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STUDY REPORT

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FIRE ENGINEERING DESIGN OF REINFORCED AND PRESTRESSED CONCRETE ELEMENTS

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PREFACE

This report on a project carried out at the Building Research Association of New Zealand describes an investigation into rational design methods for calculating the fire resistance of structural concrete elements.

This report is intended primarily for fire and structural engineers, while parts will also be of interest to code writers.

FIRE ENGINEERING DESIGN OF REINFORCED AND PRESTRESSED CONCRETE ELEMENTS

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ABSTRACT

Advances in knowledge relating to fire engineering design of concrete structures have led to the development of rational design methods as an alternative for time-consuming and expensive full scale fire testing or the use of simple tabulated forms of fire resistance data. Rational design methods are also better able to accommodate the effects of continuity and restraint, and the location of reinforcing or prestressing steel can be optimised, particularly for prestressed construction where prestressing steel rapidly loses strength at high temperatures and elements tend to be more slender. However, rational design methods are unlikely to be warranted where the required fire resistance is less than about two hours for reinforced concrete and about one hour for precast prestressed concrete. This report summarises the existing state of knowledge and discusses possible design procedures ranging from selecting an appropriate design fire, to predicting the thermal and structural response of the member. Recommendations are also made supporting a review of existing New Zealand tabular forms of fire resistance data contained in MP9 for designing concrete structural elements.

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INTRODUCTION

Major advances in fire engineering of building structures have occurred in the past decade with considerable effort made in the understanding of how real structures behave in fire. The knowledge base is by no means complete but the advances that have occurred to date allow a more realistic assessment of the fire performance of structural components. This study summarises the current knowledge with respect to both reinforced and prestressed concrete structural elements although some of the discussion can be readily applied to other materials.

Concrete elements can generally be divided into two types: flexural members such as beams and floors, and compression members such as columns and walls. The ability of loadbearing elements to remain loadbearing is usually a major concern in a fire, while walls, ceilings and floors may also be required to contain fire by preventing its spread directly through the element or by restricting excessive heat transmission from the side exposed to the fire to the side not exposed.

Existing simplified practices used to confirm adequate member sizes or to design new members rely on a number of assumptions, which are necessarily conservative. In a large number of cases these practices will still be quite adequate as fire resistance requirements may not govern the design. However, there may be instances where member design is over conservative where factors such as the beneficial effects of continuity and restraint are not currently adequately assessed, or unsafe where the effects of restraint have been incorrectly assessed.

Much of the early work and effort put into investigating the fire performance of concrete members was conducted in the United States on behalf of the prestressed concrete industry. Prestressed concrete beams were known to be more susceptible to fire damage due to the rapid deterioration in strength of cold-worked prestressing tendons and wires at high temperature. Member sizes for prestressed elements also tend to be more slender than for reinforced construction. Fortunately, most of the work on prestressed concrete can easily be extended to include reinforced concrete construction.

DESIGN APPROACHES AND METHODOLOGIES

Descriptions of the various design approaches that may be adopted for the overall determination of fire resistance of structural elements are given by Malhotra (1982), Bennetts (1982) and CIB W14 (1986) which discuss the merits of different heat exposure and structural response models. A design manual for the fire safety of steel structures has also been published (ECCS, 1985).

The following provides an overview of various design approaches - in the order of least to most functionally based. A graphical summary is provided in Figure 1.

Method 1. Traditional approach based upon fire resistance ratings specified in fire regulations and standard fire resistance testing. The fire resistance requirements must be satisfied by providing evidence or proof of compliance with results from a standard fire resistance test. "Deemed to satisfy" tables of minimum section dimensions and cover to steel, based upon analysis of a large number of fire resistance tests may also be used as an alternative to demonstrate compliance with the fire resistance requirements for building code purposes.

Method 2. Semi-theoretical approach still based upon satisfying specified fire resistance ratings (as for Method 1.) but with an analytical calculation of temperature within the structural element using a standard design fire and load (e.g., ISO 834). The fire resistance is determined such that the temperature of reinforcing or prestressing steel is limited to a specified critical value. Tabular "deemed to satisfy" data may also be developed and used to simplify or eliminate the need for calculation.

Both Methods 1 and 2 usually assume that the structure is subjected to the full design load at the time of the fire.

Method 3. As for Method 2, except that the analytical determination of fire resistance is based upon an assessment of the loadbearing capacity of the element at elevated temperature from first principles, conducted to establish that loadbearing capacity is maintained for the duration of the standard fire, but considering the likely load on the structure at the time of the fire.

Method 4. A procedure based upon a natural fire process (using ventilation, compartment thermal properties and fire load characteristics to determine fire duration) instead of traditional classification, but representing the natural fire duration as an equivalent time of standard fire exposure. An assessment of the element at elevated temperature is then conducted to establish that loadbearing capacity is maintained for the equivalent duration of the standard fire, again considering the likely load on the structure at the time of the fire.

Method 5. A procedure based upon a natural fire process (using ventilation, compartment thermal properties and fire load characteristics to determine fire duration). An assessment of the time to failure of the element at elevated temperature is conducted to establish that loadbearing capacity is maintained for the duration of the natural fire, again considering the likely load on the structure at the time of the fire.

The current approach adopted in New Zealand is primarily method 1 above. The fire code, New Zealand Standard NZS 1900: Chapter 5: 1988 Fire Resisting Construction and Means of Egress (SANZ, 1988) specifies fire resistance requirements for structural elements depending on: occupancy (or fire risk), location and function of element, location of the compartment, area and number of storeys of the fire compartment. In order to demonstrate compliance with the specified fire resistance requirements, "notional" means of compliance data in simple tabular form are used. The tabular data, found in SANZ Miscellaneous Publication No 9 (MP9) (SANZ, 1989 A) generally specifies minimum element dimensions and cover to reinforcing or prestressing steel.

Although Method 1 is the usual approach in New Zealand, there have reportedly been instances where rational fire engineering design (M3 to M5) have been allowed by approving authorities. These instances have generally been dealt with on a case by case basis.

Also, in recent times, Method 3 above has been used for calculating the fire resistance of concrete slabs acting in composite with profiled steel sheet decking. The thermal response of the concrete slab is estimated using simplified tabular data for the temperature after a specified duration at a specified depth. The procedure (Clifton et al, 1988) is based on ECCS recommendations (Twilt, 1984) with some modification for New Zealand use, and is referenced as an acceptable means of compliance procedure in MP9.

The adoption of any one method is not necessarily the best approach. It is likely that a two-tier system would be more desirable. Such a system should make use of simple tabular data for designers to demonstrate compliance with code requirements but it should also include an alternative means of calculation, using anticipated loads and structural engineering principles to assess loadbearing capacity, for those fewer cases where such an approach is desired or warranted.

Malhotra (1982) points out that in practice, detailed rational design and calculation of fire resistance for up to about two hours is unnecessary for reinforced concrete as the concrete member sizes required to satisfy general loading requirements may possess sufficient fire resistance already. In this case, a simple check against tabular data is all that is required.

Forrest and Law (1984) further suggest that the minimum practical width of a reinforced concrete beam is 200 mm, and this is likely to be also true in New Zealand. However, for precast prestressed construction this minimum width can be in the order of 90 mm for the ribs of prestressed double tee sections for example. Cover to steel in structural concrete is also controlled by requirements other than fire resistance, such as durability. The concrete design code (SANZ, 1982) gives minimum cover to principal longitudinal reinforcement as shown in Table 1. The minimum required cover ranges from 30 mm for precast construction protected from the weather and using bars with diameters no greater than 12 mm, to 50 mm for cast-in-place construction exposed to the weather.

By examining the tabular data for fire resistance in New Zealand in MP9 (SANZ, 1989A) and using 30 mm minimum cover and 200 mm minimum member width it can be seen that reinforced concrete members would in practice possess an inherent fire resistance of about 60 to 90 minutes or about 120 minutes for cast-in-place construction exposed to the weather. Therefore rational design methods would not be warranted for periods of required fire resistance less than about 90 to 120 minutes for reinforced concrete. This confirms Malhotra's earlier comment. On the other hand, rational design may well be justified for prestressed members requiring at least 60 minutes fire resistance.

The next section of the report discusses the various parts which make up the rational design approach.

SELECTION OF THE DESIGN FIRE

The Standard Time-Temperature Curve

In New Zealand and many other countries, fire resistance requirements are currently stated in terms of the duration of exposure to a standard fire following a prescribed temperature-time relationship. The relationship described in International Standards Organisation Standard ISO 834 Fire-resistance tests - Elements of building construction (ISO, 1975) is used in New Zealand, and is the same as that used in Australia (SAA, 1990). Other standardised temperature-time relationships are described in (ASTM, 1983) for USA and Canada and in (BSI, 1987) for the United Kingdom. There are also other curves used in Europe and Japan but they are all basically similar to the ISO 834 curve.

The ISO 834 time-temperature relationship, shown in Figure 2, can be represented by the following mathematical expression:

$$T - T_o = 345 \log_{10} (8t + 1) \quad \text{where:} \qquad [1]$$

t = time from the start of test (min)
T = temperature of the furnace gases at time t (°C)
T_o = initial temperature in furnace (°C)

This time-temperature relationship is not necessarily representative of a real or natural fire but it is generally regarded, for building enclosures, as being a "severe fire". Standard fire testing has resulted in vast amounts of data built up over many years and consequently there would be major problems and disadvantages in changing to another fire specification, even if such a fire is more realistic. While such a change in testing practice is likely to be unwarranted, with continued research and experimentation there will remain the scope for using natural fires in the future for theoretical assessments, and the standard fire will continue to serve a useful role in comparing the performance of different building elements under a standard set of conditions.

The cooling or decay period in a fire is usually ignored during a fire resistance test with the rating obtained from the result achieved at the end of the heating period. According to Anderberg (1988), the loadbearing capacity of a fire-exposed concrete structure reaches its minimum during cooling. This thermal lag effect is caused by the temperature within the structural element continuing to rise beyond the end of the heating period, and is a factor which should be accounted for in design methods 3, 4 and 5 described earlier. The inclusion of a cooling phase in the description of a natural fire is discussed in the next section.

Natural Fires

The gas temperature history in fires in rooms or compartments can be shown to depend on the nature and distribution of the fuel, enclosure size, location and size of openings as they affect ventilation, and thermal properties of the enclosure surfaces.

In Sweden, temperature-time curves have been developed (Pettersson et al, 1976) by solving heat balance equations for compartments. The heat produced by the fire is equated to that absorbed by the enclosure surfaces, plus that lost by radiation and convection through the openings. Examples of the curves derived by Pettersson et al, for different fire load densities (fire load per unit boundary surface area) and opening factors, are shown in Figure 3 (from Malhotra, 1982). The curves presented are for a particular fire compartment (with set thermal properties), however, procedures are available for conversion for use with fire compartments with different thermal properties.

Simpler gas temperature-time relationships are described (Lie, 1988) with analytical expressions which can be used to approximate the shape of the gas temperature-time curves. A brief description of these expressions follow. The derivation of the curves for ventilation controlled fires is based on a method described by Kawagoe and Sekine (1963).

The rate of burning, R, (generally the mass loss rate which occurs in the time interval between 80 and 30 percent of the fuel's original weight) of combustible material (wood-equivalent) in an enclosure can be given by:

$R = 330\, A_w\sqrt{H}$ where: [2]

R = rate of burning (kg/hr)
A_w = area of openings in the enclosure (m^2)
H = height of openings (m)

The duration of the fire, D, is then given by:

$D = Q\, A_t \;/\; R = Q\, A_t \;/\; 330\, A_w\sqrt{H}$ where: [3]

D = fire duration (hr)
Q = fire load per unit area of enclosure boundary surfaces (kg/m^2)
A_t = area of internal enclosure boundary surfaces including openings and floor (m^2)

However, the duration of the fire can also be expressed as:

$D = Q \;/\; 330\, F$ where: [4]

F = opening factor = $A_w\sqrt{H} \;/\; A_t$ ($m^{1/2}$)

The increase in gas temperature of the fire as a function of time can be approximated by the following:

$$T = 250(10F)^{0.1\ F^{0.3}}\,\epsilon - F^2 t[3(1-\epsilon^{-0.6t}) - (1-\epsilon^{-3t}) + 4(1-\epsilon^{-12t})] + C\sqrt{600/F} \qquad [5]$$

where:
T = gas temperature (°C)
t = time since start of fire (hr)
F = opening factor defined above ($m^{\frac{1}{2}}$)
C = 1 for light enclosure boundary materials (density < 1600 kg/m^3)
C = 0 for heavy enclosure boundary materials (density > 1600 kg/m^3)

The above expression is only valid for $t \leq 0.08/F + 1$, if $t > 0.08/F + 1$ then let $t = 0.08/F + 1$.

The above expression is also only valid for $0.01 \leq F \leq 0.15$, if $F > 0.15$ then let $F = 0.15$.

When the time of fire duration, calculated in equations [3] or [4] above, is reached, then a new expression is required to account for the decay period. Generally, the rate of decay increases as the fire duration decreases and vice versa. The following expression for gas temperature during decay may be used.

$T = -600\ (t/D - 1) + T_d$ where: [6]

T_d = temperature at time, D, at which decay starts (°C)

Where T < 20°C then let T = 20° C.

An example of a gas temperature-time curve developed from using these expressions is shown in Figure 4. Drysdale (1985) compares Lie's theoretical time-temperature curves with Pettersson's et al and notes that while they are not as refined as Pettersson's they can be used to obtain a rough sketch of the course of the fire. The equations can easily be handled by computer and while Lie (1988) notes that they may be somewhat conservative they would be satisfactory if used for design curves for fire resistance.

Equivalent Time of Fire Exposure

As a considerable amount of information has been obtained from conducting standard fire resistance tests it is somewhat attractive to be able to express the effect of natural fires as an equivalent period of exposure to a standard fire resistance test.

Harmathy (1987) compares the methods of Ingberg, Law, Pettersson, DIN 18230 and his own normalised heat-load method and concludes that the latter is the more accurate of the five with the added advantage of being able to be easily applied in probabalistic design.

The various methods of determining equivalent fire severity are described by Malhotra (1982), Kirby (1986) and Shields and Silcock (1987), while Wickstrom (1985) describes how natural fires can be expressed using the standard fire curve. Pettersson also gives a useful summary of assessing fire severity using the concept of equivalent time of fire duration in the publication by FIP (Cement and Concrete Association, 1978). It is not the intention of this report to discuss these methods in further detail and the reader is referred to the above references for further information.

FIRE TESTING VERSUS ANALYTICAL MODELLING

Fire Testing

The main advantage of fire testing is in providing a real test of the proposed construction. Unfortunately it can only do this for a very limited set of conditions, if a large number of tests are to be avoided. Disadvantages of fire testing include: furnace design and capacity may not be able to accommodate the full-sized test specimen; fire tests are expensive and time delays may be involved before results can be obtained and used; a large number of tests may be required to examine construction variations; and differences in furnace design can make the comparison of results obtained from different furnaces difficult.

Analytical Modelling

Analytical modelling addresses some of the deficiencies attributable to fire resistance testing. It enables a large number of variations to be assessed at a relatively modest cost; it is able to extend the results of fire testing by interpolation or extrapolation; it can deal with the cooling process and its effects more readily; results may be available within a short period; and it enables a more realistic consideration of the effects of restraint and continuity or other changes to boundary conditions. The main disadvantage of analytical modelling at the present time is that comprehensive models are not widely available, and they require fire engineering expertise to be available to undertake the modelling and interpret results. At the present time such expertise is not widespread within New Zealand or elsewhere.

Before analytical methods can be widely adopted, an assessment of their ability to represent performance in real fires is required. Validation studies, comparing model prediction with fire test results is a necessary part of this assessment process and therefore the need for fire resistance test facilities will continue.

FAILURE CRITERIA

Fire Testing

There are three categories of test failure (or limit states) commonly used in fire resistance testing. They are loadbearing capacity (also known as structural adequacy or stability), integrity, and insulation.

Only loadbearing capacity is applicable to loadbearing beams and columns. In its simplest form, this means that the member must not collapse during the fire test. However, there are two other means which are sometimes used in combination with the above "collapse" state. Limits may be placed on the maximum deformation or rate of deformation of the member or limits may be placed on the maximum temperature of the reinforcing or prestressing steel during the test.

For separating elements such as walls or slabs, in addition to loadbearing capacity, the criteria of integrity and insulation are also usually applied. The purpose of integrity is to prevent fire spreading through holes or gaps which may develop, while insulation is intended to prevent fire spread due to excessive heat transmission through a separating element leading to the ignition of combustible materials on the unexposed side.

Performance criteria used by various fire resistance test standards for different types of element are shown in Table 2. The standards considered were: ASTM E119 (ASTM, 1983), BS 476 Part 20 (BSI, 1987), ISO 834 (ISO, 1975) and AS 1530 Part 4 (SAA, 1990). The latter three are generally accepted for use in New Zealand.

The controls on deflection or rate of deflection included in some of the above mentioned standards have generally been derived in order to pre-empt actual collapse of the loadbearing member. This is advantageous to prevent damage to the fire testing furnace and associated equipment, but has little relevance to real building fires.

Rational Design Methods

The principal failure criterion used in rational design methods is that of the ultimate loadbearing limit state. Most commonly, failure is assumed to occur in horizontal flexural members when the reduced moment capacity of the member due to the effects of elevated temperature becomes less than the resulting moment due to the applied load. This assumption relies on the mode of failure being in flexural tension rather than in compression or shear. These methods will be discussed later in this report.

ELEVATED TEMPERATURE MATERIAL PROPERTIES

The properties of concrete and reinforcing and prestressing steel at temperatures likely to result from a fire are of interest in order to provide input data to thermal and structural response models. This subject has been dealt with extensively in the literature and useful summaries have been given by Abrams (1979), Lie (1972), Malhotra (1982), Schneider (1985), CEB (1987) and Harmathy (1988). However, there are still gaps in current knowledge and data from different sources can be highly variable. Only a brief summary of the information required to be used in conjunction with thermal and structural response models will be given here. The reader is referred to the above references for more detailed explanations or descriptions of properties not specifically included in the following discussion (e.g., thermal expansion and creep behaviour).

Concrete

The thermal conductivity of concrete depends on the type of aggregate used, the moisture content and the porosity of the concrete. The thermal conductivity generally increases as the concrete density increases as illustrated in Figure 5 from Lie (1972).

Specific heat (also known as specific thermal capacity) generally increases with temperature, with aggregate type having only a small influence. Figure 6, again from Lie (1972), shows the variation of specific heat with temperature.

The product of specific heat and density is called volumetric heat capacity and together with thermal conductivity is used as material property input to thermal response models. The ratio between thermal conductivity and volumetric heat capacity is called thermal diffusivity and is a measure of the rate at which heat is transferred through the material.

Information about the change in compressive strength of concrete with temperature is required as input to the assessment of loadbearing capacity of a structural concrete element. Abrams (1973) conducted a detailed study of the compressive strength of concretes and found that the original strength of concrete had little influence on the percentage of strength retained at high temperatures. Figure 7 from Abrams (1973) shows data for siliceous, carbonate and sand lightweight aggregates. Figure 8 from BS 8110 Part 2 (BSI, 1985) shows a recommended design curve for compressive strength reduction for dense and lightweight aggregate concretes. Similar (but not identical) design curves are given by CEB (1987) and Malhotra (1982).

New Zealand Concretes

The Building Research Association of New Zealand (BRANZ) has undertaken a study of the fire performance of New Zealand concretes (Woodside, de Ruiter and Wade, 1991) in order to confirm or update requirements in MP9 (SANZ, 1989 A). Experimental work, comparing 130 mm thick unloaded concrete slabs of differing aggregate type and mounted vertically, currently indicates that the fire performance of New Zealand concretes is in close agreement to overseas concretes of similar density and type.

Comparing fire resistance of New Zealand concretes with overseas data can be complicated by variations in the characteristics of the various furnaces used internationally in fire testing which could mask differences that may exist between the performance of concretes from different countries but of the same basic type e.g., siliceous.

There has been no similar work done on the performance of New Zealand structural concrete beams and columns compared with overseas data. However, if the overseas performance of concrete slabs can be shown to be similar to New Zealand performance, then it is reasonable to assume that the performance of concrete beams and columns will also be similar. Furthermore, New Zealand is a small country with limited resources in this area, and it is unlikely that extensive testing of beams and columns for comparison with overseas results could be justified.

