Brookfield Place, Perth

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ABSTRACT: *Brookfield Place is a bold new commercial development in the heart of Perth's CBD. The centerpiece of the development is a 250m tower overlooking the Swan River. The unique tower design with its slender, offset, open-faced RC core and external concentrically braced frames allows the architectural vision and function to be fully recognized. The tower is surrounded by a multilevel RC basement and podium structure providing public space, retail areas and parking. The project started construction in 2008 and was completed in early 2012. This paper outlines the design of the building from concept to construction and the challenges faced to deliver this iconic Perth structure.*

KEYWORDS: Brookfield Place, Tall Tower, Offset Core, Composite Structures, Steel, Reinforced Concrete

1 INTRODUCTION

The Brookfield Place development in Perth's CBD provides the newest addition to the river front skyline. The 250m tall, 5-star green star, tower is the tallest commercial building in Australia. The unique tower design has a height to base ratio of 9:1, is one of only a handful of towers in Australia with an offset concrete core and one of a few to have open-faced lift cores which expose peoples movements up and down glass elevators within the building.

The design is made possible due to a reinforced concrete core which is offset to the North, load sharing with two external concentrically braced frames (CBF) consisting of composite tube mega columns. The offset core maximizes panoramic river views from the whole floor plates to south and the open faced core connect the building with the city to the north. The large rectangular floor plates allow ultimate flexibility for tenant fitout designs and by positioning the core to the Northern façade reduces cooling requirements for the building by absorbing solar load.

The Brookfield Place development delivers an iconic Perth structure which offers a premium, practical and sustainable space to serve as the new home of BHP Billiton in Western Australia to use now and into the future.

Architectural Perspective During Construction

***Figure 1** Brookfield Place Tower*

2 DESIGN FEATURES

2.1 FOUNDATIONS

In the early 90's another development was planned for the site but never completed. However the construction of diaphragm walls, basement excavation and pile installation beneath the previous tower design were complete and this was the starting site conditions for the Brookfield Place development. This existing structure was reused in the new design to save on time, cost and material to help reduce the environmental impact where possible.

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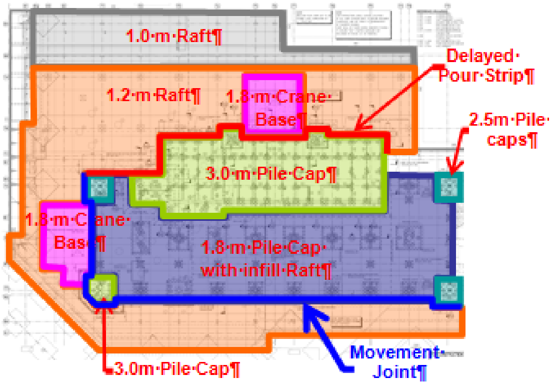
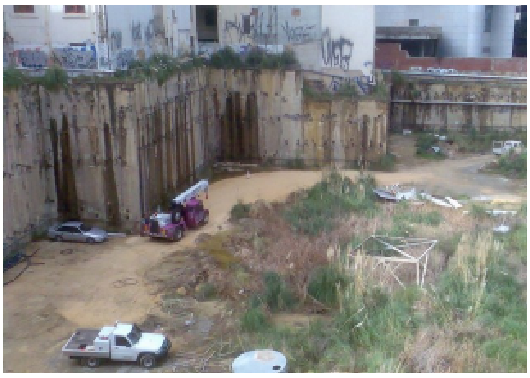
The foundation system for main tower consists of a 1.8m-3m deep pile cap supported on a system of 450 750mm diameter piles. These extended down approximately 20m in length to the solid Kings Park Siltstone. The majority of the surrounding basement structure is supported upon a 1.2m raft



a) Foundation Plan



b) Site Prior to the Start of Construction

Figure 2 *Brookfield Place Tower*

The existing diaphragm wall structure surrounds three sides of the site. It is a 500-800mm thick wall restrained by rock anchors anchored into the surrounding sites as can be seen in figure 2(b). The existing walls were incorporated into the design with the basement floors being supported off the walls and connected with temporary expansion joints.

Once the tower was complete these joints were grouted along with the delayed pour strip to the tower, shown in figure 2(a), at which time the wall anchors were be de-stressed allowing the wall to then be restrained by the tower structure. The wall required a condition survey at the early stages of the project to assess the condition of the existing anchors and wall. A number of new anchors where required and the wall movement was continually monitored throughout construction to ensure construction activities did not move the wall outside acceptable limits.

