

## VULNERABILITY ASSESSMENT OF CONCRETE TALL BUILDINGS SUBJECTED TO EXTREME LOADING CONDITIONS

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### Abstract

This paper presents a vulnerability/survivability assessment procedure based on the analysis of a typical tall building in Australia. The structural stability and integrity of the building was assessed by considering the effects of the failure of some perimeter columns, spandrel beams and floor slabs due to blast overpressure or impact. The criterion of the analysis is to check if failure of any primary structural member will cause progressive collapse propagating beyond one story level above or below the affected member vertically, or to the next primary structural member vertically. The overall stability of the structure will rely on continuity and ductility of these elements to redistribute forces within the structure. The authors suggest some methods to improve the impact resistance of concrete walls and slabs, as well as the rotation capacity of the beams, columns and joints which have been found from this study of crucial importance for collapse prevention.

**Keywords:** Tall buildings, Concrete, Blast, Impact, Progressive collapse, Extreme loadings

### 1. Introduction

Progressive collapse is characterized by the loss of load-carrying capacity of a relatively small portion of the structure due to an abnormal load which can trigger a cascade of failures affecting a major portion of the structure. Several buildings have collapsed in this fashion such as the Murrah building in Oklahoma (1995) and the recent collapse of the World Trade Centre (2001). Lessons learnt from these events were that special attention must be given to the behavior of the structural elements to improve their redundancy, toughness, and ductility under extreme events. The ultimate goal of the protection is to minimize injuries and loss of life and facilitate the evacuation and rescue of survivors. The casualties that will occur to occupants in the immediate vicinity of the explosion or impact may be unavoidable, but by preventing progressive collapse, the remaining occupants may be spared injury or death.

An ongoing study on the performance of typical Australian tall buildings under the extreme events has been carried out at the University of Melbourne. Several extreme event scenarios involving a bomb blast or an aircraft impact were identified, and their effects were investigated. A typical floor of each building was examined to determine the vertical load capacity of the beam-slab system. The objective of this study is to develop a preliminary method of assessing the structural consequences of extreme event impacts with focus on progressive collapse prevention and suggest design directions for enhancing the performance of existing and future buildings. A vulnerability assessment procedure has been proposed, which consists of three main steps: (i) Determination of hazard levels and load conditions, (ii) Global and local damage assessment, and (iii) Progressive collapse assessment.

### 2. Determine hazard levels – Extreme load cases

#### 2.1 Determine hazard levels – Performance based approach

The technical hazards to tall buildings may range from an accidental gas explosion to a car bomb, an impact of a missile to a jet airplane collision. For these assaults, the source can originate either external or internal to the structure.

The difference between technical hazards (accidental or terrorist) and other natural hazards is that the risks of technical hazards are very hard to quantify. For these types of hazards the performance-based approach can be used as a rational method for assessment or design of buildings against extreme events. Example of the performance level – hazard matrix of a bomb blast event is shown in Figure 1.

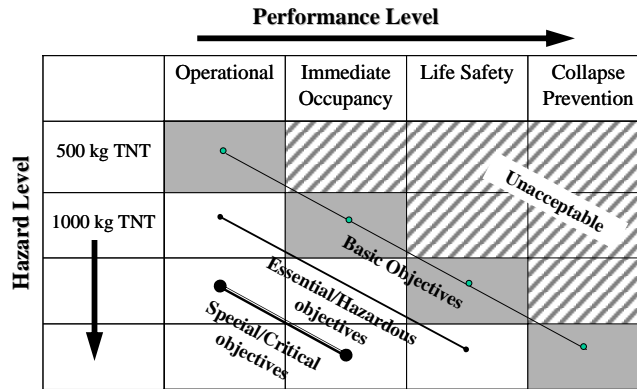


Fig. 1 Performance-based approach

2.2 Blast loading

The threat for a conventional bomb is defined by two equally important elements, the bomb size (or charge weight (W), which is normally measured using the equivalent amount of TNT), and the standoff distance (R) between the blast source and the target. For example, the blast occurred at the basement of World Trade Centre in 1993 has the charge weight of 816.5 kg TNT. The Oklahoma bomb in 1995 has a charge weight of 1814 kg at a stand off of 4.75m (Longinow, 1996).