Steel

As it is usual to assume that the steel temperature is the same as the temperature of the surrounding concrete, for the purposes of determining the temperature distribution in the member, only the mechanical properties of the steel will be considered here.

It has been observed that the decrease in strength with increasing temperature is more rapid for prestressing steels than for reinforcing steels. Reinforcing bars are usually hot-rolled mild steels while prestressing wires are usually cold-drawn steel. Prestressing tendons may be high strength alloy bars. Figure 9 from Fleischmann (1988) shows the strength variation with temperature for three American steels. A recommended design curve from BS 8110 Part 2 (BSI, 1985) is shown in Figure 10. A similar design curve for structural steel in MP9 (SANZ, 1989A) is also shown on Figure 10. Due to differences in the structural design methods used for reinforced and prestressed concrete, it is the variation with temperature of the yield strength of reinforcing steel and the ultimate strength of prestressing steel that is mainly of interest.

THERMAL RESPONSE OF CONCRETE ELEMENTS

On selecting a design fire, (if regulations permit a choice) the next step is to consider what effect it will have on the building structure so that the mechanical properties of steel and concrete can be used for structural design akin to that normally undertaken at ambient temperature. This section of the report will consider available techniques for calculating temperature profiles as a function of time, through structural concrete members.

Theoretical Calculation

Heat transfer theory can be applied to predict the thermal response of a structural element, exposed to a prescribed fire environment. The principles of this approach (from a fire viewpoint) are explained by Shields and Silcock (1987), Drysdale (1985) and Malhotra (1982). The transfer of heat into the structure is mainly by modes of radiation and conduction and will depend on the temperature of the fire gases, the thermal properties of the structural material, and heat transfer properties of the enclosure boundaries and surfaces of the heat-receiving elements.

According to Malhotra (1982), heat transfer calculations for simple cases where thermal properties are not temperature dependent can be made without much difficulty. But since thermal properties of most materials are temperature dependent, the calculations can become very complex, requiring the use of numerical methods which are best handled using computers for speed, ease of use and convenience. Generally the presence of the steel in a concrete element is ignored for heat transfer purposes. The temperature of the steel is assumed to equal the temperature of the concrete at the location of the steel. This is likely to be a conservative assumption as noted by Ellingwood and Shaver (1976) who say that steel temperatures in beams estimated from those in the surrounding concrete can be as much as 40% too high. They explain that the steel acts as a heat sink and longitudinal conductor and that moisture in the surrounding concrete condenses around the steel providing a layer of insulation.

Computer Programs

There are a number of computer programs developed which use these numerical methods to provide information about the temperature distribution across the member cross-section. In general, the cross section of interest is divided into thin layers (one-dimensional) or quadrilaterals or triangles (two-dimensional) or cuboids (three-dimensional). For each layer or node, a heat balance equation is formulated and solved using a time-step integration scheme. The better known computer programs are presented here.

PC-TEMPCALC (Anderberg, 1988, 1989) is a two-dimensional finite element program developed by the Institute of Fire Safety Design in Sweden. It has been developed with commercial applications in mind and is reportedly easy to use with good presentation of results. The program is written in Fortran 77 and can be used on IBM compatible PC or AT personal computers.

TASEF (Wickstrom, 1979) is also a two-dimensional finite element program available from the Swedish National Testing Institute. An explicit forward difference time integration scheme is used. It is similar to PC-TEMPCALC in terms of basic theory so ought to produce much the same results although it is reportedly not as easy to use. TASEF can also be used on IBM compatible personal computers.

FIRES-T3 (Iding et al, 1977) is a three-dimensional finite element program developed at the University of California, Berkeley, for thermal response of fire-exposed structures. An implicit backward difference time integration scheme is used and the evaporation of moisture in humid concrete is not able to be analysed accurately. The program is not readily available on personal computer.

These types of computer-based analysis techniques are able to adjust boundary conditions to enable, typically, one-sided exposure of slabs or walls, three-sided exposure of beams and three or four-sided exposure of columns to be considered.

PC-TEMPCALC and TASEF can account for phase changes and chemical reactions by adjusting the material thermal properties. Lim (1975) has also developed a three-dimensional transient non-linear finite element thermal analysis program, Wakamatsu (1987) a two-dimensional, finite difference method, and Lie (1977) a finite difference method for calculating the temperature history of protected steel columns or solid concrete beams, columns or walls. Of the latter three, only Wakamatsu's method considers the effect of moisture evaporation as occurs in concrete elements.

While a simplified one-dimensional heat transfer analysis (Lie, 1978; Munukutla, 1989) is likely to be acceptable for concrete slabs or walls, where the element thickness is small, relative to the width of the element, generally at least a two-dimensional analysis of cross sections through beams and columns is required, where the edge effects cannot be ignored.

In order to illustrate the use of a one-dimensional predictive tool, Figure 11 compares the predicted temperature in a 130 mm thick normal weight concrete slab with experimental data obtained from the BRANZ study mentioned earlier. The model used is similar to that described by Lie and Williams-Leir (1979) with some minor changes incorporated by the Author. The model does not include the effect of latent heat of vaporisation. Latent heat can be considered by artificially increasing the specific heat of the concrete over the approximate range 100°C to 150°C. An example of the model with some adjustment for the effects of latent heat compared with the BRANZ results is shown in Figure 12. Further work is required on the development of this and similar models, particularly for other thicknesses, if they are not to be too conservative. Likewise, the more sophisticated models such as TEMPCALC or TASEF may still require some calibration and validation work, using available data from tests on New Zealand concretes before being widely used in New Zealand.

Alternative Methods

There is a need for simpler methods of estimating the temperature of steel in structural concrete members to be available which do not require detailed heat transfer calculations or computers. Such methods will of course not be as accurate as the numerical procedures. A common way of providing the temperature data is by graphical presentation of the form shown in Figure 13 for concrete slabs (ACI, 1981). The temperature at a depth in the slab can be read directly off the figure for the required period of exposure. This data is derived from measurements taken during large numbers of fire resistance tests by the Portland Cement Association (Abrams and Gustaferro, 1968). Temperature design data of this sort can either be empirically based (i.e., derived from the results of standard fire tests) or analytical (generated from theoretical models).

An empirical expression for the variation in the temperature within a normal weight siliceous aggregate concrete slab is given by Purkiss, Claridge and Durkin (1989) as:

$$T = 558 \log_{10} t - (6.82y + 373.77) \quad \text{where:} \qquad [7]$$

T = temperature in slab (°C) (for $250 < T < 950$)
t = time (min) (for $30 < t < 240$)
y = distance from fire-exposed face (mm)

Unfortunately, beams and columns are a little more complex because they are usually heated from more than one side and temperature distributions depend on the width of the beam as well as the distance from fire exposed surfaces. Graphical means can still be used to estimate steel temperature but the process may involve a transformation of the information. Figure 14 (ACI, 1981) shows the temperature distribution along the centre-line of a structural concrete beam, for various widths of beam. From this, it is possible to construct an isotherm diagram for the temperature distribution throughout the cross-section. The procedure is described by ACI (1981).

Similar graphical methods are described by Gustaferro and Martin (1977). Examples of isotherm diagrams for beams have also been published by Comite Euro-International Du Beton (CEB, 1987), Abrams (1979) and Malhotra (1982) who provides some useful graphs, reproduced in Figure 15, for temperature distribution in dense rectangular concrete beams. For lightweight aggregate concrete, it is suggested (Abrams, 1979; Forrest and Law, 1984) that the temperature corresponding to the distance from the exposed face of a dense or normal-weight concrete be reduced by 20%. In applying these charts, Harmathy (1979) cautions that the type of concrete, moisture content, and the design of the test furnace are known to have a substantial influence on the temperature history of beams in fire tests.

Lie (1972) describes an analytical method which uses graphical information and calculation to determine temperatures in slabs, rectangular beams and columns or circular columns. The method is an approximate one which uses heating at a constant temperature equal to the average value of the standard time temperature curve. Wickstrom (1987) has also developed some simpler analytical expressions to approximate the results from TASEF.

STRUCTURAL RESPONSE AND DESIGN PROCEDURES

Concrete structures have a very good record for their performance in fire. There are likely to be a number of reasons for this:

(a) Standard fire resistance testing has traditionally treated structural elements in isolation, treating failure of an individual element as unacceptable, even if the structure as a whole remains satisfactory due to the presence of structural redundancy. In a real building, the failure of one structural element is not necessarily indicative of structural collapse of the whole building.

(b) At higher temperatures concrete elements become more flexible, due to a reduction in elastic modulus and are therefore capable of greater structural deformations.

(c) The imposed load may be much less than the design load assumed in the determination of fire resistance (see next section).

Bresler (1976) states that the structural response of concrete structures exposed to fire depends on variations in thermal coefficients of expansion, stress-strain relationships, creep, inelastic deformations associated with unloading, and fracture. He notes that cracking of the interior concrete due to thermal gradients greatly reduces strength and stiffness and the phenomenon is influenced by fire characteristics such as rate and duration of heating, peak temperature and rate of cooling.

Structural Design Loads for Fire

In standard fire resistance testing, loadbearing structures are usually loaded to produce their maximum permissible stresses, thus assuming that the full design load is present at the time of the fire. The Institution of Structural Engineers (ISE, 1975) attributes one of the reasons for the good performance of many concrete structures to the fact that the imposed load on the structure in a fire is often much less than the full design load.

In New Zealand, the stringent design and detailing requirements for earthquakes mean that in many cases, the actions of gravity loads on structural elements are relatively small in comparison with earthquake and wind load actions. Therefore, under fire conditions, there is an additional margin of safety commonly present which leads to increased fire resistance.

As fire can be considered an accidental load, then its simultaneous occurrence with other accidental loads can generally be ignored. Buchanan*

** "Loads for Structural Design of Fire Resisting Structures" (draft) - readers are referred to Dr A Buchanan, University of Canterbury, NZ.*

discusses loads for structural design of fire-resisting structures and includes a comparison of existing load combination requirements and recommendations contained within a variety of different documents published in New Zealand and overseas. He recommends that where loadbearing elements are required by the fire code to have fire resistance, then these elements should have sufficient ideal strength and stability to resist the following combinations of loads:

(1) $D + F_T + \Psi L$
(2) $D + F_T + 0.33\ W$

where: D = dead loads
L = live loads
Ψ = live load combination factor (0.6 for storage occupancies and 0.4 for all other floors and roofs)
W = wind loads (for buildings higher than 15 m or as otherwise required by the fire code)
F_T = fire induced forces or internal stresses (e.g., thermal expansion and thermal thrust)

Buchanan's recommendations are a further development on those currently proposed in the draft loadings code (SANZ, 1989 B) as follows:

(1) $D + \Psi L$
(2) $D + 0.5W$

(D, L, W, Ψ, as defined above)

Both Buchanan's and SANZ's (1989 B) recommended load combinations for the fire limit state include a wind load component, while most overseas documents tend to ignore wind loads.

Structural Behaviour of Beams and Slabs

The possible ways in which a structural concrete beam or slab might fail when heated on the tension side include: 1) flexural failure, being either formation of a plastic hinge when the yield strength of the reinforcement is reduced to the value of its working stress (as a result of the elevated temperature) or rupture of the prestressing steel at the bottom part of the beam; 2) a failure in shear; and 3) bond or anchorage failure. The last two would rarely be expected to occur in practice.

The rational design methods discussed in the following sections primarily rely on the first mode of failure i.e., in flexure, thus the designer needs to ensure this is the most likely mode in the general structural design of the member. Adequate detailing of the reinforcement is also very important and designers are referred to a publication by the Institution of Structural Engineers (1978) which considers this subject in some depth as well as other possible but less likely failure modes (e.g., shear or combined shear and flexure).

Simply-Supported Beams and Slabs

Simply-supported reinforced concrete beams and slabs are not commonly used, rather they tend to be tied together with in-situ construction techniques. Simply-supported precast and prestressed elements are more likely to be found in practice.

The expressions used to describe the moment capacity of the beams are described by Gustaferro and Martin (1977), ACI (1981), Abrams (1979), and Gustaferro (1986) and are given in the following discussion.

If the underside of a structural concrete beam is exposed to fire, the bottom face of the beam will expand more than the top face, due to the higher temperatures experienced there, causing the beam to deflect. The tensile strength of concrete and steel will also decrease as their temperatures increase. The point of flexural structural collapse is reached when the strength of the steel reduces to equal the stress resulting from the applied loads.

Moment capacity, $M = A_s F_y (d - a/2)$ for reinforcing steel [8]

$M = A_s F_{ps} (d - a/2)$ for prestressing steel, where:

A_s = area of reinforcing or prestressing steel
F_y = yield stress of reinforcing steel
F_{ps} = stress at ultimate load in prestressing steel
d = distance between centroid of steel and extreme compression fibre
a = depth of the equivalent rectangular stress block
$= A_s F_y / 0.85 f'_c b$ or $A_s F_{ps} / 0.85 f'_c b$
f'_c = specified compressive strength of concrete
b = width of the beam

For prestressing steel $F_{ps} = F_{pu} [1 - A_s F_{pu} / 2 b d f'_c]$ where: [9]

F_{pu} = ultimate tensile strength of prestressing steel

To calculate the reduced moment capacity due to fire, the value of F_y or F_{ps} at the applicable elevated temperature is used. The values of A_s, d and b usually remain unchanged at elevated temperature where the compression zone is protected from the fire (e.g., by a ceiling/floor slab), except that if the compression zone of the concrete is heated above 760°C (1400°F), the concrete above this temperature should be ignored in the calculation, and reduced values of f'_c, b and d ($f'_{c\theta}$, b_θ and d_θ) should be used. The subscript θ indicates the effect of high temperature.

The procedure outlined in CEB (1987) is similar but varies in the following respects: (1) Concrete heated to above 500°C is ignored in the calculation of loadbearing capacity (i.e., values of b and possibly d need to be reduced as shown in Figure 16), while concrete with lower temperature can be assumed to retain its ordinary room temperature strength. Thus a step function for concrete compressive strength is assumed changing from 1 to 0 at 500°C. (2) It is noted that using practical design curves for steel strength, as described earlier in this report, can lead to structural design which is too conservative, therefore a critical stress approach is recommended in which F_y in the above expression is replaced with the critical stress given in Figure 17 (CEB, 1987) as a function of steel temperature and a cross section parameter = $A_s/b_\theta d_\theta f'_c$. Readers are referred to the CEB publication for further detailed information.

The Institution of Structural Engineers (1978) also recommends that the maximum depth of the compressive stress block (at elevated temperature) should not exceed 0.5 x effective depth. They utilise a gradual compressive strength reduction of concrete with increasing temperature (as per the PCI method but unlike CEB) but also arbitrarily neglect a surface layer of concrete of about 25 mm in their calculations. The Institution of Structural Engineers (1978) also suggest using an aging factor of 1.2 to be applied to 28-day compressive strength values for concrete in fire design calculations.

CEB (1987) and ACI (1981) indicate that a capacity reduction factor of 1.0 should be used in connection with the calculation of moment capacity in fire design because factors of safety are already included in the fire resistance rating.

Figure 18 shows the applied moment and moment capacity for a simply-supported beam with a uniformly distributed load. Collapse is presumed to occur when the reduced moment capacity at mid-span reaches the value of the applied moment with the formation of a plastic hinge at mid-span.

Purkiss et al (1989) also describe a simple method for calculating the fire resistance of simply supported one-way spanning slabs which takes into account the load level. They also considered the effects of variations in concrete strength, steel strength and temperature profiles and concluded that the largest effect was due to variations in the calculation of temperature profiles, a lesser effect due to variation in steel strength and a negligible effect due to concrete strength.

Continuous Beams and Slabs

A beam continuous over its supports possesses a much greater fire resistance than if simply supported. This is because restraint against rotation provided at the supports causes a redistribution of the applied moments, increasing the negative moment at the supports as the positive moment decreases due to elevated temperature. See Figure 19. The fire will tend to have a greater effect in reducing the positive moments rather than the negative, since the positive moment reinforcement is more exposed to the fire than the negative. Gustaferro and Martin (1977) indicate the procedure that should be followed for checking the strength of a continuous beam. The procedure is summarised here (refer to Figure 20 for moment diagrams and meaning of symbols).

Given a preliminary design of beam -

1. Determine the positive moment capacity, M_θ^+ at time required using equation [8].

2. Determine the required negative moment capacity (after moment redistribution).

   At interior support of an end bay, $M_\theta^- = \frac{1}{2}wL^2 - wL^2 \sqrt{2M_\theta^+ / wL^2}$ [10]

   At support of symmetrical intermediate bay, $M_\theta^- = wL^2/8 - M_\theta^+$ [11]

3. Determine the amount of negative reinforcement (or prestressing steel) needed to provide the required negative moment capacity using equation [8].

4. Determine the position (maximum value) of the inflection points, (X_o) and thus the necessary lengths of reinforcement.

   Within an end bay, $X_o = L - 2M_\theta^- /wL$ [12]

   Within a symmetrical intermediate bay, $X_o = ½L - ½ \sqrt{8M_\theta^+/w}$ [13]

   For maximum value of X_o, the minimum value of the service load (w) should be used. The negative reinforcing bars must be long enough to accommodate the redistributed moments and the change in the position of inflection points. It is recommended that at least 20% of the maximum negative moment reinforcement be extended throughout the span.

5. Ensure that flexural tension governs design.

   To avoid a compressive failure of the concrete, the negative moment reinforcement should be small enough so that:

   $A_s \; F_{y\theta} \,/\, b_\theta \, d_\theta \, f'_{c\theta} < 0.30$ [14]

Restrained Beams and Slabs

In this context, restraint refers to axial restraint against thermal expansion. When a fire occurs in a localised area within a building, heating the underside of a beam or slab, that part of the element will try to expand. The expansion will be resisted by the surrounding cooler structure. The resistive force is known as the "thermal thrust".

Currently the accepted method for calculating thermal thrust is semi-empirical utilising test data from a large number of reference specimens. According to Issen et al (1970) who conducted the early experimental work done at the Portland Cement Association, the thermal thrust varies with the initial modulus of elasticity and the heated perimeter, leading to equation [15]. The following method is included in material presented by Gustaferro and Martin (1977), and ACI (1981).

$$\frac{T^1}{A^1 E^1} = \frac{T^o Z^o}{A^o E^o Z^1}$$

where:
$Z^o = A^o / S^o$
$Z^1 = A^1 / S^1$
S^o = heated perimeter of the reference member
S^1 = heated perimeter of the member of interest
T^1 = maximum thermal thrust in member of interest
T^o = maximum thermal thrust in the reference member
A^o = cross sectional area normal to thrust of reference member
A^1 = cross sectional area normal to thrust of member of interest
E^o = modulus of elasticity of reference member
E^1 = modulus of elasticity of member of interest

The nomographs in Figure 21 (reproduced from Concrete and Cement Association (1978)) are used to solve the expression.

Provided that the line of action of the thrust is below the resultant of the compressive stress block, the thrust will increase the moment capacity of the beam and therefore increase its fire resistance. The moment due to the thermal thrust is equal to the thrust force multiplied by the distance between the line of action of the thermal thrust and the centroid of the compression block.

$$T = \frac{M_t}{(d_t - D - \tfrac{1}{2}a_\theta)} \quad \text{where:} \qquad [16]$$

T = thermal thrust
M_t = moment (required) due to thrust force
d_t = distance between extreme compressive fibre and line of action of thrust
D = the deflection of the slab
a_θ = $(T + A_s F_{y\theta})/(0.85 f'_{c\theta} b_\theta)$ - see equation [8]

The midspan deflection of the beam, D, is required as input to equation [16]. It can be estimated for simply supported beams with minimal restraint using:

$$D^1 = \frac{l^2 D^o}{89 Y^1_b} \qquad [17]$$

D^1 = midspan deflection of beam of interest (m)
D^o = midspan deflection of reference specimen (m) as given in Figure 22 (reproduced from Concrete and Cement Association (1978))
l = beam span (m)
Y^1_b = distance of the centroidal axis to the extreme compressive fibre for the beam of interest (m)

Where the thrusts are greater than minimal, alternative expressions are provided by Gustaferro and Martin (1977). In the calculation of thermal thrust the outstanding input data remaining is d_t which requires knowledge of the location of the line of thrust. The same publication, Gustaferro and Martin (1977) provides some guidance here, and states that fire tests have shown that when only minimal thrust occurs, the line of thrust is near the bottom of the member throughout the fire exposure. If the thrust is greater than minimal, the thrust line will be near the bottom at the start of the fire with its position rising slowly as the fire progresses. Carlson et al (1965) also review the effects of restraint with respect to prestressed concrete.

