Lend Lease (Millers Point) Pty Limited

Barangaroo South – Concept Plan Amendment (MP06_0162 MOD 4)

Geotechnical Report

ARUP

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Geotechnical Report

July 2010

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Job number   220316-00

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1 Introduction

1.1 Background

On the 20 December 2009, Lend Lease (Millers Point) Pty Limited (Lend Lease) was appointed as the preferred proponent to develop Barangaroo South comprising of Blocks 1 to 4 and associated public recreation areas.

The area of land that is subject to the Concept Plan Amendment is indicatively shown in Plate 1, and is herein referred to as “Barangaroo South” or the “Site”. It comprises an open apron which is largely reclaimed over water and is identified in the existing approved Concept Plan as Blocks 1 – 4 and the immediately adjacent public recreation area. Barangaroo South also extends beyond the western edge of the existing apron and includes a north-west oriented intrusion into the existing waters of Darling Harbour (see Plate 1).

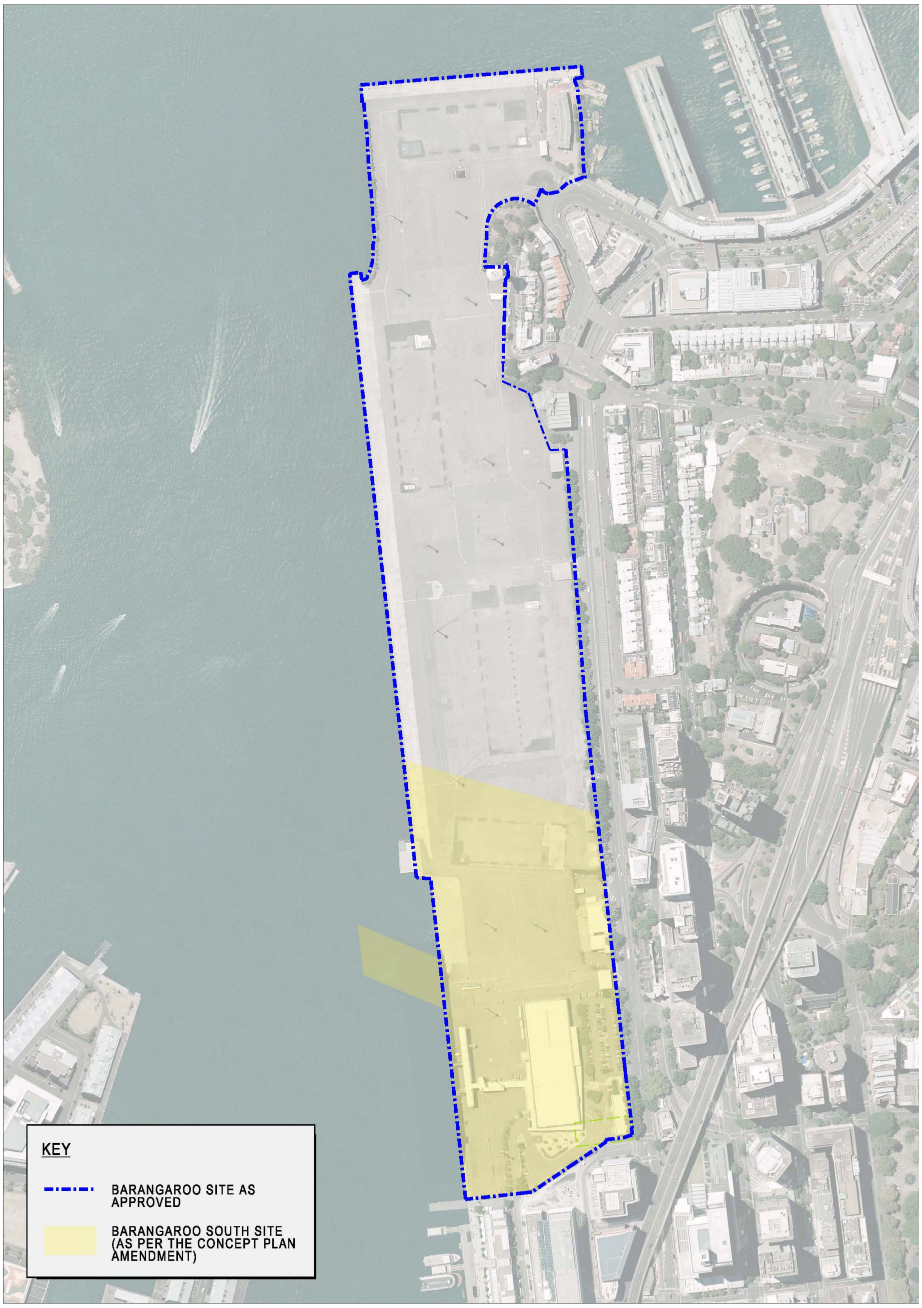

Plate 1: Indicative Site Boundary for Barangaroo South

1.2 Planning History

On 9 February 2007 the Minister approved a Concept Plan for the site and on 12 October 2007 the land was rezoned to facilitate its redevelopment. The Approved Concept Plan allowed for:

- a mixed use development involving a maximum of 388,300m^2 of gross floor area (GFA) contained within 8 blocks on a total site area of 22 hectares;
- approximately 11 hectares of new public open space/public domain, with a range of formal and informal open spaces serving separate recreational functions and including a 1.4km public foreshore promenade;
- maximum building heights and maximum GFA for each development block within the mixed use zone; and
- public domain landscape concept, including parks, streets and pedestrian connections.

A condition of consent also required two enlarged water intrusions into the Barangaroo site, one at the northern end and one at the southern end and the creation of a natural northern headland.

Modification No. 1 was approved in September 2007 which corrected a number of minor typographical errors.

On 25 February 2009 the Minister approved Modification No. 2 to the Concept Plan. The Approved Concept Plan as modified allowed for a mixed use development involving a maximum of 508,300m^2 of gross floor area (GFA) contained within 8 blocks on a total site area of 22 hectares.

On 11 November 2009 the Minister approved Modification No. 3 to the Concept Plan to allow for a modified design for the Headland Park and Northern Cove. The Approved Concept Plan as modified allowed for a mixed use development involving a maximum of 489,500m^2 of gross floor area (GFA) contained within 7 blocks on a total site area of 22 hectares.

The proposed Concept Plan Amendment (MP 06_0162 MOD 4) seeks the Minister's consent for:

- additional GFA within Barangaroo South, predominantly related to an increase in residential GFA;
- redistribution of the land use mix;
- an increase in height of a number of the proposed towers within Barangaroo South;
- the establishment of the new pier and landmark building extending into the Harbour; and
- reconfiguration and activation of the public waterfront area through the introduction of uses including retail and residential to the west of Globe Street.

1.3 Purpose

This report has been prepared in support of the Concept Plan Amendment (MP06_0162 MOD 4) for Barangaroo South. It addresses the Director General Requirements and it has been prepared on the basis of existing information that is available for the site, information from adjacent sites and other geological / geotechnical information available within the public arena.

1.4 Scope

This geotechnical report has been prepared to support the Planning Application for the Concept Plan Amendment and to inform the early stages of design. The report includes the following:

- Site description including discussion on topography and regional geology, soils and groundwater regimes;
- A brief discussion of the historical development of the site;
- Summary of existing investigative information for the site and the surrounding area;
- Development and discussion of a preliminary geological model for the site including preliminary design parameters for the various soil and rock units encountered;
- Development and discussion of a preliminary hydro-geological model for the site and discussion on potential impacts of the development on those conditions; and
- Preliminary engineering advice appropriate to the Planning Application for Concept Plan Amendment and preliminary engineering; and
- Specific comment on the Landmark (Hotel) Building to address geotechnical issues associated with the construction of the building within Sydney Harbour.