With the detonation of a mass of TNT at or near the ground surface, the peak blast pressures resulting from this hemispherical explosion decay as a function of the distance from the source as the expanding shock front dissipates with range (Fig. 2). The incident peak pressures are amplified by a reflection factor as the shock wave encounters an object or structure in its path. The reflected pressure is at least twice that of the incident shock wave and is proportional to the strength of the incident shock, which is proportional to the charge weight. The blast pressure decays exponentially and eventually becomes negative as shown in Fig. 3. This then subjects the building to pressures acting in the direction opposite (suction pressure) to that of the original shock front. Peak blast loads may be several orders of magnitude larger than the largest loads for which conventional buildings are designed (Table 1).

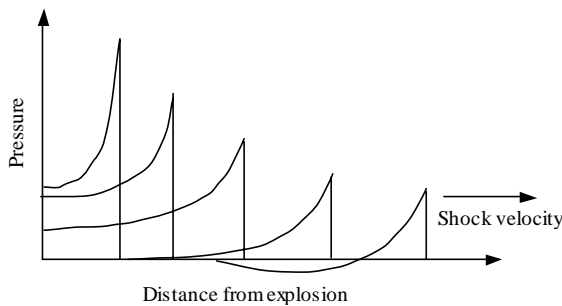


Fig. 2 Variation of pressure with distance

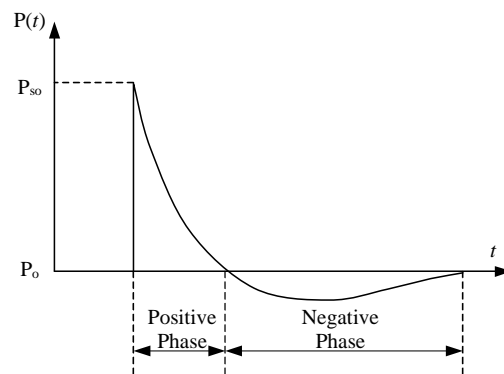


Fig. 3 Blast wave pressure – Time history

Table 1. Peak reflected overpressures (MPa) with different W-R combinations (TM5-1300, 1990)

R \ W	100kg TNT	500kg TNT	1 ton TNT	2 ton TNT
1m	165.8	354.5	464.5	602.9
2.5m	34.2	89.4	130.8	188.4
5m	6.65	24.8	39.5	60.19
10m	0.85	4.25	8.15	14.7
15m	0.27	1.25	2.53	5.01
20m	0.14	0.54	1.06	2.13

### 2.3 Aircraft impact loading

Design loads resulting from aircraft impacts are governed by the absorption of kinetic energy from the aircraft by the building at its maximum deflection. These loads are limited by the yield, buckling and crushing of the aircraft. Total impactive load  $F(t)$  at the interface of the collapsing aircraft and the building is given by (Kar, 1979):

$$F(t) = F_c + \mu [m(t)] V(t) \tag{1}$$

in which  $m(t)$  is the mass of the aircraft reaching the building per unit time;

- $\mu$  is a coefficient for change in momentum (which can be taken conservatively as 1);
- $F_c$  is a constant which can be determined from the design acceleration for failure of the aircraft;
- $V(t)$  is the velocity of the aircraft.

