated load Mp	
P_2	Concentrated load Mp	
$\bar{P}_2$	Concentrated load Mp	
P_{all}	Allowable load according to Swedish Concrete Standards	Mp
P_{ult}	Theoretical ultimate load $2\ P_{all}$ Mp	
q	Fire load $Mcal/m^2$ bounding area	

Greek letters

α	Surface coefficient of heat transfer	$kcal/m^2 \cdot h\,^oC$
γ	Weight per unit volume	kg/m^3
c_p	Specific heat capacity	$kcal/kg \cdot {}^oC$
γc_p	Thermal capacity	$kcal/m^{3\,o}C$
λ	Thermal conductivity	$kcal/m \cdot h \cdot {}^oC$
σ_s	Yield strength	kp/cm^2
σ_B	Ultimate strength	kp/cm^2
ϕ	Diameter of reinforcement	mm

Transformation factors

1 Mp	$= 9.8 \cdot 10^3$ N
1 kp/cm^2	$= 98 \cdot 10^3\ N/m^2$
1 $Mcal/m^2$	$= 4.2 \cdot 10^6\ J/m^2$
1 $kcal/m^2 \cdot h \cdot {}^oC$	$= 1.16\ W/m^2 \cdot {}^oC$
1 $kcal/kg \cdot {}^oC$	$= 4.2 \cdot 10^6 J/kg \cdot {}^oC$

1. THE PROBLEM AND ITS BACKGROUND

The existing published literature on fire-exposed reinforced concrete structures has mpstly been limited to the case of statically determinate structures. The existing investigations have predominately been characterized by treating the subject in an undifferentiated way through an internationally standardized fire process. Furthermore the objective of the investigations have as a rule been limited to determining the duration of fire resistance. However, during the recent years there has been a growing interest in a closer study of the structural behaviour and load-bearing capacity of fire-exposed structures, including statically indeterminate reinforced concrete constructions.

The statically indeterminate structures subject to fire have the characteristic property that the deformations caused by heating normally cannot develop freely and thereby give rise to restraint forces and moments which are usually of a complicated nature. These forces and moments depend among other things on the flexural stiffness of the structure and for reinforced concrete constructions they change continuously through a gradually growing crack formation in the tensile zone of concrete and through a successively decreasing in module of elasticity of the heated concrete and reinforcement. Changes in strength and microstructure together with more pronounced creep and shrinkage phenomena at elevated temperatures make the behaviour of the structure much more complicated.

As mentioned in the beginning, the published investigations have all been limited to an internationally standardized fire process with a uniquely fixed temperature-time curve. With respect to the characteristics of fire process, the investigations have been further limited to a pure heating ignoring the effect of cooling which follows a heating phase. This is true also for investigations whose purpose has been a more thorough study of the behaviour of constructions subject to fire.

Among the experiments with the above descriptions the works by Selvaggio et al. (1963 and 1966), C.C. Carlsson et al. (1965) and Gustaferro (1970) at Portland Cement Association (PCA) are worth mentioning. They studied the behaviour of prestressed and reinforced concrete slabs and plates exposed to fire by measuring the fixed end actions. By using results from experiments, Issen, Gustaferro & Carlsson (1970) have worked out a method for calculating axial restraint forces for a construction subject to fire at a given allowable longitudinal expansion with a maximum error of about 15%. The method is applicable to beams and plates guided uni- or biaxially and thus rendering the axial restraint forces.

Experimental studies, including those at PCA have been performed to determine the structural behaviour of continuous reinforced concrete beams without longitudinal restraint. The experiment is illustrated in Fig. 1 which fragmentary shows the behaviour of a symmetrically designed and loaded beam in three spans subject to fire underneath the mid-span. In the experiment, the external supports were simulated through concentrated loads P_1 and P_2 applied to end cantilevers. During the fire process $\bar{P}_1$ and $\bar{P}_2$ were continuously changed in such a way that the vertical displacement of cantilever ends was prevented (FIG. 1a). The technique facilitates determination of the change in bending moment distribution during the experiment. Moment distribution and bending moment capacity of the beam before start of the experiment can be observed in FIG. 1b. FIG. 1c illustrates how moment distribution and bending moment capacity change during the fire process. The duration of fire resistance for the above mentioned experiment turns out to be 3.5 hours and only 1.5 hours for the corresponding simply supported beam with an equivalent design load. A study of the structural behaviour of reinforced concrete beams subject to fire on three sides, simply supported on one end and fixed on the other, is described by Ehm & von Postel (1965). The test arrangement used in the experiment is illustrated in FIG. 2 which shows that the problem of the end fixing at the right support of the beam was technically solved by a stiff cantilever with an external variable force $\bar{P}$ applied at its end and varied in such a way that angular rotation over the right support of the beams was prevented at any time during the fire exposure. The test equipment was formed in such a way that the axial change in length of the beam due to heating remained unaffected. The results of experiments on two reinforced concrete beams with different stiffnesses are illustrated in FIG. 3 which shows the required variation of the cantilever load $\bar{P}$ as a function of time corresponding to the forced thermal bending moment during a continuous heating according to standard fire procedure. It is observed from the figure that in the two experiments the maximum value of $\bar{P}$ is attained already after 20 minutes and that this value is then maintained for a further 80 minutes. This value of $\bar{P}$ corresponds to moment bearing capacity of the beam at the right support. FIG. 4 shows the bending moment distribution for Beam 1 calculated from the measured $\bar{P}$ values, corresponding to fire times 0, 20, 100 and 165 minutes according to standard fire exposure.

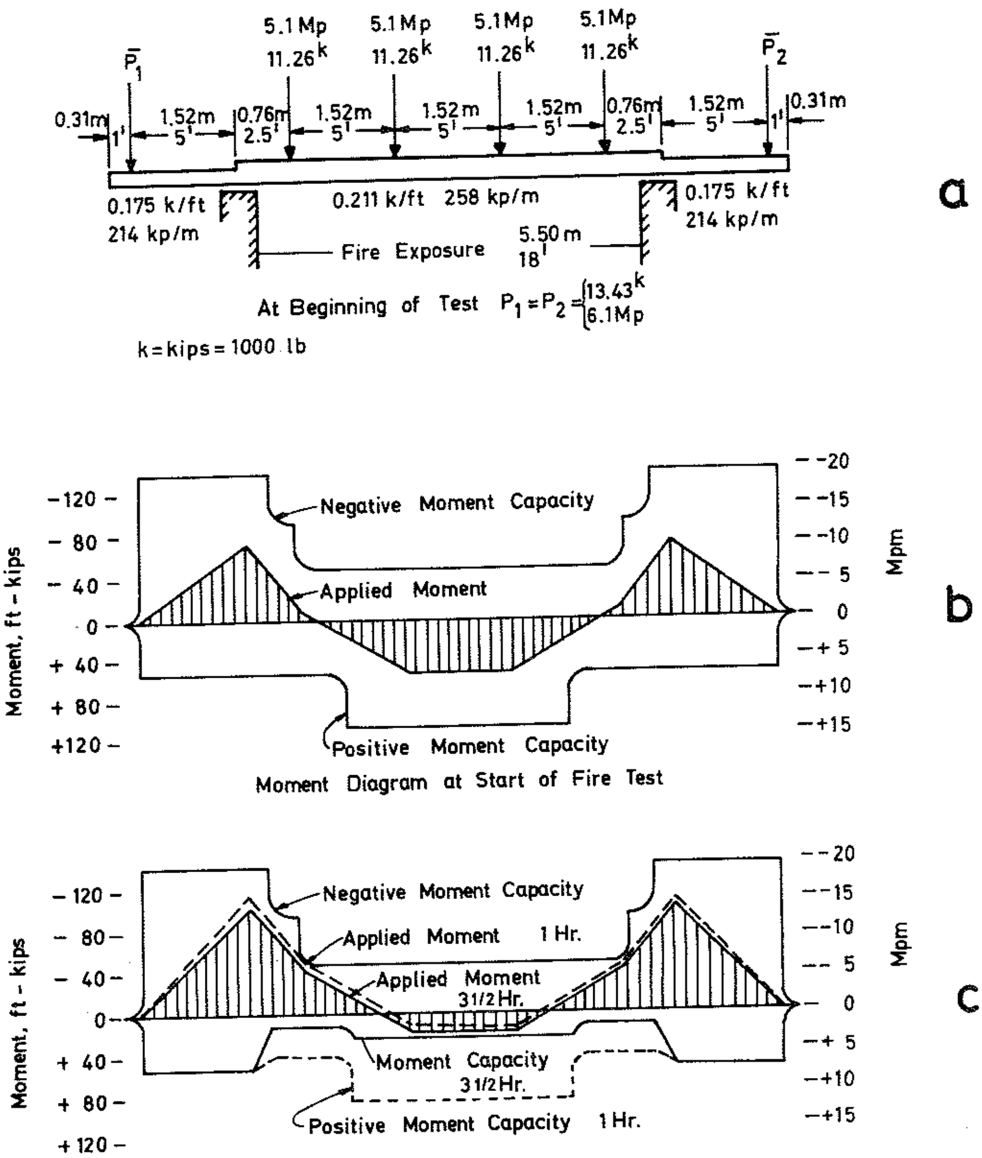


FIG. 1

a) Data on a transversally loaded reinforced concrete beam before fire test.

b) Bending moment distribution and bending moment capacity at start of fire test.

c) Bending moment distribution and bending moment capacity at 1 and 3 1/2 hours after start of fire test. From Gustaferro (1970).
k = kips = 1000 lb

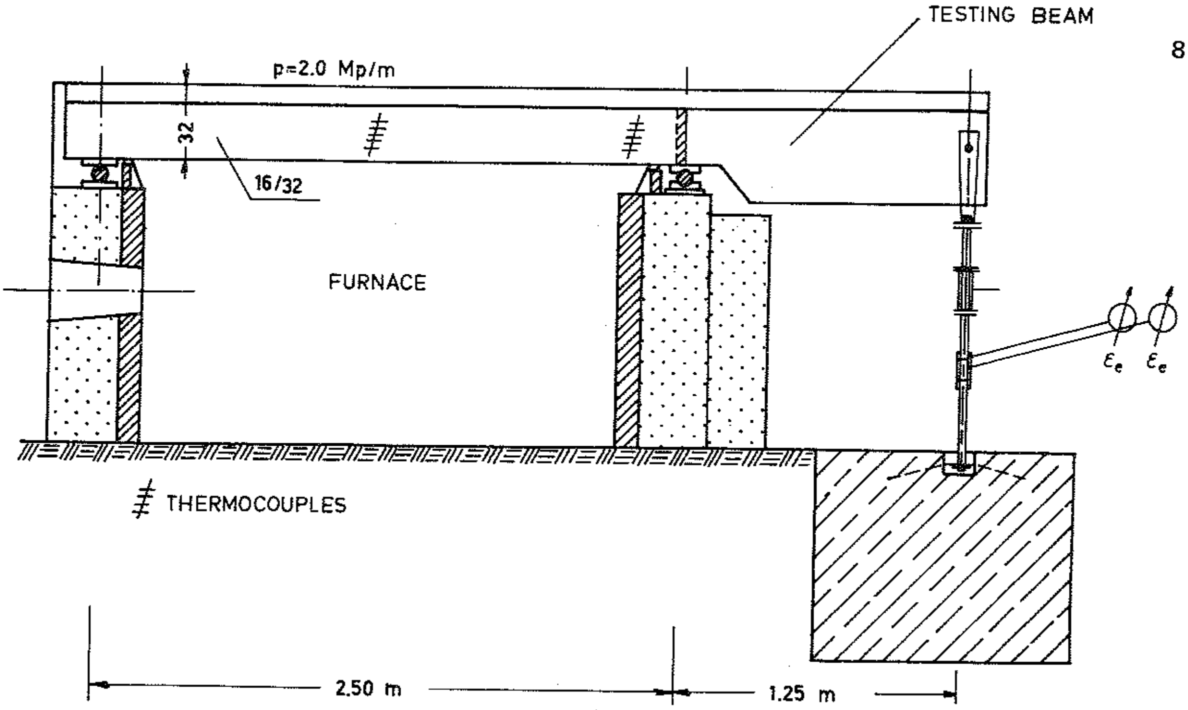


FIG. 2 Test arrangement for a fire test. From Ehm & von Postel (1965).

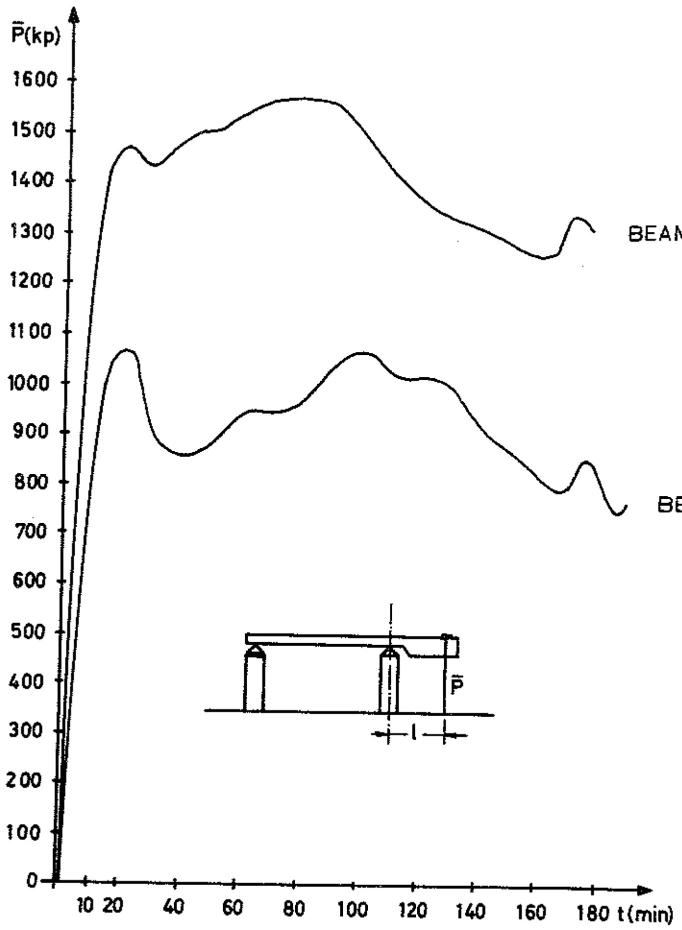


FIG. 3 Variation in the cantilever tension force as a function of time for Beam 1 and 2 during a fire exposure. From Ehm & von Postel (1965).

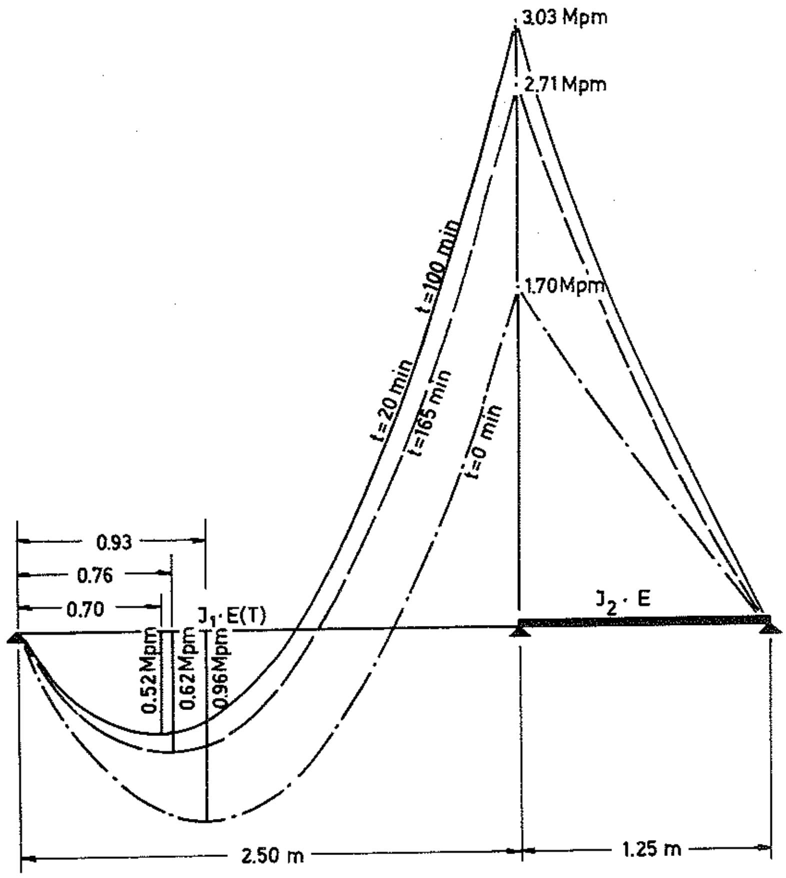


FIG. 4 Continuous Beam 1 - Bending moment distribution calculated from test results, t = 0, 20, 100 and 165 min after beginning of the fire test. From Ehm & von Postel (1965).

Experimental studies of statically indeterminate reinforced concrete beams subject to fire described in the literature are valuable illustrations of the complicated relations which characterize behaviour and bearing capacity for this type of structure. Studies have been limited to an unrealistically simplified standard heating process of fire exposure. Even within this limit the obtained results do not naturally provide the foundation for such general conclusions, which is required as a basis for a theoretical, structural fire engineering design.

However, certain general conclusions can be drawn from the reported studies. As an example it may be observed that the magnitude of the restrained forces and restrained moments which arise within a span of a continuous beam during fire exposure depends mainly on the stiffness of the part of the structure not exposed to fire and in a less degree on the properties of the unheated part. Furthermore it is true that an optimum fixed end degree is obtained if yield joints appear simultaneously at the support and in the span between the supports. Ignoring the extreme exceptional cases, the statically indeterminate structures possess a higher fire resistance than the corresponding statically determinate ones assuming the same utilization degree. This characteristic has been pointed out in the literature by many others including Harmathy (1966), who at the same time, emphasizes the difficulties encountered at the existing fire laboratories, to realize practical, representative and well defined fixed end conditions during fire technological testings. Due to this fact, most of the fire technological testings for classification are of extensive proportions performed on statically determinate structures. This has the consequence that most of the practical applications have resulted in conservative design.

To sum up, it may be concluded from the above statements that systematical and long term planned investigations, concerning the structural behaviour and load-bearing capacity of statically indeterminate structures during fire exposure, have a high priority. Thus, it is important to survey the behaviour and load-bearing capacity corresponding to varying fire parameters comprising the complete fire process, including a determination of the residual stress conditions and the residual carrying capacity after fire. The investigation accounted in the present publication is intended as a contribution towards such a systematical study. The objective plan of the investigation is explained in Section 2 below which follows by a description of the testing facilities. Next, the performed experiments are described in Section 4 after which an illustrated account and analysis of the test results is given in Section 5. In Section 6 the difficulties with a theoretical treatment is discussed and in Section 7 future plans for further studies is reported. Finally a summary and conclusions are presented.

2. OBJECTIVE AND LAYOUT

For a more detailed analysis of the complex structural behaviour of statically indeterminate concrete structures subject to fire it is required to investigate the subject on the basis of a series of fundamental studies which take into account all the significant factors. The significance of the above statement can be realized by observing the many existing investigations on the subject in which a great number of factors have been allowed to act simultaneously and consequently the results have been confined to particular structures under consideration, leaving the more general aspect of the problem untouched.

This paper presents a first partial account of the fundamental studies regarding the behaviour of statically indeterminate concrete structures subject to fire on one side. These studies have been going on for some years at the Division of Structural Mechanics and Concrete Construction, Lund Institute of Technology and during this period 4 graduate thesis (Anderberg et al. (1969), Denker et al. (1970), Erntsson et al. (1971) and Bernow et al. (1972) also have been performed and the results are included in the present report. For a first principal analysis concerning the behaviour of constructions of this type, an experimental investigation is performed, comprising a study of one in this connection extremely pure type of structure, namely a reinforced concrete strip exposed to fire on one side and completely fixed against rotation at both ends while the longitudinal movement is allowed to take place freely. Technically the end rotation is prevented through two controllable external concentrated bending moments varied so that the change in fixed end moments can be followed quantitatively during the whole fire process. In the present investigation a substantial progress compared with the existing published experiments may be noted partly by a study of the subject based on a differentiated fire process and partly by considering the whole fire process including the cooling phase.

Significant parameters studied in an investigation of this kind consist of characteristics of the fire process, plate thickness, concrete composition, reinforcement quality, proportion of reinforcement, thickness in concrete cover and age of the test specimen. The decisive factors governing the characteristics of fire process are, factors such as the size of fire compartment, shape and ventilation properties characterized by an opening factor given by the formula $A\sqrt{H}/A_t$, thermal properties of the surrounding constructions and the fire load, q. Here A denotes the total opening area, H, a weighted mean value of the vertical dimensions of these openings and A_t, the total bounding area of the fire compartment including the openings. The existing works by Magnusson & Thelandersson (1970) provide now a sound basis for fire engineering design in the form of gas temperature-time curves describing the complete fire process with varying opening factors $A\sqrt{H}/A_t$, fire load, q, and thermal properties for the enclosing constructions. The basis theoretically evaluated for fire load of mainly wooden fuel is illustrated in FIG. 5 in which the material enclosing the fire compartment possesses a thermal conductivity of λ = 0.70 kcal/m·h·°C and a thermal capacity of γc_p = 400 kcal/m^3 °C. These values are representative of the mean properties in the current temperature range. As shown in FIG. 5, the characteristics of the fire process are chosen as the basis for the investigation accounted in this paper.

The experiment is performed both for unloaded and loaded reinforced concrete strips. The loading consists of two concentrated loads which are applied symmetrically and the strips are fixed against rotation at both ends. The parameters which have primarily been varied during the experiment are the characteristics of the fire process, the age of the specimen, and concrete mixture. Thus, with respect to concrete mixture, partly the variations in water-cement ratio with constant cement quantity and partly the variations in cement quantity with constant water-cement ratio are studied. The experiment schedule includes 5 test series with following characteristics.

Test series A1 and A2:	Nil-loaded plate strips. The fire process is varied by changing the opening factor and fire load.
Test series B and C:	Nil-loaded plate strips. Concrete composition and age of the specimen are varied.
Test series D:	Loaded plate strips. Loading levels and fire process are varied by changing the opening factor.

The purpose of the experiment has been to observe the structural behaviour of a statically indeterminate reinforced concrete construction subject to fire in the most possible precise and systematical manner when fire process, load and material characteristics vary according to the above mentioned schedule. The observation has thus included a study of the structural behaviour under a complete fire process, namely both the heating phase and the subsequent cooling phase. A significant part of the experiment deals also with determining the residual bearing capacity, residual stress and residual deformation states which are characteristic features of a structure subject to fire after cooling to ordinary room temperature. Besides, knowledge about these residual states is fundamental for estimating the servicability of a fire-exposed structure after fire and for estimating the possible required reparations.

In order to make a more detailed analysis of the test results possible, the restrained thermal bending moment caused by heating, temperature gradients, deformation state of the concrete strips subject to fire and the fire

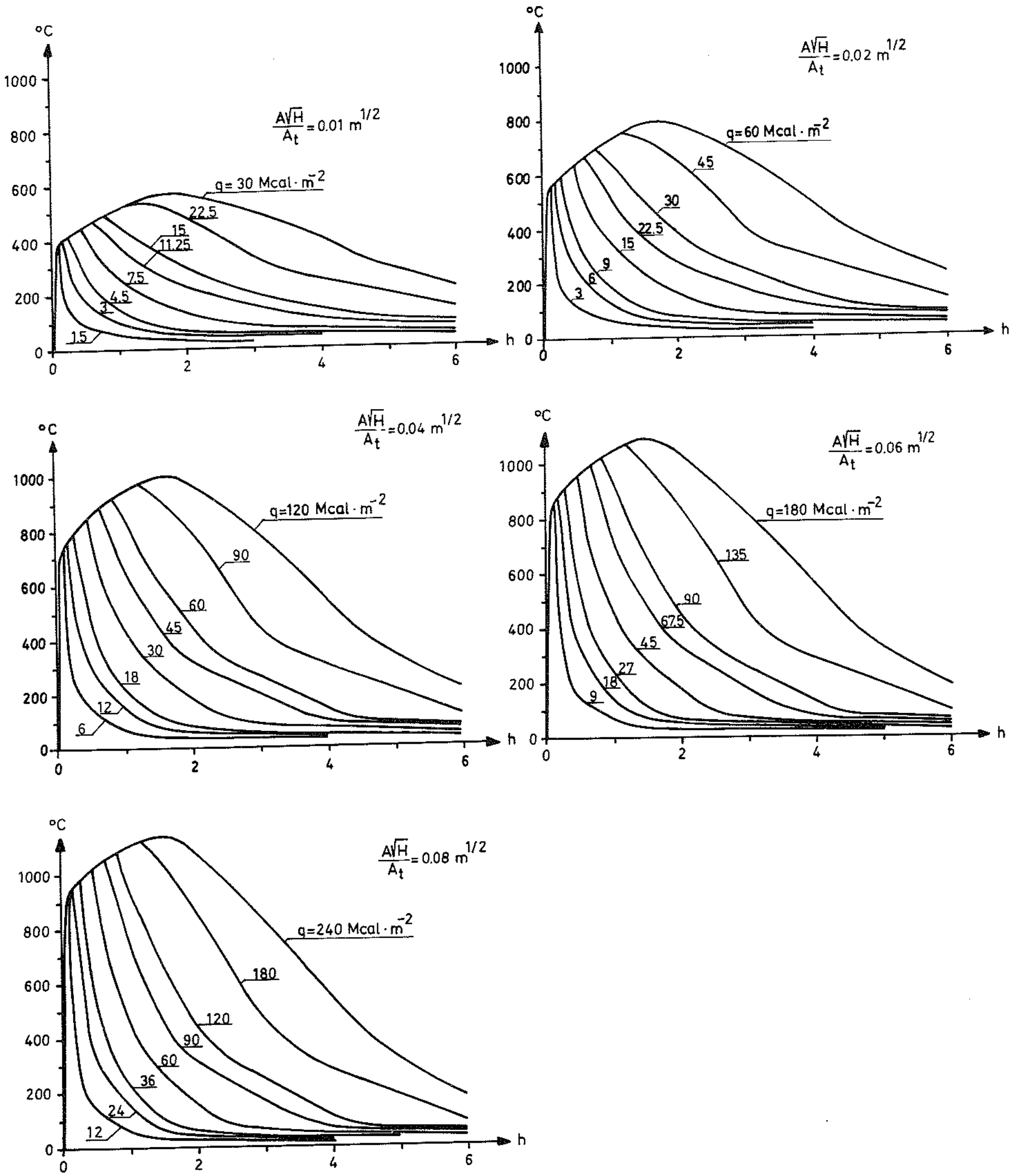


FIG. 5 Calculated time graphs of temperature of combustion gases for type A of enclosed space at different opening factors $A\sqrt{H}/A_t$ = 0.01, 0.02, 0.04, 0.06 and 0.08 $m^{1/2}$. From Magnusson & Thelandersson (1970).

process characteristics belonging to the gas-fuelled furnace used in the experiment, have been continuously measured. The residual bending stiffness and the residual load-bearing capacity of the plate strips have been determined after cooling down to room temperature.