1.5 Proposed Development

The Concept Plan Amendment and associated cross sections indicate the following site features:

- The development will comprise commercial, residential, tourism and retail facilities;
- Three large commercial towers (C3 to C5) with indicative maximum heights of up to 200m and a number of smaller ancillary structures around the site;
- A spilt level basement to a reduced level of approximately -6m AHD towards the west and centre of the site, with a deeper level basement extending to approximately -25m AHD towards the east of the site adjacent to Hickson Road/Sussex Street.
- A narrow section of the basement to the south west of the Metro corridor planned to be excavated to a depth of approximately -17mAHD
- A Landmark building, towards the north west of the site, located within Sydney Harbour.

1.6 Limitations

This report contains an interpretation of existing available geotechnical information of the site. The ground is a product of continuing natural and man-made processes and therefore exhibits a variety of characteristics and properties which vary from place to place and can change with time. Geotechnical and environmental engineering involve gathering and assimilating limited facts about these characteristics and properties in order to understand or predict the behaviour of the ground and groundwater on a particular site under certain conditions. Arup may report such facts obtained by inspection, excavation, probing, sampling, testing or other means of investigation. If so they are directly relevant only to the ground and groundwater at the place where and at the time when the investigation was carried out and is believed to be reported accurately. Any interpretation or recommendation given by Arup shall be understood to be based on judgment and experience and not on greater knowledge of the facts than the reported investigations would imply. The interpretation and recommendations are therefore opinions provided for our Client's sole use in accordance with a specific brief. As such they do not necessarily address all aspects of ground behaviour on the subject site.

2 The Site

2.1 Site Description

The Barangaroo South site is located to the north-west of the Sydney CBD (see Figure 1). It is bounded by Millers Point to the north, Sydney Harbour the north and west, the new Macquarie Group Building development, KPMG Tower and King Street Wharf to the south, and by Hickson Road/Sussex Street in the east.

The proposed Barangaroo South site is located at the southern end of the wider area, and covers approximately eight hectares.

There are a number of existing structures on the site comprising the following:

- The Cruise Terminal and Function Centre located towards the southern end of the site, comprising a terminal building and a pedestrian gangway, which are proposed to be demolished under consent S07/01342-1.
- A number of low rise commercial / industrial buildings exist along the eastern boundary of the site associated with the former port operation, which are proposed to be demolished under consent S07/01342-1.
- A number of significant lighting towers generally located near the western edge of the site, which are proposed to be demolished under consent S07/01342-1.
- The existing caisson walls that form the edge of the existing wharf area as described in Section 3.2.1. This includes the walls of the infilled Wharf 7 that are anticipated to remain buried towards the northern extent of the site.
- In addition to the structures described above, it is anticipated that a significant number of remnant structures related to the long maritime usage of the site remain in-situ. These structures may include dock walls and piles of various types and sizes.

For the purpose of reference, a general arrangement of the proposed development is illustrated in Figure 2.

2.2 Topography

The ground surface over the entire site lies at an elevation of approximately +2mAHD. The surface of the site is paved with either asphalt or concrete.

The surrounding landform rises rapidly towards the east. A significant sandstone cliff exists on the eastern side of Hickson Road/Sussex Street which is in the order of 8-10m in height. This is consistent with the sandstone landform which weathers along the sub-vertical defects in the rock mass to create a series of steps. It is likely that this stepped structure continues beneath the fill and alluvial materials present on the site.

2.3 Regional Geology and Geomorphology

The Sydney Geological Map scale 1:100,000[1] (see Figure 1) and the Sydney Geological Map scale 1:250,000[2] indicate that the site is underlain by the Hawkesbury Sandstone of the Wianamatta Group, which is overlain by Quaternary sediments and manmade fill. The geological map indicates that the majority of the Barangaroo South site is underlain by manmade fill. This is consistent with the knowledge that the majority of the site has been reclaimed successively since the late 1800's.

1 *Geological Map of Sydney, Geological Series Sheet 9130, N.S.W Department of Mineral Resources, First Edition 1983*

2 *Geological Map of Sydney, Geological Series Sheets S1 56-5, Geological Survey of N.S.W, Third Edition 1966*

The Barangaroo South site is located adjacent to the Sydney Harbour. The Sydney Geology Sheet Memoir (Herbert, 1983) states that Sydney Harbour is a drowned river valley system characterised by steep sided valleys in Hawkesbury Sandstone.

The Hawkesbury Sandstone is a medium to coarse grained quartz sandstone, with occasional minor shale and laminite lenses. The sandstone comprised massive, bedded and cross-bedded units (massive and sheet facies). Structure within the Hawkesbury Sandstone generally comprises sub-horizontal undulating bedding plane seams and cross bed partings and two sub-vertical joint sets with general orientations of approximately 100 - 140° and 350-20°.

Several well documented structural lineaments pass through the Hawkesbury Sandstone beneath the greater Sydney CBD[3], generally striking in a NNE direction. These features generally comprise significant closely spaced sub-vertical jointing and faulting with significant vertical and horizontal continuity. The rock mass surrounding these features may be more weathered and of lower strength than adjacent areas.

In addition a number of igneous dykes are present, generally orientated approximately orthogonal to the structural lineaments. These intrusions are generally sub-vertical with thicknesses of 0.5m to several metres. The dyke material is often highly weathered to significant depths and can comprise high to very high strength rock at depth. The rock mass around these intrusions can be disturbed with significant sub-parallel fracturing, associated deeper weathering and reduced strength.

Both the structural lineaments and dykes can be associated with instability and elevated groundwater inflows when encountered in excavations.

The location of structural lineaments and igneous dykes within proximity of the site are illustrated in Figure 1.

The Lunar Park Fault Zone has been mapped in the Lunar Park excavation, Towns Place basement excavation, Star City basement and in a rail cutting near the Fish Markets[4]. Thrust faulting, significant shear zones and significant sub-vertical jointing have been recorded in observations of the Lunar Park Fault Zone. In addition, the fault zone is associated with significant reductions rock strength. Extrapolation of the alignment of the fault zone shows it to cross the Barangaroo site near the northern extent of Barangaroo South.

The Pittman LIV Dyke traverses the CBD, sub-parallel to Grosvenor and Bridge Streets. The location of the dyke is inferred to cross the Barangaroo area approximately three hundred metres north of the current passenger terminal building, near to the northern extent of Barangaroo South.

2.4 Soil Landscapes

The Sydney Soil Landscape Map scale 1:100,000[5] indicates the site is underlain by disturbed soils (see Figure 1).

The acid sulphate soil risk map for the site (Prospect/Parramatta River 1:25,000 Scale, 1997) indicates the site is underlain by disturbed terrain. A high probability of acid sulphate soil is associated with estuarine sediments in Sydney Harbour which are likely to underlie the manmade materials.

The presence of disturbed ground is in agreement with the known progressive reclamation of the area since the late 1800's.

3 *Map and Selected Details of Near Vertical Structural Features in the Sydney CBD*

4 *Sydeny metro Authority, CBD Metro, Geotechnical Interpretative Reportr CBD-2110-GHD-R-GE-001-[F], GHD Ceotechnics*

5 *Soil Landscapes Map of Sydney, Soil Landscape Series Sheet 9130, Soil Conservation Service of N.S.W., First Edition 1983.*

2.5 Hydrogeology

Data obtained from the Department of Natural Resources in 2006[6] identified 32 registered groundwater bores within 4km of the site. Of these 22 were for monitoring, 8 for recreation and 3 for irrigation. None of these boreholes were registered for drinking water purposes. No registered groundwater bores were identified on or adjacent to the site. It is understood that this dataset was updated to include some monitoring wells that were installed on the site during recent site investigations. The revised data has been requested but has not yet been received for review.

Considering the proximity to the harbour and the adjacent steep landform to the east, the hydrogeological conditions beneath the site are anticipated to be influenced by both groundwater flow from the east beneath the CBD and tidal fluctuations associated with the harbour. The groundwater profile would be expected to be approximately coincident with the harbour level near the western edge of the site, rising slightly to the east as the topographic influence takes effect.

