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8 Chifley – Sustainable Structural and Fire Engineering

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Andrew is a Principal of Arup and structural engineer with a passion for design integrating innovation in architecture and structural efficiency to create better buildings. Andrew's structural expertise includes tall buildings across a broad cross-section of regions and environments including Australia, Asia, Middle East, and the UK. Many of these projects have been delivered as integrated multi-disciplinary teams, allowing highly integrated and effective building solutions. Andrew also has significant experience on long-span structures for stadia and exhibition buildings.

Abstract

8 Chifley's unique aesthetic has attracted much of the attention, however the 34 storey 150m tall building's overall quality, sustainable performance, and structural efficiency, both in construction and in final form, are similarly significant. Realizing a project of this vision required innovation in design technology throughout construction. The first building to be completed in Australia by renowned architect Rogers Stirk Harbour & Partners in conjunction with Lippmann Associates, 8 Chifley possesses a number of unique features. These elements of the building and the engineering solutions that facilitated them directly reduced capital cost, and increased lettable area – without which the project may not have been commercially viable for the owner or appeal to the tenant market.

Keywords: Dematerialization, Fire Safety, Structural Engineering, Sustainability

The north facing side core tower is uniquely located at the end of a city block of less than 1,300m^2, with streets on three sides. Unusual for contemporary urban sites, the building was governed by maximum allowable floor area as opposed to maximum height; therefore an innovative solution to office floor plate efficiency and maximizing lettable area was paramount. The proposal at competition stage for open-air fire stairs realized a concession in the planning regulations at the time, meaning that the fire stair area was not deducted from the net lettable area – which would have been the case if the stairs had been fully enclosed.

Highly efficient open plan floor plates in two "stacked" commercial modules of 9 and 12 floors respectively, are raised at both street and mid-level on expressed jump structures. Each primary module is further subdivided into a number of three and four storey "vertical villages" within these modules. Each village comprises total floor areas of up to 2,700m^2 to maximise the ability to attract tenants that might otherwise prefer large contiguous floor plates that are not available in this part of the Sydney CBD.

The use of expressed external structural steel framing - freeing up the interior space planning - with a desire for this steel structure to be legible within the architecture promoted a performance based structural design solution outside of generic codified procedures to omit fire protection, and in turn further dematerialise the building. The engineering solutions that are so integral in achieving the success of the building are on display from the moment you

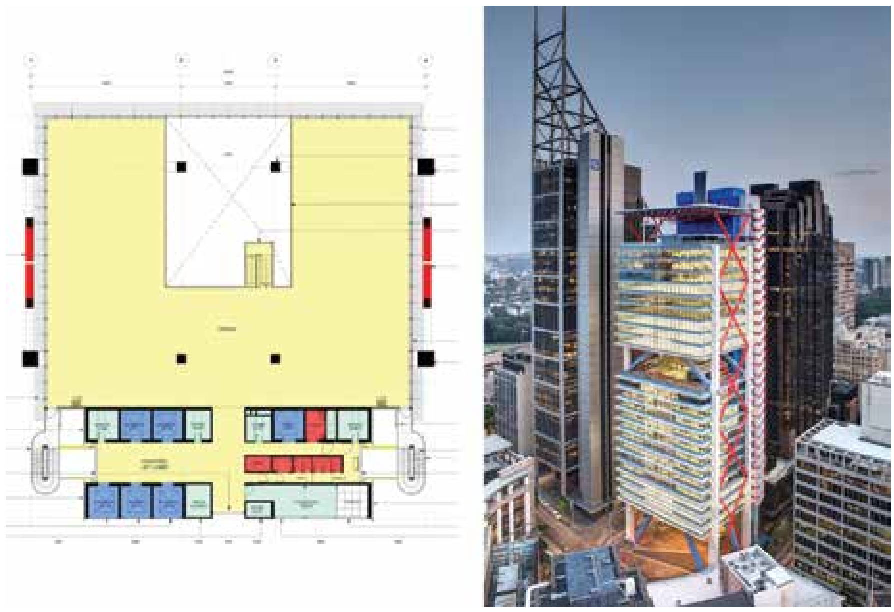

Figure 1. Typical village floor plan at architectural concept, and completed building. Only two columns are located within the floorplate on the village floors (Source: RSHP & Brett Boardman)

