Fire Safety of Concrete Buildings

Cement Concrete & Aggregates Australia

Guidelines on the Fire Safety of Concrete Buildings

Graph showing the relative strength of concrete at different temperatures.
Cement Concrete & Aggregates Australia is a not-for-profit organisation established in 1928 and committed to serving the Australian construction community.

CCAA is acknowledged nationally and internationally as Australia’s foremost cement and concrete information body – taking a leading role in education and training, research and development, technical information and advisory services, and being a significant contributor to the preparation of Codes and Standards affecting building and building materials.

CCAA’s principal aims are to protect and extend the uses of cement, concrete and aggregates by advancing knowledge, skill and professionalism in Australian concrete construction and by promoting continual awareness of products, their energy-efficient properties and their uses, and of the contribution the industry makes towards a better environment.

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Executive Summary

The findings from real fire investigations of concrete buildings have been studied as have recent experimental data on the behaviour of structures and elements in fire. Overseas data have been complemented by the findings from a recent fire experiment funded by the CCAA which involved high-strength concrete columns and prestressed floor slabs. The results of these studies and experiments are presented as are the findings from the review of overseas tests and real fire experience. Essentially, the key findings are:

- Serious fires in buildings are rare and there have been few cases where significant building failure has occurred for concrete buildings. Such fires should be avoided since rapid fire spread and intense smoke will pose a significant threat for the building occupants. The use of other fire safety measures required by building regulations (eg sprinklers) has an important role in avoiding the development of such fires. None of the buildings considered in this publication were fitted with a sprinkler system.

- The behaviour and resistance of concrete under elevated temperature conditions is complex but sufficient testing has been undertaken to provide the basis for describing the reduction in strength with temperature and simplified stress-strain relationships which can be used for design or advanced analysis. This is true for both normal and high strength concretes.

- The possibility of spalling has been long recognised in relation to both normal strength and high strength concrete with the likelihood of spalling increasing as the strength of the concrete increases.

- The addition of polypropylene fibres to the high strength mix had a dramatic effect in reducing the level of spalling. This finding is supported by recent Australian testing. A dosage rate of 1.2 kg/m$^3$ of 6 mm monofilament polypropylene fibres is recommended.

An overview of the ways in which buildings may be designed for fire safety is also presented. It is noted that buildings may be designed in accordance with the deemed-to-satisfy (prescriptive) provisions of the Building Code of Australia (BCA) or an Alternative Solution may be developed using the fire engineering process. Sometimes, this will have implications for the building structure allowing it, or parts of it, to have a lesser fire resistance that might be required by the prescriptive provisions. In many cases, it will be sufficient to use the provisions of AS 3600, it being noted that the new version of AS 3600 provides a more flexible range of solutions for members such as columns and walls compared with earlier versions.

The assessment of member resistance presented in this publication only relates to isolated member behaviour. In real fire situations, due to the fact that fire will often be limited to part of the building and/or its heating effects will vary, building structures can exhibit a level of resistance which is greater than that predicted by considering isolated member behaviour. However, the evaluation of such improved response is difficult and requires the use of complex finite element models but also a better understanding of fire growth and development.

Guidance is given to enable structural engineers to assess the structural adequacy of most building elements when subject to either a ‘real’ design fire curve or a period of standard fire exposure, should the situations being considered not be covered adequately by the prescriptive provisions of the BCA in combination with the tables and charts in Section 5 of AS 3600.
Section 1
Introduction

In Australia there have been no failures of concrete buildings in fire. The common view of concrete structures amongst both engineers and regulators is that these structures are inherently fire resistant and that high levels of fire resistance can be achieved by adopting certain member dimensions and cover to reinforcement. The reason for this is that concrete has both low thermal conductivity and high heat capacity, and that concrete elements are therefore naturally resistant to temperature rise due to fire exposure.

In the past, except perhaps for slender members such as blade columns, the design of concrete elements was rarely governed by fire requirements. The design of concrete structures for fire resistance was, and continues to be, a straightforward process – merely requiring selection of the correct member dimensions and covers for each member within the fire compartment. The positive perception of concrete structures and fire resistance has been reinforced by design standards for concrete structures which, at least in the fire section of the standards, have been relatively simple with tabular solutions being provided for various members – beams, floors, walls and columns.

Concrete technology appears to be advancing at a relatively high rate. Since the 1970s, there has been a greatly increased interest in high-performance concrete where concrete mixes are designed to achieve increasingly greater strengths and performances. This has been driven by the desire to construct buildings taller (to better utilise the land footprint) with smaller columns and thinner core walls (to increase the rentable area and functionality) and with minimum floor-to-floor height (to provide more floors within the building envelope). This has largely been achieved through the use of high-strength concrete (HSC) columns and post-tensioned floors. These developments in concrete technology have not gone unnoticed by overseas researchers interested in the behaviour of structures in fire, resulting in questions and issues being raised about the likely behaviour in fire – particularly of higher strength concretes [1].

In addition to the developments in concrete technology, the fire engineering design of buildings has become commonplace. Many major buildings, irrespective of the materials of construction, will be the subject of a fire engineering design. However, for concrete building structures, it is not often that the fire engineering process considers the building structure. This is because the prescriptive requirements – ie the deemed-to-satisfy (DTS) provisions – of the Building Code of Australia (BCA) [1] are usually easily met and durability requirements often govern the required axis distance to the steel reinforcement, ie the DTS fire provisions do not usually control the design. There are certain situations where this might not be the case and where the fire engineering process may need to consider the behaviour of the concrete structure. For example, the need to maximise the overall energy rating of the building may justify a fire engineering assessment of the situation to reduce the size of members. Also, some members may be lightly loaded or column cross-sections chosen on the basis of the requirements of AS 3600 [2] to satisfy the DTS provisions, but a different cross-sectional shape is desirable from a functional point of view. It is also sometimes found that older concrete buildings, which are the subject of refurbishment or modification, incorporate members that will not achieve the fire-resistance levels required by the DTS provisions of the BCA when assessed by AS 3600.

Research, funded by CCAA, has been undertaken to:

- investigate the behaviour of various elements of construction in fire;
- explain and develop simplified fire engineering procedures for concrete structures;
- investigate the effect of real fires on concrete structures.

This Guide presents the findings from the above work and from overseas studies in order to provide a better understanding of the performance and design of concrete structures in fire. This publication is not a design standard referenced by building regulations (cf AS 3600). It should be used as a guide to enhance understanding in combination with other publications such as AS 3600. It is intended for engineers and [1] In this Guide, the phrases 'normal strength concrete' (NSC) and 'high-strength concrete' (HSC) are frequently used. In Australia, HSC refers to concrete with a 28-day compressive strength of 60 MPa or greater, whereas in the United States, this has been taken to refer to concrete with strength greater than 41 MPa. There is also no sudden transition in behaviour between normal strength concrete and high-strength concrete. Therefore, wherever possible in this Guide, the strengths assumed to be associated with the use of these terms will be noted.
building practitioners who are involved with the design of concrete buildings for fire safety and who wish to gain a greater understanding of fire engineering and aspects of the behaviour of concrete in fire.

The Guide comprises two parts – Part A reviews the behaviour of concrete structures and elements under real and experimental fires. Part B presents details regarding the design of concrete structures for fire including the use of fire engineering principles where the prescriptive requirements may prove to be onerous. However, as an introduction to these two parts it is considered necessary to understand some basics about fires and fire safety.

Section 2
Fires and fire safety

2.1 Introduction

The term ‘real fire’ is commonly used to refer to fires that occur within buildings and that are initiated either accidentally or deliberately. Such fires are differentiated from the ‘standard’ fire which is the fire that is referenced by the DTS provisions of the Building Code of Australia (BCA) and AS 3600 and which is characterised by a standard fire temperature versus time relationship Figure 1. The latter relationship is given in AS 1530.4 which covers a standard test method for determining the fire resistance of elements of construction. A member that is able to resist exposure to this standard fire temperature versus time curve for a period of time \( t \) is said to have a fire resistance of \( t \). The various fire resistance levels (FRL) or grading periods recognised in the standard are 30, 60, 90, 120, 180 and 240 minutes and to achieve one such FRL, an element must have a fire resistance equal to or greater than the FRL grading period. There are three relevant failure criteria in relation to concrete structures – structural adequacy (the ability to resist load), integrity (the ability to resist the passage of flames) and insulation (the ability to prevent fire spread due to unacceptable temperature rise of the non-heated face). Thus AS 1530.4 and the BCA potentially reference three numbers with respect to a required FRL. For example, a floor in an office building may be required by the BCA to have an FRL of 120/120/120 whereas a column supporting that floor is required to have an FRL of 120/-/-.. The latter criteria (ie integrity and insulation) are not listed since they are not relevant for a column since its purpose is to

![Figure 1: Standard time temperature curve (AS 1530.4)](image)
support the level above rather than specifically provide fire separation between two levels. Furthermore, it is implicitly assumed by the DTS provisions of the BCA that fire compartmentation is effective such that elements within a compartment need to be designed to resist only a fire within that compartment [2]. Thus floors and walls at the compartment boundaries are designed assuming fire on one side only, not on both sides at the one time.

In practice, real fires are rare and many are extinguished before they become significant [3]. Considering fires that result in an alarm to the fire brigade, it is known that 77% of these in office buildings are self-extinguished or extinguished by human activity without the assistance of special measures such as sprinkler systems. The presence of a sprinkler system makes it much less likely again, that the building structure will be subject to a fire generating significant heat. The provision of a sprinkler booster connection for the fire brigade means that the fire brigade can supplement the pressure within the system should the sprinkler pumps fail to operate. Such ‘active’ fire safety measures are required by the DTS provisions of the BCA for tall buildings (>25 m in effective height) and large area buildings. Enhancements to the sprinkler system, which are not required by the DTS provisions, are possible and these can further reduce the likelihood and severity of a fire. For example, the use of subsidiary monitored valves for each floor of the building and associated test ‘valves’ reduce the probability that the sprinkler system will be isolated on the floor of fire origin [4] and reduce the chance that a severe fire could spread to the floor above [5]. The provision of ‘tap valves’ provides a means of checking that water is present on a floor. These comments emphasise the fact that, in Australia at least, severe real fires in significant buildings, are very rare. Even in low-rise office buildings – where sprinklers are not required – such fires are rare.

2.2 Real fire characteristics

The characteristics of significant real fires and how they vary from those associated with the standard fire temperature versus time curve are considered below.

2.2.1 Extent of burning

As will be noted in the review of some of the building structure fires described in Part A of this Guide, real fires may extend throughout the building. This may be the result of the nature of the building, ie where there is little compartmentation or where compartmentation is relatively ineffective. In respect of ineffective compartmentation, it needs to be noted that it is the edges of multi-storey buildings that present the greatest potential point of weakness due to gaps between the edges of the floors and the facade (a common occurrence), but more particularly the inability of the edge details (spandrels if provided (a)) to prevent flame spread to the level above.

