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DESIGN OF REINFORCED CONCRETE SLABS EXPOSED TO NATURAL FIRES

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Abstract. *The use of prescribed solutions for fire design has allowed engineers to simply provide an assembly of elements which meets a building codes required fire resistance without necessarily understanding or evaluating the development of a fire in a compartment, the corresponding temperatures reached and the implications of such temperatures. A process is presented which uses simple thermal and mechanical models to determine the fire resistance of simply supported reinforced solid concrete slabs. The calculated fire resistance is compared to prescribed provisions for slabs exposed to the standard fire and an equivalent fire severity method for slabs exposed natural fires.*

1 INTRODUCTION

The resistance of building members to the effects of fire is an important part of any structural and fire safety design. The New Zealand Building Code states overall objectives which must be met in order to provide a comprehensive fire safety design for buildings [1]. Compliance may be demonstrated in the following ways; design based on recognized prescribed solutions referred to as acceptable solutions, simplified calculation methods for simulating structural members and advanced methods for simulating the behavior of parts or the entire structure. Alternatively these may be based on fire tests or a combination of tests and calculations.

The use of prescribed solutions allows engineers to simply provide an assembly of elements which meets the code required fire resistance without necessarily understanding or evaluating the development of a fire in a compartment, the corresponding temperatures reached and the implications of such temperatures. The important concept of equivalent fire severity used to compare natural fires to the standard fire test is also overlooked by many [1]. Research has shown that compartment fires which have a short-duration, high temperature profiles can significantly exceed the ISO 834 standard fire test temperatures and the limitations of equivalent fire severity methods [2].

In light of low cost computers and software ranging from spreadsheets and specifically designed computer programs for fire and structural analysis it is proposed that the original concepts of fire severity and fire load, are important but are technically obsolete [2]. These concepts are however the basis for many of the fire resistance requirements specified in building codes and any proposed process needs to relate back to the current specification system in order to be useable for the foreseeable future.

To assist in the evaluation of reinforced concrete (RC) slabs exposed to post-flashover fires a design procedure is presented that uses a natural fire model based on the consideration of the physical parameters specific to a particular building compartment. It approximates a realistic fire much better than a nominal fire and includes a cooling phase [3]. The process is intended to provide structural capacity calculations based on fundamental first principles in order for engineers to make design decisions based on design capacity.

2 BACKGROUND

Concrete is used in the construction of many parts of a building from foundations, main structural frames, walls, floors, and ceilings. One of the main advantages of concrete is its relatively high compressive strength. However it has a low tensile strength; being about one-tenth of its compressive strength [4]. Steel reinforcing bars are used to form the fundamental tensile strength of the member to withstand bending loads [4]. These bars are located close to the external faces of the structural concrete members in order to distribute and carry tensile forces generated by bending loads applied to the member effectively.

The increase in steel reinforcing bar temperature caused by exposure of the concrete slab to fire has a significant impact on structural adequacy; this is due to the increased ductility and reduced strength of steel at elevated temperatures [5]. Insulation of the reinforcing steel from heating under fire loads is required in order to prevent the steel reaching a critical temperature which could cause member failure. Due to concretes noncombustible nature and low thermal conductivity it not only acts as part of the structural element but with sufficient cover of the steel bars also provides insulation to the steel reinforcing.

2.1 Fire Resistance

The fundamental concept in designing structures for fire safety using prescribed solutions is to verify the fire resistance of the structure is greater than the fire severity the structure is exposed too [5].

Fire resistance is a measure designed to prevent the spread of fire and structural collapse in fire, it is a measure of the ability of building elements to resist a fire. Fire resistance is most often quantified as the time for which the element can meet certain criteria during exposure to a standard fire [5]. The required fire resistance rating (FRR) in order to achieve structural adequacy is specified in tables giving minimum axis distance, the distance from the exposed face of the slab to the center of the reinforcing rods, in NZS 3101 [6].

Fire severity is a measure of the destructive potential of a fire and is often related to the standard test fire [5]. The damage caused to a structure is largely dependent on the amount of heat absorbed by the structural elements. Out of the three modes of heat transfer; conduction, convection and radiation, radiation heat transfer dominates in post-flashover fires. Radiative heat transfer from the fire and released gases is proportional to the fourth power of the absolute temperature. This means that post-flashover fire severity is largely dependent on the temperatures reached and the duration of the higher temperatures.

