Geotechnical Engineering
Volume 167 Issue GE2

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ice | proceedings

Proceedings of the Institution of Civil Engineers
Geotechnical Engineering 167 April 2014 Issue GE2
Pages 194–204 http://dx.doi.org/10.1680/geng.13.00031
Paper 1300031
Received 26/03/2013 Accepted 12/09/2013
Published online 02/12/2013
Keywords: anchors & anchorages/diaphragm walls/excavation

ice
Institution of Civil Engineers
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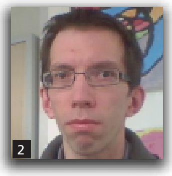
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At Barangaroo South on Sydney's central business district waterfront, Australia, Lend Lease is working with the New South Wales government on the A$6 billion (£4 billion) transformation of a former container port into a thriving business, residential and leisure precinct. The vision is that Barangaroo South will enhance Sydney's position as an internationally appealing, globally competitive, environmentally sustainable city. The development will be Australia's first large-scale carbon dioxide neutral precinct. Barangaroo South will feature three commercial towers ranging in height from 39 to 49 storeys, sharing a common, two-level basement to be retained around its perimeter by an approximately 770 m long diaphragm wall, socketed into Sydney sandstone. As the basement excavation will be below sea level, it is essential that the site retention wall for the basement also acts as a cut-off against groundwater ingress. This paper provides an overview of the design and construction challenges facing the installation of the diaphragm wall and the innovative solutions adopted to overcome these challenges.

1. Introduction

Barangaroo South is a major mixed use development located at the southern end of the Barangaroo development project in Sydney, Australia (Wong *et al.*, 2013). It is located on Sydney's Central Business District waterfront on the foreshore of Darling Harbour as shown in Figure 1, and has a site area of over 7·5 ha. The vision of the developer for the project, Lend Lease, is that Barangaroo South will enhance Sydney's position as an internationally appealing, globally competitive and environmentally sustainable city. The development will be Australia's first large-scale carbon dioxide neutral precinct.

Figure 1. Location of Barangaroo South

Located on the southern third of the 22 ha Barangaroo site, stage 1A of Barangaroo South will feature three commercial towers ranging in height from 39 to 49 storeys, sharing a common, two-level basement to be retained around its perimeter by an approximately 770 m long diaphragm wall, socketed into Sydney sandstone. As the basement excavation will be below sea level, it is essential that the site retention wall for the basement also acts as a cut-off against groundwater ingress. This had to be considered when designing the reinforcement and the temporary ground anchor support required prior to construction of the permanent floor slabs.

A number of challenges had to be overcome during the design and construction of the diaphragm wall, including difficulties associated with old harbour walls and buried wharf structures, steeply dipping rock levels and buried cliff lines, accommodating the proposed future Sydney Metro tunnels, the close proximity of existing building basements, as well as complex bulk excavation staging. Stringent wall deflection and settlement criteria needed to be met, as well as the need to consider seismic events and sea level rise due to climate change.

2. Site history

Barangaroo has a long post-European settlement history of industrial use, predominantly as wharfage but also as a gasworks. The original, undeveloped natural shoreline comprised rocky outcrops and mangrove swamps. 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 gasworks were constructed in 1840 on the coastline of the time and substantial reclamation of the area took place during the mid- and late nineteenth century, with a sandstone block harbour wall and numerous wharves constructed at this time.

Following the demolition and backfilling of the gasworks in 1925, timber wharves were constructed in its place and further northward, such that by the end of the 1930s the entire shoreline in the site area was fronted by timber wharves extending from a sandstone harbour wall. In the early 1970s a concrete caisson harbour wall, which still forms the western boundary of the current site, was constructed parallel to and some 150 m out from the former harbour wall and the reclamation formed by end dumping of fill. The timber finger wharf piers were left in place and a variable depth of predominantly sandstone rubble fill up to 21 m was placed between the two harbour walls. Prior to redevelopment, Barangaroo was occupied by a shipping and overseas passenger terminal, with the majority of the site covered by hardstand, with areas of suspended concrete slab.

3. Geological overview

The geological conditions at Barangaroo are complex. The site is underlain by fill and Quaternary alluvium overlying Triassic age Hawkesbury Sandstone – a fine- to coarse-grained quartzose sandstone deposited in 1–3 m thick beds. Structurally, there are two main rock joint sets, one set trending NNE and an orthogonal set trending ESE. The orientation of these joint sets and the bedding spacing of the Hawkesbury Sandstone, together with erosion and weathering processes in a foreshore environment, results in a steeply stepping rock profile with overhangs, detached blocks and wide, open sub-vertical joints around the harbour foreshore (Figure 2).

The presence of buried cliff lines results in a large variation in rock level across the site, from about 1 m to 2 m below ground level in the north-eastern corner of the site to over 30 m below ground level in the north-western corner – a distance of only some 150 m. In addition, a NW–SE trending palaeochannel crosses the site, extending to a depth of approximately 18 m below ground level and infilled with predominantly loose to medium dense alluvial sand, interbedded with subordinate firm clay. There is potential for the alluvial deposits to be acid sulfate bearing, which may be a problem if they are allowed to oxidise following exposure to air.

The fill material used for the land reclamation is highly variable, both in nature and thickness; many buried obstructions, including