It is possible therefore for a severe fire on one level to spread vertically and affect the levels above (and possibly below) so that structural elements, such as columns and walls, may be heated over significant lengths at the one time. Similarly, it is possible that floors may be heated simultaneously from above and below. Whether such situations are acceptable or not is considered in Section 2.3.

2.2.2 Rate of temperature rise

The rate of rise of fire temperature is dictated by the rate at which fuel is pyrolised (ie solid mass is converted to combustible gas) and burnt when these combustible gases come into contact with air. Once burning takes place, there will be an acceleration of pyrolysis due to the heating and, given more air, more heat will be generated. The factor that has by far the greatest impact on the availability of air for burning in a multi-storey building is the breaking of external windows. This is a function of the air temperature developed and as this increases, more and more windows will break. However, the breaking of windows incorporating toughened glass is not instantaneous. A test conducted on an enclosure with a substantial toughened-glass facade [6] found that window breakage to the point that the fire reached its maximum severity took about 18 minutes, at which point the air temperatures reached around 1000°C at various locations. The final climb of air temperatures from about 300°C to 1000°C took around

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[2] A compartment may be considered as a box which seeks to contain the heating effects associated with a fire. However, since all compartments have openings in order to make them functional (eg windows and doors, service penetrations), such openings provide points of weakness.

[3] A significant fire is defined as one where the associated air temperature will result in loss of strength of an exposed member. Tests have shown that fires that are extinguished by occupants or by sprinklers will have no effect on exposed members.

[4] The most common cause of sprinkler failure is that the system has been isolated to permit tenancy upgrade work. If only one floor is isolated compared with all of the floors associated with the sprinkler zone, there is a lower chance that the floor on which a fire starts will have been isolated.

[5] The severe fire will have occurred on a floor where sprinklers have been isolated but the level above is still active and should be able to resist the spread of fire to this level.

[6] The DTS provisions of the BCA do not require spandrels if the building is a sprinklered building.
Admittedly, this is only one test result, but it demonstrates that rapid rises in air temperatures are possible in real fires. It is worth noting that the standard fire specifies a fire temperature of 700°C at 15 minutes and 745°C at 18 minutes. It follows that the temperature rise rate in real fires can be considerably greater than that associated with the standard fire.

### 2.2.3 Duration of burning

The effective rate of burning is a function of the available ventilation and other factors such as the geometry of the enclosure. A discussion of the duration of burning can be found in Appendix E of Reference 5.

The duration of burning is fundamentally a function of the total fuel available to be burnt divided by the effective rate of burning. The rate of burning refers to the mass loss rate of the fuel as it is pyrolysed to form a combustible gas. Not all of the combustible gas will burn inside the fire enclosure but a significant amount may burn outside the openings where it comes into contact with fresh air. Higher rates of burning will give rise to lower burning durations for the same quantity of combustibles. As a general rule, higher rates of burning will be achieved for enclosures with:

- Higher exterior windows – burning rate is dependent on, where \( \sqrt{h} \) is the height of the potential openings associated with a window;
- A greater area of external windows – burning rate is dependent on the area of windows;
- A lesser depth of the floor measured perpendicular from the potential facade openings (as a general rule, the rate of burning will reduce as the depth from the potential openings increases).

An illustration of the differences in fire duration that might be expected by varying these conditions is given in Figure 2. If the situation shown in Figure 2[a] has a burning duration of around 30 minutes, then for the situation shown in Figure 2[b] it would be expected to be 170 minutes.

If the floor area was larger, then the burning duration would be proportionately larger.

It follows that the duration of burning is highly variable and dependent on the extent of external glazing. Under the DTS provisions of the BCA, elements must survive the standard fire for 120 minutes in the case of offices and 180 minutes for retail premises.

### 2.2.4 Fire temperatures

From experimental fires within enclosures, it appears that although air temperatures may exceed 1000°C occasionally, the average fire temperature is 1000°C or less. It is commonly assumed that fire burning conditions are uniform within an enclosure. Although this is a reasonable approximation for small enclosures it is not the case when enclosures are very large or deep. Testing 6 has found that burning will take place adjacent to the openings with lower temperatures being experienced away from the openings. The effect of this is to increase the duration of burning, with members at the exterior of the building being subjected to hot gases for longer than those within the interior.

Higher fire temperatures may be reached in well-ventilated conditions involving the burning of hydrocarbon fuels but this situation is unlikely in commercial buildings since the fuel is still predominantly cellulosic.

Testing has provided important guidance and insights on factors that influence the rate of burning. There is, however, no definitive mathematical model able to accurately describe the development of a real fire within a building from initiation through to full fire development 7.

### 2.3 Risk management imperatives

As noted in Section 2.2, it is possible for real fires to:

- extend beyond compartments and possibly throughout the building (uncontrolled);
- have a duration of burning that could be well in excess of that anticipated by building codes; and
- have a relatively rapid rise of temperature with time compared with that associated with the standard fire temperature versus time curve.

It is not reasonable to expect a building to resist any fire that is possible. Nevertheless uncontrolled fire spread is considered unacceptable and effective measures need to be put in place to minimise the occurrence of such fires – especially in buildings with many occupants. Such fires are unacceptable.
because they will have a potentially disastrous effect on occupants. This is particularly the case with tall buildings where evacuation could take a substantial time, even if the stairs remain free of smoke.

Fire is a 'load' on a building that should be addressed by measures other than simply considering the fire resistance of the structure. This is because the associated fire and smoke will have at least as great an effect on the occupants as it does on the structure. Fire fighting within the building becomes almost impossible. The provision of fire sprinklers is perhaps the most important measure, as noted in Section 2.1. There are other measures that also reduce the likelihood of a fire spreading throughout a building. It is therefore not considered reasonable to expect a building structure to resist an uncontrolled fire throughout the building. However, it is reasonable to assess the performance of the building structure in terms of the expected fire characteristics within a level [7]. Nevertheless, as will be noted in Section 3, there have been very few catastrophic failures of tall buildings in fire.

Although there are many fires in buildings, only the unusual or spectacular ones are reported in the public and technical literature. It can be assumed therefore, that for every reported major fire incident, there are many others where nothing of note occurred. Some of the notable incidents are now presented. Most of these are mentioned by Beitel et al in a paper which gives a brief description of the various incidents including several involving concrete structures, viz the Katrantzos Sports Building, The Minion Department Store, CESP Building 2 Sao Paulo and the Windsor Tower, Madrid.

From a consideration of these fires and others associated with steel building construction, the following observations are made:

- In all cases, the fires were very extensive and either there was no compartmentation or it was almost totally ineffective. None of the buildings had a sprinkler system fitted at the time of the fire.

- The buildings were unoccupied at the time of the fire. This further increases the chance of a large fire since occupants usually detect and take measures to extinguish a fire.

- As a general rule, fires that are permitted to become very severe through the absence of key fire safety measures have a greater chance of spreading throughout the building since:
  - more potential openings between compartments will be exposed to heat and flames and increase the likelihood that one or more penetrations may be breached;
  - external flaming will be greater and this increases the chance of the fire spreading to levels above via the external openings;
  - thermal expansion will tend to open gaps and this may assist with the spread of fire;
  - it becomes increasingly more difficult for the fire to be extinguished due to the amount of heat being released and the difficulty of access.

- Given the apparent duration and severity of the fires, it is surprising that there was not more damage or that failure did not take place earlier.

[7] Note that this is the philosophical position implied by the DTS provisions.
This illustrates that the performance of a building will often be better than that suggested by the fire resistance of the individual elements.

Spalling was noted to have occurred in some cases but did not seem to have been the cause of failure.

There does not appear to be any justification for requiring the systematic use of advanced analysis to assess the behaviour of concrete buildings in fire.

It is noted that all of the buildings mentioned above had low to medium strength concrete. No examples are available of severe fires in buildings with high strength concrete columns.

Section 4

Fire test on concrete building

There have been very few full-scale tests on concrete structures. The only one reported is that conducted at Cardington in the large test enclosure. This test is reported by Bailey. The test structure was a six-storey reinforced concrete building with high-strength concrete columns and reinforced concrete flat slabs. The concrete used for the slabs had 28-day cube strength of 61 MPa. The building was constructed within the large test enclosure at Cardington. The concrete used in the columns had a compressive strength of 103 MPa at 28 days. The columns incorporated 2.7 kg/m³ of polypropylene fibres within the mix. No fibres were added to the concrete mix used for the floors as this was considered not to be a threat with respect to spalling (having a permeability of 300 times greater than the concrete used for the columns). A limestone aggregate (which is known to perform well in fire and be relatively resistant to spalling) was used in the concrete for the columns.

Part of the ground floor of the building was compartmentalised to form a fire enclosure measuring 15 m x 15 m. Details are given in Figure 3. The resulting fire is estimated as having a heat release rate of between 50–85 MW and resulted in very significant spalling of the floor such that the bottom reinforcement was exposed—with some of it being draped towards the floor. The temperatures within the test enclosure were unable to be measured beyond a certain point due to the loss of instrumentation.

Despite the loss of concrete from the floor due to spalling, the floor continued to support the imposed load via the ability of the floor to form compressive forces within the upper part of the slab. This was possible because the edges of the floor were able to sustain the compressive force within the slab. The floor expanded considerably with one of the columns having an outwards lateral residual displacement of 57 mm at the floor location. Bailey suggests that the spalling of the floor slab was exacerbated by the development of compressive stresses within the slab due to expansion of the floor on heating.
The other explanation could be that spalling of the floor was due to the particular aggregate used in the concrete mix for the floor slab. Spalling is a phenomenon that is very difficult to predict. It appears to be most dependent on moisture content and the permeability of the concrete but may also be influenced by the coarse aggregate characteristics. Testing suggests that some aggregates perform better than others when heated. For example, aggregates which have a high coefficient of thermal expansion compared with the cement matrix or which contain compounds that when heated result in rapid expansion and fracture of the individual pieces of aggregate are likely to be less resistant to spalling. Because aggregates vary over the world, no systematic study has been done on the relationship between aggregate type and spalling. No spalling occurred with the high-strength concrete columns.

Figure 3 Cardington test – column deformation and area of burning

Figure 4 Spalling of flat slab with reinforcing bars draped towards floor of test room
5.1 Introduction

It is helpful to consider recent literature on the effects of fire on concrete elements as this is relevant to understanding the potential effects of fire on modern concretes. Recent testing has been fundamentally concerned with considering the strength and spalling behaviour of high-strength concrete under various fire exposure conditions. A summary of the findings are now given and discussed in relation to modern concrete buildings.

