AGNSW

Art Gallery of NSW Expansion Project - Sydney Modern

Development Application - Structural Report

247039

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Contents

Executive Summary

This report has been produced to support the State Significant Development Application (SSDA) Number SSD 6471 on behalf of The Art Gallery of NSW (AGNSW).

The concept structural design has been prepared in full consideration of the existing infrastructure, with particular reference to the ongoing performance and design life of the land bridge over the Eastern Distributor (stakeholders RMS and Transurban), and the fuel bunker located on the northern portion of the site at Lincoln Crescent.

This report specifically responds to the SSD SEARs Key Issue number 10, which reads:

(10) Infrastructure

- *Detail any infrastructure proposed to service the development and demonstrate that the site can be suitably serviced.*
- *Detail the existing infrastructure on site and identify any possible impacts on infrastructure (particularly on the Cahill Expressway/Eastern Distributor) arising from the construction of the proposed building.*
- *Where the proposed works affect existing infrastructure, the application should detail any mitigation works proposed including service relocations.*
- *Prepare an infrastructure Management Plan. The applicant shall provide information on the required water and waste water services, electricity and gas and any augmentation of Sydney Water and RMS infrastructure that may be required for the proposed development.*

This report has addressed the Key Issue number 10 with particular reference to the structural solution and proposed loading on, and works to, the existing land bridge to adequately protect the asset and serve the development.

1 Detailed Description of Works

The Art Gallery of NSW proposes to undertake a major expansion of the existing art gallery adjacent to the Phillip Precinct of the Domain. The expansion is located north of the existing gallery, partly extending over the Eastern Distributor land bridge, and includes a disused Navy fuel bunker located to the northeast of this land bridge.

The new building comprises a new entry plaza, new exhibition spaces, shop, food and beverage facilities, visitor amenities, art research and education spaces, new roof terraces and landscaping, and associated site works and infrastructure, including loading and service areas, services infrastructure and an ancillary seawater heat exchange system.

Development consent is sought for:

- Site preparation works, including:
    - Site clearing, including demolition of former substation, part of road surfaces, kerbs and traffic islands, pedestrian crossings, foot paths, retaining walls, stairs, and part of disused underground former Navy fuel bunkers;
    - Tree removal;
- Excavation and site earthworks;
- Remediation works;
- Construction of the building comprising:
    - Covered public plaza;
    - Five building levels, including entry pavilion, following the site topography down to Lincoln Crescent;
    - Retention of part of existing disused underground former Navy fuel bunker for use as gallery space and support spaces;
    - Art exhibition spaces;
    - Outdoor publicly accessible terraces;
    - Shop and café;
    - Multipurpose space;
    - Education spaces;
    - Ground level loading dock (accessed via Lincoln Crescent) with associated art handling facilities, workshops, service parking, plant, and storage areas.
- New open staff and administration visitor carpark to rear of Art Gallery building;
- Landscaping and public domain improvements including:
    - Continuation of the east-west pedestrian link between the Domain and Woolloomooloo Bay, including dedicated lift structure for universal access;

  - Improved public access of the north-south pedestrian link;
  - Enhancement of the public open space on the land bridge to create a landscape and art connection between the two buildings;
  - Hard and soft landscaping to roofs and terraces;
  - Plantings and new pathways;
  - Increased landscaped area to the forecourt of the existing Art Gallery building, and removal of car parking;
  - Relocation of selected trees to the south-eastern corner of the site;
  - Sound barrier to edge of land bridge.
- Upgrade works to parts of Art Gallery Road, Cowper Wharf Road, Mrs Macquarie's Road, and Lincoln Crescent, including new pedestrian crossings;
- Provision of vehicle drop off points including a taxi stand, private vehicle drop off and bus/coach drop off at Art Gallery Road;
- Installation of an ancillary seawater heat exchange system to act as the new building's cooling system, adjacent to and within Woolloomooloo Bay;
- Diversion, extension and augmentation of physical infrastructure and utilities as required.

2 SEARs Issues Addressed

This report addresses the following issues identified within the State Significant Development Application (SSDA) Number SSD 6471, as contained within the SEARs issued by the NSW Department of Planning and Environment on 8 June 2016.

(10) Infrastructure

- *Detail any infrastructure proposed to service the development and demonstrate that the site can be suitably serviced.*
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- *Where the proposed works affect existing infrastructure, the application should detail any mitigation works proposed including service relocations.*
- *Prepare an Infrastructure Management Plan. The applicant shall provide information on the required water and waste water services, electricity and gas and any augmentation of Sydney Water and RMS infrastructure that may be required for the proposed development.*

3 The Site

3.1 Topography

The site is located to the north of the existing Art Gallery of NSW, bounded by Art Gallery Road to the west, Lincoln Crescent to the east, and the existing zone substation to the north. The site slopes severely from west to east, from approximately RL24 at Art Gallery Rd at south-west extent of the site, to RL2.9 on Lincoln Crescent to the east.

The new building is required to be located partially over both the existing land bridge constructed over the Eastern Distributor in 1999, and the disused WWII fuel bunkers located north of the Woolloomooloo exit ramp of the Eastern Distributor.

Figure 1: Site contours and preliminary borehole locations

Figure 2: East-west section (Ref: Architectus drawing DA_2000 Rev D)

3.2 Land Bridge

A portion of the southern side of the new building is proposed to be located on the existing land bridge, currently owned by RMS and managed by Transurban. The land bridge was constructed in 1999 with a structure consisting of:

- Reinforced concrete pad and strip footings on Class III sandstone;
- Vertical structure comprising in situ reinforced concrete columns and headstock beams, and reinforced concrete walls located between traffic lanes;
- Horizontal structure of precast Super Ts and precast planks spanning between headstocks and walls with an in situ concrete structural slab.

Above the structure, the existing loading condition comprises nominally 600mm of deep soil landscaping, with zones of deeper soil and trees located strategically over headstock lines. A membrane, screed protection, and drainage layer are provided beneath the soil and connected to a stormwater drainage outlet at the north-eastern (lowest) corner.

The extent of new works has been designed to be accommodated by the existing capacity of the land bridge without the need for strengthening works that will impact the operation of the Eastern Distributor or condition and performance of the land bridge. Figure 3 below shows the extent of new building proposed to be located over the land bridge.

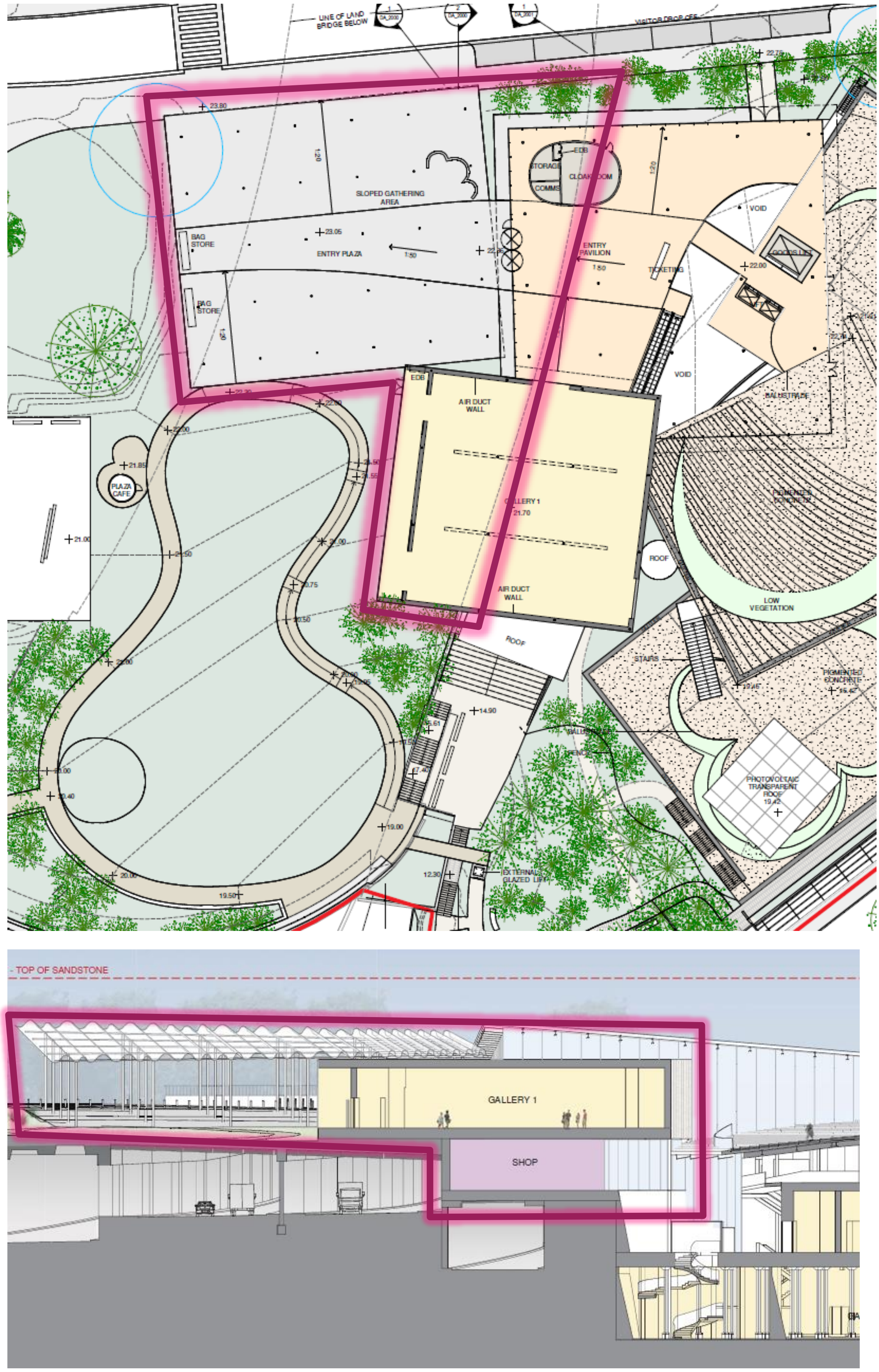

Figure 3: New construction footprint over Eastern Distributor land bridge (Ref: Architectus drawing DA_1001 Rev D top, DA2001 Rev D bottom)

3.3 Fuel Bunkers

The existing WWII fuel bunkers are located on the site of a former sandstone quarry and are of reinforced concrete construction comprising a base slab at approximately RL0.9, perimeter gravity and central dividing walls, reinforced concrete interior columns on an approximately 4.5m grid, and a reinforced concrete flat plate roof with column capitals.

