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PERFORMANCE OF CONCRETE FLOORS EXPOSED TO REAL FIRES

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SUMMARY

Methods are currently available for the design of concrete floors and beams to resist collapse when exposed to fire. However, such methods are based on results and temperatures obtained from standard fire resistance tests. There is a need to develop appropriate thermal fire design data reflecting temperature distributions within concrete floors and beams when exposed to non-standard or "real" fires.

This paper presents results obtained by using finite element numerical methods giving estimated temperatures within concrete elements as a function of concrete depth (cover), fire load density and compartment opening factor. An indication of how this data may be used in conjunction with current design methods is also given.

INTRODUCTION

Methods [1,2,3] previously developed for the structural design of reinforced and pre-stressed concrete floors require knowledge of the likely internal temperatures reached within the concrete element in order to calculate its load bearing capacity. The basic philosophy is one which applies conventional design theory, but uses reduced strengths for concrete and steel at elevated temperatures.

There is currently ample information available on the temperature distributions within concrete elements exposed to standard fire resistance tests, but if the performance is to be assessed for non-standard or "real" fires then the corresponding temperature distributions for these design fire scenarios are also required. This paper sets out to present this data in a form useful for fire protection engineers.

FIRE SEVERITY OF REAL FIRES

The time-temperature histories of post-flashover fires have been presented by several researchers [4-7]. In general, these methods use conservation of energy principles to solve for the fire gas temperature in a compartment. While energy balance methods are more scientific and to be preferred for calculating post-flashover compartment temperatures, for simplicity in this study, it was decided to use a simpler analytical expression. The expression is given by Lie[8,9] and is an approximation of time-temperature curves developed using the energy balance described by Kawagoe[7]. Lie[8] suggests "the use of these curves as basic exposure curves for fire-resistive design will reasonably assure that building components will not be exposed to temperatures higher than those represented by the curves during the life of the building." The expression follows.

$$T = 250(10F)^{0.1/F^{0.3}} e^{-F^2 t} \left[3(1-e^{-0.6t}) - (1-e^{-3t}) + 4(1-e^{-12t})\right] + C\left(\frac{600}{F}\right)^{0.5} \quad (1)$$

Where
T = the fire temperature (°C)
t = time (hr)
F = opening factor ($m^{1/2}$)
C = a constant taking into account the influence of the properties of the boundary materials on the temperature.
= 0 for heavy materials ($\rho \geq 1600$ kg/m³)
= 1 for light materials ($\rho < 1600$ kg/m³)

The above expression is only valid for

$$t \leq \frac{0.08}{F} + 1. \ \textit{If } t > \frac{0.08}{F} + 1 \text{ then } t = \frac{0.08}{F} + 1. \quad (2)$$

This means that after a certain time, the temperature remains constant at the peak value until all the fire load has been consumed, and the fire starts to decay.

Similarly the above expression is only valid for

$$0.01 \leq F \leq 0.15. \ \textit{If } F > 0.15 \text{ then } F = 0.15.$$

The expression is dependent on the opening factor, F, which is determined from the openings in the fire compartment of interest as follows.

$$F = \frac{A_w \sqrt{H}}{A_t} \quad (3)$$

Where
A_w = area of openings in the enclosure (m²)
A_t = area of internal enclosure boundary surfaces including openings and floor (m²)
H = height of openings (m)

The duration of the heating period of the fire can be expressed as:

$$D = \frac{QA_t}{R} = \frac{Q}{330F} \quad \textbf{(4)}$$

Where
D = fire duration in hr
R = rate of burning of wood = $330A_w\sqrt{H}$ (kg/hr) from Reference 4
Q = fire load per unit area of enclosure boundary surfaces including openings and floor (kg/m²)

Alternatively the duration of heating can be given in terms of fire load per unit floor area which is more commonly used for specifying fire loads in buildings.

$$D = \frac{Q_f \times \dfrac{A_f}{A_t}}{330F} \quad \textbf{(5)}$$

Where
Q_f = fire load (wood equivalent) per unit floor area (kg/m²)
A_f = floor area of the enclosure (m²)

The gas temperature during the decay phase may be represented by the following expression also from Lie[8,9].

$$T = -600\left(\frac{t}{D} - 1\right) + T_d \quad \textbf{(6)}$$

Where T_d = temperature at time D, when decay commences (°C).

While this straight-line representation is not a very realistic decay function, it will be adequate given it is not critical to the peak temperatures reached in exposed structural members.

The time-temperature curves used in this study are given in Figures 1, 2, and 3. They have been generated using Lie's expression and therefore assume certain properties and dimensions of the compartment.