Most of the testing that has been undertaken has been based on the standard fire test conditions with testing sometimes being done to other standard heating curves such as one of the ‘hydrocarbon’ heating curves or that used in Europe for the testing of tunnel linings. However, these other heating curves are difficult to reproduce in the laboratory, and in any case, the temperatures achieved by these curves are considered to be significantly higher than those likely to be achieved in most building fires. The initial rate of heating associated with these curves may be more applicable to that which may be achieved in real fire situations compared with the rate of temperature rise associated with the standard fire temperature versus time curve Figure 1, but not the high temperatures associated with either the hydrocarbon or tunnel heating curves.

In two cases reported in the literature, testing was undertaken under real fire conditions.

5.2 Overseas testing

5.2.1 Member behaviour

The issue of spalling under fire conditions and its potential effect on member behaviour has long been recognised. Early codes and standards such as AS 1480 dealt with such issues by requiring members with larger covers to incorporate secondary mesh. Such a provision was based on overseas practice at the time and recommendations from the Experimental Building Station in Sydney. However, these measures resulted in significant corrosion problems for concrete members, and in any case, were found to have little justification as a means of effectively controlling spalling. Subsequent codes such as AS 3600 deleted the need for secondary mesh and incorporated tables based on European recommendations such as those given by FIP/CEB. This resulted in significant increases in covers and member sizes for higher levels of fire resistance. As part of the evaluation of these provisions and prior to their incorporation in AS 3600, a detailed review was undertaken of standard fire tests on concrete columns and compared with the proposed recommendations.

In Reference 16 the results of 172 standard fire tests on concrete columns were reported. Of these tests, spalling was reported for 60 of these tests, with ‘severe’ spalling being reported in about eight cases. In some of these, the tests had to be stopped well before the intended test duration. Ignoring the cases where severe spalling had occurred it was found that the proposed requirements of AS 3600 (based on the FIP/CEB recommendations) gave an adequate representation of performance – irrespective of the presence of light or medium spalling. The range of concrete strengths associated with the tested columns varied from 15 to 45 MPa, which was representative of the strength of concrete being used in most buildings at the time.

Kodur has reported the results of a series of fire tests conducted in normal- and high-strength concrete members in Canada at the fire testing facility at the National Research Council of Canada (NRCC). This facility allows the fire testing of columns under load. The columns were mostly subjected to the standard fire test heating environment. The normal-strength concretes had a compressive strength of around 35 MPa whilst the high-strength concrete had a compressive strength of around 95 MPa. Limestone aggregates were used for some of the columns; a lesser amount of spalling was detected in these columns. In the high-strength concrete columns there was no significant spalling in the first 30 minutes but as the tests proceeded, spalling progressed and became significant just before the end of the test. It resulted in significant loss of cross-section and buckling of the longitudinal reinforcing bars. Spalling was much less pronounced with normal-strength concrete columns. Comparing the behaviour of high-strength columns with and without polypropylene fibres, it was found that the fire resistance was greatly improved with the addition of such fibres. The use of fitments at 135 mm centres was also found to have a significant effect and both measures were recommended.

Jensen et al tested small beams that were both reinforced and prestressed but the number of tests was limited. The aggregate used in the concrete was mostly lightweight although a normal-weight-
aggregate concrete was tested. All beams were tested under hydrocarbon testing conditions. Some beams were passively protected and some incorporated polypropylene fibres. The nominal strength of the concrete tested varied from 50 to 95 MPa. The purpose of the testing was to determine the strength of the beams after fire exposure [8]. The inclusion of polypropylene fibres in the concrete mix was noted as improving the performance.

Aldea et al. report the results of standard fire tests on six short columns made from concrete varying in strength from 30 to 90 MPa. All columns were loaded during the tests to 50% of their design load. Some columns were reinforced with four 25-mm bars and some with eight 12-mm bars. The aggregate used for the concrete mixes was limestone with river sand. Silica fume was added for the 90 MPa specimens. Since all specimens were loaded to the same relative level compared with their design loads, a similar performance in terms of fire resistance time would have been expected. For the 30 and 50 grade concretes, there was only a small amount of spalling and a similar fire resistance was obtained. In the case of the high-strength-concrete columns (grade 90) spalling occurred within 12 minutes with loss of the concrete at the corners. The resulting fire resistance as measured in time to collapse was around 65% of that achieved by the normal-strength-concrete columns. The performance of the column with four bars was worse than that having eight bars, presumably because loss of concrete at the corners resulted in greater heating of the reinforcement.

Ali reports the results of one-half scale columns of both high-strength concrete (nine columns with strength of 106 MPa) and normal-strength concrete (nine columns of 43 MPa). The columns had cross-sections of 127 mm x 127 mm. All specimens were loaded and heated in accordance with the standard time temperature curve. The columns were loaded to 40% of the ultimate design load and in some cases were axially restrained. The latter fact resulted in additional compressive forces in the columns. These additional forces appear to be around 20% of the design load. The moisture content in the columns varied from 5–6% by weight. Some of the columns were restrained so that the axial force increased with heating. Virtually all of the columns spalled and it was concluded that it was difficult to determine whether the spalling behaviour was worse (or better) for HSC than for NSC. No details of aggregate type or mix were given.

Lennon discusses the poor perception of the behaviour of precast hollowcore slabs [9] in fire and reports the results of two full-scale fire tests undertaken to assess the behaviour. The issue of spalling was apparently raised regarding the performance of such slabs under fire conditions. These concerns arose from two previous tests conducted at the BRE’s Cardington facility. The slabs tested had been designed to achieve a two-hour fire resistance. Spalling occurred within 20 minutes of the start of the test which had to be stopped at 40 minutes due to extensive cracking of the slab and with the bottom reinforcement completely exposed. The explanation for this behaviour was that insufficient time had been allowed for curing and that the moisture content was too high. The testing reported by Lennon was aimed at repeating the fire tests but with specimens having lower moisture content. The two specimens were constructed from concrete having a cube strength of around 43 MPa (ie not high-strength concrete). The specimens were reinforced, not prestressed. The floor slabs were placed on top of a fire enclosure of 6 m x 6 m plan dimensions being one way continuous slabs supported by a beam at midspan location. Details of the test specimen curing are not specified. No significant spalling occurred in either test. The conclusion appears to be that the previous test specimens must have had too high a moisture content. There was no sign of shear failure. This is a reasonable conclusion only if it can be demonstrated that shortly after construction the moisture content reduces significantly to values that are not expected to give rise to spalling.

As far as prestressed floors are concerned, there is some information on which to make a judgement regarding their likely behaviour in real fires. These slabs will normally be constructed from NSC. The application of heat to the bottom of any slab causes compressive stresses to develop at the heated surface [10]. In the case of prestressed floors, greater compressive stress will exist at the heated surface but not significantly so. It has been postulated in the literature that the presence of compressive strains at the heated surface inhibits the opening of cracks to relieve internal pressure due to heated moisture. Restraint of overall expansion of the heated slab will induce additional compressive stresses.

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[6] The strength after heating and cooling is known as the residual strength.

[9] Apparently, previous standard fire tests of such slabs in Europe showed problems with respect to premature shear failure.

[10] Since shear deformations in slabs are negligible, it is reasonable to assume that plane sections before deformation will remain plane after deformation. A non-linear thermal gradient within the cross-section will require the development of a non-linear stress state to ensure that plane sections remain plane. Such stresses will result in zero axial force and zero bending moment.
Ellobody and Bailey\textsuperscript{22} have presented the results of 12 standard fire tests on unbonded and bonded post-tensioned one-way concrete slabs. The key variables considered were the type of aggregate (limestone and Thames River Gravel), whether the tendons were bonded or unbonded, and whether the slabs were longitudinally restrained or not. The slabs were 160 mm thick, 1.6 m wide and 4.3 m long. The concrete strengths varied from 36 to 48 MPa and each slab incorporated three multi-wire tendons – one at the centre and one at 530 mm from each side. The distance from the outside of the slab to the centre of each tendon at midspan (ie the axis distance) was 42 mm since the tendons were parabolic. The slabs were loaded to approximately 50% of their ultimate load as determined by testing of similar slabs under ambient conditions. No significant spalling resulted during these tests including the six tests undertaken on slabs with Thames River Gravel. The performance of the slabs indicated that the tabulated values of prEN 1992-1-2\textsuperscript{23} for simply-supported slabs underestimated the actual fire resistance. The testing was used as part of the verification of a finite element model developed to better understand the behaviour of prestressed slabs in fire.

5.2.2 Material behaviour

Strength and deformation models

Normal-strength concrete

The strength and behaviour of materials under elevated temperature conditions is not determined by subjecting the specimens to actual fire conditions but by increasing the temperature of the specimen using some form of heating (e.g., electrical resistance heating) such that the specimen is soaked at a given temperature so as to achieve close to uniform temperature conditions throughout the specimen. Material testing of concrete under fire conditions is normally undertaken using small cylinders that are heated and loaded under various conditions. Based on such testing, various complex models have been developed such as those proposed by Anderberg and Thelandersson\textsuperscript{24} and Schneider\textsuperscript{25}. Application of heating over time results in creep (time-dependent deformation) although in typical fire situations this is limited due to the relatively short time at which concrete is at elevated temperature. The effect of creep is to increase the strain associated with any level of stress. Stress-strain relationships which incorporate a level of creep implicitly have been developed for normal-weight concrete\textsuperscript{23} and give a satisfactory basis for calculation. These are shown in Figure 5.

High-strength concrete

A literature review of the early research (up to 1996) into the effect of elevated temperature on the compressive strength of high-strength concrete is presented by Phan\textsuperscript{26}. The following observations were made:

- Testing of high-strength concrete (HSC) has found that the compressive strengths obtained from stressed tests\textsuperscript{[11]} are higher than those associated with unstressed or unstressed residual tests – at least for temperatures up to 400°C.
- HSC where the mix contained silica fume was found to give rise to a greater loss of strength for increasing temperature.
- AS 3600 provisions for NSC are not applicable to HSC.
- Phan\textsuperscript{27} notes that the reduction of strength of HSC when heated under stress is around 30% from 100–400°C and then is similar to that for NSC.