The existing fuel bunkers are proposed to be retained as far as possible within the constraints of the proposed works as follows and shown in Figure 4.

- Roof slab and interior columns retained for the southern half.
- Central wall retained for full width. Local demolition for door openings may be required.
- Base slab retained for the full extent other than the sea water chamber associated with the harbour heat rejection.
- Perimeter wall generally retained, with several exceptions. Part of the western wall will be removed near the location of the lifts and stairs. The north wall will be cut down to accommodate the loading dock access. An opening will be made in the south wall for a new egress ramp.
- Roof slab and columns demolished in the northern half (north of the central dividing wall) to suit space planning of new works.

Figure 4: Extent of existing fuel bunker. Northern half to have roof slab and columns removed. (Ref: Architectus drawing DA_1009 Rev A)

3.4 Ground Conditions

3.4.1 Geotechnical

The Sydney 1:100,000 Geological Sheet indicates that the site locality is underlain by Hawkesbury Sandstone described as medium to coarse grained quartz sandstone with very minor shale and laminite lenses.

Based on the preliminary boreholes summarised in the geotechnical report by Coffey (reference GEOTLCOV25037AA-AF, issued originally on 13 June 2014 and updated to Rev 3 for SSDA on 24 October 2017), the following geotechnical units were identified in the northern portion of the site:

Unit	Material	Description	Approximate Unit Thickness[2] (m)
1	Fill	Sandy Fill. Tile and concrete 0.6 m thick in BH08 only	0.6 – 3.2
2a	Bedrock	Class V / Class IV Sandstone[1]: extremely to highly weathered, very low to low strength, absent in some boreholes.	0 – 4.0
2b	Bedrock	Class III Sandstone[1]: moderately to slightly weathered, medium strength with 0.8 m to 3.4 m thick Class IV Sandstone[1] interbeds where bedding parting and jointing were encountered	5.8 – >18.75 Not penetrated in some boreholes
2c	Bedrock	Class II Sandstone[1]: moderately weathered to fresh, medium to high strength, encountered in BH01, BH06, BH07 and BH08 only	Up to 5.5
2d	Bedrock	Class I Sandstone[1]: generally fresh with some moderately to slightly weathered layer, high strength, encountered in BH01 only	Not Penetrated

Figure 5: Geotechnical Units

3.4.2 Groundwater

Coffey noted in the report that groundwater levels of 8.3m to 12m below the ground surface or RL12.3m to RL13.2m were measured in some of the boreholes (BH04, BH06 and BH08). It was unclear whether this was perched water table or migrating water following rainfall.

Further investigation into groundwater and hydrology has been undertaken during the design period leading to the planning submission to inform the design and construction methodology. This investigation comprised the drilling of two new groundwater monitoring boreholes in Lincoln Crescent, monitoring of groundwater levels with the existing tanks and pump-out sump, groundwater sampling and chemical analysis, and analytical work to investigate design groundwater flows. Reference is made to Coffey reports:

- Sydney Modern – Groundwater monitoring adjacent to former fuel bunkers, ref GEOTLCOV25037AC-L01 dated 19th May 2016;
- Art Gallery of New South Wales Sydney Modern – Fuel bunker inflow assessment, ref GEOTLCOV25037AB-AE dated 20th May 2016.

These reports provided the following conclusions of relevance to the structural and hydraulic design of the building:

- Groundwater flow and gradient is expected to be toward the east/north-east down the hill to Woolloomooloo Bay.
- Groundwater inflow into the tanks responds directly to rainfall events, indicating that this is flow down the hill.
- It was unclear where the entry points into the tanks were, but this is likely to be primarily through previous environmental monitoring holes, with minor ingress through the roof and wall to floor junctions.
- Measured inflow was up to 4.3m^3 per day (3.1 litres per minute) based on current measurement, and upper bound predicted inflows up to 7m^3 per day based on current tank condition.
- Groundwater levels in the Lincoln Crescent measuring points were measured at up to 1.86m below ground level (approximate RL 0.9 to 1.0m), similar to existing tank base slab surface level.
- Seawater was not encountered in any of the samples and groundwater levels do not respond significantly to tide.

3.4.3 Contamination

Reference is made to environmental assessments undertaken by Coffey as listed below. Historical assessments have also been undertaken by GHD, and Woodward Clyde.

- Art Gallery of New South Wales Sydney Modern, Stage 1 Preliminary Environmental Study, ref GEOTLCOV25037AA-AG dated 6th June 2014;
- Sydney Modern – Groundwater monitoring adjacent to former fuel bunkers, ref GEOTLCOV25037AA-AH dated 2nd July 2014;
- Sydney Modern – Groundwater monitoring adjacent to former fuel bunkers, ref GEOTLCOV25037AC-L01 dated 19th May 2016.

4 Structural Description

4.1 Excavation & Retention

Significant excavation is proposed north of the land bridge. The height of excavation is up to approximately 20m in the north-western area of the site, reducing to approximately 8m in the north-eastern corner adjacent to the zone substation.

To protect the existing land bridge structure, Arup has proposed that a nominal 6m clear zone from the rock excavation/face of the abutment wall on the north (road) side of the Woolloomooloo off ramp to the new excavation line on the western side of the existing fuel bunkers is maintained. This 6m zone provides a pillar of minimum Class III sandstone that supports the existing abutment structures and clears the existing rock anchors installed from the road side. Class III sandstone is generally able to stand vertically unrestrained, however localised rock bolts and grouting/shotcreting at any weathered zones is likely to be required. It is expected that previous excavation associated with the fuel bunker and Eastern Distributor works has reduced horizontal in situ stresses and that relief during excavation is likely to be minor.

A preliminary geotechnical assessment of the above approach was undertaken by Coffey, described in a memo dated 27 April 2016 and in the geotechnical report. Three cored boreholes were drilled in the vicinity of the proposed rock pillar and the subsurface profile was found to be fill overlying Class V/IV and Class III sandstone. The assessment indicates that the proposed rock pillar work is feasible and can likely be carried out using rock slope protection techniques typically used in Sydney with retention of Class IV/V sandstone. Appropriate investigation, design, and site inspection and monitoring techniques will be required in future stages.

The proposed excavation and retention techniques are summarised in Table 1. At excavated faces, it is intended that a ventilated cavity is provided between the cut rock face or retention and the gallery external walls. Where there is a shoring system, the slabs extend through the cavity to prop the shoring system, and an external drainage system is provided in the cavity, dropping any water to a subsoil drainage system at the lowest excavated level.

Due to the significant out-of-balance earth pressure loads, a permanently anchored western and north-western wall may be appropriate as an alternative to a propped system. This would require permanent stressed anchors that in areas would extend over the boundary into Royal Botanical Gardens land. This is considered practical and may be cost effective, and will be investigated further during future design stages.

Table 1: Excavation and retention summary

Excavation Face	System
South-west	Vertical rock cut is proposed in Class II sandstone with accessible ventilated cavity nominally 600-900mm wide between rock and building envelope. Rock bolts may be required subject to site inspection of the rock face with drainage at base connected to subsoil drainage system. Backfill from behind existing retaining walls (land bridge abutment) is to be removed. Vertical anchors/rock-bolts through toe of existing land bridge cantilever retaining walls are required. The southern lift shaft and risers will require local excavation to approximately RL0 at Lower Level 4.
South-east	The existing south wall of fuel bunker is planned to be reused. An opening is to be made in this wall for connection to a new egress ramp outside the bunker. The egress ramp will be below ground level, so will require vertical excavation in the sandstone.
West	Contiguous pile wall or soldier piles and shotcrete infill are proposed from surface at RL24 to approximate RL14.8 to retain fill and Class IV/V weathered sandstone. Contiguous piles of ∅600mm diameter with two rows of steel walers is proposed. Alternatively, if fill material is sufficiently stable, ∅750mm soldier piles at 2.25m centres with 150 min shotcrete infill could be adopted. This system requires two rows of temporary anchors, with the addition of walers for the contiguous wall option. Note that in the permanent condition, the piles will be propped by Entry Level and Lower Level 1 slabs with loads transferred to shear walls. *The above solution requires the net out-of-balance earth pressures to be resisted by the building structure. As an* ***alternative,*** *a permanent anchor solution could be considered that mitigates the earth pressures on the building but requires ground anchors on Royal Botanical Gardens land*. Sandstone Class III and better is proposed to be vertically cut to from RL16 to approximately RL9.7 at Lower Level 3 with drainage at base connected to a subsoil drainage system.
North	The retention system is similar to the west for the western half of the northern boundary but following the slope, with height of rock excavation reducing with drainage at base connected to subsoil drainage system. For the eastern half it is proposed to batter the overburden to rock level for a vertical cut. A reinforced concrete retaining wall will be built above the vertical rock cut (as a minimum) as a permanent retaining wall and providing a cut-off to the cavity below where needed.

4.2 Foundations

4.2.1 Lower Level 4

At Lower Level 4, new foundations are generally located within the footprint of the fuel bunker, with a small excavation to the west for the primary service core. Future investigation will be undertaken to determine opportunity to found new primary grid columns directly on the tank base, however at this stage it is assumed that the base slab of the tank will be demolished locally for the new footings for these columns, with any lightly loaded columns supporting only Lower Level 3 founded on the slab only.

Pad and strip footings on medium to high strength sandstone are proposed, and are expected to be supported on Class II Sandstone, with 8MPa allowable bearing pressure interpreted directly from BH1.

New loads over the fuel bunker that cannot be supported by the existing columns will transferred directly to foundation on new columns and footings. Strengthening or augmentation of existing foundations is not currently proposed, however it may be necessary to investigate this in future stages.

4.2.2 Lower Level 3

The northern area is proposed to have pad and strip footings based on 8MPa allowable bearing pressure on Class II Sandstone. Where necessary bearing pressures will be de-rated where footings are adjacent to deeper excavations, or will be extended to lower levels.

4.2.3 Lower Level 2

The north-west area at RL 9.7 is proposed to have pad and strip footings founded in Class II Sandstone. Where necessary bearing pressures will be de-rated where footings are adjacent to deeper excavations.

Elements are also supported by the existing tank walls.