The time for which an element may need to meet the fire resistance criteria is referred to as the fire design time. The fire design time may not be clearly stated in regulations and may require engineering judgment based on the consequence of failure, the importance of a structure or the owners requirements. In performance based designs the fire design time is usually complete burnout of the compartment [5].

2.2 The standard fire and equivalent fire severity

Building elements are often evaluated in testing furnaces by exposure to a fire whose severity follows a varying temperature time profile curve known as a standard fire. This is also the case for the prescribed values given in NZS 3101 [7].

The National Institute of Standards and Technology (NIST) developed the concept of equivalent fire severity to define the severity of natural fires with various temperature histories. Two fires with differing temperature histories were considered to have equivalent fire severity when the areas under their temperature time curves were similar. However the impact of fire on a surrounding structure is a function of heat transferred into the structure, the heat transfer to the structure from a short hot fire can be significantly greater than one from a long cool fire even though the areas under the temperature time curve are the same [2]. A number of other equivalent fire severity concepts have been developed but are a crude approximate method of introducing real fire behaviour into fire engineering calculations [5].

It is more accurate to make designs from first principles [5]. The minimum load capacity concept is based on calculating the capacity of the member in this way, where the equivalent fire severity is the time of exposure to the standard fire that would result in the same reduction in load bearing capacity [5]. The proposed design method uses the minimum load capacity concept for predicting fire resistance.

3 PROCESS

The design process shown in Figure 1 is indicative of other processes identified during the literature review, Figure 1 [5]. The flow chart provides a system to evaluate complete burnout of a fire compartment.

The process of calculating the structural fire behaviour has three essential components. The fire model, heat transfer model and the structural model. The calculated load capacity is compared with the load capacity requirements of a structural code such as NZS 1170.0 to verify whether the design is suitable or not [8].

The Eurocode standards are seen as the international benchmark for structural design. They contain design data, models and procedures which can be used in conjunction with the proposed design process for full analytical design of concrete structures exposed to fire [9] [10]. Due to its permitted use in the current New Zealand concrete design standard NZS 3101, the Eurocode was selected as the core resource for development of this design procedure.

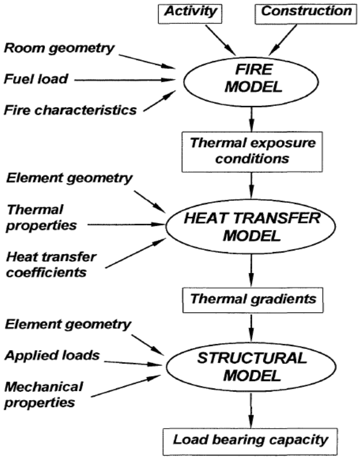

Figure 1 Design Process Steps [5]

4 FIRE MODEL

In order to solve the structural design problem the fire model must predict gas temperature development in the compartment so the heat transfer to the fire-exposed concrete member can be calculated [5]. The Eurocode allows one of the following advanced fire models; One-zone, Two-zone and computational fluid dynamic (CFD) models [11]. The Eurocode parametric fire is a simple one zone model which allows the calculation of the evolution of gas temperature as a function of time for the entire compartment, assuming a uniform temperature distribution. It is hence only applicable for post-flashover fires.

The Eurocode parametric fire model was developed to give a good approximation to the temperature profile and duration of the Swedish fire curves developed by Magnusson and Thelandersson [5]. They were the most widely reference time temperature curves for natural fires before the development of the Eurocode fire [5]. The general shape of the Eurocode parametric fire curve is shown in Figure 2 [5]. The following parameters are included in development of the fire curve; the fire load present in the compartment, ventilation openings present in the walls and/or roof and the type and nature of the different walls of the compartment. The Eurocode parametric fire model is seen as being state of the art and is used in the design process to calculated compartment temperature development.