These can be found in Reference 8 and are summarized in Table 1. The assumed fire

Table 1: Enclosure Parameters Assumed

Compartment volume	1000 m^3
Window height	1.8 m
Thermal conductivity of boundary material (heavy)	1.16 W/mK
Volumetric specific heat of boundary material (heavy)	2150 kJ/m^3K
Total inner surface area including openings	1000 m^2
Emissivity for radiation transfer between hot gases and boundary surfaces	0.7
Coefficient of heat transfer (both faces)	23 W/m^2K
Thickness of boundary material	0.15m
Initial Temperature	20°C
Heat released in the enclosure by burning 1 kg of wood	10.77 MJ/kg

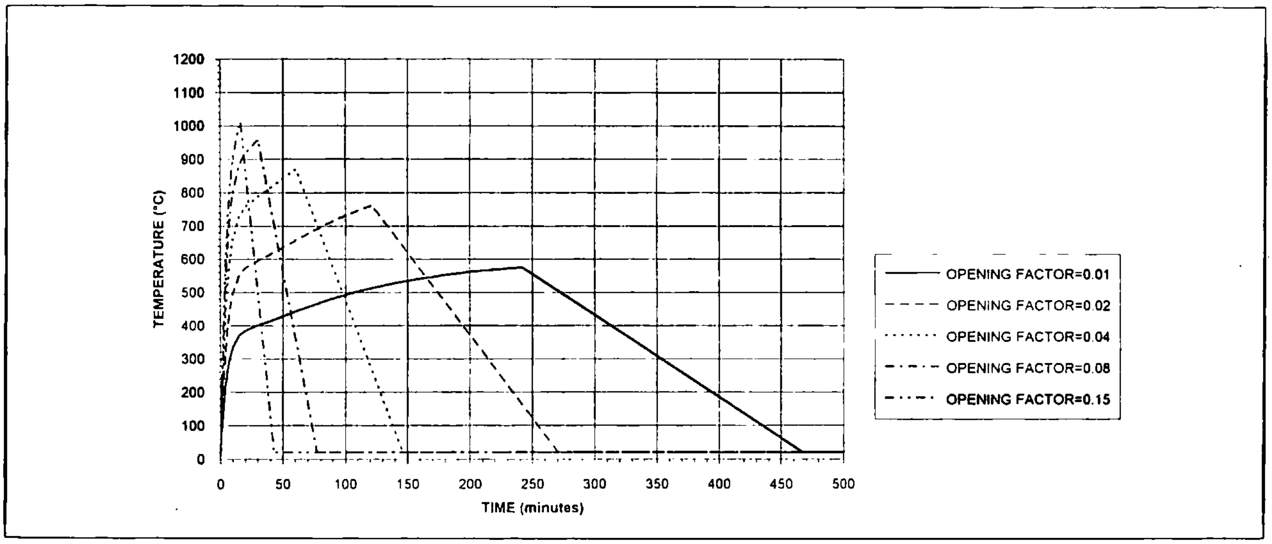


Figure 1. Time Temperature Curve of the Fire for Different Opening Factors (Fire Load of 37 kg/m² Wood Equivalent).

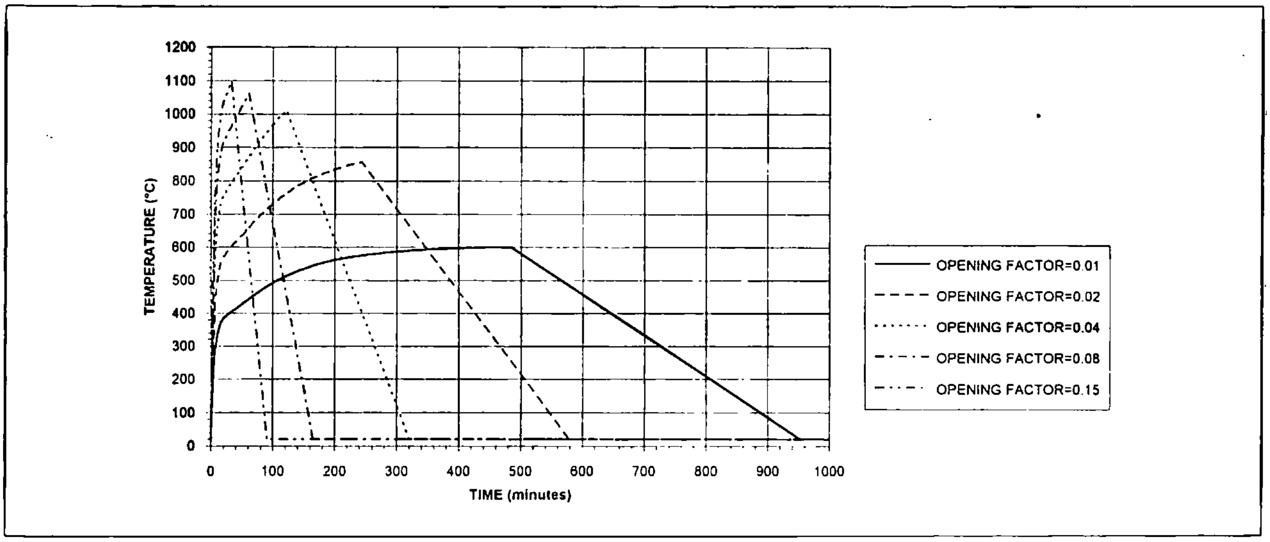


Figure 2 . Time Temperature Curve of the Fire for Different Opening Factors (Fire Load of 74 kg/m² Wood Equivalent).

