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SSDA Structural Report

TOGA Central

Issue: A – For Information

15 July 2022

Prepared For: TOGA

Project No.: 19231

Document No.: RBG-ST-REP-00000001[A]-SSDA Report

Report Amendment Register

Issue Ref	Amended Section(s)	Issue / Amendment Details	Author(s)	Reviewer	Date
P01	ALL	Draft for Review	Aubrey Bullen	Mitchell Starkey	15/07/22
A	ALL	Issued for Information	Aubrey Bullen	Mitchell Starkey	15/07/22

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Executive Summary

This Structural SSDA Report has been prepared by Robert Brid Group to accompany a detailed State significant development (SSD) development application (DA) for the mixed-use redevelopment proposal at TOGA Central, located at 2 & 8A Lee Street, Haymarket (the site). The site is legally described as Lot 30 in Deposited Plan 880518, Lot 13 in Deposited Plan 1062447 and part of Lot 14 in Deposited Plan 1062447. The site is also described as 'Site C' within the Western Gateway sub-precinct at the Central Precinct.

The purpose of this report is to provide an overview of the proposed structural design, discuss compliance with the Western Gateway Sub Precinct requirements and describe the structural design criteria and inputs used to develop the proposed structural design which has been incorporated into the architectural plans during development of the SSDA design submission.

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1 Introduction

This report has been prepared to accompany a SSD DA for the for the mixed-use redevelopment proposal at TOGA Central, located at 2 & 8A Lee Street, Haymarket.

The Minister for Planning, or their delegate, is the consent authority for the SSD DA and this application is lodged with the NSW Department of Planning and Environment (DPE) for assessment.

The purpose of the SSD DA is to complete the restoration of the heritage-listed building on the site, delivery of new commercial floorspace and public realm improvements that will contribute to the realisation of the Government's vision for an iconic technology precinct and transport gateway. The application seeks consent for the conservation, refurbishment and adaptive re-use of the Adina Hotel building (also referred to as the former Parcel Post building (fPPb)), construction of a 45-storey tower above and adjacent to the existing building and delivery of significant public domain improvements at street level, lower ground level and within Henry Deane Plaza. Specifically, the SSD DA seeks development consent for:

- Site establishment and removal of landscaping within Henry Deane Plaza.
- Demolition of contemporary additions to the fPPb and public domain elements within Henry Deane Plaza.
- Conservation work and alterations to the fPPb for retail premises, commercial premises, and hotel and motel accommodation. The adaptive reuse of the building will seek to accommodate:
  - Commercial lobby and hotel concierge facilities,
  - Retail tenancies including food and drink tenancies and convenience retail with back of house areas,
  - 4 levels of co-working space,
  - Function and conference area with access to level 7 outdoor rooftop space, and
  - Reinstatement of the original fPPb roof pitch form in a contemporary terracotta materiality.
- Provision of retail floor space including a supermarket tenancy, smaller retail tenancies, and back of house areas below Henry Deane Plaza (at basement level 1 (RL12.10) and lower ground (RL 16)).
- Construction of a 45-storey hotel and commercial office tower above and adjacent to the fPPb. The tower will have a maximum building height of RL 202.28m, and comprise:
  - 10 levels of hotel facilities between level 10 – level 19 of the tower including 204 hotel keys and 2 levels of amenities including a pool, gymnasium and day spa to operate ancillary to the hotel premises. A glazed atrium and hotel arrival is accommodated adjacent to the fPPb, accessible from Lee Street.
  - 22 levels of commercial office space between level 23 – level 44 of the tower accommodated within a connected floor plate with a consolidated side core.
  - Rooftop plant, lift overrun, servicing and BMU.
- Provision of vehicular access into the site via a shared basement, with connection points provided to both Block A (at RL 5) and Block B (at RL5.5) basements. Primary access will be accommodated from the adjacent Atlassian site at 8-10 Lee Street, Haymarket, into 4 basement levels in a split-level arrangement. The basement will accommodate:
  - Car parking for 106 vehicles, 4 car share spaces and 5 loading bays.
  - Hotel, commercial and retail and waste storage areas.
  - Plant, utilities and servicing.
- Provision of end of trip facilities and 165 employee bicycle spaces within the fPPb basement, and an additional 72 visitor bicycle spaces within the public realm.

- Delivery of a revitalised public realm across the site that is coordinated with adjacent development, including an improved public plaza linking Railway Square (Lee Street), and Block B (known as 'Central Place Sydney'). The proposal includes the delivery of a significant area of new publicly accessible open space at street level, lower ground level, and at Henry Deane Plaza, including the following proposed elements:
  - Provision of equitable access within Henry Deane Plaza including stairways and a publicly accessible lift.
  - Construction of raised planters and terraced seating within Henry Deane Plaza.
  - Landscaping works within Henry Deane Plaza.
- Utilities and service provision.
- Realignment of lot boundaries.

2 The Site

The site is located within the City of Sydney Local Government Area (LGA). The site is situated 1.5km south of the Sydney CBD and 6.9km north-east of the Sydney International Airport within the suburb of Haymarket.

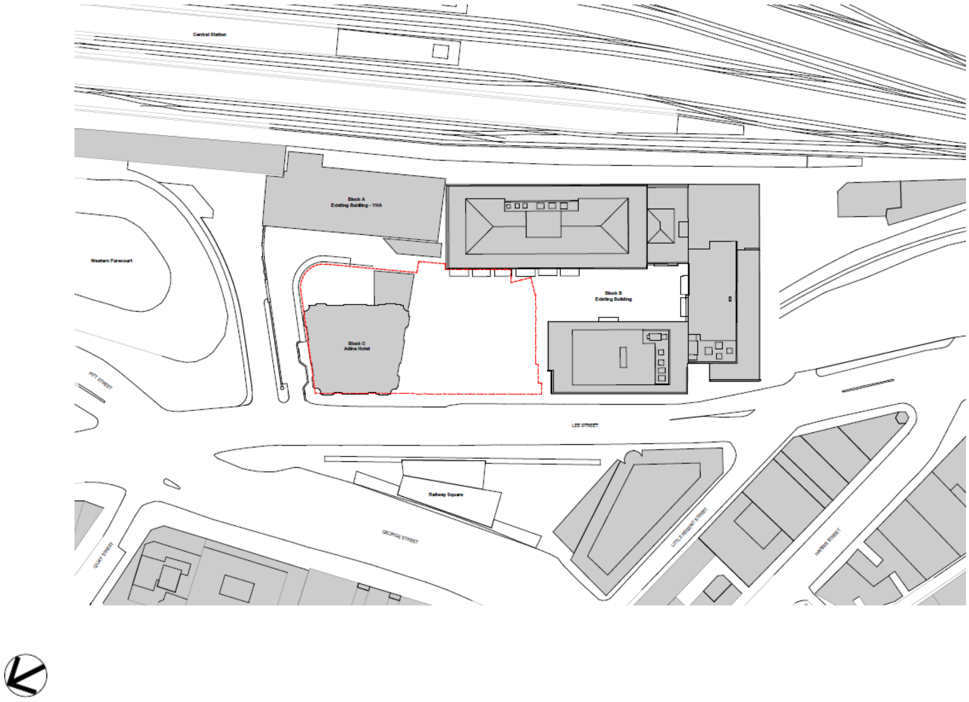
The site is located within the Western Gateway sub-precinct, an area of approximately 1.65ha that is located immediately west of Central Station within Haymarket on the southern fringe of the Sydney CBD. Immediately north of Central Station is Belmore Park, to the west is Haymarket (including the University of Technology, Sydney and Chinatown), to the south and east is rail lines and services and Prince Alfred Park and to the east is Elizabeth Street and Surry Hills.