The above equations can be used, for instance, to determine the expected expansion and thrust forces required to increase the moment capacity of a member to the minimum required for a specified period of fire resistance. In this case, the required increase in moment capacity is known, the midspan deflection and depth of compressive block can be calculated, and the line of action of the thermal thrust can be estimated, allowing the size of the required thermal thrust to be calculated. Figure 21 then uses the thrust parameter and ratio of cross sectional area to heated perimeter to give the ratio ($\Delta \ell / \ell$) i.e., the expected thermal expansion.

An assessment is still required as to whether the restraining elements can withstand the calculated thrust force without deforming more than the amount calculated. This is often the most difficult part of the exercise because it requires knowledge of the extent of fire spread within the building, and the restraining capabilities of the cool part of the structure. If a fire is considered to occupy a whole floor of a multi-storey building, for example, then there is likely to be insufficient structure available to provide the required axial restraint.

It should also be noted that if the line of action of the thrust is above the centroid of the compression block, then axial restraint is extremely unsafe, and any thermal expansion may cause premature failure. This can occur if the deflection is too large or if the member is supported near the top of the cross section (in flange supported double tee sections, for example). The method for estimating restraint which has been described here has recently been questioned by Anderberg and Forsen (1982) who found much smaller thermal strains in a detailed analytical study. This subject still requires further detailed research.

Structural Behaviour of Columns and Walls

Structural failure of columns and walls (compression elements) under fire conditions will usually fall into two categories; compressive failure, and buckling (instability) - as for ambient temperature design. As Malhotra (1982) explains, in normal temperature design, a distinction is made between long and short columns on the basis of slenderness ratio, with the long column susceptible to a buckling mode of failure. During fire exposure, the outer layers of concrete lose strength and their contribution to resisting axial load diminishes, - this effectively increases the slenderness ratio of the column with the possibility of columns which were previously short, now becoming long and exhibiting failure by buckling.

Approximate design procedures similar to that already described for flexural elements for calculating fire resistance of concrete columns and walls are generally not as well advanced as for beams or slabs. However, more sophisticated computer-based analytical methods are being developed. The National Research Council of Canada has been attempting to develop a general method for the calculation of fire resistance of concrete columns and walls which is described by Lie et al (1984). In this method the strength of the element is calculated from a load-deflection and stress strain analysis of the cross section. The calculation of temperature distributions uses a finite difference method similar to those discussed earlier in this report. At the present time, the complete computer-based method is being developed by a software house, before it is released more widely.

Lie and Lin (1985 A) and Lie (1989) found that load, cross section size and type of aggregate have the largest influence on the fire resistance of reinforced concrete columns with the influence of moisture being relatively small.

Both experimental and theoretical work jointly undertaken by the National Research Council of Canada and the Portland Cement Association (Lie and Lin, 1985 B) indicate that full restraint against axial thermal expansion has little influence on the fire performance of columns. They found that the maximum stresses in a fully restrained column at the time the restraining load is a maximum, are considerably lower than those at the time of failure of the column.

Malhotra (1982) recommends empirical expressions for dimensioning reinforced columns assuming notional cover and reinforcement. The expressions are:

$$b = 150 + 1.6(t-30) \text{ min} \quad \text{for dense concrete} \qquad [18]$$

$$b = 150 + 0.86(t-30) \text{ min} \quad \text{for lightweight aggregate concrete} \qquad [19]$$

where:

b = column width (mm)
t = fire resistance (mm)

These expressions generally produce more conservative values of column width than do MP9 (SANZ, 1989 A) requirements and there seems little advantage in recommending their use in New Zealand.

Munukutla (1989) describes the basis for analytically modelling fire performance of concrete walls (and in principle also columns exposed to fire on one side). He describes both a simple and detailed design method for New Zealand conditions. The simple method involves checking MP9 data (SANZ, 1989 A) for adequate insulation of the wall, and also checking against a design chart which gives fire resistance as a function of wall thickness, axial load, end conditions, and height of the wall (Figure 23, for example). Munukutla's detailed method involves the use of both a thermal model (HEAT) and a structural model (FIREWALLS, discussed in next section).

Computer Programs

PC-DESIGN (Anderberg, 1989) is a program for determining the load-bearing capacity of concrete and steel beams as a function of time during and after exposure to fire and is intended to be used in conjunction with PC-TEMPCALC (discussed earlier). At the present time it is not commercially available, unlike PC-TEMPCALC.

FIREWALLS (Munukutla, 1989) is a program to predict the structural performance of a wall exposed to fire originally developed by O'Meagher and Bennetts (1987) but modified to include various boundary conditions at the top and bottom of the wall. The program also accommodates variation in wall thickness, height, load magnitude and eccentricity, reinforcement quantity and cover, and concrete and reinforcement properties. Axial restraint forces due to thermal expansion are not included. The program uses a numerical procedure where the wall is divided into segments and at each time step a moment-axial load interaction diagram is produced and the moment curvature relationship derived.

SAFE-RCC. Purkiss and Weeks (1987) describe a computer program, SAFE-RCC which allows the study of column behaviour in a fire under structurally induced restraint. The program is capable of handling both pin-ended and fully restrained columns with axial and rotational restraint considered separately. Purkiss and Weeks used a modified output from the FIRES-T3 thermal analysis program as input to SAFE-RCC.

CONFIRE (Forsen, 1982) is a non-linear structural analysis program which accounts for non-linear material properties, temperature dependent stress-strain relationships, creep, and thermal and transient concrete strains. Time dependent stresses, strains and displacements are obtained at each time step. Practical use of the program is demonstrated by Anderberg and Forsen (1982).

FIRES-SL (Nizamuddin and Bresler, 1979) is a program developed for predicting displacements and stress histories in reinforced concrete slabs exposed to fire. The program is a nonlinear finite element method coupled with time step integration, and is designed to be used with thermal response data generated by FIRES-T3. The coupling of both bending and membrane action can be evaluated.

FIRES-RC (Becker and Bresler, 1974) is a computer program designed to evaluate the structural response of reinforced concrete frames when exposed to fire. The analysis is non-linear with time step integration. Within each time step, an iterative approach is used to find a deformed shape which results in equilibrium between forces associated with external loads and internal stresses. The program is capable of providing time-dependent displacements, internal forces, stresses and strains in the concrete and steel reinforcement and assumes that boundary conditions can be simulated by a set of springs to represent the stiffness of the surrounding structure. The program is designed to be used with thermal response data produced by FIRES-T3 (discussed previously).

SPALLING OF CONCRETE

Spalling is the loss of concrete from the heated surfaces of concrete structures exposed to fire. It is detrimental to the fire resistance of concrete structures because the effective member cross-section is reduced below that assumed in the calculation of loadbearing capacity, and the temperature of reinforcing or prestressing steel rises more rapidly and thus strength is lost more rapidly than would otherwise occur. The basic types of spalling are -

1. Aggregate splitting; where splitting of silica containing aggregates in dense concrete occurs due to thermophysical changes. Its effect on structural performance can usually be ignored.

2. Surface spalling; where pitting, blistering and localised removal of surface layer material occurs. Compressive stresses (exceeding the compressive strength of the concrete) resulting from the thermal gradients occurring near the fire-exposed surface may be a contributing factor.

3. Explosive spalling; where pieces of concrete are expelled from the heated surface of the concrete member due to steam and other gas production within the concrete. The phenomenon is thought to be related to the nature of the aggregate, concrete porosity, moisture content and level of induced stress.

4. Sloughing off; where the surface layer of concrete is eroded away after prolonged exposure to fire (i.e., at corners of beams and columns).

Lie (1972) describes the main factors that promote spalling as:

a) high content of free water
b) restraint
c) low porosity of material
d) low permeability of material
e) rapid temperature rise at the exposed surface
f) closely spaced reinforcement

It is not proposed to closely examine these factors in detail here and the reader is referred to further discussion of the subject by Malhotra (1982), Lie (1972), Malhotra (1984), Copier (1980) and Saito (1965).

In general, analytical methods for calculating fire resistance do not account for spalling of concrete members, and in fact rely on the non-occurrence of serious spalling. For this reason, spalling needs to be considered separately and if necessary, steps taken to ensure the risk of spalling is low.

To reduce the risk of spalling associated with a large amount of unreinforced cover, Morris et al (1988) recommend additional protection against spalling in concrete beams when the cover to the outermost steel exceeds 40 mm for dense concrete and 50 mm for lightweight concrete.

In general, spalling is more likely to occur in members with slender or small cross sections or where there are sudden variations in cross section. Also, MP9 (SANZ, 1989 A) states that there have been no reported instances of spalling in members made of lightweight concrete in New Zealand. Fire testing of a wide range of New Zealand concrete slabs of uniform thickness (Woodside, de Ruiter and Wade, 1991) did not indicate any occurrence of explosive spalling, but some localised surface spalling was observed for pumice and limestone aggregate concretes. In these tests, 55 mm clear cover was provided to reinforcing steel.

TABULAR DATA FOR FIRE RESISTANCE

As mentioned previously in this report, the availability of tabulated data giving minimum required dimensions of member sizes and concrete cover for a stated period of fire resistance is desirable for quick solutions, especially where fire resistance does not govern the member sizing. However, tabulated data may produce more conservative results as the location and amount of steel in the concrete member cannot be optimised to the same degree.

Tabulated fire resistance data for structural concrete columns and beams for use in New Zealand is published in MP9 (SANZ, 1989 A). A comparison of MP9 data with a variety of equivalent overseas data is presented in Tables 3, 4, 5, 6, 7 and 8 for reinforced beams, prestressed beams, columns, reinforced slabs, prestressed slabs, and walls respectively. The other documents examined were: CP 110 (BSI, 1972); BS 8110 Part 2 (BSI, 1985) which superseded CP 110; BRE report (Morris et al, 1988) which provided the technical basis for BS 8110; Forrest and Law (1984); FIP/CEB's Guides to Good Practice (FIP, 1975); CEB's model code for concrete structures (CEB, 1987); Supplement to the National Building Code of Canada (NRCC, 1985); the Uniform Building Code (International Conference of Building Officials, 1988); and AS 3600 (SAA, 1988) the current Australian Concrete Structures Code.

While differences between the MP9 data and overseas data exist, they are not widely disparate which is not surprising since the MP9 figures were originally obtained from these sorts of overseas sources. Strict comparison of the data is complicated by the compensating effect that higher minimum dimensions can have in reducing the necessary steel cover.

Many of the overseas documents make allowance for a trade off between the minimum dimension of concrete beams and the required steel cover as a larger heat sink will reduce the rate of temperature rise on the steel. For example, Morris et al (1988), BS 8110 (BSI, 1985) and Forrest and Law (1984) recommend the variation in cover shown in Table 9. Importantly, it is noted in the above references, that the cover must not be less than that required for a plain soffit floor of the same fire resistance.

For prestressed and reinforced beams and slabs, many of the documents also provide additional tabular data for members continuous over supports, rather than simply supported. As discussed previously, this effect is beneficial and may allow reductions in member size and cover.

Neither trade off in cover with member size nor allowance for continuity is accommodated in the tabular data in MP9 although is referred to in the text and some designers do make use of it for one-off designs. The increased flexibility for designers by encouraging these options is desirable and the MP9 data should be reviewed, and updated accordingly.

RECOMMENDATIONS

A two-tier approach to determining fire resistance of concrete structures is recommended. Firstly, the use of tabular data (status quo), and secondly, the option for designers to use rational design methods. New Zealand building codes should facilitate such an approach.

Pending the availability of completely analytical tools, approximate design methods for concrete walls and concrete beams and slabs should be published.

The approximate design method for concrete walls should be based on the simple design method described by Munukutla (1989).

A technical recommendation should be produced by BRANZ, outlining an approximate design method for rational design of structural concrete beams and floors. The publication should include a simplified method of estimating temperatures within beams during a fire (e.g., graphs) and it should provide suitable elevated temperature properties of steel and concrete and be able to account for both continuous and simply supported elements. The rational method should be predominantly based on ACI (1981) and Gustaferro and Martin (1977) documents and should apply to both reinforced and prestressed construction.

Existing tabular data in MP9 should be reviewed and updated to better reflect the research effort over recent years. With respect to beams it should provide for additional flexibility by accommodating a trade off between beam width and cover to steel. With respect to both beams and slabs the effect of continuity over supports should be accommodated more explicitly.

The building control system should also be amended to specifically permit the use of rational calculation methods, such those referred to in this report.

Finite element thermal analysis software specifically designed for predicting temperatures in fire engineering problems (e.g., PC-TEMPCALC) should be obtained by BRANZ and used as a tool to assist New Zealand designers.

CONCLUSIONS

Rational design and analytical methods for calculating fire resistance of structural concrete members have many advantages over full-scale fire testing, and their further development and use should be encouraged.

Rational design methods are more established for flexural concrete members such as beams and slabs than they are for compression elements such as walls and columns. Furthermore, such methods for prestressed and reinforced concrete flexural members are now sufficiently developed and established overseas, that they could also be used in New Zealand.

Detailed analytical methods have been developed overseas for calculating the fire resistance and structural behaviour in fire of concrete columns. These methods require specialised software not yet readily available. The basis for a simplified method for concrete walls has been documented.

Rational design methods for reinforced concrete are unlikely to be warranted in cases where the required fire resistance is less than about 90 to 120 minutes, and about 60 minutes for prestressed concrete.

Tabular data for fire resistance of concrete structures must continue to be available. There are opportunities for the flexibility of such data to be increased by providing a trade off between minimum size of member and cover, and in the accommodation of continuous members.

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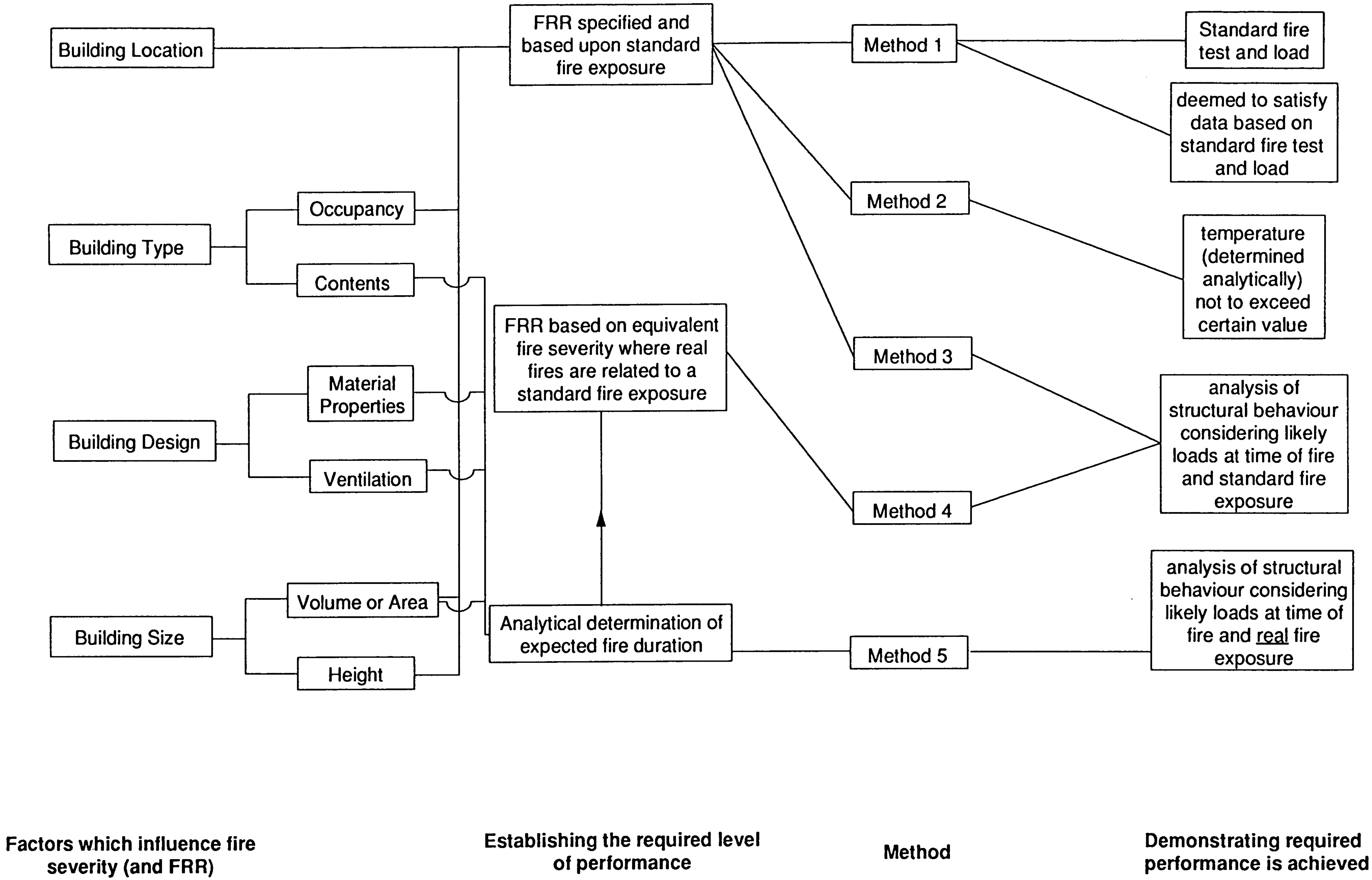