2.2 BASEMENTS AND PODIUM

There are four basement levels beneath a podium structure which surrounds the tower footprint, as shown in figure 3. The floor to floor heights for the basements were governed by the neighbouring building, of which it shares the basement access, and was set at 3m. Construction access was critical to the site as it is 'landlocked' by the neighbouring properties. This affected the sequencing of the basement construction which was accounted for in the design.

The basement structure and podium were designed as a combination of conventional reinforced concrete and post-tensioned concrete. The 2-way floors varied in thickness form 280mm to 360mm with band beams where required. This allowed for flexibility in layout, floor spans and loadings which vary from 2.5kPa to 20kPa.

During construction the design of some areas of the podium underwent a value engineering exercise so the design could be modified to allow podium construction to be accelerated to suit sequencing. These areas were modified to a top down type construction where podium could be built prior to the basement levels below which allowed basement construction to continue unaffected by constructi9on occurring overhead. After reviewing a number of schemes, steel frames with composite floors and columns were used in a design similar to the tower. This allowed podium to be built at 12m above the raft, before the basement levels, which would have been difficult to carry out with conventional formwork.

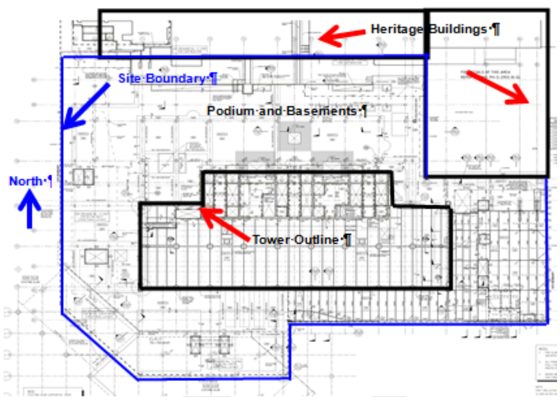


Figure 3 *Site Plan*

2.3 GRAVITY SYSTEMS

The tower structure has its offset core to the North with the floor structure to the south. The floor structure consists of steel frames with composite slabs, beams and columns. The main drivers for the decision to use composite systems were cost, programme and speed. The 120mm composite floor spans 2.8m between secondary beams which form the 8.4m primary grid and 11.4m secondary grid as shown in figure 4 below.

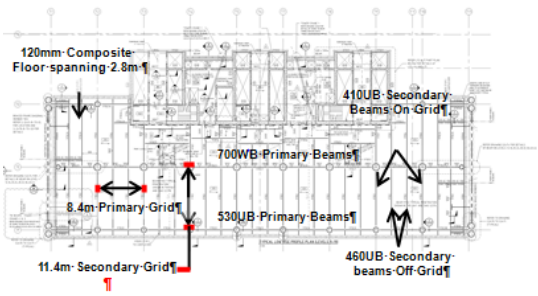


Figure 4 *Tower Floor Plan*

The design of the floor structure was analysed to optimize the design while meeting the design criteria and drivers. This included assessing floor vibration so movement was not perceived by the end user as well as allowing for penetrations to coordinate services through the steel support beams. The final design allowed for simple construction sequencing and coordination the services that followed. The supporting composite columns consist of spirally wound steel tubes filled with reinforced concrete. Columns range from 1350mm to 500mm with a tube thickness of 12-16mm. The use of composite columns allows a simple and fast onsite construction process with the tubes acting as permanent formwork. Also the composite columns utilize fire performance and compressive resistance to minimise the column dimensions to provide an economic solution and increase the functionality of the floor space.

2.4 LATERAL SYSTEMS

The lateral resistance scheme consists of the offset concrete core working with the two external CBF frames. The two frames load share with the core to resist loads in the North-South direction and then resist torsion cause by the offset core for loads in the East-West. Approximately 70% of the load is resisted by the core and 30% by the two CBF frames. An extensive parametric study was undertaken where many schemes, varying lateral resisting systems and configurations, were considered and taken to a high level of design before this final scheme was adopted. The final scheme included significant collaboration with the client, architect and the builder to achieve the best overall outcome.

