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Structural Design of the 84 Storey World Tower in Sydney

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Abstract

Completed in June 2003, Sydney's 84 storey 260m high World Tower is the tallest building in Australia. The structural engineering design represents an example of innovative structural engineering and technically advanced use of reinforced concrete. Its most striking feature and greatest challenge is its slender design, with a width of only 28 metres in the east-west direction and a slenderness ratio of 9:1. Due to the slenderness and the height of the building, the design team faced significant technical challenges to resist lateral loads and to keep building movements within acceptable levels under wind loads. This paper describes the solutions to the technical challenges.

Keywords: high-rise, slenderness, shortening, dynamic response, wind-engineering

1. Introduction

The 260m high World Tower building has eight below-ground parking levels, nine commercial/retail podium levels with 80 commercial suites, and 665 apartments on 67 levels, with associated plant rooms and recreational facilities, and a construction cost of AUD$350 million. Construction of the structure was completed in June 2003, and the apartments are expected to be completely fitted out by mid-2005.

The plan dimensions of the typical floors in the tower are 55m x 28m, with large cantilevered balconies. The typical residential level floors are post-tensioned slabs of flat plate construction spanning nine metres between the core and perimeter columns. The vertical structure of the tower comprises a central core of reinforced concrete, shear wall elements and 20 architecturally expressed columns.

2. Lateral Load Resisting System

Engaging the building structure on the perimeter was identified as the key issue, and extensive parametric studies were carried out to identify both the method of engaging the perimeter structure, which elements to engage, as well as the configuration of structural elements utilised to do this. The chosen solution includes:

- Central reinforced concrete core boxes coupled by header beams.

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Fig. 1. World Tower South Elevation

- 2 pairs of 8 storey high diamond shaped post-tensioned outrigger walls at mid-height and three-quarter height of the building, which link the lift core to the perimeter columns.
- 2 storey high belt walls located at the tips of each outrigger at the level 37 and 60 plantroom levels, which engage the entire east and west facades to the outrigger walls. Connection of the belt walls to the outrigger walls occurs by means of intricately reinforced wish-bone corbel members.
- Outrigger trusses consisting of wind columns and inclined tower columns between levels 14 and 9.
- Perimeter belt beams at the level 37 and 60 plant levels.

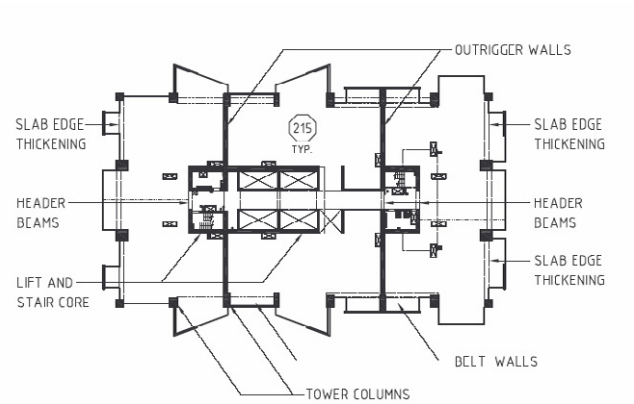

Fig. 2. Floor plan showing elements of lateral load resisting system

With the lateral load resisting system adopted, approximately 70% of the total overturning moment acting on the building under lateral loads in the critical direction is resisted by a push-pull couple generated by compression and decompression forces in the perimeter tower columns.

Of the 70% of total overturning moment resisted by the perimeter columns, 8% is due to frame action generated by the edge beams, 13% is due to the outrigger truss located at 1/4 height of the building, 30% (the most significant contribution) is due to the lower outrigger walls, and 19% is due to the upper outrigger walls.