Detailed test results for the compressive strength and modulus of elasticity are given for high-strength concretes in Phan and Carino\textsuperscript{28}. These tests were undertaken as part of the NIST investigation into the behaviour of higher strength concrete in fire and

\[\text{Figure 5 Stress-strain curves for normal-strength concrete at various temperatures}\]

\(\text{[11]}\) Stressed tests refer to tests where the test specimen is loaded during heating (as in real structures) whereas unstressed tests are where the specimen is heated and then loaded to failure. Residual tests are where the specimen is heated, then cooled, and then loaded to failure. For normal-strength concrete, the application of load during heating has been found to improve the concrete strength.
considered the behaviour of concretes varying in strength from 47 to 98 MPa. Figure 6 shows the results of these and other investigations into the effect of heating on the strength of HSC. The results shown in Figure 6 are for specimens not stressed during heating while Figure 7 shows the results for specimens stressed during heating. As with NSC, the presence of stress during heating appears to improve the strength under elevated temperature conditions. The relative strength for normal strength concrete (prEN 1992-1-2) is plotted in both of these figures to allow a comparison between NSC and HSC. Cheng et al. have reported the stress-strain curves for HSC at elevated temperature and noted that, except for concrete at high temperatures, it is very difficult to determine the unloading path (which appears to be very steep). Stress-strain curves ‘fitted’ to the results obtained by Cheng are shown in Figure 8. A comparison of Figures 5 and 8 demonstrates the much lower deformation capacity of HSC compared with NSC.

**Resistance to spalling**

The resistance to spalling is normally studied by subjecting small or larger specimens to heating in a furnace which is constrained to follow a particular gas temperature versus time curve such as that shown in Figure 1 or perhaps a more severe curve in terms of rate of temperature rise or furnace temperature achieved.

Phan in reviewing the early test data found that:

- Explosive spalling failure may occur more in HSC than in NSC – although this was not found in all test programmes (approx 50%). Where spalling occurred, it was found to occur in stressed, unstressed and residual tests. In test programmes where spalling occurred, it did not occur for every specimen – indeed, sometimes with nominally identical specimens tested under nominally identical conditions one would spall while another would not.
- Concrete with dense pastes due to the addition of silica fume are more susceptible to explosive
spalling. The behaviour was noted as being more likely in large specimens than small specimens and if the heating rate was higher.

- The type of aggregate also has an effect on the tendency to spall.

Phan et al. reported the findings of an International Workshop on the Fire Performance of HSC and observed that fire tests in Germany (TU Braunschweig) have demonstrated that secondary steel within the cover of columns does not prevent spalling. It was noted that the addition of polypropylene fibres (citing the work of Conoco) to the mix appeared to reduce the incidence of spalling.

Phan et al. summarised the results of the NIST test program on HSC. These tests were undertaken using 100-mm x 200-mm cylinders. The concrete mix varied but for all mixes limestone aggregates were used as the coarse aggregate with river sand as the fine aggregate. Various quantities of silica fume were added to the mixes as cement replacement. The strengths of the mixes varied from 47 to 100 MPa at 400 days. Various additions of polypropylene fibres were considered (0, 1.5 kg/m³ and 3 kg/m³) and heating rates varied from 5°C/min to 25°C/min as far as the air temperature within the heating furnace was concerned.

The key outcomes from this work were:

- Explosive spalling was considered to be primarily a function of the build-up of water pressure within the matrix but also possibly due to the effect of differential thermal expansion within the matrix.
- The tendency to spall was found to increase as the water-cement ratio was reduced. This was associated with decreased permeability.
- The addition of up to 1.5 kg/m³ of polypropylene fibres to the mixes appeared to be effective in relieving steam pressure and avoiding spalling.
- The presence of significant compressive load during heating appeared to reduce the extent of spalling.

Bostrom et al. present the results of an evaluation of spalling behaviour for concretes using different mixes and specimens. The work was undertaken to consider both the use of concrete in buildings and for tunnel linings. For concretes evaluated for tunnel linings, specimens were subject to a severe fire test considered to represent a tunnel fire. Other specimens were subject to the standard time temperature curve identical to that specified by AS 1530.4 and referenced by the BCA. The cube strengths of the concretes tested varied from 73 to 107 MPa. For specimens tested under standard fire test conditions, spalling was common except where polypropylene fibres were incorporated. In this case, the test specimens were small slabs and long cylinders. However, even with polypropylene fibres, spalling occurred with specimens subjected to the more severe heating curve. However, the amount of spalling was much less than that associated with specimens without fibres.

It was found that the likelihood of spalling is increased if the specimen is subjected to compression as it was postulated that this limits the formation of cracks on the exposed face. It is noted that this differs from the findings of Phan (see Section 5.2.2 Resistance to spalling) who considered that the application of compressive stress reduced spalling.

As far as the effect of water within the concrete matrix is concerned and the associated development of pore pressures leading to explosive spalling, a good description is given by Flynn. The following is a verbatim quote from this publication:

‘Consider a concrete slab that initially may have small temperature gradients, eg due to indoor-to-outdoor temperature differences. Further, the extent of hydration of the concrete may not be uniform throughout the structure. Because of the heat released during initial curing and drying out of the concrete near one or both surfaces, there may be a higher amount of hydration in the middle of the material than near the surfaces. The free moisture content may also vary throughout the concrete if the different surfaces have been exposed to different levels of humidity.

‘At the beginning of a fire, the temperature of the exposed side of the concrete slab will rise rapidly. Free moisture, both liquid and vapour, will migrate towards the cold side of the concrete. Initially this moisture movement occurs by diffusion processes, where the driving force may be considered to be the gradient in moisture content. As the temperature of the fire-exposed side increases, any free liquid water will boil off and migrate towards the colder side where some of it will condense. The latent heat required to boil the liquid water will retard the rate of temperature rise at that location. When water vapour is transported into a colder region, some of it is absorbed into the concrete, with the heat of sorption being approximately equal to the latent heat associated with condensation of free water vapour into liquid, so that

[12] The performance of limestone aggregate in fire is better from a spalling viewpoint probably because the heating of limestone results in heat being absorbed and therefore it is less sensitive to potential thermal gradients.

[13] ‘Explosive spalling’ is a term that refers to the sudden and intermittent loss of concrete accompanied by an explosive sound.
significant heat is released. As moisture moves into the slab and the interior temperature rises towards 100°C, portions of the slab may experience additional hydration (conversion of free water to chemically bound water), with the attendant release of heat. When the temperature of any portion of the concrete slab exceeds the boiling point of water some dehydration (release of chemically bound water) will begin to take place, with an attendant absorption of heat. The dehydration reaction continues to temperatures in excess of 800°C, with the pronounced reaction being the dehydration of Ca(OH)₂ between 400°C and 600°C. The free water introduced into the concrete tries to diffuse toward the cold side. However, high strength concrete is not very permeable to water vapour and is even less permeable to liquid water. Thus the moisture cannot escape as rapidly as it is being released and the pore pressure in the concrete will rise substantially.

5.2.3 Summary of findings

The following are apparent findings from the reported testing of members and concrete under fire conditions:

- The relative strength of HSC under elevated temperature conditions is reduced to a greater extent than that associated with concrete with a compressive strength of around 40 MPa. However, this is the case only for temperatures up to 400°C. For higher temperatures, the proportion of strength left appears to be similar to that for NSC.

- Of greater significance is that it appears that HSC may have a greater tendency to spall than NSC and that this is due to the concrete matrix having a much lower permeability. Loss of concrete section due to spalling is likely to result in a sudden and significant reduction in loadbearing capacity. Testing to date has looked primarily at concretes having strengths of up to 100 MPa at the time of test.

- Other factors that may exacerbate spalling are the rate of temperature rise to which the concrete surface is exposed and the aggregate characteristics.

- The likelihood of spalling of HSC can be minimised by the addition of polypropylene fibres to the concrete mix. It is thought that these fibres melt, allowing moisture to more readily escape from the heated surface. Other measures include the provision of external render that will provide a buffer against high temperature gradients but this measure is unlikely to be cost effective.

- Spalling of NSC floor slabs has also occurred in real fire tests but the coarse aggregate chosen for these tests (ie Thames River Gravel) has a history of spalling in fire. Excessive moisture content has also been advocated in the related literature as a potential cause. However, the moisture content will always be high immediately following construction and there may not be an alternative choice of aggregate.

5.3 Recent testing of Australian concretes

5.3.1 Introduction

The testing of HSC columns and post-tensioned slabs described above has been conducted in overseas test laboratories using overseas aggregates. Common aggregates used in Australia include basalt in Victoria and river gravels such as Nepean River gravel in New South Wales.

The purpose of the testing described in this section was to gain a better understanding of the performance in fire of post-tensioned slabs and high strength concrete columns made from concrete using common Australian aggregates. This testing was principally aimed at assessing the extent, if any, of spalling associated with such members under simulated real fire conditions and at providing some guidance on the effectiveness of various measures to limit spalling or limit the effects of spalling.

The purpose of the testing described in this section was to subject a combination of stressed columns and post-tensioned slabs to severe real fire conditions. This was considered to be a more efficient and representative way to test multiple elements rather than conducting multiple fire tests in a standard fire test environment. The details of the specimens and the design of the test situation are described below.

5.3.2 Test set-up

Test specimens

Due to the cost associated with undertaking large-scale fire testing it was decided to include a number of variables were included in the one test set-up. In particular, both high strength concrete columns and post-tensioned slabs constructed from both Nepean River gravel and basalt aggregate were tested. In addition, for the columns, various measures were adopted to reduce the incidence of spalling or limit its effects, including the use of polypropylene fibres and closer spacing of column fitments. The strength characteristics of the various mixes are summarised in Table 1.

Table 2 summarises the various test specimens and the variables covered. In all, twelve column specimens and three post-tensioned slabs were constructed. Six of the columns and one slab were made from concrete using Nepean River gravel whereas six columns and two slabs were made from concretes using a basalt aggregate from Deer Park in Victoria.
All of the columns were constructed of HSC whilst the slabs were constructed of NSC. Four of the columns contained polypropylene fibres but no fibres were added to any of the concrete mixes for the slabs. It should be noted that four of the columns have ties at a closer spacing of 120 mm to see whether closer tie spacing would minimise the effects of spalling.

Each column contained normal reinforcement and a high strength micro-alloyed (‘Macalloy’) bar that was tensioned approximately one month after casting. All slabs contained some nominal reinforcement to enable handling after casting and a centrally placed flat duct containing three 12.7 mm strands. The strands were tensioned approximately one month after casting.

The reinforcing details for the columns and slabs are shown in Figure 9. Four mixes of concrete were supplied.

