

CI/SfB | | Hq4 | (K) Date February 1991

TECHNICAL RECOMMENDATION

NO: 8

METHOD FOR FIRE ENGINEERING DESIGN OF STRUCTURAL CONCRETE BEAMS AND FLOOR SYSTEMS

Colleen Wade

RESTRICTED

METHOD FOR FIRE ENGINEERING DESIGN OF STRUCTURAL CONCRETE BEAMS AND FLOOR SYSTEMS

RELEVANCE

The rational design procedure described in this document applies structural engineering principles and material properties at elevated temperature to the calculation of fire resistance of reinforced or prestressed concrete beams and floor slabs.

Current methods used in New Zealand usually rely on notional tabulated data provided in SANZ Miscellaneous Publication MP9 (SANZ, 1989) for reinforced or prestressed concrete structural elements. This tabular data specifies fire resistance according to the minimum width or thickness of the element and the amount of concrete cover provided to tensile steel. In many cases, the use of this tabulated data will be sufficient, particularly when fire resistance requirements do not govern the design of the element. When fire resistance governs the dimensions of the element then a rational design approach may well be appropriate particularly for members requiring fire resistance of more than about 90 to 120 minutes for reinforced concrete and about 60 minutes for prestressed concrete.

When fire resistance requirements do govern cover requirements for flexural elements, the rational fire design procedure can be used to most advantage by optimising design to take into account a range of properties (such as restraint, amount of steel and degree of loading) other than just element width or thickness or concrete cover to tensile steel. All of these parameters can have an influence on the amount of fire resistance achieved.

<u>ACCEPTANCE</u>

It is recommended that concrete beams and floors designed for fire resistance in accordance with this document be accepted as a means of compliance with New Zealand building code requirements.

The failure criterion used by this design procedure is that of 'Loadbearing Capacity' (or collapse) for slabs and beams, and in addition 'Insulation' for slabs. The latter requires that the temperature rise on the side not directly exposed to fire be limited to 140° C. For concrete separating elements, such as slabs, it is assumed that adequate detailing and absence of severe spalling will ensure that 'Integrity' is maintained (i.e., no holes or gaps develop which would allow the passage of flames or smoke). These criteria are consistent with internationally accepted criteria for fire resistance testing of beams and floors, except that vertical deflection of the element is not considered. This factor is not regarded as significant as an increasing rate of deflection will normally precede the collapse of the member.

The mode of failure is assumed to be in flexure and the time at failure is taken as the time at which the moment capacity of the element, which reduces as the

temperature within the section increases, becomes less than the moment applied to the element. Shear failure of flexural members in fire rarely occurs, but can be checked for in the usual way using material properties at elevated temperatures and factored loads.

As safety factors are already incorporated into the fire resistance period and the normal design loads, in fire design it is usual to allow the load factors and design live loads to be reduced.

BACKGROUND

The design method and theory described in this document are largely based on that described by Gustaferro and Martin (1977) and ACI (1981) with minor additions and modifications. The basic procedure has been developed and verified from a large number of fire resistance tests sponsored by the Portland Cement Association in the USA.

Simply Supported (Unrestrained) Elements (Figure 1)

During the course of a fire, the moment capacity of a simply supported element reduces as a result of a reduction in steel strength at higher temperatures. Since the applied moment remains the same as the fire progresses, collapse (flexural failure) is expected to occur when the moment capacity of the element becomes less than the applied moment.

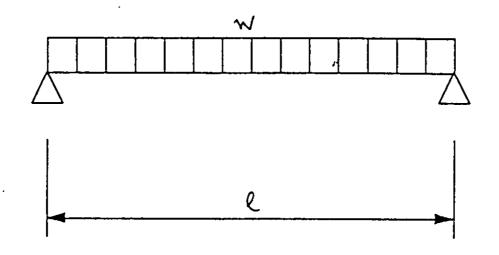
Restraint Due to Moment Continuity (Figures 2 and 3)

Continuous members undergo changes in stresses when exposed to fire. These changes result from temperature gradients within the members and also from different material properties at high temperatures. During a fire, members tend to curve downward due to the thermal gradient which produces different amounts of thermal expansion on the hot and cool faces, and thereby increases the moment at interior supports of continuous members. If the fire is intense, the negative moment reinforcement over interior supports will yield. This change or redistribution of moments generally occurs early during a fire.

During the fire, the positive moment reinforcement is heated and its strength diminished. As a result, the positive moment capacity diminishes. However, the negative moment capacity diminishes more slowly because the top reinforcement tends to remain cooler. Thus because of the redistribution of moments, the fire resistance of a continuous element is significantly greater than that of a simply supported element having the same dimensions and loaded to the same moment intensity.

Restraint Against Thermal Expansion (Figure 4)

When a fire occurs in a localised area within a building and heats 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 with a resistive force known as the thermal thrust. Provided that the line of action of the thrust falls below the



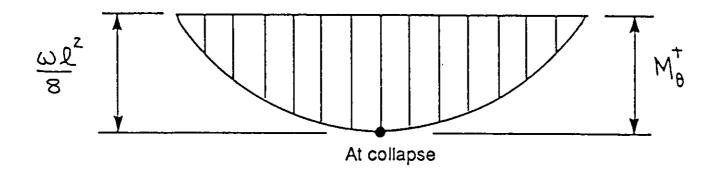
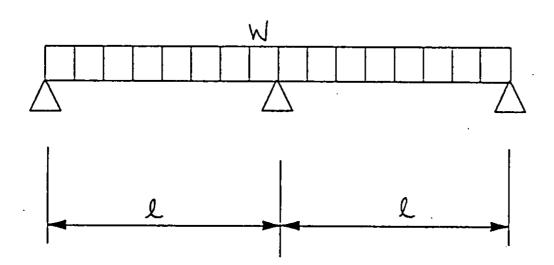


Figure 1



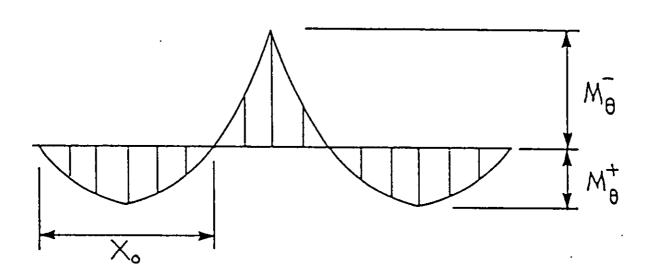
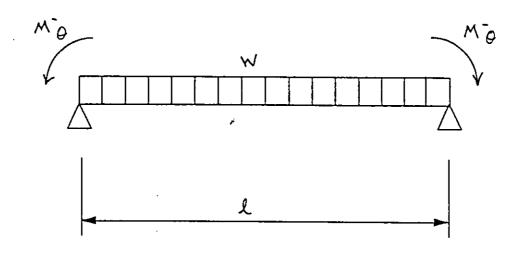


Figure 2



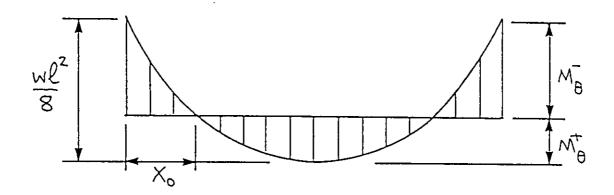
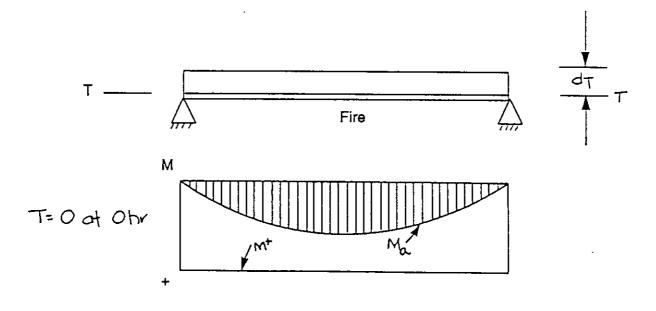
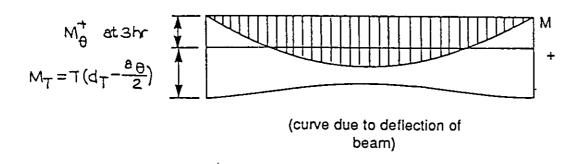


Figure 3





Moment diagrams for axially restrained beam during fire exposure. Note that at 3 hr M_{θ}^{\dagger} is less than M_{a} and the effects of axial restraint permit the beam to continue to support load.

resultant of the compressive stress block of the element, the thrust will increase the moment capacity of the element and therefore increase its fire resistance. For this reason some proprietary floor systems have different fire resistance ratings for 'restrained' and 'unrestrained' conditions. Designers are cautioned against detailing the element in such a way that the line of thermal thrust falls above the resultant of the compressive stress block, leading to premature collapse of the element when exposed to fire (e.g., flange supported double tee sections). Furthermore, columns and spandrel beams in the exterior bays of buildings have only a very limited ability to provide restraint against thermal expansion during fire. If restraint is required, the building in question must be checked to find out whether it can provide that restraint, considering the likely size of the fire and the arrangement of stiff lateral load resisting elements.