Central Station is a public landmark, heritage building, and the largest transport interchange in NSW. With regional and suburban train services, connections to light rail, bus networks and to Sydney Airport, the area around Central Station is one of the most-connected destinations in Australia.

The site is located at 2 & 8A Lee Street, Haymarket and is legally described as Lot 30 in Deposited Plan 880518, Lot 13 in Deposited Plan 1062447 and part of Lot 14 in Deposited Plan 1062447.

The land that comprises the site under the Proponent's control (either wholly or limited in either height or depth) comprises a total area of approximately **4,159sqm**.

The location of the TOGA Central site is illustrated in **Figure 1**.



Source: Bates Smart

Figure 1 - Site Identification Plan

The site currently comprises the following existing development:

- Lot 30 in Deposited Plan 880518 (Adina Hotel building): the north-western lot within the Western Gateway sub-precinct accommodates a heritage-listed building which was originally developed as the Parcels Post Office building. The building has been adaptively re-used and is currently occupied by the Adina Hotel Sydney Central. The eight-storey building provides 98 short-stay visitor apartments and studio rooms with ancillary facilities including a swimming pool and outdoor seating at the rear of the site.
- Lot 13 in Deposited Plan 1062447 and part of Lot 14 in Deposited Plan 1062447 (Henry Deane Plaza): the central lot within the Western Gateway sub-precinct adjoins Lot 30 to the south. It accommodates 22 specialty food and beverage, convenience retail and commercial service tenancies. The lot also includes publicly accessible space which is used for pop-up events and a pedestrian thoroughfare from Central Station via the Devonshire Street Tunnel. At the entrance to Devonshire Street Tunnel is a large public sculpture and a glazed structure covers the walkway leading into Railway Square. This area forms part of the busy pedestrian connection from Central Station to Railway Square and on to George and Pitt Streets, and pedestrian subways.

The site is listed as an item of local significance under Schedule 5 of the *Sydney Local Environmental Plan 2012* 'Former Parcels Post Office including retaining wall, early lamp post and building interior', Item 855.

The site is also included within the Central Railway Station State heritage listing. This is listed on the State Heritage Register 'Sydney Terminal and Central Railway Station Group', Item SHR 01255, and in Schedule 5 of the *Sydney Local Environmental Plan 2012* 'Central Railway Station group including buildings, station yard, viaducts and building interiors' Item 824.

The site is not however listed independently on the State Heritage Register. There is an array of built forms that constitute Central Station, however the Main Terminal Building (particularly the western frontage) and associated clocktower constitute key components in the visual setting of the Parcel Post building.

3 Reference Documents

The following documents are to be read in conjunction with this report.

- Bates Smart Architectural SSDA Report and Drawings (Revision A) dated July 2022.
- RWDI Preliminary Wind Results Report (RWDI #190273 April 19,2022).
- Douglas Partners Geotechnical Investigation Report Rev 2 15th July 2022.

4 Geotechnical Conditions

RBG's understanding of the site-specific geotechnical conditions is based on the Douglas Partners Geotechnical Investigation Report Rev 2, Douglas Partners Pty Ltd (DP) prepared for Toga Pty Ltd on July 15th 2022.

The site and depth of the proposed basement means that the structure will typically be founded on good quality rock meaning that high level pad foundations are likely. Footings within the western portion of the basement (beneath Henry Deane Plaza) will be relatively simple to design and construct. Special consideration will be needed for the design and construction of footings founded within and directly adjacent the existing Parcels Post building that are near the existing footings.

Extended groundwater monitoring observation showed that the standing water level is in range between RL12.6-RL13.9. Ground water measurements within and adjacent to the site indicate the proposed design floor level of 'Basement 4' at a minimum elevation of RL1.0m will be below the permanent groundwater table. A drained basement with perimeter water-tight cut-off walls socketed in slightly fractured or unbroken sandstone has been adopted for the SSDA structural design following advice in the Douglas and Partners Geotechnical Investigation Report.

5 Proposed Structural Design

5.1 Substructure

The basement substructure consists of four basement levels on the southern side of the site and three basement levels on the Northern side of the site. The lowest basement levels are at RL1.0m and RL1.5m respectively. The lift pit for a portion of the commercial core lift is located at RL0.9m. The basement retention system consists of perimeter shoring walls socketed at least 2.0m into slightly fractured or unbroken sandstone to provide a watertight shoring wall system.

The site shares a boundary along the East with the Atlassian development and boundary along the South-East with the Dexus-Frasers Development. The assumed programming of the works would see both the Atlassian and Dexus-Frasers basements built ahead of the TOGA Central Basement. In this case a basement shoring wall along the Atlassian and Dexus-Frasers shared boundary would not be required. A concrete wall or columns will be built adjacent to the Atlassian/Dexus-Frasers shoring walls to provide support to the TOGA suspended basement slabs. If Toga proceeds with construction ahead of either adjoining owner, Toga will construct a shoring wall to the relevant sides of the basement as required. The basements will be linked at B03 to provide a connection for services, deliveries, and general movement on traffic in the temporary and permanent case.

The lowest basement level slabs will be reinforced concrete slabs cast on ground. The suspended basement slabs will be reinforced concrete slabs and band beams supported on reinforced concrete columns, reinforced concrete core and perimeter shoring walls.

Columns are supported on reinforced concrete pad footings founded on Class II Sandstone. The reinforced concrete core is supported on a pad footing with active ground anchors along the Eastern Edge to resist uplift under wind lateral loads.

5.2 Superstructure

On the upper commercial levels, the floor structure consists of a post-tensioned concrete slab and band beams supported on reinforced concrete columns and reinforced concrete core walls. Columns can be conventional reinforced concrete or concrete filled tube (CFT) could be adopted to reduce the size and increase serviceability performance. Soft spots have been allowed for within the structural design to provide vertical circulation between commercial levels.

The typical hotel floor plates consist of a post tensioned flat slab supported on reinforced concrete blade walls and reinforced concrete core walls. There is a large void present in the central area of the hotel floor plate.

On the North/West Side at Level 10 on the lowest hotel floor, the concrete blade walls transfer onto two Y columns which run through the existing fPPb and are supported on pad footings bearing on Class II sandstone. Horizontal in-plane forces from the Y columns are restrained by a combination of the belt truss located at level 19 and framing action via. the slabs at each tower level. RBG and the project team explored various options for supporting the portion of the tower which sits above the fPPb which are discussed further in Section 9.