3. TESTING FACILITIES

The principal form and function of the testing facilities can be observed from FIG. 6 and FIG. 7. The tests were performed for reinforced concrete plate strips which were symmetrically located on a roller support at A and a horizontal hinge support at B and supplied with cantilevers AC and BD (FIG. 6). The plate strips were exposed to fire underneath in the area AB through a gas burner in a furnace whose longitudinal upper wall edge approximately coincied with the upper side of the concrete strips. Location of the hinge and roller supports of the plate strips were directly placed against the short wall of the furnace. The desired degree of restraint against rotation on supports A and B was realized by applying concentrated bending moments on the cantilever ends C and D, through concentrated forces P_1, applied by means of loaded wires fastened to the cantilever having lever arms denoted by F. During the experiment these concentrated moments were continuously varied such as to maintain the rotation at support A and B at zero value, controlled by water levels which were located on the supports. In test series D in which the plate strips were vertically loaded, the loads were applied in the form of two symmetrically located parallel loads P_2 according to FIG. 6. FIG. 7 gives a more detailed picture of the whole testing equipment. The intermediate zone between these concentrated loads corresponds to a constant bending moment. Under the preliminary test series there arised many technical problems which led to a successive improvement and completion of the testing facilities.

3.1 HEATING ARRANGEMENT

The furnace used in the experiment is built of 30 cm long refractory bricks and externally covered by sheet-metal to protect it against external wear. The detailed picture of the furnace is illustrated in FIG. 8. On every short side of the furnace there is an opening through which the burners may be applied. These burners are adjustable within a wide range so that the desired temperature-time curve of the fire cell can be followed. The smoke is exhausted through a chimney which leads from one of the long sides of the furnace and discharges in the open air. By continuously adjusting the air supply and cross sectional area of the chimney a prescribed temperature-time curve of the furnace can be followed satisfactorily. Heating of the furnace is attained through gas burners which are specially manufactured for this purpose (FIG. 9) by the Oil Consumers (OK). The gas and air supply of the burners can be controlled by a steering

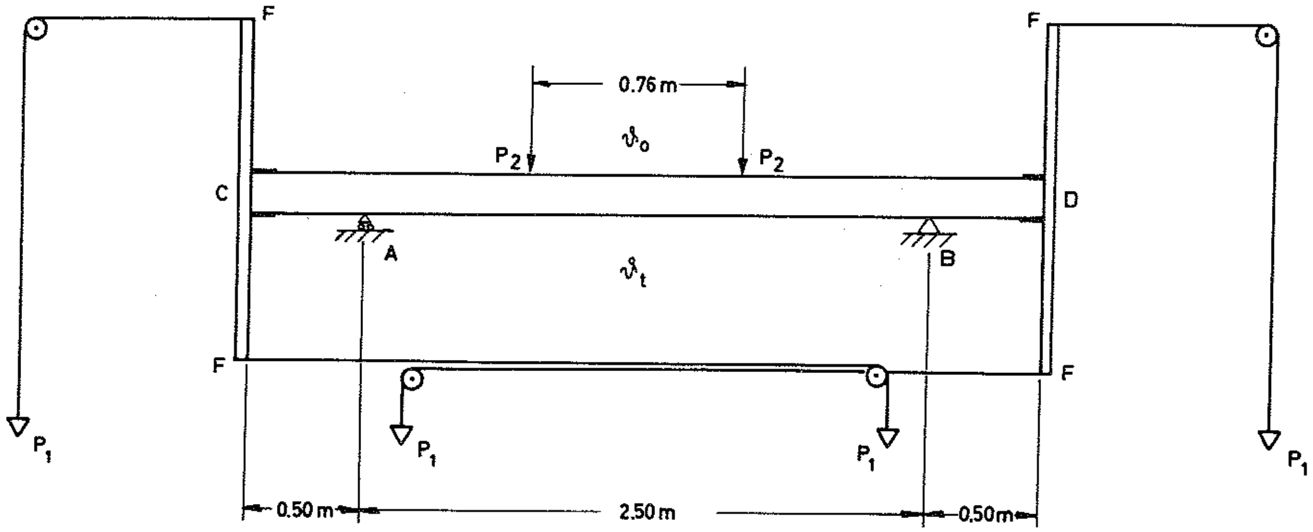


FIG. 6 Principal form of test arrangement.
ϑ_o = room temperature, ϑ_t = furnace temperature.

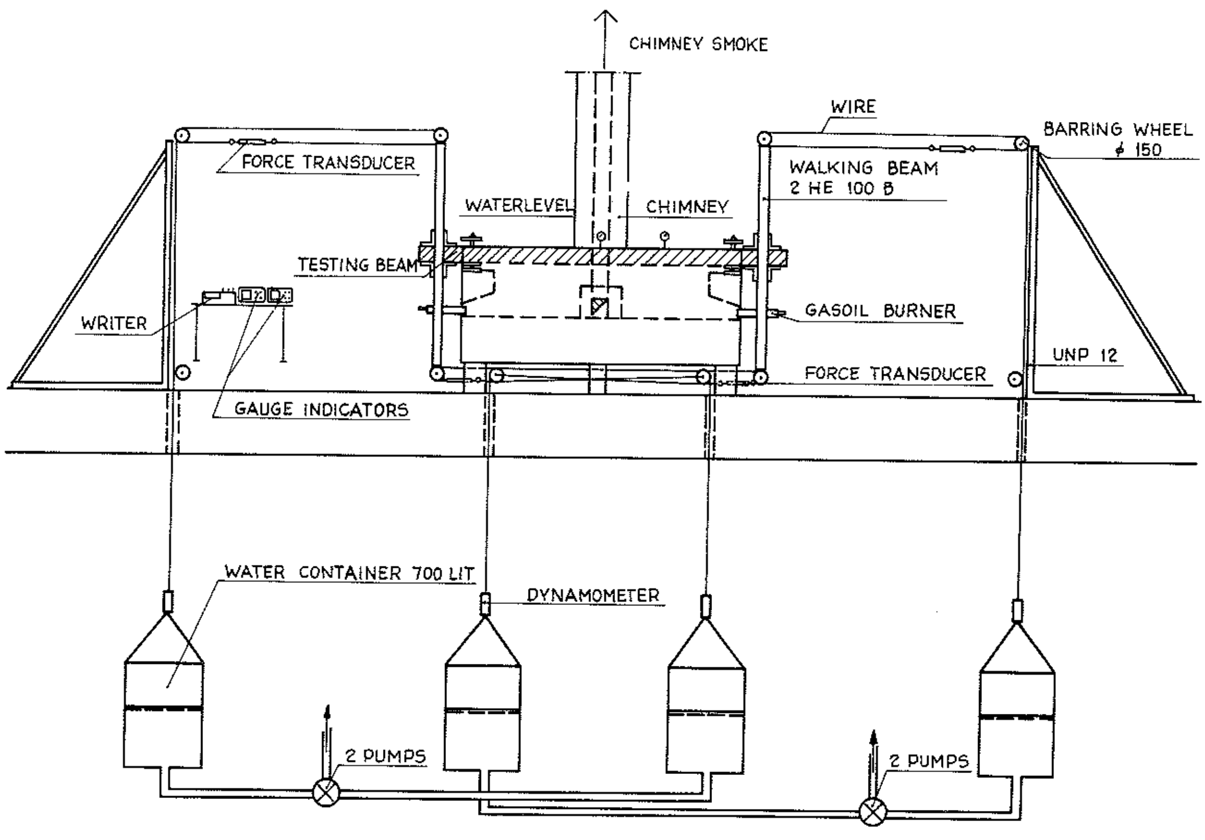


FIG. 7 Testing facilities.

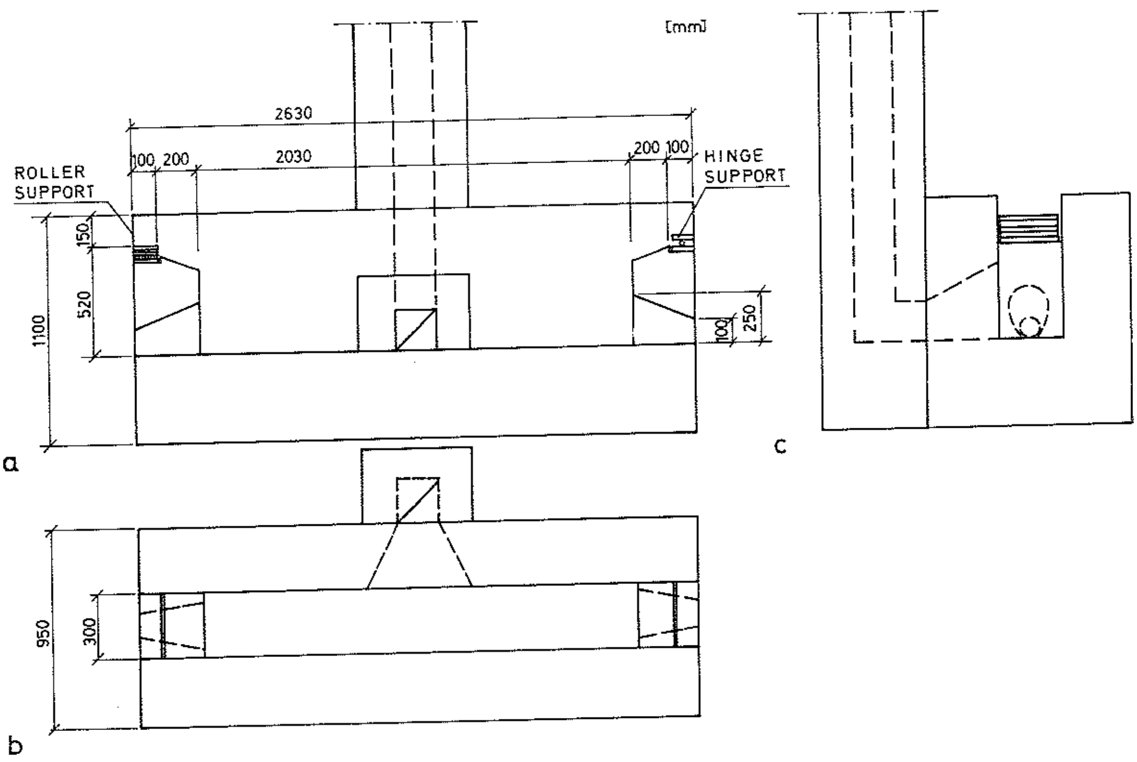


FIG. 8 The furnace seen in three sections namely a) from the long side b) from above c) from the short side.

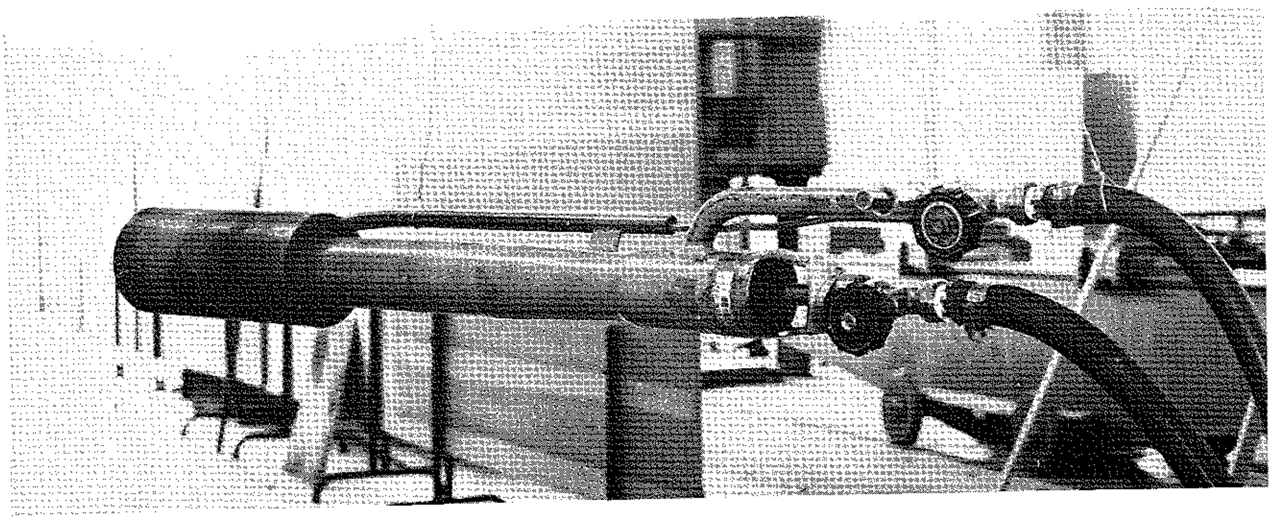


FIG. 9 Gasoil burners manufactured by the Oil Consumers (OK).

system. The maximum capacity of these burners amounts to 7-8 l gas-oil/min., and 1 l gas corresponds to 6 kcal effective thermal value. The gas pressure often used is 1 kp/cm^2.

The use of the above mentioned heating system in the case of too slow or too rapid fire processes may bring about the risk of temperature falls of short duration due to the fact that a burner may become extinguished owing to pressure changes in the furnace. This temperature fall may last for some minutes and may in some cases have an appreciable influence on the testing process. In an intermediate range of the fire process most commonly used, the burners have proved to function quite satisfactorily. By setting 5 thermocouples (4, 20, 8, 16 and 12) of chromel-alumel-type in a Kanthal pyrometer conduit and placing it in the longitudinal symmetrical section of the furnace according fo FIG. 10 the furnace temperature was registrated by a 24-channeled printer of Honeywell model manufactured by Braun Electronic. The detailed design characteristics of the furnace and the location of burners and chimney openings during the experiment rendered the highest furnace temperature in a region between thermocouples 16 and 20 which was selected as the reference region for the desired temperature. Starting from this region and proceeding towards the short sides, the temperature somewhat decreases. FIG. 11 illustrates more closely how temperature distribution in the furnace varies at a certain temperature-time curve. FIG. 12 shows time curve for the heat transfer coefficient α of that area of plate strips which is exposed to fire at a certain temperature-time curve of the fire compartment. The reason for fluctuation of α-value is due to non stationary conditions in the furnace.

3.2 ARRANGEMENT FOR END RESTRAINT

In order to simulate fixed-end conditions experimentally without preventing the axial change of length, lever arms attached to cantilever end of the plate strips are used as shown in FIG. 6 and FIG. 7. The lever arms are 2.25 m long and consist of two I-beam sections (HE 100 B) threaded over cantilever ends and clamped by means of four angle profiles (L 120). A porous pad was placed between the plate strip and the angle profile in order to achieve a more uniform load transmission. As shown in FIG. 6 and FIG. 7 the pair of forces P_1 producing the cantilever moments were applied through cables fastened to the ends of the lever arms. These cables lead through pulleys to a cellar where they are connected to four water containers of 700 l capacity each, serving as weights. The detailed assemblage of the pulleys is shown in FIG. 13 from which it is observed that the pulleys are firmly locked around an axis the pivoted ends of which are placed in a ball bearing cup that reduces friction to a minimum. In the initial tests an alternative solution was used in which the pulley could rotate around a fixed axis. But the friction developed by using this method was too large

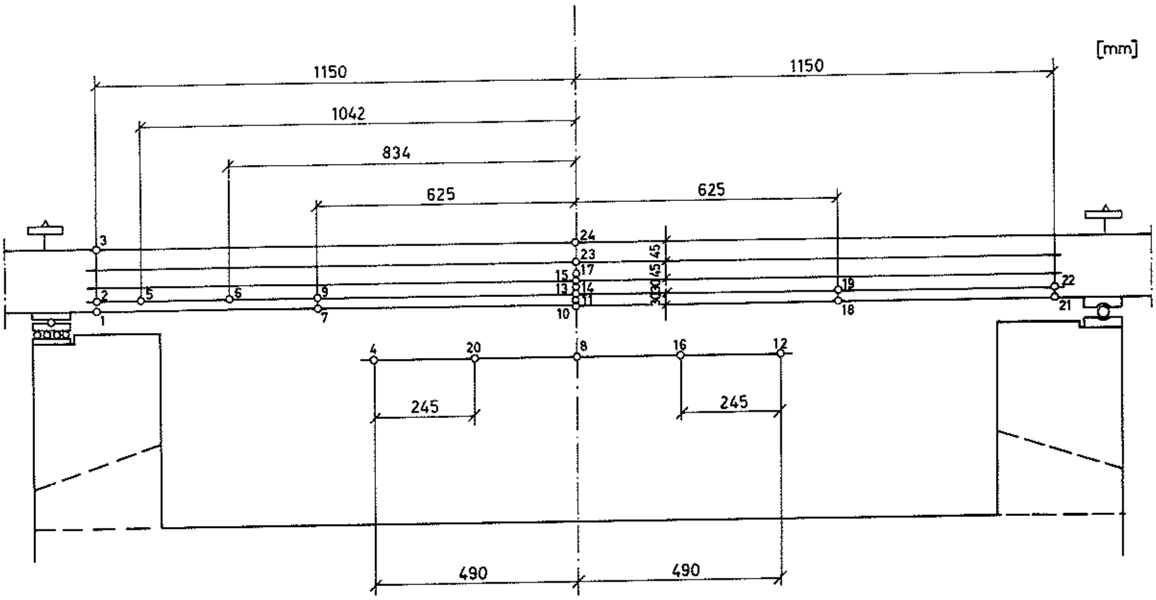
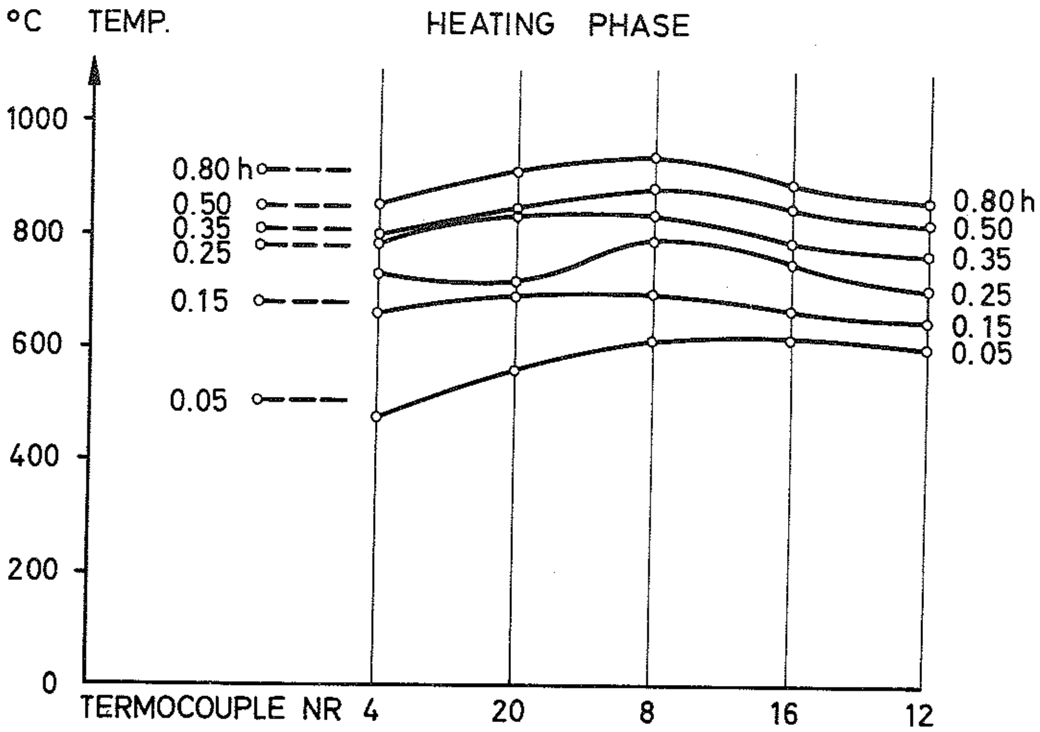


FIG. 10 Location of thermocouples (type Chromel-alumel) inside the furnace and the concrete plate strip.



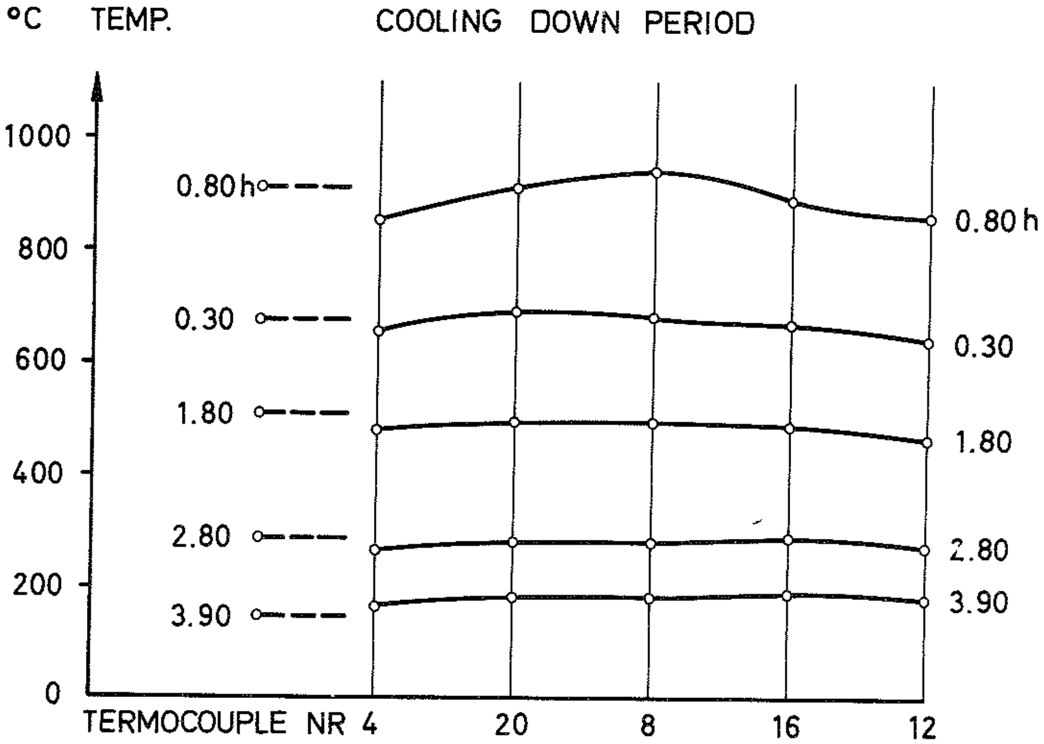


FIG. 11 Measured temperature distribution inside the furnace during a fire test. Top: Heating phase. Bottom: Cooling down period. Dashed lines indicate desired temperature.

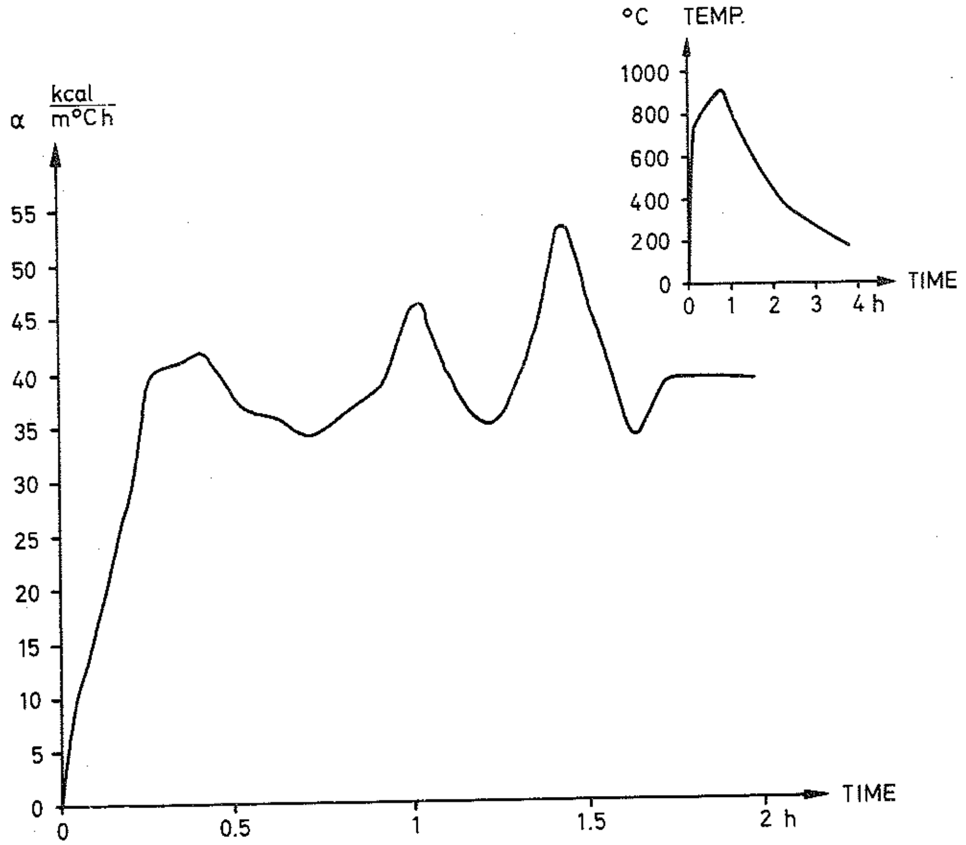


FIG. 12 Variation in surface coefficient of heat transfer α in that area of the plate strips which is exposed to fire at a certain temperature-time process of the furnace.

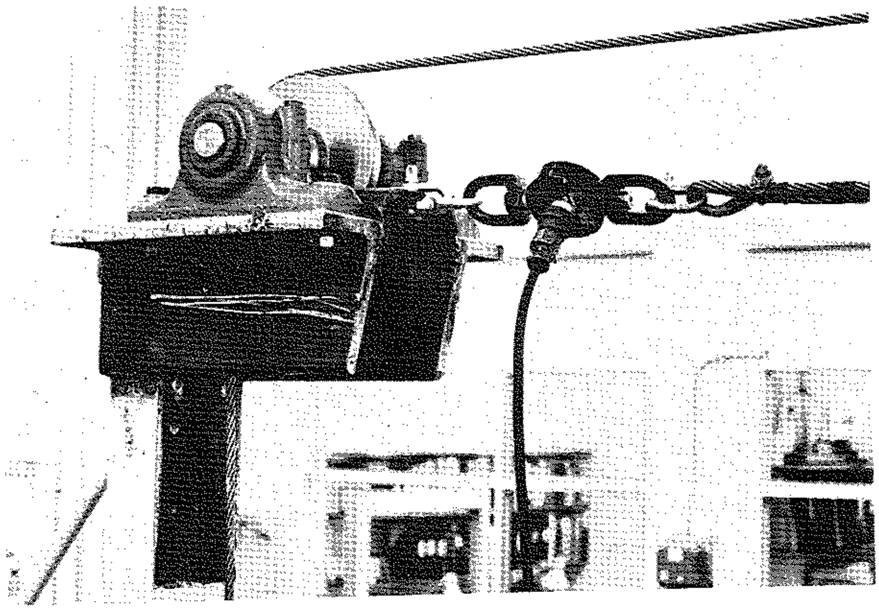


FIG. 13 Assemblage of the pulleys.

to be accepted. According to the detail in FIG. 13, the maximum friction produced is only 5 kp per pulley which is considered satisfactory for the present investigation. Final choice of the detail was preceded by a series of loading tests in order to study the different factors influencing the magnitude of friction, namely the cross sectional area of the cable, size of the pulley and length of the axis. From these studies it could be concluded that the above mentioned factors had a negligible influence on the friction produced in the ball bearing cup described above.