### TABLE 1 Concrete strengths at maximum age of (cylinder) testing

<table>
<thead>
<tr>
<th>Concrete mix</th>
<th>Age (days)</th>
<th>Strength (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>HSC (Basalt)</td>
<td>56</td>
<td>96</td>
</tr>
<tr>
<td>NSC (Basalt)</td>
<td>29</td>
<td>34</td>
</tr>
<tr>
<td>HSC (Nepean river gravel)</td>
<td>56</td>
<td>89</td>
</tr>
<tr>
<td>NSC (Nepean river gravel)</td>
<td>28</td>
<td>43.5</td>
</tr>
</tbody>
</table>

### TABLE 2 Detail of test specimens

<table>
<thead>
<tr>
<th>Ref no.</th>
<th>Aggregate type</th>
<th>Concrete strength</th>
<th>Polypropylene fibres</th>
<th>Fitment spacing (mm)</th>
<th>Cross-sectional dimensions (mm)</th>
<th>Length (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>C1</td>
<td>basalt</td>
<td>high</td>
<td>no</td>
<td>200</td>
<td>350 x 350</td>
<td>3000</td>
</tr>
<tr>
<td>C2</td>
<td>basalt</td>
<td>high</td>
<td>no</td>
<td>200</td>
<td>350 x 350</td>
<td>3000</td>
</tr>
<tr>
<td>C3</td>
<td>basalt</td>
<td>high</td>
<td>no</td>
<td>120</td>
<td>350 x 350</td>
<td>3000</td>
</tr>
<tr>
<td>C4</td>
<td>basalt</td>
<td>high</td>
<td>no</td>
<td>120</td>
<td>350 x 350</td>
<td>3000</td>
</tr>
<tr>
<td>C5</td>
<td>basalt</td>
<td>high</td>
<td>yes</td>
<td>200</td>
<td>350 x 350</td>
<td>3000</td>
</tr>
<tr>
<td>C6</td>
<td>basalt</td>
<td>high</td>
<td>yes</td>
<td>200</td>
<td>350 x 350</td>
<td>3000</td>
</tr>
<tr>
<td>C7</td>
<td>Nepean</td>
<td>high</td>
<td>no</td>
<td>200</td>
<td>350 x 350</td>
<td>3000</td>
</tr>
<tr>
<td>C8</td>
<td>Nepean</td>
<td>high</td>
<td>no</td>
<td>200</td>
<td>350 x 350</td>
<td>3000</td>
</tr>
<tr>
<td>C9</td>
<td>Nepean</td>
<td>high</td>
<td>no</td>
<td>120</td>
<td>350 x 350</td>
<td>3000</td>
</tr>
<tr>
<td>C10</td>
<td>Nepean</td>
<td>high</td>
<td>no</td>
<td>120</td>
<td>350 x 350</td>
<td>3000</td>
</tr>
<tr>
<td>C11</td>
<td>Nepean</td>
<td>high</td>
<td>yes</td>
<td>200</td>
<td>350 x 350</td>
<td>3000</td>
</tr>
<tr>
<td>C12</td>
<td>Nepean</td>
<td>high</td>
<td>yes</td>
<td>200</td>
<td>350 x 350</td>
<td>3000</td>
</tr>
<tr>
<td>S1</td>
<td>basalt</td>
<td>normal</td>
<td>no</td>
<td>400</td>
<td>1000 x 150</td>
<td>6000</td>
</tr>
<tr>
<td>S2</td>
<td>basalt</td>
<td>normal</td>
<td>no</td>
<td>400</td>
<td>1000 x 150</td>
<td>6000</td>
</tr>
<tr>
<td>S3</td>
<td>Nepean</td>
<td>normal</td>
<td>no</td>
<td>400</td>
<td>1000 x 150</td>
<td>6000</td>
</tr>
</tbody>
</table>
On each day of casting, the four columns without polypropylene fibres were cast first. Polypropylene fibres were then added to the mixing truck at a dosage rate of 1 kg/m³ of concrete and then the concrete was mixed for 10 minutes prior to casting the last two columns. The fibres added to the mix were 6-mm monofilament polypropylene fibres. No fibres were added to the normal strength concrete mixes for the slabs.

Views of the columns and slabs prior to, and following pouring are given in Figures 9 and 10.

Stressing was undertaken one month after casting. The resultant tensioning force in the case of the Macalloy bars was about 900 kN whilst in the case of the strands was about 156 kN per strand. Allowing for the hole through the centre of each column specimen for the prestressing bar and for the presence of the four reinforcing bars, the prestressing resulted in an axial stress some distance away from the ends of almost 7 MPa. Similarly, away from the ends of the slabs, the compressive stress induced by the prestressing was about 3 MPa.

Test enclosure

The fire test enclosure was constructed at the CESARE Large Fire Test Facility at Fiskville Figure 11. The building is 70 m long, 40 m wide and 22 m high at the ridge. It contains roof and wall vents and has been designed to allow the conduct of significant fire tests in a controlled environment.

The test enclosure in which the columns were placed is shown in Figure 12 and the positions of the columns in Figure 13. The test enclosure consisted of symmetrical ventilation openings on two sides but also included some low level openings on the other sides. The internal dimensions of the enclosure were 4.25 m x 5.4 m x 3.3 m high.

It was possible to reduce the height of the openings during the test by means of external dampers as shown in Figure 12. This was done to give some control over burning rate during the test. The test enclosure was clad with two layers of fire-resistant plasterboard and clad with a single layer of 25 mm ceramic fibre insulation blanket. The three post-tensioned slabs formed part of the roof to the enclosure with block-off panels at each end Figure 14.

The fuel used for the test consisted of 20 wood cribs and represented a very high fire load density of 124 kg/m² of wood per square metre of floor area. This level of fire load was chosen to ensure a long test duration. A typical wood crib is shown in Figure 15.

Apart from video cameras, the instrumentation consisted of thermocouples to measure air temperatures within the enclosure. These were located in two layers – 400 mm and 1500 mm below the soffit.

Figure 10 Casting of slabs and columns

Figure 11 Exterior of CESARE large fire test facility
Figure 12 Test enclosure in which columns were placed

Figure 13 Plan view showing location of columns in enclosure; orientation for all columns

Figure 14 Placement of post-tensioned slabs over enclosure

Figure 15 Typical wood crib

Figure 16 Shortly after start of test
Description of test

The fire was initiated by igniting the methylated spirits in each of the aluminium trays at the bottom of the cribs by remote electrical ignition. A view of the burning cribs shortly after ignition is shown in Figure 16 and of later external flaming in Figure 17.

The measured fire temperature versus time relationship is shown in Figure 18. The temperatures at both levels were similar and showed significant variability.

Also shown in this figure is the standard time-temperature curve that was described earlier. It can be seen that at various locations and times, the temperatures exceed the standard time temperature curve but also that at many locations, they were lower. Despite the fact that every measure was taken to ensure uniformity of temperature within the enclosure, such a situation cannot be realised. As the test progressed the temperatures increased. The test was stopped around 70 minutes after initiation of the fire.

5.3.3 Test findings

All of the columns except for those with polypropylene fibres suffered significant spalling. The comparison in behaviour is illustrated in Figure 19 which shows column C10 (no fibre) and column C12 (fibre). Generally the maximum depth of spalling for the columns without fibre was to the rear of the fitments with there being little difference in the depth of spalling between columns made from basalt aggregate concrete and those made from concrete utilising Nepean River gravel.

In the case of the columns made from concrete with polypropylene added, the column that behaved best was C12 with no loss of concrete whilst the other columns (C5, C6, C11) lost concrete at particular locations from one corner (C5, C11) or two (C6) Figure 20. However, such spalling was localised and it is considered that such occurrences could be eliminated by adding a slightly greater mass of fibres into the mix and ensuring good mixing and good workability during casting.

![Figure 17 Significant external flaming](image)

![Figure 18 Individual thermocouple readings at lower level within the enclosure](image)

![Figure 19 Comparison of columns with and without fibre](image)

- [a] C10 top
- [b] C12 top
- [c] C10 bottom
- [d] C12 bottom
As far as the slabs were concerned, Slab S2 exhibited no spalling at all Figure 21 whereas Slab S1 exhibited spalling at the ends Figure 22 but not elsewhere. The spalling at the east end of the slab measured approximately 750 mm x 600 mm in area and was 30 mm deep whilst that at the other end was slightly less in area but 40 mm deep.

Figure 22 shows Slab S3 after the test and the associated extensive spalling. In this case the reinforcement and prestressing duct were completely exposed with the depth of spalling exceeding one half the depth of the slab. The exposed surface of the slab suggested that most of the exposed surface had experienced a ‘pitting’ type spalling and this corresponded to a staccato spalling noise heard during the testing.
Kelly and Purkiss\textsuperscript{34} noted the advent of severe spalling with a post-tensioned slab containing Thames River Gravel – a common river gravel aggregate in the UK. Similar behaviour was demonstrated by Slab S3. It is interesting to note that the test reported by Bailey\textsuperscript{13} showed similar behaviour for the slabs which were also constructed from Thames River Gravel Figure 4. In that case the slab was reinforced rather than prestressed and the columns, which were also constructed from Thames River Gravel, incorporated polypropylene fibres. The floor slab had a cube strength of 61 MPa whilst the columns had a cube strength at 28 days of 103 MPa. The floor spalled extensively but the columns did not experience spalling. This behaviour is consistent with that observed in the test described in this Guide. One would expect that the addition of polypropylene fibres to the slab concrete mix would overcome the spalling problem. However, in contrast to this, the post-tensioned slabs tested by Ellobody and Bailey\textsuperscript{22} did not exhibit severe spalling even though six of them contained Thames River gravel.

5.3.4 Conclusions

- The addition of fibres to the high strength mix had a dramatic effect in reducing the level of spalling. This is consistent with overseas test data. It would appear that the dosage rate should be increased above 1 kg/m\textsuperscript{3} to provide a higher concentration of fibres within the mix but so as not to further reduce workability. It is considered that a dosage rate of 1.2 kg/m\textsuperscript{3} is appropriate in this regard. The workability of the mix should be such as to facilitate good compaction.

- The placement of column fitments at closer spacing did not reduce the level of spalling.

- The post-tensioned slab containing Nepean River gravel spalled badly and consideration should be given to incorporating polypropylene fibres in concrete mixes for this type of construction.

- One of the slabs constructed from basalt aggregate exhibited no spalling whereas the other slab, a nominally identical construction, spalled at each end of the slab. No reason is offered for the difference in behaviour. Again, it is expected that the addition of polypropylene fibres to the concrete mix would have eliminated the spalling at the ends of Slab S1.

Section 6
Conclusions to Part A

Based on a review of the accounts of major fires in concrete buildings and the literature and research findings presented in Sections 4 and 5, the following conclusions are drawn regarding the effect of fire on concrete, concrete elements and concrete buildings:

- Serious fires in buildings are rare and there have been few cases of significant building failure in concrete buildings. Such fires should be avoided since rapid fire spread and intense smoke will pose a significant threat for the building occupants. The use of other fire safety measures required by building regulations (eg sprinklers) has an important role in restricting the development of such fires. None of the buildings considered in Part A was fitted with a sprinkler system.

- The behaviour and resistance of concrete under elevated temperature conditions is complex. Sufficient testing has, however, been undertaken to provide the basis for describing the reduction in strength with temperature and simplified stress-strain relationships that can be used for design or advanced analysis.