Figure 1. Summary of Design Approaches

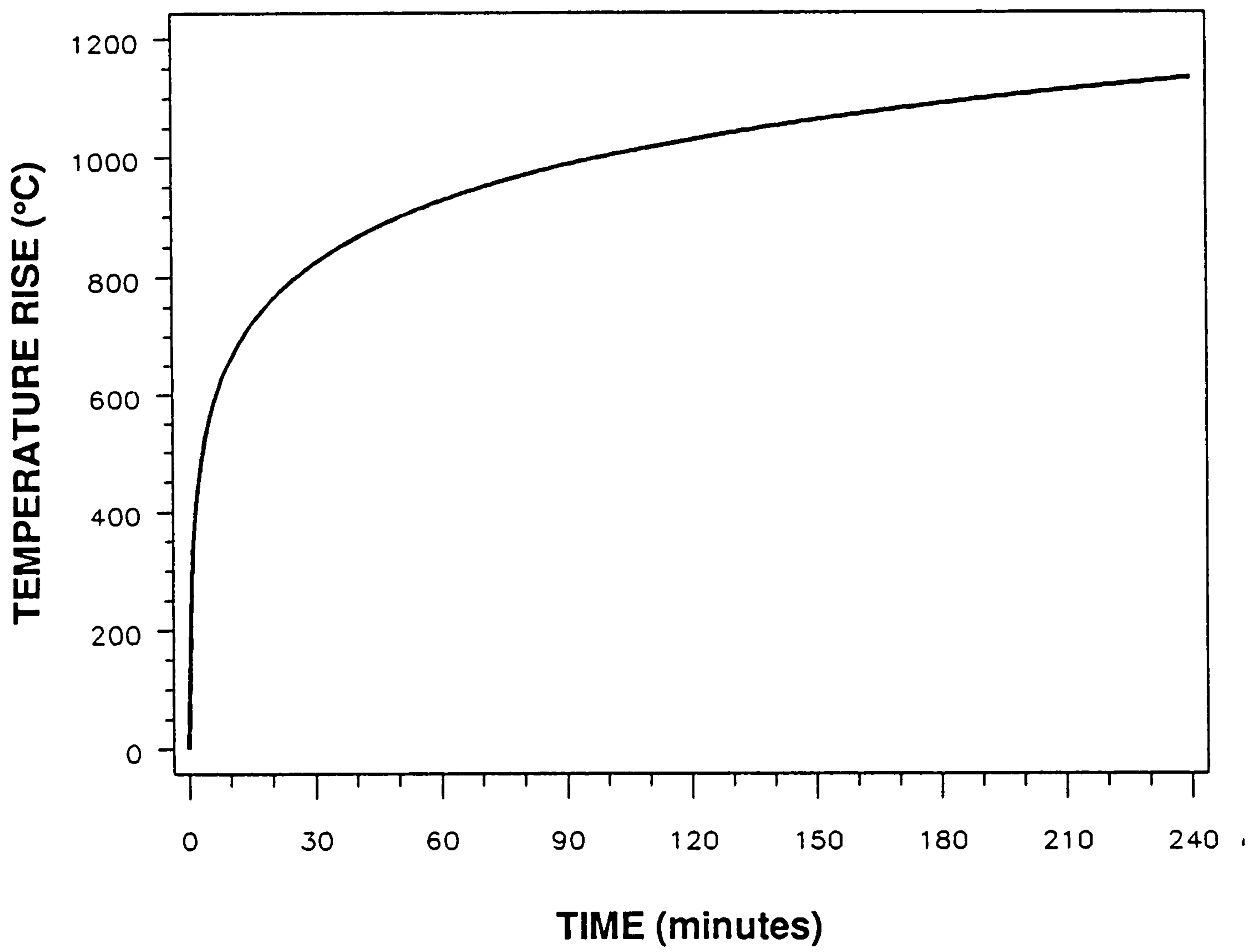


Figure 2: ISO 834 Standard time - temperature curve

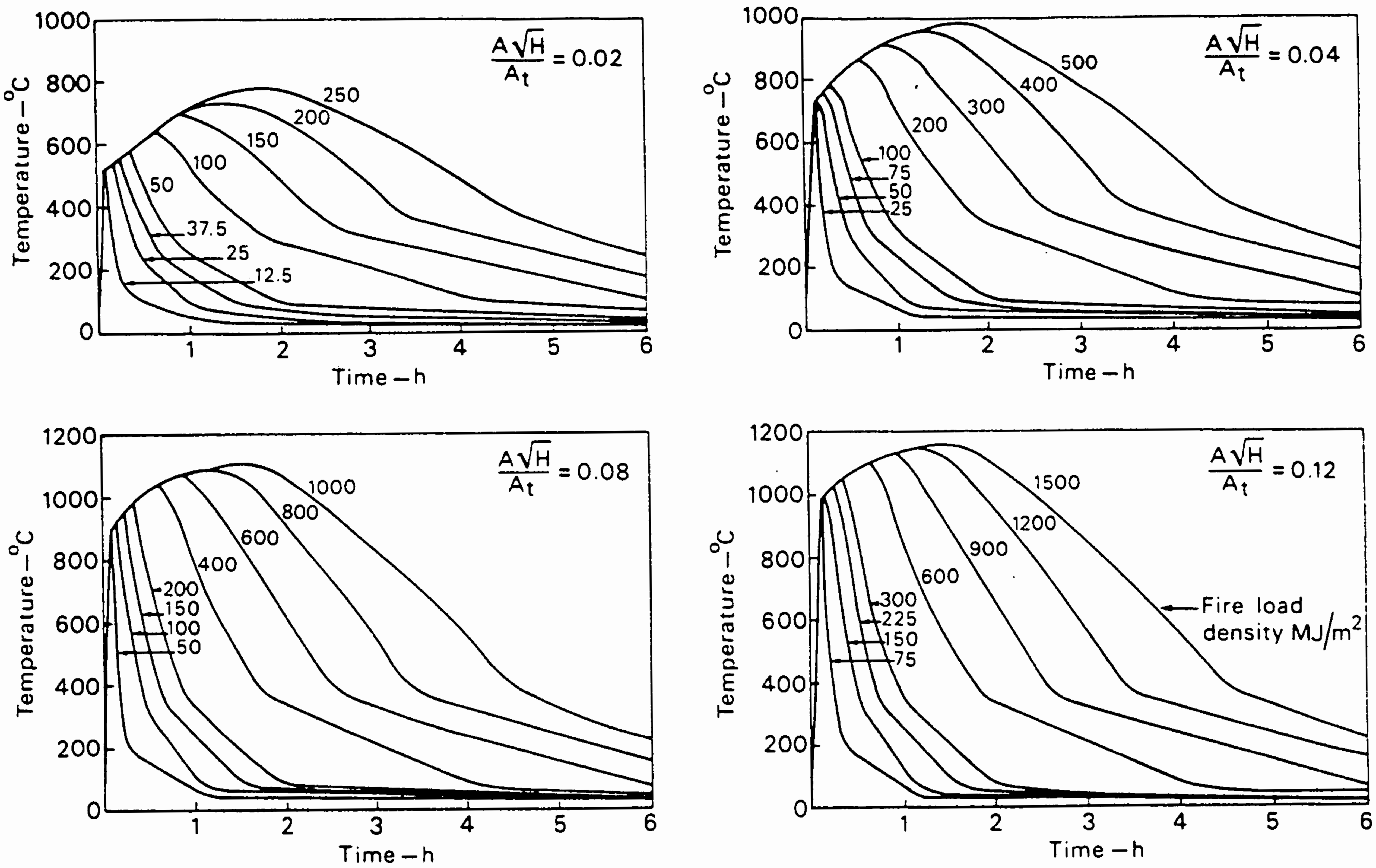


Figure 3: Theoretical temperature-time curves for different fire load densities and opening factors (from Malhotra, 1982)

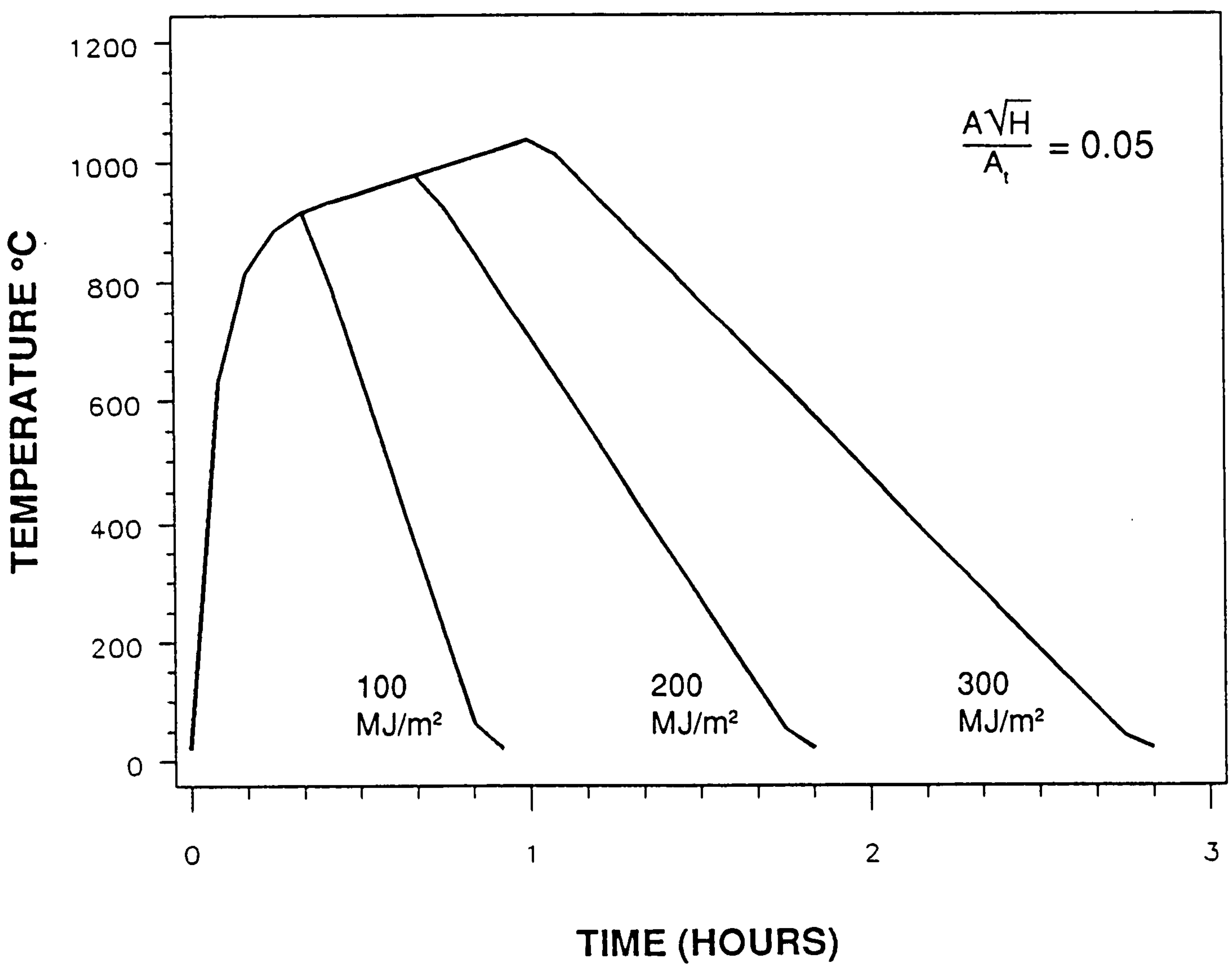


Figure 4: **Time-temperature curve for fire loads of 100, 200 and 300 MJ/M^2 and opening factor = 0.05 m$^{0.5}$**

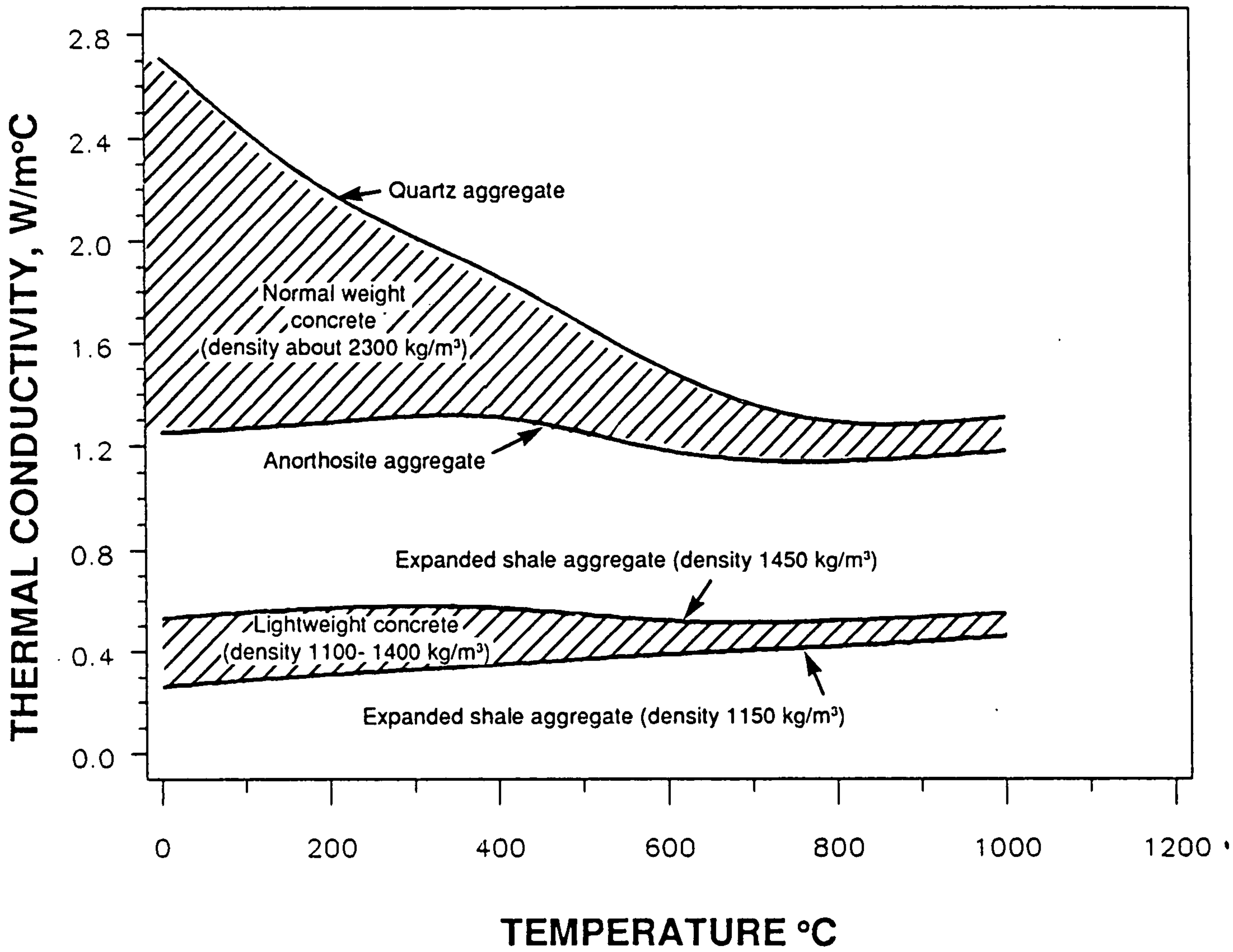


Figure 5: Limiting thermal conductivities of normal-weight concrete (2300 kg/m^3) and light-weight concrete (density 1100 to 1400 kg/m^3) as a function of temperature (redrawn from Lie, 1972)

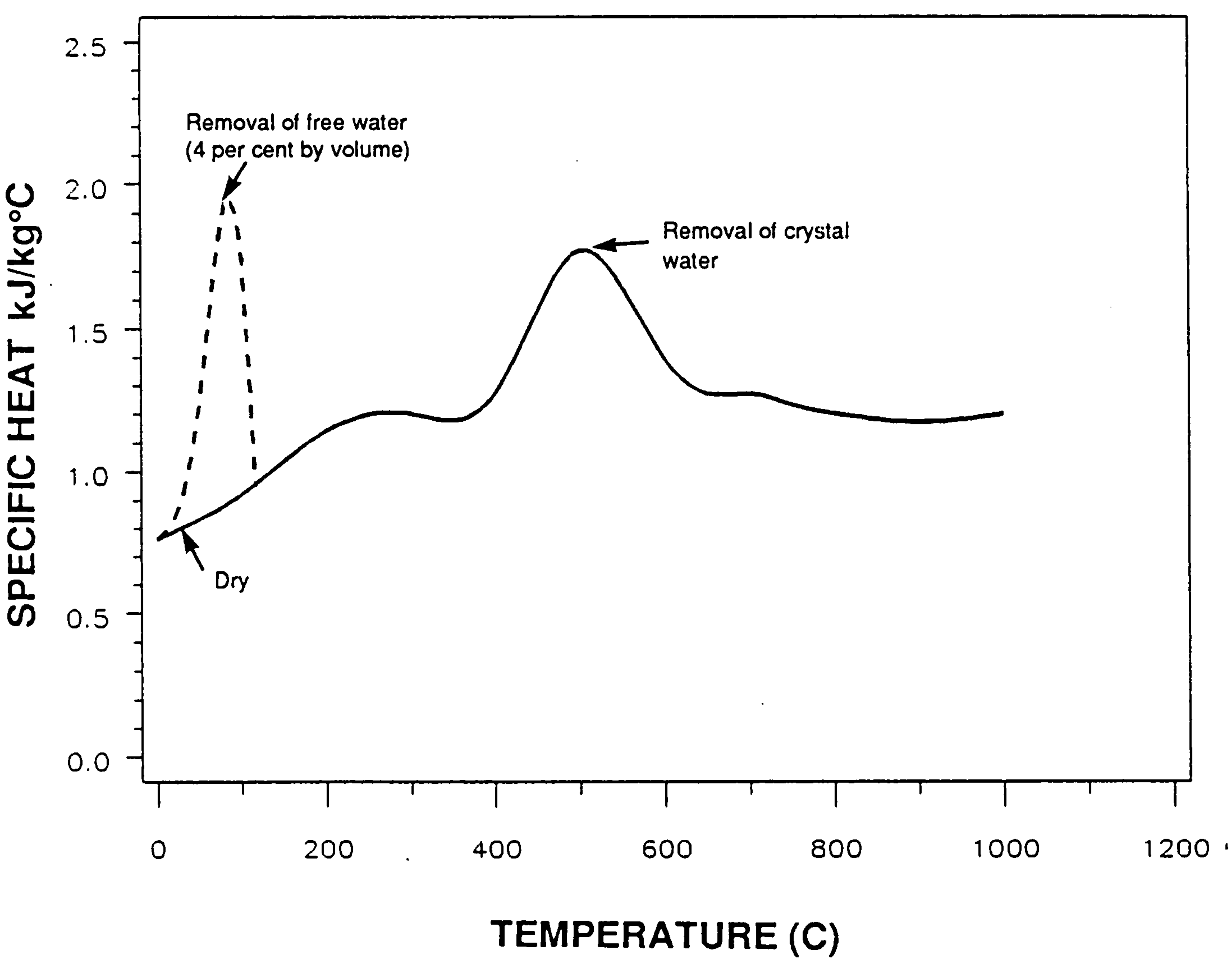


Figure 6: Specific heat of normal-weight concrete as a function of temperature (redrawn from Lie, 1972)

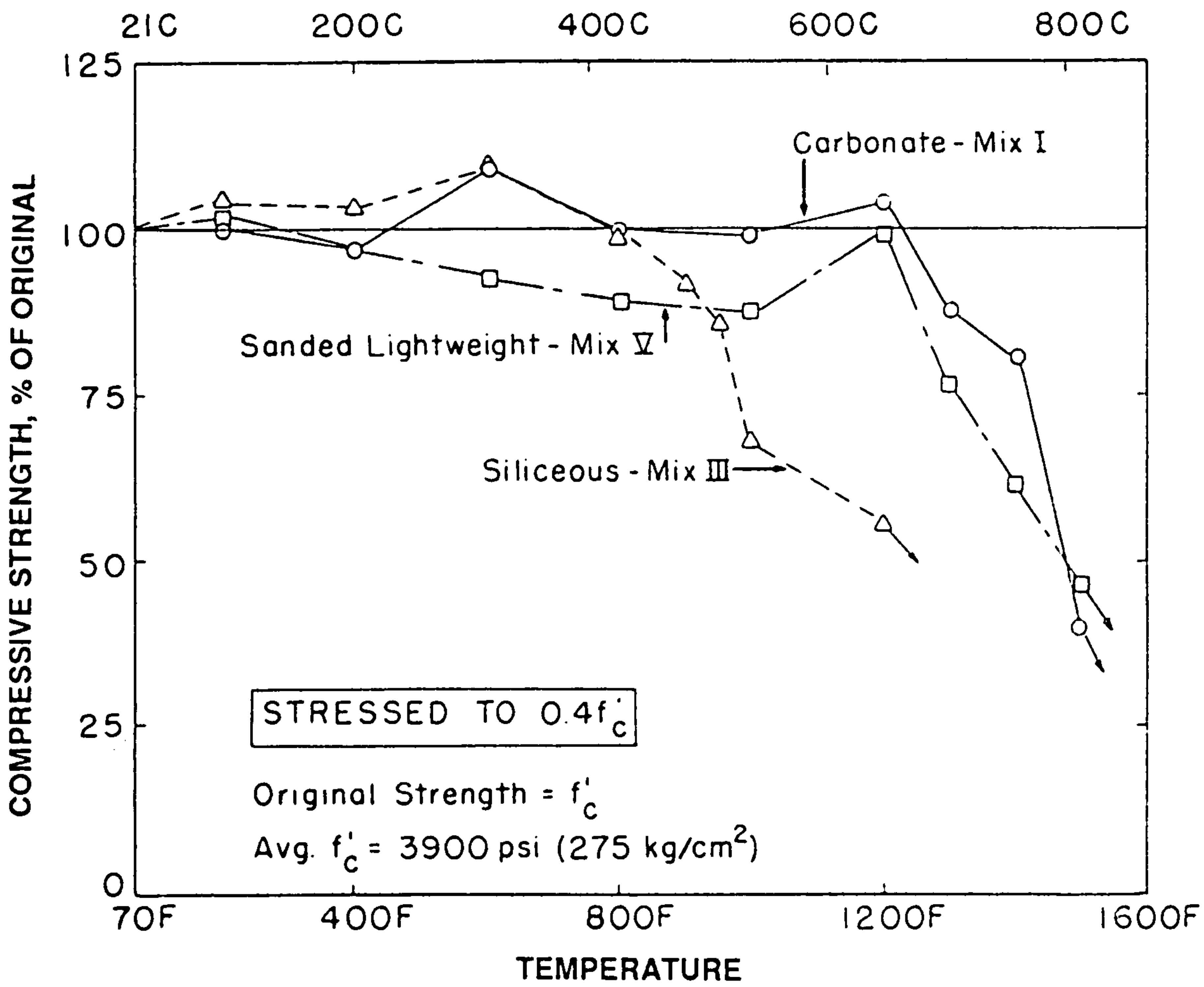


Figure 7: **Strength-temperature relationships for a carbonate, siliceous and sand-lightweight concrete (from Abrams, 1973)**