The forces P_1 were applied by filling the water containers with water through connected rubber tubes. The filling could be adjusted by four taps located nearby the furnace, one for each water container (FIG. 14). In order to obtain loads of the same magnitude in the cables connected to the same lever arm, assuring that no longitudinal forces are introduced on the plate strips, the water containers are communicating with eachother in pair.

Emptying water tanks is achieved by four pumps, with a capacity of 55 l/min each. In the preliminary tests, different measuring procedures have been used in determining the magnitude of the load, including the use of dynamometers with a precision of ± 5 kp, attached to the cables. In order to reduce the amount of manual data collection, electrical loading cells are now used instead, by connecting the cells through gauge-indicators KRG-4 from Bofors with a measuring range of 0 - 2000 kp to 2-channeled Servogor printers which continuously register the load with a precision of ±3 kp.

In certain cases the restrained moment changes direction during the cooling phase of the fire process. Therefore, in order to be able to follow the behaviour of the plate strips in a complete fire process subject to such conditions, a special arrangement is required to reverse the direction of application load. For this purpose a switching arrangement has been designed, which is shown in FIG. 15, for the upper end of the lever arm (cf. with FIG. 14). This switching construction enables the forces in the wires to be directed toward the furnace and render a positive fixed-end moment. The same principal idea have been used for the lower end of the lever arm.

In order to assure that no end rotation took place in the strips, two water levels fastened to T-profiles were placed on each support. T-profiles were poured in the concrete section over the support. Two parallel water levels placed over each support enabled the control of possible warping in the plate strips. Should, during the test crack formation take place in the section over the support, the measurement with this method could result in lower precision. To cope with this problem and assure further precision, a third water level was placed just outside each support. This test arrangement was developed after a series of less successful

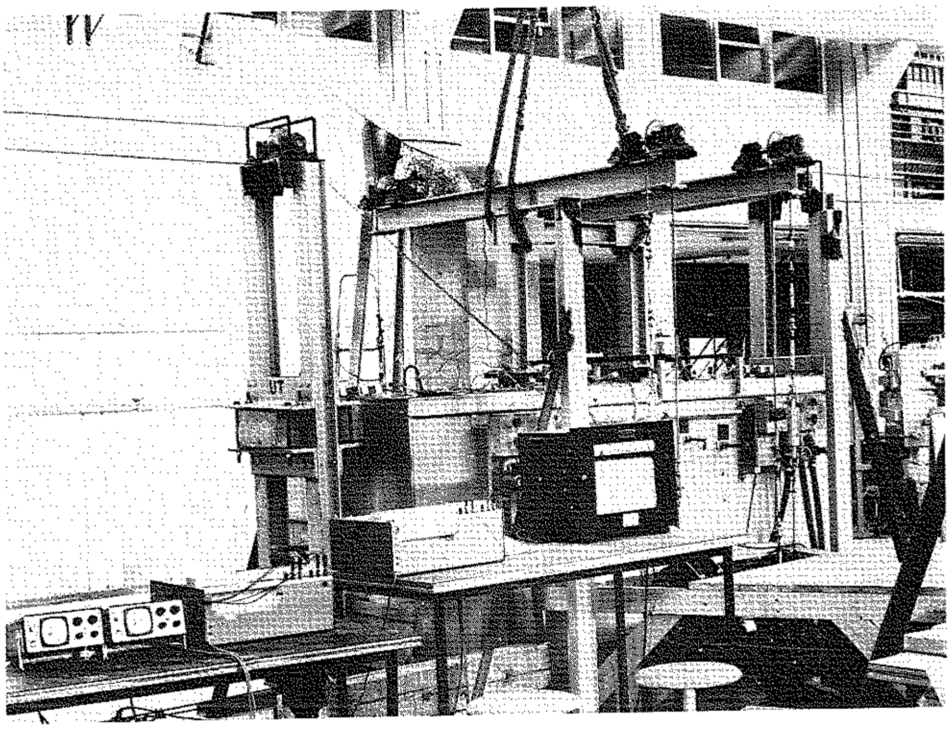


FIG. 14 Picture of testing facilities including load arrangement.

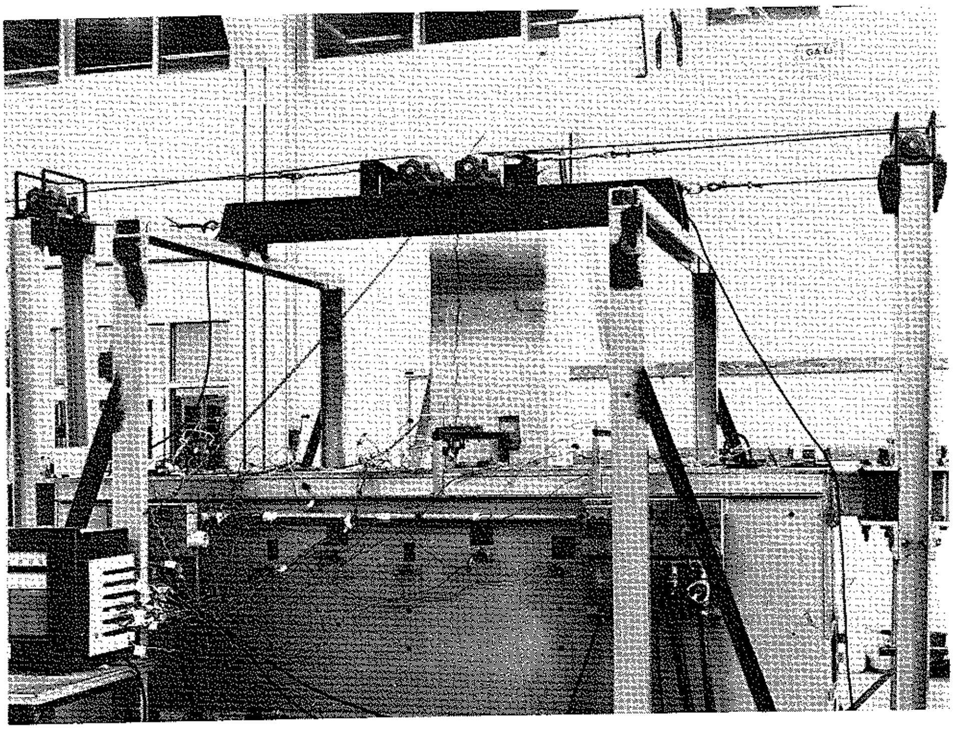


FIG. 15 Picture of the switching construction for the upper end of lever arms, which enables the forces in the wires to be directed toward the furnace and render a positive restrained bending moment.

experiments using other alternative arrangements for preventing end rotation. This finally adopted procedure, however, keeps two persons busy controlling and maintaining water levels at zero-angle position by continuously filling the containers with water or emptying them through the taps as shown in FIG. 7. The purpose of the future experiments is to automatize the procedure in such a way that photocells through a measuring device control the whole process of filling or draining the containers. This can be achieved by impulses sent to automatically controlled magnetic taps which fill or drain the containers.

3.3 LOAD ARRANGEMENT

FIG. 16 and FIG. 17 show the basic construction used in series D for the application of symmetrically placed vertical loads P_2 shown schematically in FIG. 6. The arrangement consists of two load-transmitters connected to a steel beam (HE 100 B). The load is applied through cables passing over pulleys above the transmitters, after passing over the pulleys fastened in the floor, continue down to the celler where they are attached to a beam (HE 100 B), about 4 m long, on which an arbitrary load may be applied (FIG. 16 and FIG. 17). To this the dead weight of the load-transmitters, which amounts to a total of 230 kg should be added. A more detailed picture of the load arrangement is illustrated in FIG. 14.

3.4 TEMPERATURE MEASUREMENTS

In order to continuously determine and register the temperature-time field of the strips, chromel-alumel thermocouples were poured in the strips. The location of the thermocouples is shown in FIG. 10. Thermocouples 10, 11, 13, 14, 15, 17, 23, 24, and 1, 2, 3 rendered the temperature gradients in the middle section and the section over the support respectively. Thermocouples 1, 7, 10, 18, 21, and 2, 5, 6, 9, 13, 19, 22, and 2, 24 on the other hand, yielded the temperature distribution along the strip at different depths. All the thermocouples belonging to the plate strips together with those of the furnace were connected to a Honeywell 24-channelled printer which registrated the temperatures with a printing speed of 24 channels per 3 minutes.

3.5 DEFORMATION MEASUREMENTS

In order to measure the deformation produced in the plate strips, 4 gauge indicators manufactured by the machine company Karlebo were used. The precision of these indicators is 10^{-2}mm. Two of the indicators yielded the deflection at the mid-section of the strips while the other two yielded the deflection at 1/4-section. The possibility of warping in the strips is controlled by double gauges located at every measuring section. The gauge readings were performed manually. For future investigations it is planned to use potentiometers instead provided that they can satisfactorily be protected against heat and vapour diffusion resulting from the plate strips. Thereby a direct registering of the deflection process on the printer is achieved.

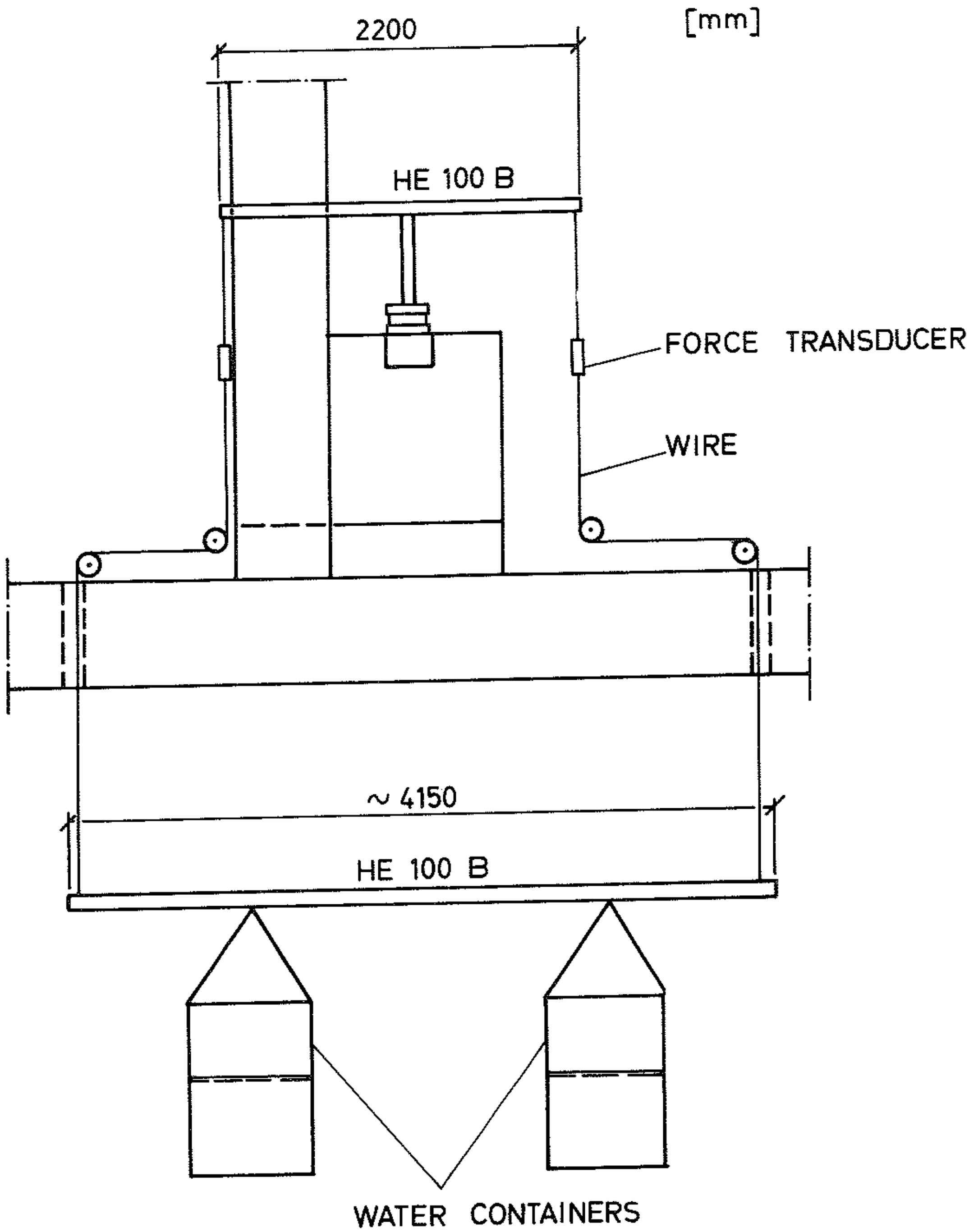


FIG. 16 Construction for load arrangement. Section through the short side of the furnace.

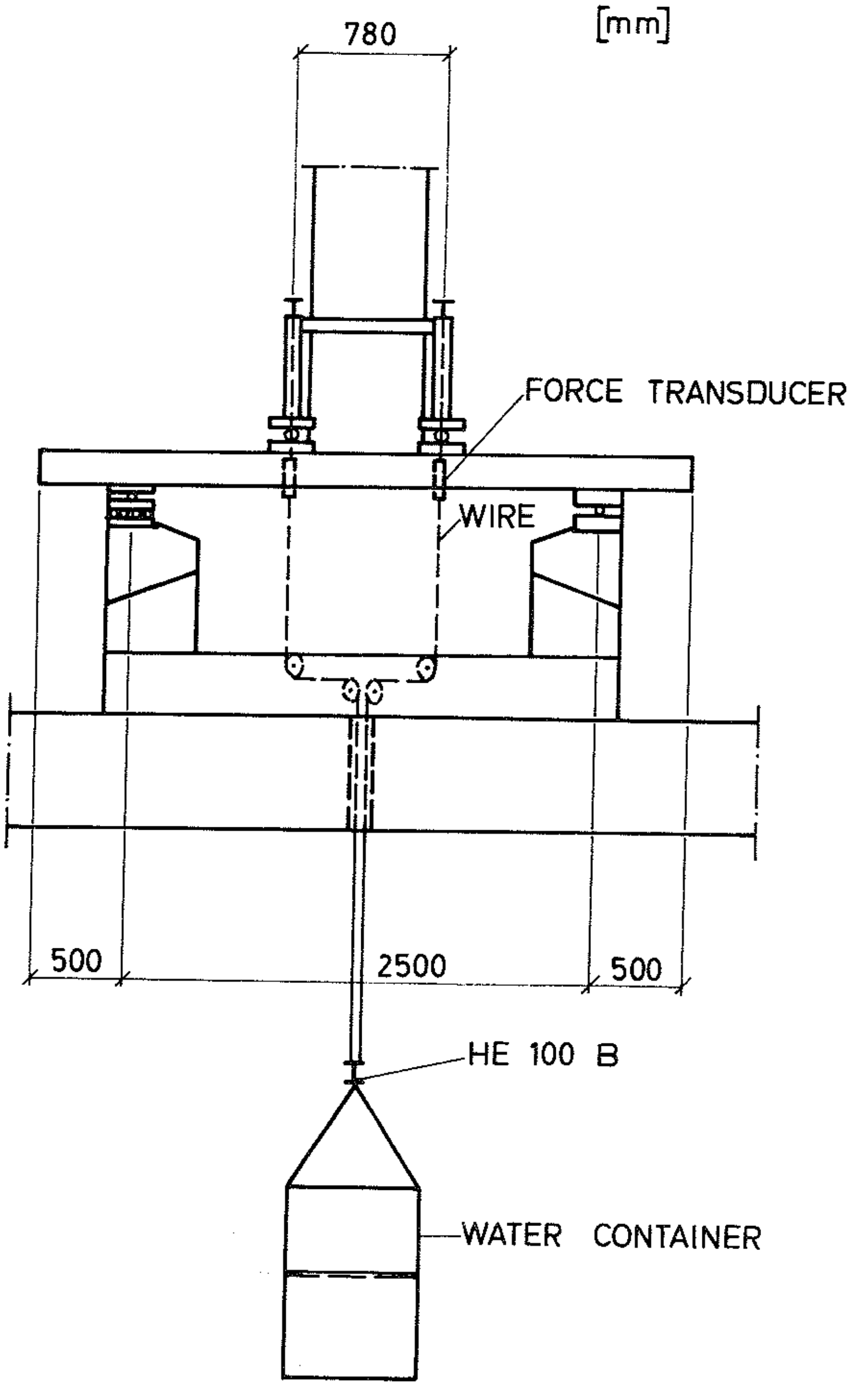


FIG. 17 Construction for load arrangement. Section through the long side of the furnace.

4. DESCRIPTION OF THE EXPERIMENT

4.1 FUNDAMENTAL MATERIAL DATA

In the test series performed, all the strips have had a length of 2.5 m between the supports and a length of 0.5 m for the end cantilevers; the cross sectional area of all strips being 300 mm in breadth and 150 mm in depth (FIG. 18). 5 Ø10Ks40 reinforcement bars have been symmetrically placed along the whole length of the strip both at the lower and the upper side with a protecting concrete cover of 25 mm. In order to make the cantilevers stronger compared with the part of the strips subject to fire and reduce crack formation and deformation in them, each cantilever was further provided with 5 stirrups Ø10Ks40. A representative σ-ε-curve for the longitudinal reinforcement in series A1 is illustrated in FIG. 19. The values of yield and ultimate strengths belonging to the longitudinal reinforcement may be observed from the following table.

Test series	Reinforcement Yield strength σ_s kp/cm^2	 Ultimate strength σ_B kp/cm^2	Remarks
A1	4690	7040	Mean value of 6 tests
A2	4900	7660	-"-
B and C	5080	7510	2 tests
D	4580	6810	

In order to control the concrete quality, for every strip, 6 cubes 15 cm in length were poured to determine the compressive strength and 6 cubes 15 cm in length poured to determine the tensile strength (test series A1 only) according to split-cube test, and two nonreinforced beams (800 x 150 x 100 mm) poured to determine the bending tensile strength. All the strips underwent 5 days of water curing and thereafter air curing at a temperature of 20°C and the relative air humidity of 50%, up to the time when the fire tests were performed.

The data regarding the concrete strength is illustrated in TABLE 1 where water-cement ratio, cement-paste quantity and age of the specimens may also be observed. Concrete composition in 1 m^3 of concrete is as follows:

Water-cement ratio			0.63		0.77	0.52
Cement-paste quantity	(ℓ)	257	277	296	277	277
Water	(ℓ)	170	183	196	195	172
Standard cement	(kg)	270	290	311	255	330
Gravel	(kg)	994	968	942	968	968
Macadam	(kg)	973	948	923	948	948

(d_{max} = 16 mm)

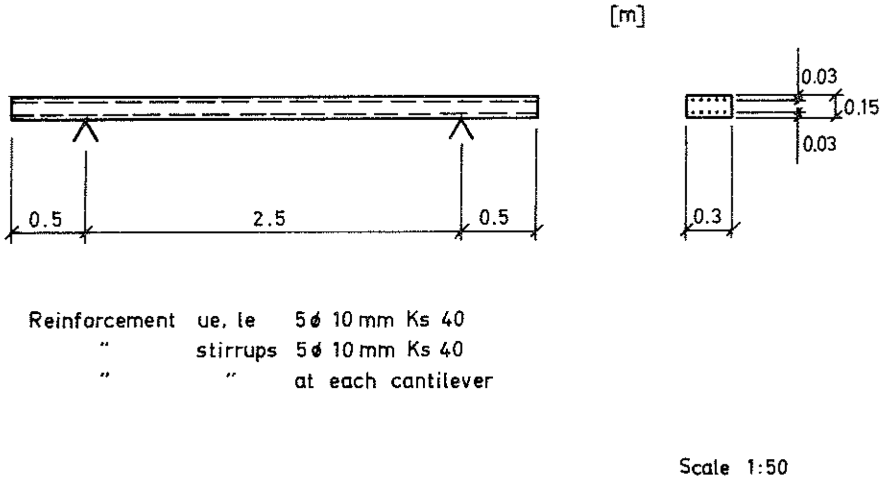


FIG. 18 Reinforced concrete fire test specimen used to simulate a plate strip.

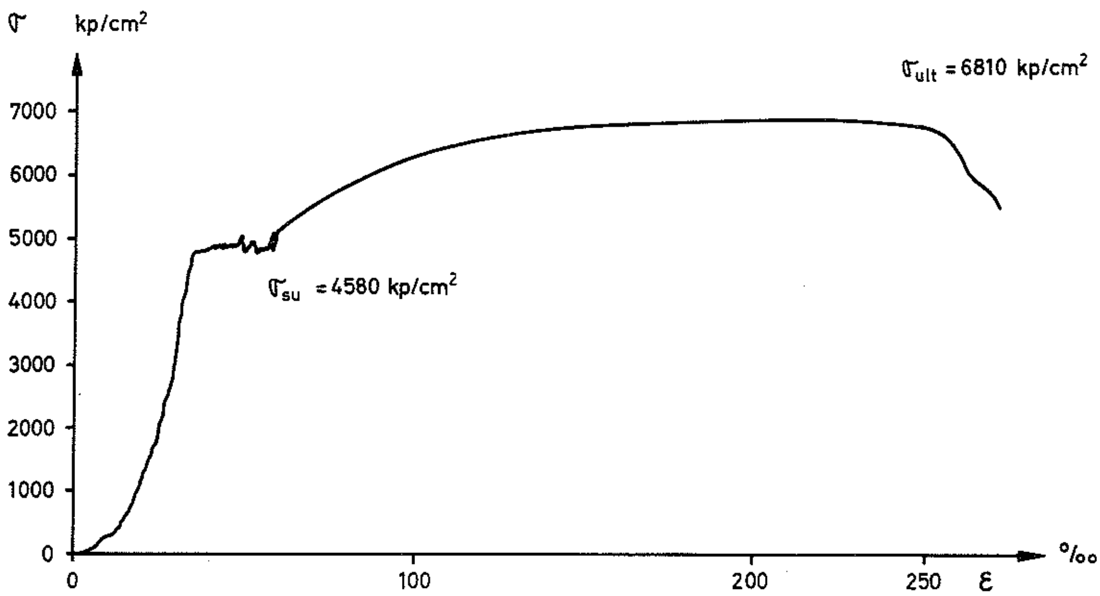


FIG. 19 σ-ε-curve representing the longitudinal reinforcement used in test series A1.

TABLE 1 Material data

Test series		Compressive strength: Mean value kp/cm^2	Compressive strength: Relative standard deviation %	Tensile strength: Mean value kp/cm^2	Tensile strength: Relative standard deviation kp/cm^2	Tensile bending strength: Mean value kp/cm^2	Water cement ratio	Amount of cement-paste ℓ	Testing age months
A1	1	350	3	22	15	43	0.63	280	2
	2	390	5	24	6	45	"	"	2
	3	380	3	21	3	41	"	"	2
	4	400	5	25	8	45	"	"	2
	5	400	4	25	9	53	"	"	2
	6	350	3	21	13	45	"	"	2
	7	390	4	24	10	53	"	277	2
	8	385	-	-	-	38	"	"	2
	9	-	-	-	-	-	"	"	5
	10	450	3	-	-	51	"	"	6
	11	460	5	-	-	60	"	"	6
A2	1	286	2	-	-	33	"	277	1
	2	280	4	-	-	35	"	"	1
	3	291	2	-	-	35	"	"	1
	4	252	2	-	-	28	"	"	1
	5	307	5	-	-	36	"	"	1
	6	279	3	-	-	35	"	"	1
	7	279	5	-	-	31	"	"	1
	8	282	4	-	-	25	"	"	1
	9	278	4	-	-	37	"	"	1
	10	297	2	-	-	35	"	"	1
B	1	420	-	-	-	53	0.63	277	3
	2	530	-	-	-	55	0.52	"	3
	3	220	-	-	-	35	0.77	"	3
	4	360	5	-	-	39	0.63	"	1
	5	220	3	-	-	29	0.77	"	1
	6	400	5	-	-	44	0.52	"	1
	7	450	4	-	-	40	0.52	"	1/2
	8	260	3	-	-	27	0.77	"	1/2
	9	350	4	-	-	41	0.63	"	1/2
C	1	490	4	-	-	46	0.63	277	1
	2	460	3	-	-	49	"	257	1
	3	430	2	-	-	44	"	296	1
	4	422	8	-	-	60	"	277	3
	5	455	5	-	-	63	"	257	3
	6	342	6	-	-	48	"	296	3
	7	415	7	-	-	58	"	277	4
	8	356	6	-	-	55	"	257	4
	9	492	5	-	-	62	"	296	6
D	1	328	6	-	-	40	0.63	277	1
	2	306	3	-	-	33	"	"	1
	3	354	9	-	-	40	"	"	1
	4	371	5	-	-	44	"	"	1
	5	372	2	-	-	39	"	"	1
	6	364	4	-	-	46	"	"	1
	7	424	9	-	-	54	"	"	1
	8	388	3	-	-	50	"	"	12
	9	328	6	-	-	34	"	"	12
	10	345	2	-	-	35	"	"	1
	11	366	5	-	-	35	"	"	1
	12	335	2	-	-	33	"	"	1

The gravel and macadam were composed of a material rich in quarzite.

In certain cases concrete test samples for determining the coefficient of thermal conduction, λ, and coefficient of thermal expansion, corresponding to zero-loaded specimens, have been made. The specimens were sent to the Central Laboratory at Höganäs AB, where the coefficient of thermal expansion, was determined within the temperature range 200-1000° C using a device constructed by Stålhane-Pyk (1923). Besides, the coefficient of thermal conduction was determined at room temperature by Christiansen's plate-apparatus method (1955). The coefficient of thermal expansion was determined within the temperature range 20-1000°C for nil-loaded specimens. The results concerning the coefficient of thermal conduction, λ, for a specimen from test series A1 is illustrated in FIG. 20, where the variation of λ both during the heating and the cooling process may be observed. It may be noted that the coefficient of thermal conduction decreases intensively up to 500°C and, at higher temperatures, it is characterized by insignificant variations. However, during the cooling phase when the temperature is less than 300°C, λ seems to increase somewhat. After investigations concerning the thermal properties of concrete, Ödeen and Nordström (1972) have recently published rather extensive test results regarding the specific heat and coefficient of thermal conduction of concrete with variations in the aggregate type, aggregate gradation, and water-cement ratio.

