Fire performance of high strength reinforced concrete walls

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ABSTRACT: An increasing interest in fire safety engineering can currently be identified in Australia and overseas. However the question of structural adequacy of concrete walls in fire has generally been ignored and the fire-resistance of walls has been taken to refer to performance with regard to insulation and integrity only. A general numerical procedure to analyse the structural behaviour of load-bearing reinforced concrete walls under elevated temperatures is developed in a project jointly conducted by the University of Melbourne and Monash University. The program will also incorporate high-strength concrete (HSC), which is becoming very popular around the world due to its many advantages over normal strength concrete (NSC). It will therefore take into account the significant behavioural differences between HSC and NSC, most notably the sudden spalling under elevated temperatures, whereby pieces of hardened concrete explosively dislodge. The important findings up to now are presented in this paper.

1 INTRODUCTION

1.1 *Importance of fire design*

Fire safety engineering is a very important aspect of designing structures. In Australia, about 150 fatalities related to fire occur every year. The cost of fire and fire protection to the community is typically 0.8% of the GDP, which amounts to about A$ 1.6 billion per year, and direct property losses due to fires are approximately A$ 300 million per year (Leicester 1989). These statistics show how important fire is in designing structures, both in terms of loss of life and loss of property.

1.2 *High-strength concrete and walls*

There is an increasing use of load-bearing reinforced high-strength concrete (HSC) walls in the design of buildings. The increase in HSC ($f'_c \geq 50$MPa) use is because it has many advantages and it has also become easier to manufacture, due to the availability of a variety of additives such as silica fume and water reducing admixtures. Over the years, reinforced concrete walls have gained greater acceptance as load carrying structural members due to the increased research undertaken on concrete walls and subsequent increases in allowable design stresses incorporated in various concrete codes. The reason for the newfound popularity of using load-bearing reinforced HSC walls is mainly due to the trend towards reinforced concrete core walls in high-rise buildings and the increased acceptance of tilt-up and other types of precast structures. In addition to providing a load-bearing function, these walls usually provide a fire separating function between compartments in modern structures, and they must therefore satisfy all three fire safety requirements, namely integrity, insulation and structural adequacy, at high temperatures (AS 3600 2001).

Currently, the Australian Concrete Structures Code (AS 3600) and other major codes around the world cover only normal strength concrete (NSC). Some designers use the Norwegian, New Zealand or the Canadian Code to design HSC members. The provisions relevant to HSC walls are given in the next section. The Canadian Concrete Code however, does not contain any clauses relating to fire design. Several researchers have however concluded that with the exception of spalling, which is the detachment of pieces of the hardened concrete surface exposed to fire, it is acceptable to use formulae based on NSC to design HSC members. This is of course, still debatable and participants of the NIST workshop in 1997 (Phan et al. 1997) have recognized that the amount of test-data on fire-exposed HSC is insufficient relative to the number of variables.

Despite the increasing use of load-bearing HSC walls in buildings, there have been no investigations reported on HSC walls subjected to fire conditions, both locally and internationally (Phan et al. 1997), except for an experimental study at Monash University (Crozier & Sanjayan 2000). The numerical program developed in the project reported in this paper will simulate thermal and structural behaviour of concrete walls in fire, incorporate HSC and will be compared with experimental results obtained from tests of load-bearing slender reinforced concrete walls in fire conducted at Monash University.

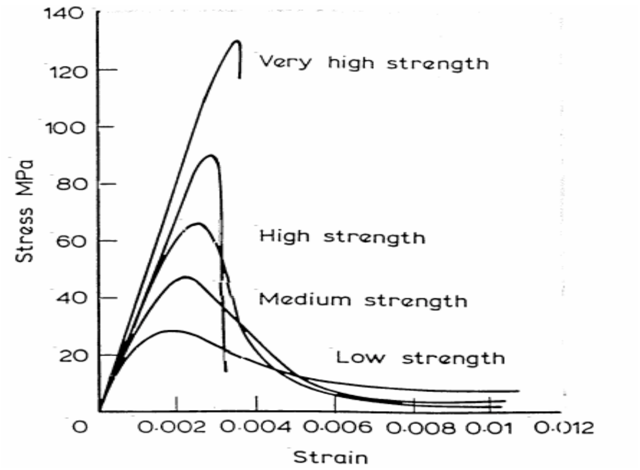

Figure 1. Uniaxial stress-strain curve for concrete

The experimental results will be used to calibrate a stress-strain curve for HSC, which has different behaviours from that of NSC, as shown in Figure 1 (Pendyala 1997). More efficient charts for the fire design of reinforced concrete walls will also be developed, since many parameters that are ignored in the codes are taken into account in the numerical model.