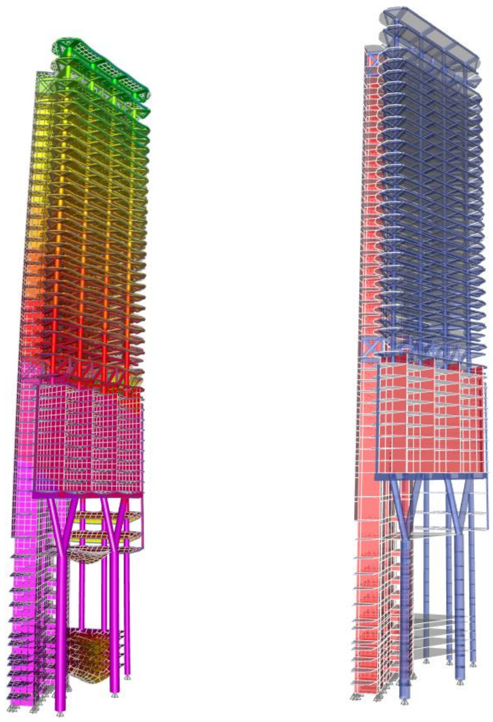


Figure 2 - ETABS Finite Element Anlaysis Model of TOGA Central

On the South Side at the Level 10 hotel floor, the four RC blade wall transfer onto central CFT columns. The blade walls and concrete slabs from L10-19 are used to stabilise the out of balance forces from transfer at Level 10. The four CFT columns are supported on pad footings at B4 bearing on Class II sandstone.

The reinforced concrete core is positioned on the Eastern edge offset from the centroid of the floor plate. The positioning of the core results in the core walls supporting minimal gravity loads due to the floor structure when compared to the columns. The area of concrete in the core and hence stiffness is much greater than the area of concrete in the columns The size of the columns and hence stiffness is much less than the core. This imbalance of vertical load to structural stiffness results in more vertical settlement on the western side of the tower and a tendency for the tower to lean towards the west. Building services, lifts, façade, finishes and other internal fit out items are to be designed and detailed to accommodate these movements. A detailed building movement report will be developed at later design stages to inform all disciplines of the predicted building movements for incorporating within their design.

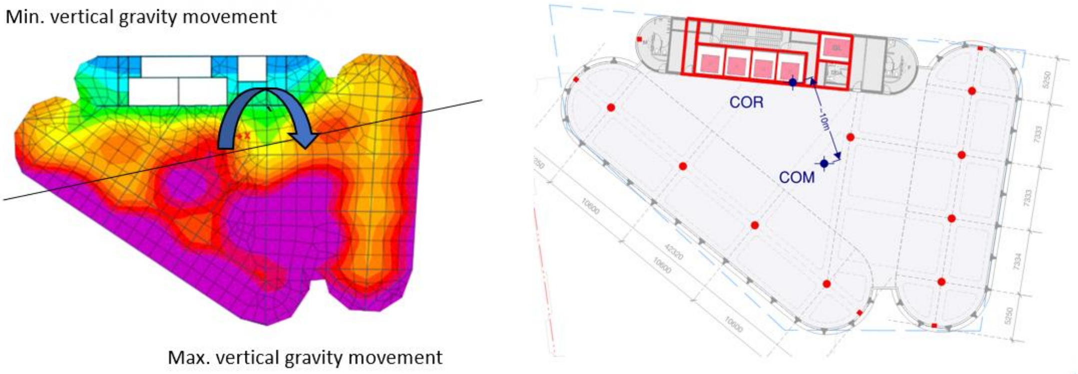


Figure 3 – Location of maximum vertical gravity movement on a typical floor plate

5.3 Lateral Stability System

The lateral stability system consists of an offset reinforced concrete core on the Eastern side of the building with outriggers linking the Northern Columns to the core at the two plant room levels (Level 19 and Level 44). The outriggers may consist of steel trusses, which provide more flexibility for services reticulation through the plant room or reinforced concrete walls.

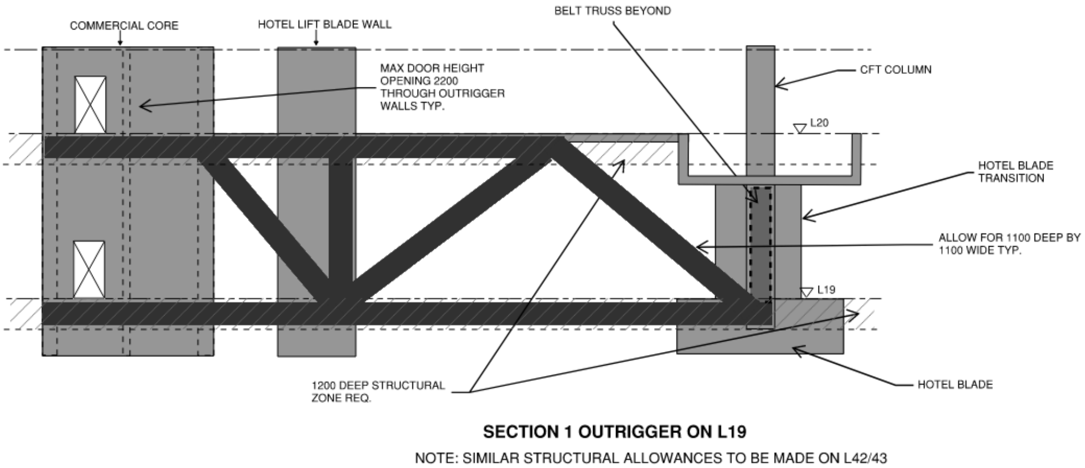


Figure 4 - Section Showing Outrigger (Steel Truss Option) Connected to Columns and Core at Plant levels

The two internal reinforced concrete walls running in the East-West direction within the hotel lift shaft are connected to the core with concrete link beams below Level 22 and contributing to the lateral stability of the building over the lower levels.

5.4 Interface with the former Parcel Post Heritage building (fPPb)

5.4.1 fPPb and Interface Description

The fPPb is a heritage listed building located on the Southern position of the site constructed between 1912 and 1913. A refurbishment of the building took place in the early 2000's which included demolition of portions of the existing structure and installation of a new steel portal structure on the roof, relocation of stairs and lifts and other minor works.

The structural frame consists of concrete encased steel columns and steel beams. It is anticipated the concrete has been provided for fire rating purposes only as was the building practice at the time. On typical floors secondary beams span in the East- West direction dividing the slab span into 3 spans per primary bay. The slab is a reinforced concrete slab of varying thickness. Columns and walls are founded on high level pad footings, allowable rock bearing capacities have been nominated on the existing structural drawings appended to this document.