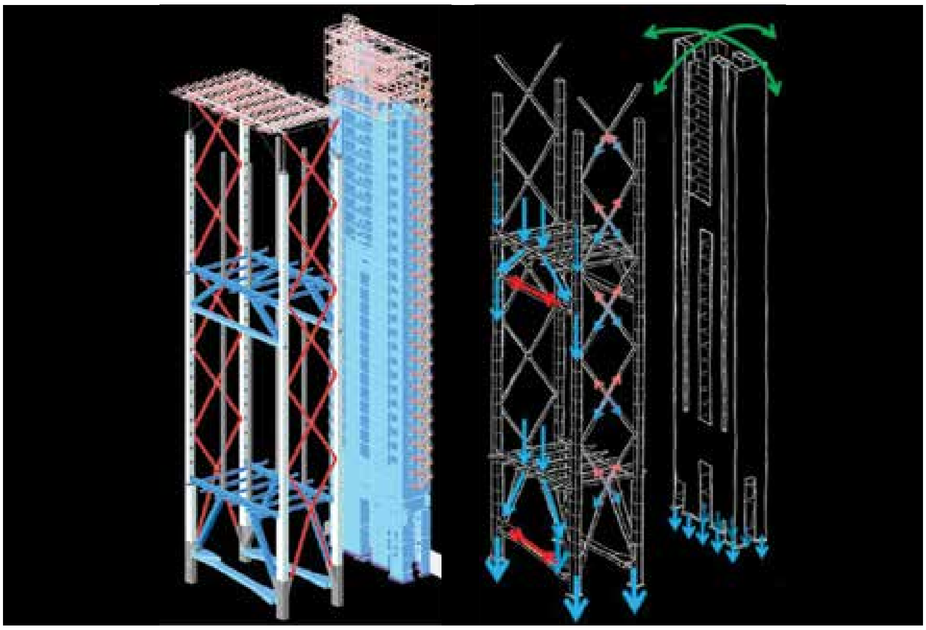

Figure 2. Stability and Gravity System (Source: Arup)

first read the building in the city skyline with the distinctive red bracing and expressed cantilevered fire stairs. This only increases as you approach the main lobby - a floating transparent frameless glass box utilizing performance based fire engineering to assist minimizing the visual obstruction of supporting framing.

Figure 1 indicates a typical office village floor, and a view of the completed building.

Structural System

The structural system is a hybrid assembly comprising insitu concrete, precast concrete, composite steel and concrete, and structural steel frame. Each of the structural materials were selected based on a combination of structural performance requirements, compatibility with overall design intent, cost-effectiveness, buildability, and speed of construction (see Figure 2 & 3).

The gravity and stability system comprises:

- Four external insitu concrete megacolumns on an 18m by 37.5m grid. The insitu concrete is poured inside permanent precast shell formwork located outside the building façade. At foundation level these columns support all of the office floor gravity load due to the two primary transfer structures;
- Steel cross-bracing in the north-south direction between the external megacolumns. Below ground level the bracing transitions to reinforced concrete shear walls between the mega-columns. These orthogonal braced frames resist over 60% of north-south building overturning at the base, and up to 95% over the top half of the building (see Figure 4);
- Four interior columns are located in each of the two stacked modules, providing clear 12m wide floor planning zones on each wing. These columns are transferred to the external megacolumns at the Level 18 and Level 6 jump structures, maximizing gravity loads on the east and west braced frames;
- Expressed composite steel transfer columns, post-tensioned ties, and composite steel and precast concrete jump-start decks from Ground to Level 6, and Level 18 to Level 21. The inclined columns and primary beams are filled with reinforced concrete for the fire load case. The secondary beams and their connections are not provided with passive fire protection, and behave compositely with the precast and insitu slab;
- A reinforced concrete southern core, comprising of the lift and service risers. This core was proportioned primarily for gravity loads and east-west stability;
- Post-tensioned beam and reinforced concrete slab typical floor structures. Secondary beams are 725mm deep at 6m spacing and span 18m onto 800mm deep primary beams. The secondary beams cantilever a further 4.75m to the northern façade. A 150mm thick slab on permanent metal deck formwork spans 4.8m clear between the secondary beams;
- Steel framed external and open fire stairs, cantilevered from the concrete core;
- Composite steel and concrete multi-level plant room on top of the core;
- Structural steel and cable supported roof structure to house PV array;
- Pad foundations on sandstone of up to 8MPa allowable bearing pressure. Due to the system efficiency and proportioning, no tension is developed at foundation level. The stability system was proportioned such that the south core with minimal gravity load suffers some lift-off at ultimate limit states conditions.