ERM (2007)[7] indicates that the depth to water was recorded during July 2006 as ranging between 1.7 m and 2.5 m below ground level, and levels ranging from RL +0.07mAHD to RL +0.65mAHD. Short term variations in groundwater levels were reported as occurring during drilling, particularly in locations close to the seawall. This, along with the generally saline nature of the groundwater, indicates that the groundwater regime on the site is likely to be strongly influenced by tidal fluctuation.

It is anticipated that the groundwater in the fill and alluvial materials is connected to that within the underlying Sandstone.

[6] *ERM 2007 Environmental Site Assessment East Darling Harbour, Sydney, NSW. Final Report – Rev 1. Report Ref 0044432RP02 Rev01 Final.*
[7] *ERM 2007 Environmental Site Assessment East Darling Harbour, Sydney, NSW. Final Report – Rev 1. Report Ref 0044432RP02 Rev01 Final.*

3 Site History

3.1 Historical development

Early maps from around 1788 show a small settlement around the south western extent of Sydney Cove and the Tank Stream inlet. It is not until the map of 1836 that infrastructure and several small buildings are present near the eastern boundary of the site. It is envisaged that these roads and buildings are on the high ground to the east of the Hickson Road cliff line.

The historical maps and photography indicate that the majority of the site has been produced by land reclamation since the late 19th Century, and that it has predominantly been used for the berthing of ships and storage of materials from shipping.

A gas works was located in the middle of the site from 1820 to 1921.

An account of the historical development of the site is illustrated in Figure 3 to Figure 5 and is summarised in the following table.

Date	Site	Surroundings
1836	The shoreline runs approximately along the line of the current Hickson Road. Only the northern part of the site is above sea level.	There is development of the city in the Rocks and the CBD area. It appears to be largely housing with the military barracks located to the west of the current Wynyard Park and Fort Philip where the Observatory is currently located.
1840	A gasworks operated by The Australian Gas Light Company is present in the centre of the site (ERM, 2008) [8]	No information available.
1888	The land in the northern part of the site was extended by reclamation and a number of buildings were erected. Four finger wharves extended southwards from the land. The Australian Gas Light Company was located on the southern half of the site (tanks and wharves are shown on the map). To the south of the gas works a number of finger wharves were built running east-west into Darling Harbour.	Much of the current road network has been constructed from Millers Point through to the CBD. A number of small wharves had been constructed in the area that is currently known as Walsh Bay.
1922-1925	The gas works above ground were demolished and the gas holding tanks were backfilled (ERM, 2008) [9].	No information available.
1951	Ten finger wharves had been established across the southern two thirds of the site. The northern third of the site comprised of reclaimed land with warehouse type structures. The gas works are no longer evident on the site.	A main road leading to the Harbour Bridge and toll booths has been constructed along with the approach ramp to the Cahill Expressway. The current large finger wharves in Walsh Bay have been constructed.
1961	The southern part of the site was being	The Cahill Expressway has been

8 *ERM, 2008. Draft Stage 2Remedial Action Plan for Barangaroo, Hickson Road, Sydney. Report Ref 0087036R03 Draft Rev02*

9 *ERM, 2008. Draft Stage 2Remedial Action Plan for Barangaroo, Hickson Road, Sydney. Report Ref 0087036R03 Draft Rev02*

Date	Site	Surroundings
	reclaimed at this time. Four wharves remain at the centre of the site (labelled wharves 3 – 6 from north to south) The northern area of the site is unchanged.	completed. No other significant changes.
1968	Warehouses have been built, located in the southern reclaimed area.	No significant changes.
1970	The four remaining finger wharves in the centre of the site have been removed and the area was being reclaimed except for a small area between wharves 6 and 7.	Tower blocks have been constructed in the Sydney CBD such as the Observatory Hotel and Australia Square.
1986	The entire site was reclaimed and the shore was almost in its present location except for one remaining wharf (Wharf 7). The buildings on the northern part of the site have been demolished. The entire site has been covered with asphalt or concrete and four large warehouse structures erected. One on the northern boundary, one in the top half of the western boundary, one in the centre of the site and one running east west on the boundary of the old wharf 6. One of the wharves of Walsh Bay adjoining the site had been removed and the land had been extended into Sydney Harbour.	Building heights throughout the CBD increased.
2000	The final dock area has been reclaimed (Wharf 7). In the southern part of the site two warehouses had been removed and the Passenger Terminal had been constructed.	No significant changes.
2004	A short section (approx. 30m) of wharf extending westwards had been added in the location of the old finger wharf 6.	No significant changes.
2008	All buildings on the site except for the Passenger Terminal and smaller ancillary buildings have now been removed.	No significant changes.

Table 1 – Summary of site history.

3.2 Underground structures

3.2.1 Sea wall

An assessment of the seawalls was undertaken in August 2005 by Sinclair Knight Merz (SKM). The report indicates that the construction of these seawalls is as follows:

- Wharves 3, 4, 5 & 8 – The wharves comprise of a reinforced concrete caisson construction filled with a sand material and topped with a concrete slab. The concrete slab level is approx. RL+3.0m with the toe of the caisson at RL-12 to -13m. The typical cross sections indicate that base of the caisson is approximately 11.9m in width and that there is “select sandstone fill” behind the caisson. The lateral extent of this sandstone fill is not reported but is shown to be approximately 6m wide at the level of the top of the caisson with a 1:1 rear slope.

- Wharf 3 – At the western end of the wharf is a sandstone wall construction topped with a concrete slab.
- Wharf 7 – 100m in length and just to the north of Cruise Terminal and comprises five rows of steel piles topped with a suspended concrete slab. The exact location of these rows of piles is not indicated in the drawings. Beneath the suspended slab, fill material slopes down to the harbour floor at a 2:1 gradient. The surface of the slope is protected with a 1200mm thick layer of rock armour.

A typical section through Wharves 3, 4, 5 & 8 is provided in Plate 2. The coping detail of Wharf 8 is slightly different to the others sections but the general cross section is similar. A typical cross section through Wharf 7 is provided in Plate 3.

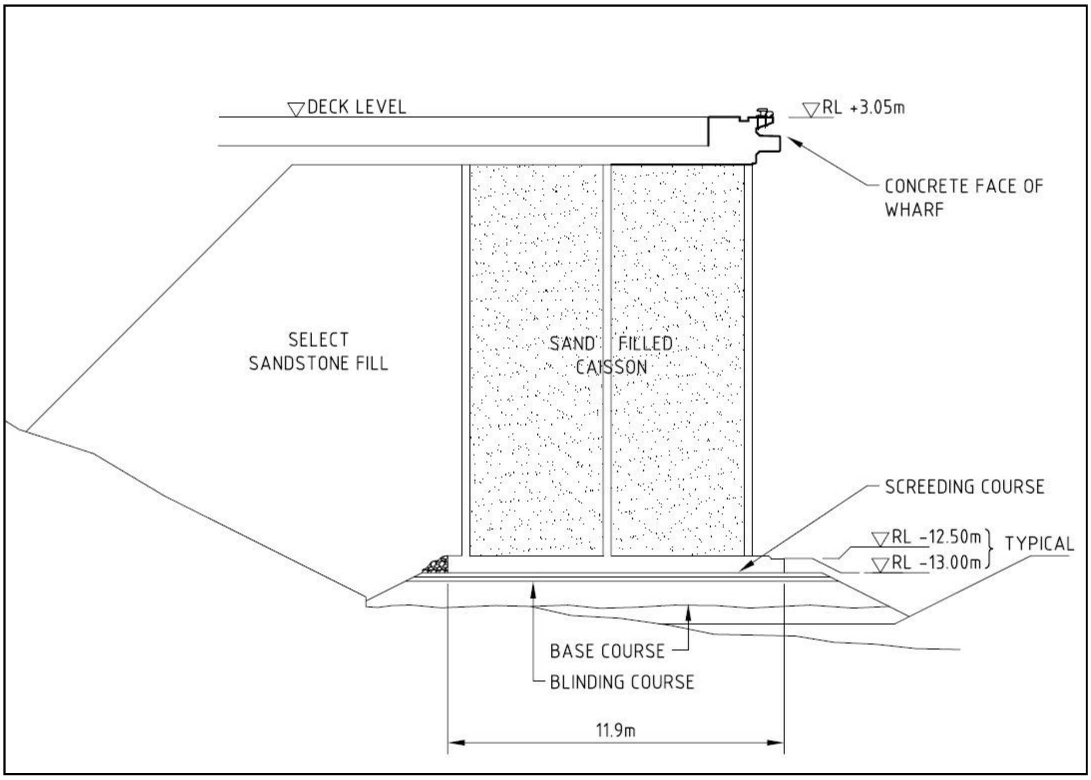

Plate 2 – Typical Section of Wharves 3, 4, 5 & 8 (SKM August 2005)