SCOPE OF APPLICATION

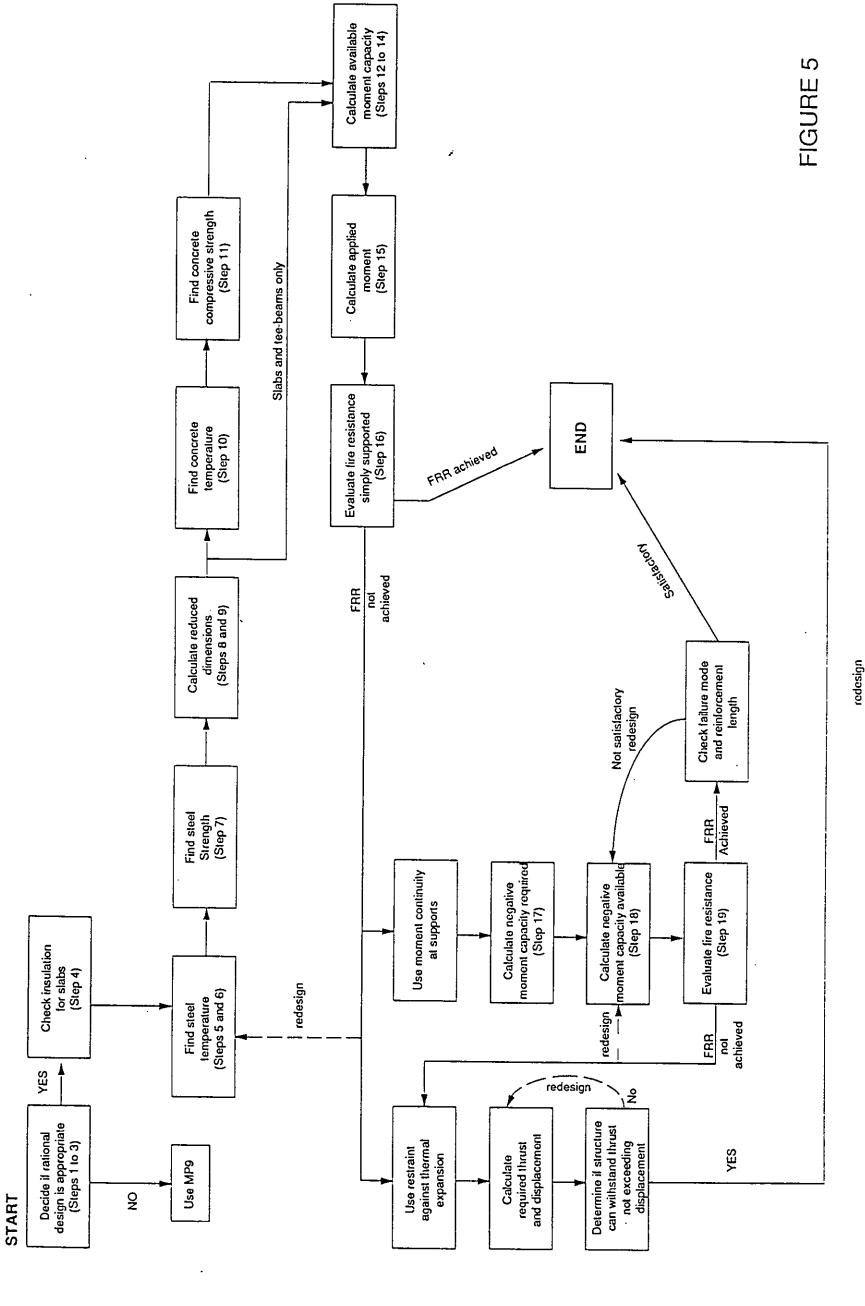
The rational design procedure described in this document applies to concrete beams or floor/ceiling slabs which:

- May be rectangular or tapered in cross-section (for beams), or T-beams, with a minimum width of 100 mm. The width should be measured at the centroidal axis of the tensile steel.
- Are made with normal-weight concrete (assumed density greater than or equal to 2000 kg/m³ i.e., includes most New Zealand siliceous aggregates) or lightweight concrete (assumed density less than 2000 kg/m³ e.g., pumice aggregate).
- ◆ Are simply supported (with or without restraint against thermal expansion), or continuous over supports including both end bays and internal bays in a framed structure.
- Include either prestressing tendons or strands or reinforcing bars.
- ◆ Are required to meet 60, 90, 120, 180 or 240 minutes of fire resistance.
- ◆ Are exposed to the time-temperature heating conditions described in ISO 834.
- In all other respects are in general accordance with the requirements of NZS 3101 New Zealand Code of Practice for the Design of Concrete Structures and NZS 4203 New Zealand Standard Code of Practice for General Structural Design and Design Loadings for Buildings.

RATIONAL DESIGN PROCEDURE

The overall design procedure is summarised in the flow chart shown in Figure 5. The basic philosophy applies conventional design theory but uses reduced strength of concrete and steel at elevated temperatures.

It is assumed that a preliminary ambient temperature design of the beam or floor has previously been carried out, and that the following information is available:



- element dimensions, span and support conditions.
- dead and live loads to be applied.
- steel type, and its location within the element cross-section.
- yield strength of reinforcing bars or ultimate tensile strength of prestressing tendons or strands.
- concrete type, density and 28-day cylinder compressive strength.

Two worked examples showing practical use of the following rational design procedure are included as an Appendix to this document. Other examples of the design procedure in use can be found in ACI (1981) and Gustaferro and Martin (1977). Readers are also referred to Wade (1991) for further background information on fire engineering design of concrete elements.

ASSESS WHETHER RATIONAL DESIGN IS APPROPRIATE

Step 1 Determine the required fire resistance rating from New Zealand building code requirements or as otherwise may be required.

Step 2 Determine the amount of cover to steel and minimum dimensions of the element to establish fire resistance by referring to the appropriate tables in Standards Association Miscellaneous Publication MP9. Similarly, if a proprietary system is to be used, check MP9 for current approvals. Floor systems meeting the cover requirements of MP9 will have an 'unrestrained' fire resistance rating.

Step 3 If the required fire resistance is achieved according to MP9 then no further calculation is necessary. If the required fire resistance is not achieved then either increase cover and/or beam dimensions until MP9 tables give required fire resistance, or alternatively use the rational design method described below.

CHECK INSULATION

Step 4 Check insulation requirements (for slabs only).

Determine the minimum required thickness of the concrete slab from MP9, which is generally based upon thermal transmission requirements for satisfying the Insulation criterion.

RATIONAL DESIGN METHOD FOR LOADBEARING CAPACITY

Determine Nominal Temperature of Tensile Steel

Step 5 Calculate the effective cover (C_e) to the tensile reinforcing or prestressing steel.

The effective cover (C_e) is taken to be the weighted average of the distances between the centres of individual steel strands or bars and the nearest surface exposed to fire.

For example, the effective cover applicable to the tensile steel in the beam shown in Figure 6 is calculated as follows:

$$C_e = \frac{A_1C_1 + A_2C_2 + A_3C_3 + A_4C_4 + A_5C_5 + A_6C_6}{A_1 + A_2 + A_3 + A_4 + A_5 + A_6}$$

where: $A_n = \text{cross-sectional}$ area of the *n*th reinforcing bar or strand; and $C_n = \text{cover}$ to the axis of the *n*th reinforcing bar or strand.

Step 6 Use the value of effective cover (C_e) to determine the nominal temperature (T_s) of the tensile steel from Figure 7 for slabs or Figure 8 for beams or ribs of tee sections. Figures 7a and 7b are redrawn from curves given in ACI (1981) for siliceous and sanded-lightweight aggregates respectively. The original tests were performed by the Portland Cement Association (PCA), USA (Abrams and Gustaferro, 1968). Figures 8a to 8e are redrawn from ISE (1978). It is stated that they are also based on information obtained from PCA tests.

Enter the diagrams on the horizontal axis with effective cover. Draw a vertical line to intersect the curve applicable to the width of the beam (Figure 8) or the required fire resistance of the slab (Figure 7). Draw a horizontal line through the intersecting point on the curve to the vertical axis. The required temperature is read from the vertical axis.

For lightweight concrete beams or webs, in the absence of specific design curves for temperature versus effective cover, multiply the temperature values obtained from Figure 8 for dense concrete by 0.8 to allow for the lower thermal diffusivity (and extra insulating capability) of the lightweight concrete.

It is acceptable to interpolate between curves (Figure 8) for intermediate widths of beam. For beams with a width greater than 300 mm, use the curve applicable to a 300 mm wide beam. For beams tapered in cross-section, the width of the beam should be determined at the height of the centroidal axis of the tensile steel.

Determine Strength of Steel at Elevated Temperature

Step 7 Determine the proportion of yield strength for reinforcing steel, or ultimate tensile strength for prestressing steel, at the nominal temperature (T_s) of the tensile steel using Figure 9. Multiply this value by the ambient temperature value to find the yield strength of reinforcing bars $(F_{y\theta})$ or the ultimate tensile strength of prestressing steel $(F_{pu\theta})$ at elevated temperature.

Determine Values of Element Width and Depth

Step 8 Determine the effective depth at ambient temperature (d) (i.e., distance from the centroidal axis of the tensile steel to the compressive face). For simply supported slabs or tee-beams, where the compression zone is not exposed to fire, go to Step 12 with $d_{\theta} = d$ and $b_{\theta} = b$ (Steps 9 to 11 not required).