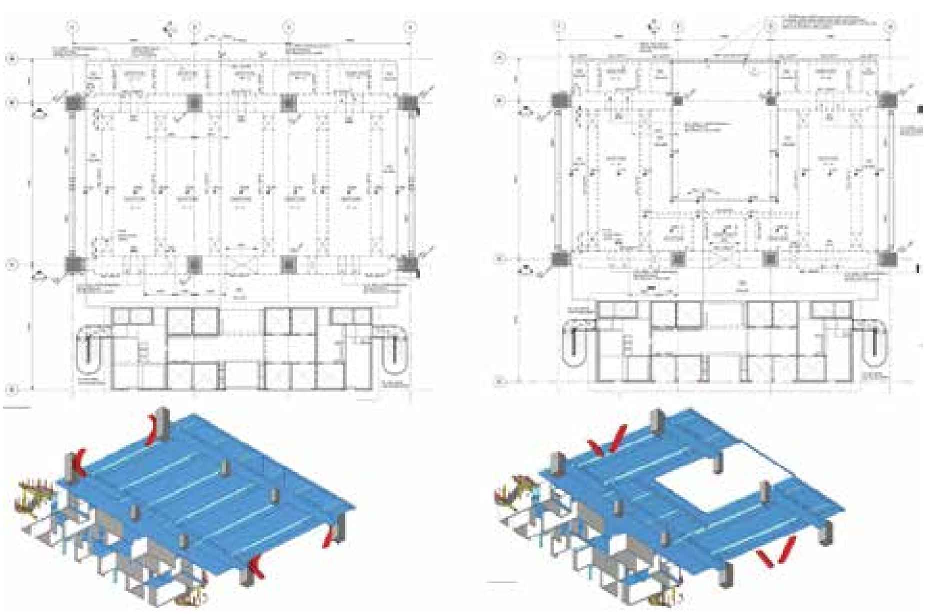

Figure 3. Typical floor structure. Full floor (left) and village floor (right). 3D views from below under (Source: Arup)

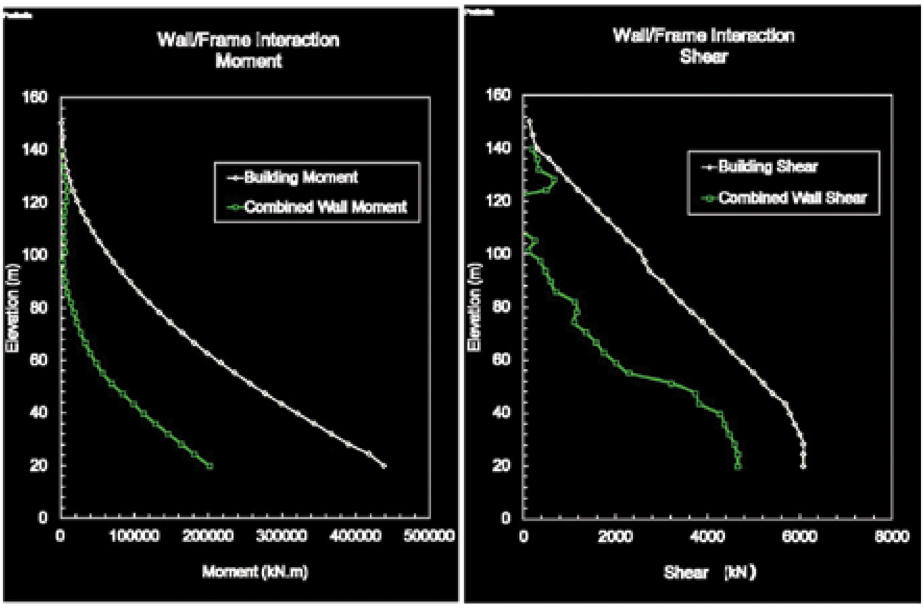

Figure 4. Distribution of moment and shear between the cores and braced frames under wind loading in the north-south direction (Source: Arup)