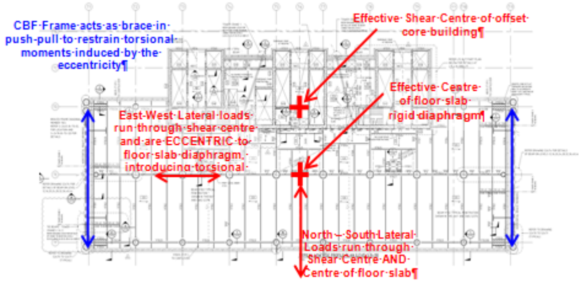


Figure 5 *Tower Plan – Lateral Resistance*

The offset, open-faced core is the key feature of the towers design which allows the architectural vision to be recognised as well as provide the required building performance. The core is approximately 47m by 13m giving it a height to base ratio of 16:1. The core arrangement tapers with the low rise portion finishing at level 23 and midrise at level 33. The core walls range between 200mm-600mm with concrete strengths of 70MPa at the base of the building and reducing to 50MPa at the top. The open face to the North of the core, shown in figure 6 below, reduced the stiffness of the core and meant the extreme ends of the walls were unrestrained and it is in these zones where wall ligs were included so these portions of walls were confined. The offset core not only provides an open, functional space with expansive views to the South but also reduce the cooling requirements of the building by absorbing the majority of the solar load from the North.

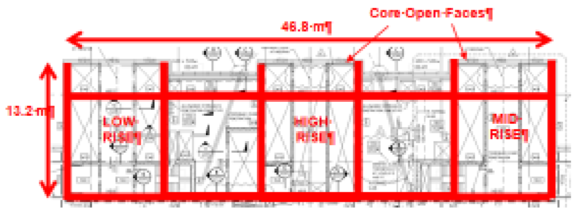
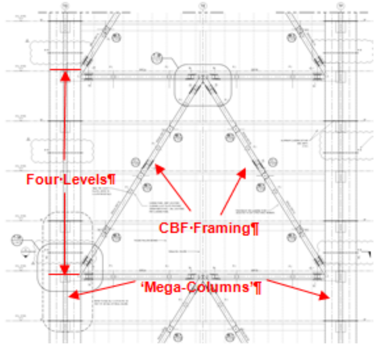
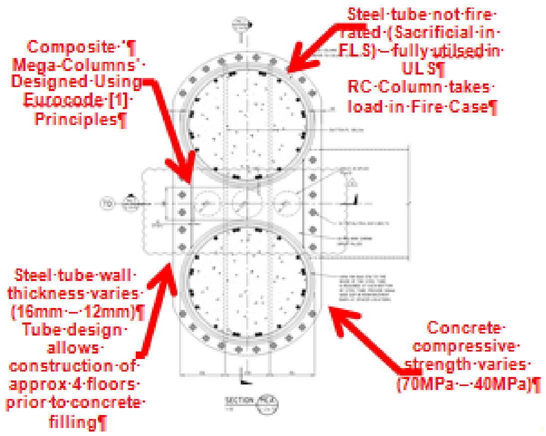


Figure 6 *Core Plan*

The two external CBF frames combine an architectural feature with a primary stability element for the tower. As shown in figure 7, the frames consist of two composite 'Mega-columns' with 400-500WC bracing between. These frames reoccur every four levels with the braces having a restraint at each floor. Each mega-column is fabricated from two 1150mm, Grade 350, spirally wound tubes then poured on site with 40MPa - 70Mpa reinforced concrete as shown in figure 7.



a) External CBF Frame Elevation



b) Mega-Column Section

Figure 7 *External CBF Frame – East and West Ends*

2.5 BUILDING 'TIARA'

Completing the architectural vision of the building is the 40m tall 'Tiara' which sits atop of the tower. The Tiara is a braced portal frame structure with structural steel box truss horizontal and raking members, composite tube columns and high strength tension bracing. The 1.2m square box truss horizontal members are fabricated from SHS sections with a warren truss arrangement front and back and vierendeel truss arrangement top and bottom. The portal knee connections required detailed finite element analysis due to the susceptibility of thin wall tubes to local buckling. The final solution used batten plates through the columns, local stiffeners below and above the joints and concrete filled tubes.

Some of the unique challenges which needed to be overcome in the design included aeroelastic instabilities, access, construction methodology and tolerances all while maintaining the architectural intent. The tiara is a wind sensitive structure due to the member shapes and to avoid aeroelastic instabilities the final solution, developed after detailed wind tunnel testing, included mass chain dampeners and modification of the cladding of square members to open it up critical regions. Permanent access was utilised during construction and a combination of welded and bolted connections were used to simplify construction methodology and increase construction tolerances.

3 Design Philosophy

3.1 Tower Design

An extensive parametric study of the tower structure was undertaken to balance cost-efficiency with constructability, usability and architectural requirements. The tower was modelled with design loads in accordance with AS/NZS1170[2], material properties in accordance with AS3600[3] and AS4100[4] with a dynamic response in accordance with ISO 6897[5].