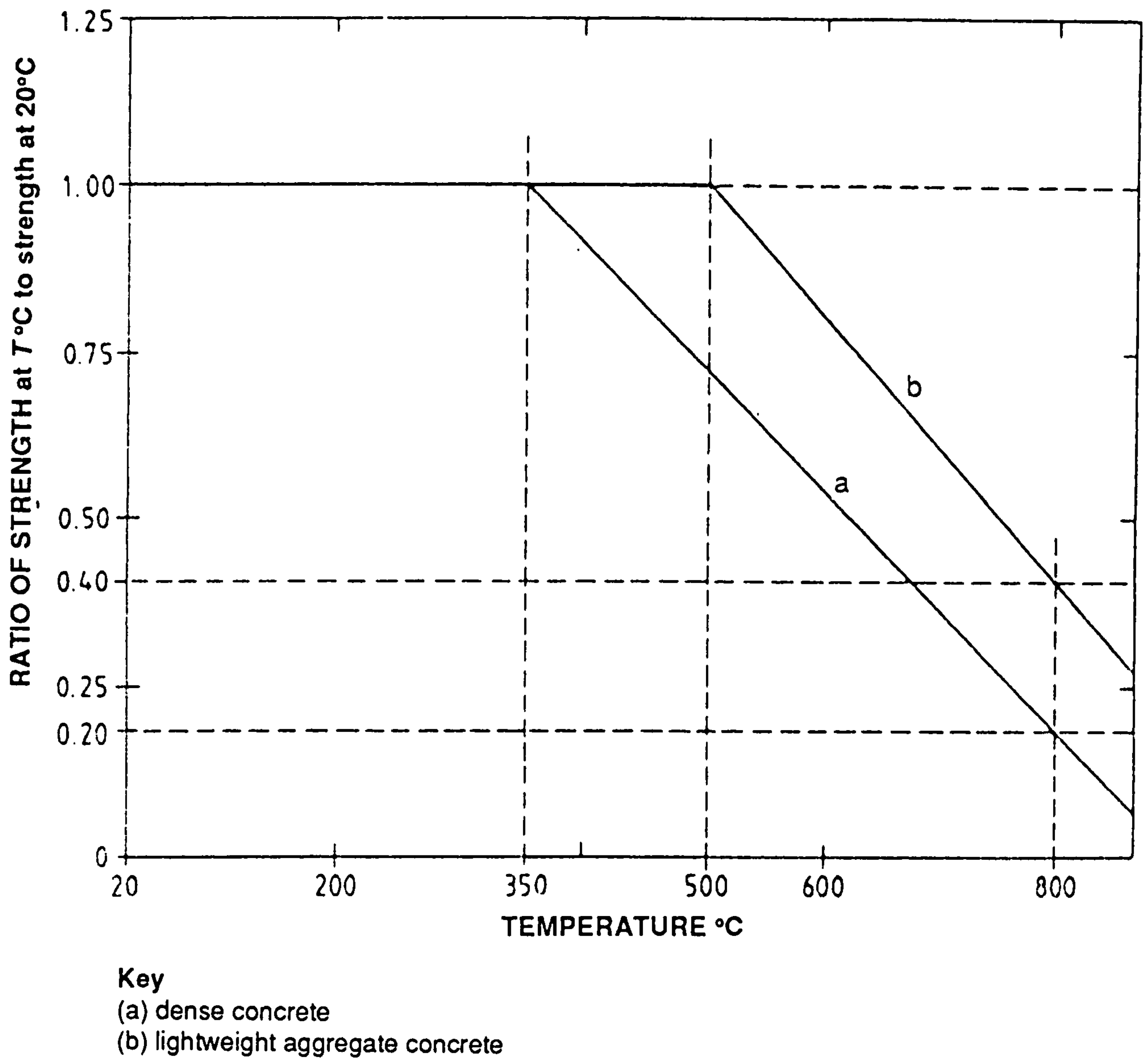


Figure 8: Design curves for variation of concrete strength with temperature (from BS 8110)

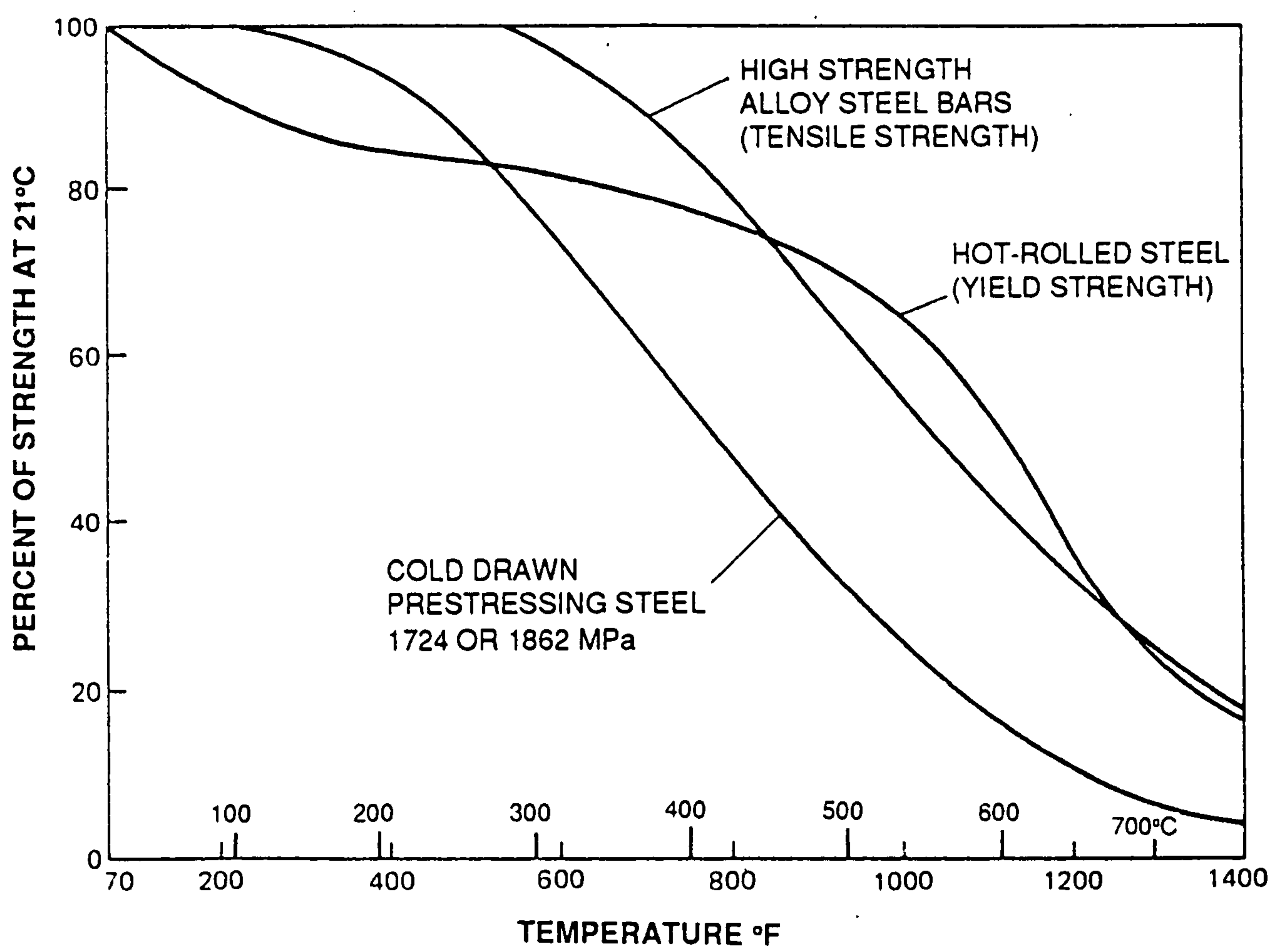
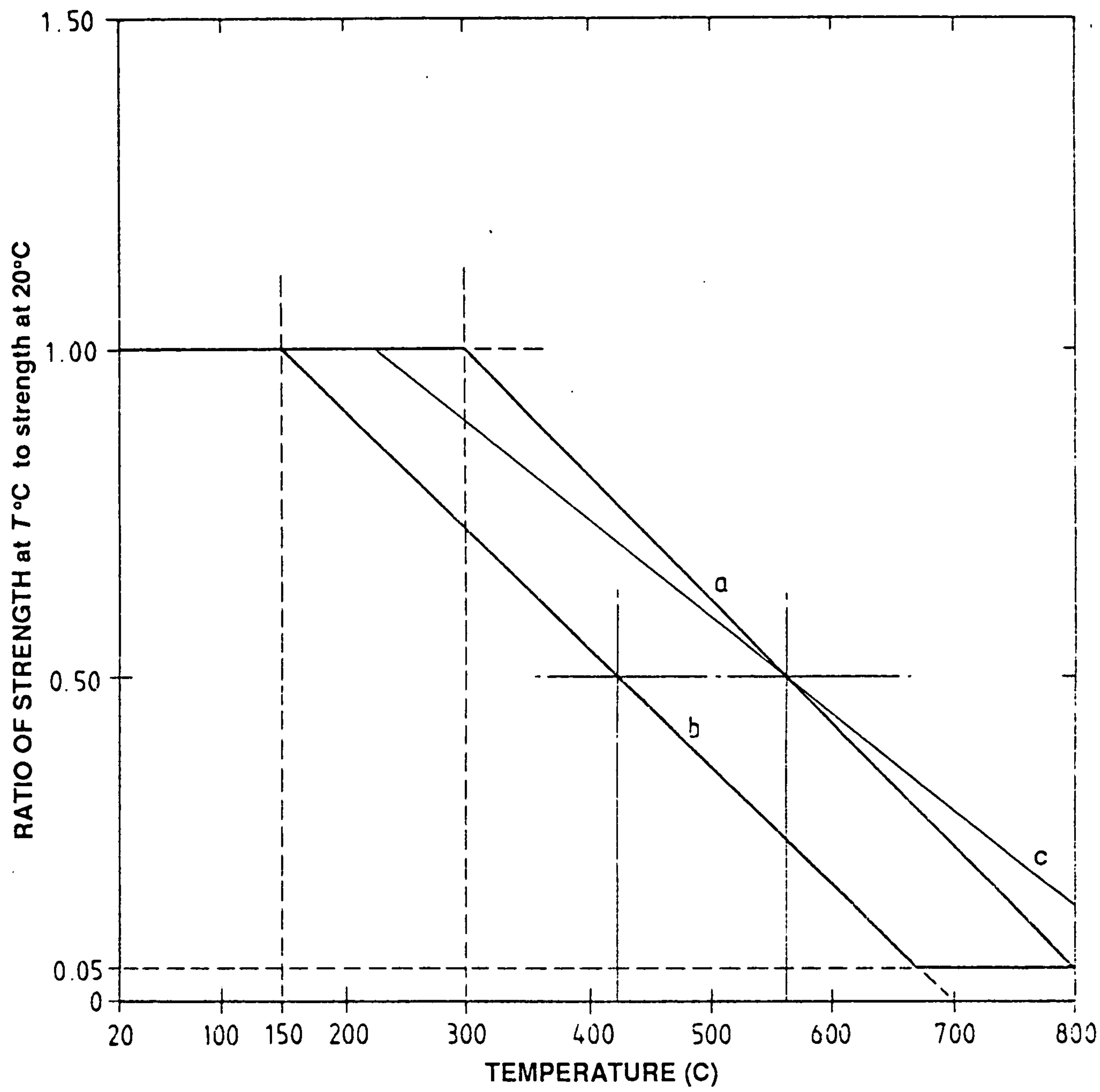


Figure 9: Strength-temperature relationship for hot-rolled, cold drawn, and high strength alloy steels (from SFPE handbook, re-labelled)



KEY

(a) grade 460 and grade 250 reinforcement and extra high strength steel bars (BS 8110)
(b) prestressing wires or strands (BS 8110)
(c) structural steel (MP9)

Figure 10: Design curves for variation of steel strength or yield stress with temperature (from BS 8110, modified)

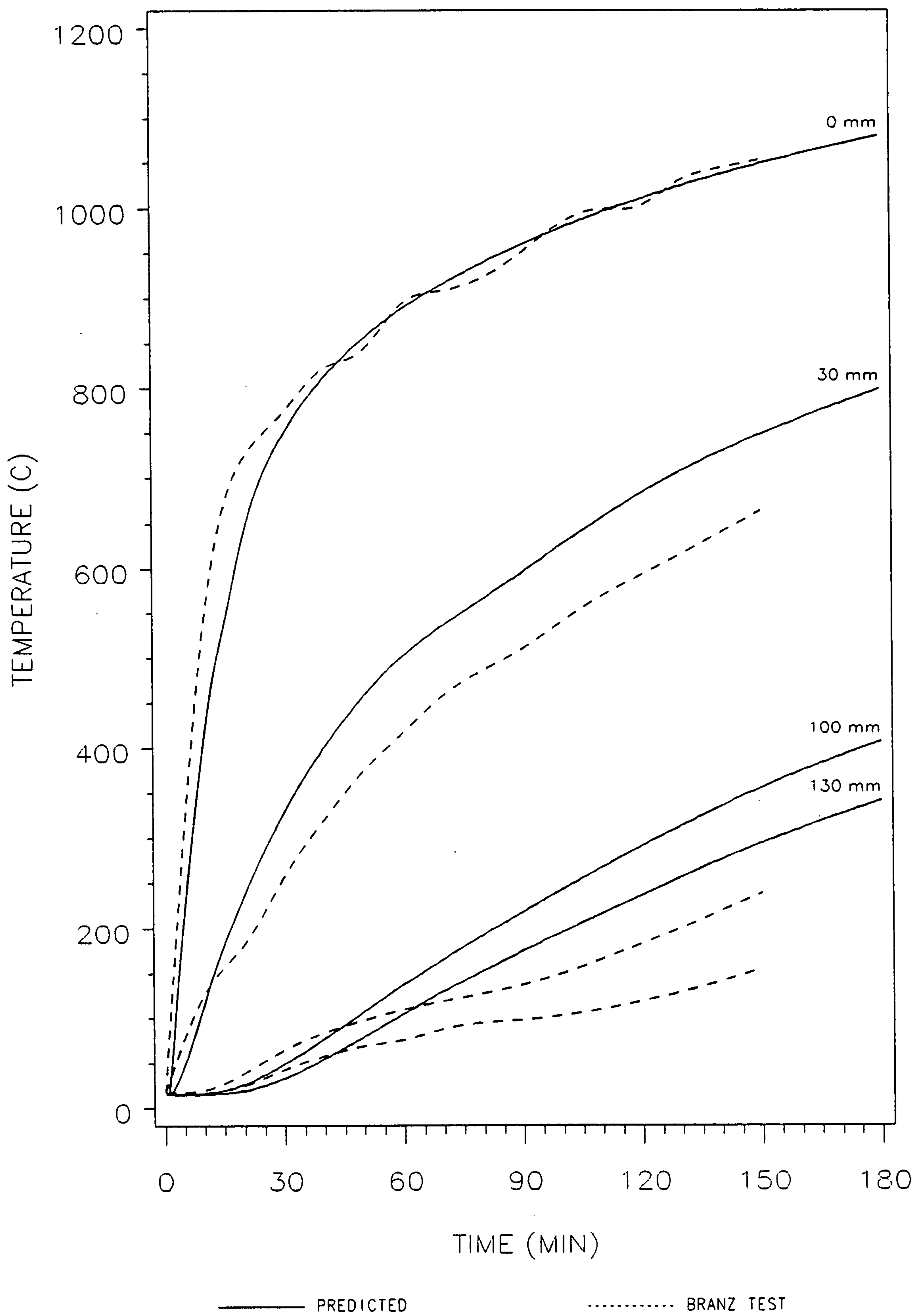


Figure 11: Temperatures in concrete slab (alluvial quartz) with no adjustment for vaporisation of moisture

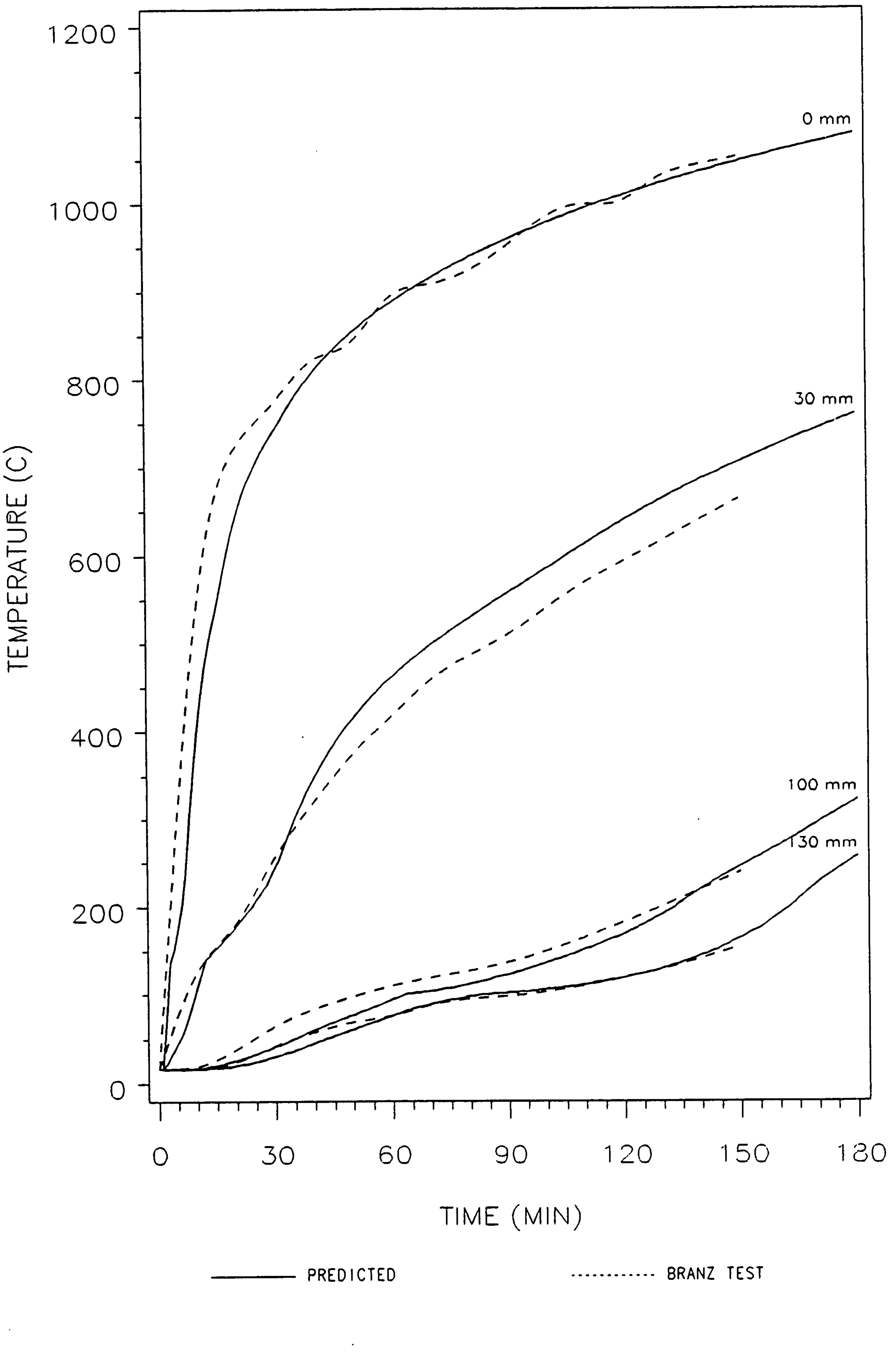


Figure 12: Temperatures in concrete slab (alluvial quartz) with adjustment for vaporisation of moisture

