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Fire Effects on Reinforced Concrete Members

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Fire Effects on Reinforced Concrete Members

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FIRE EFFECTS ON REINFORCED CONCRETE MEMBERS

Bruce Ellingwood and James Shaver

Fire ratings for structural assemblies in the U. S. are currently measured by endurance of or temperature rise in components subjected to a standard test. Analytical procedures show considerable promise for alleviating the extensive testing required, and for placing fire resistant structural design on a limit states basis. In this study, thermal and structural analyses for reinforced concrete members are validated using experimental data. Temperature distributions computed in reinforced concrete members for several realistic fire exposures are examined and compared to the distributions measured in a standard test. Parameter studies show the effect of typical variations in thermal diffusivity, emissivity and conductivity. Structural responses for the different fire exposures are also briefly compared.

Key words: Concrete (reinforced); fire load; fire resistance; fire tests; structural engineering; temperature; thermal analysis.

NOTATION

A_f, A_w = Floor area, wall area;

A_o, A_t = Area of opening, total bounding surface area;

h = weighted height of openings;

h_c = conduction coefficient;

k = thermal conductivity;

q, q_f = fire load referenced to total bounding surface area, floor area, respectively;

T, t = temperature, time, respectively;

ε = surface emissivity coefficient;

ρC_p = heat capacity;

σ = Stefan - Boltzman constant;

ψ = ventilation or opening factor.

1. INTRODUCTION

Losses due to fire in the U.S. exceed losses from other extreme environmental factors which are normally considered in structural design. In the U.S., fire ratings for structural assemblies are currently based on the time period until structural failure, loss of integrity, or excessive temperature rise occurs in structural members subjected to the standard ASTM E119 [14][1] fire endurance test. These tests, which form the basis for most code requirements, are very costly and do not provide a basis for extrapolating to situations not covered by the limited test data. Moreover, there is substantial evidence [8-12] to show that the ASTM fire test may be an inappropriate indicator of performance for many kinds of realistic fires.

In recognition of these factors, analytical procedures have been developed and are currently being extended [1, 2, 3, 4, 6] which show promise in predicting structural behavior under conditions simulating severe fire exposures. These procedures enable the effects of various material parameters on thermal and structral response to be studied without resorting to expensive laboratory testing. They also make it feasible to consider alternate fire curves (time-temperature histories) for design in addition to ASTM E119; the necessity for doing this has been recognized by various standards organizations in Europe.

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[1] Numbers in brackets denote references found at the end of this report.

Improved analysis procedures, validated by limited experimental data, make it possible for the designer to consider fire explicitly as another limit state. The development of fire resistant design procedures based on identifiable limit states of performance and realistic fire curves should lead to cost savings in certain structures and result in more uniform reliability for all structures.

The present paper contributes to the data base which is necessary for improved fire resistant design procedures to evolve in the U.S. Temperature distributions in various reinforced concrete sections are studied as functions of thermal conductivity and heat capacity. Temperature distributions corresponding to alternate fire exposure curves are also illustrated; these curves were developed utilizing data from a recent survey of fire and live loads in offices in the U.S. conducted by the National Bureau of Standards. Finally, the sensitivity of structural behavior of simple reinforced concrete members to the different fire curves will be examined briefly. These parametric studies are necessary to assess the effect of these curves on the variability in member performance in a realistic fire.

2. DEVELOPMENT OF ALTERNATE FIRE CURVES

The standard ASTM E119 fire test imposes three conditions for failure for reinforced concrete components: (1) Collapse of the component or failure to inhibit passage of flame or hot gasses; (2) Attainment of the limiting average temperature of 593°C (1100°F) in reinforcement; (3) Rise of 139°C (250°F) in the average temperature of the unexposed surface of the test component. The applied temperature-time relation (fire curve) is monotonically increasing during the rating period and is identical for walls, floors, columns and beams. This test is useful as a standard procedure for comparison purposes, but may not always provide a good indication of how the component might be expected to behave in an actual fire. Except for the first failure condition, the test failure criteria are not directly related to any physically meaningful limit state of performance. Moreover, the use of one unique increasing fire curve fails to recognize that the amount and composition of material in the compartment, as well as ventilation effects, will cause variations in the rate of heat generated and the peak temperature reached. Finally, structural and heat transmission effects which occur following the rising temperatures during the rating period are not accounted for [8, 9, 10, 11].

Numerous compartment fire tests have been conducted in recent years. These indicate that while the fire curves depend on many factors, the two significant parameters are the fire load q per unit area (described subsequently) and a ventilation or "opening" factor defined as $\psi = A_o \sqrt{h}/A_t$, in which A_o = total area of window and door openings, h = weighted average height of openings, and A_t = total area of compartment bounding surfaces [8-11]. With q and ψ defined, realistic fire curves for design can be generated [11].

The National Bureau of Standards has recently completed a survey of fire and live loads in office buildings in the U.S. [5]. The fire loads reported represent the total weight of combustible items within a room converted to equivalent weights of combustibles having a calorific value of 18.6 MJ/kg (8000 Btu/lb). This was then divided by the room floor area to obtain the load intensity (load per unit area) reported. The effects of several factors on the fire load were considered. In general, it was found that the fire loads did not depend on the geographic location, height or age of the building, nor was there any significant different between government and private office buildings. On the other hand, the magnitude of room fire loads appeared to be strongly correlated to the use of the room; moreover, file, library and storage rooms which tended to have low values of ψ were more heavily loaded than general and clerical offices which generally had high values of ψ. Table 37 of Ref. 5 shows that the mean and standard deviation of the total fire load (in equivalent weight; see preceding paragraph) for general clerical offices are 350 N/m^2 (7.3 psf) and 211 N/m^2 (4.4 psf), respectively; the 95 percentile values is 737 N/m^2 (15.4 psf) according to Fig. 14 of the same reference. The average opening factor was ψ = 0.081 $m^{1/2}$ (0.146 $ft^{1/2}$) with a standard deviation of 0.07 $m^{1/2}$ (0.127 $ft^{1/2}$), indicating that the scatter in ψ is quite large.

In order to use the method of Ref. 11 to generate fire curves, the above fire loads must be converted to load per unit area of bounding surface (wall plus ceiling plus floor). That is, with $q_f = W/A_f$ reported in Ref. 5,

$$q = \frac{W}{A_t} = q_f \frac{1}{2 + A_w/A_f} \qquad (1)$$

Table 32 of Ref. 5 may be used to estimate A_w/A_f; it was found that $(2 + A_w/A_f)^{-1}$ ranged from about 0.25 to 0.36. For purposes of obtaining a fire curve for general and clerical offices, then, $q \approx 0.3\ q_f$.

Using a derating factor of 0.9 [5], to reflect the fact that combustibles enclosed in steel furniture may not be totally consumed in a fire, we find that for general and clerical offices the mean value of q is 179 MJ/m^2 and the 95-percentile value is 378 MJ/m^2 with a mean opening factor 0.081 $m^{1/2}$. The mean and 95-percentile short duration high intensity design fire curves thus obtained (SDHI-M and SDHI-95) are compared in Fig. 1 to the ASTM E119 fire curve. The decrease in temperature following the exhaustion of the finite amount of fuel is characteristic of realistic fire curves [9, 10], but currently is not taken into account in standard fire tests, since performance is only considered during the rising temperature portion.

Similar fire curves can be generated for other room uses. Lumping the fire load data on file and storage rooms for government and private offices [5], the mean q_f = 277 MJ/m^2 (13.6 psf) and ψ = 0.03 $m^{1/2}$ (0.016 $ft^{1/2}$). Here, $q \approx 0.25\ q_f$, as file and storage rooms tend to be small. The corresponding long duration moderate intensity fire curve (LDMI-M) using the conservative derating factor of 0.9 is also compared to the ASTM curve in Fig. 1. In contrast to the fire curves for general and clerical offices, the curve for file and storage rooms indicates a lower maximum temperature but a longer fire duration. This is a reflection of the lower ventilation factor, which reduces the rate of combustion, as well as the higher fire load.