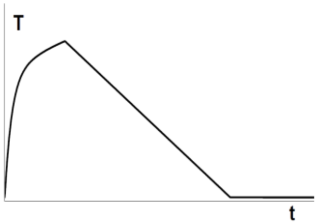

Figure 2 Typical shape of Eurocode Parametric fire [5]

5 HEAT TRANSFER AND SLAB TEMPERATURE DISTRIBUTION MODEL

To determine the capacity of a concrete slab we must first develop a model of the thermal gradients in the member throughout the duration of fire exposure. The temperature distribution through the slab thickness is calculated using a one dimensional heat transfer and temperature model based on the finite difference method (FDM).

In such a model the slab is divided it into a large but finite number of layers, Figure 3 [12]. The thickness of each layer is denoted as Δx. At the center of each of these layers is a node. Each node has a unique identifier based on its layer position and the temperature at each node is assumed to be representative of the average temperature throughout the layer. The bottom of the slab is exposed to the fire and the unexposed surface is exposed to ambient air. The finite difference method allows this problem to be solved either explicitly or implicitly. The explicit approach is easily implemented using a spread sheet software such as Microsoft Excel ©. Using the explicit approach the temperature of the node is computed directly based on the temperatures of the adjacent cells at the last time step.

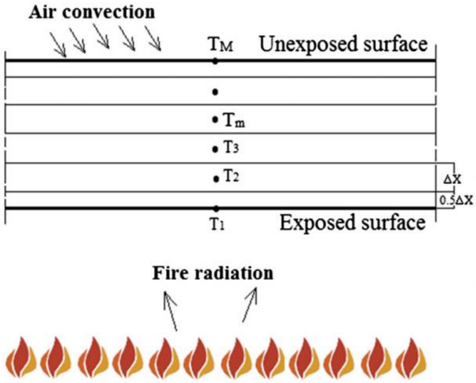

Figure 3 Heat Transfer Model [12]

5.1 Energy Balance Equation

The temperature in the system is solved by inserting a heat transfer equation between each node and solving the energy balance equations. This requires an energy conservation equation be found for each nodal point. The inputs are defined at boundary positions, the initial fire exposure surface is specified as Node T_1. Node T_1 includes the temperature transfer equations from the fire to the concrete element surface whereas the interior nodes only include conduction.

The heat transfer from the fire to the surface nodes on the fire exposed side and ambient side of the slab are calculated using the energy balance, Equation 1.

$$\dot{h}_{net} = \dot{h}_{net,c} + \dot{h}_{net,r} + \dot{h}_{net,cond} \tag{1}$$

For internal nodes this is reduced to, Equation 2:

$$\dot{h}_{net\ m} = \dot{h}_{net,cond\ m-1} + \dot{h}_{net,cond\ m+1} \tag{2}$$

Net convective heat flux $h_{net,c}$ from the fire gases to the component and from the unexposed side to ambient per unit surface area is determined using Newton's Law of cooling [11]. The radiative heat flux $h_{net,r}$ from the fire to the component per unit surface area is determined using Kirchoff's Law [11]: Conductive heat $h_{net,cond}$ component per unit surface area is determined by the following equation, Equation 3:

$$\dot{h}_{net,cond} = \frac{\lambda_c(\Theta_1 - \Theta_o)}{\Delta x} \tag{3}$$

Where Θ is the temperature in K or ^{o}C and λ_c is the thermal conductivity in W/m^{o}C

The accumulated internal energy at each node is represent by Equation 4.

$$\dot{h}_{net}^{i+1} = \frac{\rho c_p(\Theta_o^{i+1} - \Theta_o^i)}{\Delta t} \cdot \frac{\Delta x}{2} \tag{4}$$

For the surface node temperature Equation 1, the heat flux equations and Equation 4 are combined and rearranged to get Equation 5:

$$\Theta_o^{i+1} = \Theta_o^1 + \left(\frac{\Delta t}{2 \cdot \rho \cdot C_p \cdot \Delta x^2}\right)\{2 \cdot [\xi \cdot [(\Theta_r + 273)^4 - (\Theta_m + 273)^4] + (\Theta_{m+1}^i - \Theta_m^i) \cdot (\lambda_{cm+1} + \lambda_{cm}) + \alpha_c(\Theta_g - \Theta_m)]\} \tag{5}$$