Advantages of the adopted lateral load-resisting system are as follows:

- It provides a much stiffer structure minimising building sway and reducing accelerations under wind loads to within acceptable levels for occupant comfort without the need for supplementary damping.
- Avoids rock anchors under the central core for overall stability, which would otherwise have been required.
- Saves substantial amounts of reinforcement and concrete in the core walls due

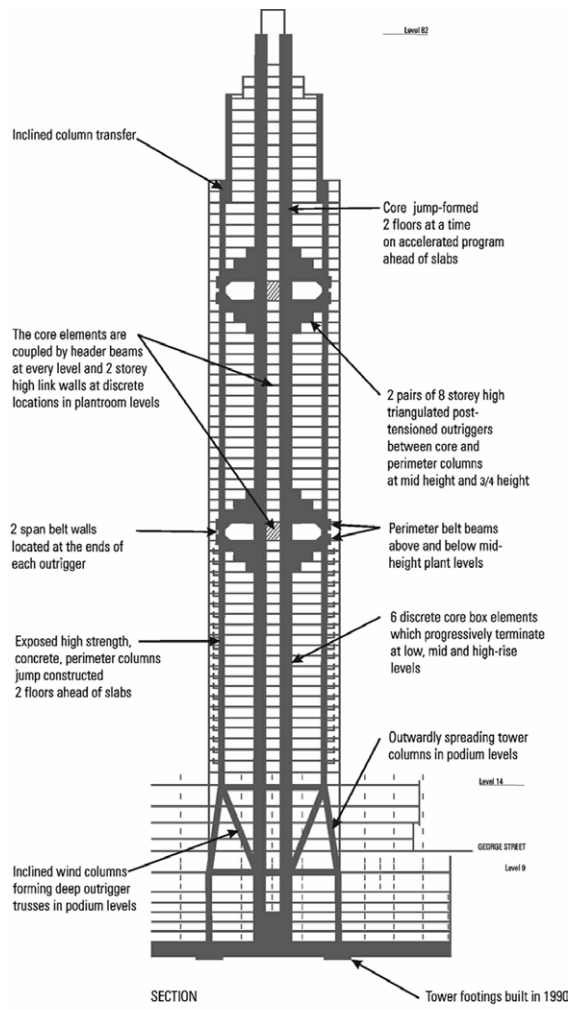

Fig. 3. Section showing elements of lateral load resisting system

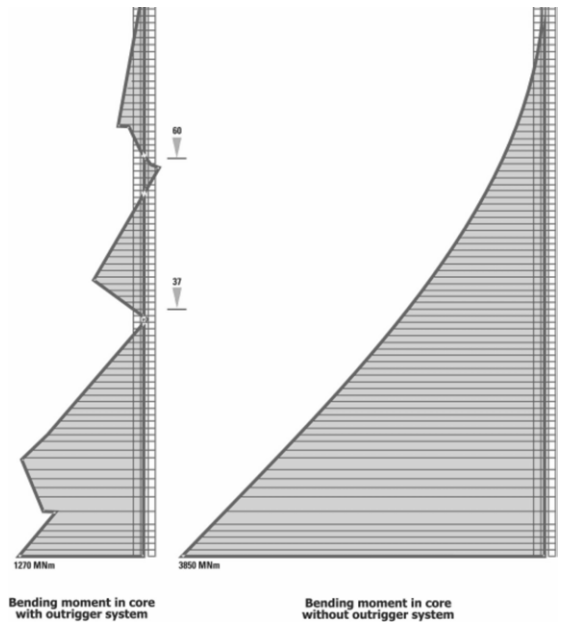

Fig. 4. Bending moments without outrigger system, and with outrigger system in world tower core

to significantly reduced design actions.

Figure 4 shows bending moments on the World Tower core for two scenarios, one with the outrigger system and one without. This provides an appreciation of the contribution made by the outrigger system in reducing the bending moments, and therefore reducing wall thickness and reinforcement in the core.

The outrigger system, though highly effective, poses some major design challenges:

- Very large loads have to be transferred across the building between the central core and the perimeter columns whilst minimising disruption to lettable floor space.
- Provision needed to be made for up to 50mm relative shortening between the core and the columns. This shortening was an ongoing effect requiring continual monitoring and management in accordance with the construction program, concrete creep and shrinkage, and building settlement.
- A stiff connection between the core and the columns is essential, with provision made to allow for the relative shortening between the columns.

3. Outrigger Walls

The 400mm thick post-tensioned concrete outrigger walls connect the central lift core to the building's perimeter columns, engaging them as part of the stability system.

To satisfy strength and serviceability requirements, other similar tall buildings that have outrigger structures generally incorporate such structural elements into the plant levels and include outrigger walls that are only one or two storeys deep, and therefore relatively inefficient.