2 CODE METHODS

2.1 *Norwegian concrete code, NS 3473E - 1992*

The design tables for fire resistance of concrete walls can be found in Appendix B.3.3 of NS 3473E-1992. The tables can be used for concrete of strength classes up to C65 or LC65 (Compressive cylinder strength of 54 MPa). Both non-load bearing and load-bearing walls are covered, but only in terms of insulation. Tables B.3 and B.4 from NS 3473E - 1992 are shown below as Tables 1 and 2 respectively. The code also provides a method for the calculation of fire resistance period of beams and slabs for concrete of strength classes up to C65 or LC65 but the tables are much simpler and more convenient to use.

As can be seen from Table 2, its use is restricted to values of slenderness ratio ≤ 25 and also to concrete strengths of 54 MPa.

Table 1. Non-load bearing walls. 1) Minimum wall thickness

	Fire resistance in minutes					
	30	60	90	120	180	240
Minimum wall thickness	60 2)	80 2)	100	120	150	175

1) Can be assumed to be rigid in its own plane
2) The height shall not exceed 3m

Table 2. Load-bearing walls. Minimum wall thickness and the associated depth of reinforcement.

Fire resistance in minutes	Concrete stress	Wall thickness/Depth of reinforcement	Concrete stress	Wall thickness/Depth of reinforcement
30		120/10		120/10
60	$\sigma_c \le 0.15\ f_{ck}$ (Accidental limit state of fire)	120/15	$\sigma_c \le 0.30\ f_{ck}$ (Accidental limit state of fire)	140/25
90		140/25		170/35
120		160/35		220/45
180		200/55		300/65
240		240/75		400/85

The table is applicable for walls with slenderness ratios $l_k/t \le 25$

2.2 *New Zealand Code, NZS 3101: Part 1: 1995*

In NZS 3101: Part 1, the wall has to satisfy insulation, integrity and stability criteria if it has a fire-separating function. Integrity requirement is assumed to have been met if the wall meets the requirements for both insulation and stability. Tables 6.1 (insulation) and 6.4 (stability) from NZS 3101: Part 1 are shown below as Tables 3 and 4 respectively.

Although NZS 3101: Part 1 states that the clauses are applicable to concrete strength between 17.5 MPa and 100 MPa it should be noted that the clauses relating to fire in NZS 3101: Part 1 are based on previous work on low strength concretes.

Table 3. Minimum effective wall thickness for fire resistance ratings for insulation

Fire resistance rating (minutes)	Effective thickness (mm) for different aggregate type		
	Type A* aggregate	Type B* aggregate	Type C* aggregate
30	50	45	40
60	75	70	55
90	95	90	70
120	110	105	80
180	140	135	105
240	165	160	120

* Aggregate types:
A-quartz, greywacke, basalt & all others not listed
B-dacite, phonolite, andesite, rhyolite, limestone
C-pumice & selected lightweight aggregates

Table 4. Minimum cover to vertical reinforcement and tendons for stability of walls

Fire resistance Rating (minutes)	Cover, *c* (mm)	
	To reinforcement	To tendons
30	20	30
60	20	30
90	35	30
120	40	30
180	45	35
240	50	50

2.3 *British Standards, BS 8110: Part 2-1985*

The design rules for concrete walls in fire are presented in Table 4.6 of BS 8110: Part 2. The design tables are based on a limited number of standard fire tests on low strength concrete only. Several design parameters including wall height, load eccentricity and load level are not directly considered. The clauses relating to fire are explained in more detail by Martin et al. (1989).

A working group appointed by the Concrete Society of UK reviewed all the clauses in the British Code and recommended extensions to cover HSC (The Concrete Society 1998). However, other than identifying the problem of spalling and lack of research in this area, no recommendations were made.

