STRUCTURAL ROBUSTNESS AND FIRE

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ABSTRACT: *Structural robustness is a performance requirement in most structural codes and standards. The consideration of fire in relation to robustness has been prompted by reviews of major fires in buildings, some of which were the result of terrorist attack. This paper examines the issue of robustness from a fire perspective by considering some of these incidents and the findings from research on the robustness of framed construction. This paper considers whether buildings can be considered to be robust under fire conditions if they meet the design criteria of BV2 of the National Construction Code 2016 in addition to the requirements of Section C (fire-resistance) of that code. It is concluded that demonstrating that BV2(a) (loss of column or similarly critical member) can be achieved under ambient conditions is sufficient evidence that loss of one such member under fire conditions will not result in disproportionate collapse provided the connection details are chosen to avoid the shearing of bolts and crippling of the webs of supporting columns as a result of compression forces induced by restraint of expansion. Suggestions are made as to how the robustness of a building can be further improved through improvements to the design of the sprinkler system and possibly also through the provision of fire-resistant edge spandrels even though these are not currently required by the prescriptive provisions of the NCC for buildings with an effective height greater than 25m*.

KEYWORDS: robustness, fire, risk, fire resistance, disproportionate collapse

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1 INTRODUCTION

The word *robustness*, in its common usage, can be considered to refer to an attribute of a product or system that will allow it to be forgiving of at least some events that may not have been specifically envisaged at the time of development of the product or system. These events have not necessarily been recognised *before* they occur. Thus a robust economy is one that is more likely to remain in sound condition despite the occurrence of an unforeseen recession. Similarly, in an engineering sense, a robust solution is one that is likely to show a level of tolerance to some events that were unforeseen or unable to be quantified at the time of design. The Building Code of Australia (BCA) used to require lift shafts to be constructed from concrete or masonry and still requires loadbearing internal and fire walls to be constructed of concrete or masonry. This longstanding requirement has been in place due to concrete and masonry being considered by some to be more resistant to damage and unauthorised alterations and therefore being of sufficient "robustness".

Performance requirement BP1.1 of Volume 1 of the National Construction Code (NCC) [1] states the following:

BP1.1

(a) A building or structure, during construction and use, with appropriate degrees of reliability, must—

- *(i) perform adequately under all reasonably expected design actions; and*
- *(ii) withstand extreme or frequently repeated design actions; and*
- *(iii) <u>be designed to sustain local damage, with the structural system as a whole remaining stable and not being damaged to an extent disproportionate to the original local damage; and</u>*
- *(iv) avoid causing damage to other properties, by resisting the actions to which it may reasonably expect to be subjected.*

*(b) The actions to be considered to satisfy **(a)** include but are not limited to—*

A list of actions is then given in clause (b) and includes those associated with dead and live load, wind, earthquakes, snow, thermal effects (ambient), liquid pressure from ground water, rainwater, earth pressure, differential movements, time-dependent effects, ground movement, construction activity and termites. It is of interest that fire is not specifically mentioned amongst these actions although it is not excluded.

In volume 1 of NCC 2016, a verification method (**BV2** Structural Robustness) has been added in relation to the above performance requirement. This states the following:

*Compliance with **BP1.1(a)(iii)** is verified for structural robustness by—*

(a) assessment of the structure such that upon the notional removal in isolation of—

(i) any supporting column; or

(ii) any beam supporting one or more columns; or

(iii) any segment of a load bearing wall of length equal to the height of the wall

the building remains stable and the resulting collapse does not extend further than the immediately adjacent storeys; and

(b) demonstrating that if a supporting structural component is relied upon to carry more than 25% of the total structure a systematic risk assessment of the building is undertaken and critical high risk components are identified and designed to cope with the identified hazard or protective measures chosen to minimise the risk.

The above title for BV2 links the term "robustness" with resisting disproportionate collapse (since BV2 relates to BP1.1(a)(iii)). BV2 also introduces the important concept of *risk assessment* as part of identifying critical elements and determining how to treat them.

In the United Kingdom, Part 3 of Approved Document A of the Building Regulations [2] requires that "The Building shall be constructed so that in the event of an accident the building will not suffer collapse to an extent disproportionate to the cause".

Figure 1 Ronan Pt failure due to gas explosion

The Guidance provided in Part 3 of that document makes reference to several industry guidance documents that link the *robustness* of a structure to its ability to resist disproportionate collapse. The issue of disproportionate collapse was first recognised as an issue after the Ronan Point apartment incident in 1968 when an unexpected gas explosion in one apartment resulted in failure of a corner of the building (Figure 1). The panels of which building was composed relied on friction only to maintain connectivity.