The final modelling of the tower was primarily carried out using ETabs using three separate models. One model for vertical loads, one model for ULS lateral loads to determine design loads of structural elements and one for SLS lateral loads to compute building modal shapes, frequencies, displacements and drifts. Within these models the structural elements were modelled with core walls modelled as 'Shell' elements, Mega-Frame and header beams by 'Frame' elements, floor slabs with rigid diaphragm constraints and piles by 'Springs'.

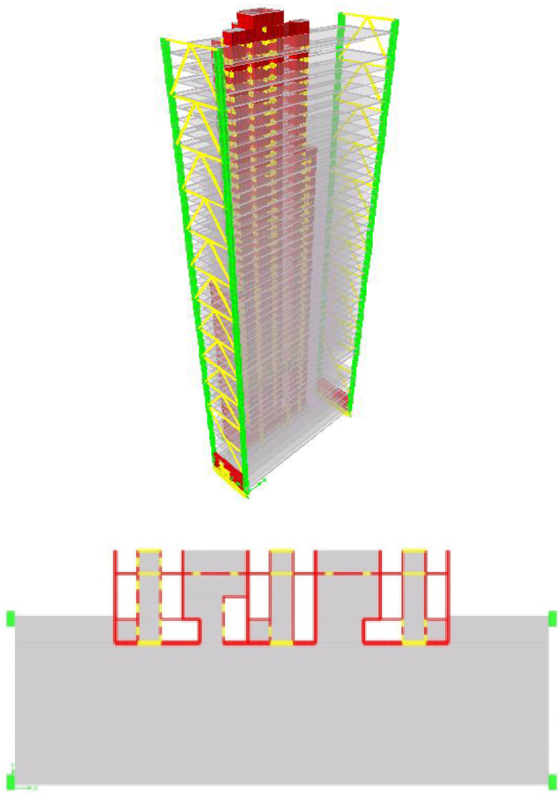


Figure 8 *Tower ETabs Model – Perspective and Plan*

Modelling secondary effects on the structure was also critical to the successful design of the tower. This included calculating the expected vertical movement of elements from axial shortening and settlement. Using load rundowns along with the construction programme the vertical movement from creep and shrinkage can be calculated over the buildings lifespan. It was found that the core

shortening was governed by shrinkage due to its relatively low axial load compared to a high level of exposed concrete where as the column movement was governed by creep due to their high axial loads with no exposed concrete due to the steel tubes.

The main concern with axial shortening and settlement is differential movement between adjacent columns or between columns and the core. If this differential movement is high it increases loads in the structure as well as effecting secondary elements such as facades, which could be at their expected level. This was evident at mega-column locations, shown below in figure 9, as these have a relatively low axial load due to each having two columns and being sized for their bracing function. Movements from differential axial shortening and differential settlement were modelled using Spacgass to assess their effects and levels where monitored on site to confirm they matched expected levels.

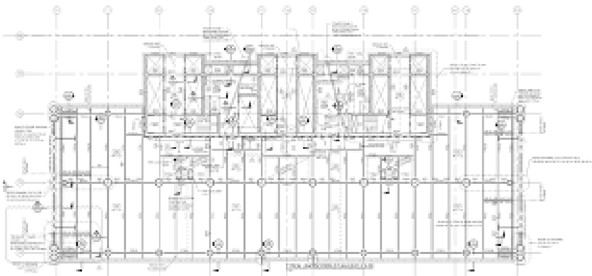


Figure 9 *Tower Plan – Column Layout*

3.2 Design Loads

The design loads for the project have been derived from AS/NZS 1170. The building is an importance level 3 structure with a design working life of 50 years. The primary use of the tower is office space which has a live load allowance of 3kPa with compactus zones on each level with an allowance of 7.5kPa.

The lateral loads on the tower include wind and seismic loadings, as well as eccentric gravity loads causing sway. The initial wind loads were calculated using AS/NZS 1170.2 using an ultimate limit state 1000 year return period. MEL Consultants then carried out wind tunnel testing on a 1/400 scale model of the tower, shown in figure 10. This enables the AS/NZS1170.2 principles to be refined to obtain more efficient design by taking into consideration the surrounding structures, wind direction, dampening and tower frequencies. There are a number of outputs which were obtained from the wind tunnel testing including general façade design pressures used in local design and then ULS design moments and shears are presented for global design as well as serviceability accelerations.