Step 9 Determine the reduced effective depth (d_{θ}) and width of the compression

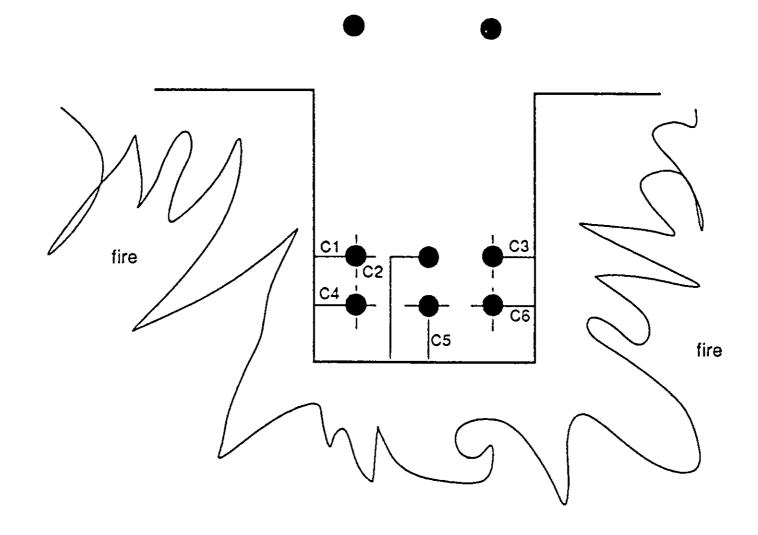


Figure 6



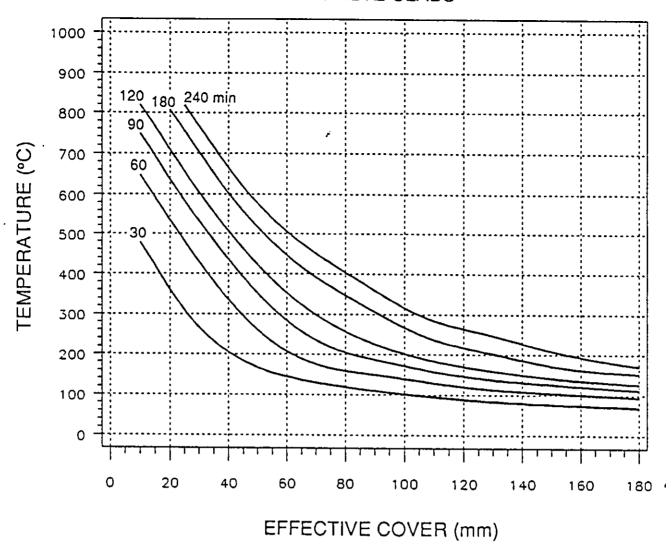


Figure 7a



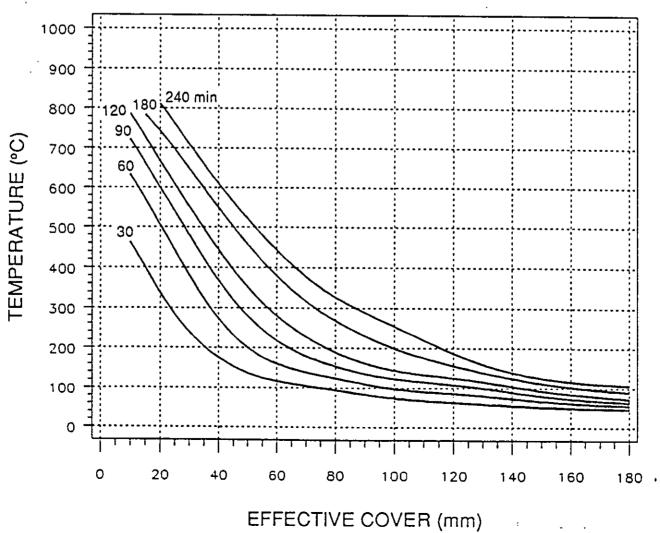


Figure 7b



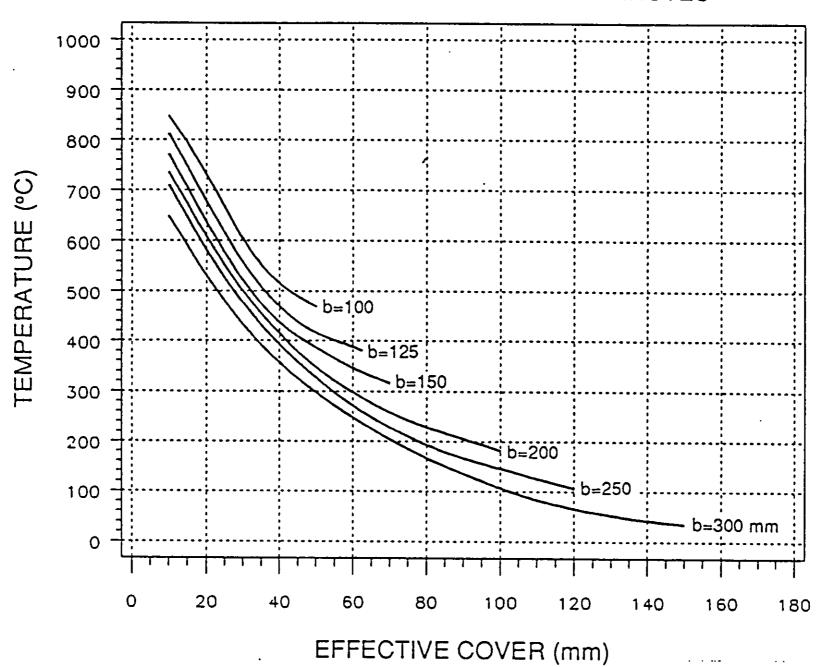


Figure 8a

DENSE CONCRETE BEAMS - 90 MINUTES

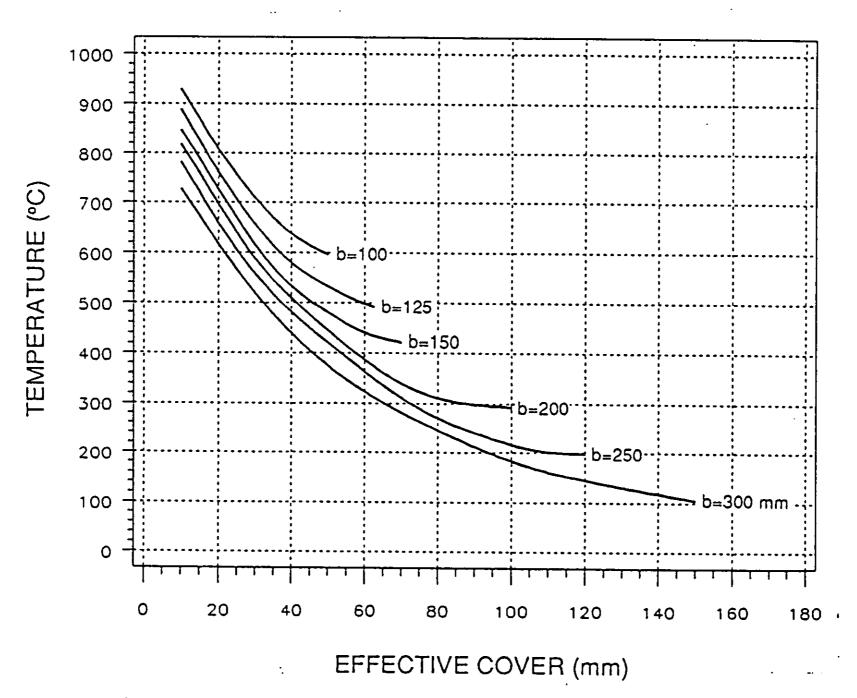
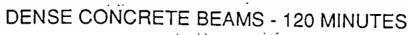