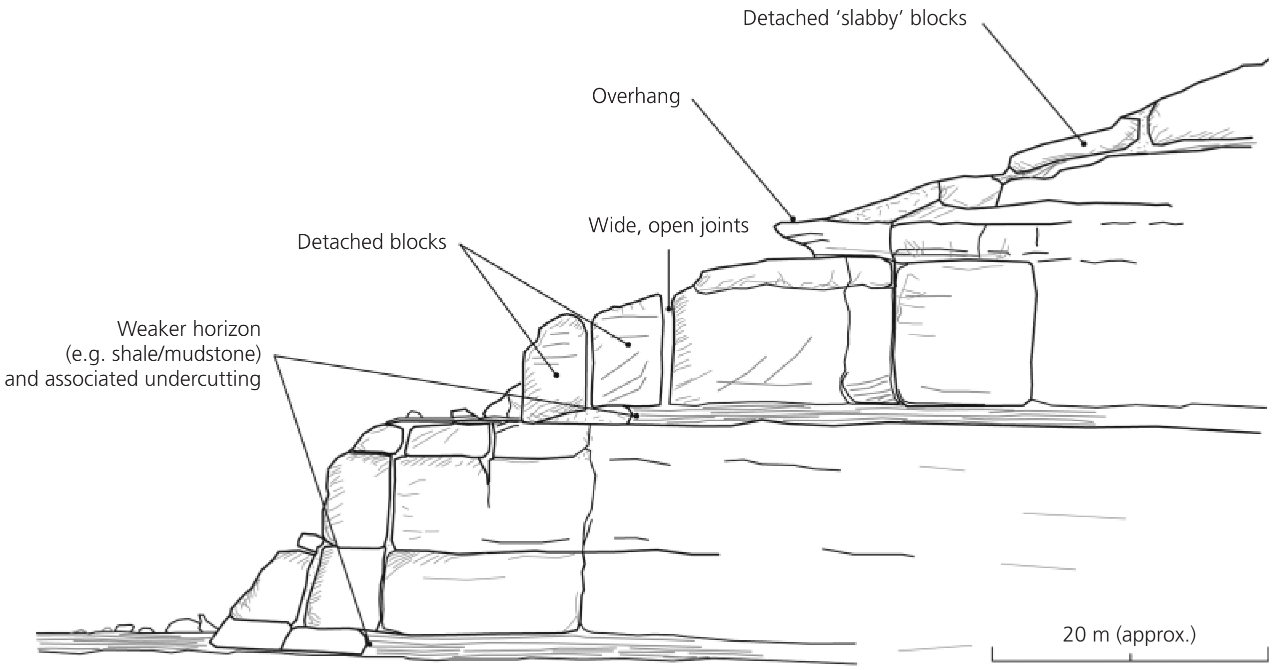

Figure 2. Sandstone cliff profile

old and existing harbour walls and timber wharves, are present. Large voids were also identified in the fill during the site investigation works, as well as wood, steel, sandstone boulders and other building rubble.

A typical geological section along the western (harbourside) boundary of the site is given in Figure 3. The deep fill profile with thick underlying alluvial deposits will provide limited lateral support for a deep foundation system. Also, the fill material across the site is highly permeable and the near-surface water table responds strongly to tidal variations. It is likely that hydraulic connection is present between the Hawkesbury Sandstone and the harbour, although the rock mass permeability is generally low to moderate (typically less than 2 uL) and governed by defects in the rock.

4. Design challenges and solutions

As the development area is large, temporary internal propping of the perimeter diaphragm wall was not considered to be effective, except possibly in the corners of the site. A top-down construction solution was considered but not adopted as the client needed to maintain flexibility in the excavation and construction sequencing, as the development layout had not been finalised at the time of diaphragm wall design.

The diaphragm wall was designed using the commercially available finite-element program Plaxis to assess structural actions and deflections and associated adjacent ground movements, as well as the loads to be carried by the ground anchors and floor slabs. Each analysis typically included 13 stages in order to simulate the temporary and long-term conditions. Check analyses using the Menard Bachy computer software based on subgrade reaction theory were completed to validate the Plaxis results.

Owing to an extremely variable bedrock level and other site constraints, design had to be carried out for a relatively large number of wall cross-sections. Typical wall thicknesses were 0·8 m and 1·2 m with individual panel lengths varying from 3·4 m to 7·8 m, constructed using either 2·8 m wide excavation grabs or Hydrofraise techniques, forming 'bites', with up to three bites required to form a panel. The general excavation level is at −5·85 m Australian height datum (AHD) (approximately 8·5 m below ground level), with some locally deeper excavations for pile caps, pump rooms and lift pits. At tender stage, it was decided to anchor the diaphragm wall with a single row of prestressed multi-strand anchors, installed just above the groundwater table at +1·2 m AHD. This resulted in an increase in steel reinforcement, as the anchors were positioned above the optimum level for bending moment distribution but avoided the high-risk operation of installing anchors below the groundwater table. The basement general layout is shown in Figure 4 and typical diaphragm wall section is shown in Figure 5.