The results of this investigation, illustrated in FIG. 21, shows the variation of the coefficient of thermal conduction as a function of temperature, when the aggregate is granite, cement/aggregate = 1/3 and water-cement ratio = 0.65 which, apart from the aggregate, is in aggreement with the concrete used in series A1. By comparison with FIG. 20, it may be observed that the differences, except for the last curve segment during cooling, are rather small.

The results obtained for the coefficient of thermal expansion of nil-loaded specimens in series A1 is illustrated in FIG. 22. It may here be noted that the coefficient of thermal expansion grows significantly up to 600 - 700 °C where it attains the maximum value, 1.4%.

For the present case, it is of vital importance to have a knowledge on the strength and deformation properties of concrete at elevated temperatures. As an example on how the compressive strength of concrete changes with temperature, the result from an investigation by Abrams(1971) is illustrated in FIG. 23, in which the compressive strength as a function of time is shown for a) nil-loaded specimens subject to heating and tested in the hot state; b) specimens subject to heating and loaded with 40% of the ultimate load and tested in the hot state; and c) nil-loaded specimens subject to heating and tested 7 days after cooling down to room temperature.

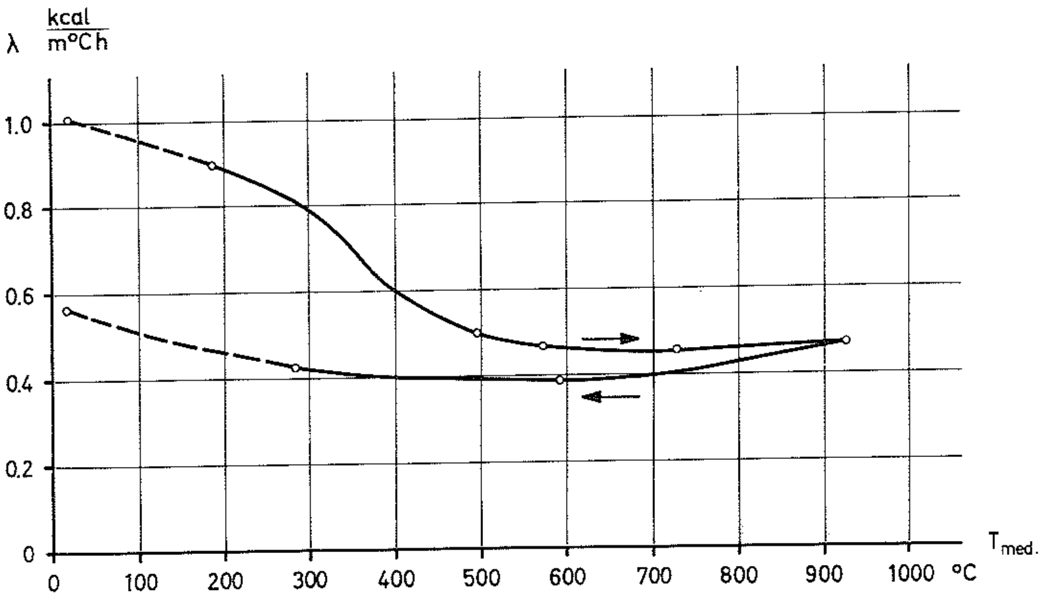


FIG. 20 The temperature-dependence of the coefficient of thermal conduction, λ, during the heating and cooling phases for a specimen representing test series A1 and determined at the Central Laboratory at Höganäs AB.
→ Heating phase
← Cooling phase

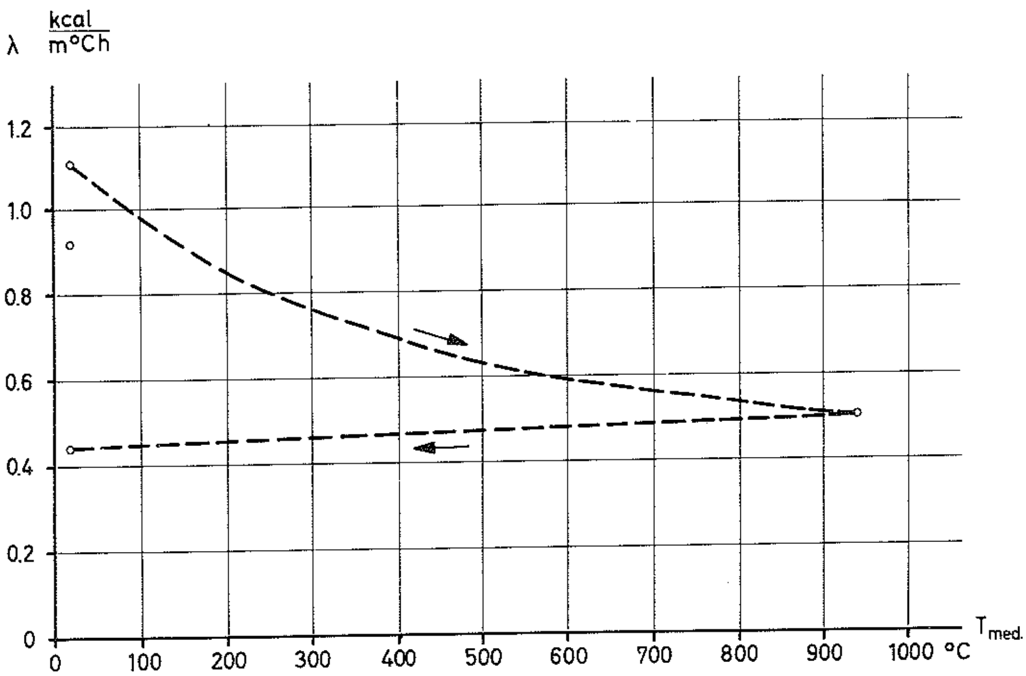


FIG. 21 The temperature-dependence of the coefficient of thermal conduction, λ, during the heating and cooling phases.
Aggregate: Granite.
Cement-aggregate-ratio: 1/3.
Water-cement ratio: 0.65.
From Ödeen & Nordström (1972).

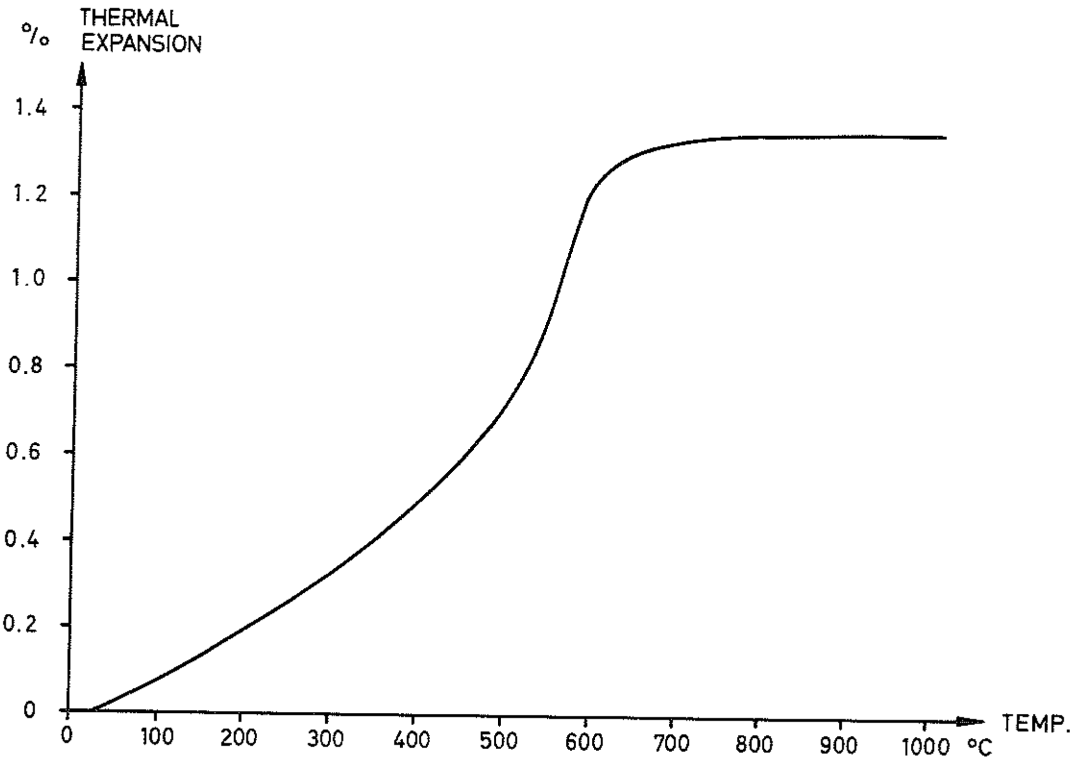


FIG. 22 Coefficient of thermal expansion of nil-loaded specimens in test series A1 in the temperature-range 20-1000º C determined at the Central Laboratory at Höganäs AB.

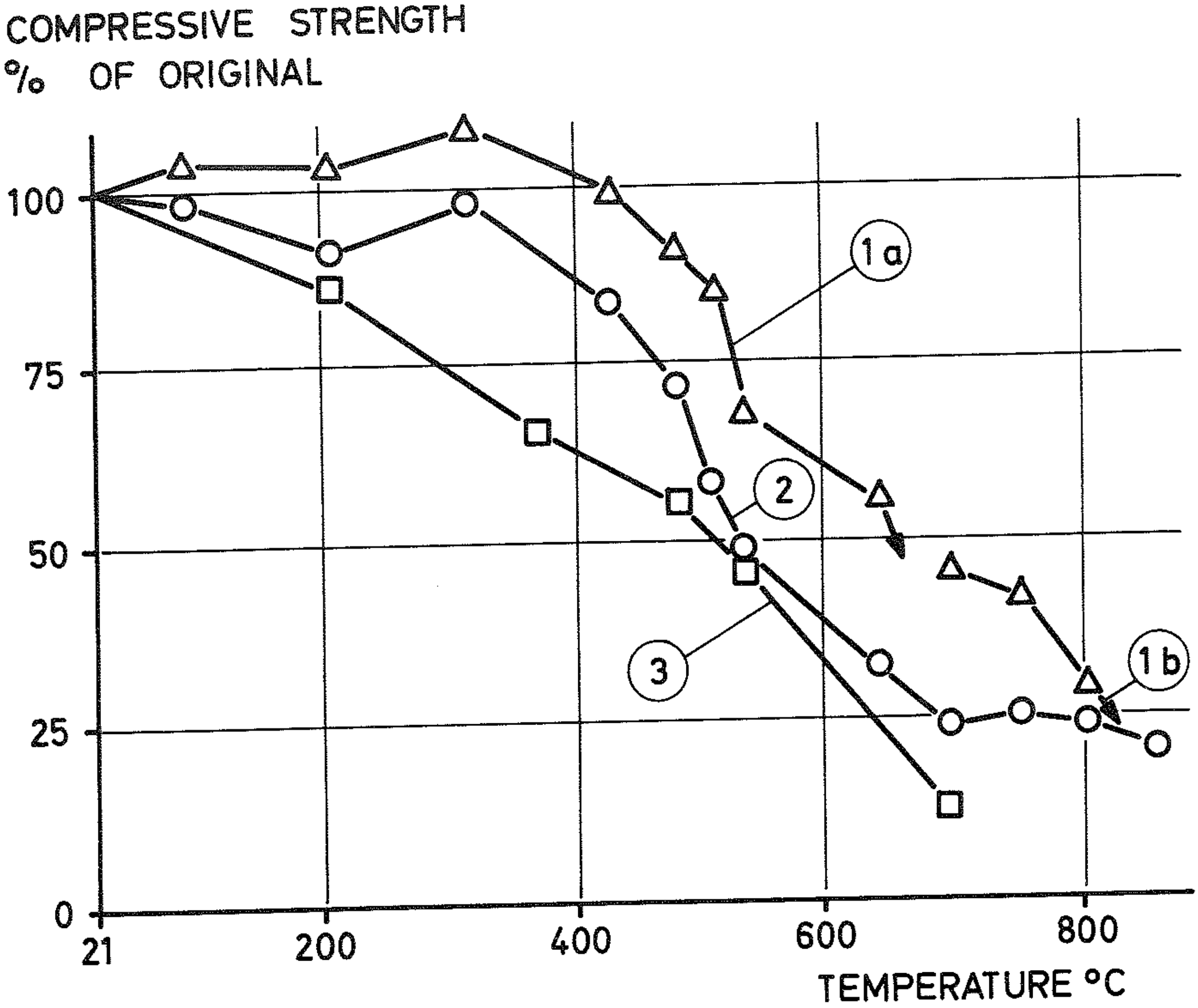


FIG. 23 Effect of temperature on compressive strength of siliceous aggregate concrete. The specimens, cylinders 76 mm in diameter, were cured in air until the relative humidity in the centre reached 75 per cent. Original compressive strength f'_c = 27.5 MN m^{-2}. During heating the maximum temperature difference within the specimen was limited to 83^o C.
Curve 1: Specimens stressed to 0.4 f'_c (1a) or 0.25 f'_c (1b) during heating and then tested in the hot state.
Curve 2: Nil-loaded specimens during heating and tested in the hot state.
Curve 3: Nil-loaded specimens during heating and cured 7 days in air before testing.
From Abrams (1971).

FIG. 24 has been reproduced from a publication by Thelandersson (1972) regarding the tensile strength at elevated temperatures, in order to illustrate the relative decrease in the tensile strength (determined through split-cylinder test) with increasing temperature, and at the same time it is compared with temperature dependence of the compressive strength determined by Weigler and Fischer for a comparable concrete. Even the aggregate, here, is composed of a material rich in quartzite and the water-cement ratio lies in the vicinity of 0.60. The Figure shows a somewhat stronger reduction in tensile strength than the compressive strength at elevated temperatures.

The deformation characteristics of concrete at elevated temperatures, at the present, is poorly studied. This fact is largely due to the complicated nature of the accelerating shrinkage and creep phenomena at elevated temperatures. An isolated illustration of deformation properties of concrete is shown in FIG. 25 which is reproduced from an investigation by Weigler and Fischer (1968). The Figure shows the linear expansion of concrete subject to 0, 1/6, 1/3 and 1/2 of the compressive ultimate load at room temperature, when heated to 600^{o}C at a rate of 120^{o}C/h and cooled with a rate of 120^{o}C/h after keeping the temperature at the constant value of 600^{o}C for 3 hours. It should be noted that already at half of the ultimate load the thermal expansion is completely neutralized by the intensive creep induced by temperature, and that large residual deformations develop in the specimens loaded in compression following heating and subsequent cooling.

4.2 MAIN TESTS

The experiment illustrated below embraces 5 test series with characteristics according to TABLE 2. Within every test series, the table gives information regarding how fire duration, fire load, opening factor, concrete composition, age of specimen and level of the vertical load are varied.

In series A1, which studies the influence of variations in fire process characteristics, 10 tests were performed. The opening factor, $A\sqrt{H}/A_t$, was varied within the range of 0.01 - 0.04 m$^{1/2}$ and the fire load, q, within the range 60 - 480 Mcal per square meter of enclosing area of the fire compartment. The corresponding fire duration is varied in the range 1 - 8 hours. The characteristics of the curve showing the gas temperature as a function of time in a complete fire process determined by using the opening factor $A\sqrt{H}/A_t$ and the fire load, q, were selected from the diagram in FIG. 26 and TABLE 2 (Magnusson et al. 1970). However for a fire duration or heating phase exceeding 2 hours a different cooling phase was selected. It corresponds to a temperature decrease of 10^{o}C/min which is in

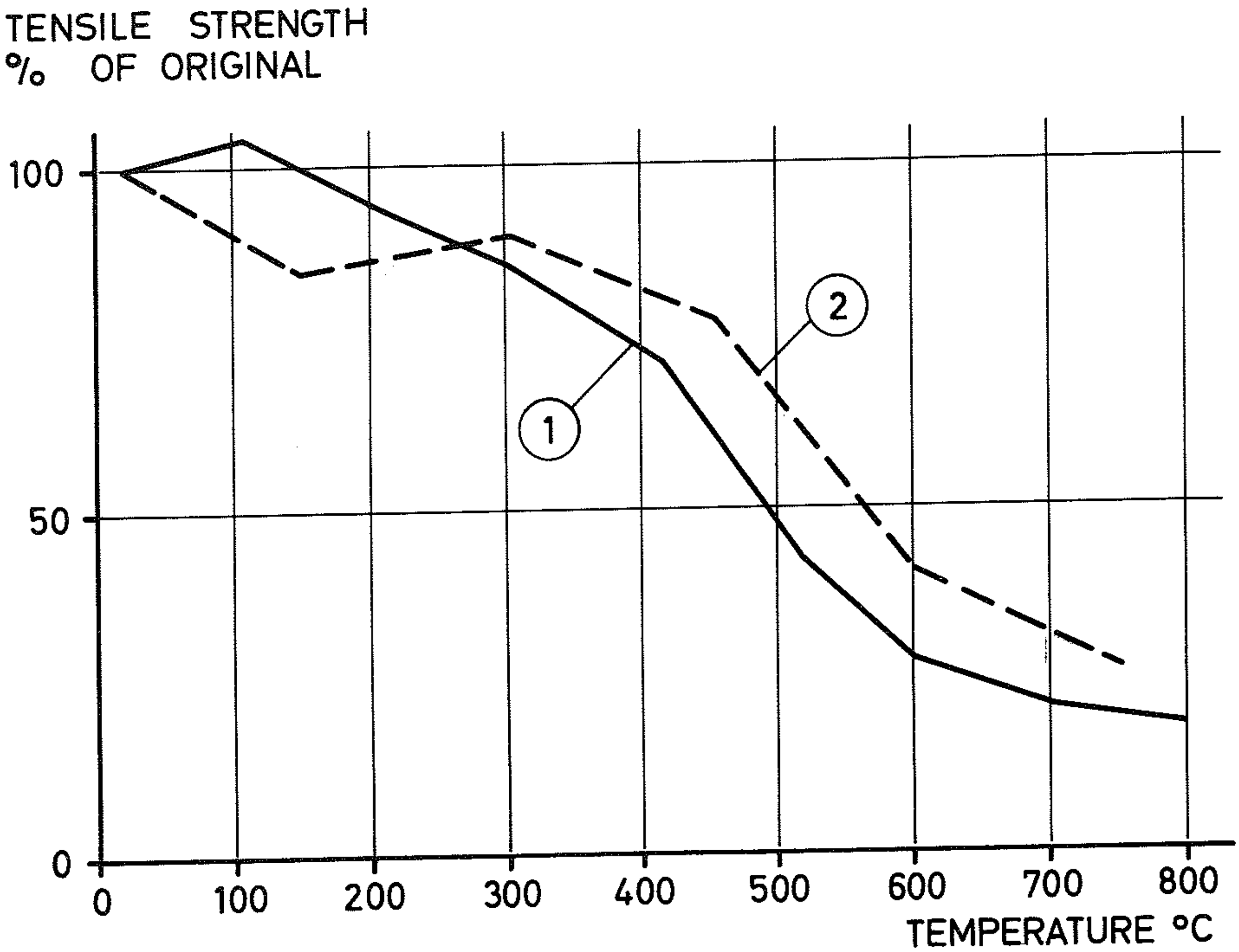


FIG. 24 Comparison between tensile strength and compressive strength.
Curve 1: Split-cylinder strength.
Curve 2: Compressive strength.
From Thelandersson (1972).

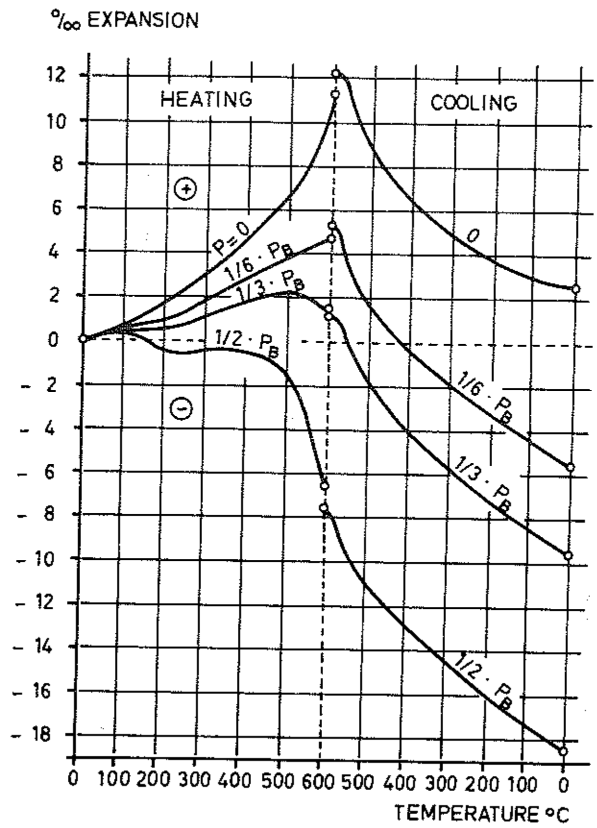


FIG. 25 Thermal expansion of concrete specimens subject to various stress levels at a heating and cooling rate of 120^{0} C · h^{-1}. The temperature was kept at 600^{0} C for 3 hours. P_B = Original compressive strength. From Weigler & Fischer (1968).

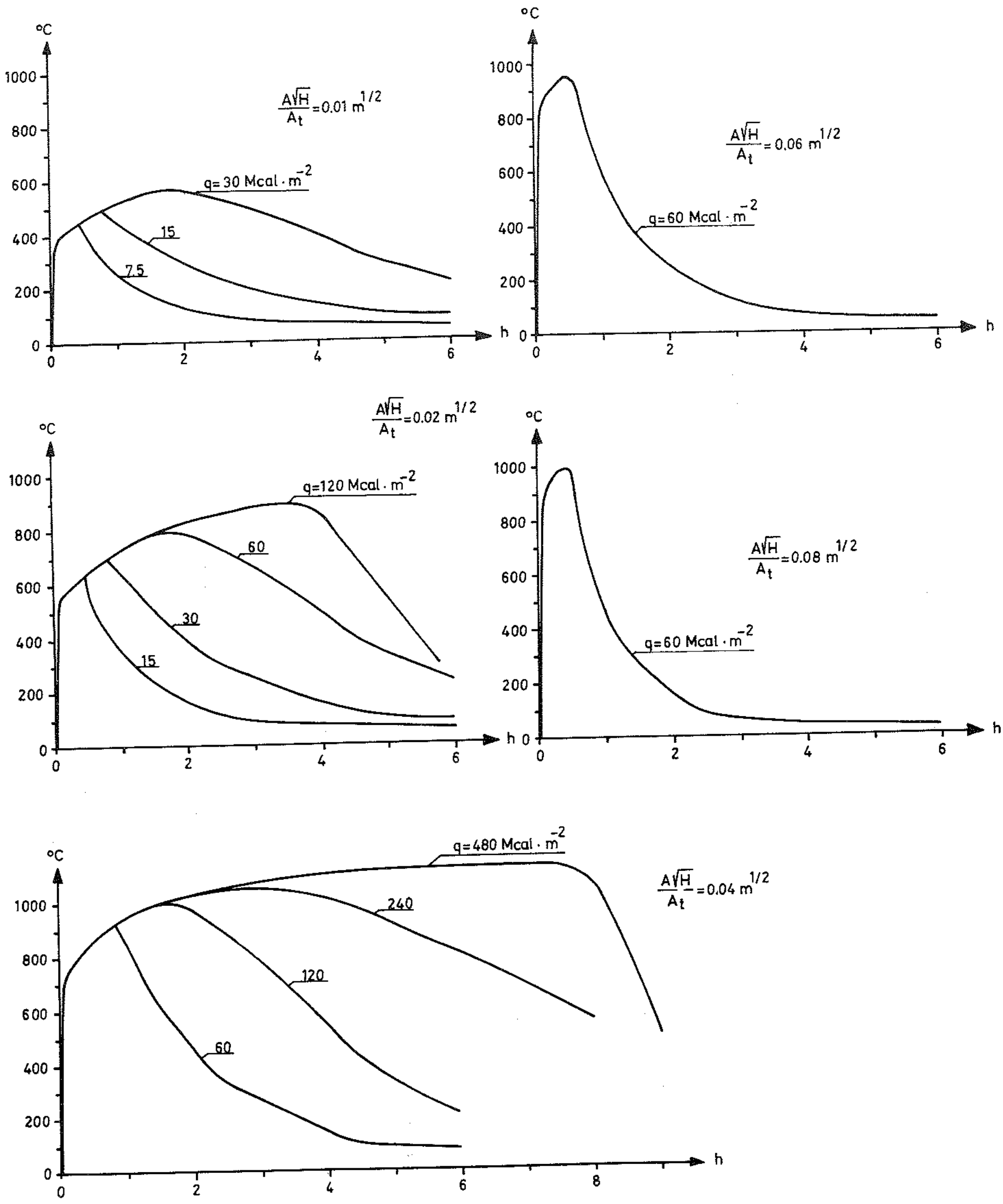


FIG. 26 Diagrams of the gas temperature-time curves at the opening factors $A\sqrt{H}/A_t$ = 0.01, 0.02, 0.04, 0.06 and 0.08 $m^{1/2}$ used in the investigation.
From Magnusson & Thelandersson (1970).

TABLE 2

Characteristics of accounted tests

Test series A1

Test No.	Fire duration h	Fire load $Mcal/m^2$	Opening-factor $m^{1/2}$	Water-cement ratio	Amount of cement-paste ℓ/m^3	Testing age months	Load P/P_{all}
1, 3 o 8	4	60	0.01	0.63	277	2	0
2, 4, (9), 11	4	120	0.02	"	"	2,,,6	0
5	1	60	0.04	"	"	2	0
6	2	120	0.04	"	"	2	0
7	4	240	0.04	"	"	2	0
10	8	480	0.04	"	"	5	0
Test series A2							
1	4	60	0.01	0.63	277	1	0
2	2	"	0.02	"	"	1	0
3	1	"	0.04	"	"	1	0
4	2/3	"	0.06	"	"	1	0
5	1/2	"	0.08	"	"	1	0
6	1/2	7,5	0.01	"	"	1	0
7	1	15	0.01	"	"	1	0
8	2	30	0.01	"	"	1	0
9	1/2	15	0.02	"	"	1	0
10	1	30	0.02	"	"	1	0
Test series B							
1	2	120	0.04	0.63	277	3	0
2	"	"	"	0.52	"	3	0
3	"	"	"	0.77	"	3	0
4	"	"	"	0.63	"	1	0
5	"	"	"	0.77	"	1	0
6	"	"	"	0.52	"	1	0
7	"	"	"	0.52	"	1/2	0
8	"	"	"	0.77	"	1/2	0
9	"	"	"	0.63	"	1/2	0
Test series C							
1	2	120	0.04	0.63	277	1	0
2	"	"	"	"	257	1	0
3	"	"	"	"	296	1	0
4	"	"	"	"	277	3	0
5	"	"	"	"	257	3	0
6	"	"	"	"	296	3	0
7	"	"	"	"	277	4	0
8	"	"	"	"	257	4	0
9	"	"	"	"	296	6	0
Test series D							
1	1/2	60	0.08	0.63	277	1	1/4
2	"	"	"	"	"	1	1/2
3	"	"	"	"	"	1	3/4
4	"	"	"	"	"	1	1
5	1	60	0.04	0.63	277	1	1/4
6	"	"	"	"	"	1	1/2
7	"	"	"	"	"	1	3/4
8	"	"	"	"	"	1	1
9	4	60	0.01	0.63	277	1	1/4
10	"	"	"	"	"	1	1/2
11	"	"	"	"	"	1	3/4
12	"	"	"	"	"	1	1

accordance with recommendations by Svensk Byggnorm 67 (Swedish Building Standard 67) when the cooling is unknown. Rigourously, this procedure, using 10^{o}C/min as the cooling rate, is an underestimation of unprevented fire processes of long duration. However, there is little probability that fires with durations more than two hours are allowed to develop undisturbed. This fact justifies the above mentioned choice of cooling rate for fires of this category. For all the tests in series A1, water-cement ratio is 0.63 and the quantity of cement-paste amounts to 277 l per cubic meter of concrete. Unfortunately, the age of plate strips on testing did not turn out to be the same, as desired. This was due to intermittent disturbances arising in connection with adjustment and successive modification of the test apparatus. Thus the prearranged time schedule was postponed at several occasions. In test series A1 which was the first to be performed, the results were not particularly satisfactory in some cases. This can be attributed to the rather large friction in pulleys which characterize the construction of the test apparatus in the preliminary phase of the investigation (cf Section 3). However, after adjusting and treating the test results, some useful qualitative information were obtained.