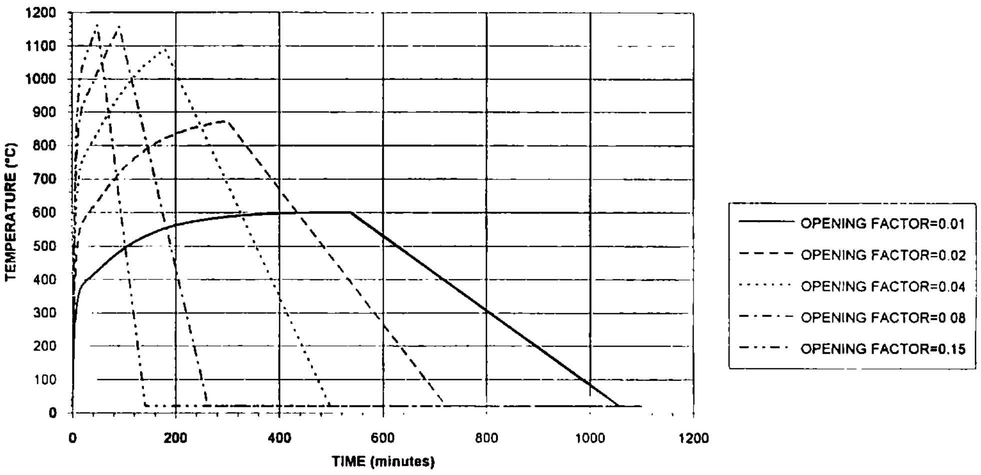


Figure 3 . Time Temperature Curve of the Fire for Different Opening Factors (Fire Load of 111 kg/m² Wood Equivalent).

loads per unit floor area are 37, 74 and 111 (kg/m²) (wood equivalent) and the opening factors used range from 0.01 to 0.15.

Numerical Modeling of Thermal Response

A commercially available general purpose finite element computer program was used to investigate the likely response of concrete slabs and beams to the time-temperature curves generated by Lie's expression. The program used was NISA[10] which is capable of taking the following factors into account:

- time dependent fire/furnace gas temperatures
- material properties which are time- and/or temperature-dependent
- 1, 2, or 3 dimensional analysis
- emissivity and convective heat transfer coefficients which may vary with temperature
- phase changes *e.g.* evaporation of moisture
- automatic finite element meshing routines

The theoretical basis of NISA will not be discussed here. The program was used in a previous study[11] to assess how useful it was for analyzing fire-resisting performance of buildings and to assess the accuracy of results obtained from the program by comparison with data recorded in standard fire resistance tests. The study concluded that NISA could be used for reasonably accurate analytical studies of the thermal response of building components exposed to fire, and for this reason was used in this study.

Comparison with Standard Fire Resistance Testing

Figure 4 compares the predicted results using the NISA program[11] with measured temperatures at nominated depths within a vertical 175 mm thick alluvial quartz concrete slab. The fire exposure followed the ISO 834 time-temperature curve. Experimental data were obtained from an earlier study[12] into the fire resistance of New Zealand concretes which identified alluvial quartz as the least insulating concrete type currently used in New Zealand. For the comparison shown, as the depth increases the agreement reduces. Part of the reason for this is the effect of moisture in the concrete. While NISA can account for the evaporation of water, it does not model mass transport of water as it is driven through the slab.