Early design development with the architect, Nation Fender Katsalidis allowed the structural engineers to devise an outrigger configuration that consisted of two eight-storey high diamond-shaped walls hidden within the inter-tenancy walls of the architectural layout. Diamond-shaped walls allowed access along the building at each floor, without compromising the performance of the outrigger walls.

The building stiffness was increased considerably by the development of the outrigger system, and enabled the design team to do away with reliance on supplementary damping systems, which have been necessary on buildings of similar slenderness.

Due to the large wind forces to be resisted, the primary tension reinforcement used in the outrigger walls consists of 50mm and 75mm diameter Macalloy bars. These were placed to match the orientation of tension forces within the walls and allows a direct load path for the tie forces. Placing tension reinforcement on the direct load path, as opposed to conventional orthogonal arrangement, maximised efficiency and provided increased stiffness to the outrigger walls.

Use of Macalloy bars allowed the bars to be post-tensioned, with the level of post-tensioning adopted in the design to avoid cracking of the concrete under serviceability wind loads.

At the connection of the outrigger walls to the belt walls, the outrigger had to be splayed out in a wishbone shape to bear on the belt wall on either side of the column in line with the outrigger. Finite element modelling of the complex geometry of the outrigger and the wishbone corbel connection was carried out using the program STRAND 7 to check strength, and assess the required prestress and reinforcement levels in the outrigger walls and the "wishbone" corbel connections.

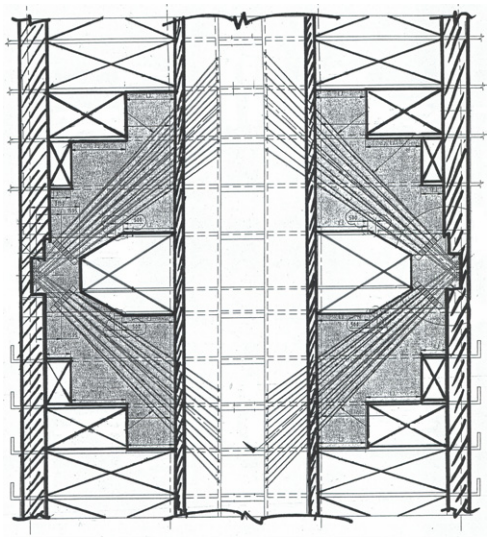

Fig. 5. Sketch elevation of outrigger wall

4. Belt Walls

Two-storey high 800mm thick reinforced concrete belt walls at the ends of each outrigger connect all the perimeter columns along the east and west facades of the building thus maximising the area of vertical perimeter structure mobilised to resist wind and earthquake forces.

5. Outrigger Trusses and Inclined Columns in Podium Levels

The outrigger trusses located at quarter height, in the podium levels, also have a significant effect on reducing building sway and accelerations. The major components of the outrigger trusses are the tower columns that incline outwards between levels 9 and 14, and inclined wind columns which complete the truss.

This unusual configuration of outwardly inclined columns adopted by the design team provides an increased spread of the base of the tower and this has a three-fold benefit. It reduces the slenderness of the

tower near the base, enables use of the original tower pad footings built some 10 years earlier for a previous scheme, and also utilises existing provisions for tower column penetrations in the as-built carpark slabs.

Increasing the overall base dimension of the tower from 29.2m to 36.5m by spreading the tower columns at the base increases the lever arm and reduces the tower slenderness from a height to base ratio of 8.9 to 7.1 at the lower levels. This has a significant effect on reducing building sways and accelerations due to increased stiffness for lateral load resistance.

The splaying of the perimeter columns had the following effect when compared to a design with the columns remaining vertical:

Fig. 6. 3-D model of superstructure

Table 1. Impact of Splayed Perimeter Columns at Base

Parameter	Impact
Tip deflection	reduced by 13%
Frequency	increased by 7%
Acceleration	reduced by 4%
Proportion of base moment resisted by the core	reduced by 35%

6. Analysis

Parametric studies of over 35 different scenarios were conducted to enable a design to be reached which resisted the loads economically, had minimum impact on the planning and avoided the use of supplementary damping. Some of these are shown below:

Table 2. World Tower Lateral System Parametric Studies

Model	Tip Deflection (mm)	Frequency (Hz)	Acceleration (milli-g)
Base case[1]	500	0.160	5.0
Addition of outrigger at L82	490	0.164	4.9
Removal of outrigger at L60	585	0.155	5.2
Addition of stiff perimeter frame	360	0.185	4.6
Double perimeter column stiffness	420	0.168	4.9
Reduce core wall thickness by 30%	515	0.158	5.1
No header beams in core	600	0.141	5.3
Reduce outrigger stiffness by half	600	0.150	5.2
Splay perimeter columns at base	440	0.170	4.85

[1]Base case is the building as designed with outriggers at levels 37 and 60 but without the splayed columns and wind columns at the base.

By appropriately understanding the impact of various parameters the final design was developed. Neither the core or columns were increased in size in an attempt to improve stiffness as the impact of this on acceleration is minimal. Outrigger stiffness was optimised at two levels and the columns splayed at the base to achieve the necessary performance.

7. Dynamic Response of Tower

Sway in the building's slender direction is caused by westerly wind which produces the highest wind speeds in the Sydney region, with a 3 second gust ultimate wind speed of 50 m/s.

Wind tunnel testing of the building has been carried out at design stage on an aeroelastic scale model at Monash University's 450kW boundary layer wind tunnel, to determine the dynamic response of

the tower to wind.

The predicted calculated dynamic behaviour of the tower is as follows:

Table 3. Predicted Dynamic Response Parameters

Parameter	Value
Natural frequency (east-west sway)	0.17 Hz
Natural frequency (north-south sway)	0.23 Hz
Damping levels for 5 year wind	1%
Acceleration under 5 year wind (RMS combined)	4.2 milli-g

On site dynamic response testing was carried out by Dr Graham Wood of the University of Sydney in January 2004 in order to confirm critical parameters of natural frequency and damping levels of the building.

The results of the testing is tabulated below. The predicted calculated natural frequencies that were calculated at design stage by modelling the structure using ETABS program are also tabulated below for comparison purposes.

Table 4. Dynamic Response Testing Results

Sway Direction	Calculated Natural Frequency (Hz)	Measured Natural Frequency (Hz) 1st Mode	Measured Frequency (Hz) 2nd Mode	Measured Damping (%) at 0.1 milli-g peak acceleration
East-West	0.17	0.22	0.76	0.77
North-South	0.23	0.32	1.15	0.70

As damping is dependent on building acceleration, extrapolation of the measured damping using the Davenport and Hill-Carroll graph, “Summary of Damping Relationships for Concrete and Steel Buildings of Tall and Intermediate Height”, correlates to a damping level of approximately 1% for 5milli-g acceleration, which is the calculated value of peak acceleration for a 5-year return period serviceability wind loads for World Tower. This corresponds with the predicted value of 1% damping adopted during the design phase.

For a damping level of 1% the 5 year acceleration prediction of 4.2milli-g is within the recommended criterion of 4.8milli-g for a period of 10 minutes in a return period of five years, proposed by ISO 6897 - 1984 " Guidelines for the evaluation of the response of occupants of fixed structures to low frequency motion".

The testing validates that the design parameters used in the design of the lateral load resisting system for World Tower are consistent with actual building performance. This provides further confidence that the actual building sway motions and accelerations under design wind conditions will be within acceptable criterion as calculated.

Fig. 7. World Tower – May 2004

8. Special Detailing to Accommodate Axial Shortening Effects

In tall reinforced concrete buildings, different rates of shrinkage and creep shortening of the columns and core walls needs to be considered in the design. Two

primary implications are the effect of this on the levels of the floor slabs, and unwanted load transfers between the core and columns.

The floors at the perimeter of World Tower were designed to be preset and constructed higher than the floor levels around the core walls, as is common on similar buildings, such that over time after the majority of the shortening has occurred, the floors end up being level.

Having a lateral load resisting system which links the lift core and the perimeter columns, while structurally efficient, has led to challenges associated with detailing to account for differential axial shortening between the core and the perimeter columns. The primary interface between the core and perimeter has been the outrigger and belt wall connections, and the outrigger trusses located at podium level. These are discussed below.