Seismic loads on the tower were calculated using AS/NZS1170.4. A return period of 1000 years was used with a ductility of 2 and the buildings natural frequencies. The seismic loads where applied to the building using equivalent static analysis. The critical load case was found to be that from wind in the North-South against the towers board faces.

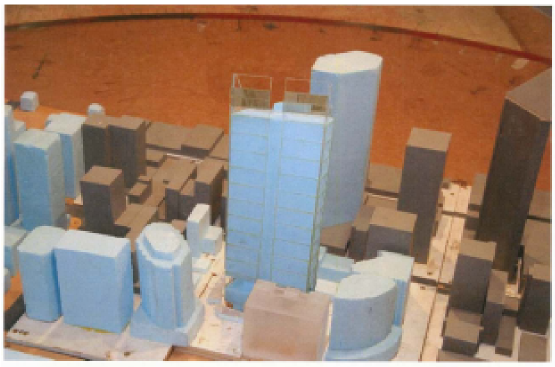


Figure 10. *Wind Tunnel Testing Model*

3.3 Fire Design

The design of the reinforced concrete elements in the building was able to be utilized to resist the design fire loads. In general there was a 2 hour fire rating requirement in the building. The fire resistance can be provided by the addition of a fire resistant material, suppression system or the structure can itself be designed to resist the high temperatures. The final design combined the above systems but the design of the floor, columns and CBF frame included inherent fire resistance.

The floor slab design included additional reinforcement to allow floor catenary action during a fire. As the temperature rises the beam strengths decrease and the beams begin to sag. This causes the floor to pull in on the ends of the beams and the reactions need to be resisted or the floor could collapse. The floor design resisted this reaction via the connection to the core at one end and then by an in plan truss spanning between columns at the other formed by additional reinforcement. This is illustrated by figure 11. The composite columns supporting the floors were designed for the tubes to be sacrificial during a fire with the internal concrete column able to resisting fire load combinations. In including these elements in design it avoided the requirement to protect all beams and columns with fire spray which was only needed at connections and beams in specific areas.

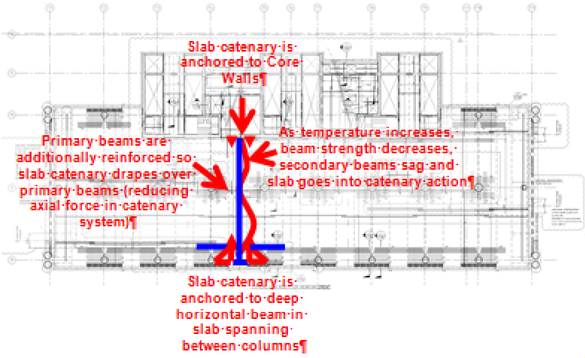
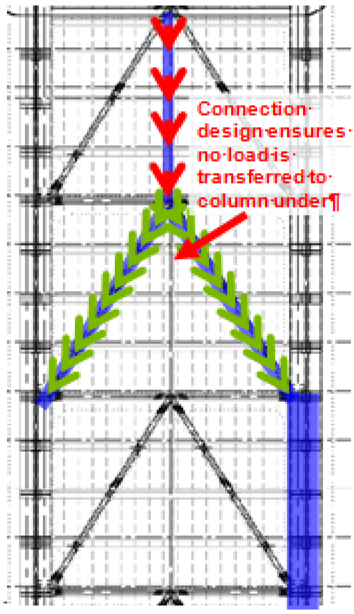
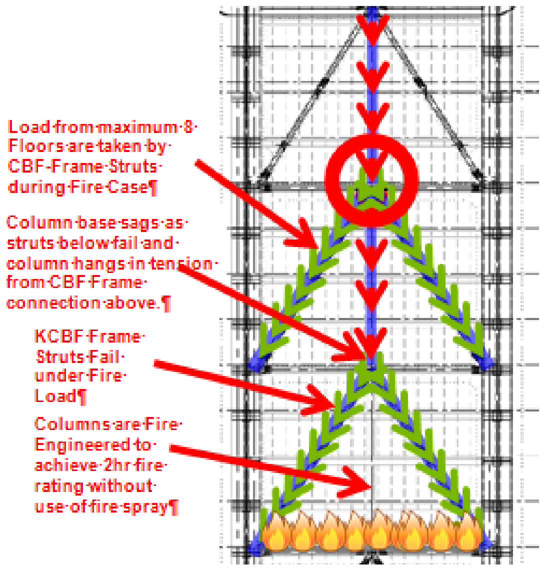