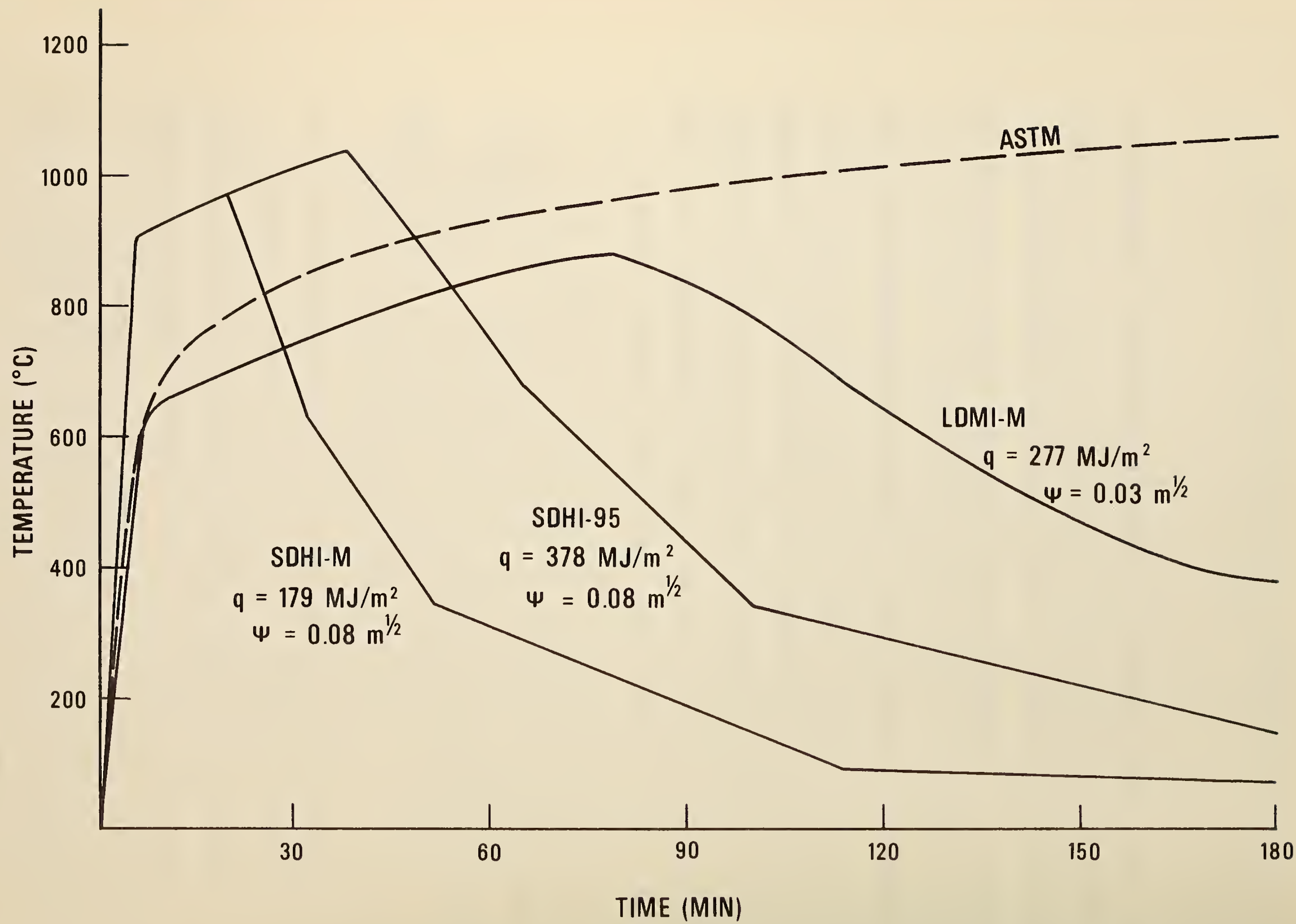

Fig. 1 Comparison of Computed Fire Curves with Standard

These fire curves are quite similar to the temperatures observed in compartment fire tests. For example, recent fire tests have been conducted at NBS in which the test compartment simulated a residential recreation room. The equivalent fire load was 225 N/m^2 (4.7 psf) referenced to the floor area and $\psi = 0.05\ m^{1/2}$. A time history of average upper air compartment temperatures measured directly during the test is shown in Fig. 2 (curve DATA).* The irregularities in the time-temperature history are typical in fire tests. Compartment temperatures were only monitored for one hour; the remaining portion of the curve (dotted line) is estimated. The trend for the portion of the curve for which data are available is quite similar to those previously discussed. Curves developed [11] with $q = 100\ MJ/m^2$ (roughly equivalent to the 4.7 psf fire load on the floor) and with $\psi = 0.04\ m^{1/2}$ and $\psi = 0.08\ m^{1/2}$ are shown in Fig. 2 for comparison. Considering the inherent scatter in fire testing, the agreement is quite good.

Conceptually, different fire curves can be generated for any combination of fire load and opening factor, including the effect of different bounding surface compositions [11]. Considering the present state of fire resistance design in the U.S., however, such refinements may not be desirable in the short term or even necessary in order to obtain more realistic estimates of structural performance during fires and to foster the adoption of limit states design procedures. The survey of fire loads in offices [5] revealed that

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* These data made available courtesy of by Dr. J. B. Fang of the Center for Fire Research, National Bureau of Standards.

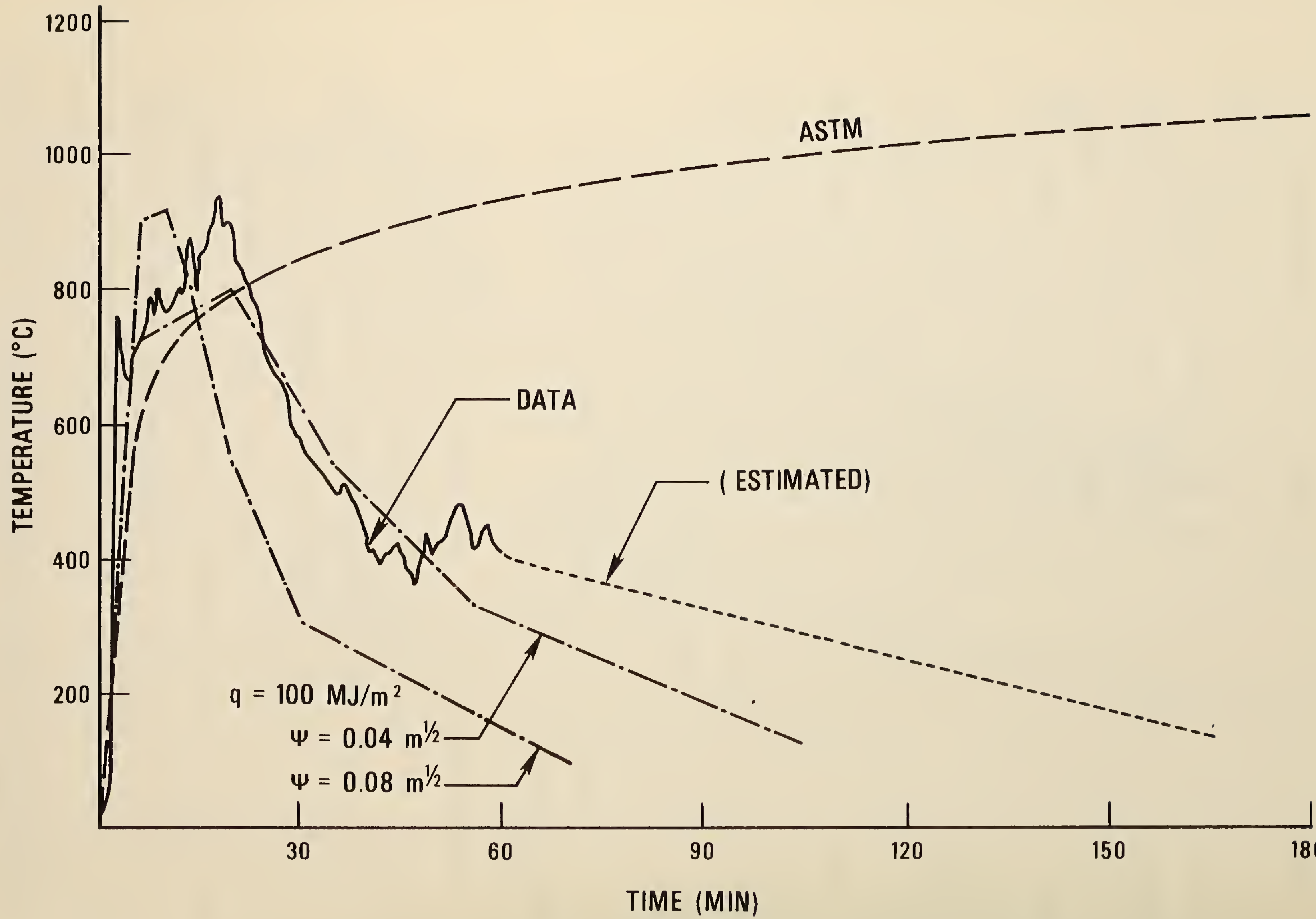

Fig. 2 Comparison of ASTM Fire Curve with Measured Fire Temperatures

areas which are more heavily loaded also tend to be poorly ventilated. Thus, it would appear that improvements and potential economies in fire resistant design for office buildings could be achieved by initially specifying just two design fire curves; the first for heavily loaded, poorly ventilated areas such as storage rooms (LDMI), and the second for lightly loaded, well ventilated areas such as general offices (SDHI). Additional curves can be specified at a later time for other occupancies (residences, commercial establishments, warehouses, etc.) by standards writing organizations if they feel the situation warrants them. In the following sections, the response of members to such curves is compared with their response to the ASTM fire curve.

3. VALIDATION OF THERMAL ANALYSIS

A thermal analysis of a non-reinforced rectangular concrete section was made to study the effect on temperature distribution of varying the thermal properties of the concrete (conductivity, capacity) and the parameters on the fire boundary (convection, emissivity). The analysis is two dimensional, and it has been assumed the material is homogenous. There are no phase changes or exothermic reactions. The possibility of cracking during the fire has been neglected. The results of the analysis are compared with experimental data from an ASTM Fire Test provided NBS by the Portland Cement Association (PCA)*.

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* Data provided courtesy of Mr. M. S. Abrams, Mgr, Fire Research Station, PCA. A paper entitled, "Temperature Distribution in Concrete Beams During Fire Tests," by T. D. Lin and M. S. Abrams documenting these tests is currently in preparation.

The thermal analysis was performed with a computer program* which solves by forward differences the nonlinear heat conduction equation:

$$\rho C_p[T]\ \dot{T} + k\ [T]\ T = Q\ [T,\ t] \qquad (2)$$

in which ρC_p = the heat capacity matrix, k = the conductivity matrix, Q = the heat input vector, t = time, T = temperature vector and the dot represents differention with respect to time. Q is defined at the boundary nodes by,

$$Q = h_c\ (T_A - T_S) + \epsilon\sigma(T_A{}^4 - T_S{}^4) \qquad (3)$$

The first and second terms in Eq. 3 are convection and radiation components, respectively; h_c = the convection coefficient, ϵ = the emissivity coefficient, σ = the Stefan-Boltzman constant and, T_A,T_S = absolute air and surface temperatures. The accuracy and stability of the solution to Eq. 2 is based on a limited time step within each time interval which is a function of the material properties and boundary conditions during the time interval.

Figure 3 shows the grid used in the analysis of the 203 mm by 406 mm (8 x 16 in) rectangular cross section and the circles denote the locations of the thermocouples on the cross-section during the PCA test. Due to symmetry only half of the cross-section was analyzed. The ASTM test is simulated analytically by exposing the bottom and side of the section to the fire while the top is exposed to the ambient air temperature.

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* The computer program was initially developed by Mr. Ulf Wickerström of Lund Institute of Technology, Sweden.