Due to the age of the building, it is unlikely earthquake loads have been considered in the original design. The building consists of a concrete encased steel frame and masonry façade. There are masonry or concrete walls located around the Western Stair that form part of the original construction and reinforced concrete walls around the central stair that were part of the 2000's works. The current lateral load resisting system is likely to consist of a combination of the façade and internal walls.

The materiality of the perimeter columns is unknown from the available existing documentation however investigation works completed within the basement suggest perimeter columns within the basement may consist of masonry piers. Further investigation works are planned for the tower to determine what these columns are constructed from.

The design of the superstructure and substructure has been designed to minimise the amount of intervention with the heritage structure where possible by having three points of vertical support passing through the existing building. The new structure is not reliant on any support for lateral loads from the existing building.

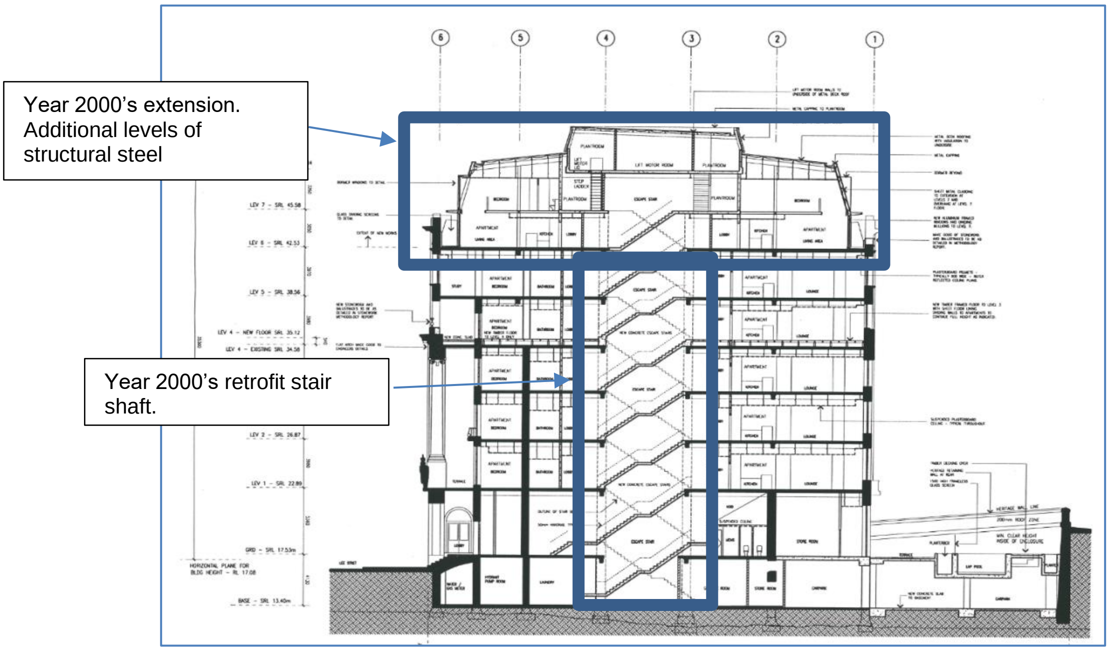


Figure 5 – Architectural section through the existing fPPb

5.4.2 Assessment of the fPPb

RBG have carried out a desktop study based on the existing documentation available. Further testing is required to verify the general condition of the structure and any information missing from the existing documentation such as connection details, grade of steel, concrete grade, thickness and reinforcement details for slabs and encasement and information on the perimeter structure.

The following observations were made from RBG's desktop review:

- Condition of steel, concrete and masonry is to be assessed for any durability issues as part of the site investigation works. Due it's age, it is expected the structure will not meet an additional 50 years of operation without ongoing maintenance over this period therefore an operation and maintenance regime will be required throughout the service life of the structure.

- The existing beams, columns and slabs are expected to have sufficient capacity to resist gravity loads.

- Most connection details between structural elements are unknown from the existing documentation and further investigation works is determined to assess capacity of connections across all elements.

- Further information is required on concrete thicknesses and reinforcement cover to determine if the building will meet the FRL's stipulated in the NCC. Should the existing structure not meet the NCC FRL's, a fire engineering performance solution and/or additional fire protection measures will be required.

- Construction type and details of structure around the perimeter of the building could not be determined from the existing documentation available. Further investigation works are required to determine construction type and details. The current masonry standard AS3700:2018 has height limitations for buildings with load bearing unreinforced masonry walls shown in Figure 6 below. If the perimeter piers are constructed from unreinforced masonry, they will not comply with this requirement of AS3700:2018. To meet this requirement, strengthening of the existing columns using methods such as reinforced concrete jacketing or fibre reinforced polymers (FRP's) could be considered or alternatively new internal columns could be constructed adjacent to masonry piers so the existing unreinforced masonry elements are not relied upon to be load bearing elements.

10.4 RESTRICTIONS ON THE USE OF LOADBEARING UNREINFORCED MASONRY

Buildings with heights greater than the values shown in Table 10.3 shall not incorporate loadbearing unreinforced masonry elements, except where the masonry complies with the following:

(a) The loadbearing masonry supports only a trafficable or non-trafficable roof or mezzanine floor that exerts a maximum of 10 kN/m permanent load to the masonry.

(b) All isolated masonry piers are constructed of reinforced masonry and designed accordingly.

(c) The area supported by the unreinforced masonry is less than 25% of the plan area of the structure on which it is supported.

(d) The unreinforced masonry is not within 3.0 m of the edge of the structure on which it is supported.

(e) The loadbearing masonry components are designed for loads derived from AS 1170.4.

TABLE 10.3

HEIGHT LIMITS FOR BUILDINGS WITH LOADBEARING UNREINFORCED MASONRY

Hazard factor (Z) (as per AS 1170.4)	Height limits, m				
	Subsoil classifications (as per AS 1170.4)				
	A_e	B_e	C_e	D_e	E_e
≥0.11	12	12	12	10	10
0.10	15	12	12	12	10
0.09	15	15	12	12	12
≤0.08	15	15	15	12	12

NOTE: These limits are not intended to apply to small loadbearing unreinforced masonry structures (e.g. plant rooms and mezzanine floors) contained within larger buildings, which may be over the prescribed heights.

Figure 6 - Limitations on use of unreinforced load bearing masonry from AS3700:2018

- The existing lateral load resisting system which is believed to consist predominantly of the unreinforced masonry façade is expected to require strengthening. Some preliminary options that could be considered individually or in combination for strengthening of the existing building are described below.
  - Use of the new structural elements such as the hotel core, commercial core and tower columns passing through the Adina could be used as part of the lateral load resisting system for the existing building. The existing concrete diaphragm may need to be strengthened as part of this option.
  - Strengthening of the existing steel frames and connections between beams and columns to resist seismic loads. The connections are currently not detailed to resist horizontal forces due to seismic loads and therefore welded moment connections would need to be retrofitted to the existing structure at the beam to column connection for this option.
  - A new steel bracing system installed within the fPPb footprint however this method on its own may not be preferable due to the impact this would have on the usage and space within the existing building.