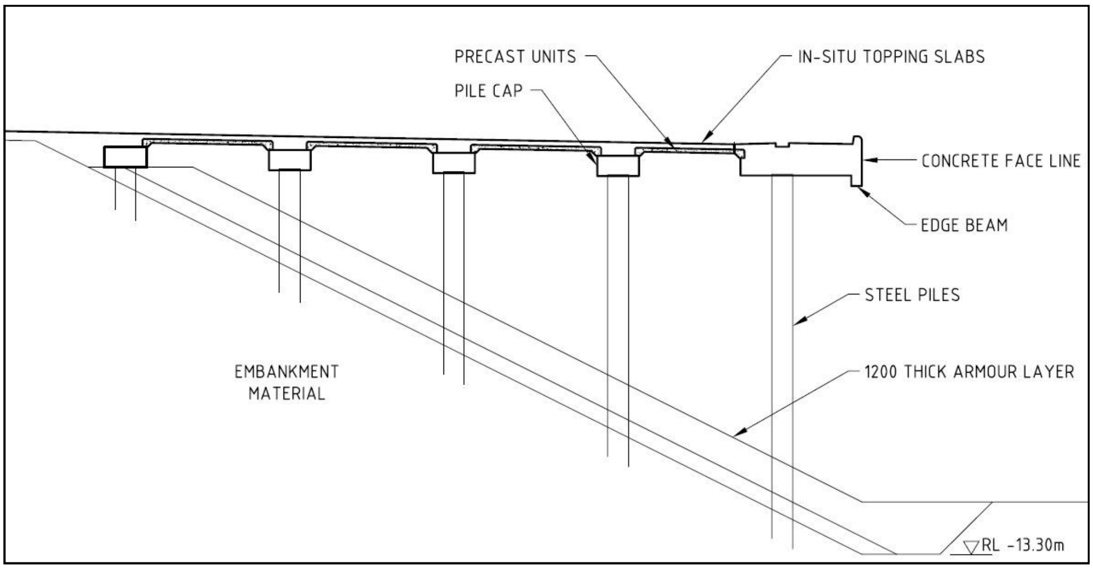

Plate 3 – Typical Section of Wharf 7 (SKM August 2005)

The SKM report does not contain information about the form of sea wall that was used to support the northern and southern side of the former dock at Wharf 7; the last of the docks to be infilled. Aerial photography from 1961 (Figure 4) clearly shows the reclamation of the area to the south of Wharf 7. Evidence of caisson units similar to those used to support Wharves 3, 4, 5 and 8 can be seen in the photograph. Similarly, aerial photography from 1970 (Figure 4) shows the reclamation of the area to the north of Wharf 7. Again, similar caisson units are clearly visible along the north extent of the former dock.

On the basis of the aerial photography, it is envisaged that the northern and southern sides of the former dock were formed using caisson units similar to those used to support Wharves 3, 4, 5 and 8.

In addition, photography from August 1982 (not published herein) shows 2 ships berthed in the former dock, one berthed along each of the northern and southern sides of the dock. This suggests that it was dredged to a depth similar to that of the other wharves.

3.3 Existing Foundations

It is likely that remnant foundations and other underground elements remain in-situ from many of these former structures including; wharf structures, historic dock walls, building foundations etc. . Reference should be made to the Non Indigenous Archaeological Assessment[7[10]] (Casey and Lowe, May 2010) in which historical maps document the development of the Barangaroo site up to the present day. The historical maps could be used to provide an indication of the location of potential obstructions for further investigation or to inform construction activities.

The exact source and nature of the fill used in the reclamation of the site is unknown. Significant construction activities ongoing in Sydney around the time of the reclamation include the construction of the Eastern Suburbs Railway, Australia Square and Centre Point. Although it could be anticipated that the majority of the fill was derived from excavation in Hawkesbury Sandstone associated with construction, it is anticipated that little control was placed on the material or placement and the presence of foreign matter such as demolition rubble, construction waste, refuse and other undesirable material should be anticipated.

3.4 Future Metro

The CBD Metro was approved on Jan 1, 2010, and subsequently cancelled by the NSW Government. However, the corridor of the proposed CBD Metro is required to be respected.

The corridor of the proposed CBD Metro runs beneath the southern end of the site. The metro will run in twin tunnels which, given the ground conditions beneath the site, are likely to be constructed using a Tunnel Boring Machine with segmental concrete lining. The tunnels of the future metro are currently approximately 6.5m diameter (excavated profile) with a variable separation between tunnels. The alignment to the east of the site lies beneath Margaret Street. As the tunnels pass beneath the site they curve towards the south. The proposed rails currently enter the eastern side of the site at approximately -15 to -16mAHD and exit the western side of the site at approximately -22mAHD before diving deeper under the palaeochannel beneath Darling Harbour. The proposed location of the Wynyard Barangaroo Underground Station is understood to be located beneath Margaret Street between Kent and Sussex Streets although the western end of the station (being the Station Box and Station Service Building Box) is understood to extend beneath the eastern edge of the site. The horizontal and vertical alignments of the metro tunnels are illustrated in Figure 14.

The tunnels will run beneath the proposed basement for Barangaroo South. It is typical that the owner of such an alignment will impose some restrictions on the founding of structures above and around the tunnels. For the CBD Metro, these zones are referred to as the 1st

10 Casey & Lowe, May 2010. Archaeological Assessment, Barangaroo, Stage 1, Draft

and 2nd Reserve Zones. A typical cross section of these zones is illustrated in Figure 14 in respect to the running tunnels and the station precinct. The restrictions that apply to each of these zones is summarised in Table 2.

Protection Zone		Construction Activities	Condition Guidelines
1st Reserve	Inside Protection Zone	NA	Construction and excavations not permitted to directly encroach upon Protection Zone except where it can be demonstrated to the satisfaction of Sydney Metro that the encroachment will not have unacceptable structural or operational impacts on the metro corridor.
	Outside Protection Zone	Surface excavation	Engineering assessment required from developer where surface excavations are proposed directly above station caverns and crossover caverns.
		Foundations, underground excavation, geotechnical investigation, ground anchor, directional drilling & demolition	Developers must demonstrate through an engineering assessment that loading from shallow foundations will not adversely impact the future metro.
		Surface excavation	An engineering assessment is generally required except where surface excavations are less than 2m in depth and excavations are not for the purpose of development of load bearing footings.
2nd Reserve		Foundations	Engineering assessment is not required if calculated bearing pressures are less that 150kPa for shallow footings and strip footings which are less than 3m by 3m in plan. For all other shallow foundations an engineering assessment is required of the developer. Engineering assessment is not required if loading from deep foundations (including shaft friction) is' transferred to below the boundary of the influence zone. Engineering assessment required from developer where the above condition is not satisfied for deep foundations.
		Underground excavation (e.g. tunnel/cavern construction), ground anchors and demolition activities	Developers must demonstrate through an engineering assessment that loading from shallow foundations will not adversely impact the future line.
		Geotechnical investigation and directional drilling	Assessment not required.