Figure 11 *Tower Floor Fire Design*

Another location where it was preferred to incorporate the fire resistance into the structural design is at the external CBF locations. The primary load path at the external frame is that the floors loads the central composite column and every four levels the load is transferred into the diagonal brace members and then out to the mega-columns as shown in figure 12. However during a fire a brace member could be compromised and this load path taken away. To design for this there is a connection at the top of the central column which does not allow it to be loaded from above due to a design gap. However the connection does allow the column to hang from above if required. In an event of a fire where a brace is removed the load path changes and the column then hangs from the transfer beam above and loads the external braces above with eight levels of floor as illustrated in figure 12.



a) General Load Path



b) Load Path During a Fire

Figure 12 *External CBF Load Path*

4 Construction

Construction for the project commenced in 2008 with the piling and enabling works. With a project of this size the construction sequencing was critical to the overall success of the project. The design stage is the best time to review the overall construction methods and sequencing to assess how efficiencies can be made and included in design to help the construction process.

The tower core was constructed using a self-climbing jump form which was split into two halves, an East and West side as shown in figure 13 below. This was required to be reconfigured twice during its ascent up the tower at locations where the lowrise and midrise cores dropped off. The jump form allows the reinforcement to be placed simply of the jump form working platforms, the formwork to be quickly put in place, concrete poured with the formwork then repositioning itself up ready for the next level. This meant a very efficient core construction with each 4.1m high level being complete in a 5 day cycle.

Figure 12 *Tower During Construction*

The core construction process combined well with the floor construction which followed closely behind. The use of composite structural systems meant an efficient, cost effective construction sequence was achieved. The composite tower floors are constructed after the core with steel columns and beams being installed followed by the composite decking, reinforcement and concrete. The beam cleats to the core walls have plates already cast into the walls with the cleats site welded to these. The double height tube columns come to site with the reinforcing cage already installed. These are spliced to the columns below with a bolted flange connection with the splice levels staggered between floors for adjacent columns. Each tower floor was split into three zones which allowed all trades to be working on a separate zone at the same time, steel installation, decking/reinforcement and concreting as highlighted in figure 13.

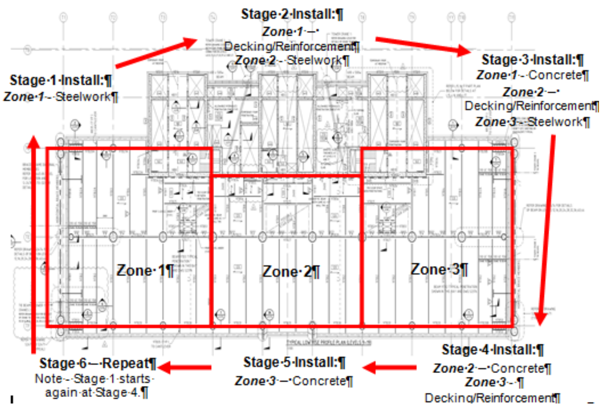


Figure 13 *Tower Floor Construction Sequence*

The Tiara construction was challenging due to it being located at 215m with heavy lifts required and limited access. The construction methodology was simplified by limiting the number of lifts, precladding where possible, simplifying connections, maximising tolerances, using permanent access systems, eliminating any need for scaffolding and minimising props. The columns were lifted directly into place, maximum lift 36T, with temporary bolted connections and FSBW permanent connections. The horizontals were lifted onto L47 in two halves, welded together and lifted into position in one piece with TF bolted splices each end. The raking members were split in two with FSBW connections top and bottom and a large bolted splice at midheight. Temporary working platforms were used and supported off the structure and a 'Mega-prop' was required on each raking member.

5 Conclusion

The vision for the Brookfield Place development, which posed challenges to achieve both the aesthetic and function for the tower, allowed for an innovative structural design to emerge. With the use of the slender, offset, open-faced core combined with external CBF frames the structure allows both of these to be achieved and the vision to be realised. Concrete played a major role in the projects success and the unique design of this iconic structure will dominate Perth's skyline for years to come.

6 Acknowledgements

The author would like to acknowledge Brookfield Multiplex for their assistance with the background information for this paper.

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[3] Standards Australia AS3600-2009: Concrete Structures.

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[5] International Organisation for Standardization, "ISO 6897-Guidelines for the evaluation of the response of occupants of fixed structures, especially buildings and off-shore structures to low-frequency horizontal motion (0.063 – 1 Hz)".