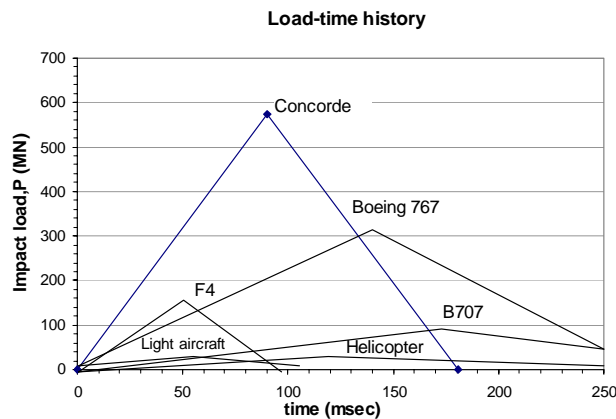


Fig. 4 Impact load-time history for aircraft impacts

The frame is classed as a soft missile which will suffer considerable deformation and a finite difference method of calculation is employed to describe its perfectly plastic impact. The engines which are considered separately are assumed to constitute a much harder missile which will undergo little deformation. Fig. 4 compares the impact loads produced by different aircrafts. The peak loads and impact durations are given in Table 2. More details are given elsewhere (Mendis & Ngo., 2002).

Table 2. Examples of aircrafts and peak impact loads

Aircraft	M (tonne)	L (m)	$V_o$ (m/s)	Peak Load (MN)	Duration (ms)
Aust. SUPAPUP Light Aircraft	0.34	5.7	51.3	4.6	111
Westland Sea King Helicopter	9.5	17	63.9	19.6	266
Boeing 707-320	91	40	103.6	92	386
Phantom F4 aircraft	22	19.2	210	145	91
Boeing 767-300 ER	187	54.9	140	320	362
Supersonic Concorde	138	62.2	344	568	181

## 3. Case study – Global and local damage assessment

### 3.1 Structural configuration

A 52 storey building (modified from a typical tall building designed in Australia) was analyzed in this study. The plan view and structural configuration of the building are shown in Fig. 5. The typical story height is 3.85m. Perimeter columns are spaced at typical 8.4m centers and are connected by spandrel beams to support the facade. The lateral loads are resisted by 6 core boxes located at the centre of the structural plan. The building is designed to resist lateral loads due to wind and seismic ground motion specified by Australian Loading Standards AS1170.2 and AS1170.4. The slab, columns and core walls are all cast-in-place concrete. High-strength concrete is used for columns and core walls at the lower stories (see Table 3). The lateral load resistance system (LLRS) of the building relies mainly on the lateral load capacity of the core walls which account for about 80% the overall capacity.