5.5 Sydney Water Interface and Devonshire Tunnel

There is a zone directly adjacent to the Southern Façade of the fPPb containing two existing stormwater pipes and an HV electrical cable at RL+11 to RL+14. The existing Devonshire pedestrian tunnel will be realigned to sit directly over these services.

Part of the basement on the Eastern edge of the site sits directly below these services. Canopy tubes will be utilised to temporarily to support the services in this area while the permanent insitu concrete tunnel structure is built below. Ongoing engagement with Sydney Water is required during the design development.

5.6 Railway Impact Assessment

A engineering impact assessment of the Togal Central development on TfNSW rail assets including CBDRL alignments is being completed by Arup described in the Arup report 'Rail Infrastructure Easement Impact Assessment at Toga Central Version 1 dated 31st May 2022'. The closest tower pad footing is set back 20.6m from the CBDRL tunnel wall. The new basement in the South West corner of the site sits within the second reserve of the CBDRL tunnel, the structure in this area of site consists of a four level basement.

Structural loads will be resolved to mitigate loadings to acceptable levels at the future CBDRL tunnels as shown in Figure 7. Where loads cannot be managed within acceptable limits (Figure 7), de-bonded piled foundations will be used to channel loads to below tunnel invert (at the tunnel) to mitigate impacts on the CBDRL.

6 Western Gateway Sub-Precinct Requirements

6.1 Pedestrian access from Lee Street to the Devonshire Street Tunnel

The existing Devonshire Street Tunnel will be realigned as part of the proposed development. The construction programme and structural design will allow for pedestrian access through the tunnel to be maintained throughout construction. There is no vertical structure located passing through the Devonshire Street tunnel zone.

6.2 Design of Columns to the South of the fPPb within Henry Deane Plaza

The structure to the South of the fPPb consists of four CFT columns spanning clear height from Ground Floor to Level 6 (discussed in more detail in section 5.2). The area atrium area is enclosed by glazing supported on secondary steelwork. Structure in this area has been minimised to maximise the view of the fPPb Southern façade from Henry Deane Plaza and Lee Street.

6.3 Options for Vertical Structure within the fPPb

RBG carried out various structural engineering studies for the proposed Block C reference massing with specific consideration to the sensitive heritage nature of the fPPb. These options are described in section 6.2.1 – 6.2.3 below.

6.3.1 Fully Cantilevered Option

A structure fully cantilevered over the existing fPPb with no vertical structure passing through was investigated as an initial solution. This structural scheme shifted all load bearing structural elements outside the fPPb footprint to the East and South areas of the site. It was found that when the structural requirements to facilitate this load path were combined with the lift shaft spatial requirements there was inadequate spatial allowance to meet the required 12m separation from the adjacent Block A as per the provisions of clause 6.53 of the Sydney LEP 2012 illustrated in Figure 7. This scheme was deemed infeasible due to the breach of the boundary conditions and the applicable planning controls under clause 6.53 of the Sydney LEP 2012.

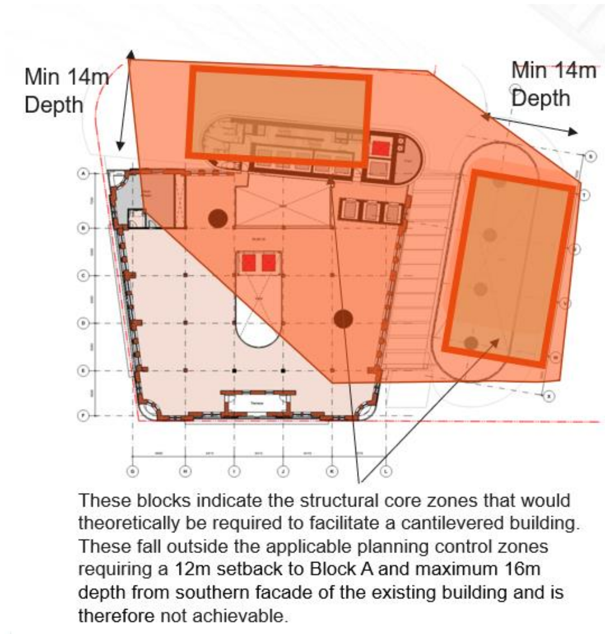


Figure 7 - Diagram showing indicatively required structure to facilitate a cantilevered building

6.3.2 Supporting Tower Structure on Existing fPPb grid

Locating tower columns on the existing fPPb grid as shown in Figure 8 was explored as an option. This option has the benefit of providing a sensitive integration of the old and new structure as the original grid is preserved and/or restored. The existing fPPb columns do not have sufficient capacity to take the tower loads and new columns would need to be circa 1300mm diameter resulting in demolition or significant retrofitting works of the existing columns. The fPPb existing column grid did not result in an efficient hotel and commercial grid meaning a complicated transfer structure is required to realign structural grids for the tower.

1300mm diameter columns

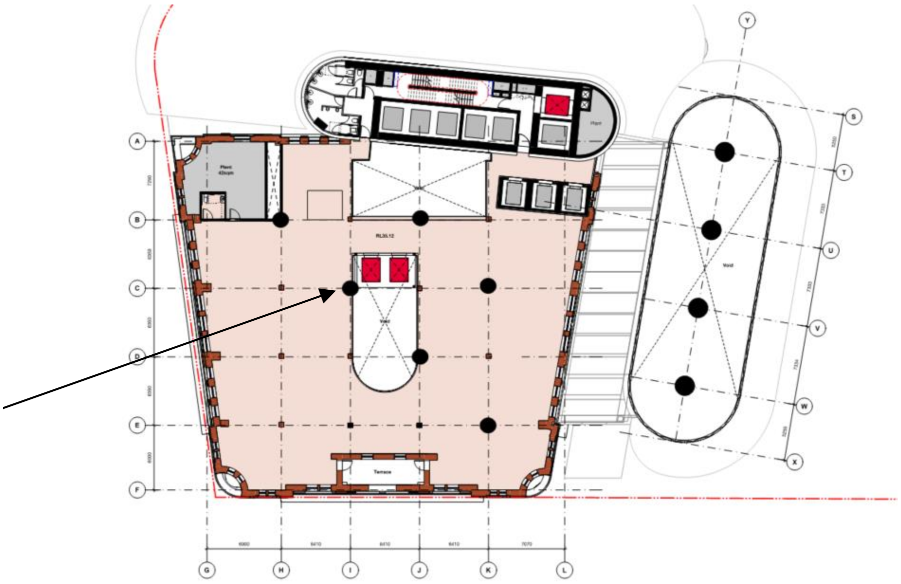


Figure 8 - Tower vertical support required within fPPb on existing structural grids

6.3.3 Supporting Tower Structure off existing fPPb grid

Options to support the tower structure off fPPb grid was explored as a third option shown in Figure 9. The number of columns required to pass through the heritage building was minimised and columns positioned in locations to suit the tower layout above. This option had the benefit of reducing the number of columns requiring either demolition or retrofitting from six to two and columns could be positioned to reduce the amount of transfer structure required between tower and the fPPb. This option was recommended by the design team and adopted in the design.