4.2.4 Land Bridge

Refer to Section 5 for more information regarding the foundation system on the existing land bridge structure. The intent of this system is to distribute column loads over sufficient areas to ensure that they are within both the local and global capacity of the existing structure. A combination of pad footings, strip footings, waffle slabs, and spreader beams and walls are proposed for this purpose.

4.3 Vertical Structure

4.3.1 Columns

4.3.1.1 Structural Grids

The new works are divided into zones with the following structural grids.

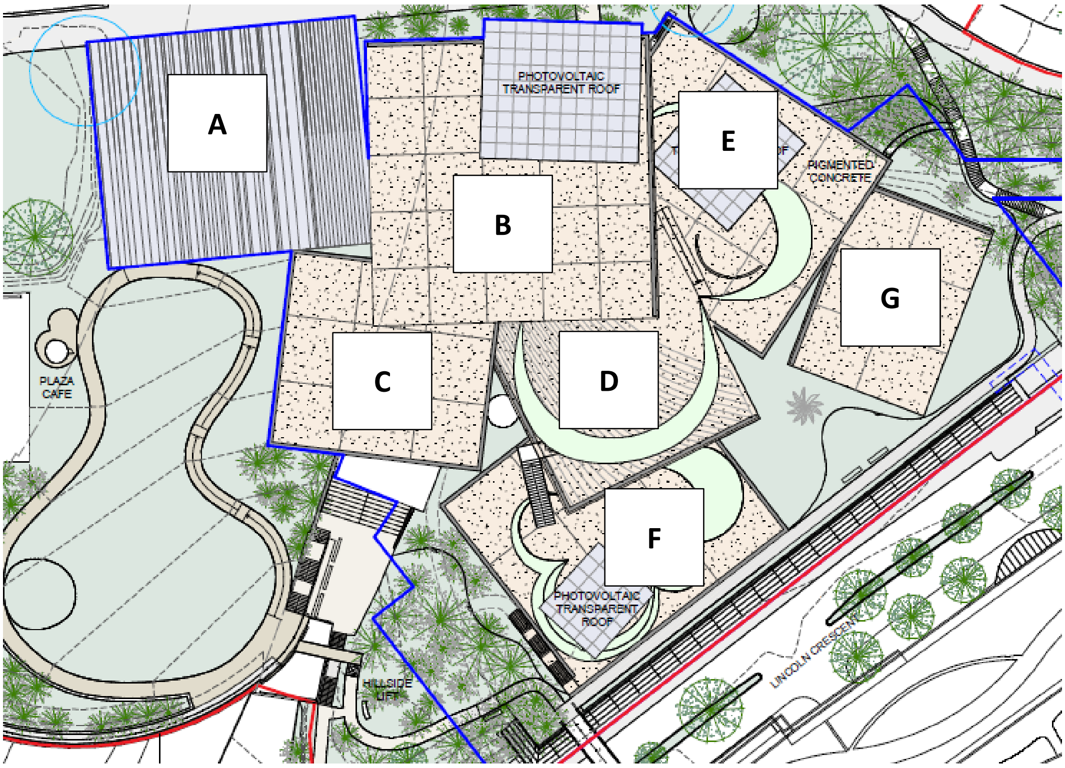

Figure 6: Zone naming

Table 2: Zone structural grids

Block	Use	Grid (m)
A	Entry Plaza (canopy only)	7.5m square
B	Entry Pavilion	9.4m square
C	Gallery 1	10 x 10.2m
D	Café, atrium	9m square
E	Gallery 2, 4 and 6, with terrace over	9.1m by 12.2m
F	Gallery 3, with café and terrace over	12m by 13.5m
G	Multipurpose space	6.5-7.6m x 17.2m

Part of Zones A, B and C lie on the land bridge.

Zones F and G lie on the existing fuel bunkers. The fuel bunkers have a nominal existing grid of approximately 4.5m square.

4.3.1.2 Reinforced Concrete Columns

North of the land bridge, reinforced concrete columns are proposed from Lower Level 4 to the floors of all gallery areas as part of an in situ concrete frame. Columns are medium to high strength concrete up to 65MPa, and are generally circular in open areas and rectangular when in walls and where required to suit planning.

Concrete columns are braced by floor diaphragms and associated reinforced concrete shear walls.

Where above uppermost gallery floors and retaining walls, expressed steel columns extend to roof level.

4.3.1.3 Fuel Bunker - Existing

The existing precast fuel bunker columns in the southern section are proposed for reuse, generally maintaining support to the existing roof slab as per current and historical function. Survey information has been provided on column size and historic photos used to assess likely reinforcement quantities. No original structural drawings have been sourced to date.

An intrusive investigation is proposed to assess the strength, durability, and fire resistance period of the existing columns, as well as walls, footings and slabs. The extent of any works to existing elements will depend on the results of this investigation.

Since Zone F overlays the southern half of the bunker which is to be reused, and the new columns do not align with the existing bunker columns, it is proposed to transfer the new column loads to the existing column grid using two-way steel beams just above the bunker roof. It has been estimated that the existing columns have sufficient capacity to carry the transfer loads, but this will be confirmed following the intrusive investigation into the fuel bunker structure, and if necessary, strengthening of the columns may be undertaken.

4.3.1.4 Structural Steel

Structural steel columns are proposed to support all pavilion lightweight and composite roof structures.

Where the columns are in internal spaces they are generally proposed to be expressed to keep sizes small. Passive fire protection, where required, will be provided by intumescent paint, possibly supplemented by concrete core filling. A fire engineered solution will be used to minimise passive fire rating requirements.

4.3.2 Reinforced Concrete Walls & Global Stability

Reinforced concrete walls are provided for various functions that include:

- Lateral stability for transient loads (wind and seismic);
- Notional load and robustness requirements;
- Resistance of global out-of-balance soil loads. This is primarily with the high level fill and residual soils to be retained on the western and northern boundaries;
- Gravity load support;
- Transfer walls; and
- Cantilever and propped retaining walls.

Walls are located and sized accordingly for their performance requirements, with concrete strength up to 65MPa.

The building is stabilised by a combination of the existing fuel bunker walls and new reinforced concrete shear walls. Roof structures are stabilised by either braced or sway frames.

4.4 Floor Structures

4.4.1 Lower Level 4

Lower Level 4 is mostly within the footprint of the existing fuel bunker. Demolition of a portion of an existing wall will be required for construction and access into the new central stair, lift, and services cores.

The top of the existing fuel bunker slab is nominally at RL0.9. With reference to Section 3.4.2, investigation is ongoing to assess site hydrology and groundwater levels. Preliminary investigation indicates that groundwater is at or near the base interior level of the tanks, and water enters the tanks after rainfall. Based on available evidence it is likely that this is entering primarily through previous investigation holes in the tank base, and possible through the roof slab, wall, and base slab junctions to a lesser extent.

The lowest habitable level has been set at RL1.25. It is intended that this is a raised slab with a drained cavity between the soffit of this slab and the top surface of the existing fuel bunker base slab. An appropriate membrane/vapour barrier will be required to the soffit of this slab, and a new subsoil drainage system installed. A ventilated cavity will also be required against the existing tank concrete walls for any spaces that require a reasonable degree of environmental control. The heritage use of the fuel bunker space may define the solutions further investigated.

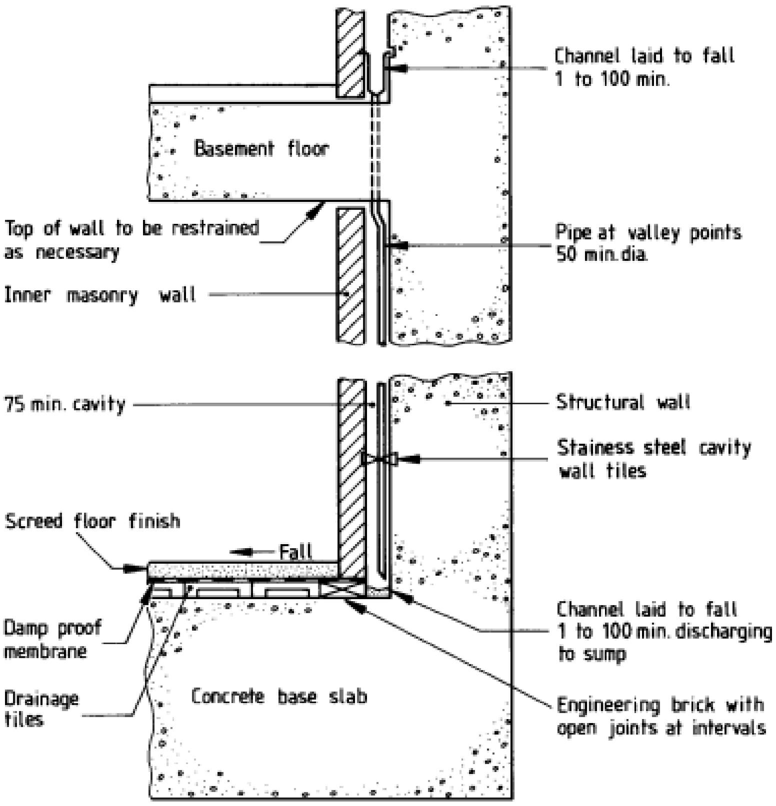

Figure 7: Typical drained floor construction

The existing tank structure will require remedial works to joints and other locations of water ingress to provide adequate control of groundwater inflow to a rate that it can be adequately managed by the drainage system. It is likely that this will be by means of joint and crack injection, and plugging of existing holes and penetrations. New foundations that are required to penetrate the existing base slab will require similar treatment.

Below RL0.9, any basement areas, tanks, or pits should be considered as fully tanked reinforced concrete elements and designed for appropriate hydrostatic pressure.

Dewatering will be required during excavation. Grouting of sandstone seams in excavated faces shall be allowed for assisting to manage groundwater inflow through the sandstone.

4.4.2 Lower Level 3

Lower Level 3 comprises a mix of work inside and outside the footprint of the existing fuel bunker. The majority of slab is within the footprint at RL4.5 for conservation, workshops, storage, and the upper level of the loading dock.

The northern loading dock at RL3.5 is north of the line of the existing fuel bunker structure and is proposed to be constructed of 250mm thick slab on grade on hardcore over the rock excavation. Bulk excavation in this area is proposed at RL3.15 and sloping down with the driveway to Lincoln Crescent.