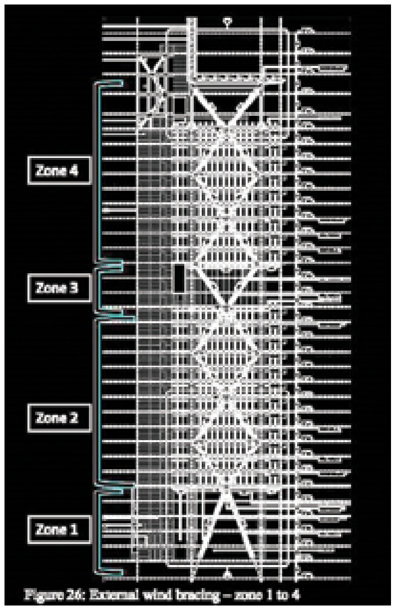

Figure 5. Fire zoning for consideration of lost bracing elements in the fire load case (Source: Arup)

Performance Fire Engineered Solutions Relating to Structure

The following elements were subject to detailed performance structural fire engineering assessment utilizing the International Fire Engineering Guidelines, and additional international standards and guidance.

- Omission of passive fire protection to steel primary wind and seismic bracing elements (megabracing);
- Omission of passive fire protection to the expressed secondary steel beams and the connections of the Level 6 reverse podium and Level 21 terrace jump structures (megaframe);
- Use of exterior unenclosed steel fire egress stairs and omission of passive fire protection to the stair structure;
- Omission of passive fire protection to steel columns supporting lightweight construction roof to glass box on Level 18 skygarden and Level 30 roof top terrace pop-up box and roof array;
- No fire rating to the stainless steel cable hangers from underside of level 6 to support glass roof to the main lobby glass box.

Megabracing

The bold red mega braces on the eastern and western elevations are a powerful design feature as well as a highly efficient stability system for this building in accommodating the eccentric core location. Running the full height of the building, the braces are geometrically set out to align with the three storey village module creating a series of six storey high ‘X’s, resisting over 60% of the building overturning moment at foundation level in the north-south direction with no net tension. The efficiency of the 18m wide braced frame width in combination with the modest building height and relatively low seismicity of Sydney, allowed bracing to be omitted from the north face to maximize views of Chifley Square and Sydney harbour. Transverse stability was provided by the southern concrete core combined with the eccentric braced frames of the jump start transfer structures, with additional torsional stability from the north-south frames.

Sitting proud of the cladding line, these external elements provided an unusual construction and design challenge. Due to the nature of the bracing forces – equally in tension and compression with no gravity load, steel was the natural material choice. However, protecting these elements from fire whilst maintaining the legibility of the material as fundamental to the architecture was key to the success of the scheme. Alternative bracing materials such as precast concrete, insitu concrete, precast concrete encased steel, and insitu filled steel elements were also considered from an early stage. Steel was finally selected based on ease of erection outside the line of the floorplate, the element function and structural size, and finally the innovative approach to performance design out the passive fire protection.

As the building contains a slender reinforced concrete core at the southern façade – selected on the basis of ease and cost of construction

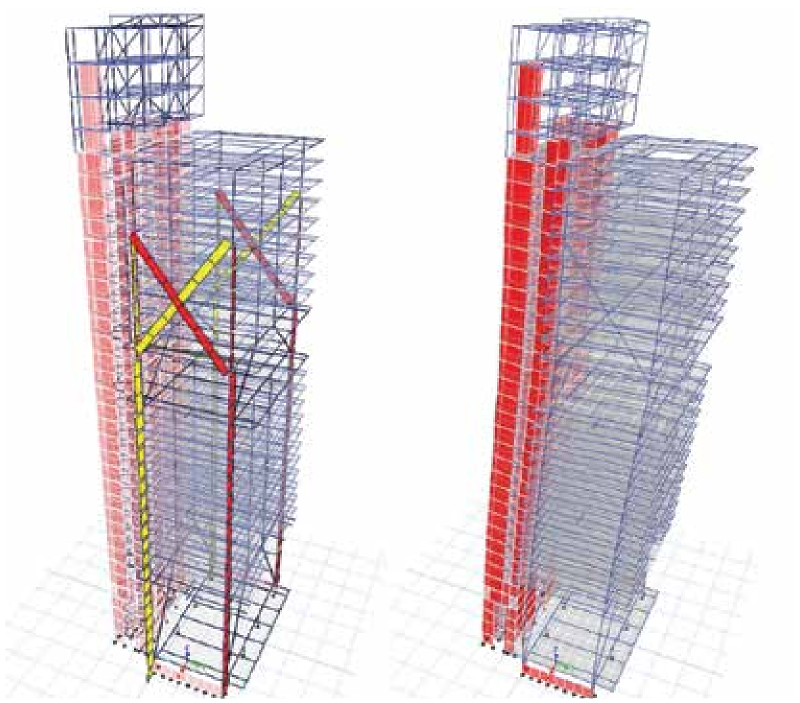

Figure 6. Structural analysis under fire load case with Zone 2 fire condition (Source: Arup)

