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From wasteland to world-class development: Barangaroo South

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Barangaroo South is a A$6 billion (£4 billion) transformation of a former container port in Sydney, Australia, into a business, residential and leisure precinct. Due for completion in 2020, it features three commercial towers up to 49 storeys providing more than 490 000 m2 of gross floor area. It is set to become one of Australia's first large carbon dioxide-neutral developments. This paper provides an overview of the civil engineering challenges facing the project and the innovative solutions adopted, including remediation of past contamination, basement retention and foundation construction in reclaimed land, groundwater control, design of foundations and transfer structures over the proposed rail tunnels, and superstructure design.

1. Introduction

Barangaroo South is a A$6 billion (£4 billion) transformation of a former container port on the waterfront of the central business district of Sydney, Australia, into a major business, residential and leisure precinct (Figure 1).

Located on the southern third of the 22 ha Barangaroo site, the 4·4 ha site features three commercial towers designed by internationally

Figure 1. Artist's impression of how the Bangaroo South development (right) will complement the Sydney cityscape in 2020 (Lend Lease)

acclaimed architects Lord Richard Rogers and Ivan Harbour. The towers, ranging in height from 39 to 49 storeys and with large, highly flexible floorplates, are targeting 6 star green star as-built ratings. When completed in 2020, they will have more than 490 000 m^2 of gross floor area and become Australia's first large-scale carbon dioxide-neutral community.

The vision of developer Lend Lease is to enhance Sydney's position as an internationally appealing, globally competitive, environmentally sustainable city. Barangaroo South is intended to become the country's leading financial and professional services hub, with highly sustainable commercial towers, a landmark hotel, stylish apartments, world-class restaurants, enticing parks and a naturalistic headland.

Stage 1a will have the International Towers Sydney at its core – a new financial and professional services centre. Construction involves installing 0·7 km of diaphragm walling for excavation retention and as the basement wall, driving nearly 1000 piles ranging from 1·0 m to 2·4 m in diameter, building a transfer structure over future Sydney Metro tunnels, and erecting the three main towers.

For a project of the magnitude of Barangaroo South many complex engineering design challenges and opportunities arise for optimisation and resolution.

2. Site remediation challenges and solutions

The New South Wales environmental protection authority has defined an area of the project as a declared remediation site (blocks 1–5) as shown in Figure 2. Blocks 1–3 (excluding the declared remediation area) are the location of the proposed towers that will be supported by a combined basement structure.

The area has a long post-European settlement history of industrial use, predominantly as wharfage but also a gasworks. The first wharves were built at Millers Point in the mid-1820s and there are reports that parts of Barangaroo were utilised as wharfage as early as 1788.

The main contamination identified is associated with the historical presence of the former gasworks located immediately north of the site and the presence of fill materials used for land reclamation. Gas manufacture from coal began around 1839 and it operated through to 1918. The gasworks occupied a portion of Barangaroo and also extended into and past a portion of what is now Hickson Road, which was built following the demolition of the gasworks with the further development of the waterfront occurring through the 1920s.

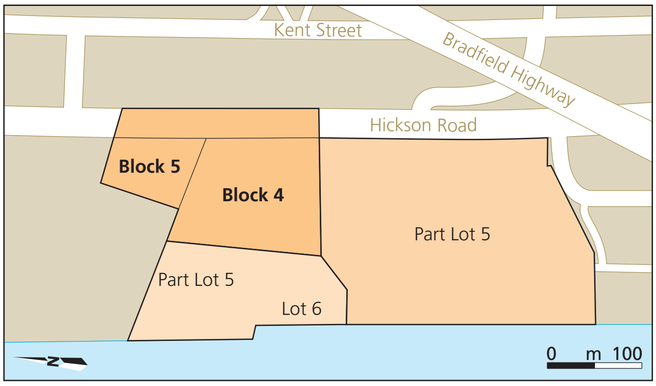

Figure 2. The development site, showing potentially contaminated areas identified as blocks 1–5 and environment protection authority declaration area in yellow shading

Contamination identified at the site was generally concentrated within the northern portion of block 3 in proximity to the former gasworks infrastructure, but was variably detected across the site.