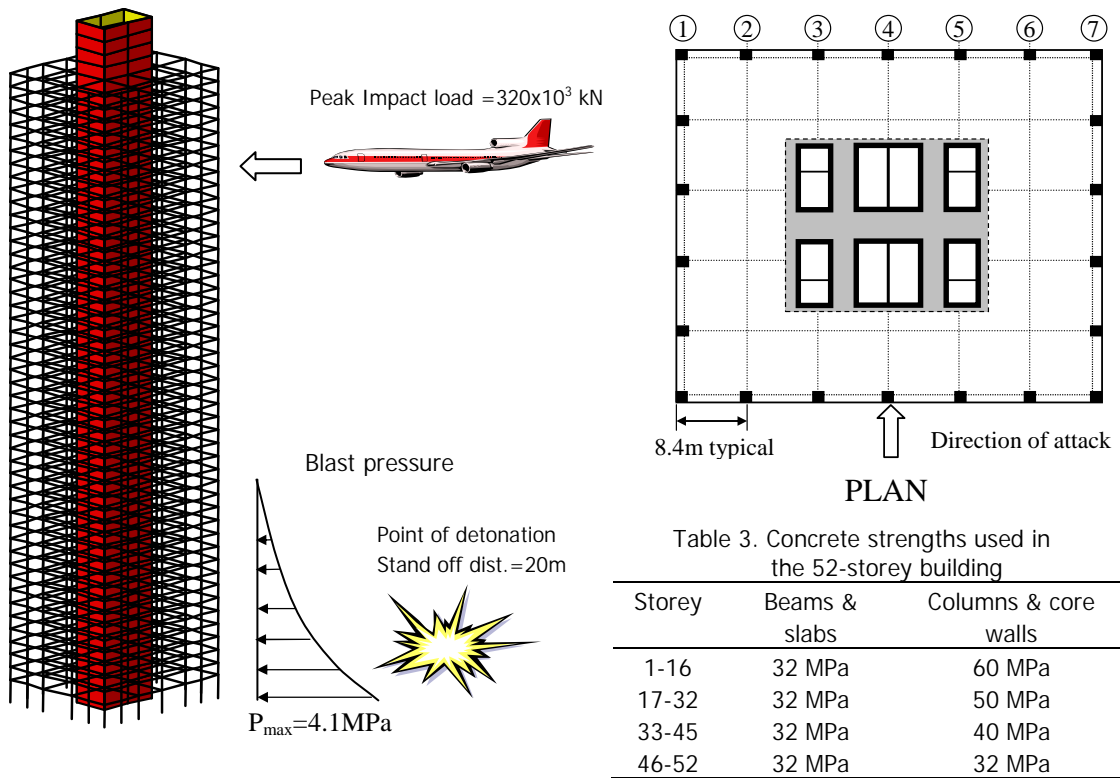


Fig. 5 Structural configuration

3.2 Non-linear time-history analysis (global assessment)

The global stability of the example building (Fig. 5) was analyzed using program RUAUMOKO-3D (Carr, 2001) which takes into account both material nonlinearity and geometric nonlinearity (P-delta effect). The effects of blast and aircraft impact were considered in the nonlinear time history analysis to obtain the dynamic load-deflection responses of the building. Two lateral extreme load cases as shown in Fig. 6 were modeled as time-history force functions. The blast load was calculated based on data from the Oklahoma bomb (Mlakar et.al., 1998) with a stand off distance of 20m. The peak overpressure is 4.1MPa at the ground level and reduces rapidly up the height of the building (Fig. 6). The average duration of loading was adopted as 15 milliseconds. The triangle shape of the blast load profile was used as the first push-over case (see Fig. 7). The impact of a Boeing 767 at level 40th was modeled as a concentrated load with the peak impact load of 320,000 kN (see Table 2). The duration of the impact was estimated as 0.36 seconds.

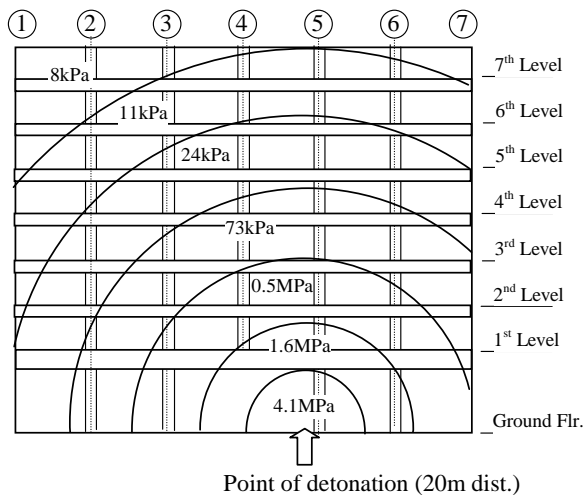


Fig. 6 Distribution of Peak Pressure

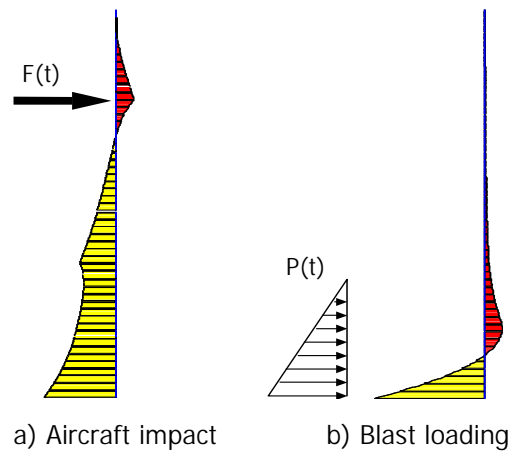


Fig. 7 Bending moment diagrams of core walls

For reinforced concrete structures subjected to blast or impact effects, response at very high strain rates (up to  $1000 \text{ s}^{-1}$ ) is often sought. At these high strain rates, the strength of concrete and steel reinforcing bars can increase significantly. For concrete, the ratio of the dynamic to static strength can

be up to 3 in compression, and 6 in tension. In this case study, the CEB-FIP-1990 model for strain rate enhancement of concrete was adopted.