3 SPALLING

Spalling is the phenomenon whereby pieces of the hardened concrete surface exposed to fire break away explosively or fall-off during the course of rapid high temperature exposure. Observations from fire tests on concrete specimens indicate that spalling ranges from progressive, where minor pieces are dislodged and there is a gradual reduction in cross-section, to explosive, where test specimens are suddenly disintegrated into fine fragments, accompanied by a sharp loud bang and the release of a sufficient amount of energy which projects the broken concrete fragments in all directions at high velocity. HSC has been found to be more prone to spalling failure than NSC when exposed to relatively rapid heating (above 1°C/min). From a review of the literature, it can be summarised that tendency for spalling is high when:

- Cover to reinforcement is increased, especially more than 50 mm;
- Moisture content of the concrete is high;
- The temperature rise of the fire is rapid;
- The concrete specimen/member is subjected to compressive stress;
- The concrete specimen/member is subjected to high thermal gradient.

It has been theorised that the risk of spalling for (HSC) is higher due to the following reasons:

- Lower permeability of HSC, causing higher moisture content at the time of heating,
- Lower porosity of HSC, causing faster build up of pore pressures, and
- HSC members are typically subjected to higher compressive stresses than lower strength concrete.

There are conflicting results in the literature on spalling of HSC. Explosive spalling of HSC is observed on some tests, while others reported no difference to the behaviour of NSC. In 1996, an extensive investigation on experimental and analytical studies on fire performance of HSC was conducted at the National Institute of Standards and Technology (Phan 1996). In the report, five out of the ten materials test programs reviewed and three out of the five element test programs reviewed reported explosive spalling. It was also observed that explosive spalling did not occur for every specimen tested under identical conditions. The reported temperature range when explosive spalling occurs is 300°C to 650°C. When compared with HSC made of normal weight aggregates, HSC made with lightweight aggregate appears to be more prone to explosive spalling. Concrete with dense pastes resulting from the addition of silica fume are more susceptible to explosive spalling. HSC specimens heated at higher heating rates, and larger specimens are also more prone to spalling than specimens heated at lower rates and of smaller size. Additionally, Phan et al. observed that explosive spalling occurs at a time when thermal gradient is at a maximum in the specimen (Phan et al. 2001).

Whether spalling occurs or not in a particular situation and the extent of spalling have a random element in them. A companion project proposed jointly by the University of Melbourne and Monash University will incorporate the theorized random effect in assessing the risk of failure of HSC specimens due to spalling of concrete.

4 NUMERICAL SIMULATION

4.1 *Thermal behaviour modelling*

Thermal simulation is concerned with predicting the variation of the temperature with time at varying depths of the concrete wall when one side of the wall is subjected to a fire. The fire in consideration follows the requirements of the Standard Fire Test Conditions (AS 1530.4 1997), as shown in Figure 2 (standard fire curve). Thermal response modelling forms the basis for assessing the insulation requirements of concrete walls exposed to fire.

The thermal behaviour of a concrete wall has been modelled using Visual Basic 6. The method used to simulate the one-dimensional transfer of heat in concrete walls is an explicit finite difference method from Harmathy (1993). It is assumed that the entire surface of one of the wall's face is exposed to the fire environment in the analysis. Temperature at a wall thickness is assumed to be the same for the height of the wall. The material models used to describe the thermal conductivity and volumetric specific heat of the concrete needed for the one-dimensional heat transfer problem are those from Lie (1992).

Figure 3 illustrates the temperature history predicted by the model at selected thicknesses of a 250 mm thick carbonate aggregate concrete slab subjected to the standard fire curve. The figure shows a high rate of heat transfer in regions close to the fire exposed surface and that this effect diminishes as the distance from the heated surface increases. Figure 4 illustrates the temperature distribution as a function of distance from fire exposed surface calculated by the numerical model. The figure shows a high rate of heat transfer during the early stages of the fire and that this effect diminishes as the fire continues. The figures correspond with the expected behaviour of concrete walls in fire.