Figure 8b



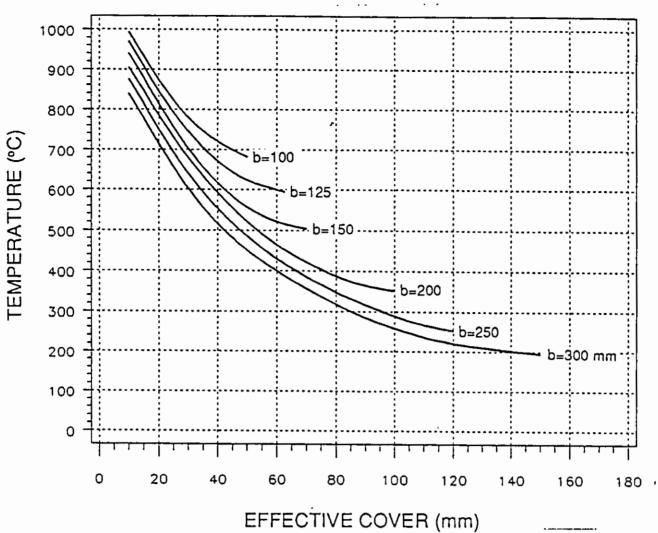


Figure 8c

DENSE CONCRETE BEAMS - 180 MINUTES

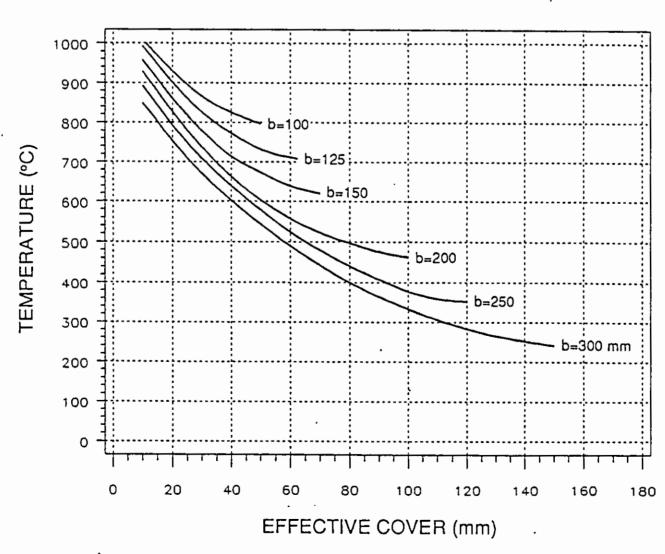


Figure 8d

DENSE CONCRETE BEAMS - 240 MINUTES

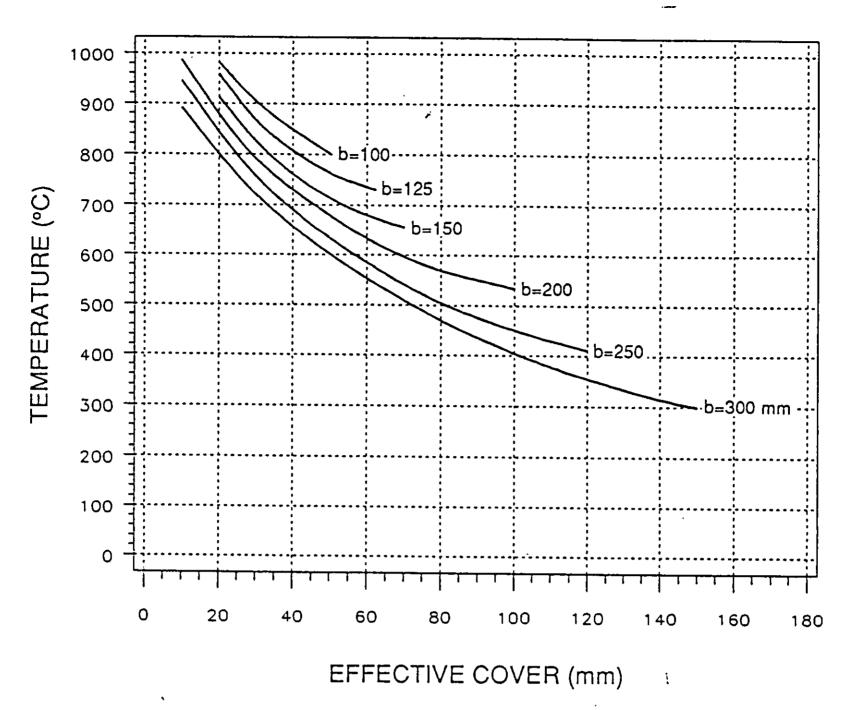
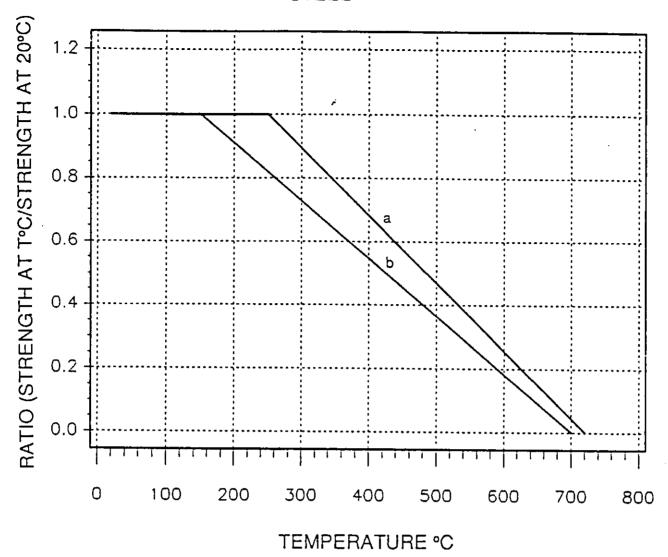


Figure 8e



(a) reinforcing bars (from AS 3600, 1988)

$$\frac{\text{Fy (T)}}{\text{Fy (20°C)}}$$
 =
 { 1.0, T≤ 250°C
 1.53 - T/470, T > 250°C

(b) prestressing steel or strands (from AS 3600, 1988)

Figure 9

zone (b_{θ}) by ignoring all concrete in the compression zone with a temperature greater than $750^{\circ}C$ by using Figures 7 and 8. Where the compressive zone of the element is exposed to fire, d_{θ} will usually be less than d.

In this case, the diagrams are used by entering the vertical axis at 750° C, drawing a horizontal line to intersect the appropriate curve and then drawing a vertical line to intersect the horizontal axis giving the depth of the surface layer which should be ignored in order to calculate d_{θ} and b_{θ} .

Estimate Nominal Temperature of Concrete in Compression Zone

Step 10 Estimate the nominal temperature of the concrete (T_c) in the equivalent rectangular compressive stress block.

For beams and ribs of tee sections, average the temperature at mid-width and at the side of the compressive stress block (usually the average of $750^{\circ}C$ and the mid-width temperature. See Figure 10). Use Figures 7 and 8.

As the nominal temperature of the compressive stress block (T_c) is influenced by the depth of the compressive stress block (a_{θ}) , and vice versa, an iterative or trial and error approach is sometimes required. Furthermore, where the compressive zone is not directly exposed to the fire, the assumption that $T_c < 350^{\circ}C$ (i.e., $f'_{c\theta} = f'_c$) can usually be made, simplifying the next step.

Determine Strength of Concrete at Elevated Temperature

Step 11 Determine the compressive strength of the concrete at the nominal temperature (T_c) of the compressive stress block by using Figure 10. Multiply this value by the 28 day compressive strength to find the compressive strength of concrete $(f'_{c\theta})$ at elevated temperature. If $T_c < 350^{\circ}C$ for dense concrete or $500^{\circ}C$ for lightweight concrete then $f'_{c\theta} = f'_c$.

Determine Available Moment Capacity

Step 12 For prestressing steel only, calculate the stress in the prestressing steel at ultimate load $(F_{ps\theta})$ at the temperature (T_s) using:

$$F_{ps\theta} = F_{pu\theta} \left(1 - \frac{A_s F_{pu\theta}}{2b_\theta d_\theta f'_{c\theta}} \right)$$

Step 13 Calculate the depth of the equivalent rectangular compressive stress block (a_{θ}) after the required period of fire exposure. For slabs, let b = 1 m and calculate moment capacity per metre width.

$$a_{\theta} = \frac{A_{s}F_{y\theta}}{0.85f_{c\theta}'b_{\theta}}$$
 for reinforcing steel

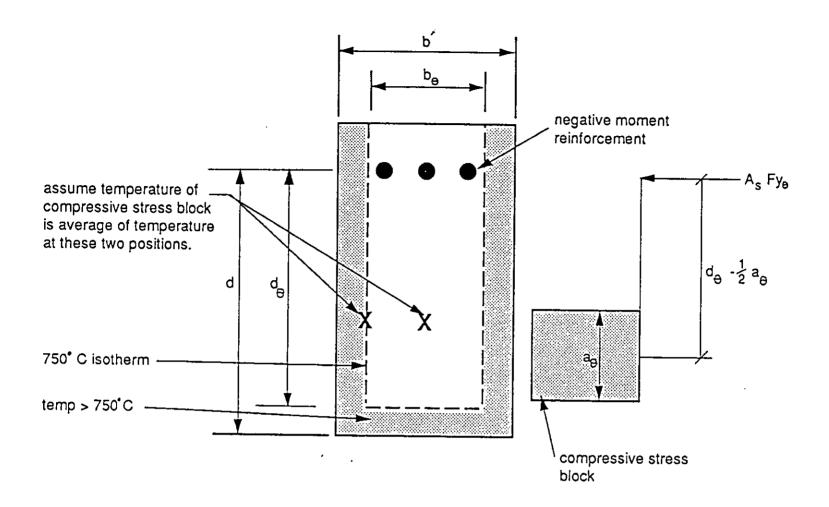
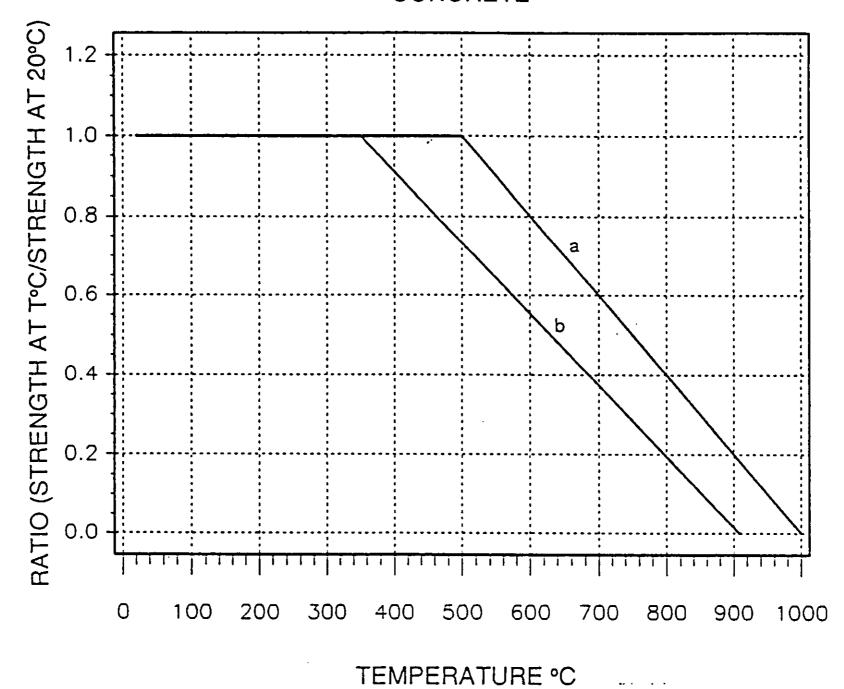