and compatibility with the concrete floors, this element can maintain a degree of lateral stability under a fire load case where capacity may be lost in the unprotected mega-braces. Further, the geometry and use of the building allowed an analysis and assessment of real fires that naturally split the building up into a series of zones considered to be limiting based on formation and spread of fire (see Figure 5):

- Zone 1: Fire at ground level lobby or street level. Analysis of vehicular and lobby fires six levels below the first commercial floor with a clear outside reverse-podium space demonstrated that spread to the low-rise building block would not occur;
- Zone 2: Fire in the low-rise office block Level 6 to Level 18. It was considered that a typical office fire commencing at Level 6 could spread to Level 18, but was highly unlikely to jump to Level 21 – the bottom floor of the high-rise – due to the three storey external space between Level 18 and Level 21;
- Zone 3: Fire on the Level 18 lobby. On the basis of the fire load at this level it was demonstrated that the fire would not jump to Level 21 – the first office floor plate of the upper block;
- Zone 4: Fire in the high-rise block Level 21 to Level 30.

Further to this analysis, the building has three street boundaries and only one shared boundary. The megabracing elements are therefore protected against fire from adjacent buildings on the basis of the separation afforded by the streets, and to the south adequate separation due to the depth of the 8 Chifley core.

Due to the proximity of the bracing to the floor plate, assessment of limiting temperatures against associated member capacities of the steel brace sections indicated that their capacity was overcome relatively quickly for typical office fires. A series of global structural stability analyses were subsequently performed with braces removed to model the structural condition under each fire load case relating to the zonal fire assessment. As there is no specified lateral loading requirement under the Australian Standards for the fire load case, lateral loads based on two cases where assessed:

- i - An applied lateral load of 33% of ultimate wind load in accordance with BS5950-80, representing a wind load of approximately 6 month return period;

Figure 7. Pinned link node (Source: Brett Boardman)

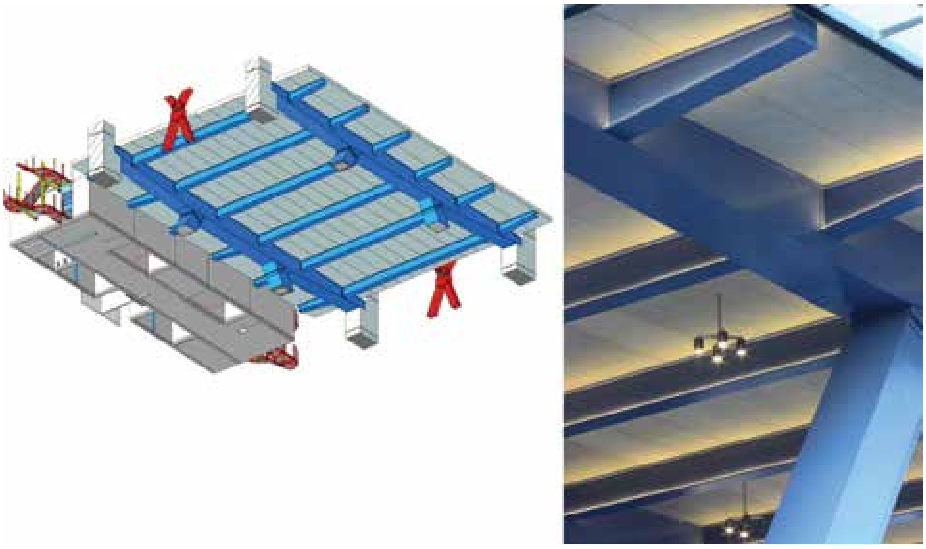

Figure 8. Reverse podium structural arrangement and completed structure (Source: Arup)