4.1 Design criteria

Stringent design criteria were specified for the diaphragm wall, which included

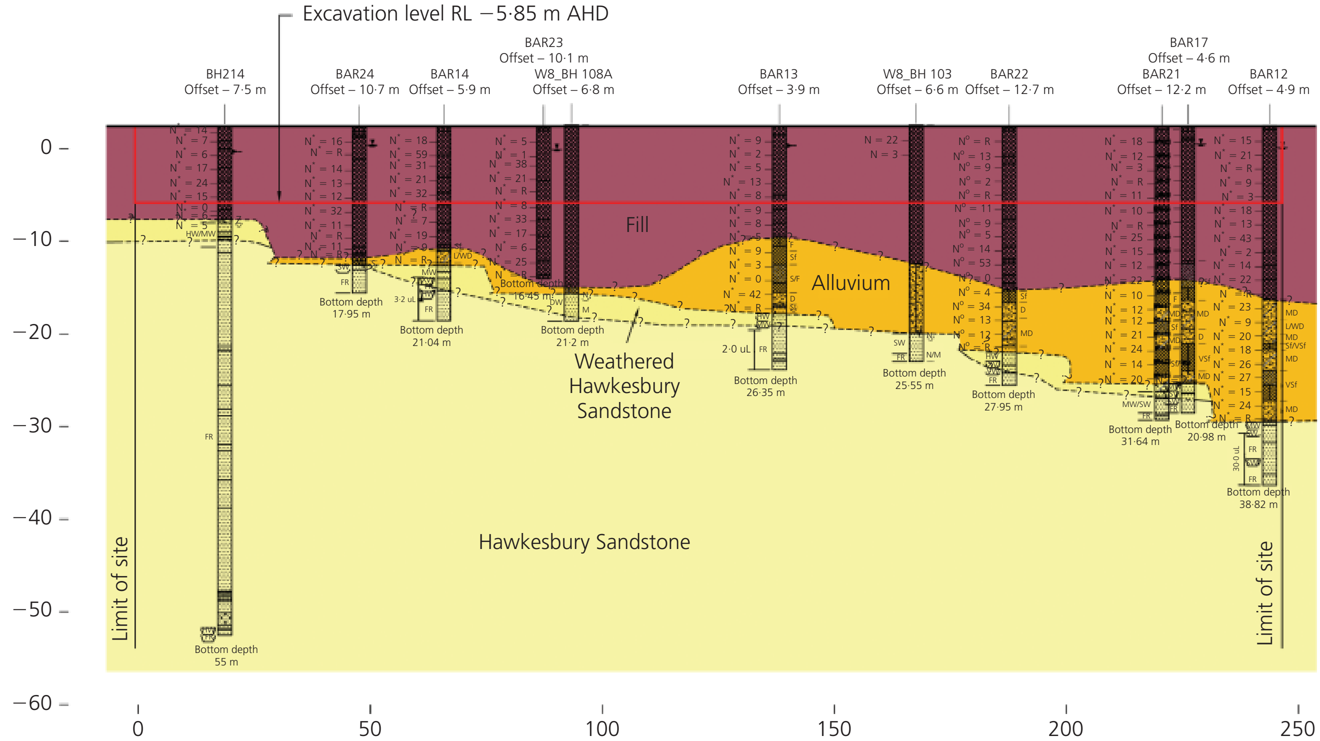

Figure 3. Typical geotechnical section

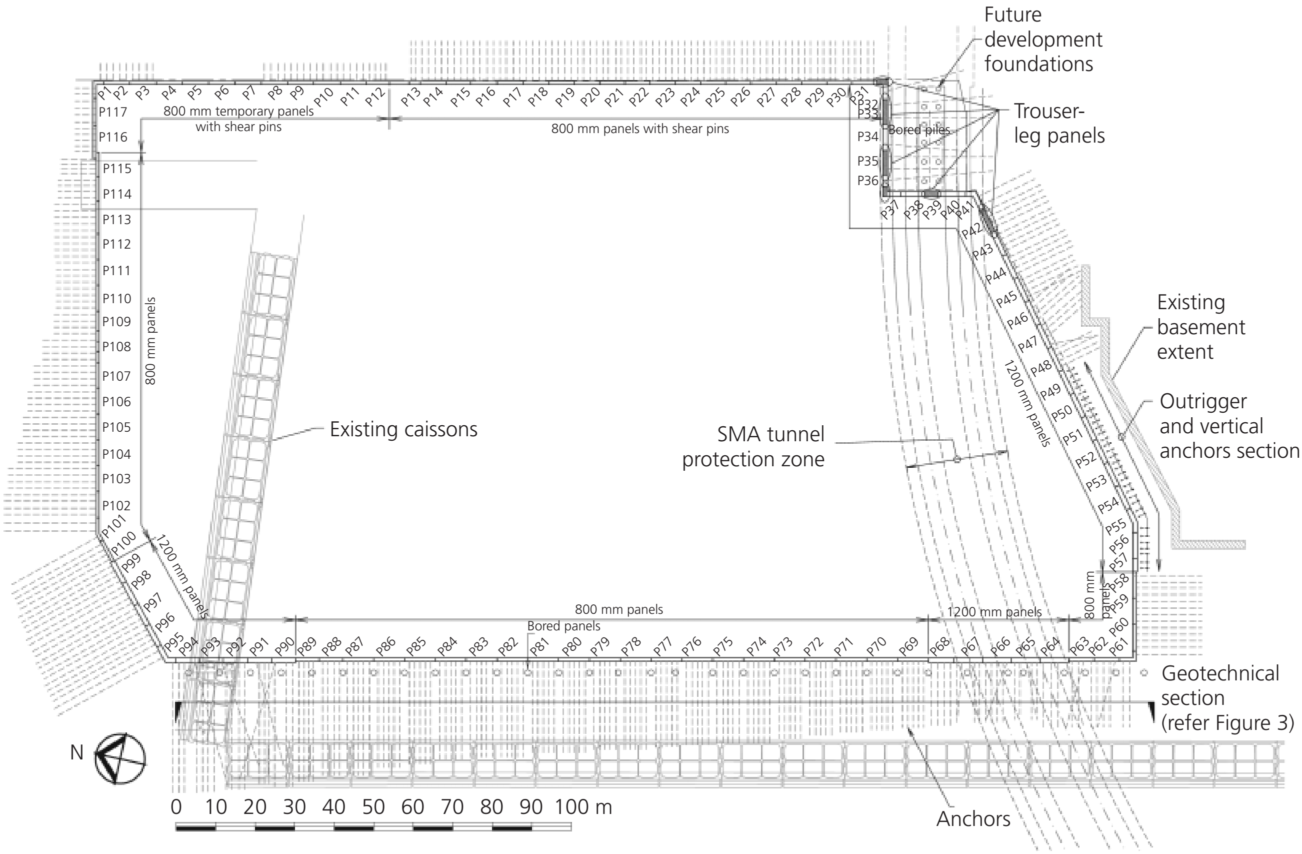

Figure 4. Diaphragm wall panel layout

(*a*) limiting lateral wall deflections to less than $H/200$, where H is equal to the excavation height
(*b*) limiting settlement at the diaphragm wall toe to less than 0·3% of the panel thickness
(*c*) design of permanent support elements for a 100-year design life
(*d*) design of the diaphragm wall that forms the basement and to 2 m below the basement floor to the requirements of Section 3·0 of AS 3735-2001 (Standards Australia, 2001): Concrete structures for retaining liquids
(*e*) diaphragm wall to be substantially watertight with water indications limited to minor damp patches on the wall surface with no visible flow of water; groundwater ingress through the diaphragm wall not to exceed 0·75 litres/min from the entire constructed wall area enclosing the basement
(*f*) building code of Australia importance level 4: 'Building or structure that is essential to post-disaster recovery or associated with hazardous facilities' with an earthquake annual probability of exceedance of 1:1500
(*g*) consideration of sea level rise due to climate change; this was assessed to result in a groundwater level increase of 1 m over the 100-year design life
(*h*) consideration of the effect of base and top slab shrinkage on the diaphragm wall.