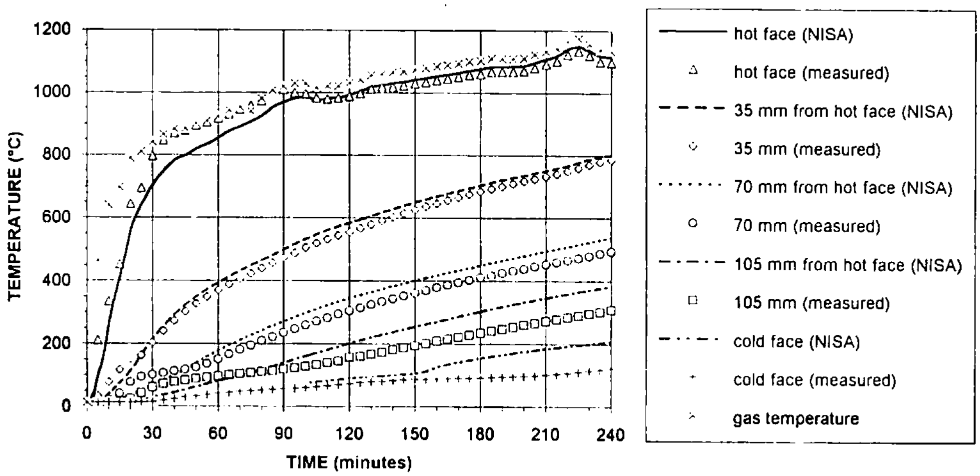


Figure 4 . Comparison Between Measured and Predicted Temperatures in Concrete Slab Using NISA.

At temperatures of importance in fire engineering (above 400°C, say) the results achieved for concrete at its in-service moisture content will generally be conservative. The temperatures predicted by the NISA program will therefore tend to overestimate the actual temperatures reached within the concrete members.

Predicted Temperature Response to Real Fires

The NISA program was used to estimate the peak temperature distribution at nominated depths within a 175 mm thick alluvial quartz concrete slab and within beam sections 100, 200 and 300 mm wide exposed to fire from beneath on three sides. The peak temperature at particular depths within a concrete slab will not vary greatly with overall slab thickness, provided the points of interest are not located close to the unexposed surface, at which point convective heat losses become important. Therefore, it is considered valid to use internal temperature data from a 175 mm thick concrete slab for slabs of other thicknesses, provided the depth of interest is not greater than about 150 mm.

Input to the NISA program included the following assumptions: initial temperature of 20°C, moisture content of the concrete 3% by mass, concrete density 2300 kg/m^3, concrete emissivity 0.9, and convective heat transfer coefficients of 7 and 10 W/m^{2}K for non fire-exposed and fire-exposed faces respectively. Thermal properties of the concrete were assumed to vary with temperature in accordance with Table 2. Thermal conductivity and specific heat data for a quartz aggregate were taken from Lie and Williams-Leir[13]. The analysis was treated as a one-dimensional problem for slabs and two-dimensional for beams.

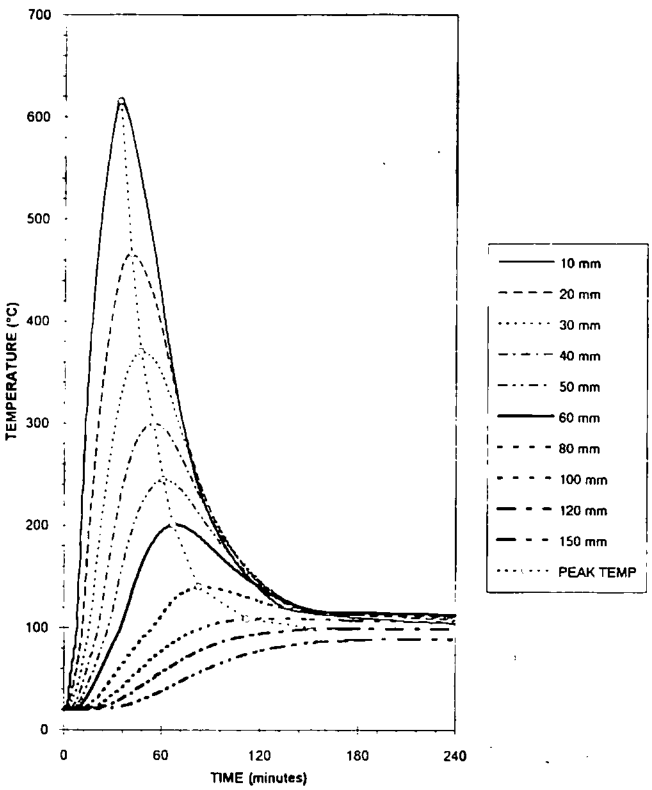
Figure 5 shows an example of how the temperature changes at different depths in the concrete slab for a fire load density of 37 kg/m^2 and opening factor of 0.08. The peak temperature reached at the depths of interest can be determined from this and similar graphs, and this information has been used to create Figures 6, 7 and 8 which show the predicted peak temperature reached at various depths in a quartz aggregate concrete slab with 3% moisture content exposed to the "real" time-temperature curves discussed earlier.