Operation of the gasworks has resulted in a legacy of subsurface soil and groundwater contamination that remains beneath parts of Barangaroo and Hickson Road. This includes gasworks tar present in historic structures (e.g. tanks and tar well). Other industrial activities included reclamation filling and demolition of former buildings potentially containing hazardous materials. There are also naturally occurring acid sulfate soils beneath the filling. A number of associated contaminants of potential concern have been identified at the site and these include

- arsenic, copper, chromium, cadmium, mercury, lead, nickel and zinc
- polycyclic aromatic hydrocarbons
- total petroleum hydrocarbons
- benzene, toluene, ethylbenzene and xylenes
- organochlorine pesticides
- organophosphorus pesticides
- polychlorinated biphenyls
- semi-volatile organic compounds
- volatile organic compounds
- acid sulfate soils.

In May 2009, the environment protection authority determined that a portion of land at Millers Point was contaminated in such a way as to present a significant risk of harm to human health and the environment. As a consequence, the agency declared the area to require remediation (declaration number 21122; area number 3221) under the Contaminated Land Management Act, 1997.

An independent, agency-accredited site auditor was appointed to undertake a review of proposed remediation works, and prepare statutory audit statements before and following completion of remediation.

The preferred remediation strategy for blocks 1–3 involves excavation of contaminated material, beneficial reuse where possible at Headland Park (northern development of Barangaroo) and/or public domain areas in the development area, as part of a specific resource recovery exemption approval (beneficial reuse offsite) or offsite disposal to a licensed landfill.

The remediation strategy for blocks 4 and 5 included two potential remediation options. Option 1 (preferred) was a combination of ex-situ (excavate and treat) and in-situ remediation methodology, with ex-situ methodology within the block 4 remediation area and in-situ methodology using surfactant-enhanced in-situ chemical oxidation within the block 5 declared remediation area and Hickson Road declared remediation area. This was an innovative approach that treated contamination where it lay below ground, without excavation. The approach was aimed at providing a more sustainable and low-impact remediation solution for specific areas of the site, particularly Hickson Road. Using an in-situ approach minimised odour management, traffic and waste generation that would otherwise have resulted from excavation. The remediation process is illustrated in Figure 3. The alternative option involved undertaking all remediation by the ex-situ method (excavate and treat) across blocks 4 and 5 and Hickson Road remediation areas.

3. Site retention and overcoming buried objects

As the basement excavation was below sea level, it was essential that the site retention wall for the basement also acted as a cut-off against groundwater ingress. A diaphragm wall, with a minimum socket of 0·3 m into medium strength or better rock, was adopted for watertightness. The diaphragm wall ranged in thickness from 0·8 m to 1·2 m depending on depth, and was constructed under bentonite slurry support.

The construction of the diaphragm wall was challenging due to the presence of open rock rubble, buried old structures including concrete seawalls, footings and old jetty timber piles. Loss of bentonite slurry through rubble fill was managed carefully by backfilling with clay soils and re-excavation, and a hydromill (rock cutting head) was used to cut through hard rock. As the diaphragm wall panels are up to 7·7 m wide and the rock levels are variable with steps and benches, a programme of advanced probing was carried out together with full-time monitoring during the works to ensure that the full width of each panel had adequate penetration into competent rock to achieve the water cut-off function. Water stops were used between panels to achieve watertightness.

Along the southern side of the site, two future rail tunnels with limited rock cover above the tunnel crown traverse across the site (Figure 4). A number of ‘trouser-legs’ (i.e. longer panels either side of the future tunnels) had to be installed below tunnel invert level to transfer the building loads away from the top of the tunnels. These ‘trouser-legs’ were successfully constructed using a hydromill to cut the high-strength sandstone.

All diaphragm wall panels were tied back using temporary ground anchors. Due to the proximity of adjacent existing building basement at one location, the designer and contractor devised an innovative ‘T’ wall section (i.e. diaphragm wall section having a short counterfort section at a right angle to the main wall) with a subvertical anchor at the tail of the T to provide overturning resistance without intruding tie back anchors into the adjacent property. Removable ground anchors were used in the vicinity of the future rail tunnels.

Following construction of the diaphragm wall, watertightness was checked by a piezometer monitoring programme and groundwater study during dewatering and excavation within the site. The contingency measure was to grout the rock below the diaphragm wall toes should excessive leakage be encountered.

4. Foundation design challenges and solutions

The core of each of the three towers is supported on groups of piles ranging from 1 m to 2·4 m in diameter. The tower columns are set on a grid of 10·5 m and carry design working loads and ultimate loads that are in excess of 80 MN and 100 MN, respectively.