In order to satisfy these criteria, close collaboration between the geotechnical and structural designers was essential to drive an efficient reinforcement design that met the design expectations.

4.2 Key design parameters

Some of the key geotechnical parameters adopted for design are summarised in Table 1. The soil and rock were modelled as elastic-plastic material with a Mohr–Coulomb failure criterion in the Plaxis analysis. The fill material was modelled using a hardening-soil model, where the elastic unload/reload modulus was three times stiffer than the primary compression modulus.

In the assessment of wall–soil interaction, both short-term and long-term concrete Young's modulus values were adopted in the numerical analysis, with values of 20 000 MPa and 10 000 MPa ascribed, respectively. For reinforcement design, a concrete Young's modulus of 34 800 MPa was used.

4.3 Watertightness

As the basement excavation will be below sea level, it is essential that the diaphragm wall also acts as a temporary cut-off against groundwater ingress during construction. Once constructed, the basement will be a fully tanked structure and designed to accommodate full hydrostatic uplift pressures. A minimum socket of 0·3 m into medium-strength or better rock was adopted to control groundwater ingress from the under-

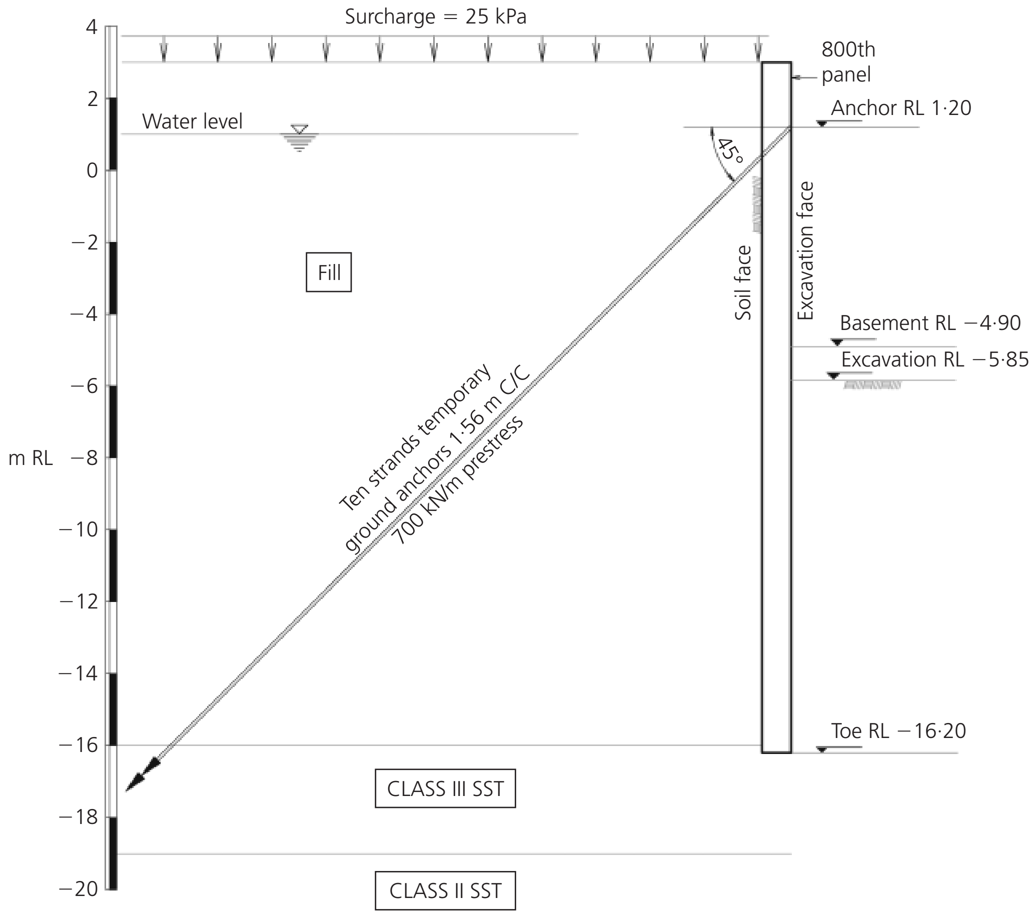

Figure 5. Typical diaphragm wall section

Material	Unit weight, γ: kN/m^3	Drained cohesion, c': kPa	Drained friction angle, ϕ': deg	Elastic modulus, E [a]: MPa	At rest earth pressure coefficient, K_0
Fill	19	2	30	10–25	0·5
Estuarine sediments	18	0	25	10–25	0·5
Alluvial sediments	20	0	31	20–30	0·5
Residual soil	20	5	25	60	0·5
Weathered sandstone	23	30	35	80	Varies[b]
Sandstone	23	200–300	35	500–1000	Varies[b]

[a] Elastic modulus values were selected considering the strain levels induced in the ground.
[b] Estimated based on in situ horizontal stresses, which can be high in Sydney sandstone, with the major stress field in the N–S direction.

Table 1. Summary of geotechnical design parameters

lying rock during construction. The embedment specification was provided by the client and was based on careful review of the geology, testing of the rock (including permeability testing) and analytical evaluation of various depths of embedment.

4.4 Seismic design

Seismic loads acting on the diaphragm wall were assessed in accordance with AS 4678-2002 (Standards Australia, 2002): Earth-retaining structures. A pseudo-static analysis approach was adopted using the Mononobe–Okabe method (Mononobe, 1924) with the calculated pseudo-static loads applied to the diaphragm wall in the finite-element analysis.

4.5 Space constraints

Along the southern site boundary, an existing basement located just 9 m from the diaphragm wall precluded the use of inclined ground anchors. To overcome this space constraint, the following options were explored

(*a*) top-down construction, with installation of the ground-floor slab prior to excavation
(*b*) raked-in struts supported on barrette portal frames installed on the excavation side
(*c*) cantilever wall with vertical anchors.