EN1991-1-7-2006 [3] defines robustness as *the ability of a structure to withstand effects like fire, explosions, impact or the consequences of human error, without being damaged to an extent disproportionate to the original cause.*

It is noted that the above definition of robustness has been broadened to specifically mention *fire*, *explosions* and *impact*. This is probably not surprising given that this definition was developed after such incidents as the bombing of the Murrah building in Oklahoma in 1995 and the attack on the World Trade Centre buildings in NY in September 2001 during which the twin towers were subjected to both impact and fire. As will be noted below, designing against terrorist attack is problematic.

Since that time there has been discussion in the engineering literature of the importance of robustness in relation to fire (see for example Refs [5, 6]) in that it is implied that there are some fire scenarios that pose unique challenges that may not be sufficiently addressed by the normal robustness requirements such as those described above in relation to BV2. The purpose of this paper is to consider whether this is the case or whether it is sufficient to meet the fire-safety requirements of the NCC (i.e. those of Section C of volume 1 of the NCC) in addition to the robustness structural requirements such as those given by BV2. This is to be done by reviewing some of the significant fire incidents and research relating to robustness and fire and then by considering the connection between risk level and robustness.

2 ASPECTS OF FIRE BEHAVIOUR IN BUILDINGS

Before considering a number of real fire incidents and the research on the observed or simulated behaviour of buildings in fire, it is necessary to consider some aspects of fire development and fire safety in buildings.

Unlike wind and earthquake and putting aside bushfires, building fires are not a “natural phenomenon” but the result of causes such as:

a) equipment malfunction due to fault or ageing (mechanical and electrical)
b) inappropriate use of heating equipment (e.g. portable heaters)
c) poor work practices (e.g. cutting and welding without adequate procedures)
d) accidents associated with the use of heating in the preparation of food
e) smoking (almost non-existent in public buildings)
f) arson (deliberate initiation of fire but not to the extent of g)
g) terrorist attack

Unlike earthquake and wind, a) can be influenced by building maintenance and c) by adequate work practices. f) can be influenced by surveillance and the deterrent effect of the presence of other people. Approximately 8% of fires in US office buildings are deemed “suspicious” but can be considered to fall well within the accepted range of fire events experienced by buildings. Fires associated with cooking are common in Apartment Buildings but rarely extend beyond the room of fire origin indicating the importance of the presence of people as an important form of detection and mitigation.

Fires associated with a) to f) begin small and develop over time. This gradual fire development makes it possible for fires to be detected and extinguished prior to becoming an *out-of-control* fire event. Detection is achieved by the presence of people as well as automatic fire detection systems and suppression can be achieved via occupant fire-fighting (extinguishers) and automatic suppression systems such as sprinklers. In the case of g), the fire growth rate may be very rapid and since almost any explosive/incendiary device is possible, it will always be difficult to design against such events.

Fires in buildings can be localised as with most fires in carparks or in large high spaces such as atria or convention centres or where the combustibles are well separated. In many spaces, if fire growth is unimpeded, the fire will continue to spread and grow reaching a “flashover” conditions where hot gases issue from the openings and a significant proportion of the fuel surfaces are subjected to pyrolysis (the application of heat to the surface to convert the solid surface into combustible gases and particles) or can become fully developed and eventually spread throughout the immediate fire enclosure. During the early stages of a fully-developed fire the rate of pyrolysis can be very high such that only a proportion of the burning takes place within the enclosure but much of it is released externally in the form of flames at the edge of the building (see Figure 2). This has the following potential implications:

(a) E*xternal* fire spread to the next level. The effectiveness of vertical spandrels (if they exist) at the edge of a building can be limited, particularly if the levels are not compartmentalised (e.g. an open-plan office).
(b) Columns adjacent to the glazing are heated more significantly than columns in the interior.

(c) Column stability is reduced if heating is over a double or greater floor height.

Figure 2 External burning - Delft University Fire

A significant related issue associated with multi-storey buildings is the use of combustible facades or curtain walls (Figure 3) which are popular from architectural and energy conservation perspectives but often poorly described (e.g. metal curtain walls with no mention of combustible content) or poorly understood. They are sometimes added without the knowledge of the fire safety engineer being described on the drawings as “metal curtain walls”. The fire shown in Figure 3 is the result of a small fire on balcony at a lower level. Should flaming such as that shown in Figure 2 occur adjacent to such a combustible façade, the fire would be expected to spread rapidly up the height of the building.

Figure 3 Vertical Spread Combustible Facade

If we consider a fire that is confined to a particular enclosure, the rate of burning is largely dictated by the available ventilation (essentially from broken windows) and the duration of burning related to the total quantity of fuel (fuel load density x area) and the aspect ratio of the enclosure. Enclosures that are deep with respect to the ventilated side will burn significantly longer than relatively shallow enclosures with burning taking place from the ventilated face and extending back into the enclosure as fuel is progressively consumed. The gas temperature versus time relationships in fully-developed fires vary with location and time and is likely to differ significantly from the time versus temperature curve associated with the standard fire test for which the temperature increases continuously with time. Extensive fully developed fires are almost impossible to fight especially if they are located well above the ground.