The remainder of the floor is suspended slab, proposed to be constructed of partially post-tensioned concrete band beam and one-way slab construction, which may or may not be designed continuously with the above 250mm thick ground bearing slab. Temporary and/or permanent separation form the existing tank structure will be necessary to minimise restraint to short and long-term in-plane shortening due to prestress.

4.4.3 Lower Level 2

The southern roof of the fuel bunker is situated at approximately RL8.9 and is proposed to be retained. Gallery 3 over has a finished floor level of RL9.7, and will require a new floor to accommodate this raised level. It is proposed that this is a composite slab on steel beams, supported on stub columns aligned with the fuel bunker columns below. Columns supporting Roof F over this level do not align with the existing fuel bunker columns, so will be transferred onto those columns via two-way steel beams within the new floor where possible.

The suspended areas of the central foyer, Gallery 6, multipurpose space and northern external landscaping are proposed to be of post-tensioned banded slab construction on the primary gallery grids, subject to headroom constraints and services reticulation requirements. If band beams cannot be accommodated outside of the loading dock area, either flat slab (drop panel) construction will be utilised or alternatively intermediate columns extended to the underside of gallery level to allow a flat plate on a 6m grid. Appropriate detailing will be required to accommodate initial post-tensioning and ongoing creep shortening.

The north-west area including Gallery 4 which is outside the footprint of the fuel bunker is proposed to be a 170 thick jointed ground bearing slab on 100 mm hard core filling.

4.4.4 Lower Level 1

This northern area for Gallery 2 is proposed to be of post-tensioned two-way banded slab construction. Some of the bands will act as transfers to account for differing column grids above and below.

Part of the southern area consists of a 170mm slab on grade for the shop and southern entry steps. An internal footbridge spans from this area eastwards to the café, which is built off the structure of Roof F.

4.4.5 Entry Level

The Entry Level works are primarily associated with the Entry Pavilion and Gallery 1, which are located on the existing land bridge structure.

Typically, this floor structure comprises a 140mm thick reinforced concrete waffle slab on void formers. The nominal 5m grid of downstand walls supporting the slab and bearing onto the existing land bridge is also utilised for spreading column loads from the suspended floor and roof over. The northern part of Gallery 1 is suspended, consisting of a composite slab on steel beams spanning from the northern wall of the land bridge over the shop and circulation space of LL1 below.

4.5 Roof Structures

With the exception of the Entry Canopy, all roof structures are proposed to be steel-framed structures, with composite steel Universal or Welded Beams supporting lightweight concrete slabs, supported by circular (CHS) steel columns. Finishes vary depending on trafficability. Stability systems depend on the availability of internal bracing elements.

4.5.1 Entry Canopy (Roof A)

The Entry Canopy consists of a freestanding roof on steel columns. Options for the roof structure include curved glass and curved aluminium on steel beams.

Stability is provided by the columns cantilevering from pad footings on the land bridge.

4.5.2 Roof B and B

Roof B is a non-trafficable roof with hard finishes. Stability is provided by sway frame action with the CHS columns.

Roof B’ is a lightweight planar roof clad in photovoltaic panels, sitting above Roof B. The framing system is structural steel stub columns and beams stabilised by frame action. Roof cladding will be either standing seam metal roofing or flat sheet below the PV panel system. The PV panels will be on a sub-frame fixed to the roof sheeting if standing seam, or on pedestals if flat roofing.

4.5.3 Roof C

Roof C is a non-trafficable roof with hard finishes. Stability is provided by bracing within the perimeter walls.

4.5.4 Roof D

Roof D is proposed to be trafficable, with a combination of landscaping and hard paving. The composite concrete slab on steel framing system allows a continuous membrane to the slab top surface beneath the drainage and landscaping build-up. It is intended that a lightweight landscaping system be used to reduce mass.

Stability is provided by reinforced concrete cores.

4.5.5 Roof E and E’

Roof E is proposed to be trafficable, with a combination of landscaping and hard paving proposed to have a combination of landscaping and hard paving, similar to Roof D.

Stability is provided by a reinforced concrete core.

Roof E’ is a lightweight roof with photovoltaic panels, built off Roof E, similar to Roof B’.

4.5.6 Roof F and F'

Roof F is proposed to be trafficable, with a combination of landscaping and hard paving proposed to have a combination of landscaping and hard paving, similar to Roof D and E.

Stability is provided by a bracing within perimeter walls.

Roof F' is a lightweight roof with photovoltaic panels, built off Roof F, similar to Roof B' and E'.

4.5.7 Roof G

Roof G is a non-trafficable roof with hard finishes. Stability is provided by bracing with internal and/or perimeter walls.

4.6 External Landscaping over Fuel Bunker

A portion of the external landscaped area over the fuel bunker may need a new suspended structure to avoid soil depths overloading the existing structure. The depth of planting is nominally 500mm spread evenly. This suspended structure would be supported from the structure below, and will need to be constructed to suit the contours whilst maintaining some structural efficiency.

The slope of the finished landscaping may vary in angle to greater than 50 degrees. This would require terracing in the structure to allow planting depths and slope stability. The proposed preliminary solution is a stepped reinforced concrete slab with steel columns located along the step lines. It is proposed that this slab is a 150mm thick folded reinforced concrete slab on a nominal 4.5m grid, matching the spacing of the existing fuel bunker columns below.

5 RMS Infrastructure – Land Bridge

5.1 Approach

On the basis of minimising any works to the land bridge, and in particular to obviate or minimise the need for works from the Eastern Distributor, the section of building to be constructed on the existing land bridge is being designed within the local and global load carrying capacity of the existing structure. To this end, Arup has reviewed all available information and conducted a detailed numerical assessment of the vertical and horizontal capacity of the existing structure to inform the development of the project. We have not relied solely on a like-for-like design load replacement.

An initial meeting with RMS and Transurban was held in February 2016 to discuss the approach described below. The proposed building scheme at that stage included two storeys of new structure on the land bridge. The scheme has since been reduced in size, and now comprises a single storey over the land bridge, covering a smaller total plan extent than before, significantly reducing the loads imposed on the existing structure. A meeting was held with RMS, Transurban and its reviewing engineer AECOM on 16 October 2017 to update them on the scheme. Further consultation with those parties will be undertaken in future design phases as required.

5.2 Gravity Load Analysis

Arup has conducted a gravity load capacity assessment of the girders, planks, columns, walls, and foundations based on the structural drawings provided by Transurban. The results of the assessment are shown in Figure 8, in the form of structural utilisations under the original design loading, incorporating both the construction phase and permanent load conditions. This assessment has informed the extent of new structure proposed on the land bridge, and the approach to local support and load distribution from the new structure over.

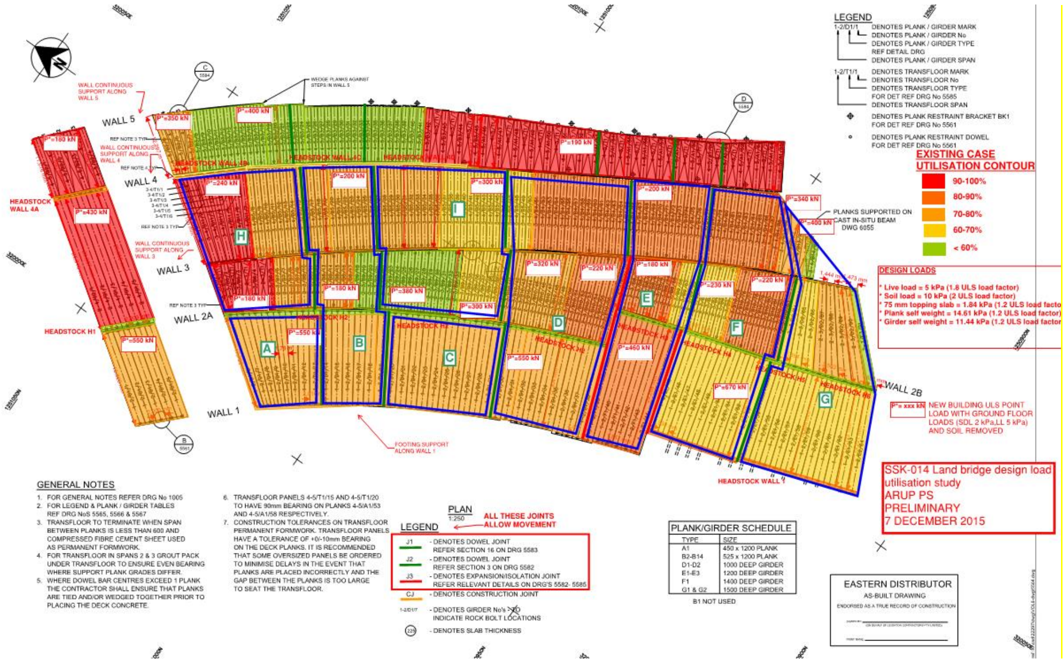

Figure 8: Structural utilisation of land bridge under original design loading (Arup sketch based on Maunsell McIntyre drawing N641/N/ST/5580 Rev AB0)

New foundations in the form of pad footings, footing beams, or grillages are required to spread proposed concentrated column loads onto the existing precast planks, girders, and composite slab, to avoid local overstress. The proposed foundations are as follows:

- Entry Plaza:
  - Pad footings poured directly on land bridge surface, in lieu of existing fill removed locally.
  - Footings support columns to Entry Canopy.
- Entry Pavilion (southern portion):
  - 150 thick reinforced concrete waffle slab over void formers on a nominal 5m grid of downstand walls. Continuous bearings (e.g. Granor or Herculslip) under downstand walls.
  - Downstands support columns to a steel-framed roof.
  - Northern portion is not over land bridge. It is a suspended structure supported by columns through internal space below.
- Gallery 1 (southern portion):
  - Waffle slab similar to Entry Pavilion, with downstands supporting columns to a steel-framed roof.
  - Northern portion is not over land bridge. It is a suspended steel-framed structure, mostly supported by columns through internal space below. Along northern edge of land bridge, suspended structure is supported by existing land bridge wall via new steel bracket columns.

The floors of both the Entry Pavilion and Gallery 1 are founded directly on the land bridge and built up using loadbearing styrene, or a simple waffle slab, providing a distributed loading on the existing structure and minimising concentrated loading. Jacking of the system using flat-jacks or hydraulic jacks to distribute load evenly may be required where depth of new structure is limited, the supporting structure is very stiff, or preloading of adjacent planks is necessary.