Figure 10

CONCRETE



(a) lightweight concrete (from ISE,1978)

$$\frac{f'c(T)}{f'c(20^{\circ}C)} = \begin{cases} 1.0, & T \le 500^{\circ}C \\ 2.0 - 0.6T, & T > 500^{\circ}C \end{cases}$$

(b) dense concrete (from ISE, 1978)

$$\frac{f'_{C}(T)}{f'_{C}(20^{\circ}C)} = \begin{cases} 1.0, & T \le 350^{\circ}C \\ 1.65 - 0.8T & T > 350^{\circ}C \end{cases}$$

Figure 11

$$a_{\theta} = \frac{A_{s}F_{ps\theta}}{0.85f_{c\theta}'b_{\theta}}$$
 for prestressing steel

Step 14 Calculate the available positive moment capacity (M_{θ}^+) (or the negative moment capacity (M_{θ}^-) at a support) after the required period of fire exposure. In fire design the capacity reduction factor can be taken as unity.

$$M_{\theta}^{+}$$
 (or M_{θ}^{-}) = $A_{s}F_{y\theta}\left(d_{\theta}-\frac{a_{\theta}}{2}\right)$ for reinforcing steel

$$M_{\theta}^{+}$$
 (or M_{θ}^{-}) = $A_{s}F_{ps\theta}\left(d_{\theta}-\frac{a_{\theta}}{2}\right)$ for prestressing steel

Simply Supported Element

Step 15 Calculate the maximum applied positive moment.

For a simply supported, uniformly distributed load.

$$M_a = \frac{wl^2}{8}$$

Note — The current draft 2/DZ 4203 specifies the following load combinations for the ultimate limit state for fire:

1.
$$w = 1.0D + \psi L$$

2.
$$w = 1.0D + 0.25W$$

where:
$$\psi = \left\{ egin{array}{ll} 0.4 & {
m for floors (domestic, office, parking and} \\ & {
m trafficable roofs} \\ 0.6 & {
m for floors (storage and other)} \\ 0.0 & {
m for non-trafficable roofs} \end{array} \right.$$

Note — these load combinations may change before final publication of NZS 4203.

Step 16 Evaluate the fire resistance (see Figure 1).

if $M_{\theta}^{+} \geq M_{a}$ the beam has the required period of fire resistance. if $M_{\theta}^{+} < M_{a}$ redesign the beam or adjust the maximum loads and recalculate moments and/or moment capacity or utilise restraint against thermal expansion if it is available.

Element With Moment Continuity

Step 17 For a continuous beam, determine the required negative moment capacity (after moment redistribution occurs) given that the available moment capacity at the location of the maximum positive applied moment is known (as calculated above).

for an end bay:
$$M_{\theta}^- = \frac{wl^2}{2} - wl^2 \sqrt{\frac{2M_{\theta}^+}{wl^2}}$$
 (see Figure 9)

for an intermediate bay: $M_{\theta}^{-} = \frac{wl^2}{8} - M_{\theta}^{+}$ (see Figure 10)

Step 18 Determine negative moment capacity available (or design such that it equals that required) repeating steps 5 to 14.

Step 19 Evaluate fire resistance. If M_{θ}^- available exceeds M_{θ}^- required then the element has required fire resistance; if not then redesign.

Step 20 The failure mode of the element is assumed to be in flexural tension. To avoid a failure of the concrete in the compressive zone the amount of negative moment reinforcement should be small enough for the following expression to be true. If this requirement is not met, the beneficial effect of compression reinforcement can be estimated.

$$\frac{a_{\theta}}{d_{\theta}} = \frac{A_s F_{y\theta}}{0.85 b_{\theta} d_{\theta} f_{c\theta}'} < 0.26$$
 for reinforcing steel

Step 21 For a continuous element determine the positions of the inflection points (X_0) , where moments change from positive to negative, and thus the required lengths of negative moment reinforcement. This length is from the support to the inflection point and the development length is still required to be added. It is further recommended that at least 20% of the total negative moment reinforcement is made continuous over the entire span.

for an end bay:
$$X_0 = l - \frac{2M_{\theta}^-}{wl}$$

for an intermediate bay:
$$X_0 = \frac{l}{2} - \frac{1}{2} \sqrt{\frac{8M_g^+}{w}}$$

The distance of the inflection point from the support increases as the load on the beam decreases therefore a minimum value of load intensity, w = 1.0D is recommended.

Elements With Axial Restraint Against Thermal Expansion

This section is reproduced (with editorial modifications) from a Cement and Concrete Association / FIP (1978) publication being based on Gustaferro and Martin (1977). The procedure is based on extrapolation of test results of the Portland Cement Association (Issen et al., (1970)). Due to recent Swedish research (Anderberg and Forsen, 1982) questioning the scope of this method, it is suggested that it be

applied only to double tee or similar floor sections until such a time as definitive guidance becomes available. In cases where the location of the thrust is difficult to determine the possible positive effects of thermal restraint should be disregarded.

When a fire occurs beneath an interior portion of a slab, the heated portion tends to expand and exert forces on the surrounding slab. The surrounding slab, in turn, pushes against the heated portion, and in effect externally prestresses the slab. When the fire starts, the line of action of the force (thermal thrust or restraint force) acts near the bottom of the slab. The effect of this force on the capacity of the slab is similar to that of 'fictitious reinforcement' located at the line of action of the thrust.

Figure 4 shows diagrammatically the effect of thermal restraint on the behaviour of a uniformly loaded member.

A series of tests to study the effect of restraint on structural behaviour was conducted by the Portland Cement Association, (Issen et al, 1970) which showed that the magnitude of thermal thrust T for a given expansion varied directly with the 'heated perimeter', s, and the concrete modulus of elasticity, E, i.e.,

$$\frac{T_1}{T_0} = \frac{s_1 E_1}{s_0 E_0}$$
 or $\frac{T_1}{A_1 E_1} = \frac{T_0 z_0}{A_0 E_0 z_1}$

where z = A/s

T = thrust

s = heated perimeter

E = concrete modulus of elasticity

A = cross-sectional area normal to the thrust

and subscripts 0 and 1 refer to the reference specimens (from Issen et al) and the member in question.

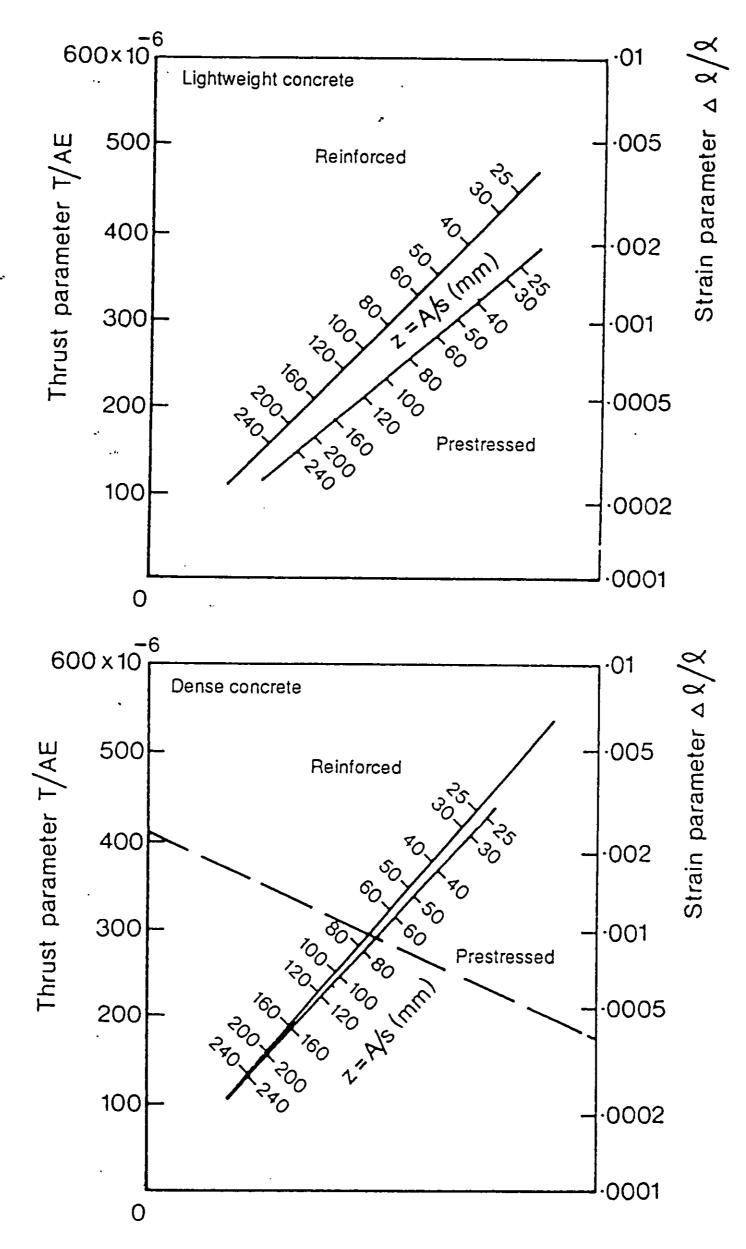
The magnitude of T for a given member is a function of the expansion.