The challenges associated with the foundation design for this project included the following.

- Rock level ranges from +1 m to –28 m relative to the Australian height datum, and is overlain by alluvium and filling (Figure 5). Rock levels can vary significantly over short distances and there is the potential for buried cliff ledges.
- With about 1000 piles to be constructed in difficult ground (saturated fill and alluvium that are prone to collapse, and the high-strength rock is hard to drill); piling represents a significant cost to the project.
- Piling in close proximity to future rail tunnels for the southernmost tower.

Due to the heavy building loads, all foundation piles are socketed into rock. The rock at the site comprises Hawkesbury sandstone, with assessed uniaxial compressive strength of about 10 MPa to 30 MPa

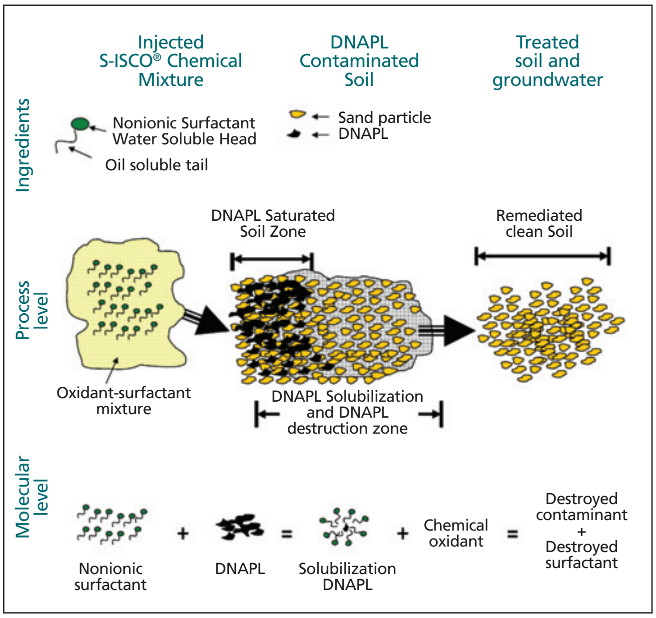

Figure 3. Surfactant-enhanced in-situ chemical oxidation contamination remediation process. DNAPL, <AQ4>

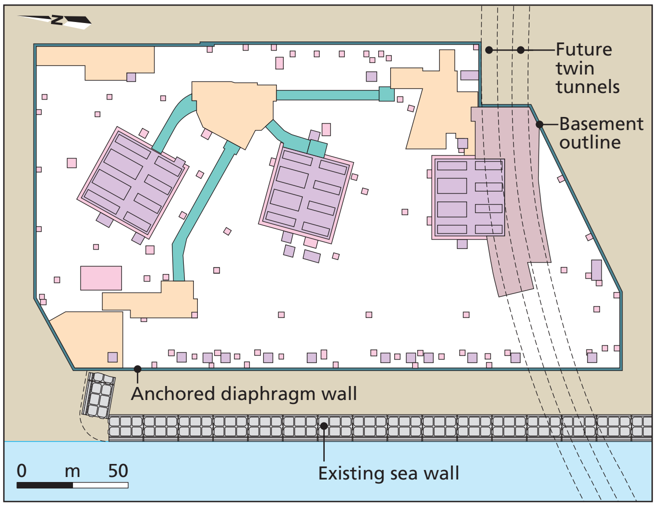

Figure 4. Barangaroo South basement B2 plan showing perimeter diaphragm wall and transfer structure over future Sydney Metro tunnels

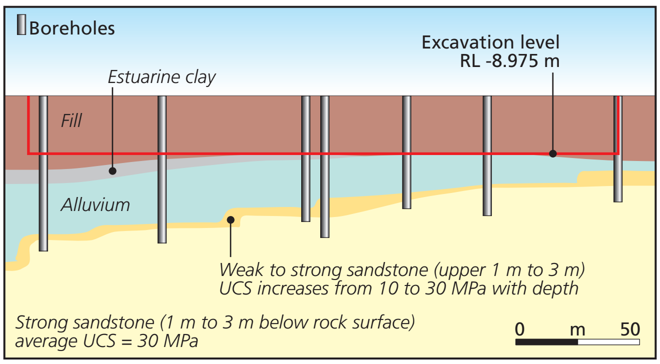

Figure 5. Typical geological cross-section, from north (right) to south (left)