3 BUILDING FIRE INCIDENTS

The concept of “robustness” has commonly been applied to buildings potentially subject to earthquake loading or terrorist attacks using high explosive. The use of the term in relation to fire has arisen after the 9/11 attacks and particularly in relation to one of the buildings on the NY site - WT7. which was not subject to direct attack but nevertheless collapsed about 7 hours after the initial fires developed. Three significant fires in high-rise buildings are now considered.

3.1 WORLD TRADE CENTRE BUILDING 7

This review is given on the basis of information reported by others [6, 7]. This building was a 47 storey steel-framed building protected with fire spray on both columns and beams. Typical floors were glazed around the entire perimeter with approximately 1.9m high glazing with approximately 1.8m high spandrel separations between each window opening and the next above. These spandrels were constructed from 50mm fire-resistant stone wool, an air gap and outer panels of granite. Cavity barriers were incorporated between the inner stone wool and the outer granite sections and at each floor slab the gap between the floor slab and the outer granite was fire stopped with stone wool. On the North and South faces the glazing was continuous across these sides on the building whilst on the East and West Sides the glazing was interrupted at approximately 2.5m centres by vertical sections of construction identical to the spandrel details having a width of 0.75m.

The steel-framed building incorporated composite floor slabs (W deck) supported by composite beams, which in turn, were supported by non-composite girders. A typical floor plan for levels above Level 7 is shown in Figure 4. Lateral load resistance in both directions was achieved by moment frame action associated with the columns and beams located just inside the building façade. The connections between these members were moment connections. The external column spacing across the North and South faces was 5m whereas that associated with the East and West sides was 2.5m. Exterior belt trusses around the perimeter were also incorporated at two levels. The fires in WTC 7 were due to the collapse of WTC 1 located 105m to the South. Not only was there physical damage to parts of the building, burning materials entered the broken façade and initiated fires on multiple levels (10 floors) however, only the fires on Levels 7–9 and 11-13 lasted until collapse.

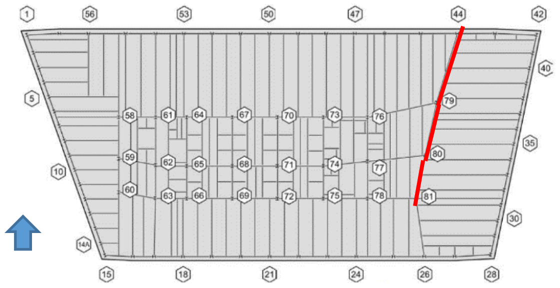

Figure 4 Typical floor layout for WTC 7

Due to the collapse of WTC 1 the main water supply to the building sprinkler system had failed and the duel tank storage provided within the building only served the sprinkler system above level 20. The fires below Level 20 were therefore not able to be extinguished by the sprinkler systems in the building. The fires burned for almost 7 hours before progressive collapse of the building occurred. The 4000 occupants had already evacuated following the attacks on WTC 1 and WTC 2.

The fire load on most levels was estimated to have been around 20kg/m^2 whilst the fire load density on levels 11 and 12 were considerably higher. The thicknesses of fire protection on the structural steel ranged from 22mm for the heavier columns, 44mm for the lighter columns and 13mm for the beams and girders. Ref [6] estimates that the temperature of the steel beams would have exceeded 600°C. The connections between the beams and girders are described as single plate shear connections (i.e. web side plates) with some double angle connections. Those between the beams and external columns were seated beam connections (i.e. bottom angle bracket with top angle bracket for lateral restraint) whilst those between the girders and the internal columns were endplate connections. All of these connections are connections designed to primarily resisting vertical reaction loads at the ends of the beams which are nominally pin-ended.

The collapse has been attributed to failure of the girders connecting interior columns 79, 80 and 81 (see Figure 4) due to instability of these girders caused by compression due to restraint of thermal expansion exacerbated by one-sided expansion of the attached beams causing twisting of the girders. Ultimately this resulted in failure of the connections with collapse of the floor and loss of restraint to the main interior columns. The failure of these columns initiated progressive collapse of the building.

Figure 5 illustrates that as steel beams expand when heated, any restraint of expansion will create compressive forces (see (a)). For example, a rise in temperature of 200 °C for a non-composite girder of 16m length will result in an expansion of around 40mm.

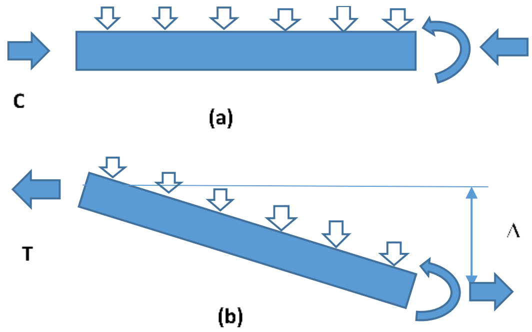

Figure 5 Actions experienced during fire

The associated compressive axial forces can be significant and may result in local or torsional buckling of the girder assuming adequate connection ductility (web or flange permitted to bear directly on column). The effect of buckling, combined with the reduction in strength with temperature is such that the bending resistance can be reduced to the extent, that in order to resist the applied floor loads, a component of the resistance must be provided through catenary action (see (b)) with axial tension forces acting in the beams and through the connections. Unless the connections can resist the combined tension and reaction loads, failure can occur.