5.3 Lateral Stability

Based on the original land bridge drawings, Arup has assessed that stability is provided by multiple concrete shear walls in the road carriageway direction, and by the rock bolts on the Gallery side of the bridge in the perpendicular (bridge span) direction. The original design drawings indicate that the land bridge was designed in accordance with the provisions of AS1170.4 (earthquake loading for building structures).

Transurban has provided Arup with a preliminary structural design report for the land bridge issued in June 1997 by Maunsell Pty Ltd and Connell Wagner Pty Ltd. The report confirms that passive rock anchors are required to resist seismic demands to AS1170.4 with the following parameters:

- *Importance factor:* 2;
- *Design category: B*;
- *Design lateral load:* 5% of the design vertical loads.

Seismic assessment of the existing structure in accordance with the current standard AS5100.2 undertaken by Arup indicates that the number of rock bolts currently provisioned has limited capacity to resist additional seismic load in the spanning direction. The original drawings also indicate a number of locations of movement joints in both directions that would be undesirable for adding significant extra loading to the structure.

Based on this assessment, the principles for ensuring lateral stability of both the land bridge and the new structure are as follows:

1. Limit the future global gravity load to within the original global design load, thus avoiding an increase in seismic demand on the land bridge. This will avoid the need for the seismic upgrading described for the previous scheme.
2. Provide a clear lateral load path from the new structures supported by the land bridge to stability structures in the new building, founded to the northeast of the land bridge.
3. Provide lateral load separation of the new structures and land bridge such that the lateral stability of each is provided independently.

The first part of the strategy is achieved as described in 5.2 above. The second part is achieved through:

a. The bearings under both pavilions, which allow relative lateral movement between the new structures and the land bridge, limiting the seismic actions that can be transferred between them.

b. Reinforced concrete shear walls under both pavilions, which provide a stability system independent of the land bridge. The key stability elements are highlighted in Figure 9. It is noted that the key stability element under Gallery 1 is proposed to be founded on the base of the existing land bridge Wall 5.

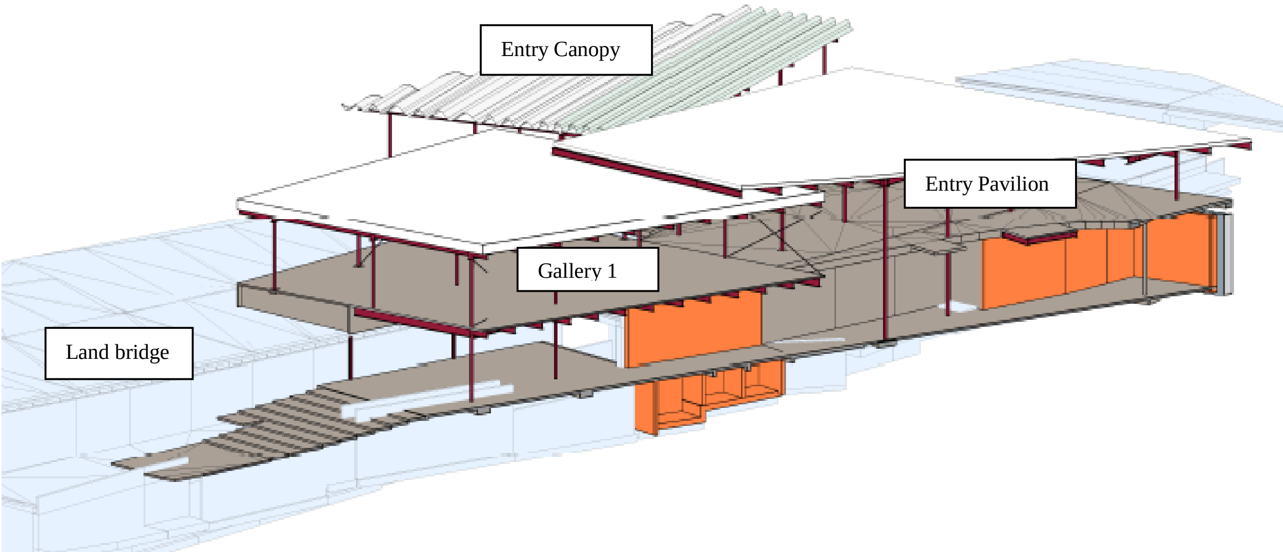

Figure 9: Key stability walls (orange). These elements resist lateral loading from new structures supported by land bridge under gravity (i.e. northern parts of Entry Pavilion and Gallery 1).

5.4 Northern Excavation and Rock Pillar

To protect the existing land bridge structure, Arup has proposed that a nominal 6m clear zone from the rock excavation/face of the abutment wall on the north side of the Woolloomooloo off ramp to the new excavation line is maintained. This 6m zone provides a pillar of minimum Class III sandstone that supports the existing abutment structures and clears the existing rock anchors installed from the road side. Class III sandstone is generally able to stand vertically unrestrained, however localised rock bolts and grouting/shotcreting at any weathered zones is likely to be required. It is expected that previous excavation associated with the fuel bunker and Eastern Distributor works has reduced horizontal in situ stresses, and that relief during excavation is likely to be significantly reduced.

A preliminary geotechnical assessment of the above approach was undertaken by Coffey, described in a memo dated 27 April 2016 and in the geotechnical report. Three cored boreholes were drilled in the vicinity of the proposed rock pillar, and the subsurface profile was found to be fill overlying Class V/IV and Class III sandstone. The assessment indicates that the proposed rock pillar work is feasible and can likely be carried out using rock slope protection techniques typically used in Sydney with retention of Class IV/V sandstone.

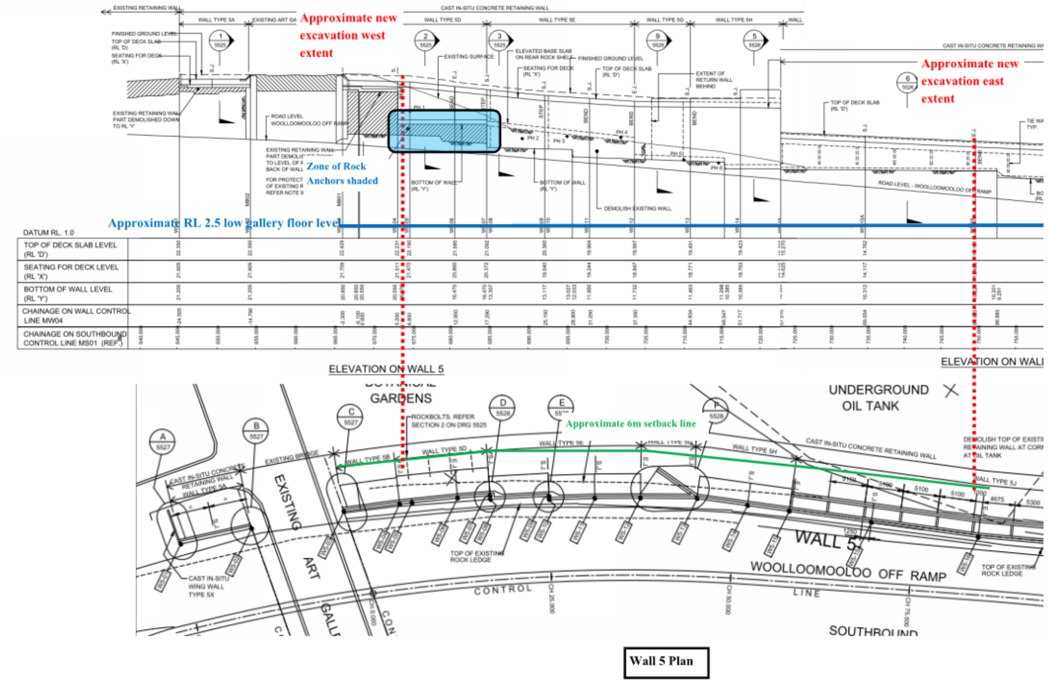

Figure 10: Proposed limit of nearest excavation below land bridge north wall footing (green line on plan)

6 Heritage Impacts

6.1 Existing Gallery

No structural modifications to the existing Art Gallery building are proposed as part of this SSDA.

6.2 Fuel Bunkers

6.2.1 Extent of demolition and reuse

The extent of proposed demolition of the fuel bunker structures is described in Section 3.3. The majority of the base slab, perimeter walls, and approximately 50% of the existing interior columns and roof slab are proposed for reuse.

Works associated with the existing walls, base slab, roof slab, columns and foundations are subject to investigation, inspection, and testing to confirm both structural capacity and condition. Arup has proposed the following preliminary testing to assess the condition and load capacity of the existing structure:

- Various test cores for strength and durability testing;
- Mapping of reinforcement;
- Contingency for some remedial patching and reinforcement replacement to maintain current capacity.

6.2.2 Roof Slab Loading

It is intended that the loading applied to the tank roof slab does not exceed the current condition, however the design adequacy and condition are currently unknown.

If major strengthening works such as flexural strengthening (for example as bonded carbon fibre strips) or shear strengthening (grouted studs or brackets) are necessary or desired, further assessment would be undertaken to review whether this provides adequate future durability, or if the roof slab should be replaced.

6.2.3 Columns

The existing precast fuel bunker columns are proposed for reuse, generally supporting the existing roof slab and not increasing their current load capacity. A representative sample of cores for strength and durability testing of concrete and reinforcement is proposed.

The extent of any works to existing columns will depend on further testing and inspection. This includes assessment of cover to reinforcement and adequacy for fire.

6.2.4 Perimeter Walls

The existing perimeter walls are proposed to be used for retention, groundwater control, gravity load support, and stability. Appropriate testing and remedial works will need to be undertaken.

6.2.5 Base Slab

The existing base slab is proposed to be retained for support of a new raised floor structure that provides a drained cavity between existing and new. It is also proposed to found more lightly loaded columns directly on this slab wherever feasible. Appropriate testing and remedial works (if required) will need to be undertaken. Localised areas of base slab in the north bunker will be demolished to allow for pad footings to new columns and for lift pit bases, as well as for the sea water chamber associated with the harbour heat rejection system.

7 Marine Structure for Harbour Heat Rejection

7.1 Introduction

A new harbour heat rejection system is being proposed for the new building. This section outlines concept level advice for the civil works associated with the intake and outfall (discharge) points.

7.2 Input Information / Assumptions

This advice has been based on the following inputs:

- OVA Harbour Heat Exchange Plan (Architectus, 20/06/16).
- Hydrographic Chart AUS 202 Port Jackson (Central Sheet) Sydney Harbour.
- A site visit on 10/05/16 – at low tide conditions.