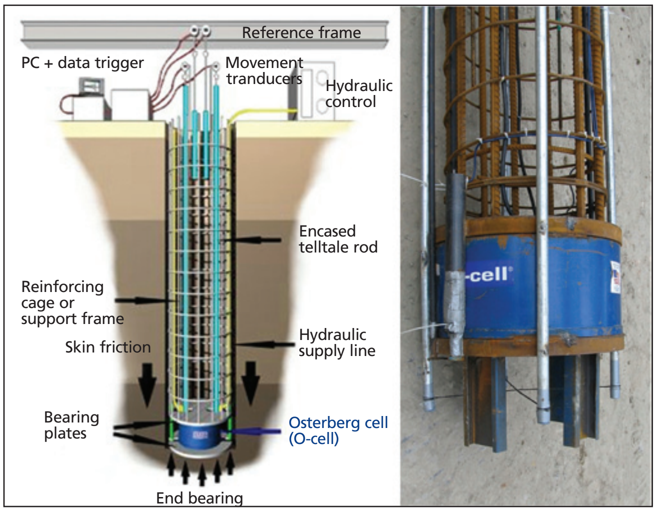

Figure 6. O-cell testing arrangement for prototype test piles

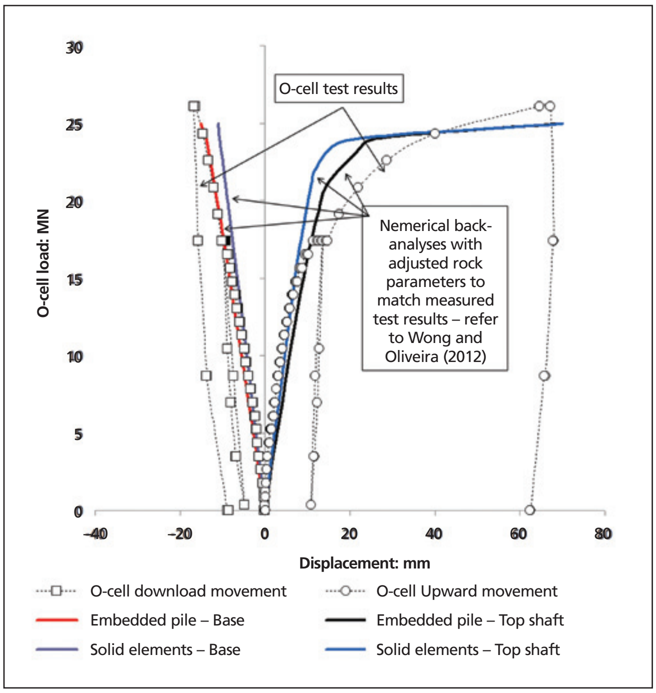

Figure 7. Back-analysis results compared with O-cell test results of one of the test piles

between 1 m and 3 m below rock surface, thereafter having an average uniaxial compressive strength of 30 MPa.

The traditional approach of designing piles in Sydney rock has been to use conventional 'serviceability' bearing pressures that correspond to a pile head settlement within 1% of the pile diameter in accordance with Pells *et al*. (1998). Because of the large number of piles to be installed, even a small reduction in pile length for each pile will result in significant cost savings. A more aggressive design approach was therefore adopted using the following strategy.

- Because of high rock strength, the strength limit state was not a governing factor for design, enabling the designer to focus on a performance-based design.
- A design settlement target of 0·3% of pile diameter at the pile base (note: pile head settlement will be higher) was adopted using presumptive rock stiffness values initially, then checking structural performance for potentially greater foundation settlement by pile load testing and back-analysis of results. The target of 0·3% is set to limit the differential settlements between the tower columns and adjacent low-rise building columns.
- Two bidirectional O-cell pile load tests were carried out on prototype 750 mm dia. piles (Figure 6), which enabled testing to approximatly 70% of the ultimate base capacity and 100% of the shaft capacity. This enabled the strength limit state to be confirmed and pile stiffness to be back-analysed for checking with building performance requirements.
- A programme of proof coring (about 25% of piles within the tower core, and each of the 2·4 m dia. tower piles) to confirm the rock profile and assess the potential for cliff ledges.
- Full-time geotechnical presence during piling to confirm founding levels and cleanliness of the pile base, and the use of appropriate drilling tools to achieve a minimum rock socket roughness of 4 mm at maximum 200 mm spacing. This socket roughness is assessed to be required to achieve the adopted shaft friction values shown in Table 1, based on the recommendations of Walker and Pells (1998).