Option (*c*) was preferred by the client as it did not impinge on excavation sequencing. This solution comprised 1·2 m thick diaphragm wall panels, embedded at least 3 m into medium-strength or better sandstone. To control wall displacements, vertical prestressed anchors were installed on the soil side behind the wall axis and connected to the wall by a cast-in-situ concrete outrigger beam (Figure 6). The vertical lock-off anchor force generates a couple at the top of the wall, which in turn reduces wall deflections. The anchor lock-off load was adjusted to maintain a lateral deflection below $H/200$. The design was carried out using subgrade reaction and finite-element analyses, both of which indicated that anchors with a working load of 1400 kN at 1·6 m spacing would be required.

4.6 Future metro tunnels

The diaphragm wall had to include provisions for the construction of the proposed Sydney metro tunnels, which cross the wall alignment in the south-east corner and the western boundary.

In the south-east corner, the diaphragm wall was constructed adjacent to and across the future tunnel alignment. The wall panels were required to carry relatively large building loads as the tunnel geometry restricted the use of piled foundations. The design was further complicated by the presence of limited rock cover above the tunnel crown. To transfer the building loads away

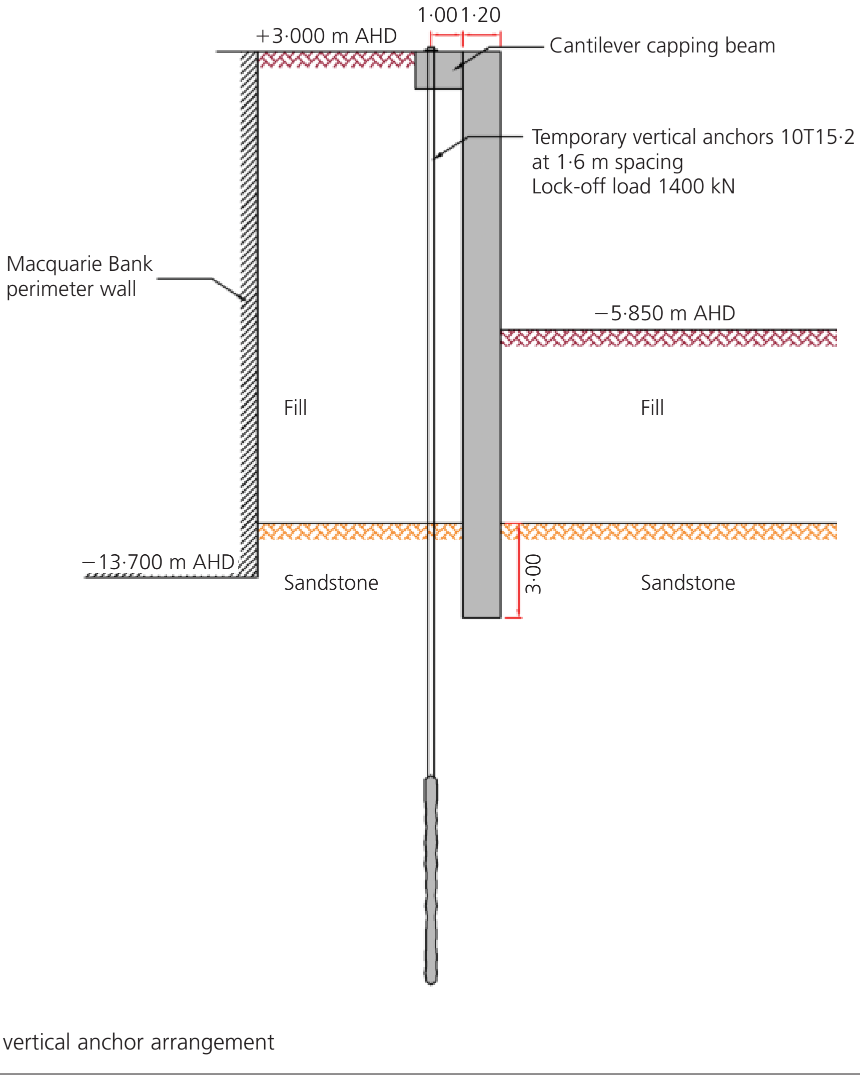

Figure 6. Outrigger and vertical anchor arrangement

from the section of diaphragm wall which crossed above the future tunnels, a number of the adjacent diaphragm wall panels were extended to a greater depth to form a 'trouser-leg' arrangement (Figure 7). These 'trouser legs' were successfully constructed using a Hydrofraise to cut the high-strength sandstone.

The future tunnels will be constructed through several diaphragm wall panels, therefore 'soft eyes' using fibre-glass reinforcement were incorporated in a number of reinforcement cages to facilitate tunnel excavation. Also, as most of the temporary ground anchors clash with the tunnel alignment, removable ground anchors were installed. After the base and ground-floor slabs are cast, these anchors will be de-stressed and the strands removed from the ground.

At the tunnel crossing on the western boundary, the tunnel crown and the toe of the diaphragm wall are separated by a minimum of 4 m of sandstone. Finite-element analysis showed that the vertical loads applied on the diaphragm wall in this area will have negligible impact on tunnel excavation and lining design. Therefore, no specific measures apart from the installation of removable anchors had to be implemented for this section of the diaphragm wall.

The future tunnels also pass within close proximity of a number of panels along the southern boundary, therefore the impact of tunnel excavation-induced ground movements on these panels had to be considered. The tunnelling-induced vertical and lateral ground movements were first assessed using analytical methods described by Mair *et al.* (1993) and Loganathan and Poulos (1998) using the volume loss concept. Using a weighted average volume loss for the tunnels of 0·9%, as the tunnels will be partly in alluvium and partly in rock, the resulting tunnel crown settlement and lateral movement at the tunnel centreline were estimated. Numerical analysis was completed to simulate the tunnel excavation process and to obtain magnitudes of soil movements that were similar to the values given by the analytical methods. Greater emphasis was placed in matching the lateral movement, as this has a greater influence on the wall panel socketed into rock adjacent to the tunnel. Deflections and stresses developed in the diaphragm wall due to tunnel excavation were assessed from the numerical analysis and the reinforcement design modified to accommodate the increased structural actions.