In the same manner Equation 2, the heat flux equations and Equation 4 are combined to get Equation 6:

$$\Theta_o^{i+1} = \Theta_o^1 + \left(\frac{\Delta t}{\rho \cdot C_p \cdot \Delta x^2}\right) \cdot [(\Theta_{m-1}^i - \Theta_m^i) \cdot (\lambda_{cm-1} + \lambda_{cm}) + (\Theta_{m+1}^i - \Theta_m^i) \cdot (\lambda_{cm+1} + \lambda_{cm})] \tag{6}$$

5.2 Properties of Concrete

The thermal conductivity and density properties of concrete are modified at each time step using the material data in Eurocode 2. The material properties are assignment based on the average temperature between each node. The Eurocode properties are used exclusively due to the limited test data available on New Zealand produced concretes.

6 STRUCTURAL MODEL MECHANICAL RESPONSE

Once the internal temperature gradients at a given time have been predicted for the concrete member, it is possible to calculate the structural capacity of the member. This is done using a conventional structural analysis procedure and using the mechanical properties of concrete and steel at elevated temperatures [5]. The material properties for steel and concrete at elevated temperatures are given in the Eurocode 2 [13]. This assessment is carried out at each time step and the structural capacity of the member is calculated. It is assumed that the temperature of the steel reinforcing is equal to that of the surrounding concrete. In order to design a suitable member it is required that:

$$\mathrm{U}_{\mathrm{fire}}^{*} \leq \mathrm{R}_{\mathrm{fire}} \tag{7}$$

Where $\mathrm{U}_{\mathrm{fire}}^{*}$ is the design force resulting from the applied load at the time of the fire, and $\mathrm{R}_{\mathrm{fire}}$ is the load-bearing capacity in the fire situation. The design force $\mathrm{U}_{\mathrm{fire}}^{*}$ maybe an axial force, a bending moment or a shear force or a combination these actions [3]. The basic combinations for ultimate limit states used in New Zealand for checking strength are given in NZS 1170 [8]. The primary cold design combinations of actions is given as:

$$\mathrm{E_d} = 1.2\mathrm{G} + 1.5\mathrm{Q} \tag{8}$$

$\mathrm{E_d}$ is the design action effect, G is the permanent action and Q is the imposed or live action. The imposed load specified for a typical office floor is stated in NZS 1170.1 Table 3.1 as 3 $\mathrm{kN/m^2}$, the permanent load is the calculated mass of the slab per unit width. The fire design combination of actions are:

$$\mathrm{E_f} = \mathrm{G} + \text{thermal actions arising from the fire} + \Psi_l \mathrm{Q} \tag{9}$$

A long term factor is used, Ψ_l reflects different aspects of the variability of action being considered, the long term factor for office floors is given in NZS 1170 Table 4.1 as $\Psi_l = 0.4$.

6.1 Simply supported slabs

The simplest reinforced concrete members to design from a structural perspective are simply supported slabs, this is also true for fire design. The conventional design procedure for concrete slabs assumes the following; concrete has no tensile strength and the parabolic compressive block in concrete can be approximated by an equivalent rectangle, which can be seen in Figure 4. It is also assumed that the compressive block does not see elevated temperatures which causes a reduction in material properties and that the flexural capacity is solely a function of the temperature of the reinforcing steel.

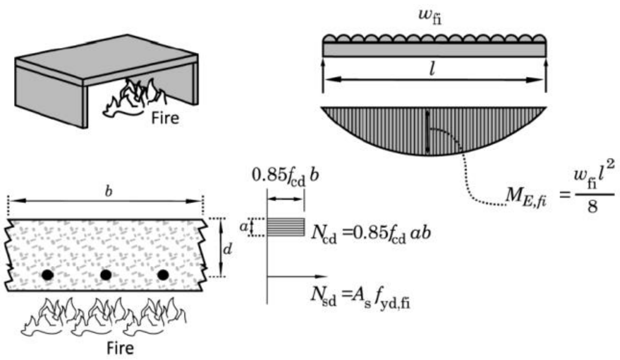

Figure 4 Simply Supported slab exposed to fire [5]

6.2 Bending moment

The design equation for a slab subjected to a bending load M_{fire} is given in Equation 10 [5]:

$$M_{fire} \leq M_f \tag{10}$$

The flexural capacity under fire conditions M_f is given by

$$M_f = A_s f_{y,T} \left(d - \frac{a_f}{2}\right) \tag{11}$$

Where A_s the area of the reinforcing steel in mm^2, $f_{y,t}$ is the yield stress of reinforcing steel for a given temperature in MPa and d is the effective depth of cross section in mm.