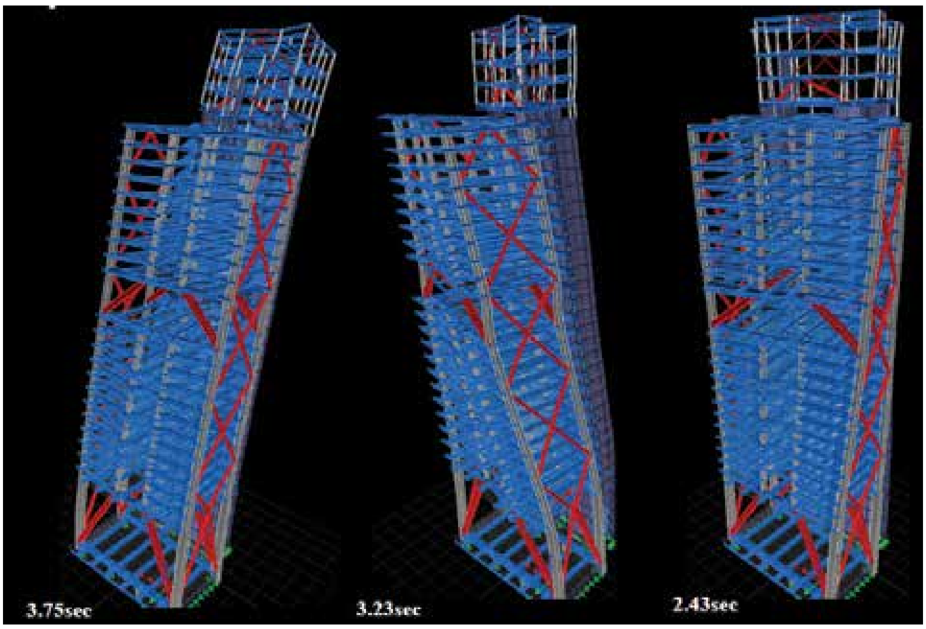

Figure 9. Fundamental dynamic mode shapes (Source: Arup)

Figure 10. Composite megaframe assembly (Source: Arup)

Figure 11. Exterior steel fire stairs (Source: Brett Boardman)

- ii - A notional load of 1% of gravity load applied laterally at each floor.

These analyses demonstrated that stability and strength were satisfied without major modifications to the detailing required under general serviceability and ultimate limit states (Figure 6).

The outcome was conventional fillet welded steel 4 plate semi-box beams conventionally painted, saving over 1300m^2 of conventional passive fire protection and cladding, or expensive external epoxy intumescent and additional steel tonnage to achieve adequate surface to mass ratio.

This steel allowed for rapid site installation – workshop to site to erection – complete - with no additional finishing trades, and provided the clear structural legibility the architects were striving for.

An articulated pinned-link node at the intersection of the X bracing geometry was developed to release elastic gravity loads, time-dependant gravity loads from shortening of the concrete megacolumns, and thermal loads, enabling the braces to be sized for wind and seismic shear only – a saving of approximately 40% in the brace tonnage and significantly reducing the end connection complexity at the interface with the concrete frame. This articulated element also assists in releasing extension of the braces in a fire, and became a defining feature of the building.

Megaframe

Occurring twice, these large expressed transfer structures create two generously proportioned column free voids; the lower public domain at plaza level and a private three storey high terrace at L18. The two prominent outdoor spaces are created by transferring the four internal columns in both the low and high rise blocks out to their respective external mega columns, delivering gravity loads to the exterior bracing frames, ensuring no tension is developed under lateral loading.

The transfer frames also provide a significant component of the lateral resisting system in the east-west direction, assisting to control the induced torsion from the southern core, demonstrated in the fundamental mode shapes (see Figure 9).

Materiality and constructability of the inclined and horizontal elements were fundamental to the construction programme and buildability, leading to the use of composite steel and reinforced in-situ filled sections for all primary elements. With the southern structural core constructed slightly ahead, the megaframe structures were designed and constructed as jump decks, allowing the insitu concrete post-tensioned floors above to be formed from this level with no back-propping below.

The steel shell for the inclined columns and primary beams acts compositely with the concrete at general serviceability and ultimate limit states. During a fire the steel shell is sacrificial. This resulted in a rapidly constructible series of hollow steel columns and beam shells which once attached to the free-standing megacolumns and bolted together on site were self-stable. The steel secondary beams were then installed, and precast planks craned in so that no temporary propping was required through the six and three storey voids at Level 6 and Level 18 respectively (see Figure 10). A self-compacting concrete fill to the columns was pumped from the bottom, whilst the primary beams were U-shaped sections and filled conventionally. An insitu reinforced diaphragm slab was then poured over the precast planks.