The pile design parameters adopted are summarised in Table 1.Class A predictions of the O-cell test piles were made before the tests, and back-analyses were carried out using finite-element analysis techniques, as reported in Wong and Oliveira (2012). The results of the tests and back-analyses are presented in Figure 7. The O-cell test results indicate some non-linearity of rock modulus and pile stiffness, with a back-analysed secant modulus of the class II sandstone reducing at high stresses as shown in note (b) of Table 1. Pile performance using the back-analysed stiffness values was considered to be satisfactory.

Soil and rock class[a]	Elastic modulus: MPa	Ultimate shaft friction: MPa	Ultimate end-bearing pressure: MPa
Fill and alluvium	20	Ignored	Not used
Class V sandstone	100	0.15	Not used
Class IV sandstone	500	0.5	Not used
Class III sandstone	1000	0.8	30
Class II or better sandstone	2000[b]	2.0	80

[a] Rock class based on Pells *et al.* (1998)
[b] Initial tangent modulus at low stress level; secant modulus back-analysed from O-cell testing were approximately 1500 MPa and 1200 MPa at load levels of 40 MPa and 59 MPa, respectively

Table 1. Pile design parameters adopted

5. Structural design challenges and solutions

The 4·4 ha site incorporates basement substructures set entirely below the adjacent harbour tidal water levels. The basements require open-plan structural layouts to enable efficient integration of the central plant systems contained primarily below grade. The basements also contain loading dock facilities and car parking. The general layout of the proposed buildings is shown in Figure 8.

The site has been mostly reclaimed during the 1960s and 1970s, with the original shoreline located close to the Hickson Road boundary. The caisson sea wall along the western foreshore of the site only extends down to the harbour floor and is supported on a gravel bed placed above the marine deposits. Consequently, it does not form a cut-off wall and land mass behind the wall is subjected to tidal inflows. In combination with the marine water environment and uncontrolled reclamation fill, the site contains adverse conditions for any buried structural elements. Exposure conditions for structures can be classed as severe, requiring the selection of high quality materials to maintain durability and reduce ongoing maintenance requirements.

A specified design life of 100 years for the basement structures is particularly onerous for the basement structures that have the lowest basement levels set generally 7 m below the projected tidal high water line. As a consequence, the basement structures have been designed to be fully tanked to provide a dry internal environment. Furthermore, the continuous concrete diaphragm wall around the perimeter of the development is not protected from chlorides but it is designed (both concrete mix and reinforcement cover) to provide 100 years design life.

The soffit of the lowest basement slab was constructed with a fully bonded membrane at the time of casting. The membrane enhances the long-term watertightness of the basement slab in contact with the groundwater and also forms a barrier that significantly reduces migration of chloride ions from the wetted surfaces to the inside of the building. The minimum slab thickness selected for the lowest basement slab is 800 mm, designed to cater for all imposed vertical building loads and hydrostatic uplift forces of between 70 kPa and 100 kPa. Thicknesses were increased for localised deeper set-down areas.

The slabs must also resist hydrostatic uplift pressures once dewatering has been terminated and equilibrium water level conditions are re-established. In the podium area where there is insufficient dead weight to resist uplift, permanent tension piles are used. To avoid punching shear that may develop from axial shortening and settlement of the supporting piles, a decomposable void-forming system was proposed during the concrete placement of the suspended basement slabs that were cast on the ground (Figure 9). A temporary drainage

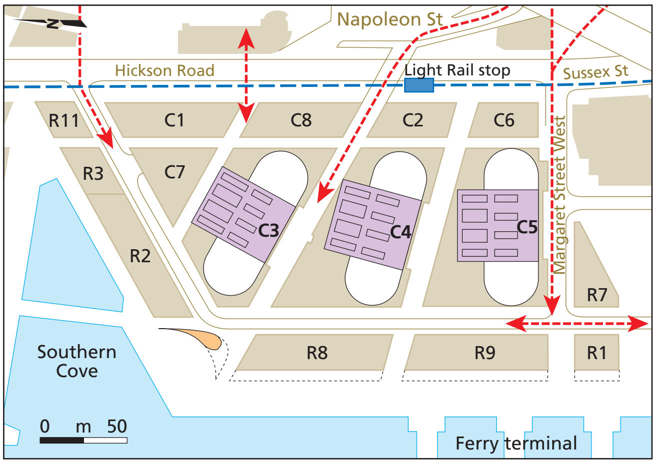

Figure 8. Barangaroo South site plan showing general layout of the proposed buildings