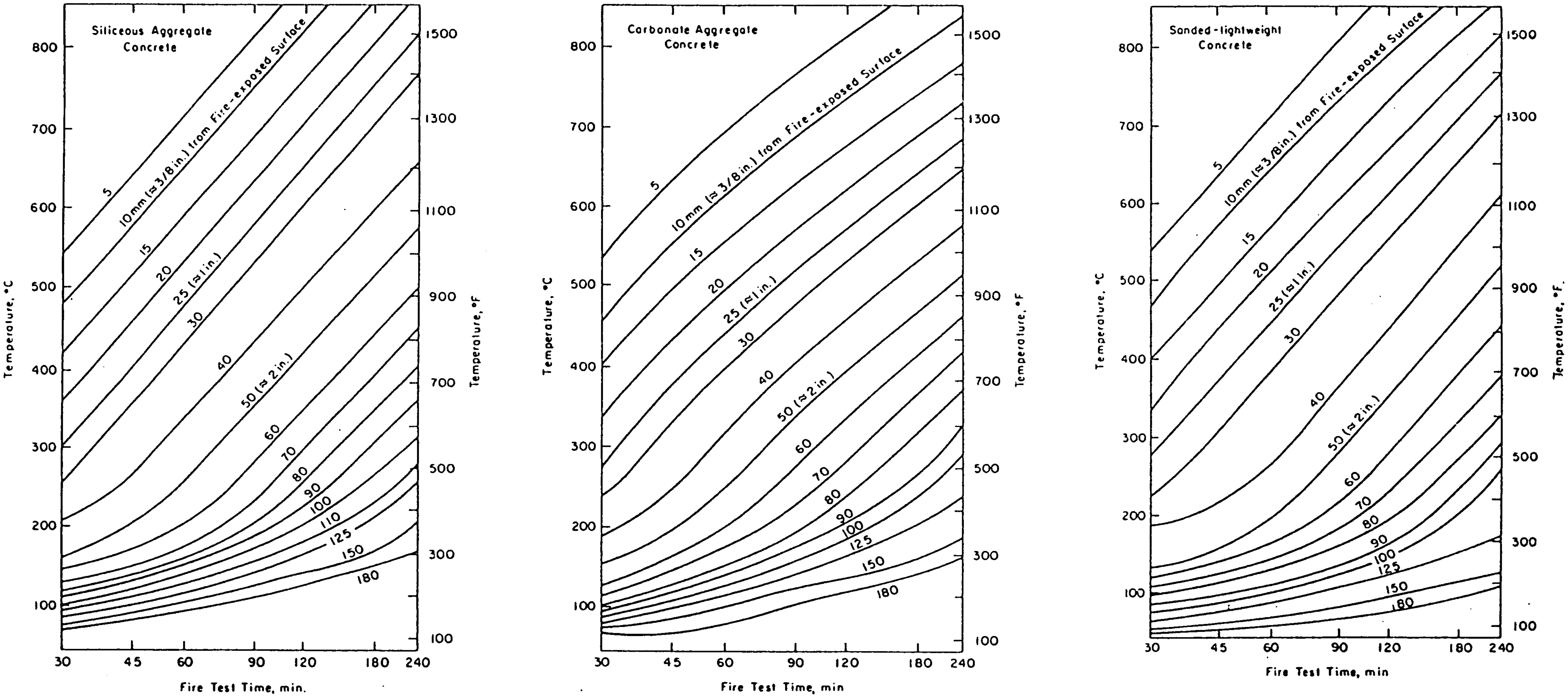


Figure 13: Temperatures within slabs during fire tests (from ACI 1981)

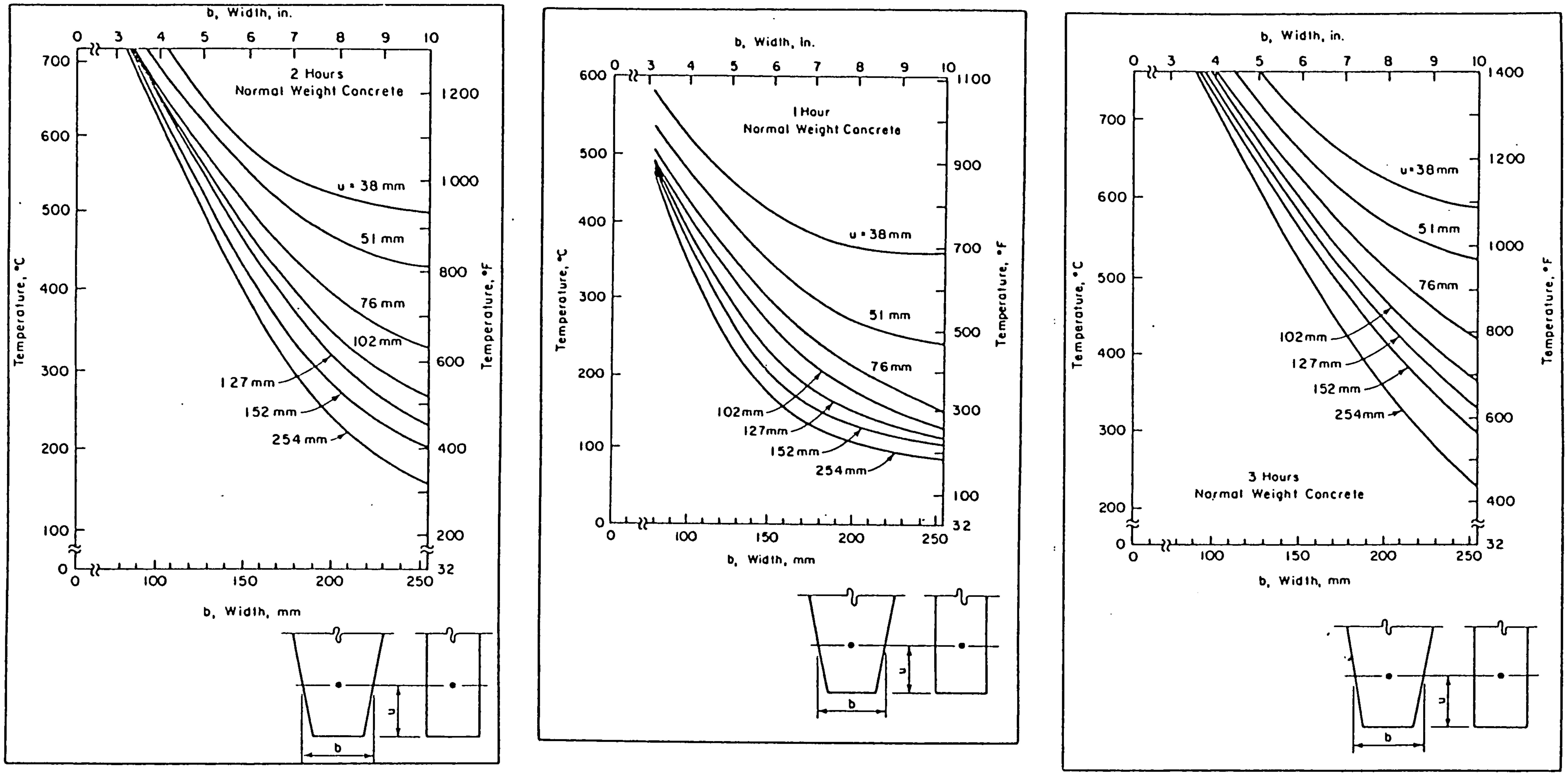


Figure 14: Temperatures in normal-weight concrete rectangular and tapered beams (from ACI 1981)

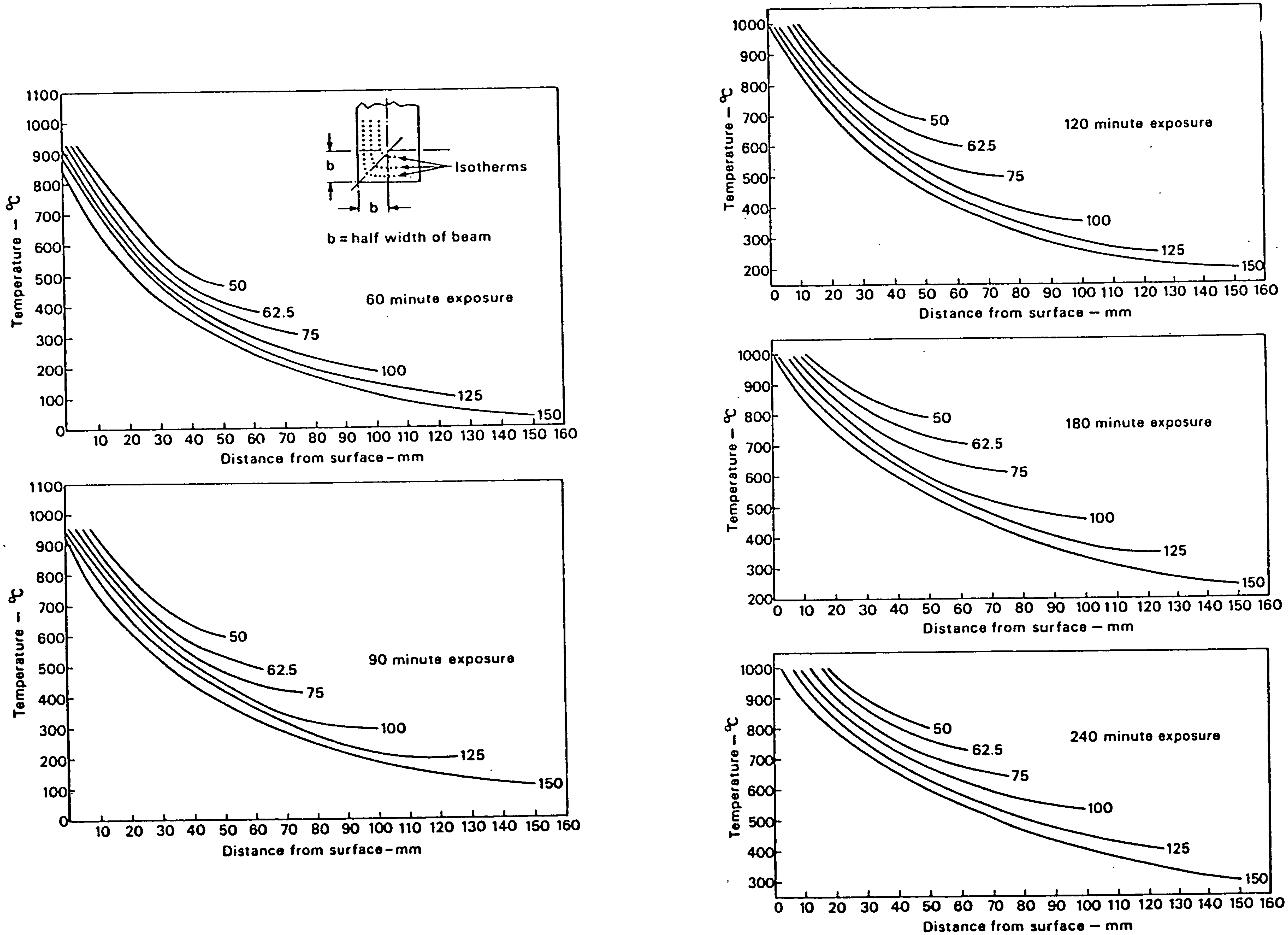
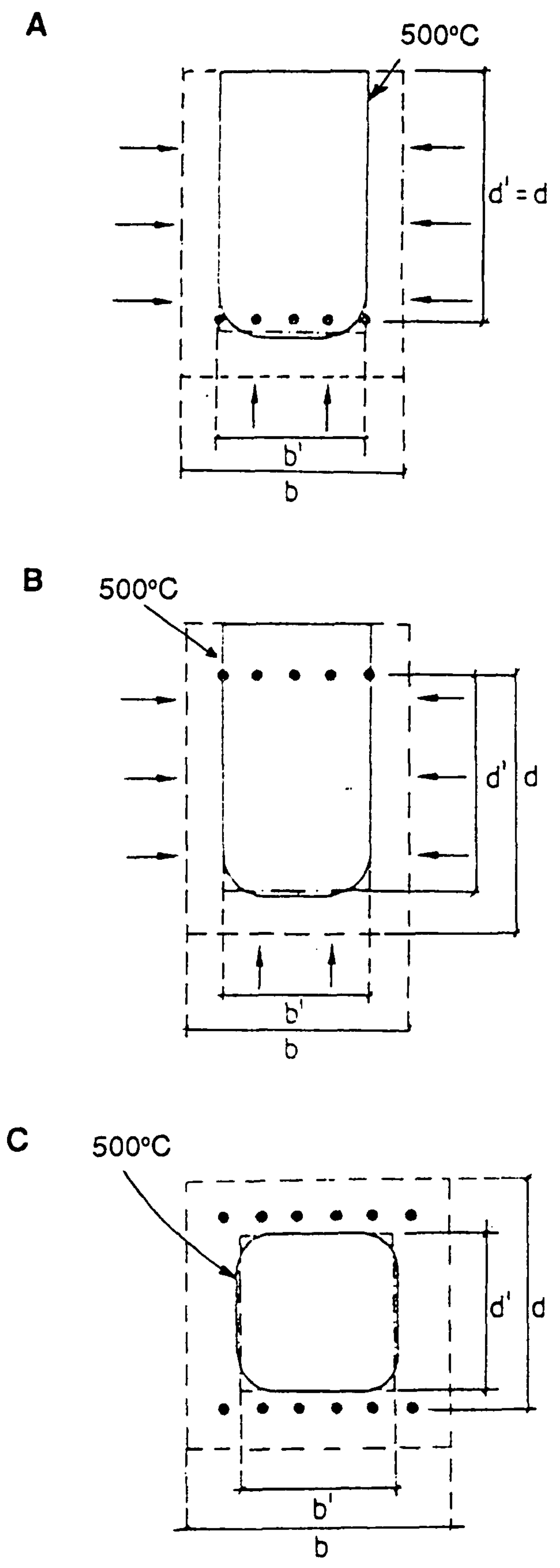


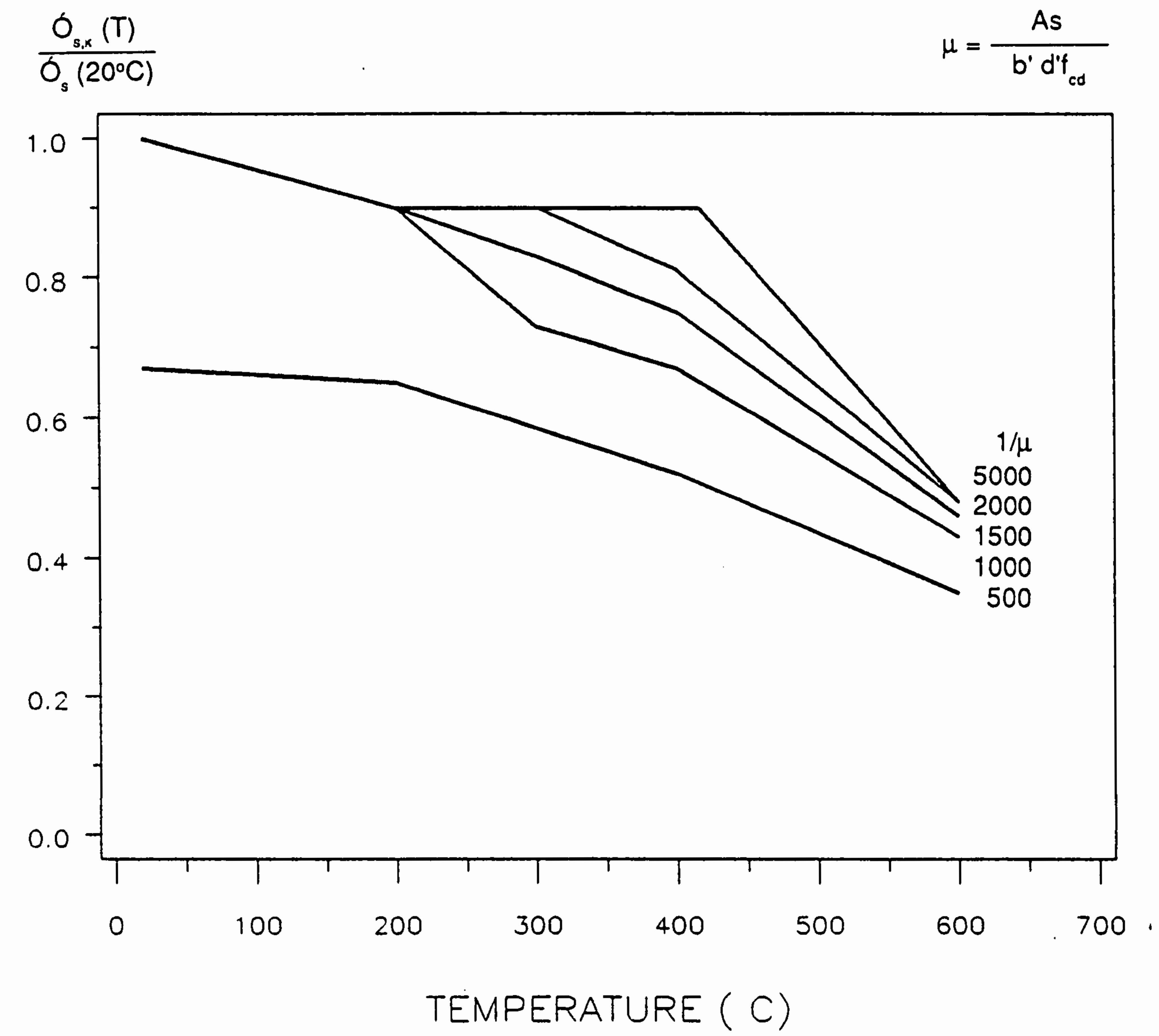
Figure 15: Temperature rise in rectangular beams of dense concrete (from Malhotra 1982)



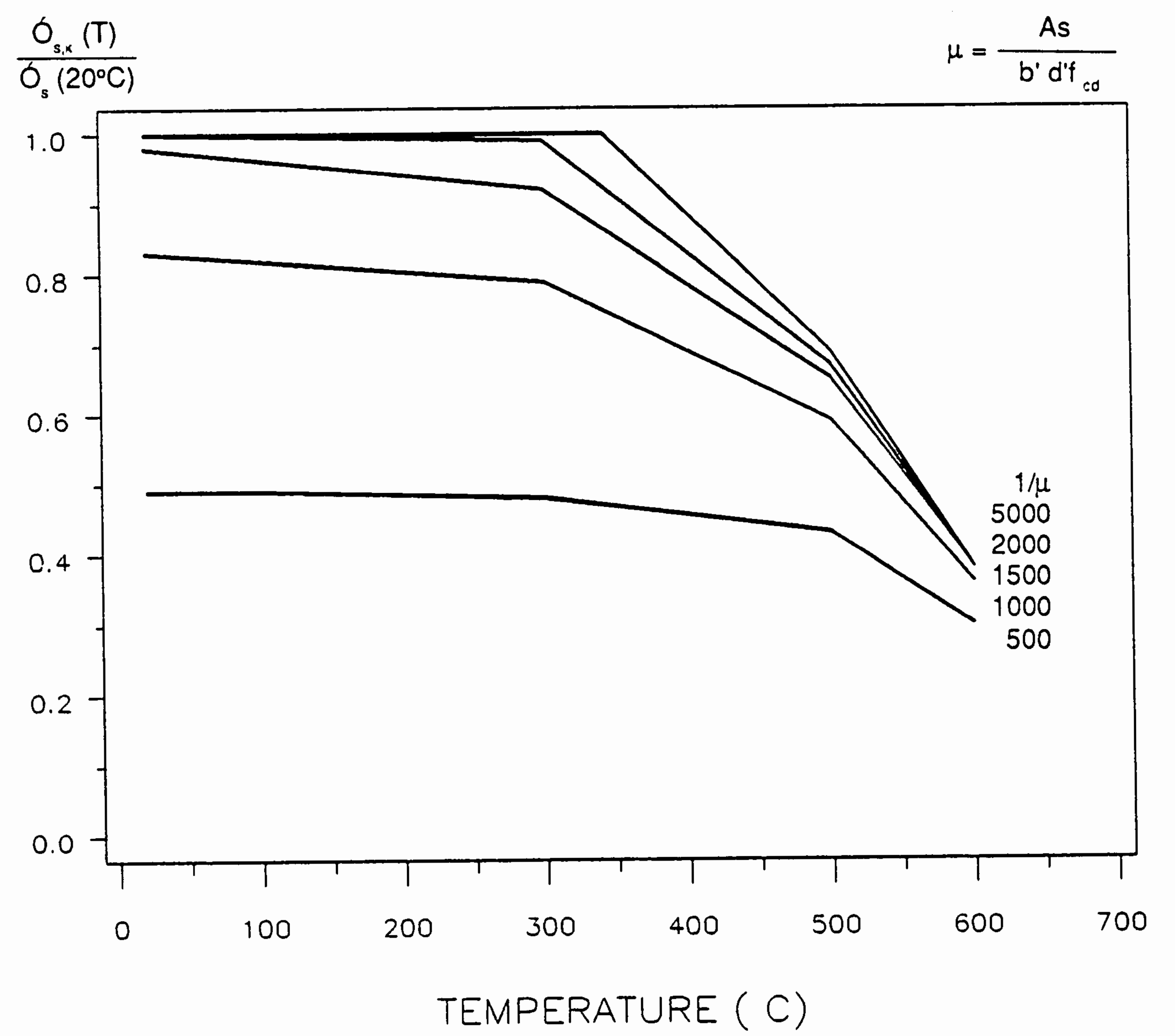
Reduced cross-section of a reinforced concrete beam at
(a) fire exposure on three sides with the tension zone expoosed
(b) fire exposure on three sides with the compression zone exposed
(c) fire exposure on four sides