Several observations can be made regarding WTC 7 and the observed behaviour:

(i) The building resisted multiple fires for almost 7 hours – a significant performance

(ii) No spread between levels due to flaming at the façade occurred

(iii) The North-West corner of the building involves significant distances to the façade and with the window areas provided at the façade would have resulted in severe fires of substantial duration and well in excess of an "equivalent" standard fire duration of 60 minutes.

(iv) The 13mm protection applied to the beams would have given a standard fire-resistance performance of around 60 minutes, albeit it was considered to give a fire resistance of 120 minutes. To achieve 120 minutes, a thickness of about 26mm would have been required. This anomaly occurs because of the US fire testing tradition of using restrained or unrestrained floor fire test results. Restrained floor fire tests give significantly better performance but the restraint conditions in the test are unlikely to be realised in practical building situations. An increase in fire protection would have probably have had a beneficial effect.

(v) As noted in Ref [6], the building structure provided no alternative load path in the event of removal of one of the main interior columns. This is significant when considering the requirements to resist disproportionate collapse.

3.2 CARACUS TWIN TOWERS

The comments here are based on information reported in Ref [8]. In 2004, a fire commenced with the East Tower of Parque Central in Caracus, Venezuela. The fire occurred on the 34th storey of the 56 storey building and eventually, despite brigade fire-fighting, spread up to level 50 consuming the combustible contents of the floors. The building was not occupied at the time of the fire and the sprinkler system had been isolated due to leakage and other issues. The basic plan area of each floor was 39.5m x 39.5m with glazing around the perimeter of the building being approximately 2.5m in height. A vertical spandrel of approximately 1.25m appears to have been provided between the glazing associated with each floor. to separate the glazing. Little is known of the fire load on the floors or the fire protection of the steelwork other than the steel was protected to achieve a fire-resistance of 120 minutes in accordance with US requirements.

The building structure has been described as a “concrete super-frame” [9] whereby the building consists of large external reinforced concrete columns with 3m high post-tensioned transfer floors (called macro slabs) every 12 storeys. Each macro slab provided vertical support to a steel-framed 10 storey building which was laterally supported by the reinforced concrete columns. These steel frames consisted of steel columns and beams supporting composite slabs. The details of connections between beams and columns are unknown. A typical floor plan showing the interior steel columns and exterior reinforced concrete columns is shown in Figure 6.

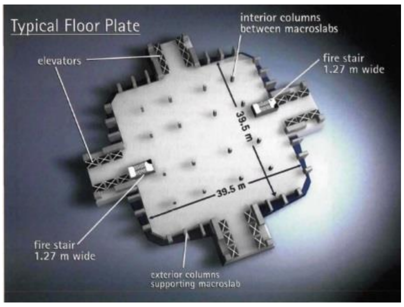

Figure 6 Typical floor plan (after [8])

Ref [8] notes that the compartmentation of each floor was compromised by the presence of floor access panels that were not fire rated. The fact that the fire spread 17 floors is of great concern and it is not clear whether this was due to spread due to weaknesses associated with the floor access panels, external flame spread or both. Building codes such as the Building Code of Australia (BCA) require each level of buildings of Type A construction (rise in storeys greater than 3) to be a separate fire compartment such that fire is contained to one level. Some structural damage on one floor may be acceptable but structural damage on multiple floors is more likely to lead to progressive collapse.

As will be noted from Figure 6, there is no central core. If combustibles were distributed across the entire floor, given the distance to the façade, a fire of long duration (approaching the severity associated with 2 hour standard fire) could have developed on each floor. The increased window area compared to WTC 7 may have reduced the comparative burning duration but, on the other hand, would have also increased the likelihood of external fire spread. Unfortunately, nothing is known about the level of fire load or its distribution across the floor. Post-fire observations indicated that two floor decks had partially collapsed and that several beams were severely deformed. No details are known of the level of fire protection actually applied to the steel but clearly it was sufficient to prevent excessive temperatures in the steelwork.

3.3 MADRID WINDSOR

The following discussion is essentially based on the description given in Ref [10]. On February 2005 at 11pm, a fire occurred in the 32 storey Windsor office tower in Madrid, Spain. Given the time of the fire, the building was unoccupied at the time of the fire. It was also the subject of a progressive upgrade program for the fire safety systems which included the protection of steelwork, fire stopping of penetrations and the installation of sprinklers. None of these tasks had been completed at the time of the fire. The building incorporated a concrete core, two reinforced concrete transfer structures (called “technical floors”) and reinforced concrete internal columns. The exterior columns around the perimeter of the façade were unprotected steel members as were internal beams spanning between the reinforced concrete columns. The floors were two-way waffle floors which were vertically supported by the core, the reinforced concrete columns in association with internal unprotected steel beams and the unprotected perimeter columns.