Fig. 8 shows the relationship between the fundamental behaviour of the kinetic energy, strain energy, energy dissipated through normal damping and plastic energy (energy dissipated through permanent deformations) due to aircraft impact. The push-over curve of impact load case is plotted in Fig. 9. As seen the global lateral load capacity of the example building are adequate to withstand these loadings. Similar analyses confirmed that the example building can withstand the blast loads globally.

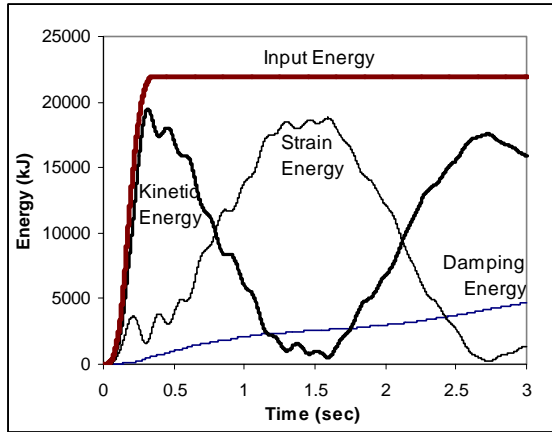


Fig. 8 Energy time history (Aircraft impact)

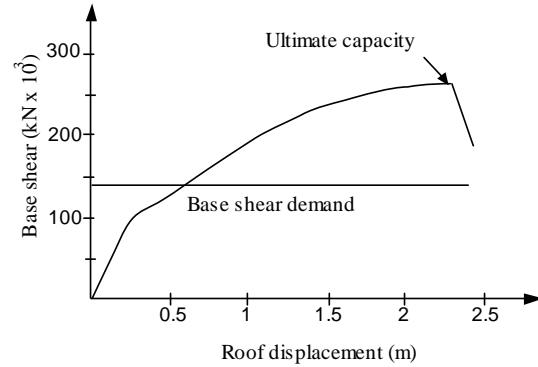


Fig. 9 Push-over curve of the aircraft impact.

### 3.3 Local damage assessment

The local damages caused by a bomb blast or an aircraft impact were assessed in order to determine the likelihood of progressive collapse to the example tall building. That included the assessment of key structural elements (columns, core walls and spandrel beams) being struck by blast pressure or aircraft parts such as engines and wings, using the finite element explicit code in LS-DYNA (2001) (see Figs. 10, 11). As aircraft engines and undercarriages are the heaviest components, they pose the greatest threat to structural elements. For column sizes commonly used in tall buildings more attention should be focused on the local resistance to impact and the design of column splices. Detailed analysis of the performance of a concrete column subjected to a bomb blast (Fig. 11) is given elsewhere (Ngo et.al., 2003).