9. Outrigger and Belt Wall Connection

In World Tower, to avoid normal perimeter column loads being transferred to the core by the 8 storey deep outrigger walls as the columns shorten more than the core, specially fabricated temporarily adjustable oil-filled flat jacks have been used at the outrigger wall and belt wall interfaces. These jacks are able to transmit the full design forces imposed by the lateral load resisting system, but are adjustable.

Ongoing monitoring of the axial shortening has been carried out during construction and the jacks have been periodically adjusted. Once the building shortening due to creep and shrinkage has fully taken place, the flat jacks will be locked off by replacing the oil with epoxy, requiring no further maintenance.

10. Monitoring of Axial Shortening

By optimising the level of stress in the core and columns, and ensuring high modulus of elasticity using advanced concrete technology, the differential shortening between the core and columns was able to be limited to be less than 20mm.

The axial shortening of the tower floors at the core and perimeter is being monitored by detailed survey, and compared with theoretical estimates in order to identify when the flatjacks in the temporary connections between the various elements of the lateral load resisting system can be locked off by removing the oil in them and filling them with epoxy. After this, the connections will require no further maintenance.

Measurements of movement at the level 13 wind column flatjacks taken over a two year period from February 2002 to February 2004 showed maximum differential movements of less than 10mm between the core and the columns.

Measurements of movement near the top of the building at level 75 taken over a one year period from April 2004 when the slab was constructed, to February 2004 showed maximum differential movements of less than 10mm between the core and columns.

The survey monitoring also indicates that the relative movements between the core and perimeter columns appear to have stabilised one year after the end of construction.

11. High Strength Concrete

The advantages of using high strength concrete of up to 90 MPa in the tower columns combined with normal strength concrete (typically 40-60 MPa) for the core has had several benefits, including:

- The high strength concrete in the tower columns has a beneficial effect of contributing to the efficiency of the lateral system, both due to increased strength and stiffness resulting from a higher Young's modulus.
- Reducing the differential axial shortening between the core and tower columns.
- Reduction of member sizes, and therefore foundation loads. On World Tower, existing pad footings built some 10 years earlier for a 60 storey tower and foundations were justified to be structurally adequate for this 84 storey building.
- Increased net floor area due to reduced core and column size, as well as reduced obstruction of views which are a prime selling point for high rise towers.
- Reduced costs. Although the cost per cubic metre is greater than lower strength concrete, significantly less concrete and reinforcement was required.

12. Low Shrinkage, Low Creep Concrete

Use of low shrinkage, low creep concrete also contributed to reduced overall axial shortening of the vertical elements, as well as reduced differential axial shortening between the core and tower columns. This was critical for this 260m tall building to better manage the risk of the floors not ending up level.

The design was based on concrete with a basic drying shrinkage at 56 days as follows:

Table 5. Drying Shrinkage used for Design

Characteristic Concrete Strength f'_c (MPa)	Basic Drying Shrinkage (microstrain)
32 to 50	650
60 to 90	550

The above values were adjusted for ambient relative humidity of 62% estimated to prevail on site for the duration of the project (ie 12% more than for standard testing conditions) using Bureau of Meteorology data for Sydney.

Tests of concrete shrinkage for a drying period of 56 days for concrete used in the project are summarised below and indicates reasonable correlation between design and tested values.

Table 6. Drying Shrinkage Measured

Characteristic Concrete Strength f'_c (MPa)	Average Drying Shrinkage (microstrain)
40	630
60	520

Summary

This paper has described the solutions to the technical challenges faced in the design of the 84 storey 260m high World Tower in Sydney. With a slenderness ratio of 9:1, the major challenge has been the design of a suitable lateral load resisting system to satisfy strength criteria and to limit building motions to acceptable levels under wind loads.

The configuration of the lateral load resisting elements and benefits of the adopted system are discussed in regard to efficiencies gained, economic benefits in regard to strength and improved performance for serviceability in regard to the dynamic response of the building structure under wind loads.

The detailing challenges associated with differential axial shortening for this building in which the core and perimeter columns were integrally linked together as part of the lateral load resisting system, and the solutions adopted are discussed.

Monitoring carried out to date on the dynamic response of the tower structure under wind loads, and differential axial shortening of the core and columns indicates that the structure is performing as predicted at the design stage.

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Client & Project Developer: Meriton Apartments Pty Ltd
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