Figure 16: Reduced cross section of a reinforced concrete beam (from CEB 1987)

A



B



Critical stress for the tensile reinforcement as a function of average temperature T_m of the reinforcement and $1/\mu$. σ_s (20°C) = characteristic value of yield stress at ordinary room temperature ($\gamma_m = 1$). In the formula for μ, A_s is in m², b' and d' in m, and f_{cd} in MPa.
(a) Hot rolled steel
(b) Cold worked steel

Figure 17: Critical stress for the tensile reinforcement (redrawn from CEB, 1987)

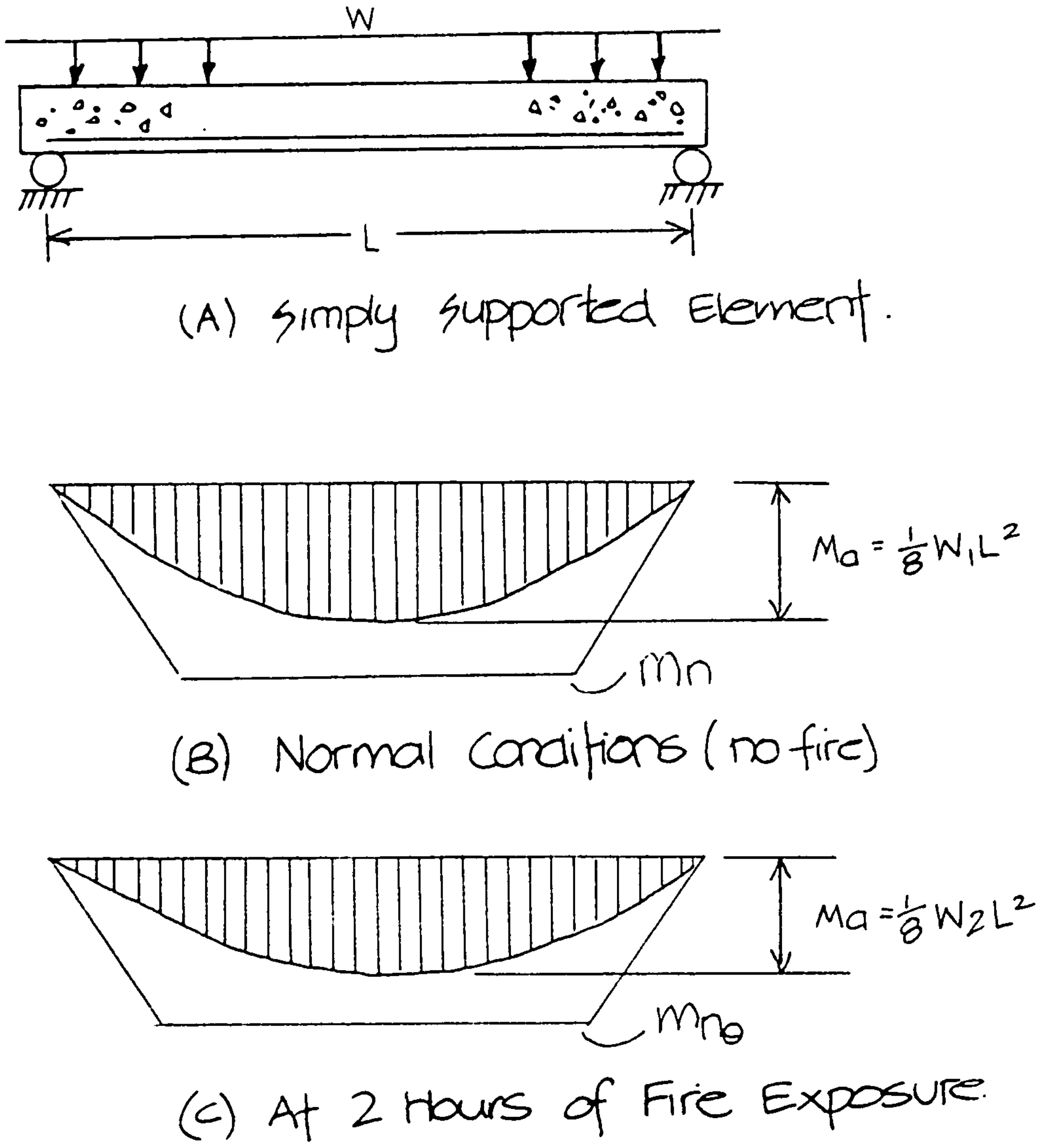


Figure 18: Applied moments and reduced moment strength diagrams for a simply supported element

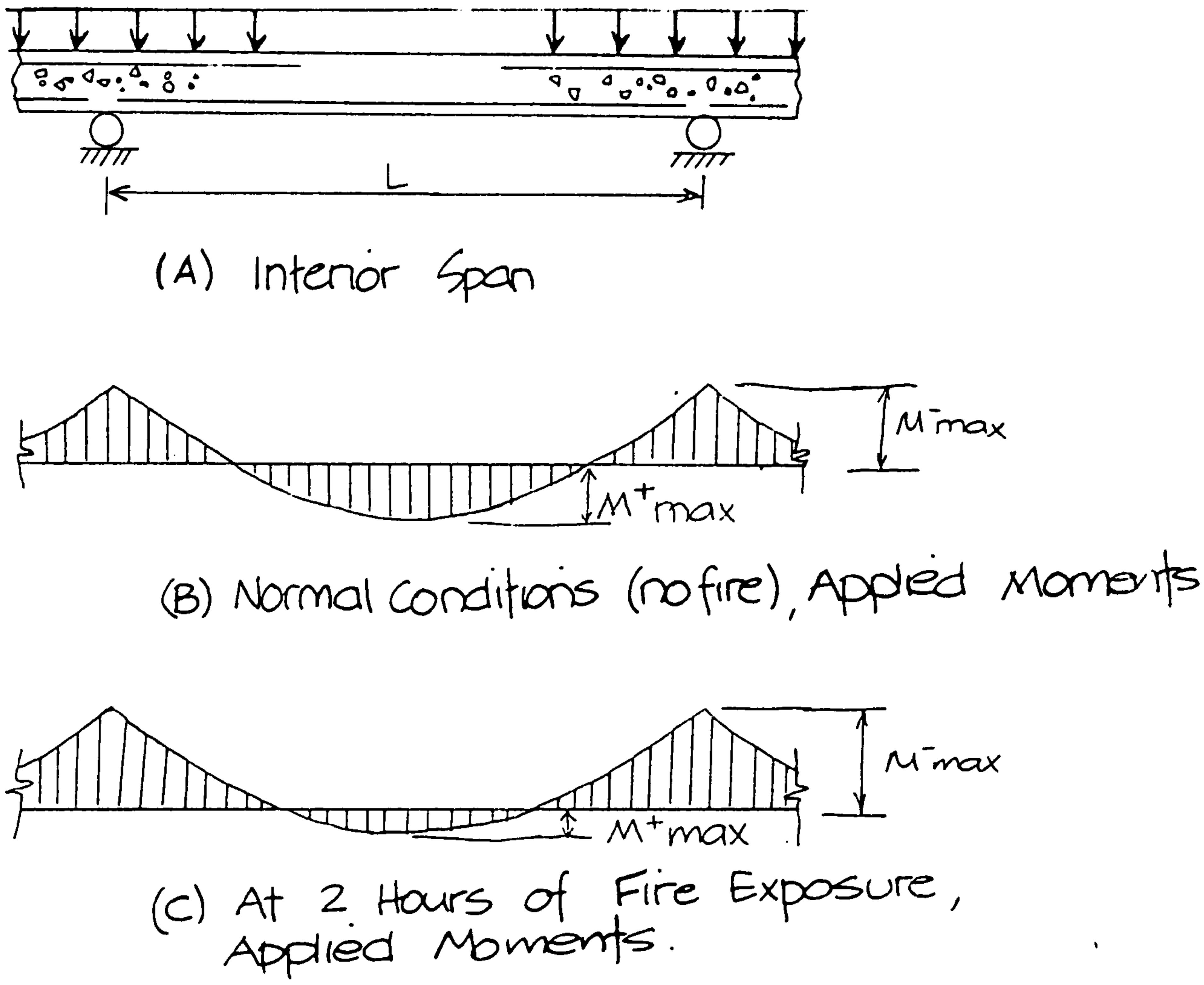


Figure 19: Moment redistribution in interior span of continuous unrestrained element due to fire exposure

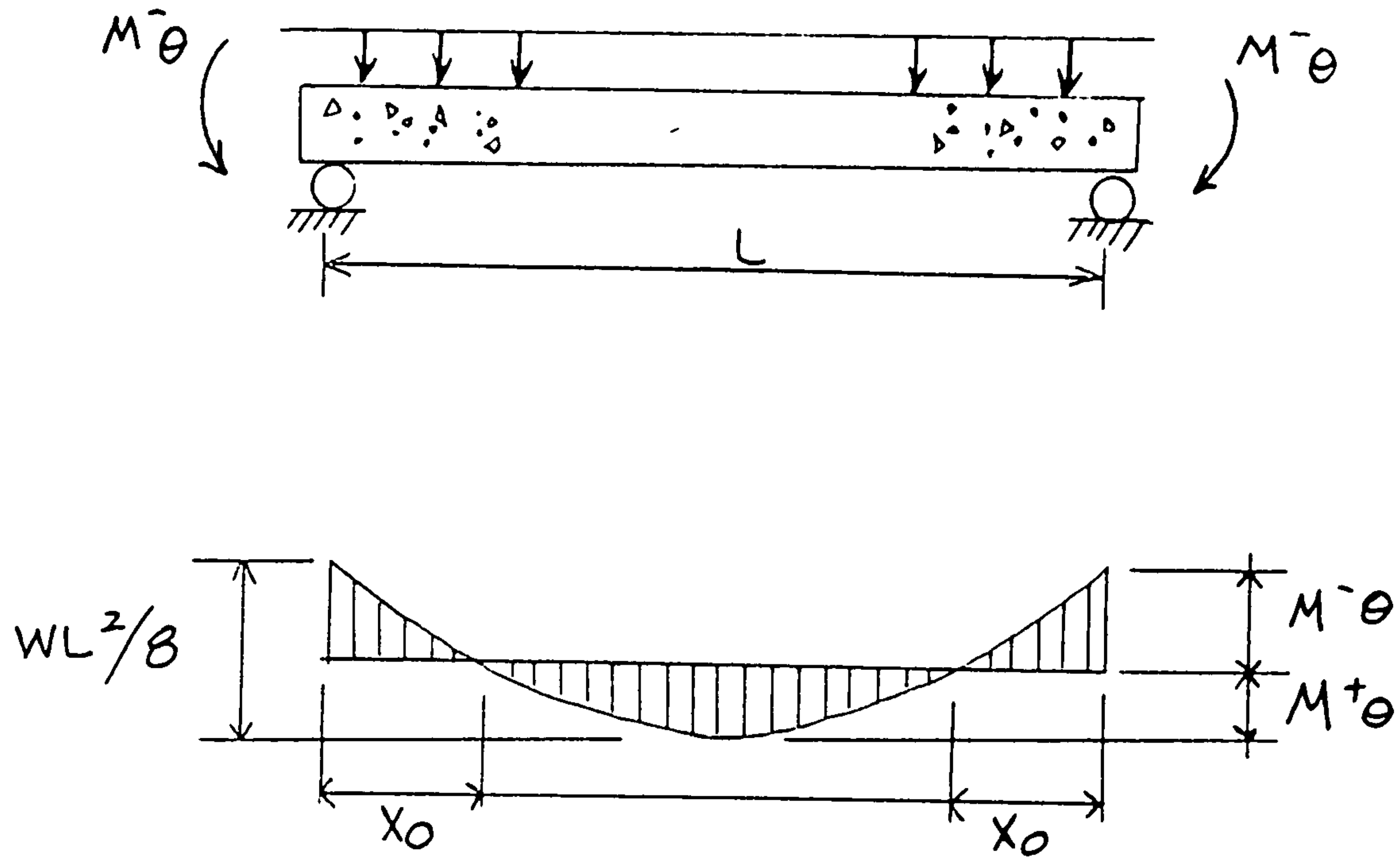


Figure 20: Symmetrical uniformly loaded member continuous at both supports

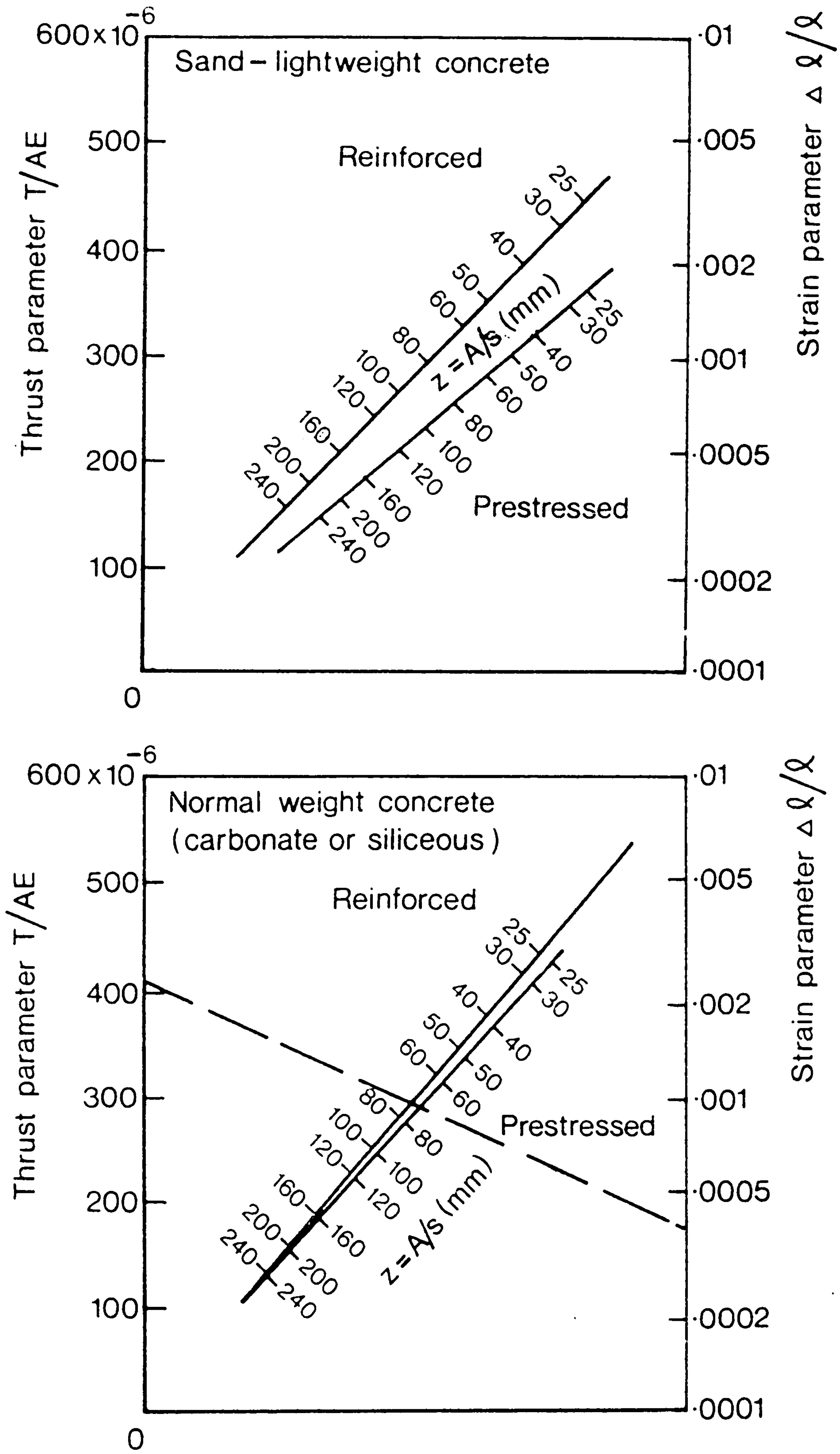


Figure 21: Nomographs relating thrust parameter, strain parameter, and ratio of cross-sectional area to heated perimeter (from Cement and Concrete 1978)

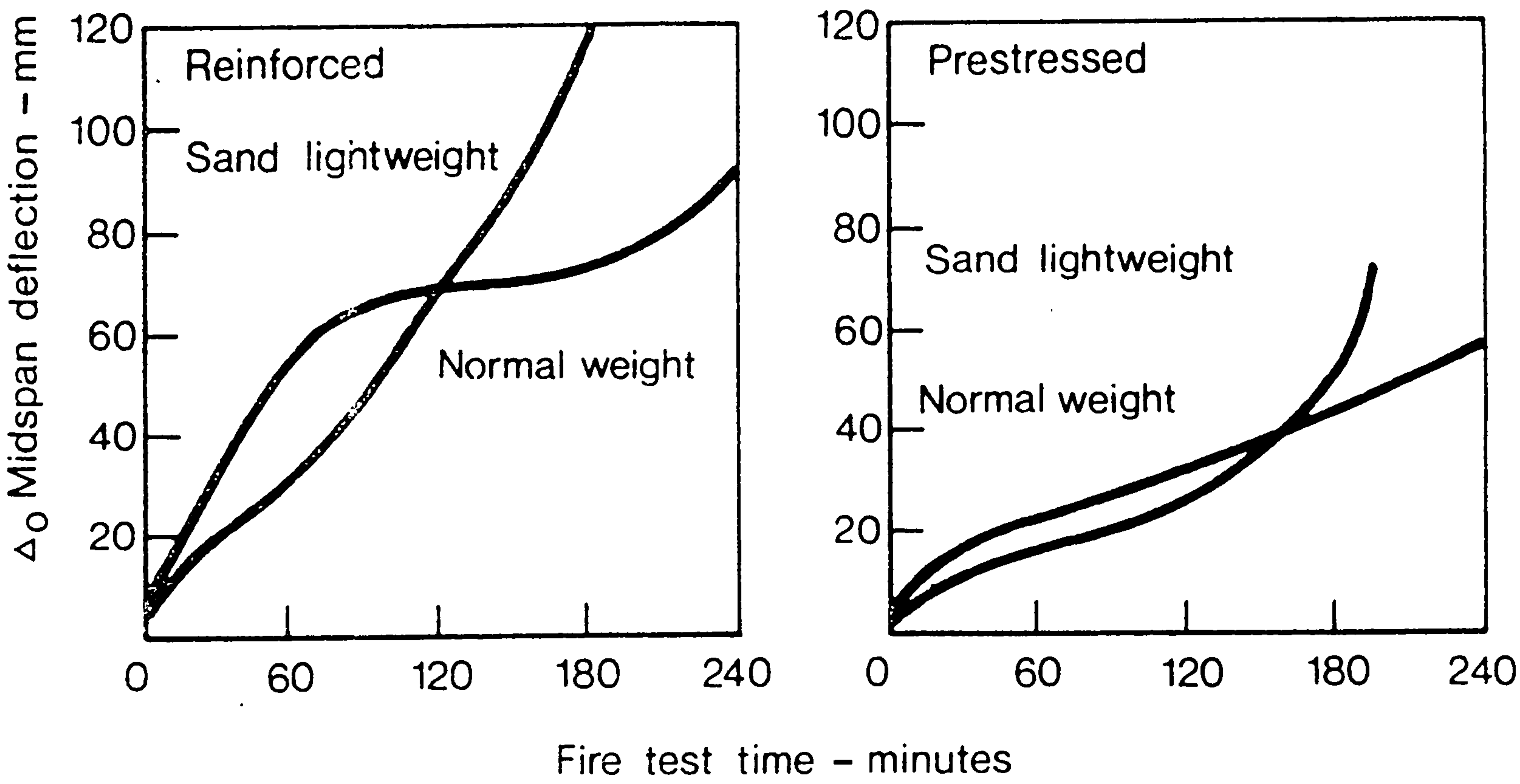


Figure 22: Idealised mid-span deflection of reference specimen with minimal restraint (Cement and Concrete 1978)