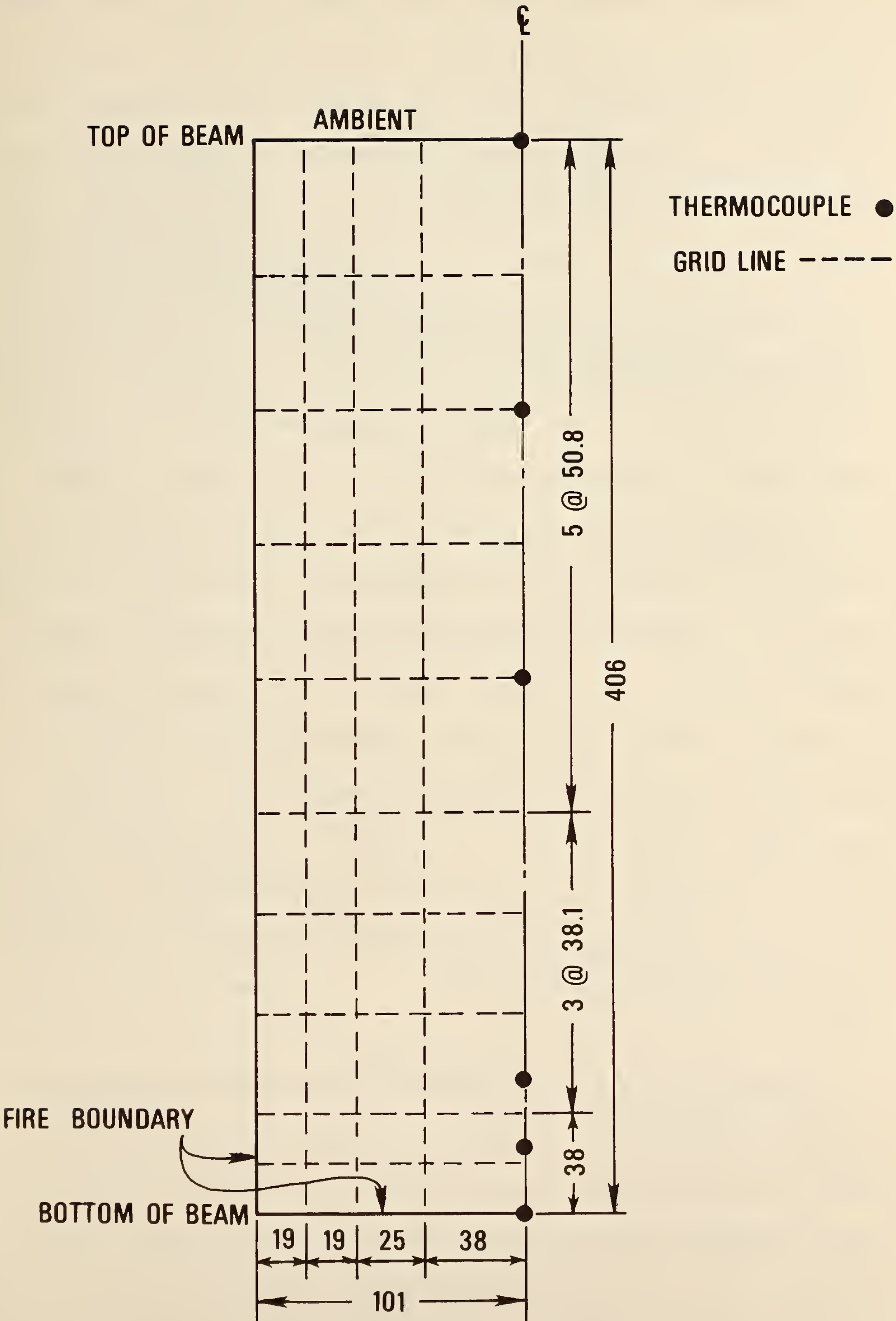


Fig. 3 Non-reinforced Rectangular Concrete Section Used in Thermal Analysis Validation

Temperature changes have a marked effect on the thermal properties of concrete and a wide range of values have been reported by different investigators. This range is due to the composite nature of concrete and varies with the properties of the aggregate and cement, their proportions, moisture content and also the history and age of the concrete. Despite this scatter it has been found that the conductivity of concrete generally decreases with an increase in temperature as shown in Fig. 4. The range of values indicated by the shaded area in Fig. 4 was reported by Bizri [3] and the dashed line represents the conductivity model used in this study. The other two conductivity models indicated by the triangle and square were reported by Anderberg [1] and Petterson [12], respectively. The heat capacity of concrete is a function of both the density, ρ, and the specific heat, C_p, of concrete. Although both change with temperature as might be expected due to drying, dehydration, etc., the product of the two, ρC_p, remains essentially constant with increase in temperature. Bizri used ρC_p = 745 W-hr/m^3-°C (40 Btu/ft^3-°F) while Petterson suggested that ρC_p = 590 W-hr/m^3-°C (32 Btu/ft^3-°F) between 0 and 400°C (752°F) and 764 W-hr/m^3-°C (41 Btu/ft^3-°F) for temperatures greater than 800°C (1472°F).

Analysis were conducted using the three diffusivity ($k/\rho C_p$) relations (numbered for reference in further figures) shown in Fig. 5 to study the effect that thermal properties and their variability have on temperature distributions. One value for thermal capacity, ρC_p = 725 W-hr/m^3-°C was used. Thermal diffusivity is an indication of ability to respond to a sudden temperature change; a

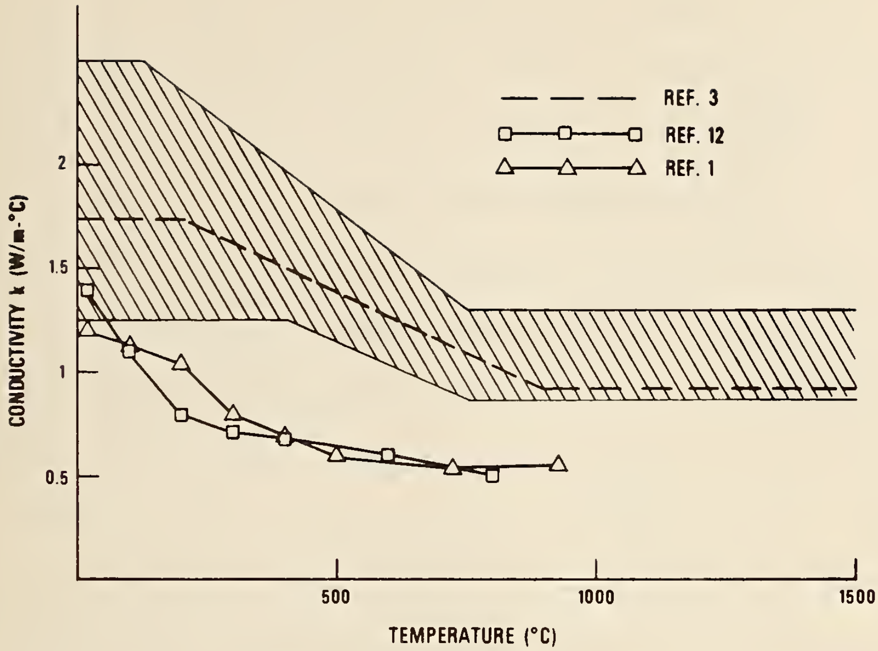


Fig. 4 Effect of Temperature on Conductivitv of Concrete

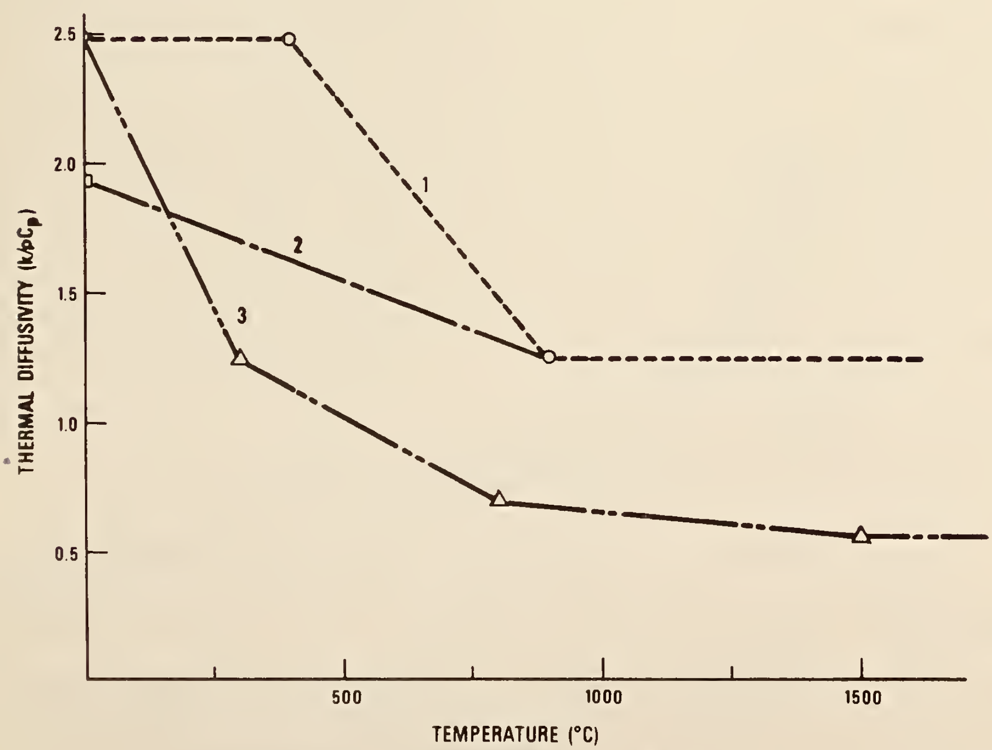


Fig. 5 Concrete Diffusivity Functions Used in Thermal Analysis Validation

low value such as that of concrete indicates slow response. Figures 6 and 7 show the computed temperature variation along the cross-section center line for the three diffusivity models along with the PCA thermocouple data. In comparison to the thermocouple data at both 2 and 4 hours, all three models over-predict the temperatures at the fire boundary (bottom of beam) and under-predict the temperature at the unexposed surface (top of beam). It should be noted that the thermocouple on the fire boundary may not give an accurate reading of the concrete surface temperature because of the care that is taken to protect it from damage during the fire test, type of thermal couple used, method of attachment, etc. Since under-reinforced beams fail primarily due to loss of steel load carrying capacity the more important region for agreement between measured and predicted temperatures is between 38 mm (1.5 in) and 102 mm (4.0 in) from the bottom of the beam where the reinforcing steel would be located. At 64 mm (2.5 in) which is representative of the reinforcing steel centroid the variation in the predicted value is $\pm$ 100°C from the measured at 2 hours while the variation from the measured is + 200°C at 4 hr. Much of this difference can be explained as a result of the material variability that is inherent to the problem (e.g. Fig. 4), i.e., the actual k and ρC_p for the PCA test beams are not known.