The fire commenced on the 21st floor but within one hour all of the floors above that level (11 storeys) were involved in the fire. In the following hours, the fire spread downwards due to burning materials dropping though unsealed penetrations. Two hours after the above fire spread, chunks of the façade began to fall off and after another 2 hours (4am) the floor slabs above level 17 collapsed but due to the presence of the technical floor below level 17, no floors below level 17 collapsed. This was also the case despite the continued spread of fire downwards. It needs to be noted that the structural steelwork below level 17 had been substantially protected as part of the upgrade program. The building has had to be replaced.

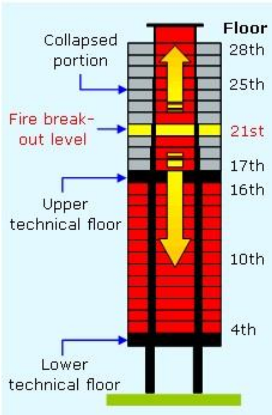

Figure 7 Progression of fire (after [10])

The following comments are made regarding this incident:

(i) It is remarkable that floor collapse did not occur until 4 hours after involvement of all of the levels above Level 17.

(ii) The spread of fire to levels above Level 17 is of great concern and highlights the importance of fire adequate fire-stopping of penetrations and between the façade and the floor.

(iii) If the fire had been limited to one level, it is quite possible that the although the façade steel elements buckled, the perimeter of the building could have been supported by the columns above the fire floor acting as tension members, provided these were connected along their length so as to carry tensile force and provided the waffle floors and connections between the waffle floors and the perimeter columns had sufficient strength under normal temperature conditions. Such robust behaviour appears to have occurred in the levels below Level 17 where the steelwork on most levels (except for two) had been protected.

(iv) The presence of the concrete core, substantial concrete columns and the technical floors clearly provided a high level of resistance against *overall* structural collapse, notwithstanding the fact that the damage was so great that the building had to be demolished and re-built.

4 RESEARCH FINDINGS

Considerable experimental and analytical work has been undertaken in Europe in relation to the robustness of steel structures in fire. Much of this work has focussed on the ductility of connections and the resistance characteristics of connections when subject to significant rotation. Although much of the focus of this research has been on steel carpark buildings subject to extreme local fires, the findings are relevant to a more general consideration of robustness. As explained when considering the fire associated with WTC 7, connections between columns and beams can experience high rotations, compressive and then tensile forces.

References 11 – 19 are a sample of the output from European-wide cooperative research programs which present the results of ambient and elevated temperature testing of a range of connections as summarised below.

Table 1 Summary of Tested Connection Types

Connection Type	**Description**	**Ref**
Web Cleat	Double angles connect beam to column web or flange	13, 14
Flexible Endplate	End plate welded only to web of beam	11, 13 14, 17
Flush Endplate	Heavier plate welded to both web and flanges of beam with all bolts located between flanges	13, 14 17, 18 19
Extended Endplate	Heavier plate welded to both web and flanges of beam but extending past top flange. Bolts as for flush endplate but with one row also above top flange	14, 17
Fin Plate	Side plate welded to column but connected to coped web of beam	11, 13, 14,18, 19
Reverse Channel	Used with tubular columns with channel flanges welded to tube	12, 19

The experimental work reported by Wald [11] is concerned with the rotational ductility of connections associated with unprotected beams designed to act as part of the “slab membrane” mechanism that develops with partially fire protected floors. This information was obtained from a natural fire test undertaken on part of the steel-framed structure used for the Cardington tests. The connections (flexible web plate and side plate) provided vertical support throughout the natural fire although cracking was noted during the cooling process.

Wang and Dai [14] conducted furnace testing of a horizontal H frame consisting of two columns connected by a beam. No fire protection was applied to the steelwork except on the top flange of the beam to simulate the shielding effect of a concrete slab. A range of connections were tested (see Table 1) with the rotation at the joint and horizontal force in the beam being determined. The aim of the testing was to determine the tensile force versus temperature response for two different size columns. Not surprisingly, significant bending of the columns occurred with the lower beam flange causing plastic deformation of the column web and flange (the columns were not protected). The best performance with respect to resisting tensile forces at high temperature and high rotations were the web cleat (double angle), the fitted endplate and the extended endplate provided higher strength bolts (Grade 10.9) were used. Such connections were described as being the most “robust”.