2500mm diameter columns

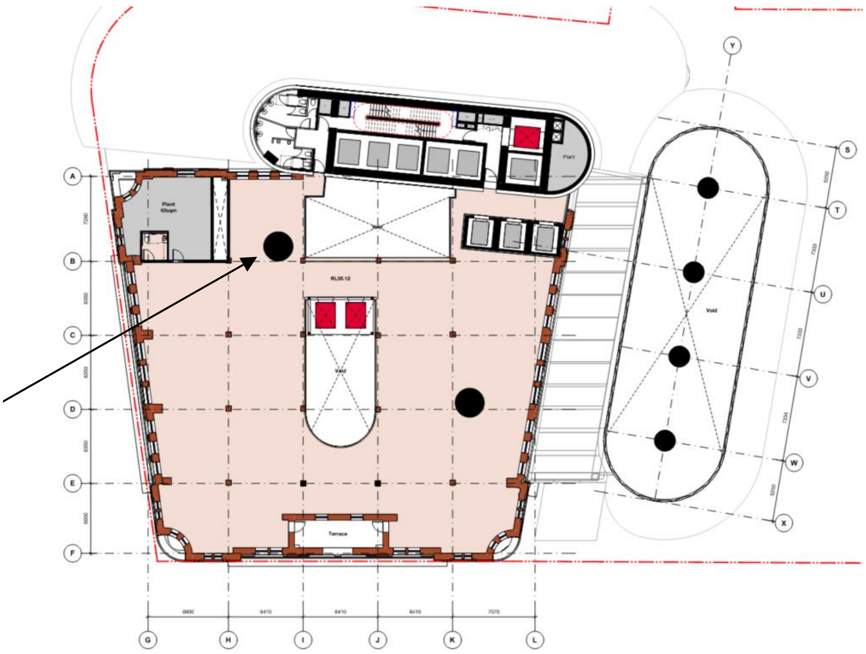


Figure 9 - Tower vertical support required within fPPb off existing structural grids

6.4 Lift Support Structure & Core through fPPb footprint

There is a hotel lift shaft located within the fPPb footprint. The internal two internal walls of the lift shaft are linked to the hotel core and being utilised to stabilise the building with the Northern internal wall linked to the level 20 outrigger.

The design team looked at various alternate options to provide the necessary stiffness to the tower structure without any core structure encroaching within the fPPb.

An option to use only the commercial core for structural stability of the building was explored without relying on the hotel lift core. To achieve the required stiffness, the core needed to grow in either width or length. The planning controls prevented the core to grow any further in width than what is currently shown. An option to grow the core in length and introduce an additional shear wall in the East-West direction was looked at by the design team during the concept design development however it was deemed to not be feasible within the building envelope due to the spatial requirements for building services located on the hotel and commercial levels.

Bracing on the South elevation of the proposed development could be introduced instead of the core walls through the hotel levels however this was not found to be a desirable architectural solution due to the impediment of the view of the fPPb façade to the South and impacts this would have on the usability of the space in the upper levels of the tower.

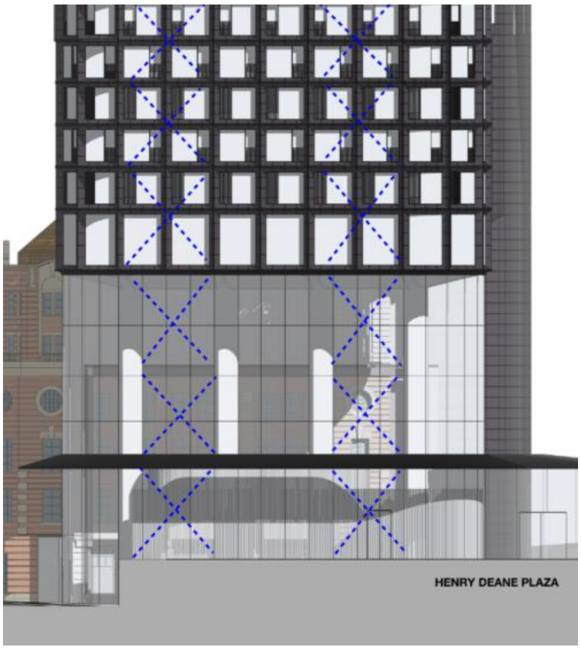


Figure 10 Indicative Cross Bracing on South Option

Reviewing all structural options, locating a small portion of the core structure within the fPPb was found to be the only viable structural option when considering the architectural, planning control and building services constraints.

The exact set out of the lift in the North-South direction is driven by a structural requirement to align internal lift shaft walls with parallel core walls to allow the level 19 outrigger to pass through the lift shaft shown in Figure 11.

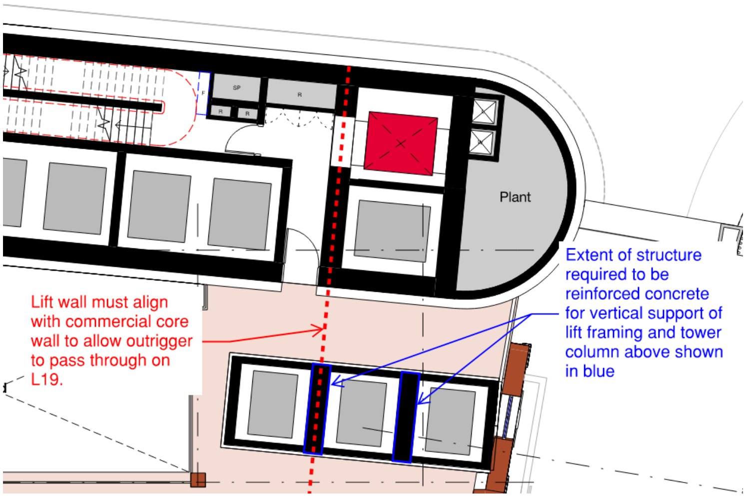


Figure 11 - Structural Constraints for hotel lift

In the Bates Smart design competition submission, the structure supporting the hotel lift shaft consisted of a reinforced concrete core box linked to the commercial core which was providing lateral stability. During development of the concept design, the concrete core walls in this location required for stability were reduced to two blade walls (Figure 11) and commercial core wall thicknesses increased to make up for the loss in structure stiffness by reducing the size of the hotel core required in the lateral resisting system.

This change was implemented to reduce the impact the new structure had on the fPPb heritage structure while still maintaining structure necessary for lateral stability and to support the lift framing and commercial column located above the hotel lift.

The lift shaft wall adjacent to the existing South heritage wall is required for vertical support of the lift shaft only and can consist of a reinforced concrete block wall or lightweight steel frame to minimise impact on the heritage façade. The lift directly adjacent to the Southern Façade termites at RL17.25 instead of RL13 to reduce the risk of undermining the existing footings supporting the heritage façade.