Because of the difference between the measured and computed temperature values on the fire boundary a brief study was made of the effect a variation in fire environment parameters has on the heat input vector Q. Figure 8 shows a plot of measured and computed

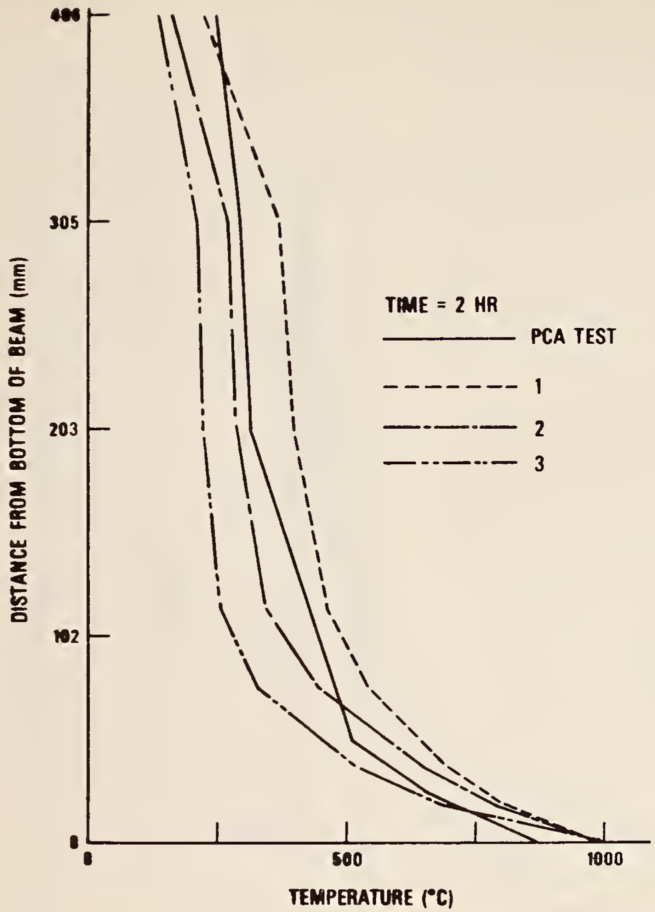


Fig. 6 Temperature Distributions on Center-Line of Cross-Section After 2 Hr. of Fire

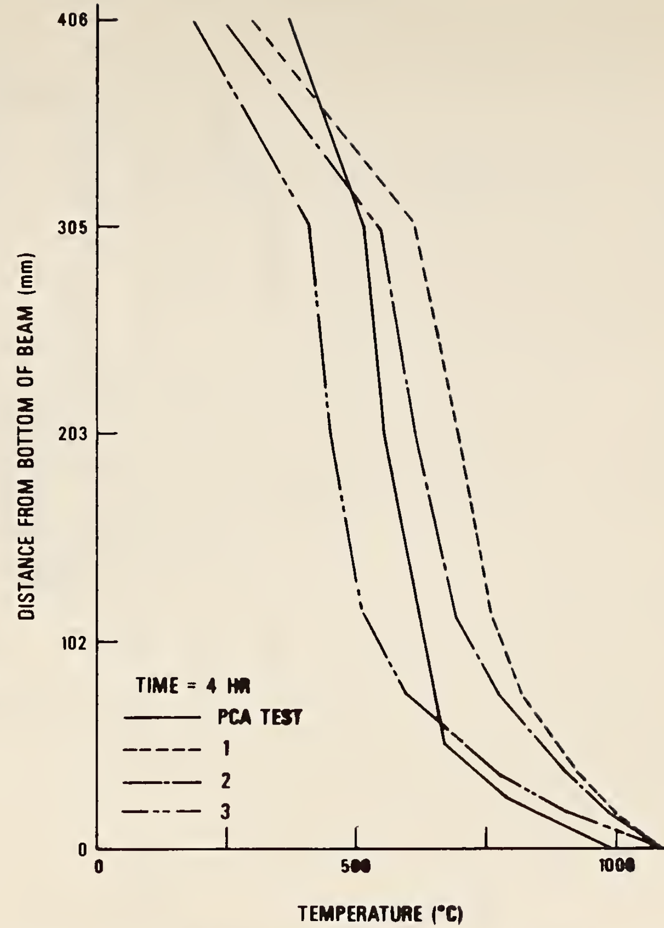


Fig. 7 Temperature Distributions on Center-Line of Cross Section After 4 Hr. of Fire

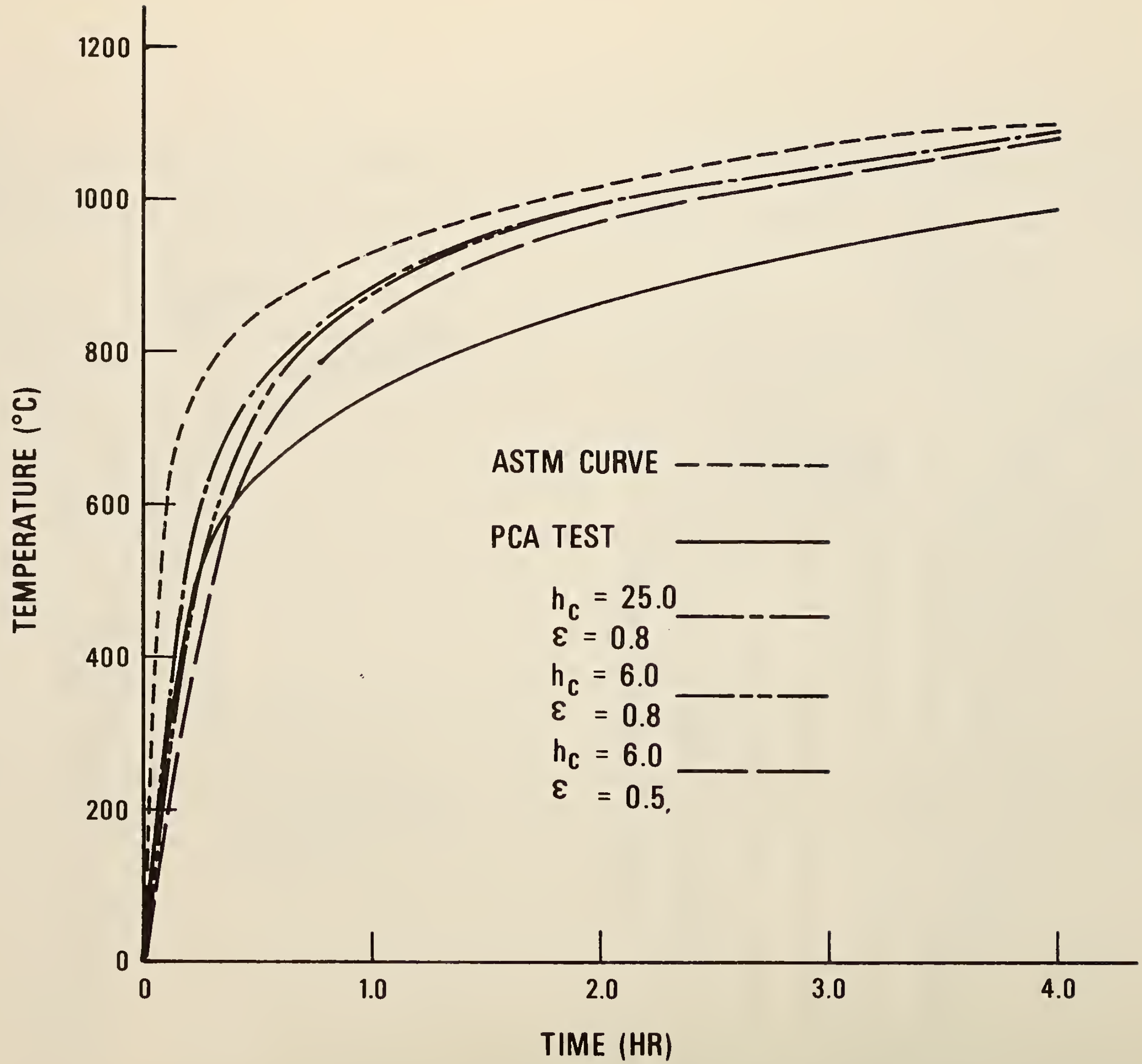


Fig. 8 Effect of Convection and Emissivity Coefficients on Fire Boundary Temperature at Center-Line

temperatures as a function of time at the center of the bottom fire boundary surface. As can be readily seen, there is a significant difference between the measured surface temperature and the ASTM fire temperature. However, the predicted temperatures follow the ASTM fire curve rather closely. A reduction of h_c has very little effect after 1 hr, as might be expected from Eq. 3, while a change in emissivity from 0.8 to 0.5 is significant up to about 3 hours.

It has been found [2] that reinforcing bars have a small effect on the temperature distribution on cross sections when the reinforcement ratio is less than about 0.04. Accordingly, their presence has been ignored in thermal analysis of the lightly reinforced members discussed in the following sections.

4. STUDIES OF STRUCTURAL RESPONSE

A series of simply supported beams fabricated with normal weight concrete and spanning 12.2 m (40 ft) with sections as shown in Fig. 9 were tested by PCA [7] using the ASTM E119 fire. The available test data enable certain aspects of the present thermal and structural analysis to be checked.