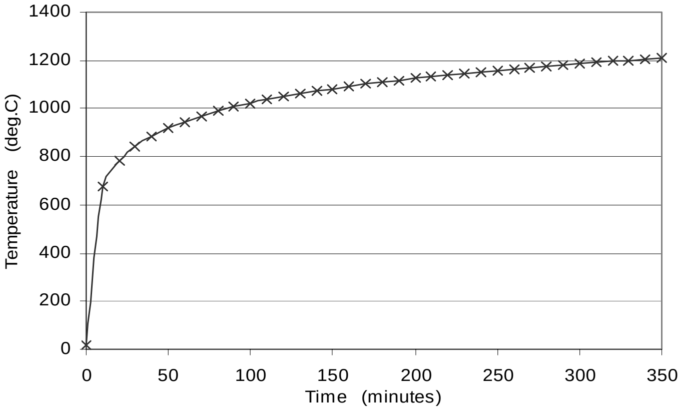

Figure 2. Standard fire curve from AS 1530.4

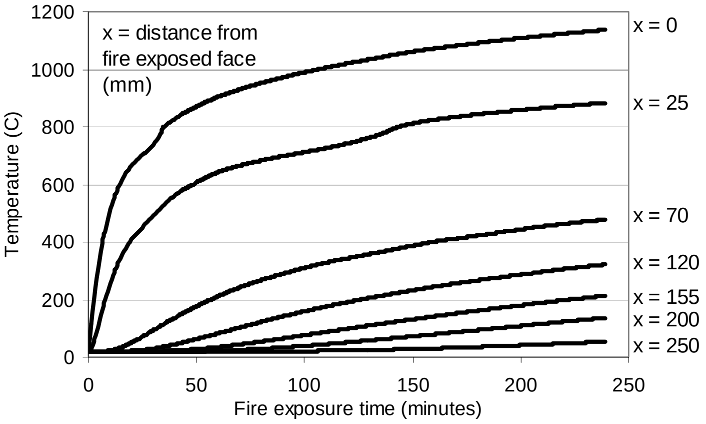

Figure 3. Temperature history at selected depths of a 250 mm thick carbonate aggregate concrete wall calculated by the numerical model.

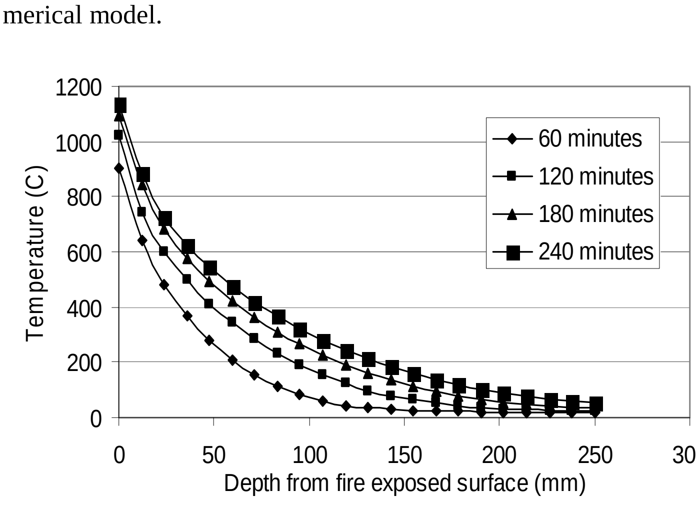

Figure 4. Temperature distribution as a function of distance from fire exposed surface calculated by the numerical model.

Traditionally, the principal fire resistance requirement of slab-like building elements such as walls is to provide a barrier against fire spread only. The fire resistance of the wall is therefore synonymous with the thermal fire resistance in such cases (Harmathy 1993). This can be readily obtained from the numerical model and a series of numerical analyses were undertaken on walls of varying thickness and aggregate type to obtain their thermal fire resistances. The results from the analyses are plotted in Figure 5, which also included the fire resistance level for insulation recommended by AS 3600 - 2001.