Ding [12] describes tests on connections between concrete-filled tubes and steel beams undertaken in the same manner as in Ref [14] using a horizontal H frame within the furnace. Connections considered were (a) special bolts and extended endplate (SHS only), reverse channel and flexible web, and reverse channel and fitted endplate. The latter is considered to give the best performance with respect to connection rotation and subsequent tensile resistance. Ref [19] is a more extensive description of the above testing and other tests undertaken on connections between steel beams and tubular sections. Burgess [13] reports the results of ambient and elevated temperature tests on flexible endplate, flush endplate, fin plate and web cleat connections where load was applied at various angles (35°, 45° and 55°) to the horizontal to determine the rotation under combined tying (tension) force and shear. The fitted endplate was found to be the least ductile but have the highest strength.

References [16] and [18] describe the development of various assessment methods for determining the “robustness” of open-deck carpark structures. Global finite element models (FEM) have been developed to seek to model a range of connections including connections in steel-framed building utilising composite slabs. These models have been “calibrated” against test results for a range of connections. Simplified models have also been developed for use by the practitioner based on a combination of theoretical studies using FEM and test data. For the purpose of the current discussion, a generalised connection is shown in Figure 8.

A type of connection considered in [16] and [18] as being capable of permitting “robust” performance for a steel frame under both ambient and elevated temperature is based on the fitted endplate connection.

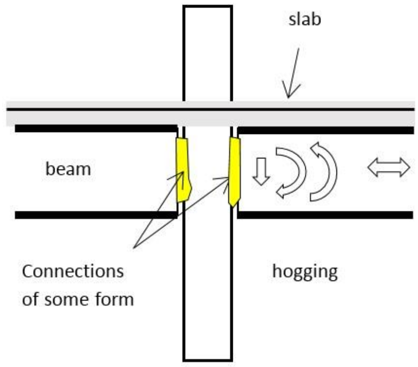

Figure 8 Typical Connection showing Actions

This connection is used in conjunction with a composite slab with ductile reinforcing bars in the slab at the connection location to enhance the hogging moment capacity. The potential failure modes with the above connection under fire exposure conditions are (a) failure of a connection component (e.g. bolts) due to tensile forces, (b) failure due to shear force (sometimes called punching shear) and (c) failure of the reinforcement due to excessive strain at a crack.

An FEM analysis has been used to study the impact of loss of a column in an open-deck carpark due to unexpectedly high temperatures. The possibility of the above modes of failure at the connection is also considered, however, for the purpose of this paper only the column loss scenario will be considered. Potential failure of an interior column on either the lowest level, a middle level or the uppermost level of an 8 storey carpark building were considered.

A fire associated with the lowest level is illustrated in Figure 9. Loss of a column at a lower or mid-level results in vertical displacement of all of the floors above with the loads normally resisted by the column now being resisted by the double span beams at the fire level and the levels above. Proportionally more load will be resisted by the ambient floors due to their greater stiffness. The vertical deformation at the column location results in sagging bending at this location and hogging bending at the adjacent columns on each level. The development of tensile membrane forces in the beams can be very beneficial but depends on the ductility of the connections (particularly for the ambient floors) under combined bending and tension and the *ability of the surrounding structure to resist these tensile forces*. The loss of an exterior column and particularly a corner column can present a greater challenge. The magnitude of membrane tensile force *required* to achieve equilibrium is a function of the deflection (or sag) at the (failed) column location and the sagging and hogging bending capacities. For example, a total in-plane extension of 32mm (e.g. 16mm at each end) of 10m span beams on each side of a column will permit a total sag of 800mm. The ductility of the connections whilst sustaining significant net tension forces is therefore critical and can be achieved under ambient conditions if appropriate connection details are used. Under elevated temperature conditions, the ductility increases whilst the bending and tensile capacity reduce. Nevertheless, application of the above approach can be used to demonstrate that loss of a column in an upper level of the building or at ground level will not result in progressive collapse, despite failure of a column. This is true under both ambient and fire conditions provided suitable connections are used.

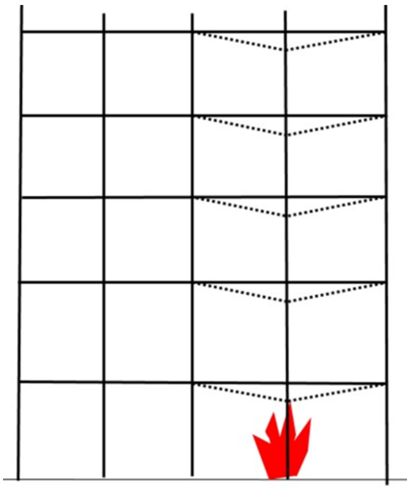

Figure 9 Local Fire Exposure

Research into the behaviour of steel and composite buildings in fire has provided information on the characteristics of various connections with respect to their ductility and ability to resist tension under both ambient and elevated temperature conditions and permits an assessment of structural robustness under either ambient or fire conditions.