Table 2 – Development conditions around the Sydney Metro

4 Existing Information

4.1 Previous Site Investigations

There have been several site investigations carried out across the Barangaroo area. The investigations include:

- ERM. 2007. Environmental Site Assessment East Darling Harbour, Sydney, NSW. Final Report – Rev 1. Report Ref 0044432RP02 Rev01 Final;
- ERM. 2008. Draft Stage 2Remedial Action Plan for Barangaroo, Hickson Road, Sydney. Report Ref 0087036R03 Draft Rev02;
- Jeffery and Katauskas Pty Ltd (J&K). May – July, 2006. For Sydney Harbour Foreshore Authority.
- Arup Pty Ltd. Jan 1986 (Arup). For Maritime Services Board of NSW;
- Coffey Geotechnics Pty Ltd (Coffey). September 2008 and June 2009. For Sydney Metro Authority.

Figure 6 shows borehole locations from these investigations.

Additional drilling has been undertaken during the first quarter of 2010 for the purposes of contamination studies. To date, Arup has not been issued with this data. The drilling was concentrated between the northern end of the Cruise Terminal and Function Centre and the northern extent of Barangaroo South.

4.2 Relevant Site Investigations at Nearby Sites

4.2.1 Walsh Bay Investigations

During the period from 1997 to 2001, Arup Geotechnics undertook several phases of investigation over the wider Walsh Bay area to inform the redevelopment of the Walsh Bay Finger Wharves and a number of the Bond buildings on the southern side of Hickson Road; in particular Bond Store 3 and 4.

Boreholes located along Hickson Road to the north of the bay revealed that the ground is generally underlain by fill, marine deposits, residual soil and sandstone. The rockhead profile varies between -16.71mAHD and +2.7mAHD dipping from south to north. Two boreholes exhibit fractured core and closely spaced high angle jointing. These boreholes are generally along the apparent strike of the Lunar Park Fault Zone.

In addition, bathymetric survey work was completed in the harbour around and to the north of the finger wharves. This survey clearly identified a significant depression in the rock head along the apparent strike of the Lunar Park Fault Zone.

4.2.2 Bond Store 1, Walsh Bay

The work carried out was part of the redevelopment of the Bond Store 1 site into a new seven storey commercial building. The development includes a four level basement excavated in Sandstone and associated shallow foundations.

The investigation was completed in a number of phases. In total, the investigation comprised the drilling of 7 boreholes, including 2 inclined holes, 4 coreholes through the southern masonry retaining wall and 7 test pits within the site.

The investigation concluded that the site was underlain by fill, residual soil and sandstone. The rockhead profile varies between -1.25mAHD and +1.7mAHD. Inclined boreholes were drilled to investigate the presence and proximity of the Lunar Park Fault Zone. No evidence of the fault zone was found within the site or within a distance that would impact of the development.

It is noted for completeness, that the Lunar Park Fault Zone was encountered along the eastern face of the Towns Place excavation immediately to the west of the site.

4.2.3 30 The Bond

30 The Bond is located at 30 – 38 Hickson Road and comprises commercial and residential buildings. Several investigations have been carried out on this site including:

- October 1999, Woodward Clyde. Three boreholes cored to approximately 12m bgl for the purpose of assessing the sandstone material for heritage Yellow Block use.
- October 1999. Woodward Clyde. A desk study based on data from previous investigations by Hyder, Axis, LHO and Woodward Clyde. This assessment was principally targeted at contamination issues with only indirect reference to geotechnical conditions.
- February 2002. URS. Two monitoring wells were installed for contamination and hydraulic conductivity testing. In conjunction with this, investigations were also undertaken to investigate a buried tar tank (one hole) close the western boundary and to determine the depth to bedrock at the northern end of the property.
- April 2002. Coffey Geosciences Pty Ltd. A desk study based on reports by Woodward Clyde, URS, and Coffey. This report included information from previous investigations of 127 Kent Street (Esso House Extension) and 189-193 Kent Street. Face mapping of adjacent exposed rock was also undertaken.

The findings of the reports indicate that the site is generally underlain by a shallow layer of fill overlying sandstone. Deeper areas of fill occur in areas where former gas works structures / excavations have been backfilled.

4.2.4 Sydney Metro Ground Investigations

Ground investigations were undertaken for North West and CBD Metros between July 2008 and February 2009 by Coffey Geotechnics. The investigations involved extensive geotechnical investigations between Central Station and Rozelle including:

- Borehole drilling
- Laboratory testing
- Water pressure testing
- In-situ stress testing;
- Borehole imaging; and
- Geophysical investigations.

The results of the investigations are published in a number of Coffey Geotechnics reports.

The investigation involved the drilling of a number of deep boreholes towards the southern end of the site including downhole borehole imaging, in-situ stress measurement, groundwater installations and laboratory testing. In addition, a number of overwater boreholes and CPT tests were completed in Darling Harbour to the west of the site.

Interpretation of the investigation is reported in GHD Report; Sydney Metro Authority, CBD Metro, Geotechnical Interpretative Report, August 2009.

In addition to the geotechnical investigation, marine bathymetry surveys have been completed within Darling Harbour to the west of the site.

Sydney Metro Authority has granted the Barangaroo Delivery Authority permission to use the information from the metro investigation for the purpose of the Barangaroo project.

4.3 Other Relevant Data

4.3.1 Confidential information

Arup are aware of a number of other ground investigations that have been undertaken on the site or on adjacent sites for the purpose of unrelated projects. These investigations include:

- KENS Development between Kent & Sussex Streets – Westpac building
- No 1 Shelley Street – Macquarie Bank building
- Wharves 8 & 9 Pyrmont – residential buildings

Permission has not been obtained from the owners of these sites to use these data sets for the purpose of the Barangaroo South development. As such we are unable to utilise this information for the purpose of the project. It is recommended that such permission be requested from the respective owners as the information is of significant value to the project.

5 Ground Conditions

5.1 General – ground conditions

The following sections are intended to provide an appreciation of the anticipated ground conditions and properties of the various soil and rock units encountered. It is noted that this interpretation is based on information from a range of different sources. The majority of the geological and geotechnical information that exists for the site has been obtained for the purpose of investigating the known contamination on the site and hence contains little geotechnical information. The interpretation provided below has been prepared to support the Concept Plan Amendment application and to inform the early stages of design. This report does not replace the need to undertake a detailed geotechnical investigation. The interpretation and associated recommendations will need to be revisited following completion of the detailed investigation.

5.2 Encountered Stratigraphy

The geotechnical information indicates the presence of a number of soil and rock types across the site. These materials have been categorised into soil and rock units with similar geological and engineering characteristics.

The entire site is underlain with a variable depth of fill which generally deepens in a westerly direction towards the existing sea wall. Underlying the fill is a variable depth of natural material. There is significant inconsistency in the logging of this material between the subsequent investigations which makes interpretation difficult. The majority of the material appears to be of marine or estuarine origin. However, some of the materials do display characteristics of residual soil. For the purpose of this report, all of these materials are considered to be marine / estuarine deposits. This is consistent with the depositional environment where any mantle of residual soil and weathered rock would most probably have been eroded during formation of the harbour, prior to the deposition of further sediments. This should be considered in further during the detailed ground investigations.

To obtain a thorough understanding of the variable stratigraphy across the site, the boreholes from the previous investigations (refer Section 4) were entered into a GIS model and from this data surfaces for the various rock and soil units have been created.

The GIS model has been utilised in conjunction with the geotechnical database for the site to produce a number of geological cross sections which are presented as Figure 7 to Figure 11

A general description of the ground conditions encountered across the site is presented in Table 3.

Unit	Description	Gauged Thickness (m)	Typical Distribution
1	Fill : concrete and asphalt, granular material, comprising sandy GRAVELS, gravelly SANDS, sand, clayey SILT, crushed rock, building rubble, re-worked marine sediments	2.4m to 21m	Increasing in thickness in a westerly direction towards the existing sea wall.
2	Marine / Estuarine Sediments: silty CLAY, sandy CLAY, silt, and clayey SILT, and clayey SAND	0m to 14.2m	Thickening in sequence to the west with the maximum thickness west of centre. Notably absent to the north east of the site.