6.5 Sustainability and Environmental Performance

Sustainability has been considered at the forefront of the development of the structural design. Key areas considered are listed below.

- Where dismantling of heritage structure elements is required to facilitate new works, existing structural elements will be reused/reinstated where feasible following investigation of condition and structural capacity of these elements.
- Use of low carbon concretes such as Envisia will be considered for concrete elements on the project.
- Use of high strength reinforcing steel for column fitments such as Viribar 750 will be considered for the project to reduce the quantity of steel.
- An efficient floor framing system within the commercial levels utilising cantilevers to balance spans has been considered to minimise volumes of concrete.
- Flexibility has been allowed for within the design of the commercial framing for future vertical connection between commercial floor plates.

7 Structural Design Loads

7.1 Superimposed Dead Loads and Live Loads

The structure will be designed for the following imposed loads.

Table 1 - Superimposed Dead and Live Loads

Area	Superimposed Dead Load	Live Load
Carparking	1.0 kPa for ceiling/services and hobs/falls	2.5 kPa
Loading Dock	2.0 kPa	As calculated for relevant use. 15 kPa minimum. Includes access to loading dock
Plant Areas	2.5 kPa partitions and plinths 0.5 kPa ceiling and services	As calculated for relevant use. 7.5 kPa minimum
Tank rooms	4.0 kPa plinths	Tank volume as calculated.
Substation and main switch room	As calculated trenches, plinths, and fire rated walls in accordance with approved substation design.	As calculated for relevant use. 7.5 kPa minimum
Stairs and landings	1.5 kPa finishes 0.5 kPa balustrades – internal residential 0.75 kPa/1.5kPa balustrades public spaces	4.0 kPa
Roads	5.0 kPa for road pavement and hard stand areas	25 kPa
Pedestrian and Bicycle Paths	4.0 kPa for external paving finishes	5.0 kPa
Retail	2.5kPa	5.0 kPa
Lobby/Foyer	2.5 kPa finishes 0.5 kPa ceiling & services	5.0 kPa
Residential/Hotel	1.5 kPa.	2.0 kPa
Offices	1.0 kPa moveable partitions 0.5 kPa raised floor 0.5 kPa services	3.0 kPa
Compactus Zones	0.4 kPa raised floor 0.5 kPa ceiling and services	7.5 kPa storage areas over 5%office floor area
Terraces (including trafficable roofs)	2.5 kPa finishes 0.25 kPa ceiling & services	4.0 kPa
Non-trafficable concrete roofs	2.5 kPa finishes 0.25 kPa ceiling & services	2.0 kPa BMU loads as provided by the supplier.

Area	Superimposed Dead Load	Live Load
Lightweight roofs	As calculated.	Generally 0.25 kPa Street awnings 1.0 kPa
Façade	1.7kPa vertically (Loads provided by the Project Façade Engineer Apex)	

Superimposed dead loads include floor finishes, celling, services, and partitions.

7.2 Wind Loads

A wind tunnel assessment has been carried out by RWDI for the project to determine ULS and SLS wind loads and results are contained in the RWDI Report 'Adina Central Redevelopment, Wind Induced Structural Responses' Dated April 19th 2022.

Wind loading parameters are provided below and are in accordance with AS/NZS1170.2

Structure Importance Level	3
Return Period (SLS)	1 in 25 years
Return Period (ULS)	1 in 1000 years
Location	Sydney
Region	A2
ULS Wind Speed	46 m/s
SLS Wind Speed	37 m/s
Structural Inherent Damping (SLS)	1.5%
Structural Inherent Damping (ULS)	3%

7.3 Seismic Loads

Earthquake loading applied to the structural elements and detailing of the seismic stability system will be in accordance with AS 1170.4 – 2007 Earthquake actions in Australia.

Structure Importance Level	3
Earthquake Design Category	III (dynamic analysis required)
Annual Probability of Exceedance	1:1000
Probability Factor, kp	1.3
Structural Ductility, μ	1
Structural Performance Factor, Sp	0.77
Hazard Factor, Z	0.08
Subsoil Class	Class Be

7.4 Blast Resistance

At this stage of the design, there is no consideration of blast loads. The assessment of blast loading should form part of a larger security assessment of the building.

8 Design Criteria

8.1 Design Standards

The structural design for the proposed development will be conducted in accordance with the current revision of all relevant Australian Standards. These standards will include but are not limited to:

AS/NZS 1170.0	Structural design actions – General Principles
AS/NZS 1170.1	Structural design actions – Permanent, imposed, and other actions
AS/NZS 1170.2	Structural design actions – Wind actions
AS 1170.4	Structural design actions – Earthquake actions in Australia
AS 1720.1	Timber Structures Code – Design Methods
AS 2121	Cold Formed Steel Structures Code
AS 2159	Piling Code
AS/NZS 2312	Guide to the protection of structural steel against atmospheric corrosion
AS 2327.1	Composite structures – Simply supported beams
AS 3600	Concrete Structures Code
AS 3700	Masonry Code
AS 3735	Concrete Structures for Retaining Liquids
AS 4100	Steel Structures Code
AS 4678	Earth-Retaining Structures Code
AS 5100	Bridge Design Set
BS 5950-8	Structural use of steelwork in building – Code of practice for fire resistant design
BS 8102:1990	Code of practice for protection of structures against water from the ground
Eurocode 4	Design of composite steel and concrete structures
BCA	Building Code of Australia
CIRIA	CIRIA Reports 139 & 140: Water Resisting Basements
CIRIA 0660	Early-age thermal crack control in concrete

8.2 Design Life and Durability

The basement and tower structure shall typically be designed for a 50-year design life.

The NCC and Standards Australia material standards will be used as the basis for the durability specification for the structures. The structural elements of the buildings shall be designed to provide adequate performance for a minimum period of 50 years as noted above.

All structure within the TfNSW State Works Zone including vertical elements supporting structure within this zone have a more stringent design life requirement and shall be designed for a 100-year design life in accordance with the Australian Standards and NCC.

The existing fPPb was built over 100 years ago and has surpassed its intended design life. Due to the age of the structure, it is unlikely it will meet an additional 50 years of operation without ongoing maintenance over this period therefore an operation and maintenance regime will be required throughout the service life of the structure.

8.3 Deflection Criteria

The structural components shall be designed and constructed to contain any deflections and deformations under service loads for the following criteria and accordance with the deflection criteria specified in AS3600 and AS4100 and Table 2.

Table 2 - Deflection Criteria

Type of member	Deflection to be considered	Deflection limitations (∆/Lef) for spans	Deflection limitations (∆/Lef) for cantilevers
All members	The incremental deflection	1/500 or 20mm maximum	1/250 or 10mm maximum
	The total vertical deflection	1/250 or 25mm maximum	1/125 or 15mm maximum
Members supporting masonry partitions	The deflection which occurs after the addition or attachment of the partitions	1/500 where provision is made to minimise the effect of movement, otherwise 1/1000	1/250 where provision is made to minimise the effect of movement, otherwise 1/500
Mullions and wind columns	Deflection under wind load	1/240	N/A
Storey drift under wind	H/500	N/A	N/A
Overall sway under wind/earthquake	H/500	N/A	N/A
Differential settlement	LT deflection less than span/500 between adjacent elements.	N/A	N/A

A detailed building movement report will be prepared as the design develops outlining the expected building deformations and design and construction tolerances.