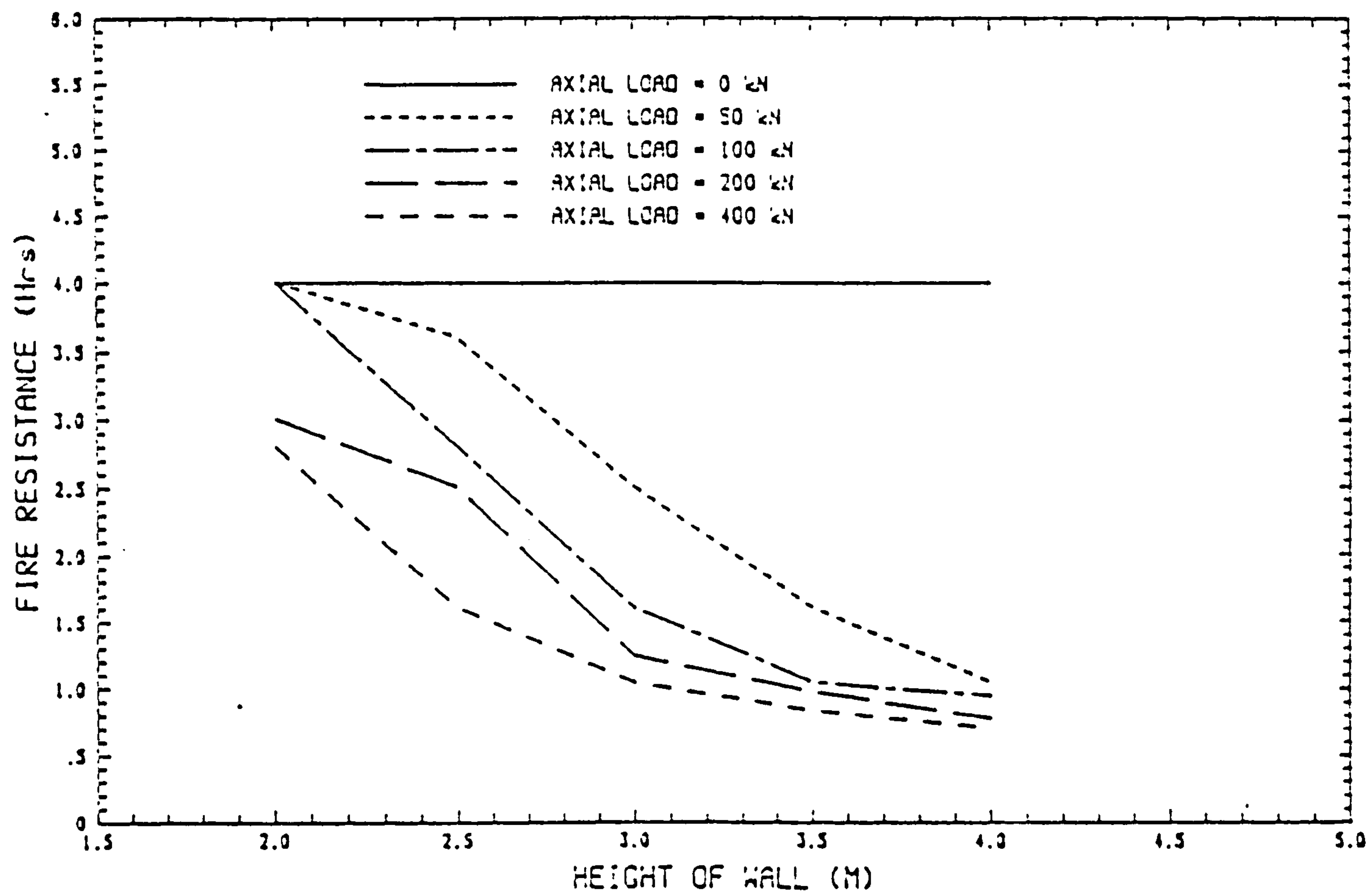


Figure 23: Effect of axial load and height on the fire resistance of a 150 mm thick wall (both ends pinned)

TABLE 1 REQUIREMENTS FOR MINIMUM COVER FROM NZS 3101

	Minimum cover to principal steel reinforcement (mm)
Cast in place	
Exposed to weather	50
Protected from weather	40
Precast	
Exposed to weather	40
Protected from weather	
> or = 16 dia bars	35
< or = 12 dia bars	30

TABLE 2

STANDARD FIRE RESISTANCE TESTING - PERFORMANCE CRITERIA

TEST METHOD	BEAM	COLUMN	WALL	FLOOR/CEILING
ASTM E119 (1983)	1 Must sustain max design load during test. 2 Assembly tested with restraint but to be used unrestrained: temp of reinforcing bar must not exceed 593 °C prestressing steel temp must not exceed 427 °C. 3 Assembly tested with restraint and to be used restrained and spaced more than 1.2 m on centres: 2, above applies up to 1 hr FRR. If greater than 1FRR temps in 2. above only apply for half FRR required or 1 hr which ever is greater.	1 Must sustain max design load during test	1 If loadbearing must sustain design load during fire test and hose stream test 2 Must be no ignition of cotton waste on unexposed face 3 No openings allowed to develop from hose stream 4 Average unexposed face temp rise must not exceed 139 °C.	1 As for beams 2 Must be no ignition of cotton waste on unexposed face 3 Average unexposed face temp rise must not exceed 139°C
AS 1530, 4 (1990)	1 Max deflection must not exceed span/20; or 2 Max rate of deflection must not exceed span $^2/9000d$ where d = distance from top of section to bottom of tension zone (mm) but doesn't apply before deflection of span/30	1 Must be able to carry applied load if axially loaded. 2 As for beams if transversely loaded.	1 Must not collapse 2 Must not develop gaps through which flames or hot gases can pass 3 Average unexposed face temp rise must not exceed 140 K. 4 Max exposed face temp rise must not exceed 180 K. 5 If loadbearing must be able to carry applied load.	1 As for beams 2 Average unexposed face temp rise must not exceed 140 K. 3 Max unexposed face temp rise must not exceed 180 K. 4 Additional requirements for incipient spread of flame apply.

TABLE 2 (Continued)

STANDARD FIRE RESISTANCE TESTING - PERFORMANCE CRITERIA

TEST METHOD	BEAM	COLUMN	WALL	FLOOR/CEILING
ISO 834 (1975)	1 Must not collapse	1 Must not collapse	1 If loadbearing must not collapse. 2 Must not develop gaps through which flames or hot gases can pass. 3 Average unexposed face temp rise must not exceed 140°C. 4 Max unexposed face temp must not exceed 180°C.	1 Must not collapse 2 Must not develop gaps through which flames or hot gases can pass. 3 Average unexposed face temp rise must not exceed 140°C. 4 Max unexposed face temp rise must not exceed 180 °C.
BS 476 (1987)	1 Max deflection must not exceed span/20; or 2 Max rate of deflection must not exceed $span^2$ /9000d where d = distance from top of section to bottom of tension zone (mm) but doesn't apply before deflection of span/30.	1 Must support load expected in service.	1 If loadbearing must support load expected in service. 2 Integrity must be maintained. 3 Average unexposed face temp rise must not exceed 140°C. 4 Max unexposed face temp rise not exceed 180°C.	1 As for beams 2 Integrity must be maintained. 3 Average unexposed face temp rise must not exceed 140°C. 4 Max unexposed face temp rise must not exceed 180°C

TABLE 3

REINFORCED CONCRETE BEAMS - SUMMARY OF TABULAR DATA

FIRE RESISTANCE RATING	0.5 H Width (mm)	Cover (mm)	1.0 H Width (mm)	Cover (mm)	1.5 H Width (mm)	Cover (mm)	2.0 H Width (mm)	Cover (mm)	3.0 H Width (mm)	Cover (mm)	4.0 H Width (mm)	Cover (mm)	NOTES
DOCUMENT, STANDARD OR CODE													
MP9: 1989 (NZ)	-	-	100	25	140	35	180	45	240	55	280	65	siliceous aggregate
CP110: 1972 (UK)	80	15	110	25	140	35	180	45	240	55	280	65	siliceous aggregate
BS8110: 1985 (UK)	80	20	120	30	150	40	200	50	240	70	280	80	dense aggregate
BRE: 1988 (UK)	80	20	120	30	150	40	200	50	240	70	280	80	dense aggregate effective cover
FORREST AND LAW:1984 (UK)	200	15	200	20	200	30	200	50	250	70	300	80	dense aggregate min width = 200
FIP/CEB: 1975 (EUR)	80 120 160 200	20 10 10 10	120 160 200 300	35 30 25 20	150 200 280 400	50 40 35 30	200 240 300 500	60 50 45 40	240 300 400 600	75 65 60 55	280 350 500 700	85 75 70 65	dense aggregate critical temp = 500°C
CEB: 1987 (EUR)	80	25	120 160 200	40 35 30	150 200 280 400	55 45 40 35	200 240 300 500	65 55 50 45	240 300 400 600	80 70 65 60	280 350 500 700	90 80 75 70	cover is to steel axis
NBC SUPPLEMENT: 1985 (CAN)	100	20	100	20	100	25	100	25	140	39	165	50	type S, N, L, concrete table 2.9A
UBC: 1988 (USA)	-	-	305	38	-	-	305	38	305	38	305	50	siliceous aggregate not monolithic with slab, table 43-A
AS 3600: 1988 (AUS)	80 700	20 15	120 160 230	30 25 20	150 200 300 700	45 35 30 25	200 240 375 700	55 45 40 30	240 300 700	70 60 45	280 350 700	80 70 55	

All data relates to three - sided exposure. Beams are simply supported.

TABLE 4

PRESTRESSED CONCRETE BEAMS - SUMMARY OF TABULAR DATA

FIRE RESISTANCE RATING	0.5 H Width (mm)	Cover (mm)	1.0 H Width (mm)	Cover (mm)	1.5 H Width (mm)	Cover (mm)	2.0 H Width (mm)	Cover (mm)	3.0 H Width (mm)	Cover (mm)	4.0 H Width (mm)	Cover (mm)	NOTES
DOCUMENT, STANDARD OR CODE													
MP9: 1989 (NZ)	-	25	-	32	-	50	-	65	-	80	-	100	unprotected beam
CP110: 1972 (UK)	80	25	110	40	140	50	180	65	240	85	280	100	siliceous aggregate
BS8110: 1985 (UK)	100	25	120	40	150	55	200	70	240	80	280	90	dense aggregate
BRE: 1988 (UK)	100	25	120	40	150	55	200	70	240	80	280	90	dense aggregate effective cover
FORREST AND LAW:1984 (UK)	200	20	200	30	200	45	200	70	250	80	300	90	dense aggregate min width = 200
FIP/CEB: 1975 (EUR)	80 120 160 200	30 20 20 10	120 160 200 300	45 40 35 30	150 200 280 400	60 50 45 40	200 240 300 500	70 60 55 50	240 300 400 600	85 75 70 65	280 350 500 700	95 85 80 75	dense aggregate critical temp = 400°C
CEB: 1987 (EUR)	80	25	120 160 200	40 35 30	150 200 280 400	55 45 40 35	200 240 300 500	65 55 50 45	240 300 400 600	80 70 65 60	280 350 500 700	90 80 75 70	cover is to steel axis
NBC SUPPLEMENT: 1985 (CAN)	- - -	25 25 25	- - -	50 39 39	- - -	64 45 39	- - -	- 64 50	- - -	- - 77	- - -	- - 102	260-970 cm^2 970-1940 cm^2 >1940 cm^2 table 2.10 A, type S,N
UBC: 1988 (USA)	-	-	203 305	44 38	- -	- -	203 305	64 50	203 305	114 64	- 305	- 76	siliceous aggregate
AS 3600: 1988 (AUS)	80 700	25 20	120 160 230	35 30 25	150 200 300 700	55 45 40 35	200 240 375 700	65 55 50 40	240 300 700	80 70 55	280 350 700	90 80 65	

All data relates to three - sided exposure . Beams are simply supported.

TABLE 5

STRUCTURAL CONCRETE COLUMNS - SUMMARY OF TABULAR DATA

FIRE RESISTANCE RATING	0.5 H Width (mm)	Cover (mm)	1.0 H Width (mm)	Cover (mm)	1.5 H Width (mm)	Cover (mm)	2.0 H Width (mm)	Cover (mm)	3.0 H Width (mm)	Cover (mm)	4.0 H Width (mm)	Cover (mm)	NOTES
DOCUMENT, STANDARD OR CODE													
MP9: 1989 (NZ)	-	-	150	20	200	30	250	38	300	50	400	50	siliceous aggregate
CP110: 1972 (UK)	150	-	200	-	250	-	300	-	400	-	450	-	siliceous aggregate
BS8110: 1985 (UK)	150	20	200	25	250	30	300	35	400	35	450	35	dense aggregate
BRE: 1988 (UK)	150	20	200	25	250	30	300	35	400	35	450	35	dense aggregate
FORREST AND LAW:1984 (UK)	150	20	200	25	250	30	300	35	400	35	450	35	dense aggregate
FIP/CEB: 1975 (EUR)	150	10	200	20	240	30	300	35	400	35	450	35	dense aggregate table 1a
CEB: 1987 (EUR)	150	25	200	35	240	50	300	50	400	50	450	55	cover is to steel axis 3m long, 20 MPa concrete
NBC SUPPLEMENT: 1985 (CAN)	150	13	200	25	250	38	300	50	400	63	500	75	overdesign factor = 1 types S, N, kh < 3.7
UBC: 1988 (USA)	-	-	305	38	-	-	305	38	305	38	305	50	siliceous aggregate table 43-A
AS 3600: 1988 (AUS)	150	10	200 240	20 15	240 300	35 25	300 400	45 35	400 500	60 50	450 600	70 60	

All data relates to a fully exposed (4 sides) column

TABLE 6

REINFORCED CONCRETE SLABS - SUMMARY OF TABULAR DATA

FIRE RESISTANCE RATING	0.5 H Width (mm)	Cover (mm)	1.0 H Width (mm)	Cover (mm)	1.5 H Width (mm)	Cover (mm)	2.0 H Width (mm)	Cover (mm)	3.0 H Width (mm)	Cover (mm)	4.0 H Width (mm)	Cover (mm)	NOTES
DOCUMENT, STANDARD OR CODE													
MP9: 1989 (NZ)	60	15	80	20	100	20	120	20	150	25	175	25	simply supported
CP110: 1972 (UK)	100	15	100	15	125	20	125	20	150	25	150	25	siliceous or calcareous solid slabs, avg cover
BS8110: 1985 (UK)	75	15	95	20	110	25	125	35	150	45	170	55	dense aggregate, plain soffit, simply supported
BRE: 1988 (UK)	75	15	95	20	110	25	125	35	150	45	170	55	dense aggregate, plain soffit, simply supported effective cover
FORREST AND LAW:1984 (UK)	75	15	95	20	110	25	125	35	150	45	170	55	dense aggregate, plain soffit, simply supported
FIP/CEB: 1975 (EUR)	60	10	80	20	100	30	120	40	150	55	175	65	dense aggregate, 1-way span, simply supported critical temp = 550 °C
CEB: 1987 (EUR)	60	10	80	25	100	35	120	45	150	60	175	70	dense aggregate, 1-way span, simply supported critical temp = 500°C cover to steel axis
NBC SUPPLEMENT: 1985 (CAN)	60	20	90	20	112	20	130	25	158	32	180	39	type S concrete
UBC: 1988 (USA)	-	-	89	19	-	-	127	25	157	25	-	32	siliceous aggregate
AS 3600: 1988 (AUS)	60	15	80	20	100	25	120	30	150	45	170	55	1-way simply supported

TABLE 7

PRESTRESSED CONCRETE SLABS - SUMMARY OF TABULAR DATA

FIRE RESISTANCE RATING	0.5 H Width (mm)	Cover (mm)	1.0 H Width (mm)	Cover (mm)	1.5 H Width (mm)	Cover (mm)	2.0 H Width (mm)	Cover (mm)	3.0 H Width (mm)	Cover (mm)	4.0 H Width (mm)	Cover (mm)	NOTES
DOCUMENT, STANDARD OR CODE													
MP9: 1989 (NZ)	60	13	80	25	100	32	120	38	150	50	175	64	normal-weight simply supported
CP110: 1972 (UK)	90	15	100	25	125	30	125	40	150	50	150	65	siliceous or calcareous solid slabs, avg cover
BS8110: 1985 (UK)	75	20	95	25	110	30	125	40	150	55	170	65	dense aggregate, plain soffit, simply supported
BRE: 1988 (UK)	75	20	95	25	110	30	125	40	150	55	170	65	dense aggregate, plain soffit, simply supported effective cover
FORREST AND LAW:1984 (UK)	75	20	95	25	110	30	125	40	150	55	170	65	dense aggregate, plain soffit, simply supported
FIP/CEB: 1975 (EUR)	60	20	80	30	100	40	120	50	150	65	175	75	dense aggregate, 1-way span, simply supported critical temp = 400°C
CEB: 1987 (EUR)	60	20	80	35	100	45	120	55	150	70	175	80	dense aggregate, 1-way span, simply supported critical temp = 400°C cover to steel axis
NBC SUPPLEMENT: 1985 (CAN)	60	20	90	25	112	32	130	39	158	50	180	64	type S concrete
UBC: 1988 (USA)	-	-	89	30	-	-	127	48	157	61	-	-	siliceous aggregate pretensioned
AS 3600: 1988 (AUS)	60	20	80	25	100	35	120	40	150	55	170	65	1-way simply supported

TABLE 8													
REINFORCED CONCRETE WALLS - SUMMARY OF TABULAR DATA													
FIRE RESISTANCE RATING	0.5 H Width (mm)	Cover (mm)	1.0 H Width (mm)	Cover (mm)	1.5 H Width (mm)	Cover (mm)	2.0 H Width (mm)	Cover (mm)	3.0 H Width (mm)	Cover (mm)	4.0 H Width (mm)	Cover (mm)	NOTES
DOCUMENT, STANDARD OR CODE													
MP9: 1989 (NZ)	-	-	75	-	100	-	120	-	150	-	175	-	siliceous
CP110: 1972 (UK)	75	-	75	-	100	-	100	-	150	-	180	-	> 1% reinforcement
BS8110: 1985 (UK)	100 75	25 15	120 75	25 15	140 100	25 25	160 100	25 25	200 150	25 25	240 180	25 25	0.4 to 1% reinforcement > 1% reinforcement dense aggregrate
BRE: 1988 (UK)	100 75	25 15	120 75	25 15	140 100	25 25	160 100	25 25	200 150	25 25	240 180	25 25	0.4 to 1% reinforcement > 1% reinforcement dense aggregrate
FORREST AND LAW: 1984 (UK)	100 75	25 15	120 75	25 15	140 100	25 25	160 100	25 25	200 150	25 25	240 185	25 25	0.4 to 1% reinforcement > 1% reinforcement dense aggregrate
FIP/CEB: 1975 (EUR)	100 60	10 -	120 80	10 -	140 100	15 -	160 120	25 -	200 150	25 -	240 175	25 -	loadbearing non loadbearing dense aggregrate
CEB: 1987 (EUR)	120 60	10 -	120 80	15 -	140 100	25 -	160 120	35 -	200 150	55 -	240 175	75 -	loadbearing, concrete stress < 0.15 x characteristic strength cover to steel axis loadbearing dense aggregrate
NBC SUPPLEMENT: 1985 (CAN)	60	-	90	-	112	-	130	-	158	-	180	-	type S concrete loadbearing and nlb
UBC: 1988 (USA)	-	-	89N	-	-	-	127	-	157	-	178	-	siliceous aggregrate Table 43-B
AS 3600: 1988 (AUS)	60	20	80	20	100	35	120	40	150	45	170	50	Limits on slenderness ratio depending on axial force

TABLE 9 VARIATION IN COVER TO STEEL		
Minimum increase in width (mm)	Decrease in cover	
	dense concrete (mm)	lightweight concrete (mm)
25	5	5
50	10	10
100	15	15
150	15	20

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