Honesty and legibility of the expressed structure dictated the jump deck floor system should mimic that of the typical concrete scheme, with steel composite secondary beams at 6m centres.

The fire engineering assessment undertaken at Zones 1 & 3 (see Figure 5) and detailed fire analysis using CFAST concluded that limiting temperatures of the secondary beams and their connections were not reached (less than 550°C, and the original specification for 660m^2 of exterior epoxy intumescent coating was saved. Therefore, a consistent paint finish could be achieved on all exterior steelwork – both concrete filled composite and plain steel elements.

Fire Stairs

A key component in realizing the building feasibility was the use of open exterior fire escape stairs, as open stairs were not counted as developable floor area under the City of Sydney planning regulations at the time – a concession that had not previously been realized in Sydney. As the site development controlling criteria was maximum area and not height, over 600m^2 was realized as additional lettable floor area on the basis of achieving an approved open stair solution – equivalent to over 50% of one of the nine full floors in the building. Over the 10 year development feasibility analysis of the project this is a substantial quantity of income in the mid to high 7 figure bracket.

Key project issues were identified that required resolution (see Figure 11 & 12):

- The stairs were carefully designed to minimize the impact of acrophobia and adverse weather impacts – particularly during fire drills. This assessment involved comparing the carefully designed balustrade to metric data for stature and eye height. A virtual model of the stairs was created to assist the team in understanding the degree of enclosure and the consequent sense of comfort and security occupants would feel while egressing;
- Following the above, a prototype geometric stair was constructed on the roof of the existing 26 storey building prior to demolition and tested by the client;
- The exterior stairways were protected by a Tyco sprinkler wall wetting system on the interior portion of the adjacent glass façade;
- The stair structure was steel, cantilevered from the concrete core in a veirendeel arrangement, separated from the interior space by 2hr fire rated walls to the core area, and separating distance and assessment of fire vs steel limiting temperature from the adjacent office space. The stair framing was originally proposed to be supported from the core on the three-storey village planning grid, however this was adjusted to a floor by floor supporting system, vertically continuous to maximize structural redundancy;
- The stairs re-enter the building core at Level 5 and egress along the street to the south boundary, necessitating entering a fire and smoke isolated corridor from outside at level 5.

Stair pressurization was alleviated with the use of external open stars, reducing energy demand and contributing to sustainable credentials of the building.

Villages

The structural geometry is driven by the multiple stacked villages - the stacked villages enabled 8 Chifley to market itself towards larger firms, by providing offices larger than those typically associated with a small site. Each village was created using three floors, one full floor and two "U" shaped floors, providing a central atrium and focal point for each. Not only do the village voids increase tenant interconnectivity, they dramatically increase the amount of daylight penetration into the floor plates, view of the façade from the deeper workstations, and consequently make the floor slabs more appealing and lettable.

Large internal floor connections of four stories, significantly greater than the prescriptively allowed two storeys under the Building Code of Australia were achieved using a

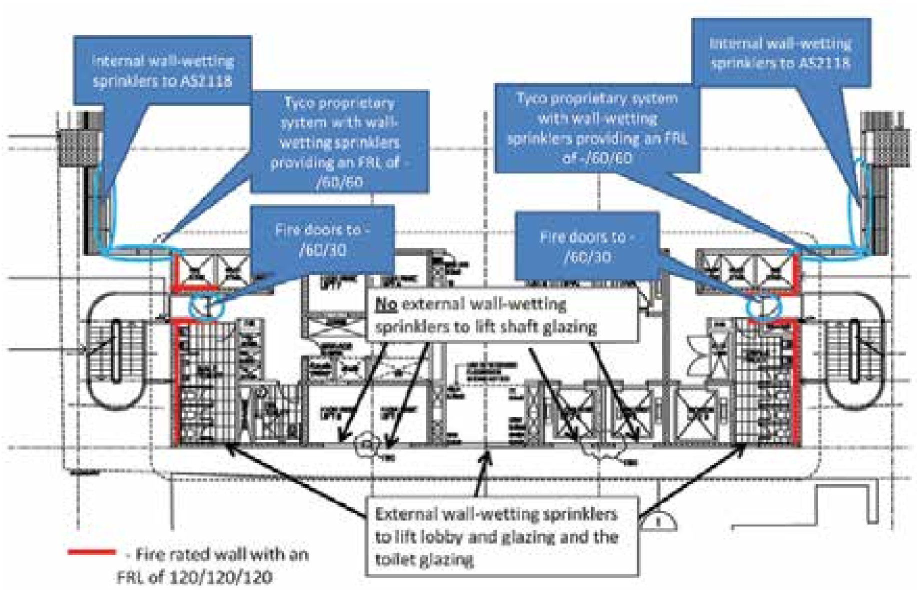

Figure 12. Fire separation to external stairs (Arup)