- The possibility of spalling has been long recognised in relation to both normal-strength and high-strength concrete, with the likelihood of spalling increasing as the permeability of the concrete reduces. This is usually the case for high strength concrete although there is some beneficial increase in the tensile strength of the concrete.

- The addition of polypropylene fibres to the high-strength mix has a dramatic effect in reducing the level of spalling. This is consistent with overseas test data. A dosage rate of 1.2 kg/m\textsuperscript{3} of 6-mm monofilament polypropylene fibres is recommended.

- It is considered that the adoption of such measures for slabs which may be more prone to spalling (eg stressed slabs with river gravel aggregates) will provide an appropriate measure to minimise such behaviour.
7.1 Overview

The conceptual framework of the BCA is one where those involved in the design and certification of the building must demonstrate conformity with the performance requirements. This is true for all aspects of the design of a building – not just those associated with fire safety.

The performance requirements, when studied closely, are statements of a qualitative nature and give no indication of the level of safety actually required. For example, performance requirement BP1.1 in Part B1 Structural Provisions states:

- **A building or structure, to the degree necessary, must** –
  - remain stable and not collapse; and
  - prevent progressive collapse; and
  - minimise localised damage and loss of amenity through excessive deformation, vibration or degradation; and
  - avoid causing damage to other properties, by resisting the actions to which it may reasonably be subjected.

This part of the BCA is concerned with the design of the building such that under non-fire conditions the building will be sufficiently safe against collapse and adequately serviceable for the occupants. Part B1 recognises that the performance requirements can be achieved by satisfying the DTS (Deemed-to-Satisfy) provisions or by developing an appropriate **Alternative Solution**, as shown in Figure 24. The DTS provisions in this part of the BCA reference Australian Standards including those related to loads (e.g., AS 1170.0\(^35\), AS 1170.1\(^36\), AS 1170.3\(^37\)) and resistances for various forms of construction (e.g., AS 4100\(^38\), AS 3600).

Satisfying the DTS provisions (and therefore the performance requirements) in relation to the normal temperature design of the building structure is achieved by designing the building to the set of standards that are referenced within the relevant part of the BCA (in this case Part B1) – and this implicitly results in the building elements being designed such that the risk of failure is considered to be sufficiently small. In Australia, loading and structural codes are considered to give rise to an average probability of failure of an element, over the life of the building, of about \(1 \times 10^{-3}\) for elements such as beams and \(2 \times 10^{-4}\) for columns\(^39\). It is difficult to avoid the inference that any Alternative Solution (i.e., one that is not based on the loading and materials standards referenced by the DTS provisions of the BCA) must achieve probabilities of failure that are at least as low as those given by the above numbers, i.e., the risk of failure should be no greater.

Wind and earthquakes can be considered to be ‘natural’ phenomena over which designers have no control except perhaps to choose the location of buildings more carefully on the basis of historical records and to design building to resist sufficiently high loads or accelerations for the particular location. Permanent and variable actions in buildings are the result of gravity. All of these loads are variable and it is possible (although generally unlikely) that the actions may exceed the resistance of the critical structural members resulting in structural failure.

The nature and influence of fires in buildings are quite different to those associated with other ‘loads’ to which a building may be subject. In most situations (ignoring bush fires), fire originates from human activities within the building or is due to the malfunction of equipment used in the building. It follows that it is possible to influence the rate of fire starts by influencing human behaviour, limiting and monitoring human activities and improving the design of equipment and its maintenance. This is not the case for the usual loads applied to a building.

In the case of a fire start, there are many factors that can be brought to bear to influence the ultimate size of the fire and its effect within the building. For example, it is known that occupants within a building will often detect a fire and deal with it before it reaches a significant size. It is estimated that less than one fire in five\(^8\) results in a call to the fire brigade, while the majority of fires reported to the fire brigade, while the majority of fires reported to the fire brigade, will...
be limited to the room of fire origin by the occupant or early fire brigade intervention. In occupied spaces, olfactory cues (smell) provide powerful evidence of the presence of even a small fire. The addition of a functional smoke detection system will further improve the likelihood of detection and of action being taken by the occupants. Fire-fighting equipment, such as extinguishers and hose reels, is generally provided within buildings for the use of occupants and many organisations provide training for staff in respect of their use. The growth of a fire can also be limited by automatic extinguishing systems such as sprinklers, which can be designed to have high levels of effectiveness. Fires can also be limited by the fire brigade depending on the size and location of the fire at the time of arrival. The point is that all of these factors can have a direct influence on whether a severe fire is likely to develop within a building and, therefore, the probability of failure of a structural member in a particular part of the building.

Moreover, time is an important element when considering fire safety since many aspects relevant to fire safety are a function of time – loss of strength of structural members, time for smoke and heat to spread, and time for the evacuation of occupants.

7.2 Fire safety

Approximately 70% of the BCA is related to fire-safety matters with performance requirements given in Part C (fire resistance), Part D (Access and Egress) and Part E (Services and Equipment). The fire safety objectives of the BCA can be summarised as follows with respect to the building design:

- Allow safe evacuation of the occupants
- Not to put the fire brigade at risk in the exercise of their duty
- Avoid the spread of fire to other buildings
- Avoid damage to other buildings.

It should be noted that the above objectives are concerned with occupant safety and preventing damage to adjacent property. They are not directly concerned with protection of the property in which the fire is initiated. Other objectives, such as property protection, may be requested by the building owner but these are not provided by the BCA provisions.

Once again, the performance requirements given in Parts C (fire resistance), D (egress) and E (services and equipment) of the BCA are qualitative statements that can be achieved by satisfying the DTS provisions of the BCA or by developing an appropriate Alternative Solution. In the case of Parts C, D and E, many DTS provisions are specified. For example, in Part C, required fire-resistance levels (FRLs) are specified for various building elements (floors, walls, beams, columns, etc) depending on the building situation (height, class of building and area) whilst the achievement of these fire-resistance levels is deemed to be satisfied if the building is designed in accordance with AS 3600 (or more particularly, Section 5 of AS 3600).

7.3 Alternative solutions and fire engineering

Fire engineering design is the process of applying engineering principles to develop a building solution (ie an Alternative Solution) which differs from that complying with the DTS provisions for fire safety. The development of such an Alternative Solution may not only be concerned with the structure of the building but will often consider all aspects affecting fire safety within the building and may propose a design that is, at many points, at variance with many of the DTS requirements. The Alternative Solution may address some or all of the following aspects:

- Control of smoke movement within the building
- Human behaviour and evacuation
- Aspects of fire suppression and detection
- Compartmentation or lack thereof
- The effect of the fire on the resistance of the building structure.

The fire engineer must demonstrate and state that the relevant BCA performance requirements will be met by the proposed design.

Given that the BCA provides no guidance an acceptable level of risk, and that such guidance is usually not given by the Building Certifier, it is most often concluded that the level of safety associated with an Alternative Solution must be similar to that associated with a design complying with the DTS provisions. Considering only the aspect of the resistance of the structure to fire (should this be important) it is tempting to suggest that the probabilities of failure should be the same as for normal temperature structural design. This would enable all of the factors that impact the initiation or growth of the fire (eg occupant intervention, sprinklers) to be taken into account. Variability in applied loads and the quantity of combustibles (termed ‘fire load’) should also be taken into account. This approach has rarely been used but the concept is further discussed elsewhere.
From a practical viewpoint, the following options are utilised by fire engineers in attempting to justify an Alternative Solution:

1. Demonstrate that the particular DTS provision that is NOT being complied with is not required to the extent specified because the presumed fire or other conditions associated with the particular provision is sufficiently unlikely to occur in the situation. This is quite possible since the DTS provisions are meant to cover a very wide range of situations and these might not occur for the particular building being considered. For example, it is possible that in a particular situation, the fire load, ventilation, etc is such that it is not possible to achieve a fire severity that is anything like that associated with the FRL (Fire Resistance Level) required by the DTS provisions.

2. Demonstrate that the absence of the particular DTS measure for the particular situation being considered will have no significant effect on the achievement of the fire safety objectives (and therefore the performance requirements) of the BCA. An example is a structural member which, if it collapsed in fire, would have no effect on the overall stability of the building or its ability to resist fire – e.g., bracing members designed to resist extreme winds.

3. Recognise ADDITIONAL features that have been incorporated into the design so as to compensate for the absence of measures required by the DTS provisions. For example, the incorporation of sprinklers within a building not requiring sprinklers will compensate for reduction in other measures such as fire-resistance levels. However, the justification of this approach will require some quantification to demonstrate that the new measures will more completely achieve the fire safety objectives than the omitted measure.

4. Recognise improvements in the EFFECTIVENESS of some of the fire-safety measures within the building that have been incorporated to compensate for a reduction in other measures – e.g., improvements in the reliability of the sprinkler system by adding additional features over and above those required by the DTS provisions.

5. Replace the particular DTS requirement with a different measure that achieves the same outcome.

The details of the assessment approaches used for each of these Alternative Solutions will differ widely and may involve a combination of qualitative argument and quantitative analysis. The fire engineering design needs to demonstrate/argue that the performance achieved is equivalent to that associated with the DTS-compliant design or other nominated safety level. In effect, it is necessary to demonstrate that the risk level is no greater for the Alternative Solution than for that associated with a ‘benchmark’ situation. If the benchmark is not the level of safety associated with a DTS compliant design, then it must be agreed and set by the stakeholders involved with the project. The relevant assessment must be undertaken by a competent fire engineer.

It should be noted that fire engineering is often about matters other than the building structure and its fire resistance and is undertaken for buildings irrespective of the material of construction. However, in some cases, fire engineering is undertaken to permit the structure to have a lesser fire resistance than that required by the DTS provisions.
Section 8

Fire engineering and concrete structures

8.1 Is there a need for this?

In most situations, designers of concrete structures will mostly adopt the DTS provisions of the BCA and design the elements of the building to meet the nominated FRLs by ensuring that the member dimensions and axis distances comply with the values given in AS 3600. However, there are some situations where adopting the DTS provisions and the associated tabulated values of AS 3600 will impose significant economic penalties or their adoption may result in a conflict with satisfying energy efficiency for the building. The design of buildings for fire safety and for energy efficiency may involve conflicts especially when considering the trend towards more open structures rather than highly compartmentalised buildings and the use of lighter rather than heavier members.

The change of use of buildings often means that existing buildings need to be assessed with respect to their suitability for a different use (eg retail occupancy with an office building) to upgrade the building to achieve the higher fire-resistance levels required by the DTS provisions may require very considerable expenditure.

Concrete structures in fire – performance, design and analysis provides guidance on the assessment of concrete structures in fire from first principles and from the perspective of European/UK design codes.