Figure 12 is a set of nomographs that relate thermal thrust, expansion, and heated perimeter for dense and lightweight concrete members that are reinforced or prestressed. The thrust and expansion are expressed in dimensionless parameters T/AE and $\Delta \ell/\ell$.

In design problems involving restraint forces, it is necessary to estimate the deflection of the member. This can be done by making use of data developed during the fire tests of the reference specimens. Figure 13 represents the deflection of the reference specimens in which minimal restraint occurred.

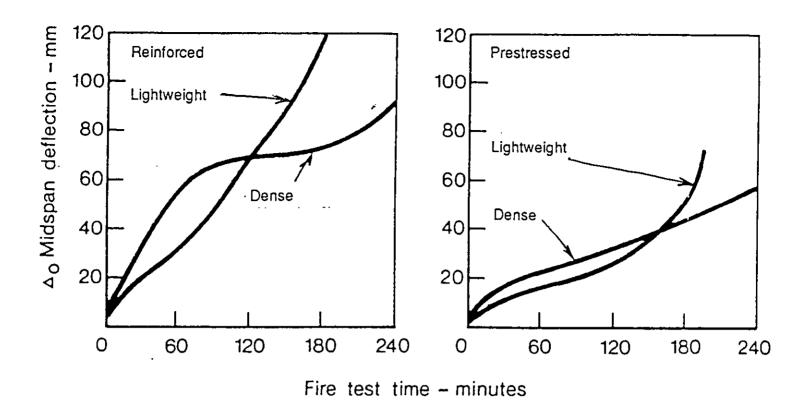
A procedure for estimating the thrust requirements for a given fire resistance for simply supported slabs or beams follows:

1. Determine the available moment capacity, M_{θ}^{+} , after fire exposure for the required period.



Nomograms relating thrust parameter, strain parameter, and ratio of cross-sectional area to heated perimeter.

Figure 12



Midspan deflection, Δ_0 , of reference specimens.

Figure 13

- 2. If the applied moment, M_a , is less than M_{θ}^+ , then the effect of restraint is not required.
- 3. If M_a is greater than M_{θ}^+ , estimate the deflection, Δ_1 , assuming that minimal restraint occurs. Use Figure 13 and the equation below.

$$\Delta_1 = \frac{\ell^2 \Delta_0}{89000 y_{b1}}$$

in which Δ_0 is obtained from Figure 13.

 ℓ = heated length of member

 y_b = distance between the neutral axis of the member and the extreme bottom fibre

- 4. Estimate the location of the thrust line at the supports. For minimal restraint, the thrust line is near the bottom of the member at the support and can be assumed to be 0.1 h above the support where h is the overall depth of member.
- 5. Calculate the magnitude of the required thrust T using the formula:

$$T = \frac{M_a - M_\theta^+}{d_T - a_\theta/2 - \Delta_1}$$

where d_T is the distance between the thrust line and the top of the member at the supports and $a_{\theta} \approx M_a/M^+ \times a$.

- 6. Calculate the thrust parameter, T_1/A_1E_1 and $z=A_1/s_1$.
- 7. Enter Figure 12 with T_1/A_1E_1 and z and determine the strain parameter, $\Delta \ell/\ell$.
- 8. Calculate $\Delta \ell$ by multiplying the strain parameter by the heated length, ℓ .
- 9. Determine independently whether the surrounding structure can withstand the thrust, T, with a displacement no greater than $\Delta \ell$. If the structure cannot withstand the calculated thrust with displacement no greater than $\Delta \ell$, then either M_{θ}^+ must be increased or the surrounding structure must be made stiffer.

PROTECTION AGAINST SPALLING

Spalling is the loss of concrete from the heated surface of concrete when exposed to fire and is mainly attributable to production of gases or vapour within the concrete, high moisture content and induced thermal stresses near the exposed concrete surface. Spalling is detrimental to fire resistance because the concrete cross-sectional area is reduced and steel temperatures will become much higher than assumed in the design calculation. Fire testing by BRANZ of unloaded concrete slabs made from commonly used dense aggregates (with 55 mm clear cover to reinforcing steel)

did not spall (Woodside, de Ruiter and Wade, 1991). Some minor surface spalling was observed for pumice aggregate. Limestone aggregate concrete exhibited severe deterioration following fire testing due to chemical changes, so should is not recommended in concrete structures requiring fire resistance.

The design procedure described in this document assumes that concrete elements have reached their equilibrium moisture content, so that the occurrence of spalling will be minimal.

RELEVANT DOCUMENTS

Abrams, M.S. and Gustaferro, A.H. 1968. Fire Endurance of Concrete Slabs as Influenced by Thickness, Aggregate Type, and Moisture. Research Department Bulletin 223. Research and Development Laboratories of the Portland Cement Association. Illinois, USA

American Concrete Institute (ACI). 1981. Guide for Determining the Fire Endurance of Concrete Elements. ACI 216R-81. Detroit, USA

Cement and Concrete Association. 1978. FIP/CEB Report on Methods of Assessment of the Fire Resistance of Concrete Structural Members. Wexham Springs, Slough, UK

Gustaferro, A.H., and Martin, L.D. 1977. PCI Design for Fire Resistance of Precast Prestressed Concrete. Prestressed Concrete Institute, Illinois, USA

Institution of Structural Engineers, The. 1978. Design and Detailing of Concrete Structures for Fire Resistance. Interim Guidance by a Joint Committee of the Institution of Structural Engineers and the Concrete Society. London

Issen, L.A., Gustaferro, A.H., and Carlson, C.C. 1970. Fire Tests of Concrete Members: An Improved Method for Estimating Thermal Restraint Forces. In Fire Test Performance ASTM STP 464. American Society for Testing and Materials, Philadelphia, USA

Standards Association of Australia. 1988. Concrete Structures. AS 3600. North Sydney

Standards Association of New Zealand. 1982. Code of Practice for the Design of Concrete Structures. NZS 3101 Part 1. Wellington

Standards Association of New Zealand. 1984. Code of Practice for General Structural Design and Design Loadings for Buildings. NZS 4203. Wellington

Standards Association of New Zealand. 1989. Fire Properties of Building Materials and Elements of Structure. Miscellaneous Publication 9. Wellington

Standards Association of New Zealand. 1991. Code of Practice for General Structural Design and Design Loadings for Buildings. 2/DZ 4203. Wellington

Wade, C.A. 1991. Fire Engineering Design of Reinforced and Prestressed Concrete

Elements. BRANZ Study Report No 33. Building Research Association of New Zealand, Judgeford

Woodside, A., de Ruiter, J. and Wade, C.A. 1991. Fire Resistance of New Zealand Concretes. BRANZ Study Report No 34. Building Research Association of New Zealand, Judgeford

SYMBOLS

```
depth of the equivalent rectangular compressive stress block at
\boldsymbol{a}
            ambient temperature (mm)
            depth of the equivalent rectangular compressive stress block at
a_{\theta}
            elevated temperature (mm)
\boldsymbol{A}
            cross-sectional area of member normal to thrust force (mm^2)
4,
            cross-sectional area of tensile reinforcing or prestressing steel (mm^2)
b
            width of beam (mm)
            width of beam (in the compression zone) at elevated
b_{\theta}
            temperature (mm)
C_{e}
            effective cover - the average of the distances between the centre of
            individual steel strands or bars and the nearest surface exposed
            to fire (mm)
d
            effective depth of beam - the distance between the extreme
            compressive fibre and the centroidal axis of tensile steel (mm)
d_T
            distance between the thrust line and the top of the member
d_{\theta}
            effective depth of beam at elevated temperature (mm)
D
            dead load (kN/m)
       =
E
            concrete modulus of elasticity (kN/mm^2)
       =
            compressive strength of concrete after 28 days (MPa)
f_{c\theta}'
            compressive strength of concrete at elevated temperature (MPa)
            stress in prestressing steel at ultimate load at elevated
            temperature (MPa)
            ultimate tensile strength of prestressing steel at ambient
            temperature (MPa)
F_{pu\theta}
            ultimate tensile strength of prestressing steel at elevated
            temperature (MPa)
F_{y}
            yield strength of reinforcing bars at ambient temperature (MPa)
F_{y\theta}
            yield strength of reinforcing bars at elevated temperature (MPa)
            span of beam (m)
       =
\Delta \ell
            allowed thermal expansion (mm)
       =
\ell
            heated length of member (mm)
       =
\boldsymbol{L}
            live load (kN/m)
M_a
            applied bending moment (kNm)
       =
M_{\theta}^{\pm}
            positive bending moment capacity at elevated temperature (kNm)
M_{\theta}^{-}
            negative bending moment capacity at elevated temperature (kNm)
            heated perimeter (mm)
S
T
            thermal thrust force (kN)
       =
```

```
nominal temperature at centre of compressive stress block (°C)
T_c
           nominal temperature of tensile steel ({}^{\circ}C)
T_{\bullet}
      =
           load intensity (kN/m)
           wind load (kN/m)
W
X_0
           distance of inflection point from beam support (m)
            distance between neutral axis of member and the extreme
y_b
            bottom fibre (mm)
            A/s (mm)
\boldsymbol{z}
           midspan deflection (mm)
Δ
\psi
            live load factor
```

APPENDIX

WORKED EXAMPLE 1

<u>Problem.</u> A simply supported reinforced dense concrete beam has been designed for ambient temperature loads. The beam is part of an external wall of a factory located 3 m from the boundary and the fire code requires 4 hours fire resistance. The beam has dimensions 680 mm deep by 300 mm wide and is required to span 12 m carrying a uniform dead load (including self-weight) of 6 kN/m and a uniform live load of 16 kN/m. Tensile reinforcement comprises 6-HD24 bars (452 mm² each) and 2-HD28 bars (615 mm² each) as shown in Figure 14. Assume the beam will be exposed to fire from three sides from below with $F_y = 380$ MPa and $f'_c = 25$ MPa.