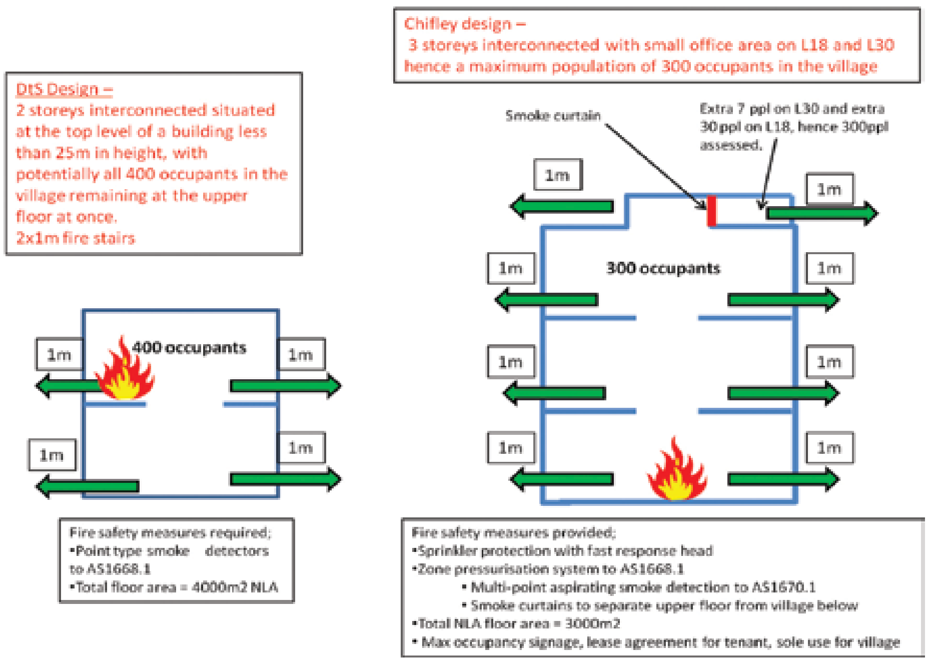

Figure 13. Comparison of 8 Chifley to DtS interconnection criteria (Source: Arup)

fire engineered risk based approach and egress assessments. These were focused directly upon risk to occupants in terms of egress and smoke exposure. The assessment quantitatively compared the number of occupants possibly affected in a Deemed-to-Satisfy (DtS) scenario compared to the proposed design. This demonstrated that with the number of occupants affected in the proposed solution, in consideration of available exit widths available, the risk of unsafe egress was no greater than in a DtS design. This enabled the safe interconnection of the floors (see Figure 13).

The fire engineering analysis allowed for costly and obtrusive mechanical exhaust systems to be omitted, with a degree of compartmentalisation provided by smoke curtains that deploy upon fire alarm. The evacuation analysis and robust smoke detection and alarm interfaces allow for quick and efficient occupant movement complimentary to the design intent

Conclusion

The engineering of 8 Chifley was integrated from the conceptual stage of the building and worked in collaboration with the architect and client. The client objectives, architectural brief, and the challenges of the site, necessitated an innovative set of solutions to deliver value and realize the feasibility of the project - not only in excellence of individual discipline responses, but more importantly through the holistic integration of these services to produce a world class building that exceeded client expectations.

The conceptual design and realization of the exterior open fire stairs contributed heavily to the commercial feasibility of the project. During the design development and detailed design of the building, performance based structural and fire engineering saved over 2,000m^2 of conventional passive fire protection and cladding or exterior epoxy based intumescent to expressed steel elements, that were instead finished with a traditional painted corrosion protection systems and successfully detailed as an inherent and legible component of the architecture. In turn, construction of the building was realized ahead of programme and has been the recipient of a series of awards in Australia for commercial development, architecture, engineering, and sustainability.

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