Unit	Description	Gauged Thickness (m)	Typical Distribution
3	Hawkesbury SANDSTONE: medium to coarse grained with layers of SILTSTONE and laminite	Unproven.	Bedrock shallow along the eastern boundary and generally increases in depth towards the west. Distinct northwest trending trough across the centre and south of the site. Bedrock drops steeply to the west towards the north of the site.

Table 3 – Summary of Stratigraphy.

The GIS model in conjunction with the geotechnical database has been used to create contoured surfaces of the following transitions:

- The base of the fill / top of the marine/ estuarine sediments – refer Figure 12
- The base of the marine/ estuarine sediments / top of the sandstone – refer Figure 13

5.3 Fill

5.3.1 Description and classification

The whole of the site is capped with a layer of asphalt and/ or concrete which is typically between 25mm and 200mm thick.

The fill beneath the site comprises a highly variable material placed during successive stages of reclamation and development. The fill generally comprises granular material such as crushed rock, sand and gravel. Occurrences of decayed organic matter, igneous gravel, furnace clinker, brick, glass, ash, tile, charcoal, steel and timber are reported, as well as sandstone boulders and concrete. Infrequent occurrences of clay and silt are noted.

During the J&K investigation concrete layers were encountered up to 0.5m in thickness in a limited number of the boreholes, below which sample recovery was not possible. It is considered that the boundary between the fill and underlying strata may vary from that indicated on the logs. The 1986 Arup investigation boreholes for the upgrade of Wharves 7 and 8, suggest a zone where the granular fill material has mixed with the underlying alluvium, however this is not indicated on the J&K borehole logs.

The concrete obstructions encountered in the J&K boreholes and a timber obstruction, 0.5m in thickness in Arup BH7 may represent former structures buried during the reclamation.

Directly behind the caissons that form the sea walls of wharves 3, 4, 5, and 8, an area of select sandstone fill is expected. The select fill forms a block approximately 6m wide before sloping downwards at a gradient of 1:1 (refer Plate 2).

The colour of the fill varies from black and tar stained through to white and orange.

The fill is generally moist, which is to be expected given the site's proximity to the harbour.

The consistency of the fill is also highly variable; reported as loose to dense, however no in situ testing was carried out with depth in the J&K boreholes and no Standard Penetration Test (SPT) results were reported on the ERM boreholes. Based on the material descriptions, numerous loose zones are present within the fill profile, with no discernible pattern. However, the loose nature of the material may be a product of the drilling method used in the J&K boreholes. The boreholes drilled as part of the 1986 Arup investigation suggest that the fill material is generally very loose to loose in density with 4 of the boreholes suggesting loose fill for the full depth and the remaining boreholes suggesting medium dense fill near surface becoming loose with depth.

SPT N values recorded as part of the Arup investigation within the fill generally ranged between 4 to 12, and where refusal was recorded this would reflect the sandstone ,brick or concrete fill within the material. Within the Coffey boreholes, SPT N values within the fill ranged between 2 to 32 and refusal.

5.3.2 Engineering properties

Preliminary engineering properties for the fill are provided in Table 4. It is noted that fill material is variable and uncertainties exist on the method and control of fill placement during the reclamation process.

Property	Fill
γ_{sat} [kN/m^3]	18
ϕ' [°]	28 (26 for loose materials)
c′ [kPa]	0
E′ [kPa]	10,000
ν'	0.3
Hydraulic Conductivity [m/day]	10^{-1} to 10^{-3}

Table 4 – Preliminary engineering properties for fill

5.4 Marine / Estuarine Sediments

5.4.1 General – marine / estuarine sediments

As discussed in Section 5.2 there is significant inconsistency in the logging between the various investigations, which makes it difficult to distinguish between materials of marine or estuarine origin and those that may be residual soil. For the purpose of this report, these materials are treated as a common unit. This interpretation may change once further information is available from the detailed investigation. It is important that the future investigation and testing regime (in-situ and laboratory) address this inconsistency as it may have a significant impact on the design.

5.4.2 Description and classification

The marine/estuarine deposits typically comprise either:

- Clayey or silty fine to coarse SAND, generally dark grey to grey and orange brown, with clay bands, trace organic matter, occasional ironstone gravel and some shell fragments; or
- Silty and sandy CLAY, generally low to medium plasticity, light to dark grey and red brown mottled grey with moisture contents above the plastic limit. Trace shell fragments noted. A peaty clay was encountered in two of the Arup boreholes varying in thickness between 1m to 1.5m and described as stiff in BH4.

Within the J&K boreholes, no insitu testing (SPT) was carried out within the marine/estuarine deposits and based on visual material descriptions noted on the ERM borehole logs, the granular material is described as loose to dense. SPT N values within the granular material recorded in the Arup boreholes ranged between 3 to 46 and within the Coffey boreholes within the site boundary ranged between 4 and 8.

The cohesive material has been described as ranging between very soft to stiff in consistency. An SPT N value of 2 was recorded within one of the Coffey boreholes.

The marine/estuarine sediments generally thicken towards the west of the site with thicker deposits located towards the centre and west of centre of the site and towards the south of the site.

5.4.3 Engineering properties

Preliminary engineering properties have been provided for both cohesive and non-cohesive materials. Given the variability in these materials, the designer should make their own assessment of the appropriate material behaviour given the ground conditions and the design under consideration. Further delineation and classification of these materials will be provided following the detailed investigation.

Preliminary engineering properties for the fill are provided in Table 5

Property	Marine Clay	Marine Sand
γ_{sat} [kN/m^3]	18	18
ϕ' [°]	27	30
c′ [kPa]	0	0
E′ [kPa]	5000	10000
ν'	0.3	0.3
c_u [kPa]	20 to 30	N/A
E_u [kPa]	6000	N/A
ν_u	0.5	N/A
Hydraulic Conductivity [m/day]	10^{-5} to 10^{-7}	10^{-1} to 10^{-3}

Table 5 – Preliminary engineering properties for marine sediments

5.5 Sandstone

5.5.1 Description

The Hawkesbury Sandstone comprises a quartz sandstone with minor shale lenses. Two sandstone facies have been identified; a massive facies and a sheet facies. Both will exist below the site. The sandstone is variably weathered and ranges from extremely weathered to slightly weathered and fresh. The depth of weathering is likely to extend at least 10m below the top of rock.

The contours of the surface of the Hawkesbury Sandstone (refer Figure 13) illustrate three troughs in the bedrock surface. At the north of the site a NE-SW trending trough is very likely to be an expression of the Lunar Park Fault. In the centre of the site an E-W trending trough is likely to be an expression of the Pittman LIV dyke. At the southern end of the site a NW-SE trending bedrock trough is not associated with any mapped geological structure. However, its trend is typical of many dykes in the Sydney CBD.

5.5.2 Classification

The sandstone has been classified in accordance with the classification system recommended by Pells[11] in order to obtain an appreciation of the nature and consistency of the rock mass. The classification is included on the geological cross section presented as Figure 7 to Figure 11.

The sandstone encountered in the investigations typically comprises orange brown to grey mottled brown red becoming light grey to grey, fine to medium massive sandstone.

In general, based on the Pells classification, about 0.5m to 5m of Class V and, or Class IV sandstone was found to overlie generally Class III sandstone. At depth the sandstone is predicted to be Class III or better. It should be noted that not all the J&K boreholes were cored, and where cored the sandstone was generally penetrated to between 1.6m to 5.9m.

11 *Foundations on Sandstone and Shale in the Sydney Region, Pells, Mostyn and Walker, Australian Geomechanics – December 1998.*

Interpretation of the Coffey boreholes suggest Class II sandstone to occur with depth, generally 2 to 3m below the upper surface of the bedrock.