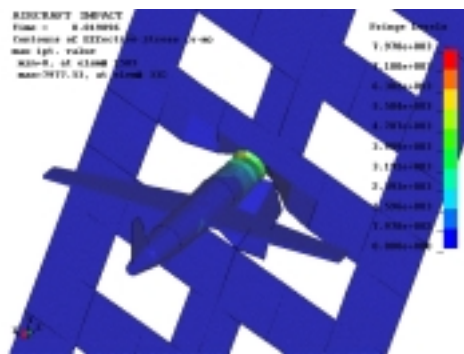


Fig. 10 Local damage of a light aircraft impact

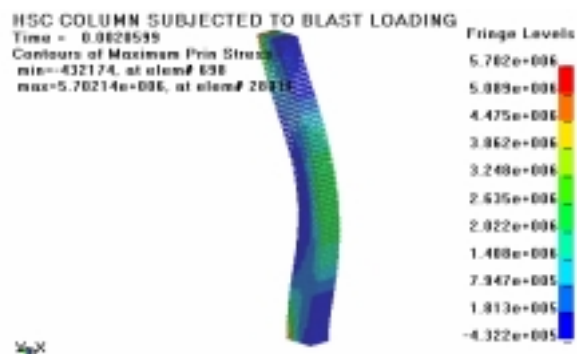
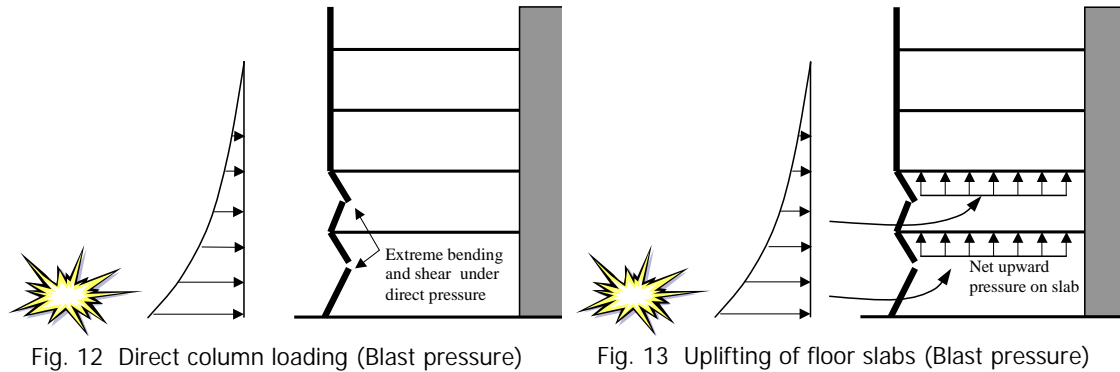


Fig. 11 Ground floor column subjected to a bomb blast

## 4. Progressive collapse analysis

Design recommendations on progressive collapse analysis have been introduced in British Standards since 1968, after the collapse of 22-storey Ronan Point apartment building. In recognition of this issue, a number of European countries, USA and Canada have incorporated progressive collapse provisions in their building codes. The American National Standards Institute (ANSI) Standard A58.1-1982, "Minimum Design Loads for Buildings and other Structures" recommends the alternative path method, in which the local failure is allowed to occur but an alternative path must be provided around the failed structural elements. It should also be noted that there are no provisions or recommendations in the current Australian standards with regard to progressive collapse.

In this study, based on the local damage assessment due to the two terrorist load cases progressive collapse analyses were performed on the example building. The structural stability and integrity of the building were assessed by considering the effects of the failure of some perimeter columns, spandrel beams and floor slabs due to blast overpressure or aircraft impact. The main purpose of the analysis is to check if failure of any primary structural member will cause progressive collapse propagating beyond one story level above or below the affected member vertically, or to the next vertical structural member.



Figs. 12 & 13 show the effect of direct blast pressure on perimeter columns, beams and floor slabs. The concrete slabs in this example building are 125mm thick supported by prestressed wide band beams. The portion of floor slabs in close proximity to the blast were directly hit by the blast overpressure. The normal glazing façade offers insignificant resistance to the blast wave so after the failure of glazing, the blast fills the structural bay above and below each floor slab. The pressure below the slab is greater than the pressure above and causes the net upward load on each slab (Fig. 13).