8.4 Fire Resistance

Structural elements are to be designed in accordance with the National Construction Code of Australia and the relevant Australian Standards to satisfy the required FRL's for fire. These levels will be advised by the Principal Certifying Authority for the project.

A fire engineered approach will be undertaken in relation to the requirements for passive protection to structural steel wherever possible. Where required, fire protection to structural steel will be provided using fire rated cladding, vermiculite (or similar) spray, or intumescent paint.

The fire resistance periods in Table 3 have been considered in the structural design to date in accordance with the NCC. These are to be further advised by the Principal Certifying Authority and Project Fire Engineer.

Table 3 – Fire Resistance Periods Allowed for in the Structural Design

Area	FRP
Hotel	120/120/120
Commercial	120/120/120
Plant Rooms	120/120/120
Retail	180/180/180

There is not sufficient information available on the existing fPPb documentation to assess whether the building will meet the FRP's set out in Table 3. Once materials investigation works are completed for the existing structure, RBG and the Project Fire Engineer will perform an assessessment to determine the existing structure FRP and whether a performance solution and/or additional passive protection is required.

8.5 Robustness

The structure shall be detailed such that all parts of the structure slab be tied together both in the horizontal and the vertical plans so that the structure can withstand an event without being damaged to an extent disproportionate to that event in accordance with Section 6 of AS/NZS 1170.0.

Exposed columns deemed to be critical structure elements which pose a risk to the public will be critically assessed for risk of accidental loading and appropriate mitigation measures adopted during future design stages.

8.6 Concrete Durability and Crack Control

The requirements of AS3600:2018 will be applied to all reinforced and post-tensioned concrete. The degree of crack control to be provided in concrete elements will be in accordance with AS3600 and AS3735 for structures required to maintain water tightness and CIRIA for high degree of restraint.

In some areas within hotel rooms and on the typical commercial levels an off form finish for the concrete is being adopted. In areas where an off form finish is proposed the concrete will be designed for a strong degree of crack control in accordance with AS3600:2018.

8.7 Steelwork Corrosion Protection

The corrosion protection system for structural steelwork will be dependent on the location of the steel elements within the structure. Systems will be selected in accordance with AS/NZS 2312 as a minimum specification.

Generally, all external steel shall be hot-dipped galvanised with an appropriate finish unless noted otherwise.

8.8 Occupancy Comfort

8.8.1 Floor Vibration

Vibration due to plant should be neither structurally critical nor unacceptable to occupants.

Floors shall be designed to ensure that there are only acceptably perceptible vibrations under footfall effects, or from other internal or external sources.

8.8.2 Floor Accelerations

As part of the RWDI Wind Tunnel assessment RWDI was engaged to predict floor accelerations for the building. The criteria set out in the standards listed in Table 4 was used to assess floor accelerations for occupancy comfort.

Table 4 - Building Accleration Criteria

Return Period	Criteria
1 Year	ISO 10137:2007 (Office Occupancy)
10 Years	NBCC (Office Occupancy)

Predicted building peak accelerations were found to be below the limits for a 1 year and 10 year return period.

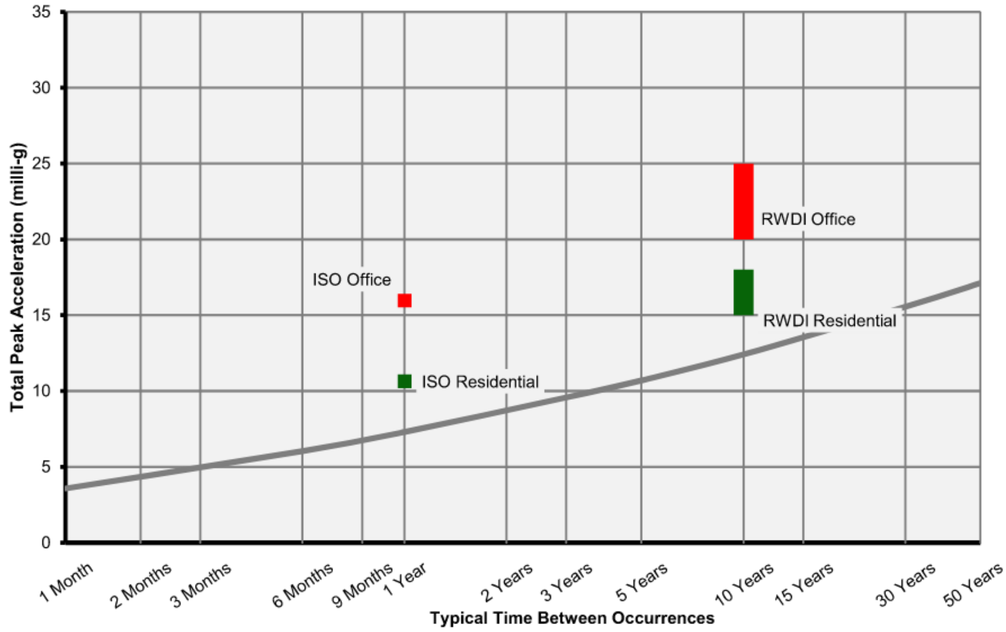


Figure 12 - Extract of predicted peak accelerations from RWDI report 'Adina Central Redevelopment, Wind Induced Structural Responses' Dated April 19th 2022.

9 Constructability

The form of the building and presence of the heritage former Parcel Post building on the site drives the need for a safe and efficient construction methodology. RBG has been working with the Toga Construction team and project design team to ensure the new structure can be built safely and in a way that has no adverse effects on the existing heritage elements on site. Some of the methodologies for construction that have been considered in the permanent works design are:

- Columns below Level 9 are constructed as concrete filled tubes as a permanent formwork system.
- The extent of the hotel core in which the building is reliant on for lateral support has been reduced to allow lightweight construction of the lift support adjacent to the heritage façade.
- The hotel lift adjacent to the heritage façade terminates at ground level to de-risk undermining of the heritage footings.

- A composite steel form-work system is proposed to construct the Level 6 and Level 9 architectural exposed honeycomb slab.

- The core will be built using a traditional jump form system.

10 Conclusion

During the pre-SSDA design development, RBG have provided advice and input into the architectural design by producing preliminary structural scheme sketches and mark-ups on the Architectural plans for the architect to incorporate into the design and attending weekly design coordination meetings.

RBG have undertaken a review of the SSDA plans and confirm that the current Bates Smart scheme has an adequate allowance for a structural feasible detailed design to be developed into future stages of the project and that the building has an adequate allowance for structural support within the proposed building envelope.

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