8.2 Determining fire resistance requirements

The fire engineer will need to assess a building and its fire safety measures and determine whether a reduced fire resistance can be justified. Assuming that it can, the outcome could be expressed in terms of a REDUCED FRL (often the case) or by means of a fire time versus temperature relationship for a representative ‘design fire’. Such a time-temperature relationship can be expressed as an initial steep rise in temperature, plateau to a constant value and a gradual decay of temperature with time. This is an approximation to the fire temperature curves associated with experimental real enclosure fires. Such a ‘real’ fire curve (ie a Design Fire Curve) will need to be determined by the fire engineer. Such curves may also be expressed as a temperature relationship that varies with time but this is unlikely to lead to markedly different temperatures within the concrete elements exposed to fire. In this publication, the simplified ‘real’ fire temperature versus time curve has been adopted.

The difference in these two ways of specifying the fire to be resisted is explained as follows. A fire that is expressed as an FRL refers to a fire exposure that is represented by the fire temperature versus time curve shown in Figure 25 as Curve 1. This curve is the standard heating curve that is assumed to apply for the FRL specified in the BCA and does not necessarily resemble the fire temperature versus time curve that might be encountered in a real fire. Any reduced FRL will have to be determined by relating its severity back to that associated with real fire.

Alternatively, as noted previously, the fire engineer could specify a design fire curve such as that shown in Figure 25. Here an average air temperature over the duration of ‘severe’ burning is taken as a constant temperature (eg 1000°C) over the duration of burning $t_{SD}$.

In very large spaces, where the combustible materials are sparsely distributed or where fully developed burning throughout the space is very unlikely, it may be more appropriate to consider a localised fire Figure 26[a] as opposed to considering that where the entire enclosure is subject to high temperatures Figure 26[b]. The former fire will result in heating of only parts of the building structure whereas the latter fire (termed an ‘enclosure fire’) will heat all members within the enclosure.

For local fires and their impact on the structure, it will be necessary for the fire engineer to determine the heating conditions (either air temperature versus time relationship or heatflux versus temperature) and the extent of heating. For enclosure fires, it is generally accepted that around 70% of the total combustibles are consumed up to the point that the air temperatures...
begin to drop. By assuming that the growth rate is very fast, and that the average air temperature is maintained until the commencement of cooling, a simplified design fire curve of the form shown in Figure 25 can be obtained. It is necessary only to determine the duration of burning. For an enclosure fire, this can be done by the fire engineer using the following steps:

[a] Estimate the total fuel (H) that could be consumed in the fire

The total fuel is normally determined by multiplying a fire load ‘density’ by the total floor area of the part of the building where the combustibles are located. The fire load density is the energy that could be released by burning the fuel associated with a unit area of floor and is normally expressed in MJ/m². For a particular situation at a given point in time, it is the average value of fire load density within the enclosure at that time that is applicable to determining the fire severity should a fire develop at that time. However, the ‘average’ values can vary widely for a particular occupancy and it is necessary that the representative value of fire load density is sufficiently high to allow for potential variation throughout the life of the building. Fire load densities for given occupancies have been measured by conducting fire load surveys. Data on fire load densities are provided in Reference 42. This publication also gives guidance on taking into account the variability of fire load density.

[b] Determine the relevant rate of burning (HRR)

The rate of burning (in megawatts (MW)) will be variable but a constant representative value has been adopted for the simplified Design Curve, up to the point that the air temperatures reduce. If the fire is a localised fire then the rate of burning can be obtained from the use of experimental burn data which gives heat release rate (HRR) versus time relationships associated with burning the combustibles in the open. In the case of an enclosure fire, the rate of burning is almost certainly governed by the ventilation available to the fire through openings such as windows (which break) and non-fire resistant doors. Various empirical relationships have been derived for the burning rate associated with a fire in an enclosure.

[c] Determine the duration of burning

For most combustibles, only a percentage (xi%) of the total fire load (FL) (kg) is consumed during the period that maximum air temperatures (see Curve 2 in Figure 25) are achieved. The duration of the maximum air temperatures can be obtained from the following equation:

\[ t_{fd} = \frac{H \times x}{100 \times HRR} \]  

where \( H = FL \times \Delta H_c \)

It is reasonable for \( x \) to be taken as 70% given the sustained high temperatures assumed for the burning phase. However, should a higher level of conservatism be required, a higher value can be adopted for \( x \). If the total fuel load is determined and the associated total heat energy, \( H \) (MJ), calculated using relevant heats of combustion (\( \Delta H_c \)), then the duration of burning (\( t_{fd} \)) is equal to the heat energy likely to be consumed during the main phase of burning (\( H \times x /100 \)) divided by the heat release rate (HRR) in MW.

8.3 Assessing the impact of fire on structural elements

8.3.1 Introduction

At the outset, it should be understood that for concrete construction, the fire resistance of a structural element is influenced directly by the key geometric dimensions of member width, depth and axis distance to the centre of the reinforcement. Of course, this latter dimension is directly related to the cover to the reinforcement. The reason that these dimensions are important is that as the member is heated, both the steel and the concrete closest to the heated surface increase in temperature with the temperature being greater the closer the location is to the heated surface. The temperature of the reinforcement is more nearly
represented by the temperature within the concrete at the adjacent location to the axis distance than by the temperature within the concrete at the cover distance from the exposed face of the member Figure 27.

As the temperature increases, the strength of the concrete (in compression) and steel (in tension) is reduced depending on temperature. The reductions in strength according to prEN 1992-1-2 are shown in Figure 28 for concrete (\(f'_{c}\)), hot-rolled reinforcing steel, cold-worked reinforcing steel, cold-drawn prestressing steel (most common) and quench and tempered prestressing steel. It is noted that the reduction in the strength of prestressing steel is greater than that associated with reinforcement due to fact that the higher strength of these steels is achieved by additional work hardening or by quenching and tempering and that such additional components of strength are more easily lost at elevated temperature.

### 8.3.2 Assessment of performance under standard fire test conditions

The simplest and most common way of assessing the adequacy of concrete members under standard fire conditions is to design them in accordance with Section 5 of AS 3600. Thus the member dimensions and axis distances for the member being designed will be chosen to satisfy the dimensions given in the Standard whilst recognising other limits such as load ratio and slenderness.

The requirements of Section 5 of AS 3600 are essentially based on the tabulated values given in prEN 1992-1-2 for various member types. These tabulated values take into account the loss of strength of both concrete and steel. However, the origins of some of the requirements are not clear, and although the requirements cover a greater range of practical members than previous versions of AS 3600, they do not cover all member sizes or fire exposure situations – particularly when only lower FRLs have to be met. For example, an investigation of the resistance of slender concrete walls under standard fire test conditions has been undertaken to assess walls having a thickness less than those covered by AS 3600 (and by prEN 1992-1-2). This model was developed also to better evaluate the beneficial effects of rotational restraint due to connected members outside of the fire enclosure. The model is similar to, but extends, the capability of previous models for assessing walls in fire. It incorporates the elevated temperature material properties given in prEN 1992-1-2. The model

![Figure 28 Reduction in concrete and steel strength – Eurocode 2](image-url)
PART B

Figure 29 Wall divided into segments and slices

Taking into account the effect of lateral displacements on wall strength with the wall being divided into elements along its length (segments) and across its width (slices) Figure 29. Equilibrium and compatibility are satisfied at each boundary using an iterative calculation. It was found that the presence of a moderate level of rotational restraint at one end will reduce the effective height of a wall to 0.80 of its actual height. In addition, a range of Alternative Solutions have been developed for walls having thicknesses of 80 and 100 mm. Such walls are relevant to multi-storey residential construction for which a fire-resistance of 60 minutes applies to the walls between occupancies.

Considering concrete structural elements generally, there may be some situations where due to low levels of load or different cross-sectional geometry, the tabulated solutions given in AS 3600 may not be directly applicable and it might be necessary to assess the structural adequacy of the element from first principles. This is most easily done by reducing the steel strength, ignoring a depth of concrete and determining the strength of the ‘effective’ cross-section using normal temperature methods for assessing the strength. A similar approach is also presented in Eurocode 2. Details of this approach are given in Section 8.3.4 below.

8.3.3 Assessment of performance under Design Fire Conditions (DFC)

If the fire engineer nominates the design fire conditions using the simplified approach described in Section 8.2, the duration of burning (t_d) will be specified. Once this is done the adequacy of the structural member could be similarly assessed by reducing the steel strength, ignoring a depth of concrete and determining the strength of the ‘effective’ member cross-section using normal temperature methods for assessing the strength. This approach is summarised in Section 8.3.4 below. In this case the fire is assumed to have a constant temperature of 1000°C compared with the gradually increasing temperature of the standard time temperature curve.

8.3.4 Assessment of fire resistance using effective sections (normal strength concrete)

Figure 35 schematically shows temperature gradients for typical column and bandbeam cross-sections with respect to fire exposed sides.

In the case of a column or wall, the strength in fire is influenced by the loss of concrete section (due to the reduction in compressive strength) and the reduction in steel strength. In the case of a beam or slab in sagging bending where the bottom steel is in tension, it is only the elevated temperature of the steel that reduces the bending strength. In the case of hogging bending, as will occur with flexurally continuous members at supports, it is the upper (cool) steel away from the fire that controls the bending strength provided the cross-section does not become ‘over-reinforced’ due to the loss of concrete in compression. The hogging bending capacity is usually not affected greatly by the fire.

An acceptable and simplified way of assessing the strength of a cross-section under fire conditions is to simply ignore the concrete that is above 500°C and adopt a reduced strength for the steel according to
This approach was determined by undertaking detailed analysis taking into account the variation of strength with temperature and comparing the results with member strength when only concrete having a temperature of greater than 500°C was ignored. This comparative analysis has been done for slabs, beams and columns including situations where the member is heated from more than one side. This approach is applicable to normal weight concrete elements and is almost identical to that given in Reference 13. The approach gives rise to an ‘effective’ cross-section that will enable the structural engineer to assess the strength in fire. It is only necessary to know which locations of the concrete cross-section have achieved a temperature of 500°C and what the temperature of the steel is at a particular location. Once the effective cross-section is determined, it is then necessary for the structural engineer to assess whether the member can support the loads to be resisted in fire. For continuous slabs and beams, redistribution of moments can be taken into account and it is only necessary to show that the combination of sagging and hogging capacities will be sufficient to maintain equilibrium in fire. AS 1170.0 gives the load factors that are relevant for the fire situation. This simplified assessment of strength is not applicable to members such as columns and walls that are heated on less than all sides since the differential thermal expansion will result in thermally induced deflections that may further reduce the strength of the member. It is, however, applicable to slabs and beams, or walls and columns that are exposed essentially to fire on all sides. In the case of walls or columns that are exposed to fire on only one side, the resistance can be calculated only by the type of approach described earlier in relation to thin reinforced concrete walls.