5. Construction challenges and solutions

5.1 Varying rock levels and buried cliff lines

Where the rock was shallow (mainly along the eastern site boundary), the diaphragm wall panels were terminated in rock 0·5 m below the excavation level and shear pins installed at their base to provide adequate toe stability (Figure 8). The shear pins consist of 127·5 mm diameter high-grade steel tubes embedded 2 m into the rock. Ducts, 200 mm in diameter, were fixed inside the reinforcement cages to allow the drilling of the shear pins through the completed panel. As these panels required considerable excavation into rock, a Hydrofraise Evolution 3 was used to increase production rates and reduce vibrations that would have resulted from conventional chiselling.

In areas where bedrock levels were deep (e.g. along the western

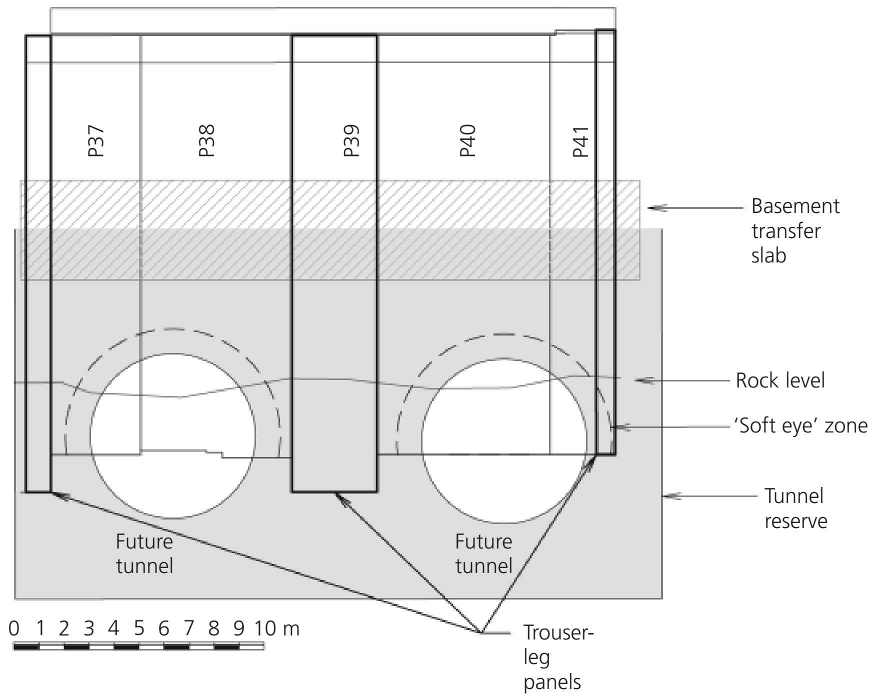

Figure 7. Diaphragm wall trouser-leg arrangement

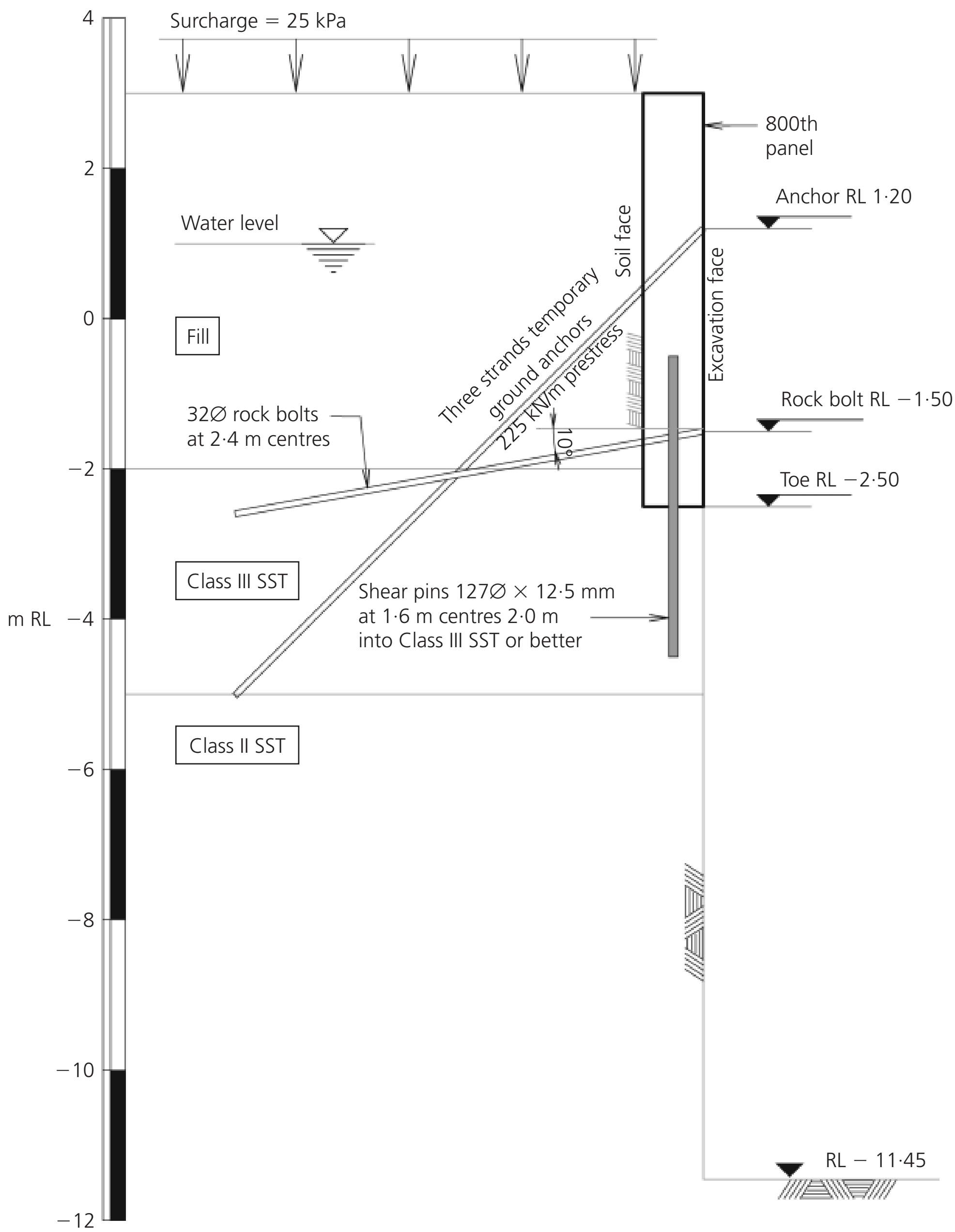

Figure 8. Diaphragm wall in areas of shallow rock

and northern boundaries of the site), the diaphragm wall panels were constructed with a minimum socket of 0·3 m into medium-strength or better rock. The rock socket provided both ground-water cut-off and bearing capacity. In a number of locations, the stepped nature of the rock surface resulted in rock level changes across the length of an individual panel. Where this occurred, steps were introduced into the panel such that each bite achieved the minimum embedment depth. As each panel was excavated in three bites, a maximum of two steps were introduced along the length of any one panel. At the peak of productivity, four mechanical grabs were simultaneously operating on site and the rock sockets were formed using grab and chiselling techniques.

5.2 Crossing of the existing caisson harbour wall

In the north-western corner of the site, the diaphragm wall crosses the existing caisson harbour wall. These caissons are reinforced concrete elements, each some 16 × 9 m in size and 15 m high, and composed of six cells infilled with sand and separated by 0·2–0·3 m thick walls, with a 0·8 m thick raft foundation founded on a gravel mattress.

The caissons had to be removed locally using rotary coring techniques to allow for the 1·2 m thick diaphragm wall to be constructed. Ten secant cores (1·5 m diameter at 0·9 m spacing) were required to form a slot greater than the diaphragm wall panel thickness, taking into account positional and verticality tolerances. Once a core was complete, the casing was gradually extracted and the hole backfilled with a mixture of clean sand and cement. During these operations, caisson movements were monitored on both sides of the trench using survey targets, and monitoring records showed that the displacements remained below threshold values.