Similarly, NISA was used to predict peak temperature distributions in beams ranging from 100 mm to 300 mm wide and exposed to fire from the underside (on three sides). The results of this analysis are presented in Table 3. The temperatures

Table 2: Thermal Properties of Concrete

Temperature (°C)	Thermal Conductivity (W/mK)	Specific Heat (J/kgK)	Enthalpy (MJ/m³) - based on 3% moisture content by mass
0	2.57	765	0
25	2.57	765	61
50	2.49	809	123
75	2.42	852	187
100	2.36	883	253
115	2.34	935	649
125	2.31	987	671
150	2.25	1061	730
175	2.19	1096	792
200	2.14	1113	855
225	2.09	1126	919
250	2.05	1139	985
275	2.02	1148	1050
300	1.99	1157	1117
325	1.96	1165	1183
350	1.93	1174	1251
375	1.90	1183	1318
400	1.86	1196	1387
425	1.81	1274	1458
450	1.76	1478	1537
475	1.70	1683	1628
500	1.64	1796	1728
525	1.57	1683	1828
550	1.52	1509	1919
575	1.49	1400	2003
600	1.45	1287	2080
625	1.43	1191	2152
650	1.41	1178	2220
675	1.38	1174	2287
700	1.36	1183	2355
750	1.31	1170	2490
800	1.28	1135	2623
850	1.28	1117	2752
900	1.28	1122	2881
1000	1.28	1139	3141
1100	1.31	1148	3404
1200	1.34	1161	3670
1275	1.36	1170	3871

Figure 5. Temperatures Reached At Different Depths From The Exposed Face For Fire Load of 37 kg/m² Wood Equivalent and Opening Factor = 0.08



correspond to those predicted on the beam vertical center line with cover measured from the bottom face as shown in Figure 9.

Figures 6, 7 and 8 show the expected decrease in temperature with increase in distance from the fire-exposed face. They also show, for a given fire load density, that at distances close to the fire-exposed face, the peak fire gas temperatures have a dominant influence on the peak temperature at that point, while at distances far from the fire-exposed face the fire duration has the dominant influence.

Application of Thermal Response Data

The design data presented in Figures 6, 7 and 8 for concrete slabs and in Table 3 for beams may be used with fire design methods[1,2,3] to assess the structural stability of floor slabs and beams. The methods compare the maximum applied moments within the floor or beam with the available moment capacity. The available moment capacity is determined taking into account

Table 3: Temperature Distributions in Concrete Beams (All temperatures in Degrees Celsius)

BEAM WIDTH 100 mm
Fire Load 37 kg/m²

OPENING FACTOR	COVER (mm) 10	20	30	40	50
0.15	799	610	509	470	451
0.08	856	733	650	609	587
0.04	824	770	728	704	690
0.02	745	728	714	705	700
0.01	568	563	558	555	553

BEAM WIDTH 100 mm
Fire Load 74 kg/m²

OPENING FACTOR	COVER (mm) 10	20	30	40	50
0.15	1008	883	792	747	724
0.08	1020	962	914	888	873
0.04	993	973	954	944	937
0.02	851	847	843	840	838
0.01	599	599	598	598	598

BEAM WIDTH 100 mm
Fire Load 111 kg/m²

OPENING FACTOR	COVER (mm) 10	20	30	40	50
0.15	1109	1030	966	931	912
0.08	1133	1099	1070	1053	1044
0.04	1080	1070	1060	1054	1051
0.02	871	868	866	864	863
0.01	601	600	600	600	600

BEAM WIDTH 200 mm
Fire Load 37 kg/m²

OPENING FACTOR	COVER (mm) 10	20	30	40	50	60	70	80	90	100
0.15	799	602	468	376	303	245	204	181	167	157
0.08	851	709	583	489	421	371	335	312	297	286
0.04	810	728	643	569	512	476	453	437	426	416
0.02	728	685	637	592	556	530	513	501	494	488
0.01	559	543	526	510	496	485	478	472	468	464

BEAM WIDTH 200 mm
Fire Load 74 kg/m²

OPENING FACTOR	COVER (mm) 10	20	30	40	50	60	70	80	90	100
0.15	1001	853	707	592	507	455	422	400	384	372
0.08	1006	915	812	718	644	591	557	536	522	512
0.04	980	932	874	816	767	729	703	685	675	662
0.02	845	827	804	781	759	741	727	717	710	705
0.01	597	594	589	585	580	576	572	569	567	565

BEAM WIDTH 200 mm
Fire Load 111 kg/m²

OPENING FACTOR	COVER (mm) 10	20	30	40	50	60	70	80	90	100
0.15	1097	985	860	745	657	596	556	532	517	507
0.08	1119	1051	970	891	825	774	740	717	701	689
0.04	1070	1039	1000	960	923	894	873	858	848	839
0.02	866	854	839	822	806	792	782	774	768	764
0.01	599	596	593	590	587	583	580	578	576	575