As an illustration of this, the predicted midspan deflections of the beam during the first two hours of the ASTM and DATA fires are compared to observed deflections [7] in Fig. 9. The agreement between the predicted and test deflections for the ASTM fire is quite close, the difference being attributable in part to the slightly higher predicted steel temperatures and accompanying

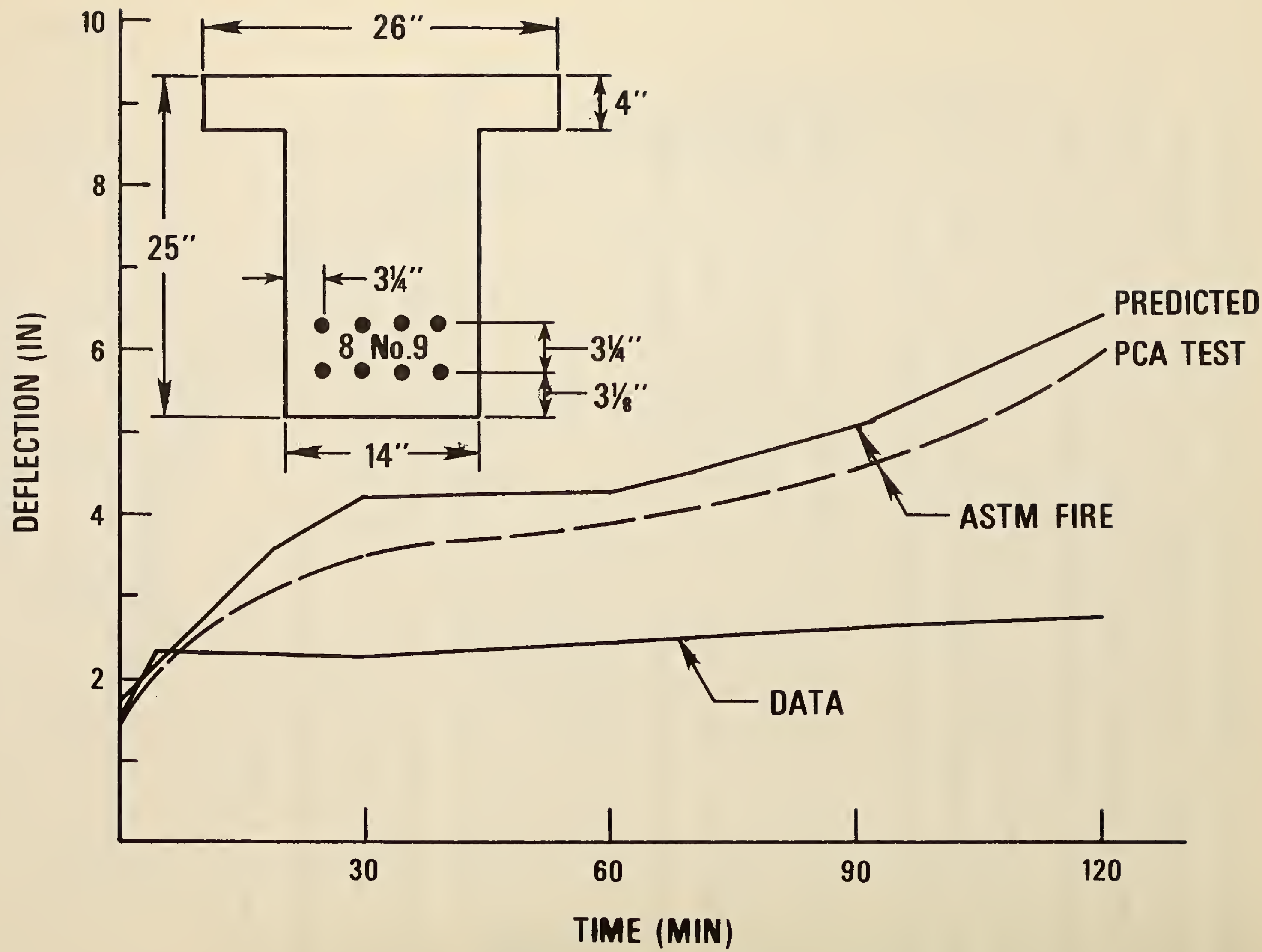


Fig. 9 Comparison of Midspan Deflection of Simply Supported Beam for Different Fire Curves

increased flexibility in the beam. In contrast with the behavior during the ASTM fire, the beam exposed to the DATA fire shows a rapid increase in deflection for the first 25 minutes or so but subsequently its deflection increases very little, due to the effect of the cooling phase. Comparisons of the predicted and observed endurance of the beam under the ASTM fire in terms of ability to carry load were also quite close [6]; predicted endurance ranged from 240 to 370 minutes, depending on material properties, while the test failures range from about 300 to 360 min.

Temperature distributions on the cross section shown in Fig. 10 were developed using the various fire curves shown in Fig. 1. This section would correspond, for example, to a section from a beam-supported slab or waffle slab floor system.

Figure 11 shows how the cross section was descretized to analyze the development of temperatures during the various fires. An additional division in the flange portion of the section, indicated by the dashed line, was introduced to examine the effect of mesh refinement on the temperature distribution in that region. This additional nodal line produced a change of only about 0.5 percent in the temperature distribution resulting from the SDHI-M fire exposures, indicating that a fine mesh is only necessary in regions of high thermal gradients which occur near the fire boundaries.

The accuracy and stability of the solution of Eq. 2 also depends on the time increment used during the solution. The

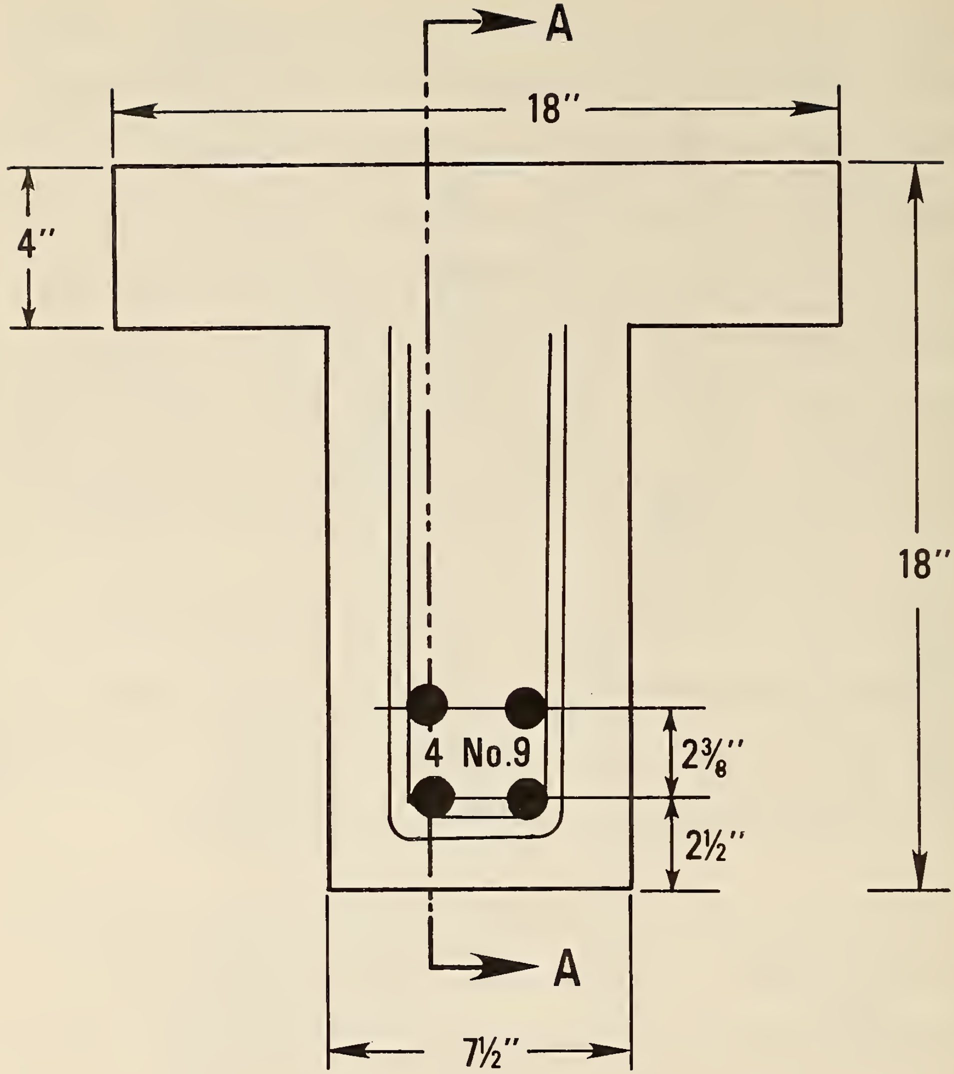


Fig. 10 Typical Beam-Slab Section from Concrete Floor System with High Initial Shear Stress