5 ROBUSTNESS IN FIRE

As noted in the introduction to this paper, BCA verification method BV2 permits the robustness of building structures to be tested by removing any individual column or other significant supporting member and demonstrating that disproportionate failure cannot occur (BV2(a)). The expectation is that such an assessment need only be done at ambient temperature. If BV2(a) can be demonstrated as being achieved under ambient conditions, is this sufficient to demonstrate *structural robustness* under fire conditions? This is designated as Question A. This question cannot be answered without considering a further question: under what circumstances will meeting the robustness test of BV2(a) under *fire conditions* prove to be important? This is designated as Question B. The answer to this question is considered first.

For buildings in which the structural members are designed to meet the prescriptive measures of the BCA relating to fire resistance levels (FRLs), BV2(a) is relevant to a greater or lesser extent in the following situations:

(i) Where defective or inadequate fire protection has been applied to *a* member. Although there is a level of quality control with the application of fire protection or placement of reinforcement to get a particular concrete cover, there is a level of uncertainty and it is remotely possible that fire protection may not have been applied correctly (or at all) to a particular member. This is considered to be the exception rather than the rule. In the event of a fire, loss of this member will not result in catastrophic failure of the building if the building has been designed to be robust with respect to considering the effect of removing a column. However, if the fire protection material has a systemic fault then meeting the requirements of BV2(a) will not prevent disproportionate collapse. Robustness is not meant to cope with systemic errors such a generally defective fire protective coating. Such errors must be minimised by adequate quality control.

(ii) Fire in part of the building that is more severe than can be resisted by a member protected to achieve the FRL required by the prescriptive provisions. If this greater than expected severity is *localised* to the vicinity of a particular member, then loss of that member may not lead to disproportionate collapse if the building has been designed to be robust under fire conditions. If, however, the severity is greater in the vicinity of several members (e.g. more than one column) then building collapse may occur.

(iii) A fire resulting from a significant destructive act (terrorist act). A building designed to be robust under fire conditions will behave better under such circumstances than one for which robustness has not be considered. However, whether the building suffers disproportionate collapse or not, will depend on the magnitude of the terrorist act (bigger aircraft or missile), the number of members lost due to impact and the number of levels experiencing severe fire exposure.

Question A is now considered for buildings in which the structural members are designed to meet the prescriptive measures of the BCA relating to fire resistance levels (FRLs).

Given sufficient time, the temperatures of the surrounding members on the level of fire origin increase and this results in both expansion and reduced strength and stiffness. For example, the free expansion associated with a 12m length of steel beam subject to an increase in temperature of 450°C above ambient is 60mm. If this expansion is restrained, compressive forces will develop in the beams and be applied through the end connections. In that case it will be important to ensure bolt shear failure under compression does not occur. Compressive forces in the lower flanges of steel beams, once contact is made with the supporting column, can result in local buckling of the beam flange or crippling of the column web. Only the latter failure mode is of concern but its occurrence depends on the thickness of the web. These are matters relevant to the detailing of steel and composite construction and are not relevant for other forms of construction such as reinforced or prestressed concrete.

Provided connection details are chosen to avoid the shearing of bolts and crippling of the webs of supporting columns (steel construction issues only), and it can be shown that BV2 (a) can be achieved for <u>ambient</u> conditions, then it can be assumed that the building can also survive the loss of a column under fire conditions.

Reference [20] discusses the issue of robustness from different perspectives and notes that measures other than *structural robustness* can and should be considered with respect to achieving a robust building design. It also recognises that a more robust building is a lower risk building. From a fire perspective this could mean that for certain buildings, a greater fire resistance may be considered for certain members than is ordinarily required and/or that any applied fire protection material should have a greater level of mechanical resistance if it is considered likely to be subject to blast or impact.

Current building regulations require mid and high-rise buildings to be designed such that structural members including the floors achieve a prescribed FRL with each level being designed to be a compartment and separated from the one above by fire-resistant floors. In Australia, fire-resistant edge spandrels are only required if buildings are not sprinkler protected. Sprinkler protection is required for buildings with an effective height greater than 25m. Disproportionate collapse can only occur in the event of sprinkler failure since only unsprinklered fires are capable of significantly heating structural supporting members. If sprinkler system failure occurs, it is more likely that disproportionate collapse could occur since fire will be more likely to spread to levels above the level of fire origin, especially if the edge spandrel is non-existent or ineffective.

A more robust building solution from a fire perspective should incorporate a *robust sprinkler system* which will have the following characteristics:

(a) Each level will incorporate a separate monitored isolation valve. This, in association with an appropriate sprinkler management protocol will increase the reliability of the system and therefore further reduce the likelihood of having a fire capable of causing member failure.

(b) The sprinkler system should be capable of delivering water at the required flow and pressure to significantly more sprinkler heads than the minimum required by AS 2118 [21] for the occupancy class in order to reduce the likelihood of fire spreading between levels should the sprinkler valve be closed on the level of fire origin due to building works on that level.

For buildings with open-plan floors, consideration should be given to incorporating effective spandrel panels at the edges of medium and high-rise buildings and paying particular attention to gaps at the edges of floor through which fire could pass to the next level in the event of sprinkler failure.