Figure 5 shows that the AS 3600 recommendations is very close to the behaviour exhibited by the siliceous aggregate concrete. This is not totally unexpected since the recommendations in AS3600 are based on the results obtained by a European-based numerical analysis, TASEF-2 (Wickström 1979) and the predominant type of concrete used in Europe is usually composed of siliceous aggregates (quartz based), which are reflected in the material properties of the TASEF-2 numerical model. Figure 5 also shows that walls composed of carbonate or lightweight aggregate concretes have improved thermal fire resistances when compared with siliceous aggregate concrete. Walls composed of lightweight aggregate concretes show the best performance among the three different aggregates, exhibiting nearly twice the thermal fire resistance of a similar thickness wall composed of carbonate aggregate concrete.

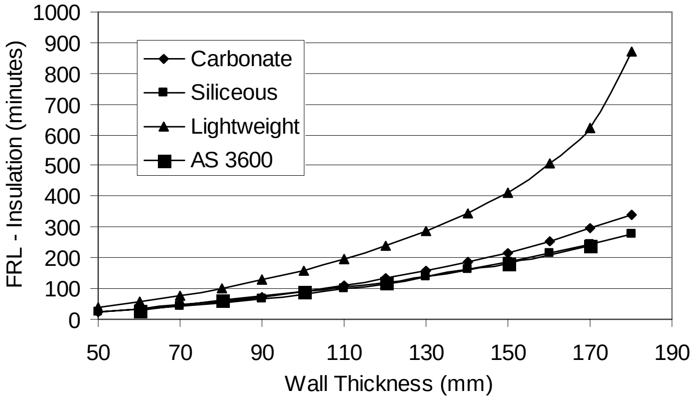

Figure 5. Thermal fire resistance of concrete walls

4.2 *Structural behaviour modelling*

The temperature data at each depth at a particular time-step obtained from the thermal model will be used to determine the temperature-dependent mechanical properties of the materials. An incremental solution procedure to assess the structural response of concrete walls in fire is adopted. For each time step during fire exposure, equilibrium is sought between the external load and internal resistance and deflection of the wall. Calculation of the equilibrium state of the wall, within a single time increment, is similar to that described by El-Metwally et al.

(1990), except that a more elaborate finite difference scheme is employed in the current model to calculate wall deflections. The incremental procedure is repeated until equilibrium of the wall cannot be maintained, thus indicating structural collapse of the wall. The time at which this occurs is termed the 'Fire Resistance Period' for structural adequacy. The steps are described in more detail below.

4.2.1 *Calculation of moment-curvature relationship*

- Divide the section into a number of discrete laminae.
- Assume plane sections remain plane after bending and obtain the linear strain profile across the section, and hence, strain at the middle of each element, for a definite value of curvature and an assumed strain at the top of the section.
- Obtain the value of stress corresponding to the strain for each element from the stress-strain relationship of the concrete.
- Obtain the value of stress corresponding to the strain for each reinforcement bar from the stress-strain relationship of the reinforcing steel.
- Determine the axial force, N, and the bending moment, M, corresponding to the assumed strain profile of the section.
- Obtain the imbalance axial force, ΔN, which is the difference of the calculated axial force, N, with the applied axial force, N_u.
- A point on the moment versus curvature curve has been obtained if ΔN is within an allowable tolerance, taken as $N_u/1000$. Otherwise, the neutral axis is increased, and the procedure for obtaining N and M is repeated until N_u is within the allowable tolerance.

4.2.2 *Calculation of deflection*

The following calculations are carried out on the half-height of the wall since it is a symmetrical problem.

- Assume a preliminary deflected shape as that of a sine curve and calculate the value of deflection at each segment.
- Calculate the value of bending moment at each segment, M_i.
- The value of the curvature corresponding to the moment is determined from the moment curvature relationship previously developed.
- Obtain the deflection at each segment by numerical double integration and compare the calculated deflection at midspan with the specified deflection to obtain Δx.
- Using this calculated value of Δx, calculate the new values for deflection at each segment.
- Repeat the previous steps and when the value of tolerance is within the allowable range, the height of the wall is determined as $L = 2n\Delta x$
- Repeat the previous steps using different values of midspan deflection to obtain corresponding heights of wall subjected to the same external loads.
- Repeat the previous steps for each time-step and the midspan deflection for the wanted wall-height can then be determined.