Series A2 is a direct complement to the test series A1. Plate strips in both of the series have the same water-cement ratio and the same cement-paste quantity. In series A2 some fires of short duration were performed. The fire duration was only 0.5 hours and the corresponding opening factor was varied in the range of 0.01 - 0.08 m$^{1/2}$. The age of all the plate strips was 1 month at the time of testing.

In test series B only concrete composition and age of plate strips prior to testing were varied, whereas the other parameters were kept constant. The tests were performed for the same temperature-time curve of the fire compartment, namely that corresponding to an opening factor of 0.04 m$^{1/2}$ and a fire load equal to 120 Mcal/m^{2}. This renders the corresponding fire duration which is 2 hours. Concrete composition was varied through changing the water-cement ratio - 0.52, 0.63 and 0.77 - while cement-paste quantity was kept constant. Age of the plate strips was varied from 1/2 to 3 months. The purpose of the test series is to find out which influence variations in water-cement ratio and age of specimen may have on the structural behaviour of concrete plate strips during fire, and thus get an idea of the significance of short-time shrinkage and short-time creep due to increased temperature and a pronounced temperature gradient.

The objective of test series C is the same as that for test series B. In this series cement-paste quantity and age of the plate strips were varied whereas the other parameters, namely fire process characteristics and water-cement ratio were kept constant.

As far as the most decisive influence of concrete composition (water-cement ratio and cement-paste quantity) is concerned, test series B and C together illustrate the influence of short-time shrinkage and creep, due to an increased temperature, on the structural behaviour of statically indeterminate reinforced concrete structures subject to fire.

Even in series B and C, some tests did not show up successfully due to the mentioned problem in an initial phase of the investigation, namely, friction in the pulleys was so large that an interpretation of the results was difficult to make.

The objective in common in test series A1, A2, B and C is a pure study and determination of the thermal bending moment which is imposed on a concrete plate strip restrained against rotation at both ends during an unilateral fire exposure. As far as possible, the test series comprise a continuous survey of the temperature-time-field, bending moment variations, and a study of deflection of plate strips with no vertical loading in a complete fire process. This is achieved by varying the fire characteristics, concrete composition and age of specimen prior to fire exposure. Test series D indicates the introduction to an extended study which also includes the influence of a vertical loading, with varying levels of loading, on the structural behaviour of the statically indeterminate concrete plate strips unilaterally exposed to fire. The vertical loading was applied through two symmetrically located concentrated loads 0.76 m apart from each other. Four loading levels were investigated namely, 1/4, 1/2, 3/4 and 1/1 of P_{all}, where P_{all} denotes the allowable load according to Swedish concrete standards. In this experiment P_{all} amounts to 1.60 Mp. In relation to the theoretical ultimate value, $P_{all} \approx 1/2\ P_{ultimate}$. In the test series, besides the vertical level of loading, the fire process has also been varied with a constant fire load, q = 60 Mcal per m^2 of the bounding area. Opening factor $A\sqrt{H}/A_t$ has been varied in the range of 0.01 - 0.08 $m^{1/2}$, corresponding to fire duration in the range 1/2 - 4 hours. The other parameters, namely concrete composition and age of the plate strips have been constant in the series. The values of these parameters were such that a direct comparison between the nil-loaded (the series A1, A2, B and C) and the vertically loaded plate strips (series D) has been possible.

4.3 COMPLEMENTARY DETERMINATION OF RESIDUAL STRENGTH AND RESIDUAL BENDING STIFFNESS

As a complement to the main experiments mentioned above, another experiment was performed on the plate strips. The purpose of the experiment was to determine the residual strength and the residual bending stiffness of the plate strips which had previously been exposed to fire, tested, and allowed to cool. For this purpose the strips were simply supported and loaded with two symmetrically located concentrated loads 0.8 m apart. During the whole loading process, the deflections in the mid-section and 1/4-section and curvature between the two concentrated loads with the load increment of 100 kp per concentrated load, were measured. These experiments were performed partly on plate strips which, in relation to the main experiment, had the right side up and partly for those which were turned upside-down. The objective in doing so was to determine the residual bending stiffness and the residual strength for loadings which render either positive or negative bending moment in the plate strips.

5. ILLUSTRATED DESCRIPTION AND ANALYSIS OF THE TEST RESULTS

5.1 OUTLINE OF THE ANALYSIS

Description of the test results is made in the following way: first a detailed general picture of the structural behaviour of the fire-exposed concrete plate strips subject to an isolated test is presented and thereafter the test results are summed up and analysed through examples.

The presentation does not claim to be complete. It only intends to illustrate some rather important test results obtained in the different test series. Furthermore the presentation intends to explain the principal structural behaviour of the plate strips during fire exposure. The analysis includes parallel interpretations of the results. As far as the zero-loaded plate strips are concerned most of the results are to be found in test series A2 with certain comparisons with the results from series A1, whereas in series B and C only some principal and rather important results and conclusions are accounted. Test series D, which is an introduction to a larger investigation, is here accounted only in brief consisting of an analysis of the most important results. Besides, an illustrated analysis and interpretation of the results belonging to the main experiment, the residual strength of the plate strips obtained from some complementary tests is presented.

5.2 DETAILED EXPLANATION OF RESULTS CORRESPONDING TO A SINGLE ISOLATED TEST

Test No. 3 in test series A2 is selected as an illustrating example showing the detailed general account of the structural behaviour of concrete plate strips subject to fire. This test is characterized according to TABLE 1 by the fact that the plate strips are not loaded. Furthermore they are influenced by the temperature-time process of the fire compartment characterized by FIG. 26 and TABLE 2 in which the fire load is 60 Mcal/m^2 and the opening factor 0.04 m$^{1/2}$ and the corresponding fire duration 1 hour. The strip was tested at the age of 1 month and the concrete composition was characterized by a water-cement ratio of 0.63 and a cement-paste quantity amounting to 277 l per cubic meter. The actual desired temperature-time curve of the fire cell together with the time curves of the furnace temperature registered by thermocouples, located at measuring points 8, 16, 24 (cf FIG. 10), is shown in FIG. 27. Already, after 15 minutes the furnace temperature reaches the value 750 - 800°C. Due to the high heat resistance, this rapid increase in temperature for the actual type of construction, and comparable structures, result in a very steep temperature gradient in the external layer close to the fire influenced area of the plate strip, whereby the external layer is affected by a large thermal expansion despite the slow response of the other parts of the

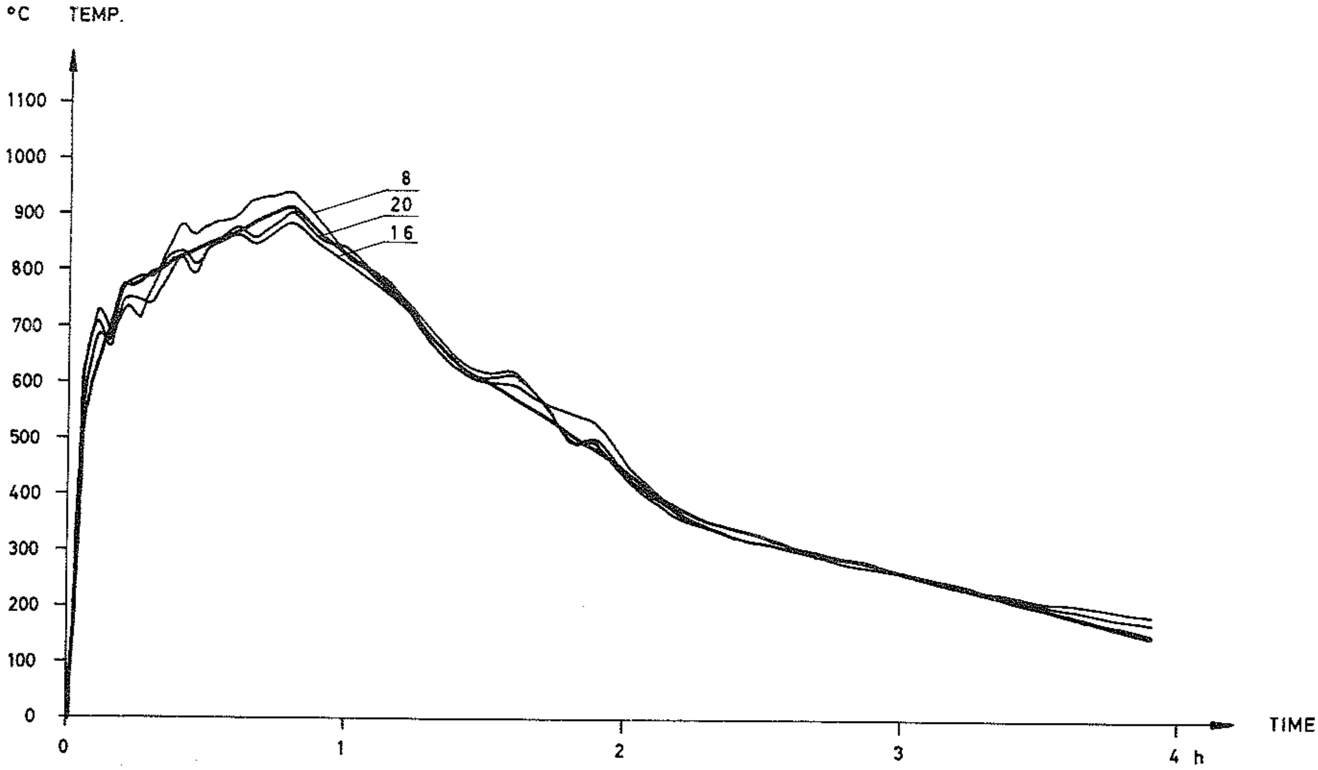


FIG. 27 Actual desired temperature-time curve and the time curves of the furnace temperature measured at the points 8, 16 and 24 for test A2:3 characterized by the fire load 60 Mcal per square meter of the enclosing area and the opening factor 0.04 $m^{1/2}$.

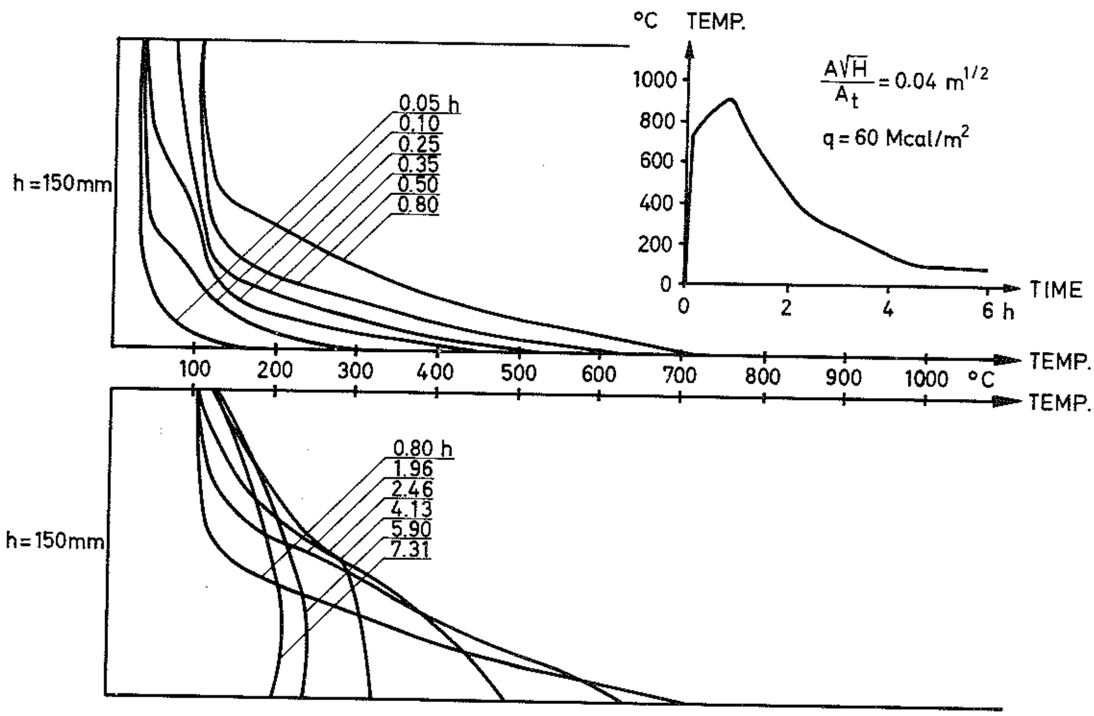


FIG. 28 The variation of temperature gradient during fire exposure for test A2:3.

cross-section which in a preliminary phase of the fire exposure are uneffected by heating. As a consequence, large compressive stresses arise within this zone which is strongly influenced by heat, whereas in other parts of the cross-section there will be tensile stresses. The large thermal compressive stresses may lead to spalling of the external layer in the beginning of a fire exposure. This, however, did not take place in this experiment. Variation of the temperature gradient belonging to the mid section of the concrete plate strips subject to fire as a function of time is illustrated in FIG. 28.

If the concrete strip subject to fire had been simply supported, the unilateral heating would have resulted in the curvature of the plate strip to be directed downwards. The structure with the actual support which is fixed against rotation and free against longitudinal displacement leads to negative moments at the supports. The magnitude of these moments depend on the temperature gradient, dimensions of the plate strip and strength and deformation properties of the actual material subject to the continuously varying fields of temperature. In this way tensile bending stresses arise in the upper cross section of the plate strip leading to formation of tensile bending cracks, in case these stresses reach the tensile bending strength of concrete. In this actual test, such crack formation appeared after the strip had been exposed to fire for 12 minutes.

The measured time curves of the negative moments at the supports induced by heating are presented in FIG. 29. From the curves, it can be observed that the increase of bending moments during the preliminary phase of fire exposure and the increase of fire-gas temperature are in harmony with each other. After 45-50 minutes of heating, about the same time as the maximum furnace temperature of 900°C is attained, the bending restraint moment reaches its maximum value which is approximately 1.15 Mpm. In the actual test, the bending restraint moments at both supports follow the same pattern during the whole heating phase.

During the cooling phase, the negative bending restraint moment decreases continuously. About three hours after exposing the strip to fire, the moment drops to zero followed by a successively increasing positive bending moment at the supports fixed against rotation. After the plate strips have completely cooled down to ordinary room temperature, there remains a positive residual bending moment, about 0.8 Mpm in magnitude, in the structure. These residual bending moments are primarily caused by large residual deformations of shrinkage and creep rendered during the fire exposure at those parts of the concrete strips most powerfully heated.

A comparison between FIG. 27, indicating time-temperature curve of the furnace, and FIG. 29, illustrating the bending restraint moment as a function of time, shows that the shrinkage and creep effects resulting from a powerful heating of short duration leads to the appearance of positive restrained moments in the plate strips. It is to be noticed that these moments arise at a time when the furnace temperature has hardly dropped to 250 - 300^{o}C, which means that there are significant temperature gradients in the structure attempting to cause deformations compatible with the negative bending restraint moment.

The pattern of the bending restraint moment at both supports are in less concordance during the cooling than the heating period. The possible explanation is the non-uniform crack distribution.

Had the heating and stiffness been identical in every vertical section of the strips located between the supports at any time during the fire exposure, the bending restraint moment would have been constant and the deflection zero at every section between the supports. The disturbances in such an ideal state are caused by nonuniform distribution of furnace temperature, crack formations in some isolated sections and the somewhat varying material characteristics possessing varying sensitivity in different sections as far as the decrease in strength and stiffness are concerned. Consequently, the actual fire exposure gives rise to deflections in the concrete strips. The variation of these deflections as a function of time is illustrated in FIG. 30 for mid-section and 1/4-section of the strip. It is observed from the curves that the deflection, to begin with, increases rapidly by increasing the temperature. After approximately 10 minutes, there will be a decrease in the deflection rate due to the fact that at this time the initial moisture begins to evaporate, leading to a local retardation in heating. A successive decrease of the slope of the temperature gradient by increasing the heating time results in a rather slow increase of the deflection. These circumstances are further illustrated by the temperature gradients in FIG. 28. A decrease in deflection starts again somewhat before the maximum value of the furnace temperature is attained. The shrinkage and creep deformations arising in concrete at elevated temperatures of short duration, as mentioned above, result in a vertical displacement upwards, during the cooling down period. There will also arise a residual upward deflection of approximately 2 mm after cooling down completely to ordinary room temperature. The deformation process is in good agreement with the time process of the bending restraint moment imposed on the strip.

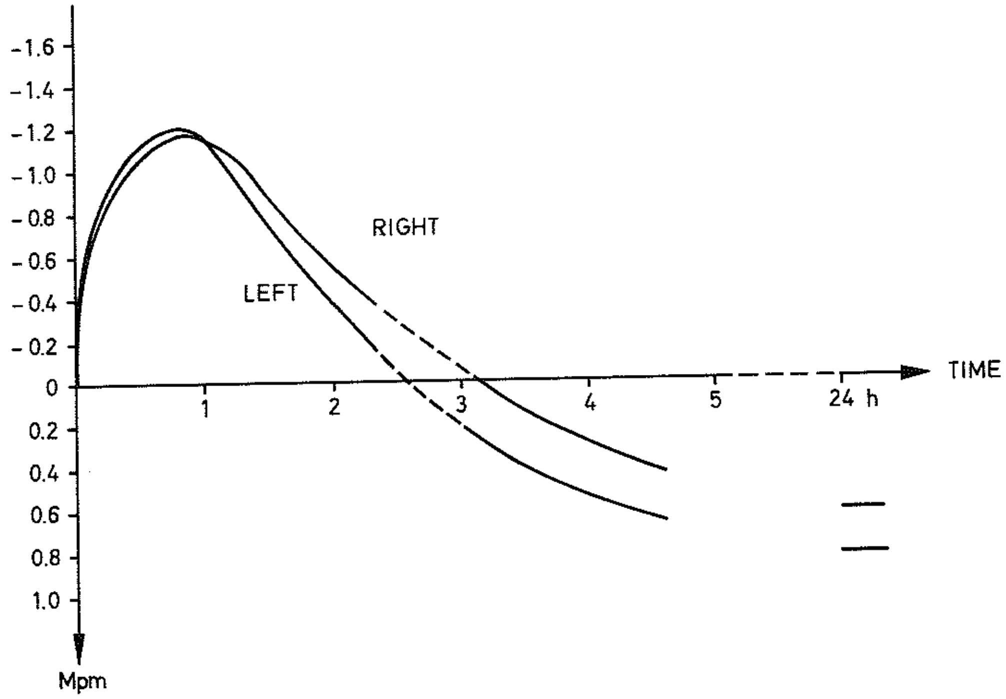


FIG. 29 Variation in bending, restraint moments with time at the left and right supports for test A2:3.

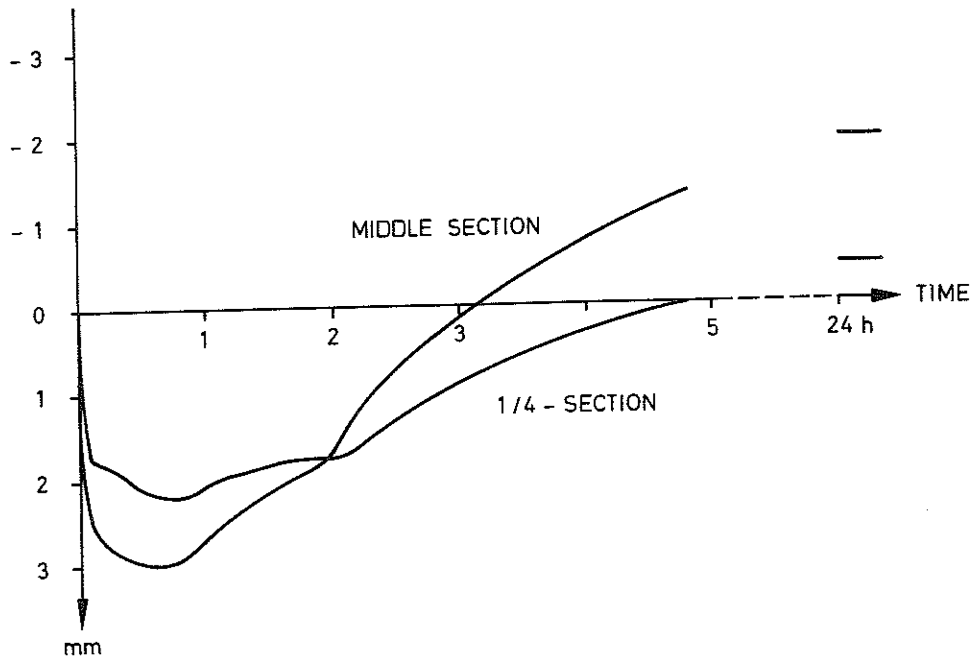


FIG. 30 Deflection in the middle- and 1/4-sections during the fire test A2:3.

The temperature distribution in the reinforcement at different times during a fire is of decisive importance for the load-bearing capacity of a fire-exposed, reinforced concrete beam or plate. This fact is illustrated for the reinforcement at the bottom side of the strip in FIG. 31, in which the temperature distribution of the reinforcement is determined through thermocouples placed as shown in FIG. 10. The different curves thus obtained correspond to measurements performed at different times during the exposure. The maximum reinforcement temperature of 440^{0}C developed after 1.4 hours, that is 30 minutes after the furnace temperature attained its maximum value. Furthermore, the temperature distribution of the fire-exposed surface of the strip is shown in the same way in FIG. 32.

5.3 SUMMARY OF THE RESULTS OBTAINED FROM NIL-LOADED PLATE STRIPS

The tests listed in TABLE 2 and performed on zero-loaded concrete strips embrace those in series A1 and A2 studying the influence of varying fire process characteristics and those in series B and C studying the influence of varying concrete composition and age of the specimen prior to fire exposure. The influence of these factors on the bending restraint moment imposed on the strip during fire exposure is briefly illustrated below while some aspects concerning the deflection of the strips are explained in section 5.5.

In FIGS. 33-37 the time curves of the bending restraint moment and the corresponding time-curves of the furnace temperature for five tests with constant concrete composition and specimen age prior to fire exposure, are reproduced from test series A2. Keeping the fire load at a constant value of 60 Mcal/m^2 of the bounding area, the opening factor has been varied within the range 0.01 - 0.08 m$^{1/2}$ and the corresponding length of heating phase or fire duration varied within the range 4 - 1/2 hours. From the results, it may be observed that increasing the opening factor from 0.01 - 0.08 m$^{1/2}$ results in the change of the maximum negative bending restraint moment from -1.25 to -1.10 Mpm, while the time required to attain this value shifts from 4 to 1/2 hours which is in good agreement with the fire duration. For the actual concrete strips this relation is generally valid for fire loads having the order of magnitude 60 Mcal/m^2 or less. According to the time-curves, the bending restraint moment changes sign at 5.40, 5.00, 2.80, 2.60 and 2.20 hours after the start of the fire exposure with corresponding opening factors 0.01, 0.02, 0.04, 0.06 and 0.08 m$^{1/2}$ respectively. After complete cooling, there will be considerable positive residual bending moments in the strips.The test A2:2 exhibits the largest residual bending moment with a fire duration of 2 hours. The residual moments obtained

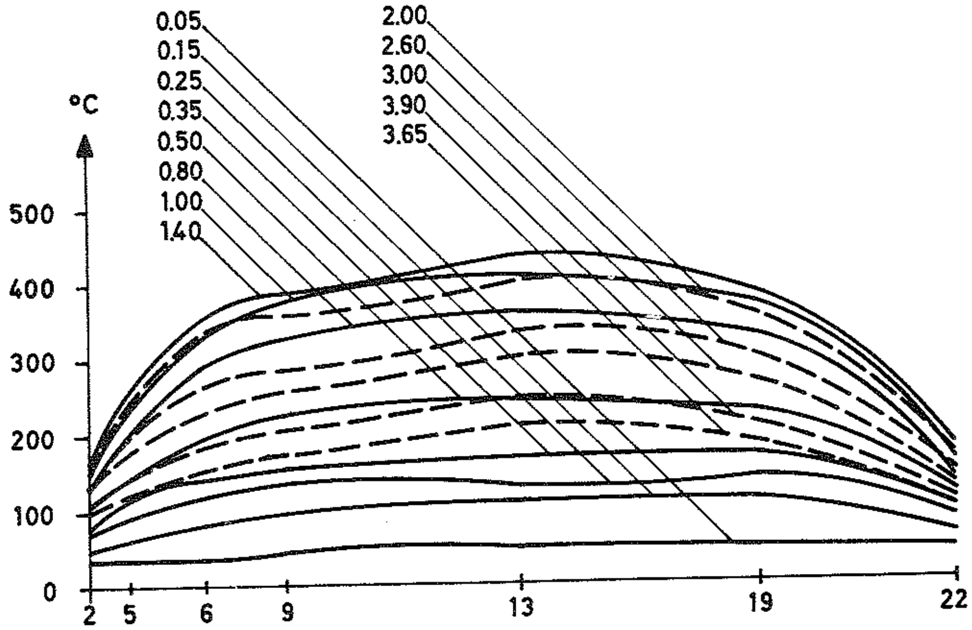


FIG. 31 Temperature distribution of the bottom reinforcement during fire exposure in the plate strip A2:3 determined by thermocouples 2, 5, 6, 9, 13, 19, and 22.