Use the rational design procedure to provide 4 hours fire resistance.

. IS RATIONAL DESIGN APPROPRIATE (Steps 1 to 3)

The minimum clear cover provided to the tensile steel is 40 mm, therefore MP9 indicates a fire resistance of only 2 hours for this cover.

Required fire resistance not achieved by MP9 therefore use rational design.

NOMINAL TEMPERATURE OF TENSILE STEEL (Steps 5 and 6)

Calculate effective cover. C_{ϵ} (mm) =

$$\frac{2(452 \times 60) + 2(452 \times 120) + 2(615 \times 54) + 2(452 \times 54)}{(6 \times 452) + (2 \times 615)}$$

$$C_e = \frac{277956 \text{ mm}^3}{3942 \text{ mm}^2} = 71 \text{ mm}$$

Nominal temperature of the tensile steel = $500^{\circ}C$ from Figure 8e at 71 mm from the fire exposed surface.

STRENGTH OF STEEL (Step 7)

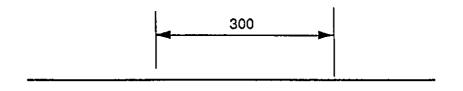
Calculate reduced yield strength of tensile steel using Figure 9.

$$F_{y\theta} = 0.47 F_y = 0.47 \times 380 = 177 \text{ MPa}$$

REDUCED DIMENSIONS (Steps 8 and 9)

Find centroidal axis of tensile steel.

$$\frac{[(2 \times 615) + (2 \times 452)] \times 54 + [4 \times 452] \times 130}{(2 \times 615) + (6 \times 452)} = 89 \text{ mm (above tension face)}$$



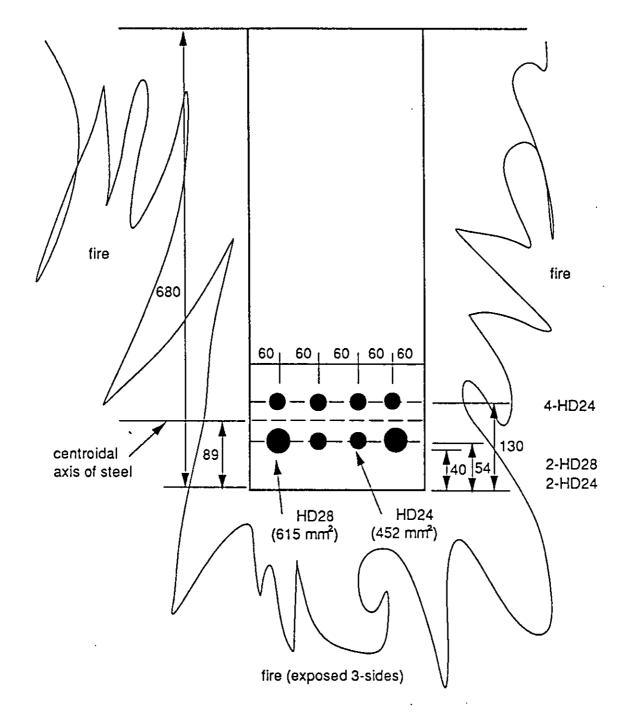


Figure 14

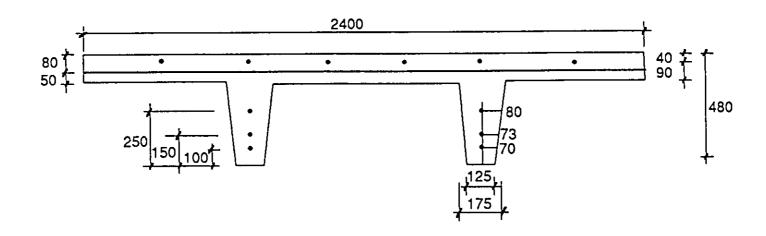


Figure 15

Determine reduced dimensions d_{θ} and b_{θ} . From Figure 8e, ignore surface layer of depth = 26 mm where temperature > $750^{\circ}C$.

$$d_{\theta} = d = 680 - 89 = 591 \text{ mm}$$

$$b_{\theta} = b - 2 \times 26 = 300 - 52 = 248 \text{ mm}$$

NOMINAL TEMPERATURE OF CONCRETE (Step 10)

Estimate the nominal temperature of the compressive stress block by averaging the temperature at mid-width and at the side (= $750^{\circ}C$) of stress block. Temperature at 150 mm (mid-width) from fire exposed face is $300^{\circ}C$ from Figure 8e.

Nominal temperature = $\frac{(750+300)}{2} = 525^{\circ}C$

STRENGTH OF CONCRETE (Step 11)

From Figure 11, $f'_{c\theta} = 0.70 \times f'_{c} = 0.70 \times 25 = 17.5 \text{ MPa}$

AVAILABLE MOMENT CAPACITY (Steps 13 and 14)

$$a_{\theta} = \frac{3942 \times 177}{0.85 \times 17.5 \times 248} = 189 \text{ mm}$$

Positive moment capacity after 4 hours $M_{\theta}^{+}=A_{s}F_{y\theta}(d_{\theta}-\frac{a_{\theta}}{2})$

$$M_{\theta}^{+} = 3942 \times 177(591 - \frac{189}{2}) \times 10^{-6} = 346 \text{ kNm}$$

SIMPLY SUPPORTED ELEMENT (Steps 15 and 16)

$$M_a = \frac{wl^2}{8}$$
 with $w = D + 0.6L = 6 + 0.6 \times 16 = 15.6$ kN/m

$$M_a = \frac{15.6 \times 12^2}{8} = 281 \text{ kNm}$$

Since $M_{\theta}^{+} \geq M_{\alpha}$ (346 \geq 281) the beam has a fire resistance of at least four hours.

Note: The use of the rational design procedure for this beam has avoided the necessity for any re-design. In order for 4 hours fire resistance to be achieved, following MP9, it would be necessary to re-arrange the steel within the existing cross-section or increase the dimensions of the beam such that 65 mm minimum cover is provided. The rational design demonstrates that neither of these options are necessary.

WORKED EXAMPLE 2

<u>Problem</u>. A prestressed dense concrete double tee floor system has been designed for ambient temperature loads and is shown in Figure 15. The prestressed double tees have dimensions 400 mm deep overall by 2400 mm wide and are required to span two bays in the building. Each span is 10 m with the double tee simply supported

over each span before the topping is cast. An 80 mm thick dense aggregate concrete topping is poured over the entire floor.

The uniform dead load (including self-weight) is 4 kN/m^2 and the uniform live load is 2 kN/m^2 . Prestressing steel comprises three 12.7 mm diameter strands in each rib (127 mm² each). Assume $F_{pu} = 1725 \text{ MPa}$ and $f'_c = 30 \text{ MPa}$.

Use the rational design procedure to provide 2 hours fire resistance.

IS RATIONAL DESIGN APPROPRIATE (Steps 1 to 3)

The minimum average cover provided to the tensile steel is 74 mm (see below), and the minimum thickness of the floor between ribs is 130 mm. The double-tee achieves 2 hours fire resistance according to MP9.

Even though the required fire resistance is achieved according to MP9 a rational design follows.

CHECK INSULATION (Step 4)

According to MP9, two hours fire resistance (Insulation) is provided by a 120 mm thick slab.

NOMINAL TEMPERATURE OF TENSILE STEEL (Steps 5 and 6)

Calculate effective cover.
$$C_e \text{ (mm)} = \frac{127(70+73+80)}{3\times127} = 74 \text{ mm}$$

The centroidal axis of tensile steel lies 167 mm above the bottom of the rib. i.e.,

$$\frac{100 + 150 + 250}{3} = 167 \text{ mm}$$

The width of the rib at 167 mm above the bottom is 148 mm (say 150 mm).

Nominal temperature of the strands, $T_s = 500^{\circ}C$ from Figure 8c (b = 150 mm and 75 mm from fire-exposed surface).