The stress block reduced by fire is given by Equation 12.

$$a_f = \frac{A_s f_{y,T}}{0.85 \cdot f_c' \cdot b} \tag{12}$$

Where f_c' is the compressive strength of concrete in MPa and b the width of the slab in mm.

The reduction of flexural capacity for bending is solely a function of the temperature of the reinforcing steel [5]. The strength properties of the reinforcing steel at elevated temperatures are stated in Eurocode 2 Table 3.2 [13].

7 TIME-EQUIVALENT FORMULAE

To allow comparison between the predictions of the proposed design process and equivalent severity methods, the Eurocode time equivalent formulae is used. The method relates the severity of a natural fire to the equivalent period of exposure in the standard fire test. The verification is then that the fire resistance is greater than the time equivalent value, Equation 13.

$$\mathrm{t_{e,d}} < \mathrm{t_{fi,d}} \tag{13}$$

Where $t_{fi,d}$ is the fire resistance of the members and $t_{e,d}$ is the equivalent time. The Eurocode 1 Annex F provides an approach to determine the equivalent time of fire exposure, given as:

$$\mathrm{t_{e,d}} = (\mathrm{q_{f,d}} \cdot \mathrm{k_b} \cdot \mathrm{W_f}) \cdot \mathrm{k_c} \qquad [\mathrm{min}] \tag{14}$$

The relative input parameters are the design fire load density $q_{f,d}$,determined in accordance to Annex E. k_b is a conversion factor which depends on the thermal properties of the enclosure and K_c is a correction factor used to allow the equation to be used for various types of materials (for reinforced concrete $k_c = 1$). The final variable W_f is the ventilation factor, in the absence of horizontal openings (roof vents) and a floor area is less than 100 m^2 this can be calculated using the expression 15.

$$\mathrm{W_f} = \mathrm{O}^{-1/2} \cdot \mathrm{A_f}/\mathrm{A_t} \tag{15}$$

Where O is the opening factor, A_f is the floor area and A_t the total surface area of the room. A similar formula is used in New Zealand C/VM2 Verification Method, however the Eurocode 1 formulae is used in order to maintain a consistent approach to the calculation of parameters used in the formulation of the parametric fire and time-equivalent method [15].

8 SLAB STUDIES

Studies were conducted to compare the fire resistances for slabs exposed to both the ISO 834 standard fire and natural fire. The fundamental parameters of the room and structural problem remained unchanged between each study i.e. design load, span, and the cold material properties. The slab was designed to meet the load actions specified in NZS 1170 for an importance level 2 building and the flexural capacity is determined in accordance with NZS 3101 for cold design and for the fire design using the modified material properties for reinforced steel in the Eurocode. The compartment is 5 x 5 meters square with a ceiling height of 3 meters and constructed of normal weight concrete. The fire load energy density is calculated using Eurocode 1 Annex E for an office area [11].

A 200 mm thick slab made from concrete with compressive strength of 30 MPa and a single row of 12 mm grade 300E reinforcing bars is used in each study. The concrete cover depth, axis distance, is increased in order to provide fire resistance. In order to maintain a constant self-weight and flexural capacity of the slab, as the cover distance increases the flexural capacity is maintained by increasing the number of reinforcing bars, refer to Equation 15.

On the primary axis of the graphs the temperature of the slabs reinforcing bars is plotted over time. On the secondary axis the flexural capacity M_f is plotted over time and a horizontal line M_{fire} shows the required minimum capacity for fire design. Once the capacity of the slab subjected to fire degrades to a value equal to M_{fire}, the slab fails and the time corresponding to this flexural capacity is the fire resistance.