In the majority of the cored boreholes, some sub horizontal and trace sub vertical joints were noted within about 3m of the upper surface of the bedrock.

5.5.3 Structure

Down hole imaging (RAXX) was carried out within 3 of the CBD boreholes drilled within the south of the site. Defect dip orientations and magnitude were recorded in addition to defect interpretation.

The log for BH214[12] indicates a Class III sandstone zone within 3m from the upper surface of the sandstone. Numerous joints were recorded within this zone based on the core description. The RAXX imaging indicates 2 shear zones within this section of the core with dip magnitudes of 9 to 15 degrees and dip directions to the southwest (196 to 267degrees) and northwest (334 degrees). The thicknesses of the shear zones were reported to range between 52mm to 145mm. Three joints were recorded in the imaging with dip magnitudes between 22 to 49 degrees generally to the south west (237 to 266 degrees). Remaining defects with depth were recorded as partings and occasional joints.

The log for BH215[12] indicates 1.5m of Class III sandstone below the upper surface of the bedrock. Seams and partings were recorded on the core description. The RAXX imaging indicates 3 joints within this zone with dip magnitudes ranging between 10 to 24 degrees with varying directions (east, south and west). A sub vertical joint was recorded dipping to the north west at 31m depth. Two shear zones with dip magnitudes of 3 and 11 degrees and dip directions generally to the south west were recorded between 19m to 24m. Remaining defects include partings and bedding laminations.

The log for BH216[12] indicates 2.5m of Class IV sandstone below the upper surface of the bedrock. Seams and partings were recorded on the core description. The RAXX imaging indicates that the majority of the defects are partings and bedding laminae. One shear zone is recorded at 13.6m with a dip magnitude of 11 degrees to the north east. Three shear zones are recorded between 28m to 32.5m with dip magnitudes between 11 to 23 degrees and directions to the north east (31 to 56 degrees). Shear zone thicknesses of between 14mm to 60mm were recorded. Subvertical joints dipping to the west were recorded below 35m

5.5.4 In-situ stress field

High in-situ stress fields in a rock mass can result in stress relief movements during excavation. Stress relief movements in the order of 0.5 – 2mm per metre of excavation can be expected during deep excavations in Hawkesbury Sandstone due to this effect. Scholey & Speechley reported significant stress relief movements during excavation on the KENS site between Kent and Sussex Streets. Movement of up to 30mm occurred in the walls of the excavation and apparent floor heave lead to the opening of bedding planes.

The presence of high lateral in-situ stresses in the Hawkesbury Sandstone is well documented by authors such as Pells (1985 and 1990), Enever (1990 and 1999) and McQueen (2004). Whilst there are a number of postulated models for the in-situ stress field beneath Sydney, it must be recognised that there are a number of features that may lead to local variations; including:

- Topographical influences including stress relief due to cliff lines and the concentration of stress beneath valleys and palaeochannels;
- The presence of geological features such as faults and dykes may cause local concentrations or local relief;

[12] Coffey Geotechnics. 2009. CBD Metro Contract 136 – Geotechnical Data Report. Central Station to Rozelle. Report Ref GEOTLCOV23558AB-AG dated 27 February 2009.

- The presence of weak layers and competent layers may lead to local reductions and local concentrations respectively; and
- Previous excavations may have resulted in localised stress relief.

Two of the postulated models for the horizontal stress field in the Sydney Basin are as follows:

Enever, 1999, suggested the following model for in-situ stresses to a depth of 200m:

- Major Horizontal Stress (σ_H) = 2.5xσ_V, orientation 28° magnetic (40° true, NNE)
- Minor Horizontal Stress (σ_h) = 1.5xσ_V, orientation 118° magnetic (130° true, ESE)

McQueen, 2004, suggested the following model for upper limit in-situ stresses:

- σ_H = 2.5MPa + σ_v down to 20 m depth, then σ_H = 6.5MPa + σ_v down to 200 m depth
- σ_h = 2 MPa + σ_v down to 20 m depth, then σ_h = 4.5MPa + σ_v down to 200 m depth

Stress testing carried out as part of the CBD Metro indicates that in general the principal horizontal stress direction (σ_H) measured is generally aligned north to north east. Stress testing was conducted in BH2103 34 towards the south east of the site on Napoleon Street (near the intersection with Kent Street). Four tests were conducted at reduced levels between -17.1m to -30.3m AHD. Two of the tests recorded the principal stress direction to the north but the remaining two results recorded stress directions varying from south west to north west.

Based on all stress tests conducted, an increase of σ_H with depth of 0.17MPa/m and an increase of minor principal stress (σ_h) with depth of 0.1MPa/m. was suggested by Coffey Mining. The results generally trend towards the upper bound model suggested by McQueen (2004) described above.

It is recommended that movement assessments are undertaken on the basis of the upper bound stress fields postulated by McQueen. The use of this upper bound stress field is considered consistent with the topographic location of the site and the possible concentration of the stress field beneath the Hickson Road cliff line and the palaeochannel beneath Darling Harbour.

5.5.5 Engineering parameters

Geotechnical parameters for the purpose of retaining structure design are presented in Table 6.

Material	γ' (kN/m^3)	ϕ' (°)	c' (kPa)
Class V sandstone	22	35	15
Class IV sandstone	24	40	30
Class III sandstone	24	50	100
Class II sandstone	24	NA	NA

Table 6 – Preliminary engineering parameter for Sandstone

Recommended allowable bearing pressures for shallow and piled foundations and allowable shaft friction for piled foundations are presented in Table 7 below for the various sandstone rock classes in accordance with Pells et al (1998). The fill material and marine/estuarine sediments are not considered suitable founding stratums.

Material	Allowable Bearing Pressure (MPa)[1]	Ultimate Bearing Pressure (MPa)[2]	Allowable Shaft Friction (kPa)[3]	Ultimate Shaft Friction (kPa)[3]	Youngs Modulus E (MPa)
Class V sandstone	1	3	100	150	100
Class IV sandstone	2.5	9.5	250	500	400
Class III sandstone	4.5	30	450	1100	750
Class II sandstone	9	60	900	2250	1500

[1] Allowable end bearing pressure for settlement of <1% of minimum pile diameter

[2] Ultimate end bearing pressure for settlement of >5% of minimum pile diameter

[3] Socket to be cleaned to roughness category R2 or better

Table 7 – Preliminary foundation design parameters for Sandstone

For foundation purposes the rock class may require reclassification to allow for seams within the defined zone of influence for the chosen foundation type. For pad footings, the zone of influence is defined as 1.5 times the least footing dimension. For piled foundations, the zone includes the length of the socket plus a further depth equal to two times the pile diameter.

5.6 Groundwater

Groundwater across the site is heavily influenced by tidal fluctuations of the adjacent Darling Harbour.

Work done by ERM in 2008[13] monitored groundwater levels across the site found that at high tide, groundwater flowed inland at a gradient of 0.003, and at low tide towards the harbour at a gradient of 0.006.

In monitoring wells MW206 and MW209 (located on the eastern border of Barangaroo South), groundwater was observed to be flowing eastwards, and a high hydraulic conductivity in this area is indicative of direct hydraulic connection with Darling Harbour.

At high tide, groundwater velocities were found to range from 3.2 and 28m/day inland and at low tide velocities were found to range from 6.3 to 57m/day. These values suggest that much of the site would be subject to significant seawater flushing.

Daily tidal ranges for Darling Harbour are typically 0.5m to 1.5m AHD. Sydney Ports give values for maximum high tide (Spring Tide) of 1.6m AHD, and minimum high tide (Neap Tide) of 1.3m AHD.

The exposed rock faces along Hickson Road were mapped for discontinuities and seepage. Minor seepage was noted from only one joint towards the northern end of the Barangaroo South area.

[13] *ERM, 2008.* Stage 2Remedial Action Plan for Barangaroo, Hickson Road, Sydney. Report Ref 0087036R03 Draft Rev02.