STRENGTH OF STEEL (Step 7)

Calculate ultimate tensile strength of steel at elevated temperature using Figure 9.

$$F_{pu\theta} = 0.36 \times F_{pu} = 0.36 \times 1725 = 621 \text{ MPa}$$

REDUCED DIMENSIONS (Steps 8 and 9)

Determine reduced dimensions d_{θ} and b_{θ} . The compression block lies within the top flange.

$$d_{\theta} = d = 480 - 167 = 313 \text{ mm}$$

 $b_{\theta} = b = 2400 \text{ mm}$

NOMINAL TEMPERATURE OF CONCRETE (Step 10)

Estimate nominal temperature of compressive stress block. Assume $T_{\rm c} < 350^{\rm o}C$ as protected from the fire.

STRENGTH OF CONCRETE (Step 11)

From Figure 7, $f'_{c\theta} = f'_c = 30 \text{ MPa}$

AVAILABLE MOMENT CAPACITY (Steps 12 to 14)

$$F_{ps\theta} = F_{pu\theta} \left(1 - \frac{A_s F_{pu\theta}}{2b_{\theta} d_{\theta} f'_{c\theta}} \right) = 621 \left(1 - \frac{6 \times 127 \times 621}{2 \times 2400 \times 313 \times 30} \right) = 614 \text{ MPa}$$

Reduced depth of compressive stress block

$$a_{\theta} = \frac{A_s F_{ps\theta}}{0.85 f_{c\theta}' b_{\theta}}$$

$$a_{\theta} = \frac{6 \times 127 \times 614}{0.85 \times 30 \times 2400} = 7.6 \text{ mm}$$

Positive moment capacity after 2 hours $M_{\theta}^{+} = A_{s}F_{ps\theta}(d_{\theta} - \frac{a_{\theta}}{2})$

$$M_{\theta}^{+} = 6 \times 127 \times 614 \times (313 - 3.8) = 145 \text{ kNm}$$

SIMPLY SUPPORTED (Steps 15 and 16)

$$M_a = \frac{wl^2}{8}$$
 with $w = D + 0.6L$
 $w = 4.0 + 0.6 \times 2.0 = 5.2 \text{ kN/m}^2$
 $= 5.2 \times 2.4 = 12.5 \text{ kN/m}$

$$M_a = \frac{12.5 \times 10^2}{8} = 156 \text{ kNm}$$

Insufficient available moment capacity exists after two hours assuming simple support, however, continuity over supports is available by providing reinforcement in the concrete topping to carry the tension forces generated by negative moment over the support and by providing mortar packing between the ends of the ribs to carry the compression forces. Note that so far, the rational design has indicated a lesser fire resistance than does MP9 (assuming simple supports).

MOMENT CONTINUITY (Steps 17 to 21)

Negative moment capacity required over central support for an end bay:

$$M_{\theta}^{-} = \frac{wl^2}{2} - wl^2 \sqrt{\frac{2M_{\theta}^{+}}{wl^2}} = \frac{12.5 \times 10^2}{2} - 12.5 \times 10^2 \sqrt{\frac{2 \times 145}{12.5 \times 10^2}} = 23 \text{ kNm}$$

Provide reinforcing bars (380 MPa, HD10's) in concrete topping, 40 mm below unexposed surface (= 90 mm from fire exposed surface).

Temperature of steel = $230^{\circ}C$ after 2 hours from Figure 7a. Strength of steel at elevated temperature (from Figure 9), $F_{y\theta} = F_y = 380$ MPa.

Ignore surface layer of 26 mm around rib, where temperature > $750^{\circ}C$ from Figure 8c (b = 150).

Reduced dimensions are:
$$d_{\theta} = d = 480 - 40 - 26 = 414 \text{ mm}$$

 $b_{\theta} = 135 \text{ (estimated)} - 26 - 26 = 83 \text{ mm} \times \text{ (two ribs)}$
 $= 166 \text{ mm}$

By trial and error, assume $a_{\theta} \approx 88$ mm ; $d_{\theta} - \frac{a_{\theta}}{2} \approx 414 - 44 = 370$ mm

Temperature in compressive stress block, $T_c \approx \frac{(750+540)}{2} = 645^{\circ}C$ (70 mm from coolest part of compressive stress block to the exposed side of the rib gives 540° C from Figure 8c (interpolated b = 140 mm).

Compressive strength of concrete, $f_{c\theta}' = 0.48 \times f_c' = 0.48 \times 30 = 14.4$ MPa

Minimum area of steel required,
$$A_s = \frac{M_{\theta}^{-}}{F_{y\theta}(d_{\theta} - \frac{a_2}{2})} = \frac{23 \times 10^6}{380 \times 370} = 164 \text{ mm}^2$$

But check minimum reinforcing required by NZS 3101:1982 (cl 5.3.32.1).

 $0.0018 \times \text{area of concrete} = 0.0018 \times 80 \times 1000 = 144 \text{ mm}^2 \text{ per metre width}$. Maximum spacing is 450 mm or $5 \times \text{ topping thickness} (= 400 \text{ mm}) \text{ whichever is less (i.e. } 400 \text{ mm})$.

Use HD10 reinforcing bars at 400 mm centres $(A_s = 471 \text{ mm}^2)$.

Depth of compressive stress block = $\frac{471 \times 380}{0.85 \times 14.4 \times 166}$ = 88 mm (as assumed).

Check
$$\frac{A_s F_{y\theta}}{0.85 b_{\theta} d_{\theta} f_{c\theta}^t} < 0.26$$

$$\frac{471~\text{mm}^2 \times 380~\text{MPa}}{0.85 \times 166~\text{mm} \times 414~\text{mm} \times 14.4~\text{MPa}} = 0.21 < 0.26~\text{(OK)}$$

Negative moment capacity, $M_{\theta}^-=471\times380\times(414-44)\times10^{-6}=66.2~\mathrm{kNm}$

Position of inflection points, $X_0 = l - \frac{2M_{\theta}^-}{wl}$ where: w = dead load = 9.6 kN/m

$$X_0 = 10 \text{ m} - \frac{2 \times 66.2 \text{ kNm}}{9.6 \text{ kN/m} \times 10 \text{ m}} = 10 \text{ m} - 1.38 \text{ m} = 8.62 \text{ m}$$

The floor system therefore has at least 2 hours fire resistance by utilising moment continuity at the supports.

Instead of using moment continuity, restraint against thermal expansion could be provided. The example follows:

RESTRAINT AGAINST THERMAL EXPANSION

Available moment capacity, $M_{\theta}^{+} = 145$ kNm as previously calculated.

Maximum applied bending moment, $M_a = 156$ kNm as previously calculated.

Estimate deflection, Δ_1 , at midspan assuming minimal restraint. From Figure 13, $\Delta_0 = 32$ mm. $y_{b1} = 357$ mm.

$$\Delta_1 = \frac{\ell^2 \Delta_0}{89000 y_{b1}} = \frac{10000^2 \times 32}{89000 \times 357} = 101 \text{ mm}$$

Assume line of thrust acts at $0.1 \times$ depth of member i.e., at 48 mm from the bottom of the ribs. $d_T = 480 - 48 = 432$ mm.

$$F_{ps} = F_{pu} \left(1 - \frac{A_s F_{pu}}{2b df_c'} \right) = 1725 \left(1 - \frac{6 \times 127 \times 1725}{2 \times 2400 \times 313 \times 30} \right) = 1675 \text{ MPa}$$

$$a = \frac{A_s F_{ps}}{0.85 f_s' b} = \frac{6 \times 127 \times 1675}{0.85 \times 30 \times 2400} = 21 \text{ mm}$$

$$M^+ = A_s F_{ps}(d - 0.5a) = 6 \times 127 \times 1675(480 - 0.5 \times 21) \times 10^{-6} = 599 \text{ kNm}$$

$$a_{\theta} \approx \frac{M_a}{M^+} \times a = \frac{156}{599} \times 21 = 5.5 \text{ mm}$$

$$T = \frac{M_a - M_\theta^+}{d_T - \frac{a_\theta}{2} - \Delta_1} = \frac{156 - 145}{432 - 2.7 - 101} \times 10^3 = 33.5 \text{ kN}$$

 $A_1 = 434500 \text{ mm}^2 \text{ and } E_1 = 26 \text{ GPa}.$

Thrust parameter, $\frac{T_1}{A_1 E_1} = \frac{33500}{434500 \times 26000} = 3.0 \times 10^{-6}$

$$z = A_1/S_1 \approx \frac{434500}{2400+4\times350} = 114 \text{ mm}$$

From Figure 12, strain parameter, $\frac{\Delta \ell}{\ell} = 0.008$

$$\Delta \ell = 0.008 \times 10000 = 80 \text{ mm}.$$

Provided the surrounding structure is able to withstand a thrust of 33.5 kN (per double tee) with a displacement no greater than 80 mm (40 mm each end), two hours fire resistance will be achieved. This capability of the structure must be independently assessed.



BRANZ MISSION

To promote better building through the application of acquired knowledge, technology and expertise.

HEAD OFFICE AND RESEARCH CENTRE

Moonshine Road, Judgeford Postal Address - Private Bag 50908, Porirua Telephone - (04) 235-7600, FAX - (04) 235-6070

REGIONAL ADVISORY OFFICES

AUCKLAND

Telephone - (09) 524-7018 FAX - (09) 524-7069 290 Great South Road PO Box 17-214 Greenlane

WELLINGTON

Telephone - (04) 235-7600 FAX - (04) 235-6070 Moonshine Road, Judgeford

CHRISTCHURCH

Telephone - (03) 663-435 FAX - (03) 668-552 GRE Building 79-83 Hereford Street PO Box 496