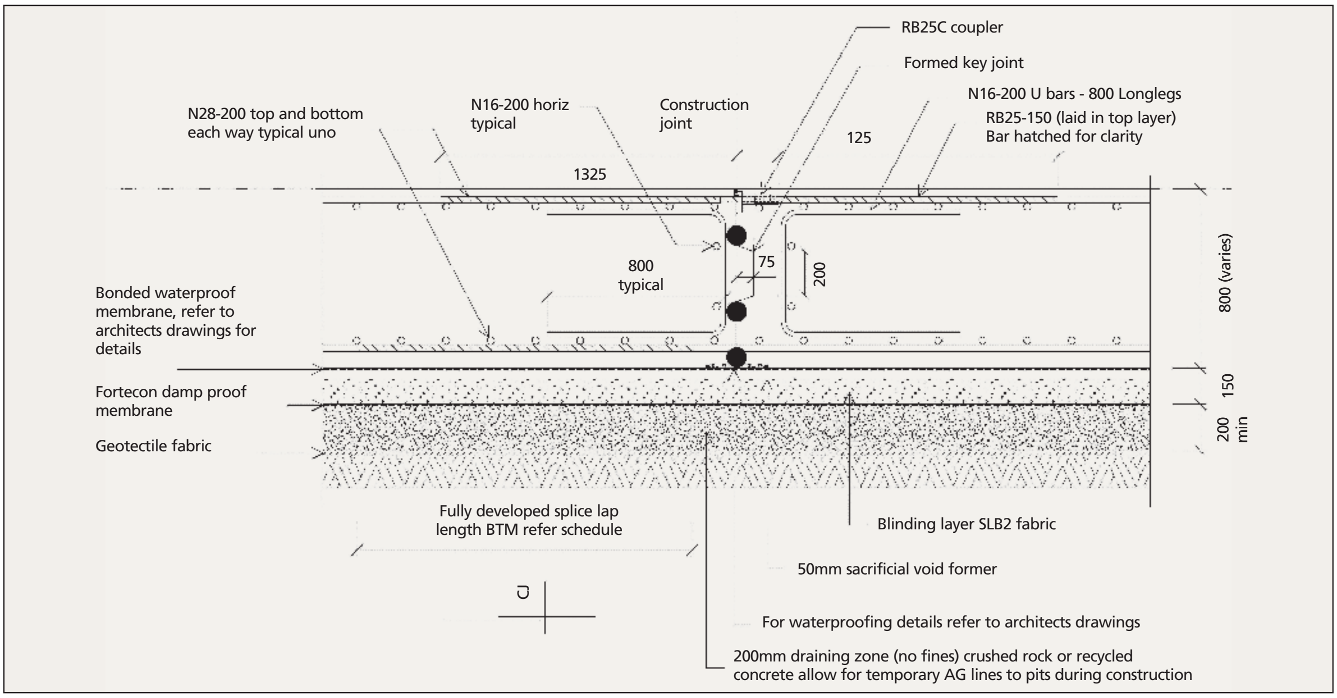

Figure 9. Typical basement slab construction joint showing proposed splicing and waterproofing details