5.3 Anchor installation and stressing

The diaphragm wall panels are anchored by a single row of temporary multi-strand ground anchors with a service life of 2 years. A total of 454 anchors were installed at an angle of 45° and bonded into 150 mm dia. boreholes drilled into medium-strength or better sandstone. The typical working capacity for these anchors ranged from 360 kN to 1700 kN, necessitating the use of as many as 12 strands, each strand being 15·2 mm dia., plastic sheathed and greased along their free length. Anchor reservations were built in the panel reinforcement cage, with appropriate horizontal reinforcement provided such that an external waler was not required.

Depending on the design cross-section, the temporary anchors were locked off at a prestress load of between 60% and 90% of the design working load in order to control wall lateral displacement. The lock-off procedure was in accordance with British Standard BS 8081 (BSI, 1989).

In the north-western corner of the site, installation of ground anchors was greatly complicated by the presence of the existing caissons behind the diaphragm wall. Installation of the anchors required drilling through several thick, reinforced concrete caisson walls. Buried steel beams and sheet pile walls were among other subsurface obstructions encountered and sonic drilling techniques were successfully used in these areas.

The depth of the bedrock in this area (over 30 m below ground level) resulted in very long anchor free lengths (up to 50 m). As the hydraulic jack stroke was limited and not sufficient to absorb all the free length elasticity, a two-stage lock-off procedure was required to perform the acceptance tests. Wedges were locked at an intermediate load step, so enabling the ram to be pulled back to its original position.

5.4 Removable anchors

As excavation of the future metro tunnels would be hindered by the presence of anchor strands, all temporary anchors had to be removed in the tunnel protection zones. Fully removable anchors were installed using methods based on single-bore, multiple-anchor (SBMA) technology. SBMA consists of a number of short unit anchors bonded to the rock at staggered depths in a single borehole (Figure 9). Each anchor unit is equipped with a 180° looped strand, greased and sheathed over its entire length, and attached to a steel saddle installed in the borehole. The anchor is prestressed by pulling on the strands at both ends and load is