The heat rejection system is proposed to be an open system where seawater is pumped via the intake to a heat exchange system and returned to the harbour via the outfall (rather than a closed system).

7.3 Civil Works Concepts

7.3.1 Intake/Outfall Locations

The current concept locations for the intake/outfalls have been overlaid on the Hydrographic Chart AUS 202, and this is shown in Figure 11. A review of the chart and site visit suggests that the proposed intake/outfall locations would be located in approximately 8-10m of water which is considered to provide a sufficient minimum water depth for operations.

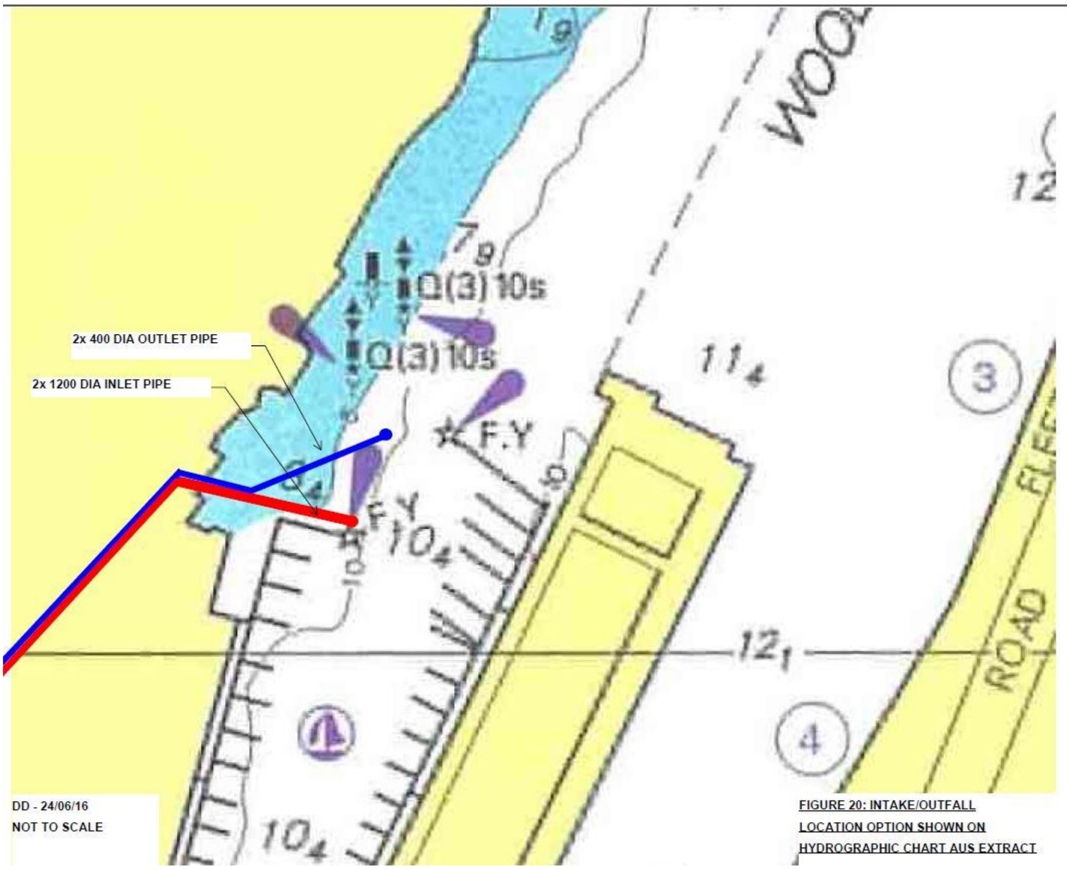

Figure 11: Intake/outfall location option shown on hydrographic chart

7.3.2 Onshore Pipework from Pump Room to Harbour

A High Density Polyethylene (HDPE) or GFRP pipe material or similar is typically adopted in such systems because of its relatively high resistance to corrosion and marine fouling. The proposed size of pipes is as follows:

- Intake pipes: 2no. x 1200mm diameter.
- Outfall pipes: 2no. x 400mm diameter.

The pipework is proposed to be buried in engineered trenches near or under the existing pavements. Refer to Figure 12.

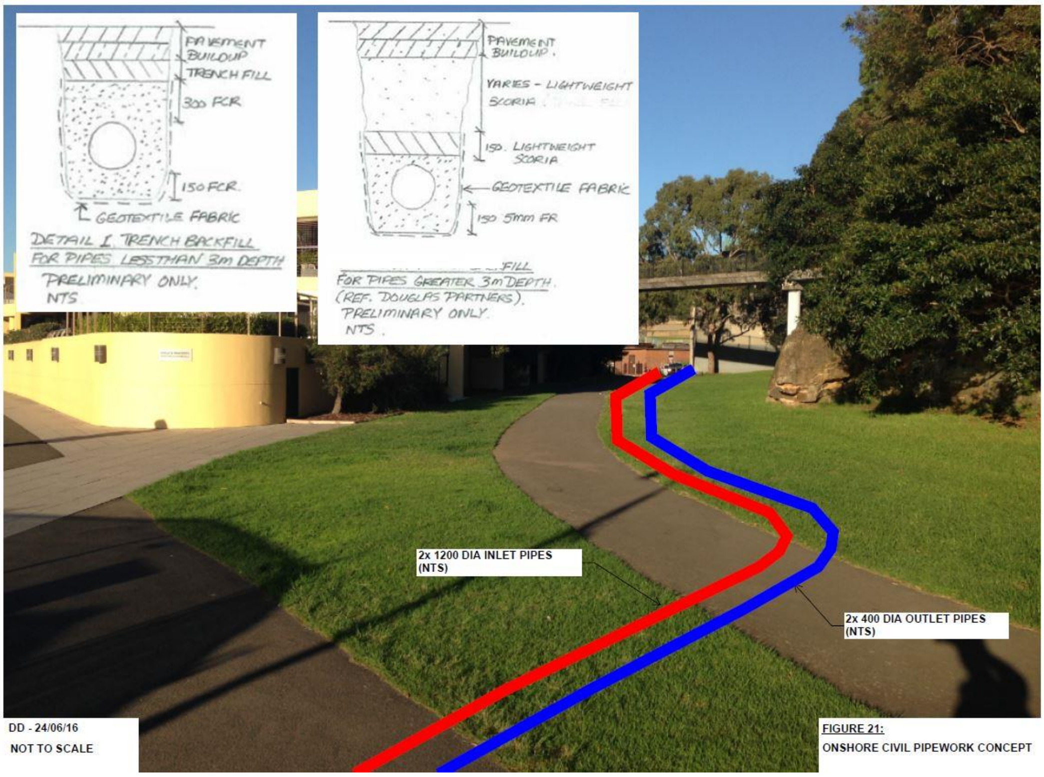

Figure 12: Onshore civil pipework concept

7.3.3 Intake and Outfall Infrastructure into Harbour

Key considerations for the design of the intake and outfall points are:

- Maintain a minimum water depth of the intake opening at Lowest Astronomical Tide (LAT) level.
- Located to be clear of harbour normal vessel navigation from potential impacts. The offshore system should be adequately marked as a hazard to navigation.
- The intake opening to incorporate an appropriate filter/gross pollutant screen.
- The intake located and positioned adequately above the seabed to minimise siltation.
- Intake and outfalls to be designed for seabed scour from vessels and harbour currents/waves.
- Minimal aesthetic impact to the existing foreshore.
- Minimal heritage impacts.
- Optimise access for maintenance.

Proposed high level concept for the maritime design of the intake and outfall system are provided on Figure 13 and Figure 14.