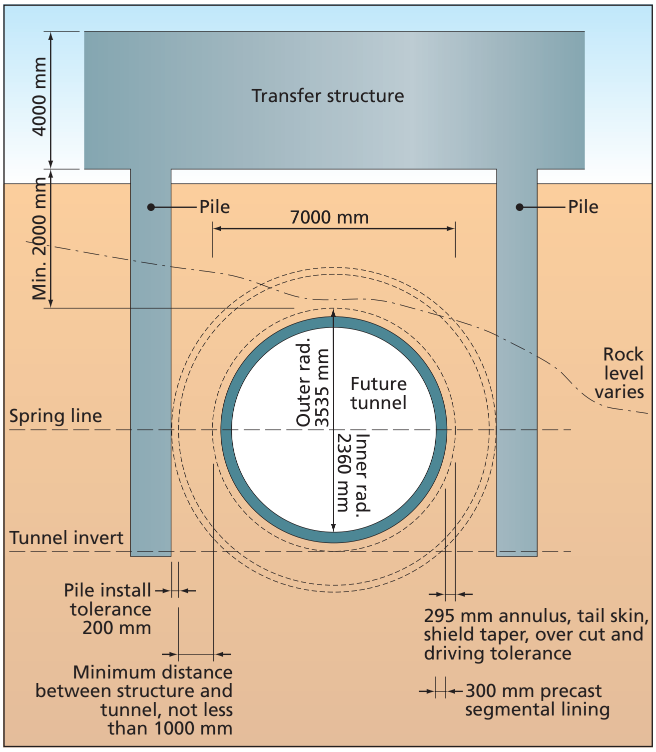

Figure 10. Typical cross-section of transfer structure over the future Sydney Metro tunnel (dimensions in mm)

Figure 11. Each of the commercial towers has a three-lift rise format, and floor-to-floor levels are set at 3·8 m giving the 50-storey northern tower (left) an overall height of 210 m above ground

layer was provided at the bulk excavation level to provide a dry working platform, and seepage through the base of the diaphragm wall was pumped out during construction only, with the permanent basement designed as a fully tanked structure.

Durability of the diaphragm wall and basement B2/B3 slabs was also enhanced by the selection of concrete mix designs with a high degree of cement replacement using graded slag and fly-ash products. The specified mixes provided a more consistent quality with respect to strength, shrinkage and permeability.

The lowest basement slab was cast in areas of approximately 500 m^2. The final slab forms a continuous horizontal diaphragm across the site. The slab is designed for a high degree of crack control to limit both shrinkage and flexural cracking. A minimum of 0·65% of reinforcement was maintained throughout all sections in both orthogonal directions of reinforcement layouts of the lowest basement slabs. Coupler connection was used at the junction between the diaphragm wall and the basement slab to provide full structural connection.

The perimeter diaphragm retaining wall system is restrained by the lowest basement (B2/B3) slabs and the ground level slab. To restrain the wall from controlled lateral movements after the removal of the temporary anchors, and to balance the differential lateral forces around the perimeter of the site imposed on the walls, the connecting slabs are designed to act as continuous plate diaphragms. The restrained and tied perimeter walls and the tower cores provide long-term overall structural stability to the basement structures.

Adjacent to the southern boundary of the site, future provision has been planned to allow the construction of the Sydney Metro tunnels located under the C5 commercial tower. Parallel rows of sleeved bored piles were installed to flank the swept path of the tunnel easement. The supporting piles are set at 1·2 m from the tunnel walls and are sleeved to avoid load shedding from the piles to the tunnel lining (Figure 10). The shaft sockets and end bearing of the piles is to be set below the railway tunnel horizontal centre line and invert level, respectively. The piles support a 4-m thick transfer raft slab from which the C5 commercial tower columns and cores are supported. The transfer raft is required to support tower column working loads in the order of 80 MN. The building structure is designed such that construction and operation of the tunnel will not affect the performance of the buildings above.

Above the basement levels, three commercial high-rise towers (C3, C4 and C5) are located in plan between a highly urbanised ground plane grid of trafficable roads and pedestrian thoroughfares that form a natural link to the city planning grid east of the site. The commercial towers form the nucleus of the development with smaller mixed-use buildings consisting of retail, residential, hotel and service centre functions located mainly around the perimeter of the site in a low-rise configuration.

The tower lobbies of each commercial building are bounded by three-storey podium buildings that follow the perimeter of the site allotment boundaries. This enables the orientation of the buildings to respond effectively to the prevailing wind conditions and yield environmentally acceptable occupant comfort conditions at grade adjacent to the podiums and the tower lobby entrances.

The variations of geometric form of the buildings above the ground plane have required a significant engineering effort to resolve a regular and efficient structural grid for the basements that can be extended vertically to support the buildings over without any major transfer structures. There are approximately 600 columns within the basement required to support various building shapes. These columns have been set generally on grid layout patterns of 7·8, 9·0 and 10·5 m to suit car parking, plant and equipment planning grids.