8.1 NZS3101 prescriptive comparisons

The design procedures calculated fire resistance for an RC slab exposed to a standard fire for the specified axis distances given in NZS 3101 are shown in Table 1. They are compared with the prescribed fire resistance ratings in NZS 3101 for each axis distance. A typical temperature time curve is shown for 60 minutes (20 mm) concrete cover, the flexural capacity of the slab M_f can be seen to fall below M_{fire} at 57 minutes, Figure 5.

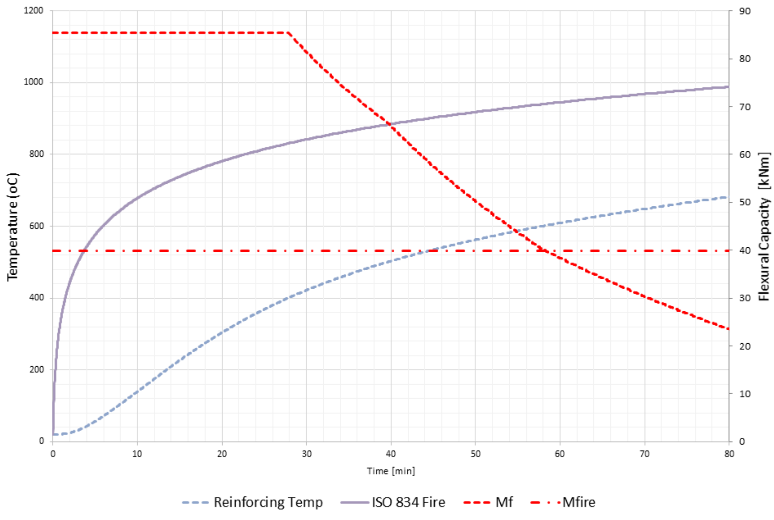

Figure 5 Slab exposed to ISO 834 fire, 60 minutes concrete cover

Comparison of the results show close agreement between the design procedures calculated fire resistance and the prescribe values in NZS 3101. The design procedure shows a strong linear relationship between axis depth and calculated fire resistance, this type of relationship has been identified by other researchers [12].

Table 1: Fire resistance for given axis distance.

Axis distance (mm) Simply supported slabs One-way	**NZS 3101 Fire resistance rating (minutes)**	**Design Procedure (minutes)**
10	30	32
20	60	59
30	90	87
40	120	124
55	180	182
65	240	227

8.2 Natural fire exposure

The RC slab was exposed to three types of natural fire profiles made up from a short duration high temperature fire, Figure 6, where the maximum temperature exceeds the standard fire and a long duration low temperature fire, where the temperature does not exceed the standard fire and one where the growth phases approximates the standard fire curve. The Eurocode fire temperature profile can be easily modified by altering the ventilation factor; increasing the ventilation factor increases the burning rate of the fuel due to the corresponding increase in available air, this produces increased fire temperatures but reduces fire duration. Decreasing the ventilation factor sufficiently produces a low temperature fire profile with an increased fire duration. Ventilation factors of 0.2 and 0.12 were used for the high temperatures and for the low temperature fires a ventilation factor of 0.04. A ventilation factor of 0.07 approximates the standard fire curve well up to the beginning of the decay phase of the fire.

The fire resistance required to provide for the complete burn out of the compartment is calculated two ways, firstly by using the equivalent time method and then using the design procedure.

The design procedure is used to find the required axis distance which maintains the slabs flexural capacity above the minimum required fire capacity M_{fire} for the given fire exposure. The equivalent fire resistance can then be found by relating the required axis distance back to the fire resistance values calculated in Table 1 for exposure to the standard fire. For example the required axis distance for a slab exposed to a Eurocode fire with a ventilation factor of 0.2 in this study is 20 mm, Figure 6. The point where the standard fire exposure crosses the capacity line M_{fire} is the equivalent fire severity on a load capacity basis [5]. The flexural capacity for a slab exposed to a standard fire with 20 mm concrete cover curve is shown plotted on Figure 6 as M_{ISO}. In Table 1 and Figure 6 it can be seen that 20 mm cover is approximately 60 minutes fire resistance for a slab exposed to the standard fire.