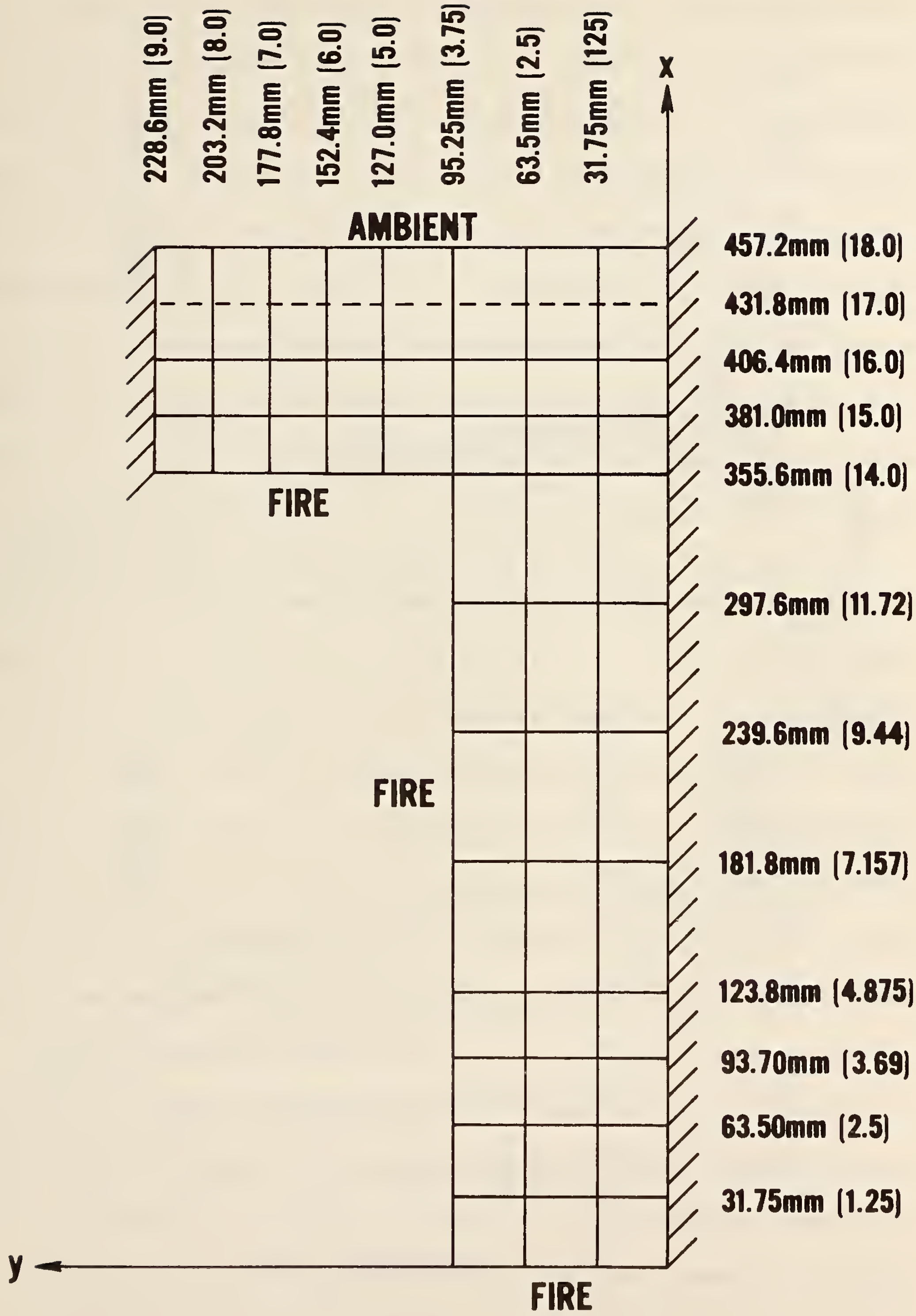


Fig. 11 Cross-Section Descretization for Beam-Slab Section

maximum permissible time increment is determined from a stability criterion which depends on the thermal properties of the material and mesh size. Of course, smaller time increments would be expected to affect the accuracy of the solution. Accordingly, a brief examination was made of the effect that reducing the time increment has on the temperature distribution in a 102 mm (4 in) flange exposed to the SDHI-M fire. A variation in time increment between 30 sec and 1.5 min produced temperature changes of approximately 0.5 percent. Therefore, the time increment can be selected simply on the basis of the stability criterion.

Temperature distributions along vertical sections A-A are compared in Fig. 12 for an exposure period of two hours for the ASTM and SDHI-95 fire curves, the latter being a severe but realistic curve. Temperatures are generally higher than they were for the PCA beam (ASTM fire in both cases) due to the reduced width of the T-stem. The cooling phase of the SDHI-95 fire curve causes significant differences after the hour of fire, particularly in regions close to the fire boundary. The interior of the section continues to heat slowly for a time while the exterior layers are actually cooling due to the decrease in fire temperature. This is similar to the inertial effect in dynamics and occurs because concrete is a relatively poor conductor. Physically, it has the effect of making the thermal strains on the cross section more uniform.

In contrast to the PCA beam (Fig. 9), the reinforcing bar cover here is the ACI minimum of 38mm (1-1/2 in). This decrease in bar

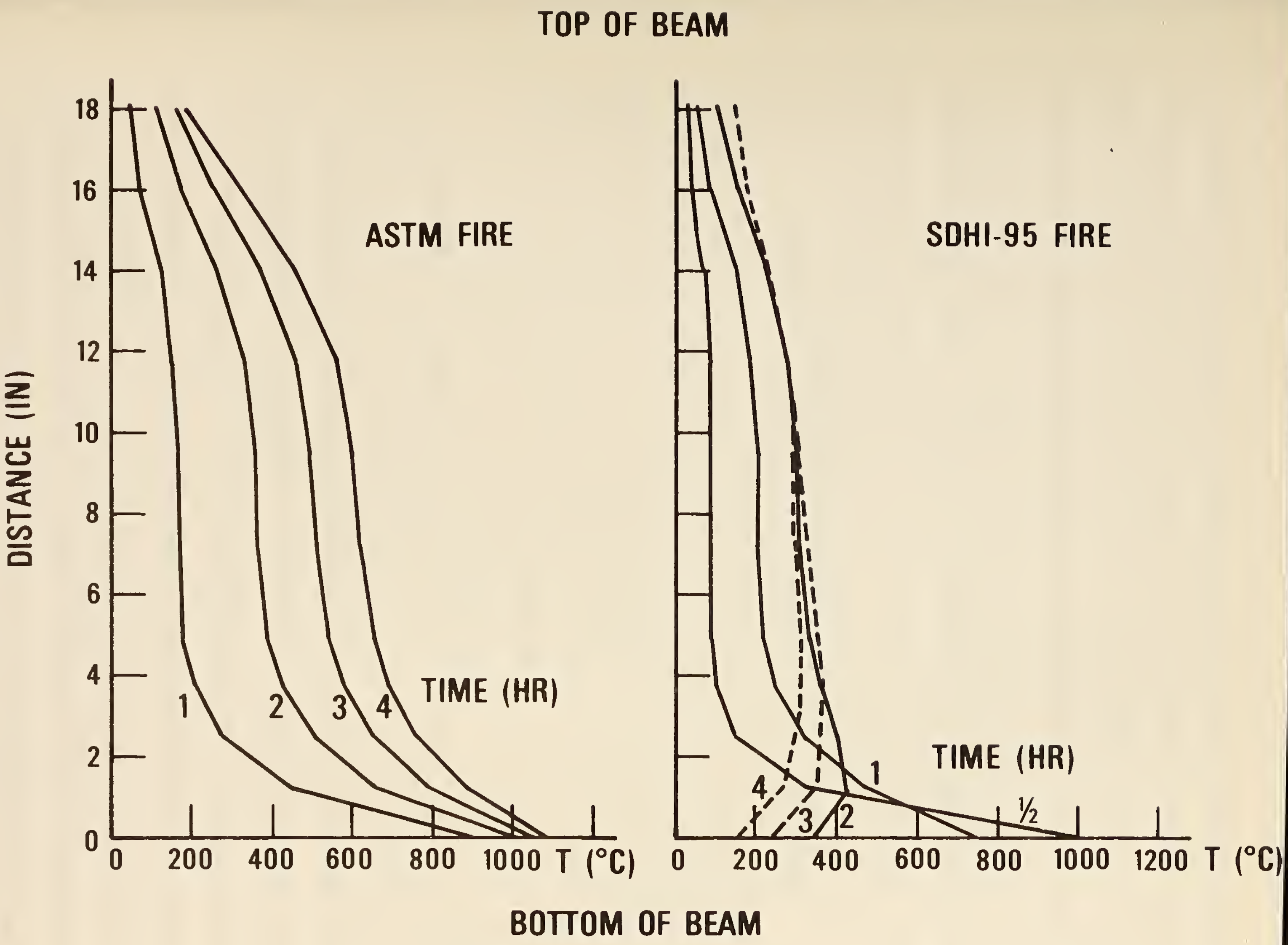


Fig. 12 Temperature Distributions on Section A-A of Beam-Slab Section

cover from 51 mm (2 in) raises the predicted temperature in the lower corner bar after 2 hr of the ASTM fire from the 360°C (680°F) in the PCA beam to 501°C (934°F); after 4 hr, the respective values are 580°C (1076°F) and 757°C (1345°F). The maximum predicted temperature for the lower corner reinforcing bars and time at which it occurs during a fire duration of 4 hr are compared in Table 1 for the different fire curves.

Table 1 - Maximum Reinforcement Temperature for Cross Section in Fig. 10

Fire Curve	Max. Temp °C (°F)	Time of Occurance (min)
SDHI-M	276 (529)	92
SDHI-95	408 (766)	114
LDMI	489 (912)	168
ASTM	757 (1395)	240

Only in the case of the ASTM fire does the steel temperature rise above the 593°C (1100°F) limit which is imposed by ASTM E119 for reinforced concrete members; this limit is reached at 155 min. Assuming that the 593°C (1100°F) has some significance from a limit states viewpoint, this suggests that ratings determined using ASTM fire curve tests for the structural component in Fig. 10 would tend to be conservative in terms of structural behavior during an actual fire.

A comparison of maximum temperatures on the unheated surface of the 102 mm (4 in) slab during fires of 4 hr duration is shown in Fig. 13. It is interesting to compare the time lag between these and the maximum fire temperatures of Fig. 1. Except for the early part of the exposure (less than 2 hr) temperatures tend to be higher for the LDMI than SDHI curves. This is because of the extended time at elevated temperature coupled with the poor thermal conductivity of concrete. The maximum temperature rise of 139°C (250°F) above the initial temperature of 20°C (68°F) allowed by ASTM E119 for floor assemblies is reached at 110 min for the LDMI fire, and is closely approached by the SDHI-95 fire at 125 min. The maximum allowable temperature rise with the ASTM fire is reached at about 90 min; the conventional rating for the slab thus would be 1-1/2 hr, assuming that heat transmission is the controlling criterion of failure.

This does not lessen the value of the ASTM test as a basic comparison tool, since the original purpose of limiting the temperature rise on unexposed surfaces was to prevent the ignition of combustibles in adjoining compartments. However, the face that in approximately 95 percent of the realistic short duration high intensity fire exposures in offices such temperatures would not develop in common reinforced concrete construction raises the question of whether some alternate test procedure ought to be developed for use when evaluating a component's resistance to fire spread in those circumstances. The use of more realistic curves could lead to less costly fire protection.