BEAM WIDTH 300 mm
Fire Load 37 kg/m²

OPENING FACTOR	COVER (mm) 10	20	30	40	50	60	80	100	120	150
0.15	799	602	468	376	302	240	192	157	131	104
0.08	851	709	583	488	416	355	257	190	154	129
0.04	810	728	641	562	496	443	357	295	258	231
0.02	727	680	627	574	526	486	423	381	354	331
0.01	554	534	511	488	465	444	408	382	365	350

BEAM WIDTH 300 mm
Fire Load 74 kg/m²

OPENING FACTOR	COVER (mm) 10	20	30	40	50	60	80	100	120	150
0.15	1001	853	706	589	500	432	328	253	210	183
0.08	1004	914	809	708	621	549	448	386	349	319
0.04	978	926	861	790	723	663	571	516	489	468
0.02	842	820	779	753	715	679	617	575	552	533
0.01	594	587	578	568	557	546	527	512	503	495

BEAM WIDTH 300 mm
Fire Load 111 kg/m²

OPENING FACTOR	COVER (mm) 10	20	30	40	50	60	80	100	120	150
0.15	1097	985	858	738	638	557	444	376	335	304
0.08	1118	1048	969	870	783	706	590	523	490	465
0.04	1068	1031	981	925	868	814	726	669	635	607
0.02	862	844	820	791	761	731	679	643	620	601
0.01	596	591	584	575	566	557	539	526	517	510

the temperature at the position of the steel reinforcement and its strength at that temperature. If the available moment capacity is in excess of the maximum applied moment, then it is expected that structural stability will have been maintained during and following the fire.

A user-friendly computer program[14] has been written in Microsoft® Visual Basic™ by the author to run under Microsoft® Windows 3.1™ following the design method described in Reference 3, and incorporating the additional design data presented in Figures 6, 7 and 8 and Table 3 such that the designer is able to select standard or non standard design fire scenarios.

Example—Design of Reinforced Concrete Floor

Problem

A simply-supported dense concrete floor slab spanning 5.5 m is required as an upper floor in a low-rise industrial building. The floor is 100 mm deep carrying a uniform dead load of 2.3 kN/m and uniform live load of 2.5 kN/m. Steel reinforcing bars, 16 mm diameter, are positioned 30 mm from the underside of the slab at 200 mm centres. The yield strength of the steel bars, F_y, is 380 MPa and the compressivestrength of the concrete, f'_c is 30 MPa. A live load reduction factor of 0.4 is to be applied. A design fire load of 74 kg/m² (wood equivalent) and an opening factor of 0.15 will be used.

Solution

The effective cover to reinforcement is 30 mm and from Figure 7 the peak temperature expected at this distance from the exposed face within the concrete slab is 446°C.

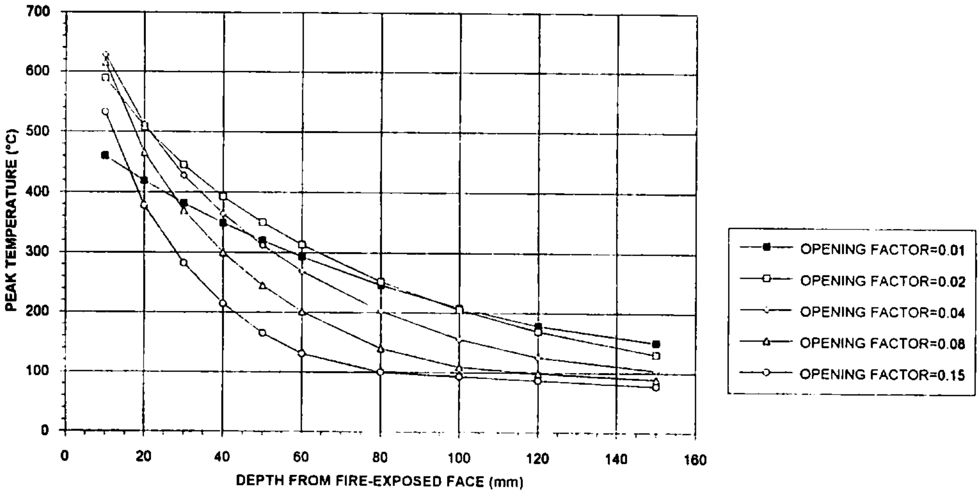


Figure 6 . Peak Temperatures Reached In Concrete Slab (Fire Load 37 kg/m²).