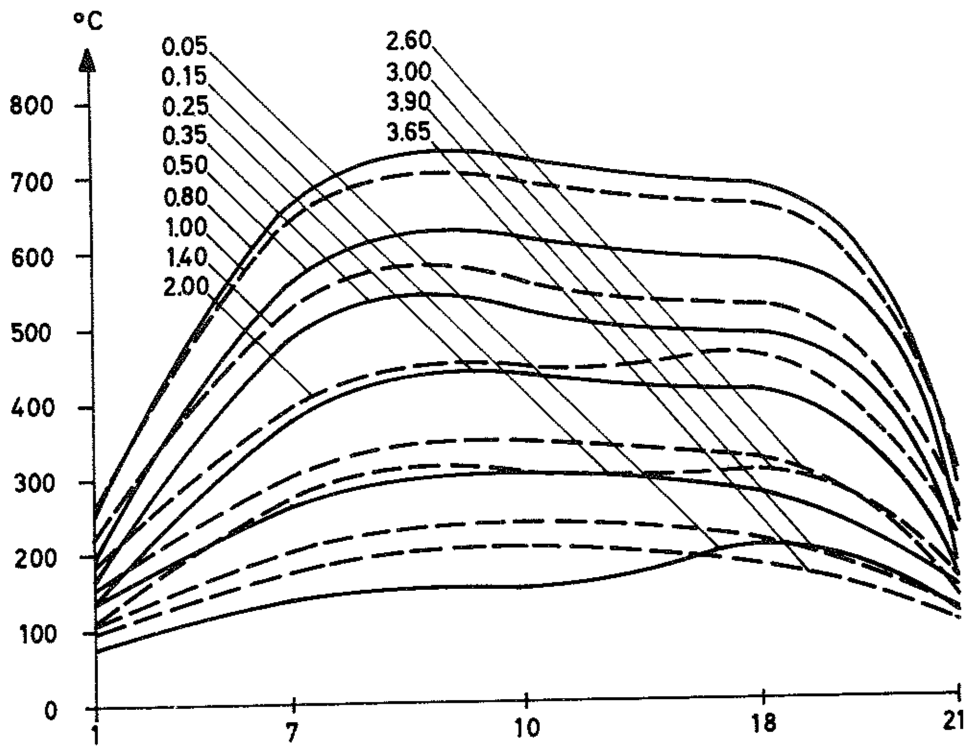


FIG. 32 Temperature distribution of the fire-exposed surface of the plate strip A2:3 determined by thermocouples 1, 7, 10, 18 and 21.

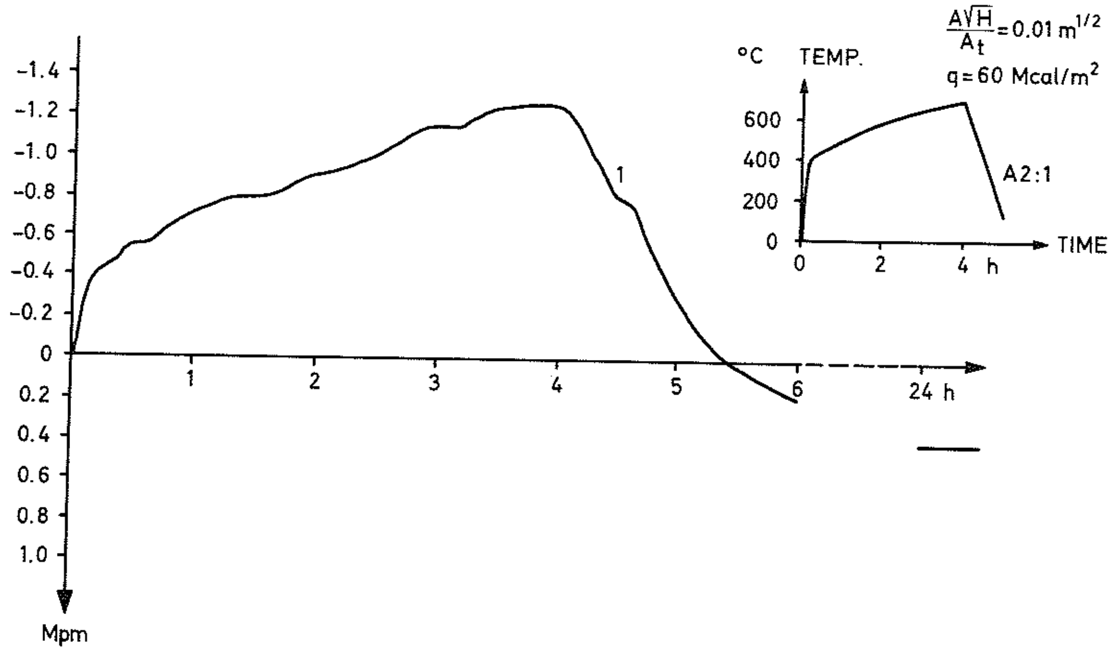


FIG. 33 The variation in bending, restraint moment during the fire test A2:1. The temperature-time curve is also shown.

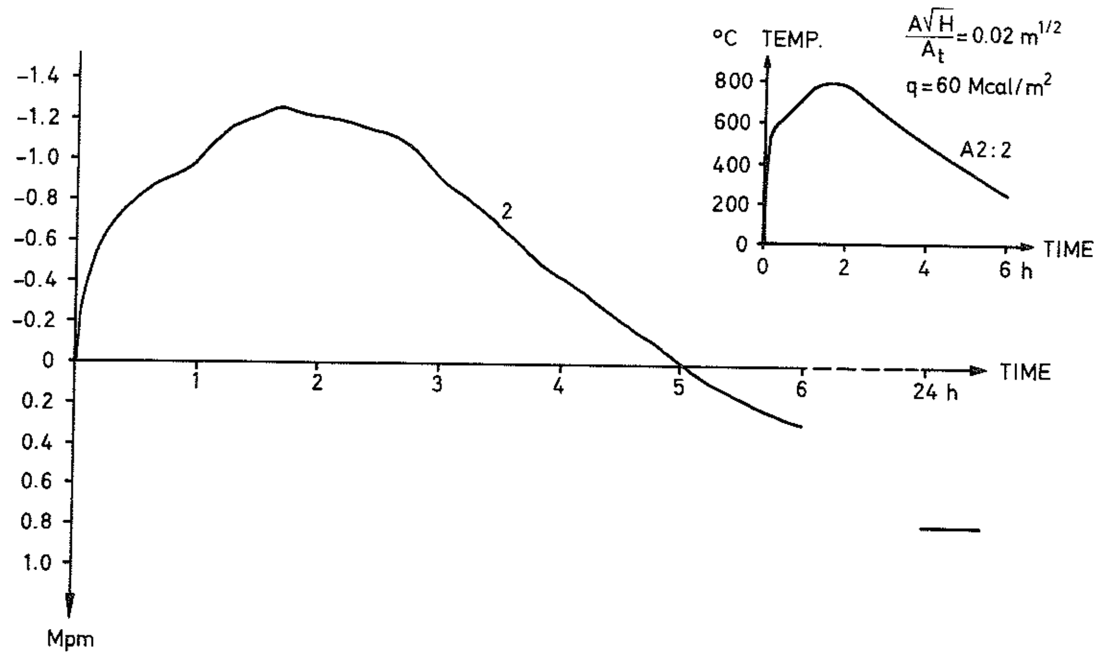


FIG. 34 The variation in bending, restraint moment during the fire test A2:2. The corresponding temperature-time curve is also shown.

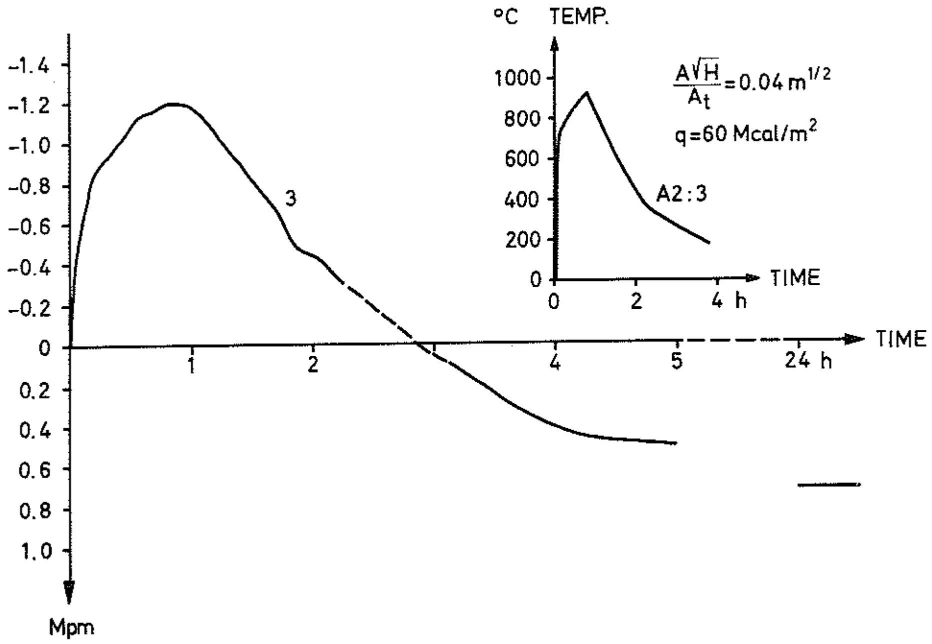


FIG. 35 The variation in bending, restraint moment during the fire test A2:3. Temperature-time curve is also shown.

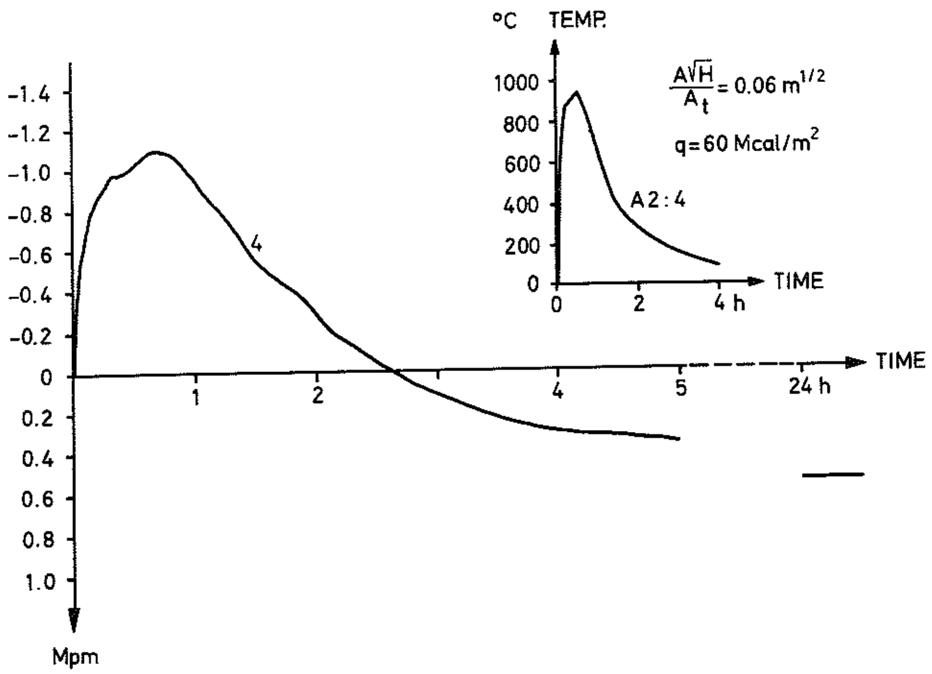


FIG. 36 The variation in bending, restraint moment during the fire test A2:4. Temperature-time curve is also shown.

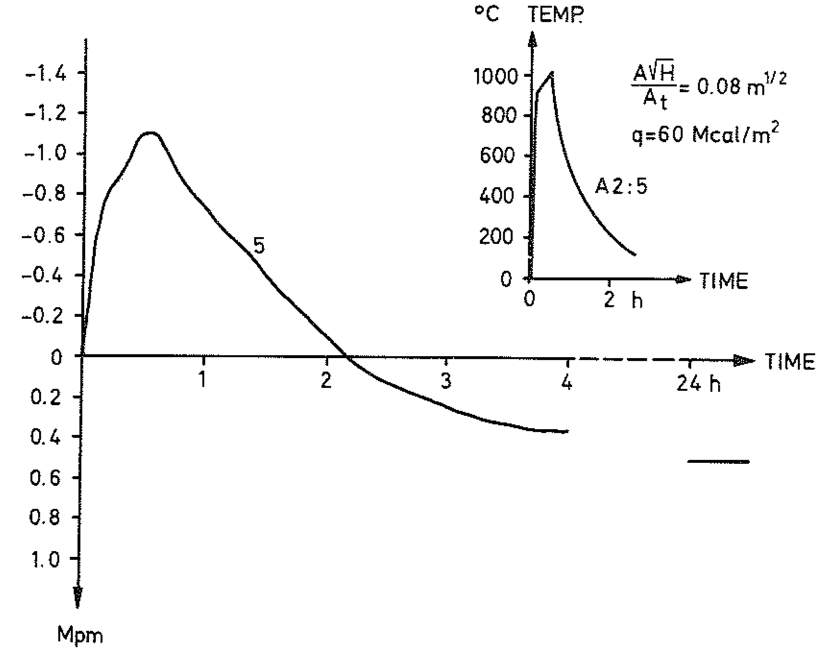


FIG. 37 The variation in bending, restraint moment during the fire test A2:5. Temperature-time curve is also shown.

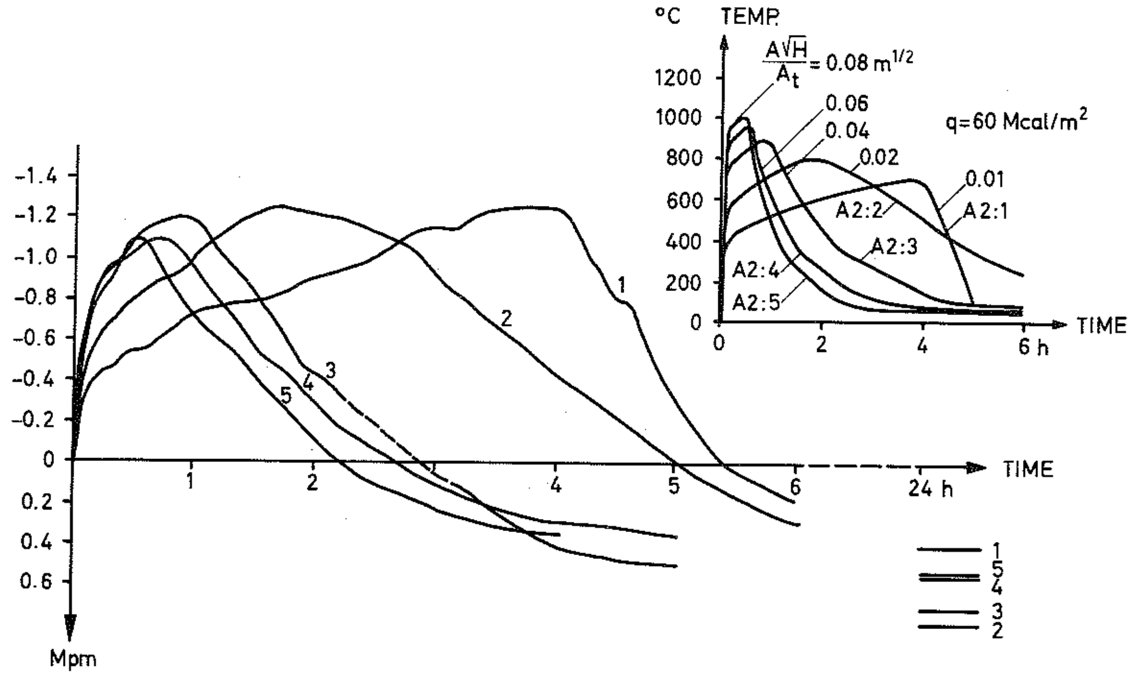


FIG. 38 The influence of fire process characteristics on bending, restraint moment shown by tests 1-5 in series 2, characterized by a constant fire load q = 60 Mcal/m^2 with varying opening factors 0.01 - 0.08 $m^{1/2}$. The corresponding temperature-time curves are also shown.

amount to 0.44, 0.80, 0.72, 0.55 and 0.50 Mpm with fire durations 4, 2, 1, 2/3 and 1/2 hours respectively. FIG. 38 illustrates the variation of bending restraint moments as a function of time for the five tests. In order to further illustrate how the maximum value of the bending restraint moment and its variation as a function of time are influenced by changes in fire process characteristics, the results of tests in which the fire load has been varied and the opening factor kept constant are summarized in FIGS. 39 and 40. The tests are included in the series A1 and A2. Thus FIG. 39 illustrates the bending restraint moments and furnace temperatures as functions of time for four tests in which the opening factor is kept at a constant value of $A\sqrt{H}/A_t = 0.01\ m^{1/2}$, with fire durations of 1/2, 1, 2 and 4 hours, and the corresponding fire load of 7.5, 15, 30 and 60 $Mcal/m^2$ bounding area respectively. From the Figure it may be observed that the corresponding maximum negative bending restraint moments obtained in the four tests, increase successively by increasing fire load and amount to -0.62, -0.75, -0.92 and -1.25 respectively. These values for the maximum negative moments are attained approximately at the same time as the cooling phase starts, that is 1/2, 1, 2 and 4 hours respectively after the start of the fire exposure. According to the Figure, the corresponding times at which the moments change sign are 1.6, 3.5, 7 and 5.40 hours respectively. The last mentioned value corresponds to the rather rapid linear cooling phase of 10^oC/minute. In the tests with the fire loads 7.5, 15 and 30 $Mcal/m^2$, the positive residual bending moments obtained are relatively small and vary within the range 0.15 - 0.20 Mpm. The reason is predominately the rather low fire temperatures used in these combinations of fire loads and low opening factors, or, the slow combustion rate which has resulted in relatively small shrinkage and creep deformations in the concrete subject to fire. In a similar test using the fire load 60 $Mcal/m^2$, the positive residual bending moment obtained is 0.40 Mpm, which is considerably larger.

Similar to the tests illustrated in FIG. 39, the curves in FIG. 40 show the bending restraint moments as a function of time for four tests in which the opening factor has been kept at a constant value of $A\sqrt{H}/A_t = 0.02\ m^{1/2}$ with fire durations 1/2, 1, 2 and 4 hours and the corresponding fire loads 15, 30, 60 and 120 $Mcal/m^2$ bounding area respectively. From the Figure, the corresponding negative bending restraint moments increase with increasing fire load, amounting to -0.75, -0.95, -1.25 and -1.40 Mpm respectively. The times, at which the moments attain their maximum values for fire loads 15, 30 and 60 $Mcal/m^2$, agree approximately with the corresponding fire durations. For the fire load 120 $Mcal/m^2$, with fire duration of 4 hours, the maximum negative moment was attained already after 2.25 hours. By continuing the heating the moment, thereupon, successively decreased in magnitude due to continuous decrease in stiffness of the strip caused by crack formations in the upper

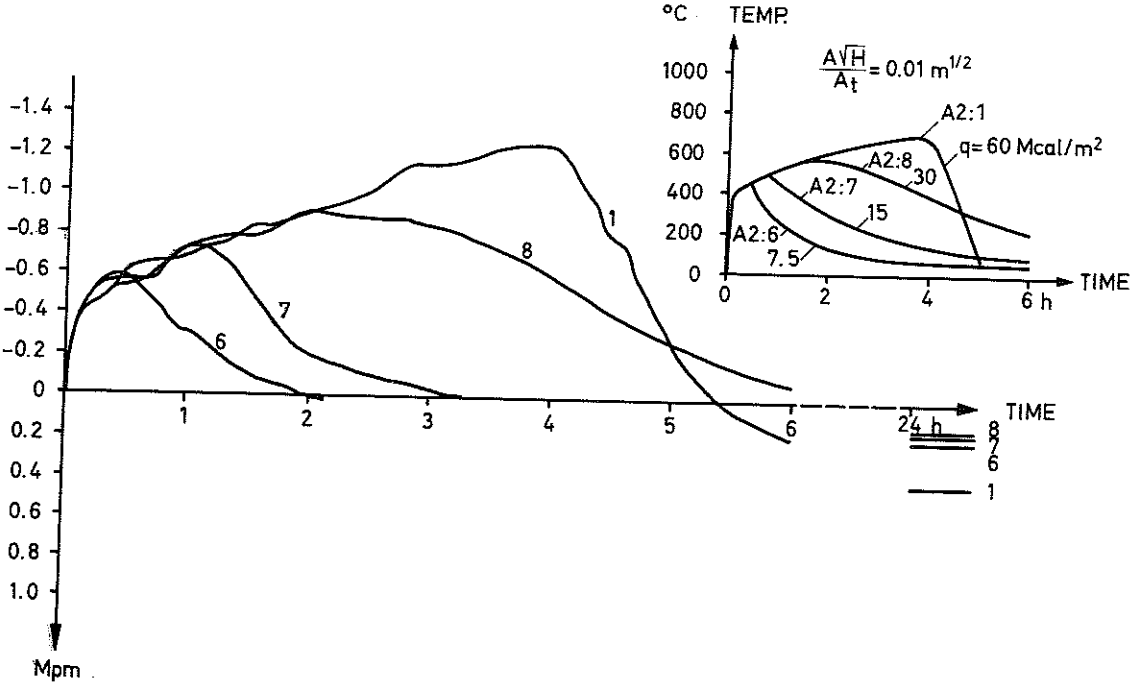


FIG. 39 The influence of fire process characteristics on bending, restraint moment shown by tests 2, 6, 7 and 8 from series 2, characterized by a constant opening factor 0.01 $m^{1/2}$ with varying fire loads within the range 7.5 - 60 $Mcal/m^2$ enclosing area. The corresponding temperature-time curves are as well shown.

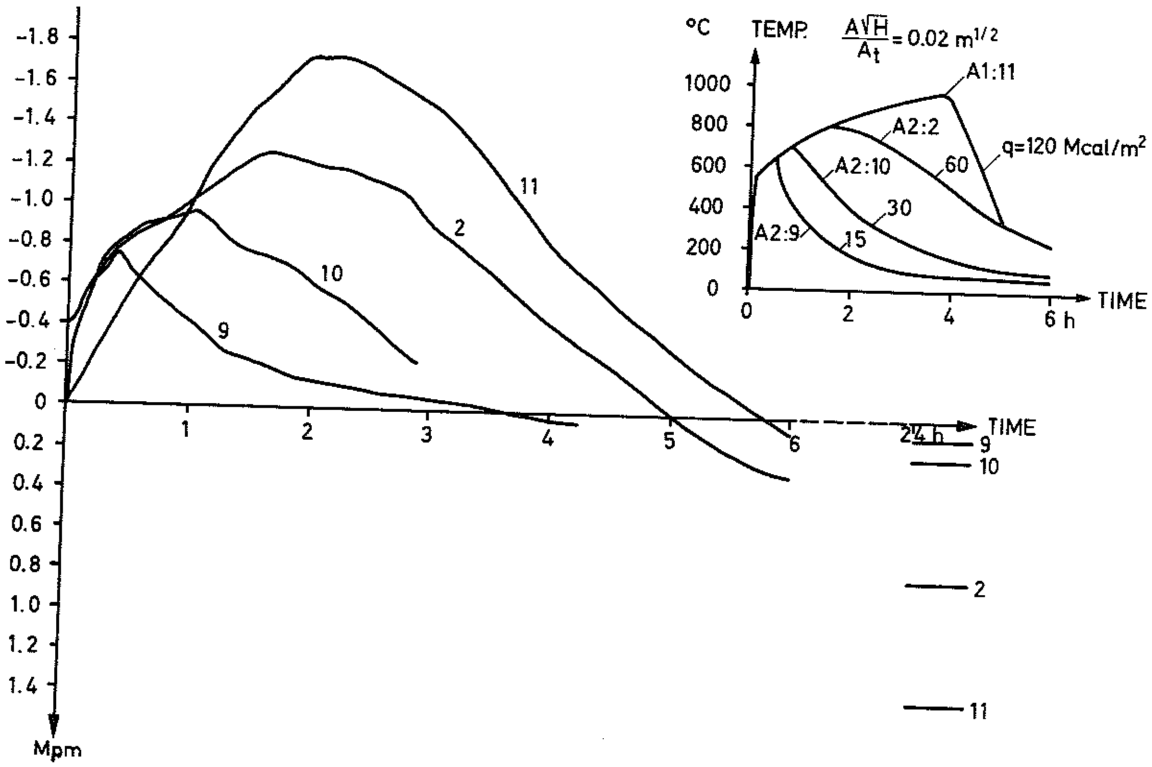


FIG. 40 The influence of fire process characteristics on bending, restraint moment shown by tests 2, 9 and 10 from series A2 and test 11 from series A1, characterized by a constant opening factor 0.02 $m^{1/2}$ with varying fire load within the range 15-120 $Mcal/m^2$ enclosing area. The corresponding temperature-time curves are as well shown.

part of the cross-sectional area, and by decrease of strength and stiffness in the lower cross-sectional area of concrete powerfully heated. After complete cooling to ordinary room temperature, the residual bending moments for the four tests amounted to 0.10, 0.20, 0.80 and 1.20 Mpm for the corresponding fire durations of 1/2, 1, 2 and 4 hours respectively.

In analysis of the results obtained from test series B, characterized by constant fire process characteristics, and varying water-cement ratio, no significant changes in the results could be observed (see TABLE 2). Thus, it follows that the influence of water-cement ratio and varying specimen age prior to fire exposure is not significantly noticeable within the acceptable precision range in the test series. In the test series, it has, however, been established that the time for the negative bending moment to attain its maximum value is delayed with increasing water-cement ratio, possibly due to increasing water evaporation and water escaping to the surface. The test series will continue with a test apparatus possessing better precision; therefore, some new conclusions are to be expected.

From the results obtained in series C, in which the cement-paste quantity is the significant variable factor, it can be observed that increasing cement-paste quantity results in larger positive residual bending moments whereas the time, at which the bending restraint moment changes sign, gets shorter. In these tests the maximum negative bending moment is around -1.5 Mpm, while the positive residual bending moment varies between 1.10 and 1.40 Mpm. Even in series C, as was the case for previous test series, varying the specimen age prior to fire exposure does not change the results appreciably within the given precision. However, it was noticed that more water left the specimens at an age of 1/2 months.