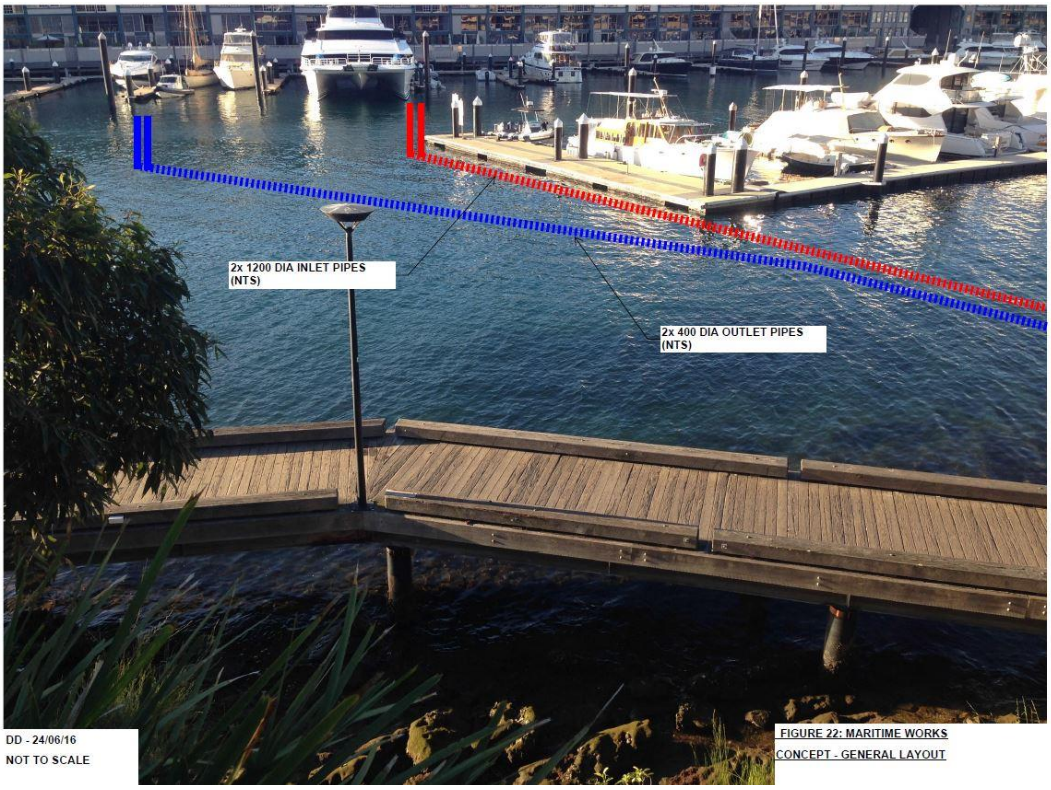

Figure 13: Maritime works concept - general layout

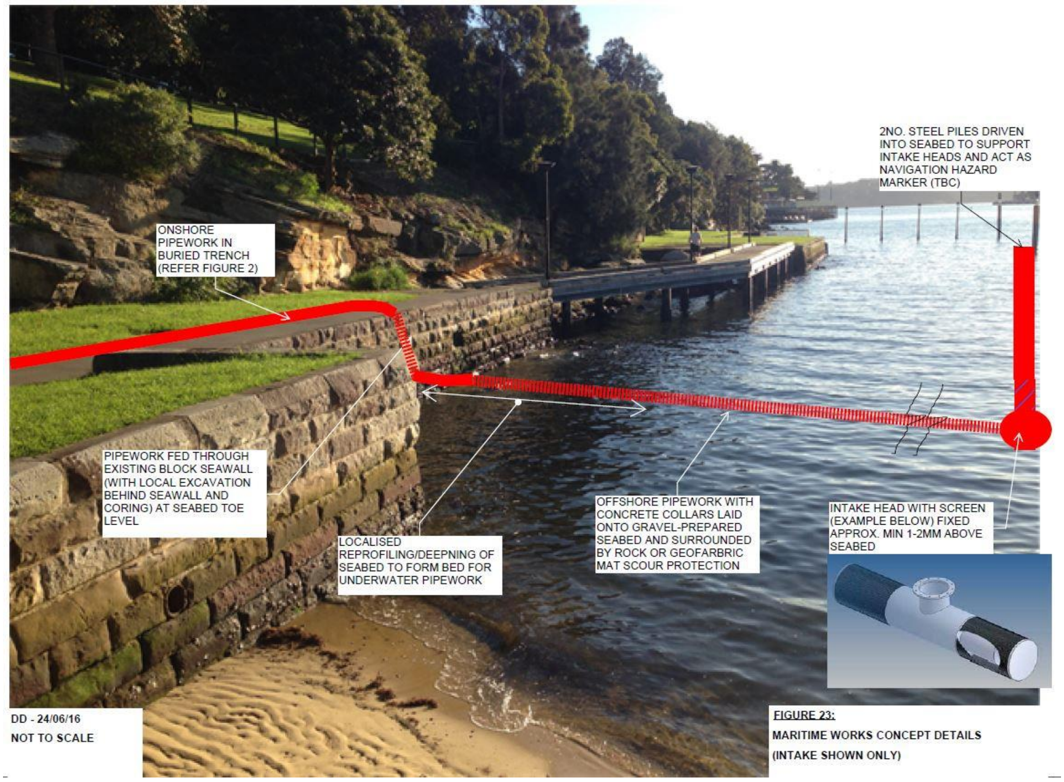

Figure 14: Maritime works concept details (intake shown only)

7.4 Further Considerations

The advice provided herein is of a conceptual level only, and the following outstanding information/issues would need to be confirmed before further design development can be undertaken:

- Confirmation of the details of the proposed heat rejection system including required pipework dimensions, flows and pumping requirements.
- Detailed topographical and bathymetric (seabed) survey of the subject site.
- Geotechnical information on land and seabed.
- As-built documentation and condition of existing seawall, wharf and other structures affected by the new works.
- Maintenance responsibilities and desired level of maintenance expected (this will influence material choices and access considerations).

8 Structural Design Criteria

8.1 Design Standards and Sources of Reference

8.1.1 General

The design and documentation of the building and associated works shall comply with all relevant Australian Standards and the Building Code of Australia (BCA). Client and design brief requirements shall be considered in addition where appropriate.

Standard Specifications or Codes of the British Standards Institute (BS) or the American Society for Testing and Materials (ASTM) are referenced only when a relevant Australian Standard publication does not exist. Current editions shall apply, as above specified.

8.1.2 BCA Structural Provisions

Importance Level of Building:

- Importance Level 3 - Structures designed to contain a large number of people

Table 3: Annual probabilities of exceedence for building

Design Events for Safety	Annual Probability of Exceedence
Wind	1:1000
Earthquake	1:500

Importance Level of Land Bridge:

- Importance Level 4 – Buildings essential for post-disaster recovery (protection of roadway below)

Table 4: Annual probabilities of exceedence for landbridge

Design Events for Safety	Annual Probability of Exceedence
Earthquake	1:800

8.1.3 Codes and Standards

The following codes and standards will form the basis for the structural design:

Table 5: Schedule of applicable Australian Standards

Code	Title
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/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 5100	Bridge design
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

8.1.4 Design References

Table 6: Schedule of applicable design references

Number	Title	Author
CCIP-016	Guide on the Vibrations of Floors	The Cement & Concrete Association
CIRIA C660	Early-age thermal crack control in concrete	CIRIA

8.2 Loads

8.2.1 General

All design loads shall be selected and applied in accordance with the relevant Australian Standard, specifically AS/NZS1170.1 to 1170.4, and the BCA.

8.2.2 Dead Loads

Dead loads should be calculated on the basis of the following densities:

- Reinforced concrete: 25kN/m³
- Steel: 78.5kN/m³
- Masonry: As calculated
- Timber: As calculated

8.2.3 Superimposed Dead Loads and Live Loads

The structure will be designed for the following imposed loads. Superimposed dead loads include floor finishes, ceiling, services, and partitions. Masonry walls are additional and as calculated from the architectural drawings.

The requirements for sculptures and three-dimensional physical artworks, and handling of sculptures that induce heavy concentrated loads during movement and installation or in final display condition, are subject to continuous development with the gallery and stakeholders and may be updated from time to time.

Table 7: Imposed loads schedule

Area	Superimposed Dead Load	Live Load	
		UDL	Concentrated load
Internal gallery spaces (not over the land bridge)	3.75kPa floor finishes (150 thick polished concrete floor) 0.25kPa hanging services 0.5kPa ceiling	7.5kPa	Exhibition loads within the 7.5kPa overall allowance 4.5kN over 350mm^2 Forklift/platform axle loads: TBC
External gallery terraces	2.5kPa floor finishes 0.25kPa hanging services 0.5kPa ceiling	7.5kPa generally 5.0kPa over land bridge	4.5kN over 350mm^2 Forklift/platform axle loads: TBC
BOH delivery and art/sculpture movement paths	2.5kPa floor finishes 0.25kPa hanging services 0.5kPa ceiling	7.5kPa generally 5.0kPa over land bridge	4.5kN over 350mm^2 Forklift/platform axle loads: TBC
Cultural plaza	2.5kPa floor finishes	5.0kPa	4.5kN over 350mm^2 Forklift/platform axle loads: TBC
Trafficable roofs	2.5kPa	4.0kPa	
Foyer (including stairs and landings)	2.0kPa finishes 0.5kPa ceiling & services	5.0kPa	4.5kN
Terraces (including stairs and trafficable roofs)	2.5kPa finishes 0.5kPa ceiling & services	5.0kPa	
External landscaped areas over suspended slabs	0.5kPa membrane 0.5kPa ceiling and services	500mm saturated soil = 10kPa	Trees/tree pits as agreed locally.
Offices	1.0kPa moveable partitions 0.45kPa raised floor 0.5kPa ceiling and services	3.0kPa	2.7kN
Compactus Zones	0.45kPa raised floor 0.5kPa ceiling and services	7.5kPa storage areas over 5% office floor area in accordance	

		with PCA Grade A space.	
Kitchens	2.5kPa finishes 0.5kPa ceiling & services	5.0kPa	4.5kN
Storage areas	0.5kPa ceiling & services	5.0kPa	4.5kN
Workshops	0.5kPa ceiling & services	5.0kPa	4.5kN
Egress stairs and landings within shafts and tunnels		4.0kPa	
Car parking	0.25kPa	2.5kPa	13kN
Loading Dock	0.5kPa	15kPa minimum to be confirmed based on vehicle type.	Concentrated wheel loads assessed on vehicle or AS5100.
Plant Areas	3.0kPa partitions and plinths 0.5kPa ceiling and services	As calculated for relevant use. 5.0kPa minimum	4.5kN
Tank rooms	3.0kPa plinths	Tank volume as calculated. Minimum 20kPa	
Substation and main switch room	As calculated trenches and plinths in accordance with approved substation design	7.5kPa	
Non-trafficable green roofs	0.5kPa ceiling & services 3.0kPa for combination of hard and soft landscaping using proprietary growth medium and not traditional soil.	1.0kPa	2.7kN
Lightweight roofs	As calculated	Generally 0.25kPa Roofs for hanging displays: 0.5kPa	1.4kN

Note: concentrated loads are applied over 350mm^2 unless noted otherwise.

8.2.4 Wind Loads

For concept stability design and design of components, wind loading applied to the structural elements will be assessed in accordance with AS/NZS 1170.2, SAA Wind Loading Code.

The design parameters in the Table 8 below have been assessed in accordance with AS/NZS 1170.2.

Table 8: Wind loading parameters

Parameter	Value
Region	A2
Basic wind speeds: Ultimate, V_{1000} Serviceability, V_{20}	 46 m/s 37 m/s
Terrain category, TC	As calculated by direction
Structure height, Z	Varies by direction
Variation of wind speed with height, $M_{(z,cat)}$	As calculated
Structural importance multiplier, M_i	1.0
Topographic multiplier, M_t	As calculated
Shielding multiplier, M_s	As calculated
Minimum internal pressure coefficient, $C_{p,i}$	+0.2/-0.3 generally
Area reduction factor, K_a #	As calculated, 0.8 minimum
Combined action reduction factor, K_c #	As calculated, 0.8 minimum
Local pressure coefficients	As calculated

Note: # denotes minimum K_a x K_c = 0.8.

8.2.5 Seismic Loading

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 for building structures:

Specific AS 1170.4 seismic data is summarised as in Table 9 below.

Table 9: Seismic loading parameters (AS 1170.4)

Parameter	Value
Importance level	3
Hazard factor, Z	0.08
Site sub-soil class	Be-Rock
Importance level, I	3
Annual probability of exceedance	1/500

Probability factor, k_p	1.0
Design earthquake category	II
Structural system	Table 6.5(A) AS 1170.4

- AS 5100.2-2004 Bridge Design: design loads for the land bridge.