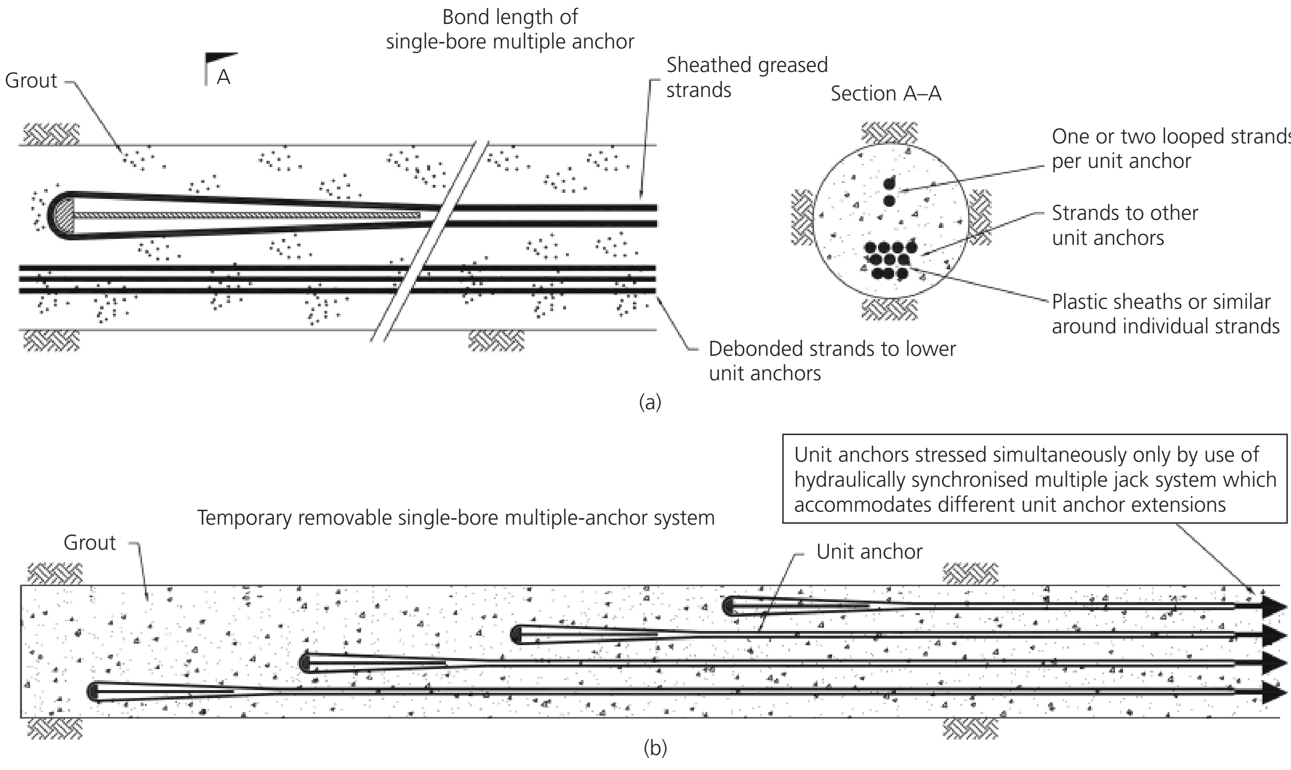

Figure 9. Single-bore multiple anchor: (a) detail of single-bore anchor; (b) section through single-bore multiple anchor

transferred from the unbonded strand to the saddle. A steel thread bar fixed to the saddle transfers the load to the grout and thus to the surrounding rock. When required, the greased, unbonded strand can be removed by pulling on one end using either a winch or a jack.

Altogether, 35 removable anchors were successfully installed in the south-east corner and on the western boundary. Depending on the capacity required, between four and 11 units were installed in a 180 mm dia. borehole and each anchor was proof tested to 25% above the working load. On completion of the load test and acceptance of the anchor, each unit was individually reloaded and locked off to 10% above the prestress load to account for friction losses and wedge draw-in.

In the south-east corner, the diaphragm wall forms a 90° angle, orientated towards the excavation, therefore the risk of cutting an installed anchor during the drilling for other anchors was a major concern. To mitigate the risk, the anchor spacing was increased and anchors were installed at different inclinations and skew angles to ensure that the distance between anchor holes was always safe. In addition, sections of the diaphragm wall had to be left anchor free to allow for the installation of future piles behind the wall prior to the removal of the temporary anchors.

The drilling of some anchors was hindered by the presence of old timber piles that once supported wharf structures. Where such obstructions were encountered, sonic drilling techniques proved successful.

6. Monitoring of diaphragm wall performance

The performance of the diaphragm wall is being monitored by inclinometers installed within the reinforcement cages and extending some 4 m below the panel toe and survey targets located on the diaphragm wall (top, mid-height and base of excavation).

At the time of writing, the formation level has been reached and lateral deflections remain within acceptable limits as defined by the client. Generally the recorded lateral deflections into the excavation are below the predicted movements inferred from the design calculations and provide a level of confidence that the diaphragm wall is performing as per the design intent. An example plot for an inclinometer installed in the western (harbourside) wall, where the formation level has been reached but permanent support is yet to be installed, is given in Figure 10, together with the deflection profile predicted from the Plaxis analysis for the diaphragm wall at this construction stage for comparison.

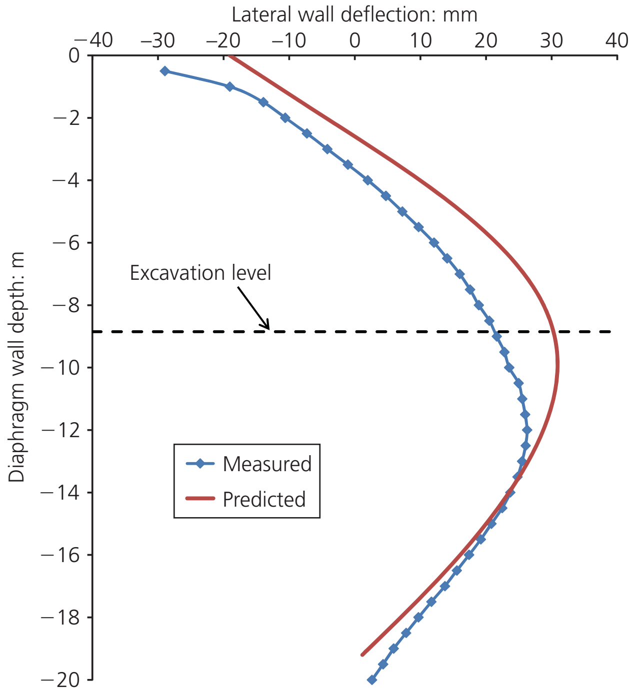

Figure 10. Measured compared with predicted diaphragm wall deflections

7. Conclusion

The collaborative approach adopted by the design and construction teams resulted in meeting a very tight delivery programme for the diaphragm wall while overcoming significant in-ground challenges. Some of the challenges faced required innovative design solutions, including vertical anchors, removable anchors and 'trouser-leg' panels, as well as detailed numerical analysis to carefully assess impacts of the diaphragm wall on future metro tunnels, and vice versa.

Some of the project achievements described in the paper are summarised below.

- Watertightness was achieved by limiting the number of anchors used below groundwater level and reinforcement design to AS 3735-2001 (Standards Australia, 2001).
- Groundwater cut-off was achieved economically with a minimum wall penetration of 0·3 m into medium-strength or better sandstone.
- Detailed assessment was undertaken of the diaphragm wall impacts on the future metro tunnels, and conversely the impacts of tunnelling-induced ground movements on the diaphragm wall performance.
- Challenging conditions for construction, including cliff ledges, building over future metro tunnels and space constraints due to adjacent basements, were dealt with by detailed geotechnical testing/monitoring and smart design.

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