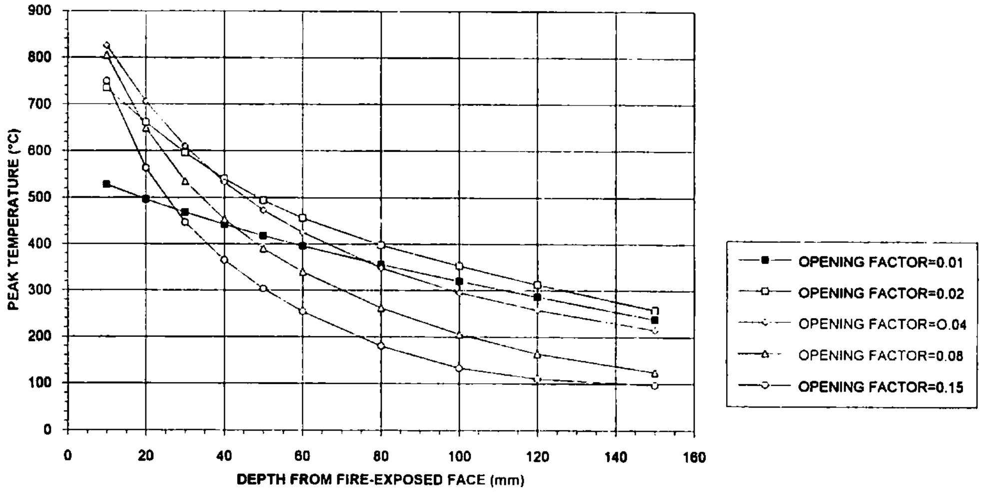


Figure 7 . Peak Temperatures Reached In Concrete Slab (Fire Load 74 kg/m²).

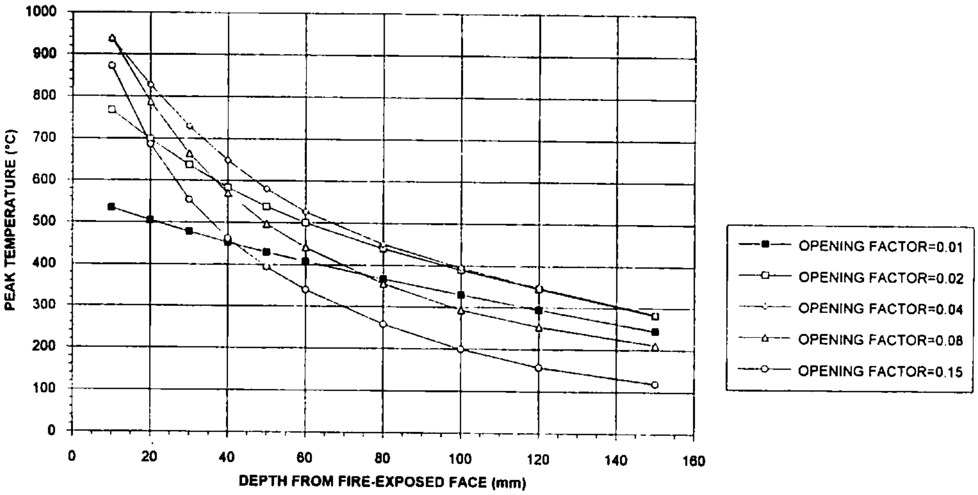


Figure 8 . Peak Temperatures Reached In Concrete Slab (Fire Load 111 kg/m²).

The reduced strength of the reinforcing steel[15] may be given by:

$$\frac{F_y(T)}{F_y(20°C)} = \begin{cases} 1.0, T \leq 250°C \\ 1.53 - \frac{T}{470}, T > 250°C \end{cases} \qquad (7)$$

where $F_y(T)$ = yield strength of steel at temperature T.

Substituting 446°C for T in the expression gives a strength reduction factor of 0.57. Multiplying by the original yield strength of 380 MPa gives 221 MPa as the elevated temperature yield strength of the reinforcing steel. The centroidal axis of the steel is 30 mm above the tension (bottom) face. The effective depth of the slab is the distance between the centroidal axis of the steel and the compressive (top) face of the slab and in this case is 70 mm.

The strength of dense concrete can be given by[16]:

$$\frac{f'_c(T)}{f'_c(20°C)} = \begin{cases} 1.0, T \leq 350°C \\ 1.65 - \frac{0.8T}{440}, T > 350°C \end{cases} \qquad (8)$$

where $f'_c(T)$ = compressive strength of the concrete at temperature T.

Since the concrete in compression (above the neutral axis) is not directly exposed to fire, the nominal temperature of the concrete can be assumed to be less than 350°C and therefore no reduction in compressive strength need be made.