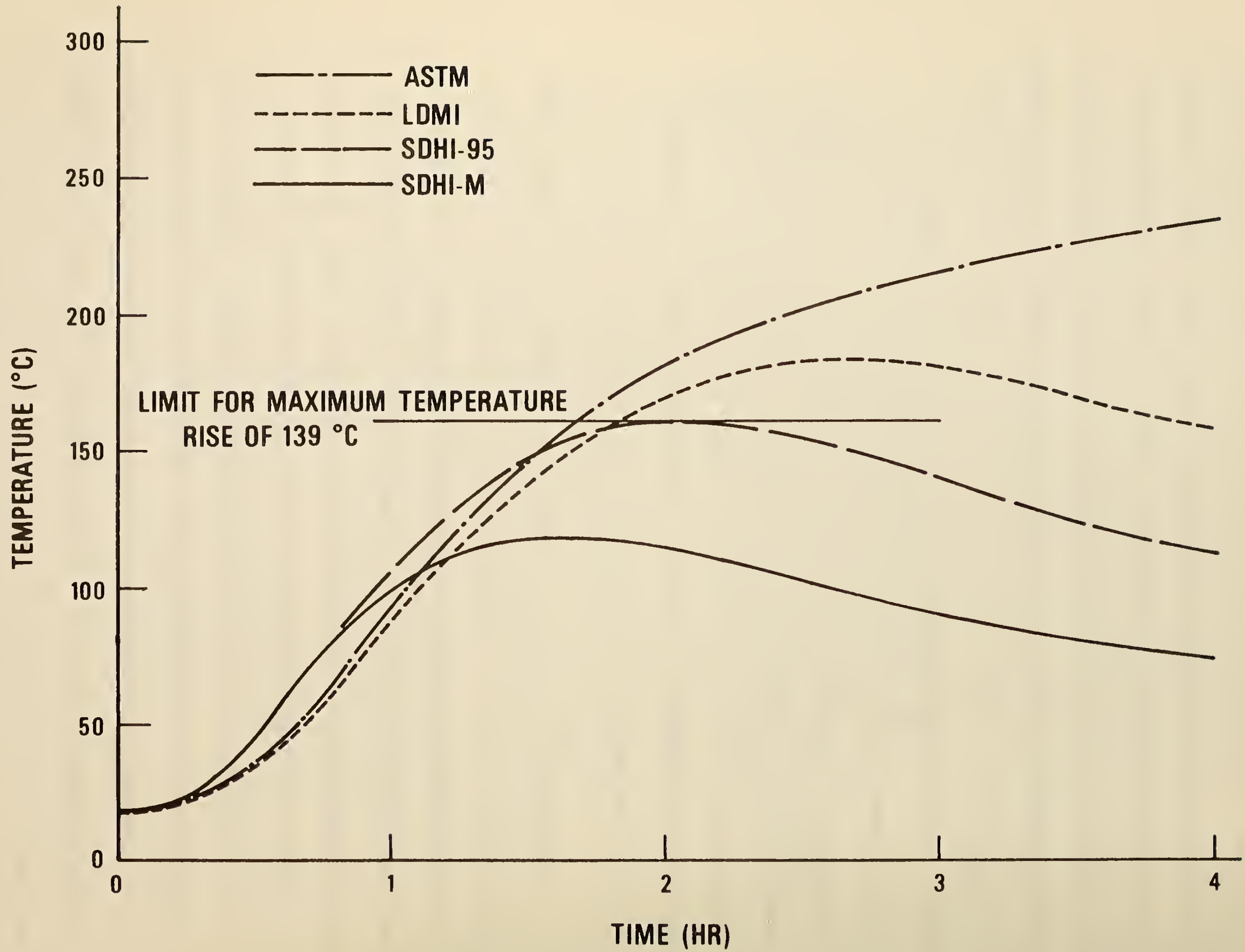


Fig. 13 Maximum Temperatures on Unheated Surface of Beam-Slab Section

The effect of flange thickness on the maximum temperature rise on the unexposed surface of the flange during the SDHI-94 and SDHI-M fires is shown in Fig. 14. Flange thicknesses of 64 mm, 102 mm and 152 mm (2.5, 4 and 6 in) were chosen as typical of the range of thicknesses that might be found in practice. The temperatures on the unexposed surface of the 64 mm (2.5 in) flange are substantially higher than those for the other two thicknesses due to the insulation provided by the increased mass of concrete. The 102 mm (4 in) flange barely satisfies the criterion for maximum temperature rise for the SDHI-95 fire (see also Fig. 13). It follows that a 89 mm (3.5 in) slab would fail to satisfy this criterion for the SDHI-95 fire but would be satisfactory for the SDHI-M fire. Note also that differences between the maximum temperatures reached during each fire decrease with increasing flange thickness.

The inherent scatter in the fire load and associated statistical variability in the time-temperature curve naturally causes variability in the reinforcement temperatures during the fire. Assuming that the fire load is normally distributed [5], the standard deviation in reinforcing bar temperature increases from about 36°C at 1 hr to 73°C at 4 hrs. This variability is in addition to that in conductivity (Fig. 4) and other material parameter uncertainties. When this is viewed along with the scatter in the reduced steel yeild stress at elevated temperatures [3, 6], it is apparent that the member load capacity during an actual fire might be quite variable. These various sources of uncertainty ought to be identified and quantified

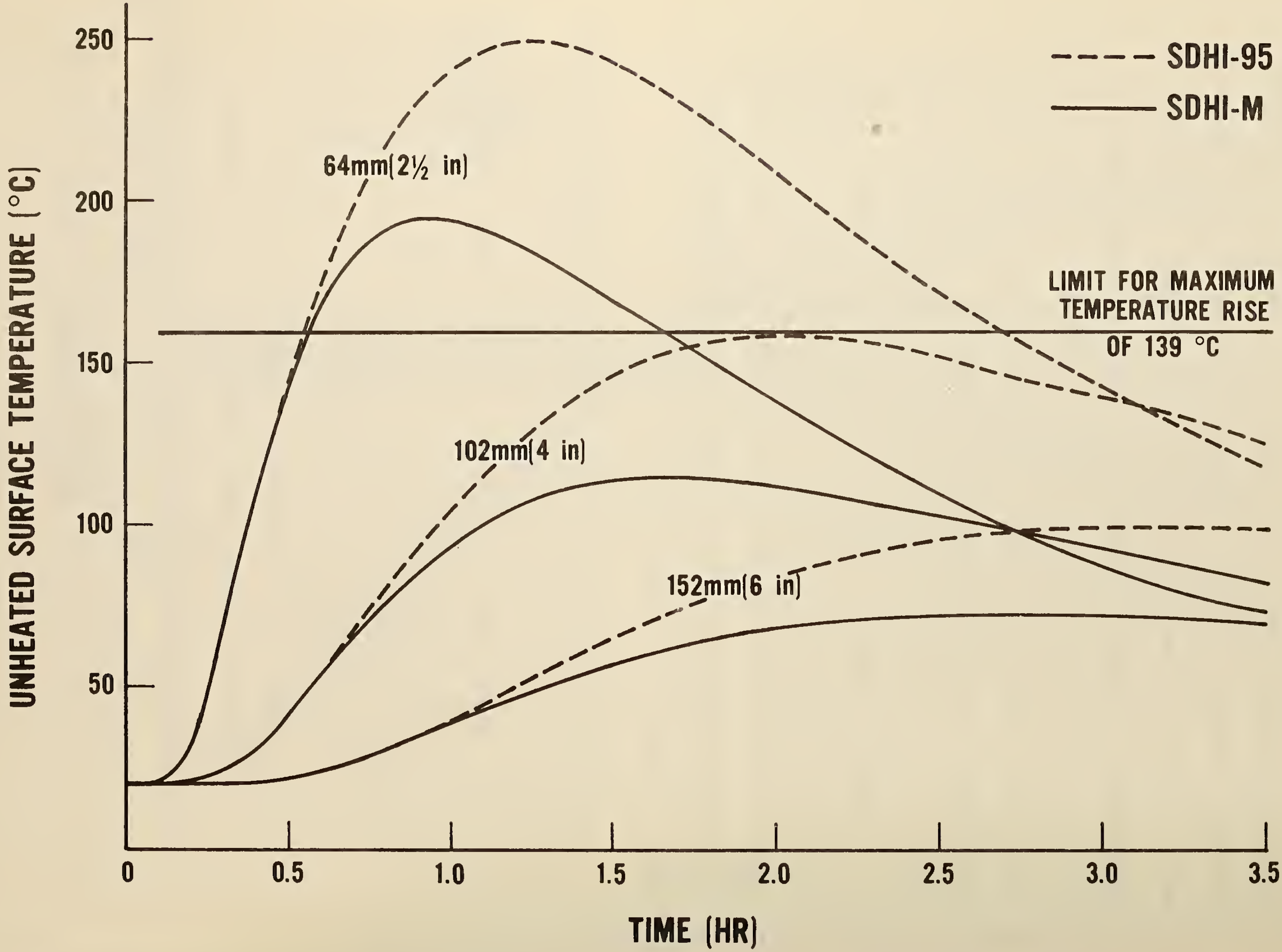


Fig. 14 Effect of Flange Thickness on Maximum Temperatures on Unheated Surface

so that appropriate fire resistant design safety criteria can be developed. Studies leading toward this objective are currently in progress at NBS [6].

Finally, the behavior of a simply supported beam spanning 6.1 m (20 ft), with the cross section shown in Fig. 10 was analyzed. However, in this example, the vertical edges of the flange were exposed to the fire. This beam carried a uniform load of 43.2 kN/m (2.96 k/ft); in contrast to the 12.2 m (40 ft) PCA beam previously studied, the nominal shear stress at room temperature was considerably higher. The beam was fabricated using Grade 60 reinforcement and nominal 4000 psi (27.6 MPa) concrete.

A comparison of the temperature on the unheated surface for the two flange boundary conditions (vertical edges exposed and unexposed) is shown in Fig. 15 for the SDHI-95 fire curve. Temperatures along the centerline (slab and T stem) of the section are virtually the same in both cases due to the insulating properties of the concrete mass. The temperatures in the flange are quite responsive to the fire curve when the flange vertical edge is not insulated. When this edge is insulated, extending the width of the flange does not materially change the maximum surface temperature, and this temperature is virtually constant more than 152 mm (6 in) from the centerline.