5.4 SUMMARY OF THE RESULTS OBTAINED FROM LOADED PLATE STRIPS

The tests belonging to series D listed in TABLE 2 embrace a preliminary study of a reinforced concrete strip subject to fire on one side and fixed against rotation at the supports, leaving the longitudinal deformation to take place freely. During the whole experiment, the strip was vertically loaded by two symmetrically placed concentrated loads denoted by $P_2 = P$ as shown in FIG. 6. With the necessary measures taken to prevent end-rotation, the vertical loads were applied to the strip one hour before the start of the heating phase and were allowed to continue loading the strip at a constant value during the whole fire process, including the time needed for the strip to completely cool down to ordinary room temperature. In the test series the fire load, concrete composition and age of the specimen prior to fire exposure were kept constant while opening factor, the corresponding fire duration and the vertical loading level, P, were varied. The influence of vertical loading on the structural behaviour of concrete strips subject to fire was studied for the cases P = 0 (test series A2), P = 1/4 P_{all}, P = 1/2 × P_{all}, P = 3/4 P_{all} and P = P_{all}. In these tests P_{all} = 1.60 Mp is the load which, according to Swedish concrete standards, is allowed to be applied to the actual strip which is fully restrained at both ends. As mentioned in section 4.1, $P_{all} \approx 1/2\ P_{ult}$, where P_{ult} is the theoretical ultimate load of the concrete strip.

FIG. 41 is intended to illustrate the results through curves describing the total bending restraint moment and the corresponding furnace temperature as functions of time for five tests characterized by the above mentioned vertical loading levels P = 0, 1/4, 1/2, 3/4 and 1/1 of P_{all} and constant conditions as far as the fire process (opening factor $A\sqrt{H}/A_t$ = 0.08 m1/2, fire load, q = 60 Mcal/m^2 bounding area and fire duration = 1/2 hours), concrete composition and age of the strips prior to fire exposure are concerned. From the Figure it may be observed that, for the loading levels 3/4 P_{all} and P_{all}, the strips attain their yield bending moment after 0.40 hours. For the lower loading levels, the magnitude of the maximum negative bending restraint moment of the strips don´t attain the yield bending moment and amounts to -1.10, -1.38 and -1.64 Mpm for loading levels 0, 1/4 and 1/2 P_{all} respectively. After cooling down to room temperature, the five tests rendered the residual bending moments amounting to +0.50, +0.35, +0.20, 0 and -0.45 Mpm for the loading levels 0, 1/4, 1/2, 3/4 and 1/1 of P_{all} respectively. It is remarkable to observe that the curve corresponding to loading 3/4 P_{all}, crosses the curve for P_{all} and ends at a value of the residual bending moment, lying between the corresponding values for loadings equal to 1/2 P_{all} and 3/4 P_{all}.

FIG. 42 describes the results of five similar tests performed with a significantly slower rate of combustion characterized through the opening factor $A\sqrt{H}/A_t$ = 0.01 $m^{1/2}$ and the corresponding fire duration of 4 hours with the fire load q = 60 Mcal/m^2 bounding area. In these tests, illustrated in FIG. 42, the negative yield bending moment of the strips, for loadings P_{all} and 3/4 P_{all}, is attained after 2 hours, and after 3.75 hours for the loading 1/2 P_{all}. For loadings 1/4 P_{all} and 0 the maximum negative bending restraint moments turn out to be -1.70 and -1.25 Mpm, which in time correspond to the fire duration of 4 hours. The residual bending moments, obtained after complete cooling, amount to 0.40, 0.30 and 0.10 Mpm for loadings 0, 1/4 P_{all} and 1/2 P_{all} respectively. The residual moments corresponding to loadings 3/4 P_{all} and P_{all} are missing.

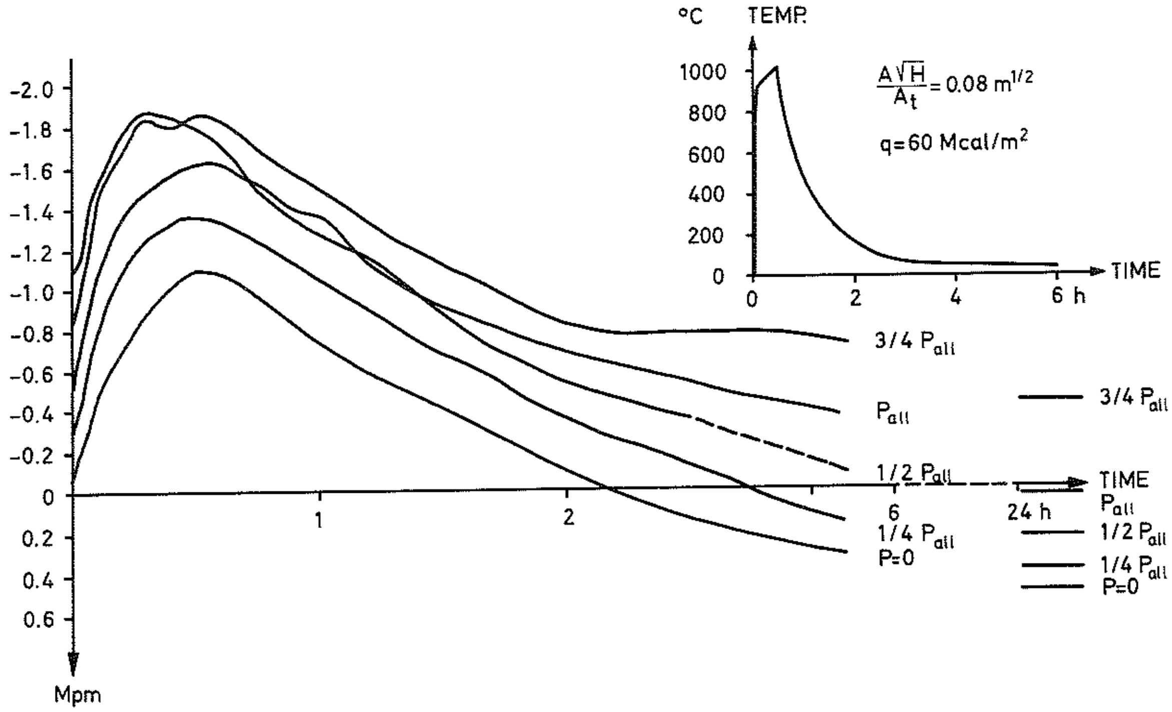


FIG. 41 Variation in bending, restraint moment with time of transversally loaded plate strips from fire tests D 1-4. The temperature-time curve is shown. P_{all} = the allowable load according to Swedish Concrete Standards.

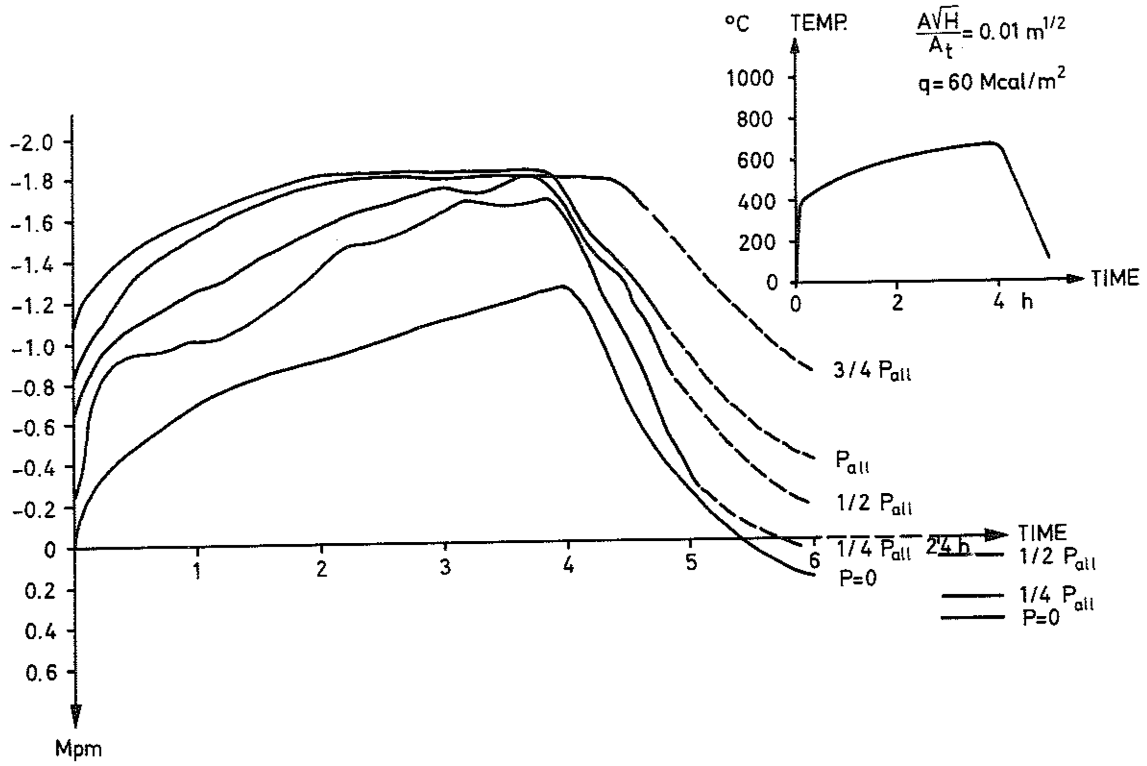


FIG. 42 Variation in bending, restraint moment with time of transversally loaded plate strips from fire tests D 9-12. Temperature-time curve is shown. P_{all} = = the allowable load according to Swedish Concrete Standards.

5.5 DEFLECTION PROCESS. SPALLING

FIG. 43 gives a complementary information on the structural behaviour of concrete strips subject to fire through curves showing the deflection as a function of time for tests 1-5 in series A2, namely, tests characterized by nil-loaded strips, constant concrete composition and specimen age prior to fire exposure, and varying fire process with constant fire load, q = 60 Mcal per square meter of the bounding area. The five deflection curves 1-5 correspond to the opening factors 0.01, 0.02, 0.04, 0.06 and 0.08 $m^{1/2}$, or, the corresponding fire durations 4, 2, 1, 2/3 and 1/2 hours respectively.

As describes in section 5.2, the deflection at any vertical crossectional area of the nil-loaded strip, located between the supports fixed against end rotation should have been zero with identical heating conditions and stiffnesses. The reason that deflections have actually developed is due to deviations from such an ideal condition in the form of non-uniform distribution of furnace temperature, crack formations in isolated sections and the somewhat varying material characteristics in different sections.

The deflection curves illustrated in the Figure exhibit trapezoidal smooth curve segments within the time interval 1/2 - 1 hour. These segments functionally correspond to periods of extensive evaporation and expulsion of the initial moisture content, which temporarily delay the heating and the deflection. Thus, in many tests it was established that in connection with the appearance of bending tensile cracks, there was an intensive escape of water to the upper surface of the strips. The maximum deflections at the mid-section of the vertically nil-loaded strips have varied from 2 to 5 mm, whereas the corresponding deflections for vertically loaded strips have been considerably larger. After cooling, the residual deflection of the nil-loaded strips is characterized by an upward deflection with the corresponding positive residual bending moments. The residual upward deflections, in tests illustrated in FIG. 43, have the following values for the given test series:

0.45 mm for test A2:1 (opening factor = 0.01$m^{1/2}$ and fire duration = 4 hours)

2.20 mm for test A2:2 (opening factor = 0.02 $m^{1/2}$ and fire duration = 2 hours)

1.90 mm for test A2:3 (opening factor = 0.04 $m^{1/2}$ and fire duration = 1 hour)

1.35 mm for test A2:4 (opening factor = 0.06 $m^{1/2}$ and fire duration = 2/3 hour)

0.55 mm for test A2:5 (opening factor = 0.08 $m^{1/2}$ and fire duration = 1/2 hour).

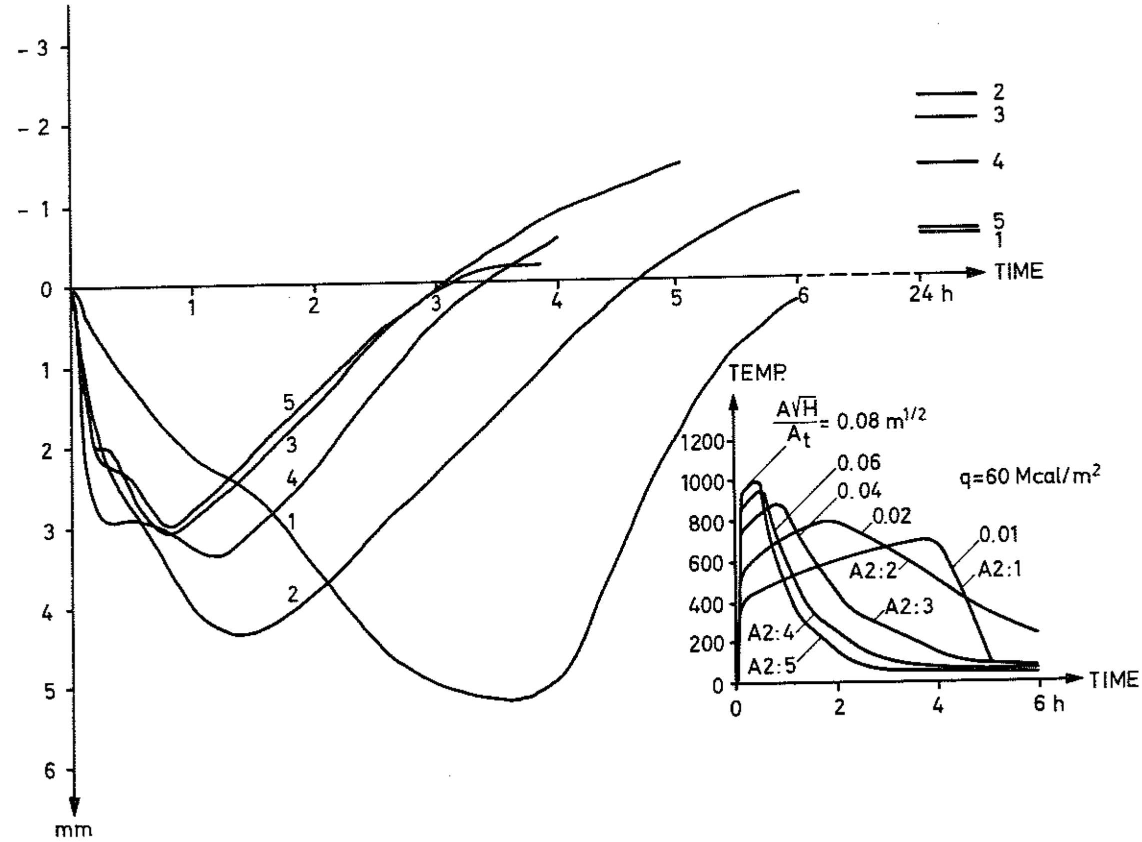


FIG. 43 Mid-point deflection curves for fire tests A2:1-5 characterized by a constant fire load with varying opening factors within the range 0.01-0.08 m$^{1/2}$. The corresponding temperature-time curves are also shown.

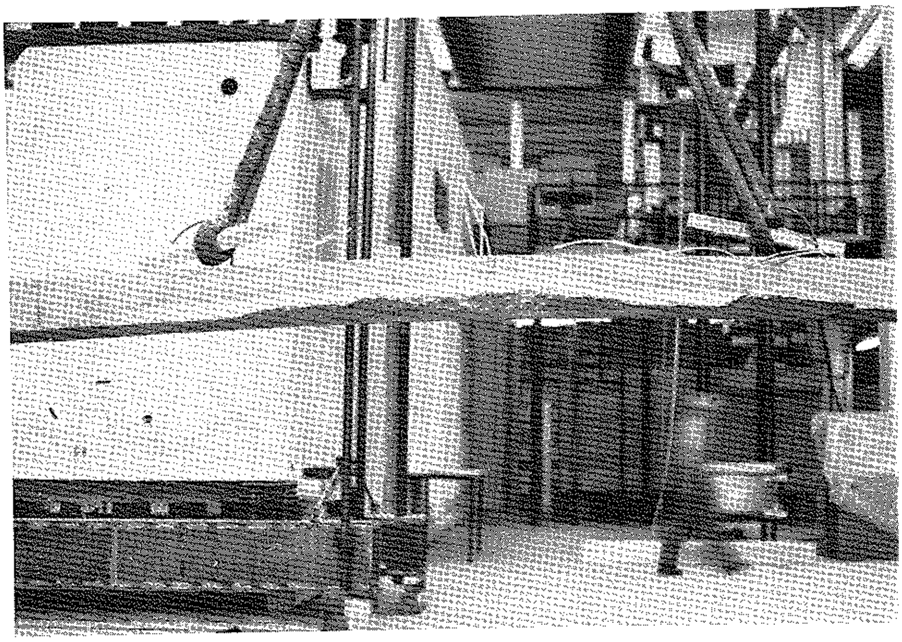


FIG. 44 Picture of the spalling developed during a fire test.

As established before, the magnitude of the positive residual bending moments, obtained in the five tests, follow the same order, cf FIG. 38. For a definite fire load there obviously exists a joint combination of opening factor and fire duration for which the influence of shrinkage and creep deformation, with respect to residual stress and residual deformation after cooling, is the optimum.

In certain cases, the spalling at the corners and surface layers of nil-loaded strips has taken place on the lower side exposed to fire (FIG. 44). All the spalling has appeared in the period between 15-20 minutes after the start of heating phase. All the spalling has been limited to fires with the opening factor within the range 0.04 - 0.08 $m^{1/2}$, that is fires with rather high rates of combustion. The spalling has appeared with such frequency and distribution which, with respect to the highly limited number of experiments, gives an impression of random character. This fact leads to the conclusion that during a fire exposure with the opening factor within the range 0.04 - 0.08 $m^{1/2}$, and the corresponding fire load, $q \geqslant 60$ Mcal/m^2 bounding area, thermal stresses close to ultimate stress of concrete develop in the surface layer directly subject to fire, and the question of whether spalling takes place or not depends on the dispersion in the test conditions, and humidity and strength of the concrete. If, at the same time a vertical load is applied, the thermal stresses due to fire exposure decrease in midspan and the risk of spalling is reduced. Instead, the compressive stresses developed by the vertical load in a region close to the support superimpose on the thermal stresses caused by heating. In this region, the furnace temperature, however, has been lower than in the span and consequently the corresponding thermal stresses are much less. In the case of vertically loaded strips no spalling of the corners and surface layers have hitherto been observed.

5.6 INVESTIGATIONS ON RESIDUAL STRENGTH

The residual load-bearing capacity and bending stiffness of a structure after fire are of great importance in estimating the extent of the subsequent reparations. Thus investigations concerning the residual state of a structure are of considerable interest and such investigations have been performed on some of the concrete strips belonging to the test series an account of which has been presented.

In order to determine the residual bearing capacity and the residual bending stiffness of the fire tested concrete strips they were loaded up to the fracture point by two symmetrically placed concentrated loads 0.8 m apart, the distance between the supports being 2.5 m .The reason why just this distance between the two concentrated loads was selected was due to the fact that within this distance the concrete strips could be exposed to a uniform fire influence and consequently there would be an approximately uniform decrease in bending stiffness within this region. During the whole loading process the deflections in the mid-section and 1/4-sections and the curvature between the two concentrated loads were determined with a load increment of 100 - 200 kp per concentrated load. In relation to the main experiments these tests were performed partly for those with the rigth side up and partly for the strips which were up-side-down with the purpose of determining the residual bending stiffness and the residual load-bearing capacity for loads which rendered either positive or negative bending moments in the strips. Later, after these investigations reinforcement bars belonging to concrete strips have in some cases been tested to determine the load-deformation properties.

As an illustration of the obtained results FIG. 45 shows the relationship between the load and deformation partly for strips not exposed to fire and partly for the fire tested strips 9, 10, 11 and 12 from test series D which were investigated about five months after they had been fire-exposed. Out of these strips, 9 and 10 were loaded with positive and 11 and 12 with a negative bending moment. From the diagram it is observed that there is a linear relation between the total load (sum of the concentrated loads) and the deflection in the mid-section up to the loading level at which a smooth transition to yielding starts, whereupon the deformations strongly increase without any significant increase of load. For the strip not exposed to fire and shown with dashed lines in the Figure it can obviously be observed that the moment corresponding to the appearance of the first cracks is attained at a load of approximately 1 Mp whereupon the stiffness decreases somewhat up to the yield moment and fracture takes place at the load of 4.9 Mp. The total ultimate load for strips 9 and 10 which have earlier been influenced by a fire corresponding to a fire load of 60 Mcal/m^2 and an opening factor of 0.01 $m^{1/2}$ (same as strips 11 and 12), was 4.5 and 4.4 Mp respectively, while the decrease in bending stiffness seemed to be very small compared with the strip not exposed to fire. On applying load to strips 11 and 12, corresponding to a negative moment, the ultimate load was reduced to 4.2 and 4.0 Mp respectively at the same time as the bending stiffness exhibited an obvious decrease caused by decrease of concrete strength in the part of cross-section situated nearest the side of the strip subject to fire which now acted as the compressive zone of concrete. For strips 9 and 10 the fracture started by the tension reinforcement attaining the yield point resulting in increased deformations whereupon rupture

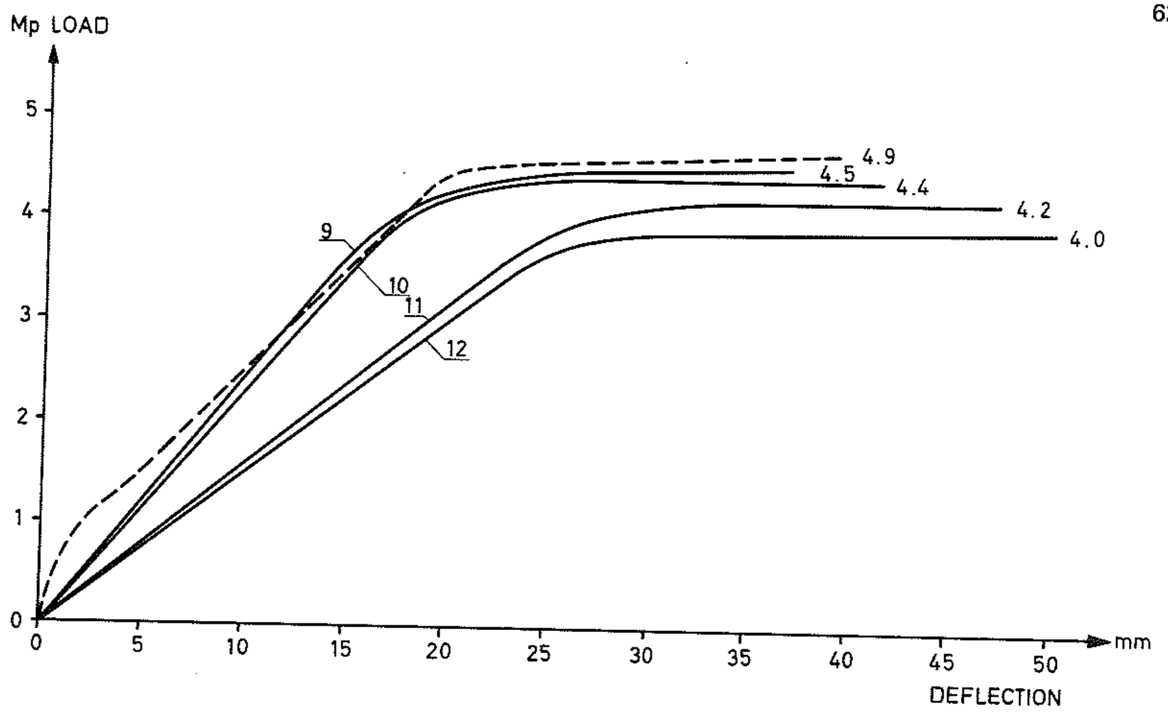


FIG. 45 Load-deflection curves at the midspan for the fire tested concrete plate strips 9, 10, 11 and 12 from series D characterized by a fire load of q = 60 $Mcal/m^2$ and an opening factor of 0.01 $m^{1/2}$. The dashed curve indicate unexposed specimen. On plate strips D 9-10 the load correspond to a positive bending moment and on strips D 11-12 to a negative bending moment.

appeared in the compressive zone of concrete. For strips 11 and 12, on the other hand, the situation was the reverse, that is, first rupture appeared in the compressive zone of concrete and thereafter the yield point was reached in the tension reinforcement. The reduction in bearing capacity amounted to approximately 10 and 20% respectively. In the investigations concerning the residual strength thus far performed on strips exposed to a fire corresponding to a maximum fire load of 120 Mcal/m^2 with the opening factor varying within the range of 0.01 - 0.08 $m^{1/2}$, it may be generally established that there will be a reduction in bearing capacity of approximately 10-15% and 20-25% for strips loaded with positive and negative moments respectively, that is vertically loaded strips with the right side up and up-side-down respectively. However, the residual bending stiffness has not decreased for strips with the right side up with the exception of the loading segment up to an approximate value of 1 Mp, while the up-side-down strips have suffered a decrease of 50-70%. The reduction in bearing capacity and residual bending stiffness may even be observed from the following table:

TABLE

Strips	Decrease in load-bearing capacity	Decrease in residual bending stiffness
Right side up	10 - 15 %	0 %
Up-side-down	20 - 25 %	50 - 70 %

Decreases in strength brought about by fire in a concrete construction are naturally extremely important to know for reclassification and fire damage investigations.