Each of the commercial towers has a three-lift rise format (comprising low, medium and high rise) (Figure 11). The tower floor-to-floor levels are set at 3·8 m giving the 50-storey northern tower (C3) an overall height of 210 m above ground. The other two towers reduce in height to form a stepped rooftop elevation towards the south.

The typical floor plates of the commercial towers are designed as post-tensioned banded slab systems. The floors, columns and reinforced concrete cores act integrally to provide lateral and torsional stability for the tower structures. A key structural feature of the typical tower floors is that the slab edges cantilever up to 4·5 m from the external face of the columns of the first internal grid line. The floor stiffness has been designed to limit the deflection of the perimeter slab edges to values that can be safely tolerated by the facade stack joints.

All structural framing and stability analysis has been carried out using Strand7 finite-element modelling. Sample framing models are shown in Figure 12.

6. Conclusion

Based on a collaborative approach between the design and construction teams, Barangaroo South is well on its way to becoming a successful, world-class urban regeneration project. As discussed in the paper, several innovative methods have been employed, resulting in significant benefits in many ways, including environmental management, energy-efficient design and major cost savings in foundation systems that contribute to ongoing sustainable engineering developments in our city. Some of the project achievements described in the paper are summarised below.

- Identification of site contamination from past industrial activities and development of suitable remediation strategies, including the potential use of an innovative surfactant-enhanced in-situ chemical oxidation remediation technology.
- Groundwater cut-off for the diaphragm wall was achieved economically with a minimum penetration of 0·3 m into medium strength or better sandstone, together with a detailed programme of groundwater monitoring and a contingency measure of grouting the rock below the diaphragm wall toe.
- Challenging conditions for diaphragm wall construction including cliff ledges along the rock surface, building over two future rail tunnels, and space constraints caused by adjacent building basement were dealt with by careful geotechnical testing/monitoring and smart design.
- Significant savings in pile rock socket lengths using innovative foundation design methods coupled with pile load testing, validation proof coring, and construction monitoring.
- Design and construction of a durable and waterproof basement for a design life of 100 years.
- Structural design challenges that were overcome included adverse conditions due to proximity of the basement to the harbour, up to 100 kPa of hydrostatic uplift pressure on the basement slabs, requirements for open-plan structural layout in the basements, and resolution of structural framing for the basements that can be extended vertically to support the buildings over to cope with the variations of geometric form of the buildings above.

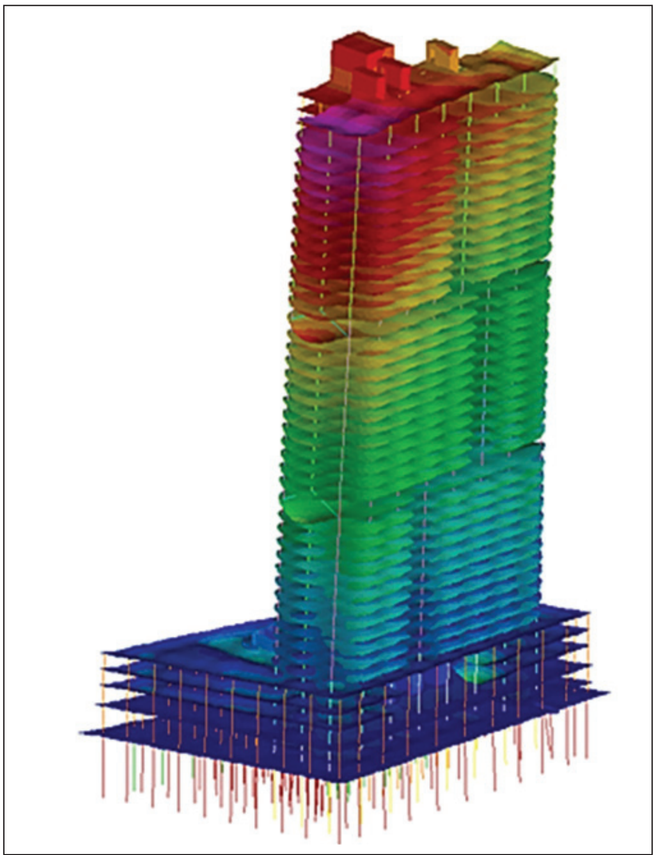

Figure 12. Example of commercial tower finite-element framing model and analysis results

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What do you think?