The nonlinearity of the temperature distribution (e.g. Fig. 12) has a significant effect on the behavior of beam or column cross

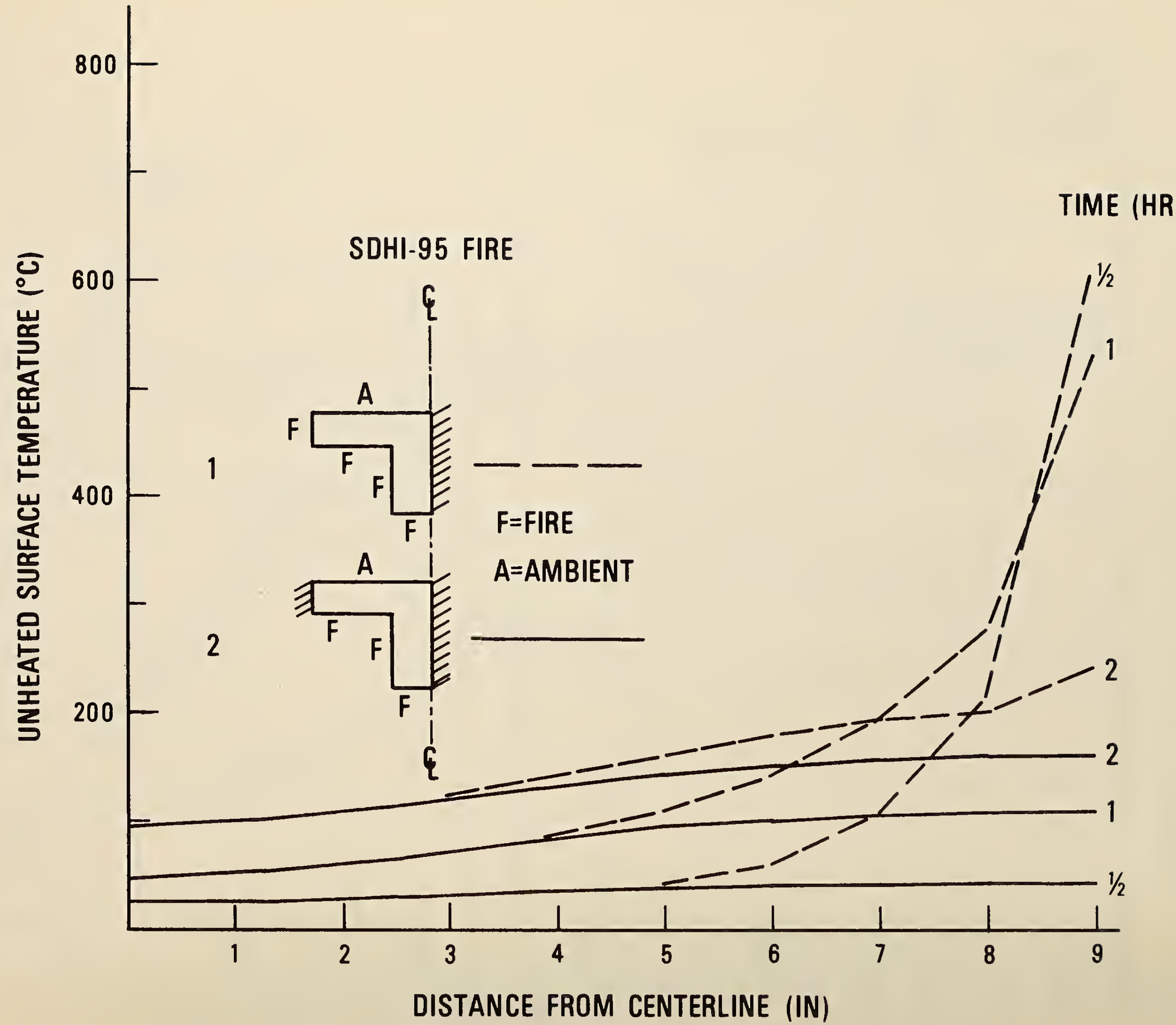
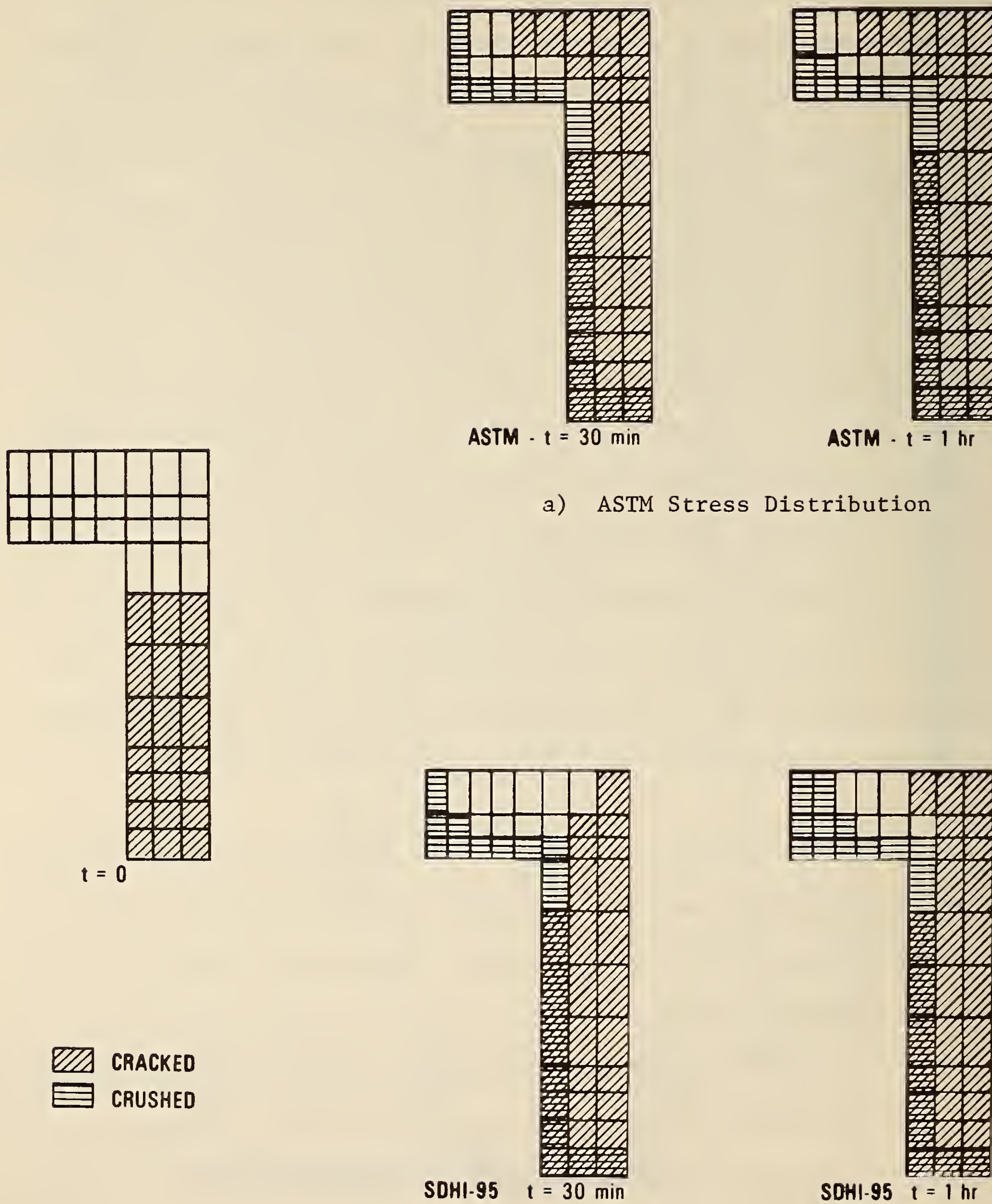


Fig. 15 Unheated Surface Temperature of Beam-Slab Section for Different Fire Boundary

sections. The exterior heated portions of the member tend to develop compressive stresses while the interior, which remains relatively cool due to the poor thermal conductivity of concrete, tends to go into tension to equilbrate stresses. This can cause certain portions of the cross section to crack or crush if the limiting strains of the concrete are exceeded, as shown in Figs. 16a and 16b for a section at the beam quarter span. The cracked portion at t = 0 is simply due to normal flexural cracking. As the fire progresses, portions of the interior of the beam which were previously in compression crack while at the same time elements on the heated boundary begin to crush. This crushing has also been observed in beams fire tested by PCA [7]. Although the degradation of the section does not appear to be as severe for the SDHI-95 fire curve as for the ASTM fire curve, in both cases relatively little of the section remains entirely undamaged.

This degradation of the section can have serious consequences in certain structural members. For very long lightly loaded spans where the shear-moment (V/M) ratio is small, significant degradation of the section can be tolerated. In fact, this was demonstrated during the PCA fire tests [7] where the 12.2 m (40 ft) spans had sufficient rotational capacity in flexure to a form plastic hinge at midspan at failure. However, this may not be possible in shorter spans with higher V/M, where the extensive cracking or crushing may cause a shear failure before the more gradual flexural failure develops. Cracking of the section, itself, does not necessarily imply a significant loss



a) ASTM Stress Distribution

b) SDHI-95 Stress Distribution

Fig. 16 Effect of Stress Distribution on Beam-Slab Section for ASTM and SDHI-95 Fire Curves of shear capacity, as aggregate interlock or interface shear transfer provides a certain amount of shear resistance even in a cracked section. However, when significant portions of the concrete on the heated boundary crush (e.g. Fig. 16) and subsequently are lost, the shear reinforcement loses much of its protection against high temperatures and may lose some of the anchorage required for effective shear resistance. Under these circumstances, extensive cracking of the core becomes a matter of concern.

Recently published guides for fire resistance design (e.g., Ref 13) have emphasized the flexure limit state, wherein the behavior of the structural member is essentially controlled by the temperature in the longitudinal reinforcing or prestressing steel [6]. The present study suggests that this procedure generally would not be sufficient for more heavily loaded structural members; in the latter case, an analysis which considers thermal strain-induced cracking and crushing, as well as steel temperatures, should be carried out in a limit states design procedure.

5. CONCLUSIONS

The calculated performance of reinforcement concrete members exposed to various fire conditions has been examined. Available test data show that reasonable predictions of structural behavior can be obtained using analytical methods currently under development at NBS and elsewhere. Conceptually, therefore, there is no reason why fire resistant structural design cannot be treated in a limited state design context provided that realistic fire curves can be developed.

In this regard, realistic fire curves have been proposed which are based on recent load surveys and fire test data. The ASTM fire curve, represents only one of many possible fire exposures and, based on heat transfer calculations, generally provides a very conservative indication of how a structural member will perform in an actual fire. Design economies could be realized if building standards were to permit the use of realistic fire curves for calculation of structural behavior in fire resistant structural design.

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16. ABSTRACT *(A 200-word or less factual summary of most significant information. If document includes a significant bibliography or literature survey, mention it here.)*

Fire ratings for structural assemblies in the U. S. are currently measured by endurance of or temperature rise in components subjected to a standard test. Analytical procedures show considerable promise for alleviating the extensive testing required, and for placing fire resistant structural design on a limit states basis. In this study, thermal and structural analyses for reinforced concrete members are validated using experimental data. Temperature distributions computed in reinforced concrete members for several realistic fire exposures are examined and compared to the distributions measured in a standard test. Parameter studies show the effect of typical variations in thermal diffusivity, emissivity and conductivity. Structural responses for the different fire exposures are also briefly compared.

17. KEY WORDS *(six to twelve entries; alphabetical order; capitalize only the first letter of the first key word unless a proper name; separated by semicolons)*

Concrete (reinforced); fire load; fire resistance; fire tests structural engineering; temperature; thermal analysis.

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