To detect the local damage, the blast analysis was carried out for perimeter columns, beams and floor slabs based on the actual blast pressures on each element. Results plotted in Fig. 14 show column lines 4, 5 of the ground and 1st levels failed due to the direct impact of the blast wave. Slabs and beams from column line 3 to 6 also collapsed. Member assessment was carried out using program RESPONSE (2001) based on the Modified Compression Field theory. More details are given elsewhere (Mendis & Ngo, 2002). The calculation also showed that if the columns were detailed using requirements for special moment resisting frames (SMRS) as given in ACI-318 Section 21, the shear capacity and the ductility would be improved significantly, thus improving the blast and impact resistance of the member. The damaged model of perimeter frame, in which failed elements were removed (Fig. 14), was analyzed to check if progressive collapse would propagate beyond one story level above or below.

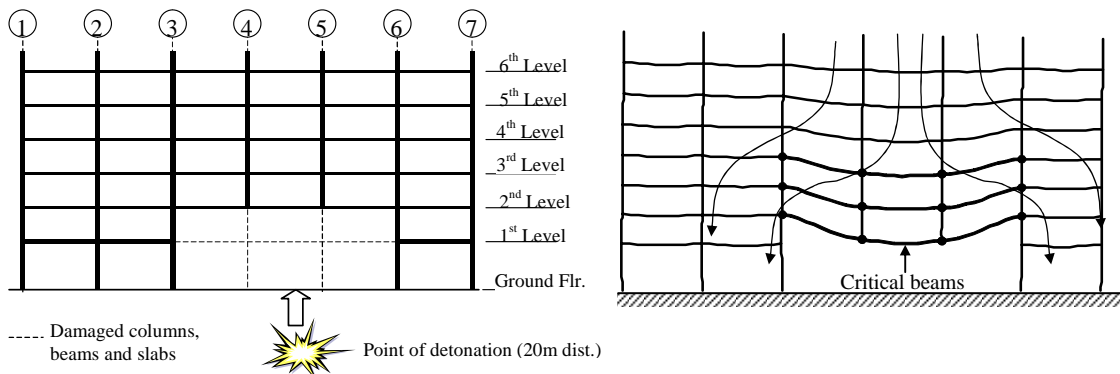


Fig. 14 Progressive collapse analysis of perimeter frame (damaged by blast load)

The progressive analysis was also carried out for the impact of aircraft. In this case, it was assumed that columns in three consecutive levels (level 39-41) were damaged by direct impact. As seen from Figs. 14 & 15, the alternative load paths go through columns surrounding the damaged area where the vertical loads are transferred. Beams and floor slabs above that area become critical due to the loss of supporting columns. The overall stability of the structure will rely on continuity and ductility of these elements to redistribute forces within the structure. The falling debris of the collapsed members also imposes severe loads on the floors below (Fig. 15). It is essential to check whether that overload can be carried without causing further collapse.

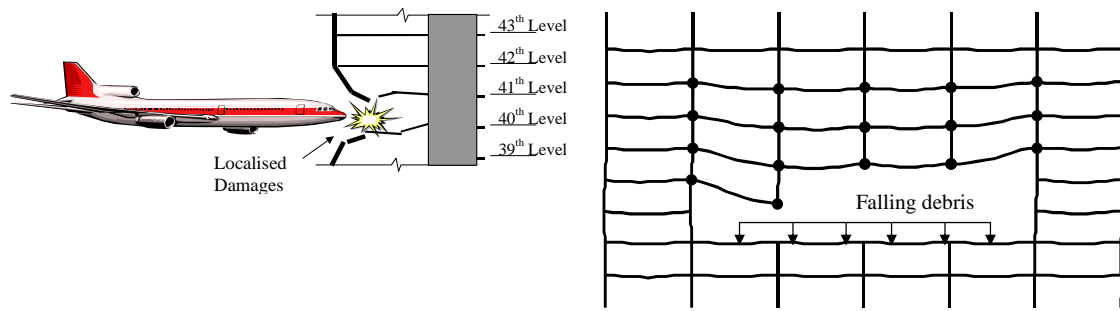


Fig. 15 Progressive collapse analysis of perimeter frame (damaged by aircraft impact)

The high temperature of burning jet fuel also results in the spalling and softening of concrete members which may lead to further instability of the structure. In addition, high-strength concrete has been found to be more prone to spalling failure than normal-strength concrete when exposed to relatively rapid heating (Phan et.al., 2001). The effect of fire will be investigated in a future study.