4.2.3 *Constitutive model*

The constitutive equation describing concrete deformation at transient high temperatures may be written as:

$$\varepsilon_{tot,c} = \varepsilon_{th,c} + \varepsilon_{sr,c} \qquad (1)$$

where $\varepsilon_{th,c}$ represents the unrestrained thermal strain and $\varepsilon_{sr,c}$ is the stress-related-strain of the concrete. The stresses in the concrete are calculated using stress-strain relations modified so that the maxima of the curves is shifted to higher strains for high temperatures to include creep effects. This form of constitutive modelling of concrete implicitly includes the transient creep strain component which is often considered separately by others (Anderberg & Thelanderson 1977, O'Meagher & Bennetts 1988).

The total strain in the steel $\varepsilon_{tot,s}$ is expressed similar to (1), with subscript c replaced with subscript s, to denote reference to the steel reinforcement. Calculation of the stresses in the reinforcement, and modelling of the variation, with temperature, of the mechanical properties of the reinforcement, is conducted in accordance with the Steel Structures code (AS4100 1990).

5 REVIEW OF EXPERIMENTAL STUDY BY CROZIER & SANJAYAN (2000)

Crozier & Sanjayan (2000) have tested eighteen large-scale slender reinforced concrete walls under standard fire conditions. The tests included conditions such as different slenderness ratios, concrete strengths and mixture proportions, reinforcement covers, and levels of eccentric in-plane load. The walls were tested under three different testing conditions. Eight walls were simply supported along two short edges and tested under combined inplane and lateral loads to investigate inplane load capacity while another eight walls were simply supported along two short edges and tested under lateral load only to investigate spalling and thermal bowing. Two walls were simply supported on three edges, two short and one long, and tested under lateral load only to investigate spalling with minimal flexural cracking. The wall thickness, t_w, ranged from 75mm to 150mm and the compressive strength ranged from 44 MPa to 70 MPa. All the test specimens had the same length (3600 mm) and width (1200mm), and the slenderness ratio ranged from 24 to 48. Structural failure was deemed to have occurred if one of the following criteria were met:

- Collapse of the loaded specimen; or

- Excessive lateral deflection of the loaded specimen occurred. The deflection limit is defined in AS 1530.4 as being deflections greater than span/20. The other failure criterion of excessive deflection rate of span/9000t_w (in mm/min) defined in AS 1530.4 did not occur in these fire tests.

The other failure criteria (insulation, which is the ability of a wall to restrict the average temperature rise on the unexposed face of the wall to less than 140ºC, and integrity, which is the ability of a wall to prevent passage of flames or hot gases through the wall when exposed to fire on one side) were not considered in the paper. The paper focuses only on the structural adequacy criterion, which is the ability of a wall to maintain its structural function during a fire.

The test results indicated that higher-strength concrete walls deflect less in fire when compared to lower-strength concrete walls. It may be due to the lower thermal expansion of higher-strength concrete and its higher thermal conductivity. Other conclusions from the tests included: thermal bowing contributes to a significant reduction to inplane load capacity; concrete strength has little influence on the inplane load capacity; and that inplane loads does not affect wall behaviour much up to approximately 25 minutes of fire exposure.

Three specimens exhibited spalling during the test. However, contrary to established knowledge, the higher-strength concrete specimen suffered relatively minor spalling, while the lower-strength concrete specimens exhibited more severe spalling. It was hypothesized by the researcher that this may be due to the different support conditions of the specimens. The lower-strength concrete specimens were supported along three edges (the higher-strength concrete specimen was supported along two short edges only), thus causing little to no flexural cracking, which eliminated a means of escape for the vapour pressure and consequently causing severe spalling.

As mentioned earlier, the tests covers both low and high-strength concrete, and looked at many different parameters. The results will be used to validate the numerical model that is being developed in this project.

6 FUTURE WORK

Presently, stress-strain curves at different temperatures that are applicable up to 100 MPa are not available. The results from the tests carried out by Crozier & Sanjayan (2000) will be used to compare and calibrate the numerical model, and also to develop a stress-strain curve suitable for HSC by modifying Lie's model (Lie 1992). The calibrated numerical model will then be used to develop guidelines for the new code. These results will be presented during the conference. Additionally, when a more accurate spalling model is developed, that spalling model will be incorporated into the numerical model.

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