Table 2: Calculated fire resistance required

Ventilation factor	**Time-equivalent $t_{e,d}$ (min)**	**Design Procedure**	
		Required axis distance (mm)	**Equivalent fire severity (min)**
0.20	17	21	60
0.12	22	15	44
0.07	29	7	27
0.04	38	6	22

The time equivalent formulae and design method are in close agreement with each other for a ventilation factor of 0.07, were the Eurocode fire approximates the standard fire for the growth phase.

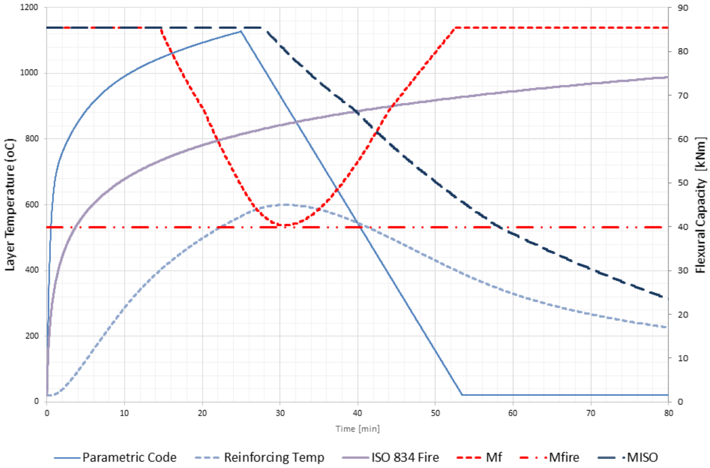

Figure 6 Eurocode fire, ventilation factor 0.2, 20 mm cover

9 DISCUSSION

As the temperature at the reinforcing axis exceeds 400 degrees the flexural capacity starts to reduce, this is temperature where the steel strength starts reducing in accordance with the values provided in the Eurocode, see in Figures 5 and 6 above. Once the fire enters into the cooling phase it can be seen that the flexural capacity continues to decrease, Figure 6. This behaviour is caused by the thermal lag between the exposed face of the slab and the reinforcing bars. The minimum flexural capacity occurs at the maximum reinforcing temperature not at the maximum temperature of the fire. Once this maximum steel temperature is reached the flexural capacity starts increasing as the reinforcing bars cool. The implications of this effect is that designs based on the predicted maximum fire duration may provide insufficient fire resistance to prevent structural failure and that fire duration needs to take into account the effect of thermal lag.

The calculated fire resistance using the two methods differs significantly for both low and high ventilation factors. As the ventilation factor decreases the time-equivalent formulae requires increased fire resistance and as it ventilation factor increases the required fire resistance decreases. This seems to be caused by the time-equivalent formulas dependence on ventilation factor for predicting the duration of the fire and the implication that the fire growth continues along the standard fire curve. This seems to be supported by the results for the design procedure with a ventilation factor approximating the standard fire, the time-equivalent method and design procedure provide relatively close values for the required fire resistance, Table 2.

It can be seen from the design processes calculated fire resistances in Table 2 and Figure 6 that short duration, high temperature fire exposure has more impact on the structures ability to resist fire than low temperature fires. The time equivalent formulae seems to under predict the required fire resistance significantly and based on these results should not be used. Inversely in compartment fires which are cooler than the standard fire the time equivalent methods tends to over specify the fire resistance requirements.

Over all the time equivalent method seems to require fire resistance values which are counter intuitive to the understanding that damage to a structure is largely dependent on the amount of heat absorbed. This problem has been identified by other authors [1].

10 CONCLUSIONS

Design processes such as the one presented in this paper are relatively simple to implement and provide significant insight into a fire development inside a compartment and the effects on structural capacity.

The Eurocode time equivalent formulae provides fire resistance values which are counter intuitive when compared to the behaviour demonstrated using minimum load capacity concept approach to determining fire resistance.

Short duration high temperature fires need to be specifically designed for where compartment temperature may exceed the standard fire temperature curve.

The possible failure of RC slabs exists after the fully developed fire begins to decay due to thermal lag.

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