Parametric studies were also undertaken to investigate the impact resistance of the floor slab, assuming a floor above had collapsed onto it. The collapsing floor was treated as falling debris, i.e. the energy applied to the floor below was the weight of the collapsing floor multiplied by the height through which it fell. To obtain an estimate of the impact load-bearing capacity of the floor slab, the structure was analyzed using a dynamic nonlinear finite element analysis (DNLFEA) procedure. In addition to material and geometric nonlinearities, the analyses considered membrane action, inertia effects, and other influencing factors. The results show that the ultimate capacity of the floor slab is approximately 16.5kPa which is 2.75 times the total floor load (dead load plus 0.4 live load). Therefore in this case study if more than two floor collapsed (Fig. 15), the falling debris of the collapsed members may impose an overload for the floor below and trigger a progressive collapse of the example building.

## 5. Discussion

The structural integrity of the example building depends greatly on the ability of structural members to deform inelastically under extreme overload, thereby dissipating large amounts of energy, prior to failure. Similarly, the prevention of progressive collapse cannot be guaranteed unless beams and columns in the immediate regions around the collapse area possess very high rotation capacities required due to excessive loads (Figs. 14 & 15).

Results from the progressive analysis shows that by increasing the ductility of the floor slab connections the floor's ability to absorb energy will increase through larger deformation before it collapses, thus reducing the amount of energy imparted to the floor below when it does collapse. This study confirmed that increasing ductility through the design and detailing whilst maintaining strength gives a more robust structure. Therefore, member ductility rather than strength is the key factor to prevent collapse of the structure. Hence detailing the spandrel frame as a SMRF would be an option to improve the robustness of tall building structures.

In this case study, a parametric study was conducted for the 52-storey building to investigate the influence of the beams, columns and core walls ductilities on the global ductility. As mentioned in Section 3, high strength concrete (HSC) was used for core walls and columns at lower stories. It has been found that vertical members such as columns and shear walls which carry high axial loads can have a marked reduction in cross section size and the amount of longitudinal steel reinforcement can be substantially reduced when high strength concrete is used (Park, 1998). However, the main concern regarding the use of high strength concrete is the reduction in ductility with the increase in compressive strength observed under uniaxial compression (Mendis, 1999).

Moment-curvature analysis was carried out to find the curvature ductility of the critical regions of concrete members. Fig. 16 shows the moment-curvatures diagrams for a typical spandrel beam and a column. Obviously, spandrel beams possess much higher curvature ductility compared to that of columns. Results from the progressive collapse and curvature analyses showed that for the aircraft impact, the rotation and curvature demands exceed the capacity of the critical beams and columns around the collapse portion of the building. Hence, further collapse of these members may trigger a cascade of failures.

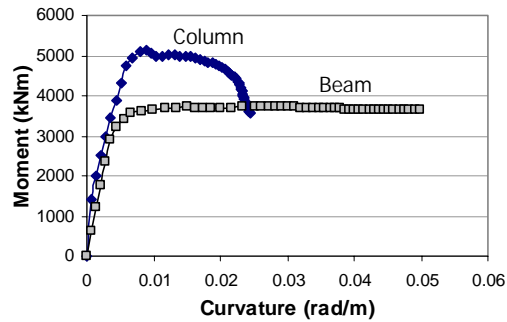


Fig. 16 Moment-curvature diagrams of beams and columns

### Concluding Remarks

For high-risks facilities such as public and commercial tall buildings, design considerations against extreme events (bomb blast, high velocity impact) is very important. It is recommended that guidelines on abnormal load cases and provisions on progressive collapse prevention should be included in the current Building Regulations and Design Standards. Requirements on ductility levels also help improve the building performance under severe load conditions. This study is continuing at the University of Melbourne.

### Acknowledgements

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