Available Moment Capacity

The available moment capacity per meter width, M^+_θ, is calculated as follows using steel area, A_s = 1005 mm² (5 steel reinforcing bars per meter width); reduced effective depth of slab, d_θ = 70 mm; width of slab, b_θ = 1000 mm; elevated temperature yield strength of steel, $F_{y\theta}$ = 221 MPa (as calculated above); and elevated temperature compressive strength of concrete, $f'_{c\theta}$ = 30 MPa:

$$M^+_\theta = A_s F_{y\theta}\left(d_\theta - \frac{A_s F_{y\theta}}{2 \times 0.85 f'_{c\theta} b_\theta}\right) \qquad (9)$$

$$= 1005 \times 221\left(70 - \frac{1005 \times 221}{2 \times 0.85 \times 30 \times 1000}\right)$$

= 14.6 x 10^6 N.mm per meter width
= 14.6 kN.m per meter width

Applied Moment

The applied moment per meter width M_a, for a simply-supported slab with a uniformly applied distributed load w, and span l, is calculated as follows:

$$M_a = \frac{wl^2}{8} = \frac{(2.3 + 0.4 \times 2.5) \times 5.5^2}{8} \qquad (10)$$

=12.5 kN.m per metre width

Since the applied moment is less than the available moment capacity, structural stability has been maintained for the duration of the design fire specified. In addition, the maximum temperature likely to be reached on the top surface of the slab can be conservatively estimated at 90°C from Figure 7 using a depth of 100 mm. The estimate is very conservative since no account is made of convective heat losses from the top surface of the slab.

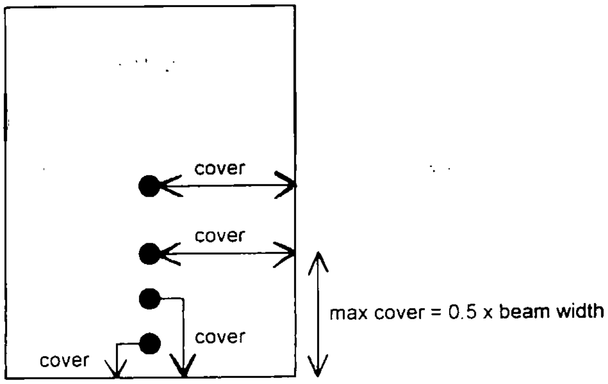


Figure 9. Measurement of Beam Cover.

The calculation is repeated for other opening factors and a summary of the results given in Table 4. It compares the available moment capacity of the slab for a range of design fire scenarios and shows that for fire compartments with favorable ventilation conditions, the impact of the fire on

Table 4: Results of Analysis of Concrete Slab Floor

Design Fire Load 74 kg/m²	Available Moment Capacity (kNm)	Maximum Applied Moment (kNm)	Stability Achieved ?
Opening Factor - 0.01	13.5	12.5	yes
Opening Factor - 0.02	6.8	12.5	no
Opening Factor - 0.04	6.1	12.5	no
Opening Factor - 0.08	10.0	12.5	no
Opening Factor - 0.15	14.6	12.5	yes
ISO 834 - 60 minutes	15.4	12.5	yes
ISO 834 - 90 minutes	10.1	12.5	no

the structure may be reduced. In this case the design is unsatisfactory when the opening factor ranges between 0.02 and 0.08. For comparison, Table 4 also includes exposure to a standard ISO 834 time-temperature curve for a period of 60 and 90 minutes. The designer has a range of options available to ensure stability is achieved, including changing the location and amount of steel reinforcement, changing the thickness of the slab, introducing moment continuity over the supports (if possible), etc.

Conclusion

Thermal fire design data for concrete floor slabs and beams has been generated from finite element analysis. The data is for use with fire engineering design methods for determining the structural stability of concrete floors during fire exposure. A computer program is available to make the design method and thermal fire design data easily used by fire protection engineers.

Acknowledgments

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NOMENCLATURE

A_f = Floor area of enclosure (m^2)

A_s = Area of steel reinforcement (mm^2)

A_t = Area of internal boundary surfaces (m^2)

A_w = Area of openings in enclosure (m^2)

b_θ = Width of slab in the compression zone at elevated temperature (mm)

C = Constant

d_θ = Effective depth of slab at elevated temperature (mm)

D = Fire duration (hr)

f'_c = Compressive strength of concrete at ambient temperature (MPa)

$f'_{c\theta}$ = Compressive strength of concrete at elevated temperature (MPa)

F = Opening factor ($m^{1/2}$)

F_y = Yield strength of steel reinforcement at ambient temperature (MPa)

$F_{y\theta}$ = Yield strength of steel reinforcement at elevated temperature (MPa)

H = Height of opening (m)

l = Span of beam or slab (m)

M_a = Applied bending moment (kNm)

M^+_θ = Available moment capacity (kNm)

Q = Fire load per unit area of enclosure bounding surfaces (kg/m^2)

Q_f = Fire load per unit floor area (kg/m^2)

R = Rate of burning (kg/hr)

T = Temperature (°C)

T_d = Temperature at time D (°C)

t = Time (hr)

w = Load intensity) kN/m)

ρ = Mass density (kg/m^3)