Table 10: Seismic loading parameters (AS 5100.2)

Parameter	**Value**
Bridge classification	Type III
Acceleration coefficient, a	0.08
Site factor, S	1.0
Importance factor, I	1.25
Design earthquake category	BEDC-2
Response factor	R_f = 3.0

8.2.6 Earth Pressure Loading

Earth retaining structures shall be designed in accordance with the recommendations in the geotechnical report.

Retention at boundaries will be designed for surcharge in accordance with AS 5100.2-2004, with a minimum magnitude of 10kPa.

8.2.7 Water Pressure Loading

Groundwater is expected to be near the existing basement level, subject to further hydrological study.

The existing fuel bunker walls are proposed to be drained, with a subsoil drainage system located within the existing tank walls to catch any seepage. Permanent electric pumps will be provided within a sump arrangement connected to the stormwater discharge system.

8.2.8 Accidental Horizontal Loads

All loading in accordance with AS/NZS1170.1. Loads on balustrades are applied in any direction.

Table 11: Balustrade loadings

Location	UDL	Concentrated load
Terrace balustrades	1.5kN/m	0.6kN
Stairs, landings, walkways in gallery and foyer areas	1.5kN/m	0.6kN
Offices	0.75kN/m	0.6kN
Egress stairs, landings, walkways	0.75kN/m	0.6kN
Maintenance walkways & stairs	0.35kN/m	0.6kN
Car park parapets, walls, barriers		In accordance with AS/NZS 1170.1. 30kN generally 40kN loading dock 240kN at base of ramps > 20m long
Delivery area columns		40kN

8.2.9 Imposed Movements

The effect of imposed movements on the structure will be considered in the calculations. These include the following types of movement as per Table 12 below.

Table 12: Imposed movements

Parameter	Value
Settlement	1% of footing width at allowable bearing pressure
Temperature (exterior elements)	Maximum range +5°C to +65°C Mean temp 20°C.
Shrinkage	As calculated for vertical structure or floor slabs
Creep	As calculated for vertical structure and post-tensioned floor slabs
Elastic shortening	As calculated for vertical structure and post-tensioned floor slabs

8.3 Serviceability

8.3.1 Design Life

The structure is to be designed for a 100 year design life. Loading will be selected in accordance with the relevant return periods to the AS/NZS1170 Parts 0, 1, 2, and 4. Material selection and concrete covers will be selected in accordance with the recommendations in AS5100.

Reference should be made to the relevant material standard regarding construction and maintenance assumptions that form the basis of the design code.

8.3.2 Deflection Limits

The deflection limits in Table 13 are proposed for the building structure:

Table 13: Deflection limits

Element	Deflection (total load UNO)	
Beams and Slabs:	Spans	Cantilevers
Generally Live load only Supporting articulated masonry Supporting unjointed masonry Supporting curtain wall and glazed assemblies Transfer structures (cumulative at location of element transferred)	L/250 L/360 L/500 (incremental) L/1000 (incremental) L/800 or 15mm max L/1000 or 12mm max	L/125 L/180 L/250 (incremental) L/500 (incremental) L/400 or ±15mm max L/500 or 12mm max
Roof beams under wind load (other than glazed roofs)	L/200	L/100
Wind columns	L/240	L/120
Overall wind sway SLS		H/500
Storey drift under wind SLS: Structures supporting glazed walls in-plane Structures supporting glazed walls out-of-plane Free roofs		 h/500 h/240 h/120
Storey drift under seismic ULS		1.5%h
Differential settlement	L/1000	L/500

8.3.3 Floor Vibrations

Vibration of superstructure floors is controlled by checking the natural frequency and response. Verification calculations will be conducted to the Cement and Concrete industry publication CCIP-016 `A Design Guide for Footfall Induced Vibration of Structures’, and The Steel Concrete Institute publication SCI P354 ‘Design of Floors for Vibration: A new approach’.

Target response factors are as per Table 14 below

Table 14: Floor vibration criteria

Space	Target Response Factor	Walking frequency	Damping
Gallery floors	R = 2 to 4	1.8 Hz	2.5% representing open-plan sparsely furnished floor.
Foyer	R = 6 R = 8	2.0 Hz 2.5 Hz	2.5% representing open-plan sparsely furnished floor.
Offices	R = 6 generally R = 8 in low sensitivity areas	2.0 steps/sec 2.5 steps/sec	3.5% representing open plan normal office fit-out.

8.3.4 Concrete Durability and Crack Control

The requirements of AS 3600 will be applied to all reinforced concrete. Where appropriate, AS 5100 shall be used for criteria that applies to the extension of design life to 100 years.

The degree of crack control to be provided in concrete elements will be in accordance with AS 3600. For exposed elements, the more onerous requirements of AS 5100 may be selected if relevant to achieve the 100 year design life.

8.3.5 Steelwork Corrosion Protection

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

Internal steelwork which is in marginally damp areas where occasional condensation may occur, such as around the building perimeter and in the vehicle and plant room areas, will require a higher level of protection than inside the air-conditioned office space which is permanently dry. For both these internal environments it is assumed that there is no access for maintenance, and either a hot dip galvanised (HDG) or multi-build paint system will be specified.

A high standard corrosion protection system is required for all exposed steelwork, and will require maintenance during the life of the building. A design life of 25 years and warranty period of 10-15 years from the coating supplier and applicator may be expected.

Care must be taken during handling, transport, and erection to minimise damage to the coating system that will require making good on site and compromise long-term performance. This may include wrapping of elements.

The paint system for corrosion protection must be compatible with any required fire protection.

8.3.6 Fire Resistance Levels for Structural Elements

Fire resistance levels for structural elements shall be determined in accordance with the Building Code of Australia, Ausgrid, and any subsequent approved relaxations based on approved fire engineered approaches.

Concrete covers are to be in accordance with AS 3600.

Structural steel elements shall be provided with passive protection or designed based on limiting temperatures. Passive protection may include:

- Synthetic vermiculite spray;
- Fire board; or
- Intumescent paint; acrylic or epoxy-based dependent on the exposure to weather.

A fire engineered approach will be adopted to reduce the extent of passive protection from the deemed-to-satisfy requirements.

8.3.7 Protection of Basements from Groundwater

The design of the basement walls and floors are to be such as to provide acceptable environmental conditions for the client.

There are no relevant Australian standards. BS 8102:2009 will be used as a design guide, which describes four grades of environment to be achieved, and three appropriate types of construction.

8.4 Materials

The following structural materials are used in the works. Typical design properties of these materials are listed. These values are to be adjusted and enhanced as appropriate during the detailed design of the structure.

8.4.1 Concrete

Table 15: Concrete properties

Parameter	Value
Grades, f'c	32 to 80MPa
Short-term E	As calculated
Coefficient of thermal expansion	10x10^{-6} per °C
Basic shrinkage strain	As calculated and specified
Basic creep factor	As calculated
Poisson's ratio	0.2
Density Mass concrete Reinforced concrete	 24kN/m^3 25kN/m^3

8.4.2 Reinforcement & Post-tensioning

Reinforcement shall comply with AS/NZS 4671 and AS/NZS 4672 respectively.

Table 16: Reinforcement and post-tensioning properties

Parameter	Value/Designation
Plain 'R" bars	R250N
Deformed 'N' bars	D500N
Welded wire fabric	D500L & D500N
Young's modulus	205 x 10^{3}MPa
Post-tensioning strand (superstrand)	∅12.7mm fpb = 1870MPa ∅15.2mm fpb = 1790MPa (min)

8.4.3 Structural steel

Table 17: Structural steel properties

Parameter	Value
Steelwork density	7850kg/m^3
Yield stress	fsy = 250 to 400MPa
Young's modulus	205 x 10^{3}MPa
Poisson's ratio	0.3
Coefficient of thermal expansion	11 x 10 $^{-6}$ per °C

8.4.4 Foundations & retention

As provided by Coffey, Ref. GEOTLCOV25037AA-AF, dated 13th June 2014.

8.4.4.1 Founding strata

Table 18: Foundation parameters

Rock Quality	**SLS (kPa)**		**ULS (kPa)**		**Young's modulus (E) (MPa)**	
	Bearing	Adhesion#	Bearing	Adhesion#	Static	Dynamic
Class IV sandstone	1,000		3,000	150	100	300
Class III sandstone	4,000		20,000	800	1,000	3,000
Class II sandstone	8,000		80,000	2,000	2,000	6,000
Class I sandstone	12,000		120,000	3,000	3,000	9,000

Notes:

- Sandstone quality in accordance with Pells et al.
- Shaft adhesion is in compression. A 0.7 factor is to be applied for tension.
- # denotes surface roughness as specified by the geotechnical engineer.
- Appropriate geotechnical factors shall be applied at ULS in accordance with AS2159.
- SLS bearing values are based on settlement of 1% of footing width.
- Spoon testing or proof coring requirements vary with rock quality. Nominally 33% of footings for Class III/IV, 50% for Class II, and 100% for Class I.

8.4.4.2 Retention

The following information is provided for drained materials. Consideration of hydrostatic pressure shall be based on both the ground water levels and drainage provision above the water table. Consideration shall be given to burst water mains.

Table 19: Retention parameters

Unit	**Bulk unit weight (kN/m^3)**	**Effective Cohesion (kPa)**	**Effective friction angle**	**Active earth pressure coefficient (Ka)**	**At rest earth pressure coefficient (Ko)**	**Passive earth pressure coefficient (Kp)**
Fill	18	0	30	0.33	0.5	3
Class IV/V sandstone	23	30	35	0.27	0.43	3.7

Note: Class III and better sandstone may be cut vertically. Adequate measures shall be provided locally for adverse jointing.

8.4.4.3 Sandstone stress relief

The new design and assessment of existing infrastructure will consider any movements arising from horizontal stress relief in the sandstone rock mass during excavation.

8.5 Greenstar & Sustainable Design

Refer to the requirements of the sustainability consultant for:

- Concrete mix requirements – Specifically Portland cement and natural aggregate replacement with industrial waste products;
- Steel reinforcement – Recycled content and prefabrication; &
- Structural steel – Recycled content and design for disassembly.

Concrete mix design shall comply with both the structural performance criteria as specified by the structural design, and the constituent requirements for compliance with the selected Greenstar rating.

9 Conclusion

This report has provided an overview of the structural systems, constraints and criteria for the proposed new building at the Art Gallery of NSW.

It has addressed the SEARs Key Issue number 10 with particular reference to the structural solution and proposed loading on, and works to, the existing land bridge over the Eastern Distributor.