5.7 RESERVE LOAD-BEARING CAPACITY DURING FIRE EXPOSURE

On investigation of the structural behaviour of a construction it is of great interest fo follow up its reserve bearing capacity during a fire exposure. As an illustration, FIGS. 46 and 47 show the moment distribution in strips 8 and 12 from test series D respectively at different times during the fire process. Negative and positive moment capacities which indicate the calculated load-bearing capacity on the basis of the temperature existing in the reinforcement at the actual time, are evaluated and entered in the Figure for a direct comparison with actual moment distribution during the fire process. In calculating the moment bearing capacity, the reduction in concrete strength in the fire influenced compressive zone has not been taken into consideration, partly due to the rather low temperatures in the compressive zone in the mid-section and the fixed-end section respectively, implying moderate

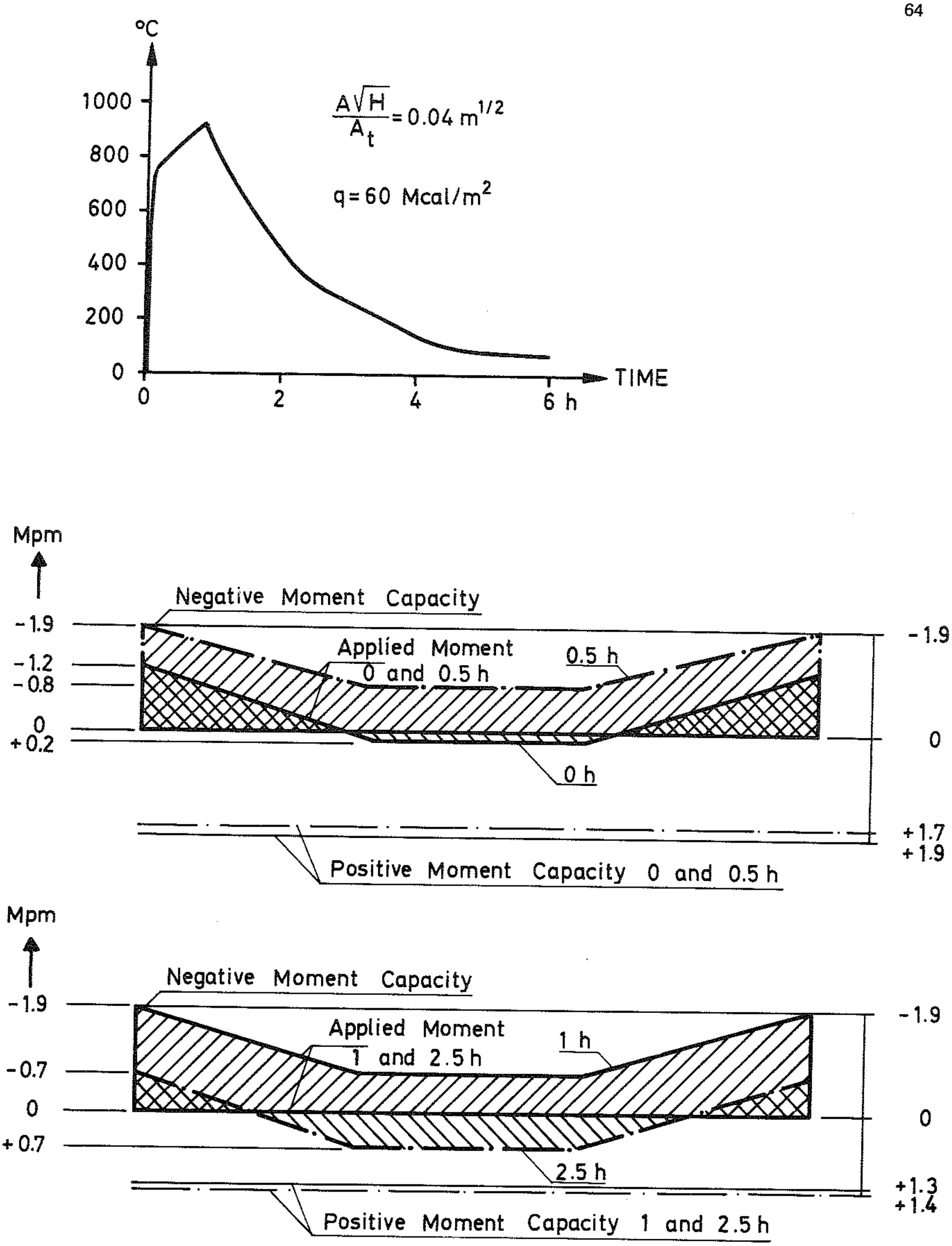


FIG. 46 Bending moment distribution and bending moment capacity during fire exposure (fire load $q = 60\ Mcal/m^2$, opening factor $A\sqrt{H}/A_t = 0.04\ m^{1/2}$) for test D 8 characterized by a loading level of $P_{all} \approx 1/2\ P_{ult}$.

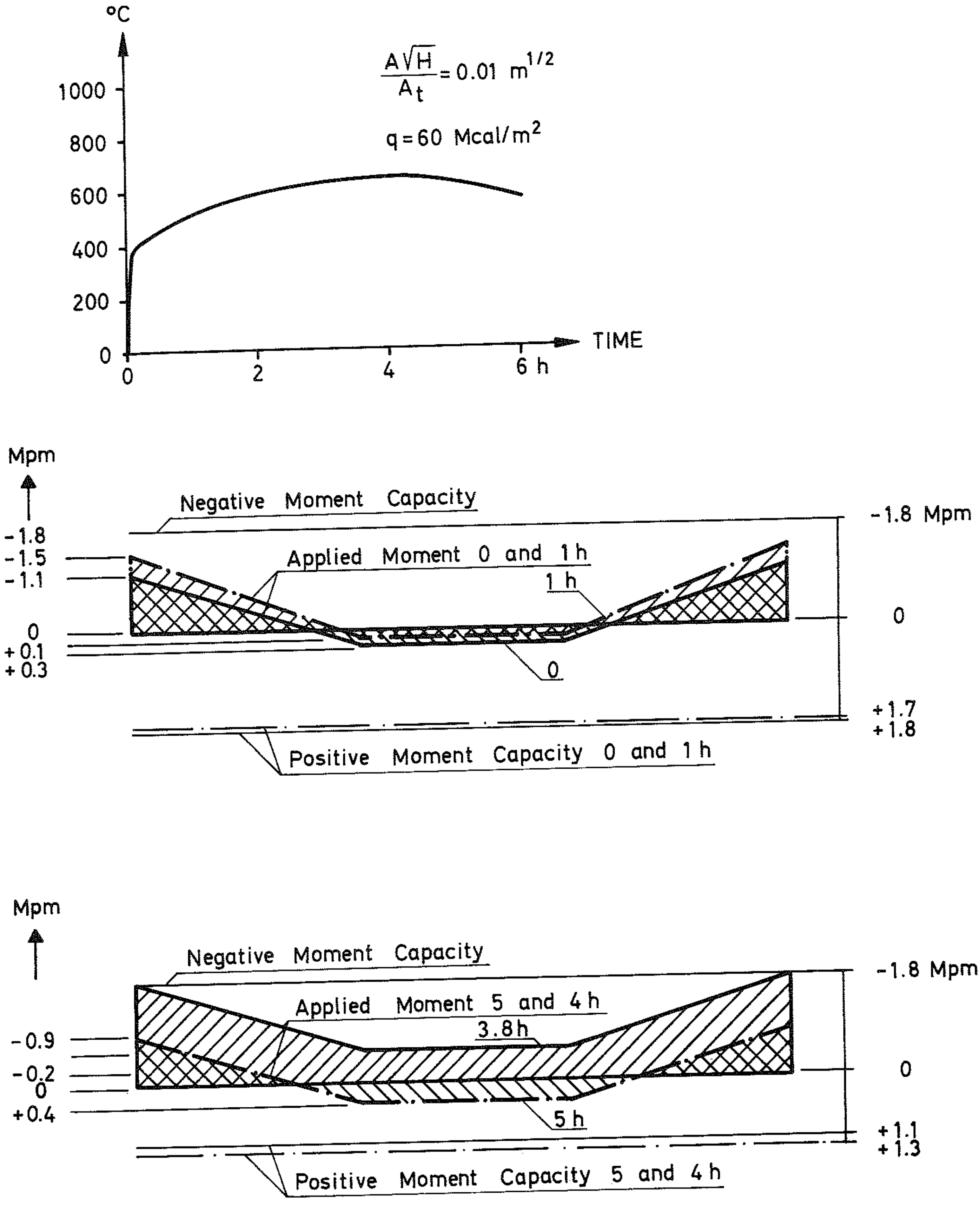


FIG. 47 Bending moment distribution and bending moment capacity during fire exposure (fire load $q = 60\ Mcal/m^2$, opening factor $A\sqrt{H}/A_t = 0.01\ m^{1/2}$) for test D 12 characterized by a loading level of $P_{all} \approx 1/2\ P_{ult}$.

reduction in strength, and partly due to the existing compression reinforcement area (= tension reinforcement area) meaning that the strip is obviously underreinforced. In the tests illustrated, the maximum temperature for the compressive zone in the mid-section and fixed-end section have been varied between 100 and 300°C. If the moment capacity is attained both at the support and in the span, fracture takes place in the strip, but in case the moment capacity is attained either at the support or in the span, the strip thereafter behaves as if it were statically determinate. For the strip 8, the actual moment distribution is shown in FIG. 46 at times 0, 0.5, 1 and 2.5 hours after the start. At the start of the test the moment distribution is not in agreement with that calculated according to the theory of elasticity for a clamped-end strip - which also characterizes test 12 in FIG. 47 - due to crack formation at the support close to the water level and unavoidable initial deformations in the strip which has resulted in increased fixed-end moment. Besides, it may be noted that the moment capacity has initially a maximum value of +1.9 and -1.9 respectively which reduces only in the span to a value of 1.3 Mpm attained after 1 hour. In this test there is no risk for fracture failure, in fact the reserve load-bearing capacity is rather large. In FIG. 47 the corresponding diagram is shown for test 12 at times 0, 1, 3.8 and 4 hours after the start. In this test the moment capacity is initially ± 1.8 Mpm and reduces to + 1.1 Mpm in the span after 3.8 hours. Even here there is no risk for fracture failure. An interesting comparison may be made with a similar test performed at PCA but with the difference that the fire exposure has continued until fracture has taken place.

6. BRIEF VIEWPOINTS ON A THEORETICAL DISCUSSION WITH A QUALITATIVE ANALYSIS OF THE STRUCTURAL BEHAVIOUR

A substantial, detailed theoretical treatment of the structural behaviour of the statically indeterminate structures subject to fire is ordinarily not feasible with the present existing knowledge. An essential reason, in this connection, is the lack of necessary knowledge of thermal strength and deformation properties of concrete at the actual temperature in a fire. It should, however, be added that during the recent years the research on the subject has been strongly intensified which means that the above mentioned properties are being discovered systematically and in greater detail. Among worthful contributions in this research field the following works are only examples: Malhotra (1956), Zoldners (1960), Hannant & Pell (1962), Harmathy & Berndt (1966), Cruz (1966), Hannant (1968), Birkimer et al. (1969), Fischer (1970), Maréchal (1970), Sullivan et al. (1971) Abrams (1971) and Thelandersson (1972).

Since concrete is a heat resistant and composite material, internal and in some cases crack generating stresses appear during heating, partly due to differences in the longitudinal expansion of the composing materials and partly due to the rather steep temperature gradients brought about by the heat resistance, especially at the beginning of a flaming fire. Besides, there will be shrinkage, creep due to internal thermal stresses and creep due to stresses produced by the external load and all these phenomena are accentuated at elevated temperatures.

A theoretical determination of the structural behaviour and load-bearing capacity of a clamped-end, reinforced concrete strip subject to fire and of the same type as those embraced by the test series contains, as a first step, calculation of the stress and the deformation state prevailing in the strip due to the external load before fire exposure. Thereafter, for every time increment of the fire process the corresponding calculation is successively made for changes in

a) the temperature gradient across the cross-sectional area of the strip,

b) the corresponding thermal expansion and thermal stresses at different points of the cross-section,

c) non-uniform shrinkage deformation induced by the temperature gradient over the cross-section,

d) stress distribution generated by the constant external load, bending restraint moment and the changes in these bending restraint moments which for the present are unknown,

e) total deflection and rotation at different cross-sections of the strip induced by thermal expansion according to b, non-uniform high temperature shrinkage according to c and stress state according to b and d where the influence of shrinkage deformation corresponding to the actual temperature gradient and stress distribution should be taken into consideration,

f) bending restraint moment determined from the condition that rotation over the support of the strips should be zero, and

g) the resultant stress and deformation state when the substituted value for the change of bending restraint moment in agreement with f.

A method of calculation adapted to computers, with basic structure as shown above, has been developed by Saito (1968) and is applicable to fire influenced beams with two supports with varying fixed-end conditions with respect to both longitudinal displacement and rotation. With the present inadequate level of knowledge as far as thermal expansion of concrete, its stress and deformation properties including shrinkage and creep deformations within the actual temperature of the fire exposure are concerned, the computation method is primarily only of scientific interest. In the future, when all the necessary material data on the subject are available, the computation method should be able to open the path towards a theoretical analysis of great importance for practical design embracing the structural behaviour of statically indeterminate, reinforced concrete structures subjected to fire in a complete fire process.

For a qualitative theoretical analysis of the structural behaviour of the strips clamped at both ends which are described in this paper, FIGS. 46-48 show the change in the positive and negative moment capacity of the strips and the bending moments in the span and at the supports induced by the vertical load during a fire process. FIGS. 46 and 47 illustrate this fact for tests 8 and 12 from the test series D with a fire process characterized by a fire load of 60 Mcal per square meter of the enclosing area and opening factors of 0.04 and 0.01 $m^{1/2}$ respectively. In both cases the vertical load is equal to P_{all}. FIG. 46 shows the distribution of the bending moment at times 0.05, 1, and 2.5 hours and even the positive and negative moment capacities are drawn. The decrease in the negative moment capacity is in this case small enough for the moment to be considered unchanged with a constant value of 1.9 Mpm, whereas the positive moment capacity decreases due to the increased temperature in the reinforcement at the bottom side and decreases from 1.9 to 1.3 Mpm. Similary FIG. 47 illustrates the distribution of the bending moment for test D 8 at times 0, 1, 3.8 and 5 hours with the positive and negative moment capacities indicated in the same Figure. In this case, the negative moment capacity is 1.8 Mpm while the positive moment capacity decreases from 1.8 to 1.1 Mpm during the fire process.

With a closer analysis of the structural behaviour it is observed that the reserve load-bearing capacity does not reach its minimum at the same time as yielding at the support is developed but often much later. This fact is illustrated in FIG. 48, in which the basic variations in the bending moments in the span and at the supports together with the moment capacity in the span and at the supports are shown for an imaginary fire process. The curves showing capacity moments are drawn with full lines while the original capacity moment is marked by dotted lines. The capacity moments at the support and in the span are decreased due to the lowering of the yield point of the reinforcement caused by the increased temperature in the corresponding reinforcement. The negative capacity moment is besides influenced by the decrease of the modulus of elasticity of concrete caused by the increased temperature.

At the beginning of the test a certain load acts upon the strip, giving rise to an initial fixed-end bending moment and corresponding span bending moment which maintain a constant difference during the fire exposure. On the time axis 10 points are marked which will now be explained closely.

When the fire test starts the restrained bending moment grows at the support continuously and at time t_1 renders a transition from the positive to negative bending moment in the span. At time t_2 the yield moment of the strip is reached at the support which are then kept constant up to time t_3, while the deflections increase, as a result of continuous rotations in hinges developed over the support subsequent to yielding. The reason why the restrained bending moment then successively decreases, resulting in a transition from negative to positive bending moment in the span at time t_4, is partly because of the decrease in slope of temperature gradient and partly because of the non-uniform distribution of shrinkage and creep effects over the cross-sectional area, strongly accentuated at this time. At times t_5 and t_6 the reserve load-bearing capacity is represented by the sum of 1 and 2, and 1´and 2´ respectively. Due to the strong decrease of capacity moment in the span resulting from the increased temperature in the reinforcement at the bottom side this capacity moment is attained at time t_7 and the decrease of capacity moment is then accompanied by the decrease of the moment in the span to the zero value at time t_8. From time t_7 chain-dotted lines are also drawn in order to indicate the process in case the capacity moment had not continued decreasing. At this point, that is to say before the beginning of time interval t_8, the cooling phase of the process is probably started, while the temperature in the reinforcement at the bottom side continues to increase up to time t_8 at which the load-bearing capacity in the span reaches zero. At continued loading the construction behaves as two clamped-end cantilever beams. At time t_9 the strip slowly starts recovering its load-bearing capacity in the span due to the fact that the cooling renders a continuous temperature decrease in the reinforcement at the bottom side. The strength recovery in the reinforcement at the top side is estimated to be noticeable for the first time at t_{10}. At time t_9 final fracture may take place, depending on how the capacity moment at the support is decreased. The example illustrates the practical possibility of how a reinforced concrete structure subjected to fire may lead to fracture during the cooling phase.

From the qualitative analysis performed, it may be concluded that if the development of the restrained bending moment and temperature-time curve of the fire chamber were known at a certain vertical loading it could theoretically be determined whether fracture would appear in the strip or not.

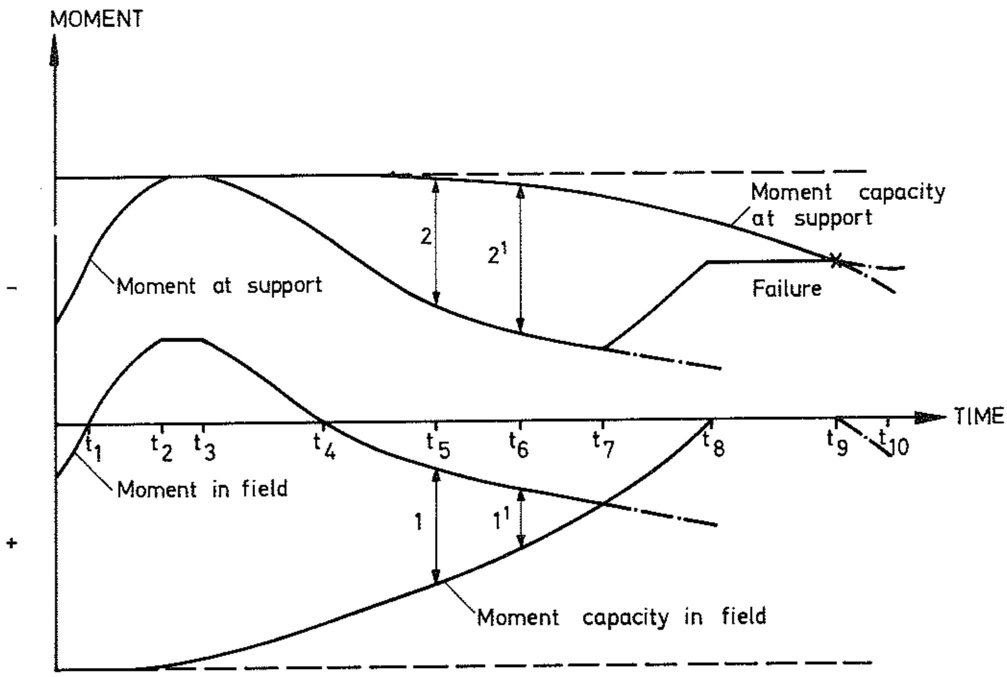


FIG. 48 An illustration to a theoretically possible development of bending moments and capacity moments at support and in field of a clamped-end beam exposed to an imaginary fire process which can cause failure during the cooling phase. The reserve load-bearing capacity at times t_5 and t_6 is represented by the sum of 1 and 2, and 1' and 2' respectively.

7. FUTURE STUDIES

For the future studies it is planned that partly experimental investigations which directly supplement the present tests should be carried out, and partly investigations performed in which such parameters as fire process characteristics, plate thickness, proportion of reinforcement, reinforcement distribution and thickness of the protective concrete layer are intended to be varied for both nil-loaded and vertically loaded clamped-end concrete strips subjected to fire on one side. There are plans to try measuring the strains developed in concrete and reinforcement at different sections of the strip by a heat resistant gauge. For this purpose a preliminary development scheme has been introduced. After calibrating the actual reinforcement by a special device for translation from strains to stresses at different temperature levels and different rates of heating such measurements should be able to give a valuable basis for more detailed studies of the structural behaviour of the strips.

Future experimental investigations of interest consist of such investigations in which partial end-fixation against rotation is simulated and those in which both end-fixation against rotation and against axial elongation of the strip are simulated. Furthermore an experimental determination of the relation between temperature gradient and bending stiffness, which is of phenomenological interest for instance in a detailed result evaluation may for example be investigated through tests performed on simply supported concrete strips.

In order to increase the possibilities for a qualified, theoretical analysis of the stress and deformation processes of the actual structure during fire exposure, further investigations concerning pure material properties such as thermal expansion, creep and shrinkage deformations of concrete at elevated temperatures are of great importance. Such pure studies have partially been introduced at Division of Structural Mechanics and Concrete Construction, Lund Institute of Technology.

8. SUMMARY AND CONCLUSIONS

The fundamental study of concrete strips presented here and exposed to fire on one side and fixed against rotation at both supports with the longitudinal displacement free to take place embraces 5 test series with characteristics according to TABLE 2 which contains information on how, within each series, the fire process characterized by fire duration, fire load and opening factor and concrete composition characterized by water-cement ratio and cement-paste quantity, specimen age and loading level are varied. In the investigation the strips are presented with a detailed construction as in FIG. 18 in which the results of the temperature-time field in the furnace and the strip, restrained bending moment, deflection process, load-bearing capacity during the fire process and the residual strength are summarized and illustrated. The characteristics according to FIG. 5 and TABLE 2 apply to the gas temperature-time curves in a complete fire process which are determined experimentally by using opening factor, $A\sqrt{H}/A_t$, and fire load, q.

The series A1 and A2 which embrace a study of the influence of variations in fire process characteristics contain totally 20 tests. The opening factor, $A\sqrt{H}/A_t$, was varied within the range 0.01 - 0.08 $m^{1/2}$ and the fire load, q, varied within the range 7.5 - 480 Mcal per square meter of the enclosing area of the fire chamber. The corresponding fire duration for the heating phase of the fire process lies within the range 1/2 - 8 hours.

The objective of test series B and C which contained 18 tests was to study the influence of variations in concrete composition (water-cement ratio and cement-paste quantity) and the effect of specimen age on the structural behaviour of concrete strips in order to get a more exact idea of the significance of short time shrinkage and short time creep at elevated temperatures as well as pronounced temperature gradients. The tests were performed for a temperature-time curve of the fire chamber characterized by an opening factor of 0.04 $m^{1/2}$, a fire load of 120 Mcal per square meter of the enclosing area and a fire duration of 2 hours. The concrete composition in series B was varied with respect to the water-cement ratio (0.52, 0.63 and 0.77) at a constant cement-paste quantity, 277 l/m^3, and in series C varied with respect to the cement-paste quantity (257, 277 and 296 l/m^3) at constant water-cement ratio of 0.63. The age of the strips prior to fire exposure was varied from 1/2 to 3 months.

The test series A1, A2, B and C were performed without presence of any vertical load. In a complementary series containing 12 tests, an extended study was carried out using simultaneously vertical loading applied in the form of two symmetrical concentrated loads, as in FIG. 6, at the four loading levels 1/4, 1/2, 3/4 and 1/1 of P_{all} where P_{all} corresponds to the design load according to the existing Swedish Concrete Standards. In the actual case P_{all} amount to 1.60 Mp which approximately amounts to half the value of the ultimate load. In series D, besides the variation in the level of vertical loading, the fire process has been varied at a constant fire load of q = 60 Mcal per square meter of the enclosing area. The opening factor was varied within the range 0.01 - 0.08 $m^{1/2}$ corresponding to the fire duration varying within the range of 1/2 - 4 hours.

As a complement to the main tests an investigation regarding the residual load-bearing capacity and residual bending stiffness of the strip was carried out after cooling down.

In FIG. 33-37 the time curves of the fixed-end bending moment imposed on the strips and the corresponding time curves of the furnace temperature characterized by a constant fire load of 60 Mcal per square meter of enclosing area and an opening factor varying within the range 0.01 - 0.08 $m^{1/2}$ are illustrated. Hence it is observed that when the opening factor increases from 0.01 - 0.08 $m^{1/2}$, the maximum negative fixed-end moment changes from -1.25 to -1.10 Mpm at the same time as the time to attain this shifts in harmony with the fire duration. After complete cooling there remains considerable positive residual bending moments which amount to 0.44, 0.80, 0.72, 0.55 and 0.50 Mpm for the fire durations of 4, 2, 1, 2/3 and 1/2 hours respectively. FIG. 38 renders a summary of the results. Thereafter, FIGS. 39 and 40 give a complementary illustration of the variation of the fixed-end bending moment as a function of time with the varying fire loads of 7.5, 15, 60 and 15, 30, 60, 120 Mcal per square meter of the enclosing area and the opening factor of 0.01 and 0.02 $m^{1/2}$ respectively. Thus it is observed that the variation of fixed-end bending moment as a function of time bears great resemblance to the variation of temperature with time. The maximum negative fixed-end bending moment and residual bending moment observed turned out to be -1.40 Mpm and 1.20 Mpm respectively, at the fire load of 120 Mcal/m^2.

In test series B it may be noted that the time for the negative restraint moment to reach its maximum value has been delayed with increasing water-cement ratio whereas specimen age and water-cement ratio have no other appreciable influence.

In test series C, where the quantity of the cement-paste is the decisive factor, a larger residual bending moment is obtained and the negative residual bending moment reaches its maximum value earlier when the quantity of the cement-paste is increased. Otherwise no other decisive effects have been observed.

The results obtained from test series D, with vertically loaded strips, where all the parameters except for the opening factor and level of the vertical load were kept constant, are illustrated in FIGS. 41 and 42. Time curves for the fixed-end bending moment and time curve for the corresponding furnace temperature may be observed in FIG. 41 at vertical loading levels of P = 0, 1/4, 1/2, 3/4 and 1/1 of P_{all}, in a fire process for which the opening factor is 0.08 $m^{1/2}$ and the fire load is 60 Mcal per square meter of the enclosing area. The negative yield bending moment of the strip was attained at loading levels of 3/4 P_{all} and P_{all} after 0.40 hours, while the residual bending moment amounted to +0.50, +0.35, +0.20, -0.45, and 0 at loading levels of 0, 1/4, 1/2, 3/4 and 1/1 of P_{all} respectively after cooling to ordinary room temperature. The corresponding five tests illustrated in FIG. 42 have had a much slower rate of heating determined by the opening factor of 0.01 $m^{1/2}$. Here, the negative yield bending moment of the concrete strips is reached after approximately 2 hours at the vertical loading levels of P_{all} and 3/4 P_{all} and after approximately 3.75 hours at the vertical loading level of 1/2 P_{all}. The residual bending moments amounted to +0.40, +0.30, and +0.10 Mpm at the loading levels of 0, 1/4 and 1/2 of P_{all} respectively.

As a complementary illustration of the structural behaviour of nil-loaded concrete strips, FIG. 43 shows time curves for the deflection at the mid-section of the strips together with the corresponding time curve of the furnace temperature for tests 1 - 5 in series A2, that is tests in which the fire process, with the constant fire load of q = 60 Mcal/m^2, was varied through the opening factors of 0.01, 0.02, 0.04, 0.06 and 0.08 $m^{1/2}$. The maximum deflections at the mid-section of nil-loaded strips have varied within the range 2-5 mm while the corresponding deflections for loaded strips amounted to 15 mm. The residual deformation state after cooling is characterized by upward deflections at the mid-section and the maximum deflection amounted to 2.2 mm for test A2:2. The positive residual bending moments obtained in the five tests have had the same order of magnitude as the corresponding residual deformations (cf FIG. 38).

During fire exposure, spalling of corners and external layers has been observed in fires with opening factor ⩾ 0.04 $m^{1/2}$ and fire loading ⩾ 60 Mcal per square meter of enclosing area. The spalling has taken place sporadically and gives an impression of random character.

In the illustrated investigation concerning residual strength, the residual load-bearing capacity and the residual bending stiffness of the fire-tested strips were determined six months after they had been poured. In this investigation it was discovered that the load-bearing capacity had decreased between 10-15% for the loaded strips with the right side up, corresponding to a positive bending moment and the decrease of residual bending stiffness was insignificant. The decrease in load-bearing capacity for the loaded up-side-down strips, corresponding to a negative bending moment lay between

20 - 25% and at the same time the residual bending stiffness had decreased considerably as shown in FIG. 45.

In this paper some viewpoints on a theoretical analysis of the structural behaviour of experimentally investigated strips during fire exposure are presented. The theoretical possibility that a strip ruptures first during the cooling phase is also illustrated in brief. This fact may be observed in FIG. 48 which also shows the basic variation of the actual bending moments at the support and in the span and the capacity moment (moment bearing capacity) during an imaginary fire process.

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