Tables 3 to 6 give guidance regarding temperatures within concrete members for both standard and DFC conditions. In these tables, Curve 1 SFC refers to the exposure time assuming the member is heated according to the standard fire curve whereas Curve 2 DFC considers heating of the member when subject to the design fire shown in Figure 25 where the duration of heating is $t_{fu}$. A gas temperature of 1000°C has been adopted, since from a comparison with the results of experimental fires, this temperature more than adequately represents the average temperature sustained over the period of burning in a naturally ventilated fire enclosure. In practice, it makes little difference to member behaviour whether a more ‘precise’ real fire temperature versus time curve is used or that proposed in this Guide. The values given in Tables 3 to 6 assume that there is no significant spalling or that this has been eliminated through appropriate measures.

If a column, wall, or band beam has an aspect ratio of 1:4 or greater then the effective section as far as loss of concrete is concerned can be taken as that associated with a slab. The same is also true for the reduction in steel strength should the steel reinforcement be located further away from the edge than the short side of the member.

In the case of continuous slabs and beams, it is necessary to consider both sagging and hogging moments and redistribution of moments can be taken into account. The sagging and hogging cross-sectional capacities can be determined from the above tables. The hogging bending capacity is usually not affected to the same degree by the fire since the steel reinforcement is cool. However, it is important to check that the cross-section does not become ‘over-reinforced’ if moment redistribution is to be taken into account.

For members resisting compressive loads such as columns and walls, the height: thickness (or slenderness) of the member can also be important. In the non-fire situation, such as that associated with a
### TABLE 3 Square columns

<table>
<thead>
<tr>
<th>Temperature (°C)</th>
<th>Fire exposure time (minutes)</th>
<th>Resultant reduced section</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>30</td>
<td>60</td>
</tr>
<tr>
<td>Concrete</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Curve 1 SFC</td>
<td>14</td>
<td>28</td>
</tr>
<tr>
<td>Curve 2 DFC</td>
<td>22</td>
<td>35</td>
</tr>
<tr>
<td>Steel</td>
<td></td>
<td></td>
</tr>
<tr>
<td>400 Curve 1 SFC</td>
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<td>38</td>
</tr>
<tr>
<td>500 Curve 1 SFC</td>
<td>14</td>
<td>28</td>
</tr>
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<td>600 Curve 1 SFC</td>
<td>8</td>
<td>20</td>
</tr>
<tr>
<td>700 Curve 1 SFC</td>
<td>3</td>
<td>13</td>
</tr>
</tbody>
</table>

### TABLE 4 Slabs

<table>
<thead>
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<th>Temperature (°C)</th>
<th>Fire exposure time (minutes)</th>
<th>Resultant reduced section</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>30</td>
<td>60</td>
</tr>
<tr>
<td>Concrete</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Curve 1 SFC</td>
<td>12</td>
<td>25</td>
</tr>
<tr>
<td>Curve 2 DFC</td>
<td>20</td>
<td>30</td>
</tr>
<tr>
<td>Steel</td>
<td></td>
<td></td>
</tr>
<tr>
<td>400 Curve 1 SFC</td>
<td>18</td>
<td>34</td>
</tr>
<tr>
<td>500 Curve 1 SFC</td>
<td>12</td>
<td>24</td>
</tr>
<tr>
<td>600 Curve 1 SFC</td>
<td>7</td>
<td>17</td>
</tr>
<tr>
<td>700 Curve 1 SFC</td>
<td>3</td>
<td>10</td>
</tr>
</tbody>
</table>

### TABLE 5 Bandbeams

<table>
<thead>
<tr>
<th>Temperature (°C)</th>
<th>Fire exposure time (minutes)</th>
<th>Resultant reduced section</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>30</td>
<td>60</td>
</tr>
<tr>
<td>Concrete</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Curve 1 SFC</td>
<td>13</td>
<td>27</td>
</tr>
<tr>
<td>Curve 2 DFC</td>
<td>21</td>
<td>34</td>
</tr>
<tr>
<td>Steel</td>
<td></td>
<td></td>
</tr>
<tr>
<td>400 Curve 1 SFC</td>
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<td>37</td>
</tr>
<tr>
<td>500 Curve 1 SFC</td>
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<td>28</td>
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</tr>
<tr>
<td>700 Curve 1 SFC</td>
<td>3</td>
<td>12</td>
</tr>
</tbody>
</table>
multistorey building where columns can be considered to be flexurally continuous, the effective length is obtained through elastic buckling analysis where the flexural stiffness of the compressive element is reduced uniformly along its length due to the presence of compressive forces. For a braced building this results in an effective length equal to the floor-to-floor height. In contrast, the fire situation is different since the compressive forces do not change (unless there is restraint of axial expansion) but only the stiffness of the heated length of the member – which typically will be one storey height or less. The stiffness of other parts of the compressive member that are not exposed to fire, do not change. The resultant effect is that the effective length of a compression member in fire is less than that associated with normal temperature conditions. It is estimated that for a member heated on all sides, the effective length can be taken as 0.75 of the floor-to-floor height for a member within a braced structure where the column is flexurally continuous at each end.

8.3.5 Assessment of fire resistance using effective sections (high strength concrete)

The approach described above in relation to normal strength concrete can be used provided that the thickness of concrete to be ignored given in Tables 3 to 6 is increased by 30%. This figure takes into account the greater loss of strength at lower temperatures for high strength concrete compared with normal strength concrete (see Figures 6 and 7) and again has been based on a comparative analysis as described previously for NSC. It is also necessary that adequate measures are taken to control/eliminate spalling (see Section 6).

<table>
<thead>
<tr>
<th>Temperature (°C)</th>
<th>Fire exposure time (minutes)</th>
<th>Resultant reduced section</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Concrete</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Curve 1 SFC</td>
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<td></td>
</tr>
<tr>
<td>Curve 2 DFC</td>
<td>23  37  47  55  70  82</td>
<td></td>
</tr>
<tr>
<td>Steel</td>
<td></td>
<td></td>
</tr>
<tr>
<td>For axis distance of reinforcement, a (mm)</td>
<td></td>
<td></td>
</tr>
<tr>
<td>400 Curve 1 SFC</td>
<td>21  40  54  65  82  98</td>
<td></td>
</tr>
<tr>
<td>Curve 2 DFC</td>
<td>30  47  59  69  85  98</td>
<td></td>
</tr>
<tr>
<td>500 Curve 1 SFC</td>
<td>15  30  42  53  68  82</td>
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</tr>
<tr>
<td>600 Curve 1 SFC</td>
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</tr>
<tr>
<td>Curve 2 DFC</td>
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</tr>
<tr>
<td>700 Curve 1 SFC</td>
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<td></td>
</tr>
<tr>
<td>Curve 2 DFC</td>
<td>12  20  26  31  40  48</td>
<td></td>
</tr>
</tbody>
</table>

8.4 Examples of assessment of members

Various situations are now considered.

**Situation 1 – Lower fire resistance level nominated**

In this case, on the basis of a fire engineering assessment, the fire engineer recommends that a lower fire-resistance level (FRL) be used for assessing the fire resistance of the structural elements. This is a simple case, since it is necessary only for the structural engineer to redesign the concrete member in accordance with AS 3600 ensuring that the members can resist the reduced FRL.

**Situation 2 – Design fire characteristics are nominated**

In this case, the fire engineer will have recommended a design fire which is characterised by a gas temperature versus time relationship as represented by Curve 2 (DFC) in Figure 25. In particular the burning duration, t_{FD}, will have been specified. Suppose that t_{FD} is 30 minutes and the member being assessed is a prestressed continuous bandbeam. The structural engineer can assess the fire resistance of the band beam by using Table 5. The following procedure is suggested:

- From Table 5 note that 21 mm of the concrete soffit must be ignored for the effective section. The remainder of the concrete section can be considered to be unaffected by the fire.
- Determine the temperature of the steel from Table 5. For the purpose of this exercise, it is assumed that the axis distance of the prestressing steel is 30 mm from the soffit. This gives a temperature of approximately 400°C.
- From Figure 28 determine the reduction in strength of the steel. This figure indicates that for cold drawn prestressing steel, the strength is reduced to 45% of the strength before the fire.
Determine the sagging and hogging bending capacities.

In accordance with AS 1170.0 determine the loads applied to the member in fire and the associated bending moments.

Check that equilibrium is achieved. If not, increase axis distance to the prestressing steel and/or thickness of the concrete.

An example of the reduced section at bandbeam cross-section after fire is given in Figure 31.

Situation 3 – FRL specified but member geometry or load level does not fall within the scope of AS 3600

In this case, it is possible to assess the fire resistance of a member using the data given in Tables 3 to 6 and the load level actually applied to the member. Either the resistance under the standard fire curve (Curve 1 SFC – Figure 25) or when subject to a design fire (Curve 2 DFC) can be assessed. It is necessary only to determine the reduced cross-section (reduced concrete cross-section and associated steel strength) to enable the strength of the member to be calculated and compared with the load applied in the fire situation. For example, if a lightly loaded square column is being assessed for an FRL of 120/-/-, then the reduced section according to Table 3 will require 54 mm of concrete from the sides of the column to be ignored. If the axis distance to the reinforcement is 40 mm, then the temperature of the reinforcing bars at this location is 600°C and the reinforcing strength according to Figure 28 is 50% of the strength before the fire. Reinforcing bars manufactured in Australia can be considered to be hot rolled steel. The structural adequacy of the member under the applied load can then be assessed taking into account the reduced section.

Part B of this publication has given an overview of the ways in which buildings may be designed for fire safety, ie in accordance with the deemed-to-satisfy provisions or by using a fire engineering process. The latter procedure will sometimes, allow the building structure, or parts of it, to have a lesser fire resistance. Such a recommendation must come from the fire engineer and be the result of a fire engineering assessment. This recommendation may be in the form of a reduced FRL or a design fire curve (ie Curve 2 of Figure 25) representing the fire situation more likely to be encountered.

In many cases, it will be sufficient to use the provisions of AS 3600, since the 2009 edition of AS 3600 provides a more flexible range of solutions for members such as columns and walls compared with the previous edition.

The assessment of member resistance presented in this Guide relates only to isolated member behaviour. In real fire situations, due to the fact that fire will often be limited to part of the building and/or its heating effects will vary, building structures can exhibit a level of resistance which is greater than that predicted by considering isolated member behaviour. However, the evaluation of such improved response is difficult and requires not only the use of complex finite element models but also a better understanding of fire growth and development.

Guidance is given to enable structural engineers to assess the structural adequacy of most building elements when subject to either the ‘real’ design fire curve or period of standard fire exposure.
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Section 10
References


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