6 CONCLUSIONS

NCC Performance requirement BP1.1(a)(iii) requires buildings to be designed to be structurally robust. Verification method BV2 provides guidance and greater clarity with respect to how robustness should be demonstrated.

The issue of structural robustness in fire has been raised in response to the performance of buildings under either terrorist attack or other major fires. This paper has reviewed some of these incidents to better understand the key contributing factors to the observed performances and any lessons with respect to robustness. Structural robustness has also been the focus of much research into the behaviour of steel and composite buildings with little or no fire protection and key findings from this research are also considered.

This paper considers whether buildings can be considered to be robust under fire conditions if they satisfy the design criteria of BV2 in addition to the requirements of Section C (fire-resistance). It is concluded that demonstrating that BV2(a) (loss of column test) can be achieved under ambient conditions is sufficient evidence that a column can also be lost due to fire exposure conditions without causing disproportionate collapse provided the connection details are chosen to avoid the shearing of bolts and crippling of the webs of supporting columns as a result of compression forces induced by restraint of expansion.

The importance of fire-safety systems such as sprinkler systems is noted in relation to their effect on reducing the risk of member failure and therefore improving structural robustness. Suggestions are made as to how the robustness of a building can be further improved through improvements to the design of the sprinkler system and possibly also through the provision of fire-resistant edge spandrels even though these are not currently required by the prescriptive provisions of the NCC for buildings with an effective height greater than 25m.

7 REFERENCES

[1] Australian Building Codes Board, National Construction Code Series 2015, Vol 1, Building Code of Australia class 2 to class 9 buildings

[2] HM Government, The Building Regulations 2010 Structure, Approved Document A (incorporating 2013 amendment), United Kingdom

[3] EN 1991-1-7 (2006) Eurocode 1: Actions on Structures – Part 1-7: General actions – Accidental actions, July 2006

[4] Scott, D. Lane, B. and Gibbons, C. “Fire Induced Progressive Collapse”, http:/www.nibs.org/resource/resmgr/mmc/wppc_scott_paper.pdf

[5] Johann, M., Rini, D. and Lane, B., “Structural Design for Fire Conditions”, Modern Steel Construction, April 2010

[6] National Institute for Standards and Technology (NIST), “Final Report on the Collapse of World Trade Centre Building 7”, US Department of Commerce, Nov 2008

[7] Gilsanz, R., Chapter 5 WTC7 in “World Trade Center Building Performance Study”, Federal Emergency Management Agency (FEMA)

[8] Mancada, J.A., “Fire Unchecked”, NFPA Journal, Mar – Apr 2005, pp 47 – 52

[9] Ali, M.M. and Moon, K.S., “Structural Developments in Tall Buildings: Current Trends & Future Prospects”, Architectural Science Review, Vol 50.3, pp 205-223, 2007

[10] Bailey, C.G., “The Windsor Tower Fire, Madrid” [available from http://www.mace.manchester.ac.uk]

[11] Wald et al, “Experimental Behaviour of Steel Joints under Natural Fires”, Connections in Steel Structures V, Amsterdam, June 3-4, 2004

[12] Ding, S., “Design for robustness of connections to concrete filled tubular columns in fire”, The Structural Engineer, 7, October 2008

[13] Burgess, I., “The Robustness of Steel Construction in Fire”, 9th International Conference on Steel Concrete Composite and Hybrid Structures, Leeds, UK, July 2009

[14] Wang, Y.C., Dai, X.H. and Bailey, C.G., “An experimental study of relative structural fire behaviour and robustness of different types of steel joints in restrained steel frames”, Journal of Constructional Steel Research, 67, 2011, pp1149 – 1163

[15] Fang, C., Izzuddin, B.A., Elghazouli, A.Y. and Nethercot, D., Fire Safety Journal, Vol 46, Issue 6, Aug 2011, pp 348-363

[16] Fang, C., Izzuddin, B.A., Elghazouli, A.Y. and Nethercot, D.A., “Robustness of multistorey carparks under localised fire – Towards practical design recommendations”, Journal of Constructional Steel Research, 90, 2013, pp 193 – 208

[17] Wang, M. and Wang, P., “Strategies to increase the robustness of endplate beam-column connections in fire”, Journal of Constructional Steel Research, 80, 2013, pp109 – 120

[18] Demonceau, J.F., Huvelle, C. et alia, “Robustness of carparks against localised fires (Robustfire)”, European Commission Report No. EUR 25864EN, Final Report, 2013

[19] Da Silva, L.S., Sandiago, A. et alia, “Design of composite joints for improved fire robustness”, European Commission, Report No. EUR26686EN, 2014

[20] Conisius, T. (Ed), ETH Course Notes, “Robustness of Structures: Structural Robustness Design for Practising Engineers”, Cost Action TU061, European Cooperation in Science and Technology, September 2011

[21] Standards Australia, AS 2118.1 – 2006, “Automatic fire sprinkler systems – General Systems