6 Engineering Advice

6.1 General

The following sections provide outline geotechnical advice that is preliminary in nature and should be reconsidered following the completion of the detailed ground investigation.

6.2 Earthworks

6.2.1 Excavation

The proposed development will comprise excavation below existing ground level for the construction of a large common basement. Based on the anticipated ground conditions, the proposed basement will be excavated in three main soil and rock conditions, namely;

- Reclamation fill: thicker deposits anticipated towards the west of the site;
- Marine/estuarine sediments; thicker deposits anticipated towards the centre of the site;
- Hawkesbury Sandstone: shallower depth to bedrock towards the east of the site.

A split level basement is proposed with a deeper basement level to the east of the site extending to approximately -25m AHD and a shallower basement level to approximately -6m AHD towards the centre and west of centre of the site. A retention system will be required to be installed prior to excavation of the basement. Retaining type solutions are considered further in Section 6.3.

Groundwater investigations completed to date have recorded groundwater at around datum and demonstrated a strong tidal influence. In addition, the anticipated permeability of the fill materials on the site suggests that groundwater flow could be significant. Water ingress during excavation will be an important construction issue. Given the hydraulic connection with Sydney Harbour, dewatering of the excavation without groundwater cut off is not considered feasible.

Various cut off systems may be feasible for the development. A slurry wall may provide an effective short and long term method of controlling groundwater inflow if the development geometry allows. The slurry wall could be founded in either the marine / estuarine sediments or the sandstone depending on final material permeability’s and inflow requirements.

It is envisaged that the excavation of the slurry wall with a long reach excavator may be required due to the presence of obstructions within the fill in the form of sandstone boulders, concrete and former foundations/structures buried during reclamation. Construction of the slurry wall with an excavator would allow for the removal of such obstructions.

On penetration of the fill, conventional diaphragm walling methods could be used to excavate within the marine/estuarine sediments and weathered sandstone. Should significant excavation be required in the sandstone to achieve the required level of cut off, a rock mill / hydrophraise could be considered.

The stability of a slurry wall excavation is generally maintained using a bentonite (or similar) slurry. On completion to depth, the slurry is either allowed to set or replaced with a less permeable material such as a bentonite / clay / cement mix to achieve higher levels of water cutoff.

Another use of the slurry trench would be to facilitate obstruction removal ahead of the construction of an embedded retaining wall.

6.2.2 Excavatability

Based on the anticipated bedrock contours, rock excavation would be required in the east and north east of the site. It is anticipated that the upper 2m to 5m of the rock may vary between very low to medium strength beyond which generally high strength (Class III or better) sandstone is anticipated. Defects within the high strength sandstone are anticipated to be limited and defects are generally widely spaced that may result in difficult excavation conditions due to the mass strength of the rock.

Excavation in the fill material, marine/estuarine sediments and Class V sandstone (UCS<1MPa) should be possible using conventional large excavation equipment (ie excavator or bulldozer). For the excavation of Class IV or better sandstone it is likely that conventional rock excavation equipment generally used in the excavation of basements in the Sydney CBD can be utilised. Selection of appropriate excavation techniques will depend partly on noise limits and vibration restrictions. Initial rock cutting carried out using rotary rock saws, will achieve a more accurate excavation line and will minimise transmission of site vibration to adjacent structures / receivers. Excavation may require the use of heavy ripping equipment (Caterpillar D9 Size or larger) provided there are no constraints on plant operation space and access. Detailed excavations may also require the use of a hydraulic rock breaker.

6.2.3 Rock face stability and support

Vertical excavations can usually be carried out in Class III sandstone or better. Retaining walls socketed into the upper 1m to 2m of the Class III or better sandstone may be required to retain the overlying weathered low strength sandstone. The potential presence of sub horizontal shear zones and joints noted from the borehole imaging within the upper 1.5m to 3m of the sandstone may require retention prior to excavation.

It is recommended that the exposed rock be inspected as excavation proceeds to ensure that localised permanent support (ie rock bolts or shotcrete) or barring down of unstable blocks can be carried out depending on the orientation, frequency and spacing of rock defects present in the rock mass. Weathered seams will need to be protected during excavation to prevent future weathering and instability. Inspections and mapping by a geotechnical engineer or geologist should be carried out for each 1.5m depth of excavation to ensure accessibility to problem areas and limit disruption from local failures.

6.2.4 Rock bolts

Rock bolting may be required during excavation if unfavourable defect orientations are exposed in the excavation face. Rock bolt locations would be recommended by a geotechnical engineer/engineering geologist during excavation.

The allowable bond stress will depend on the rock class exposed. The following ultimate bond stresses could be used for preliminary design purposes (based on Pells *et al*, 1998):

- Class II sandstone 2250kPa
- Class III sandstone 1100kPa
- Class IV sandstone 500kPa
- Class V sandstone 150kPa

6.2.5 Stress relief

Some lateral movement of the exposed rock faces towards the centre of the excavation is expected to occur during construction of the basement. These lateral movements will occur in response to stress relief of the surrounding rock mass brought about by the basement excavation. Movements in the order of 0.5 – 2.0mm per meter of excavation in rock can be expected. Movements are generally concentrated in high strength coherent rock units.

The location of the site towards the base of a cliff line and adjacent to a palaeochannel may result in a higher than anticipated stress field within the sandstone due to stress concentration effects. The varying stress directions recorded as part of the CBD Metro investigation adjacent to the site may reflect the varying topography and potentially concentrated stress below the site.

Given the nature of the rock mass beneath the site (generally high strength and coherent) and local evidence of a high in-situ stress regime, stress relief movements greater than the normal Sydney CBD trend could be anticipated. Generally stress relief movements occur during excavation but ongoing movements can cause impacts on basement structures. Monitoring could be considered during and following excavation to observe these movements. Monitoring may include face mapping, inclinometer monitoring and survey monitoring of deformation targets. Movements would be monitored in relation to movement predictions to enable evaluation of contingency measures such as the need for void formers between the rock face and abutting basement slabs.

6.3 Retention

6.3.1 Wall types

Where vertical excavations are proposed in the fill material, marine/estuarine sediments and sandstone Class V and IV, temporary or permanent lateral support will be required. Given the proposed basement extends below sea level, groundwater inflows are anticipated to be significant through the fill material and will therefore need to be controlled. Suitable retention systems that could be considered for these conditions include:

- Secant piled wall
- Bored pile wall with jet grout infill piles
- Diaphragm wall
- Sheet Piles

Where a groundwater cut off system is used, such as a slurry wall (refer Section 6.2.1),a more permeable form of retention system , such as a contiguous pile wall, may be appropriate.

A secant pile wall would comprise a 'hard-soft' pile arrangement with the piles socketed into Class III sandstone or better. Similarly, bored piles with jet grout infill piles would comprise 'hard' bored piles socketed into Class III sandstone or better with 'soft' jet grout piles in between socketed into Class V / IV sandstone. A diaphragm wall would comprise a continuous wall constructed in panels and socketed into Class III sandstone or better, however, due to the presence of obstructions within the fill, construction of a diaphragm wall on this site may be difficult.

A sheet pile wall option may also be considered on the Harbour side where appropriate in conjunction with cast in-situ basement retention systems. A similar wall may also be utilised to retain material to the western elevation of the car park.

It is likely that these retention systems may require temporary rock anchors / bolts to provide short term lateral support. If anchors are proposed that extend beneath adjacent properties, consent from the owners of those properties will need to be considered.

Continuous Flight Auger (CFA) piles are not considered suitable due to the obstructions recorded within the fill material, unless an obstruction location and removal exercise is completed in advance.

If the basement is to extend to, or close to, the western extent of the site, consideration could be given to the use of the existing caisson sea wall as a mass gravity retaining wall. This option would be complicated by seepage of groundwater below the base of the sea wall (constructed to a level of about -12m AHD within the alluvium) and seepage through the joints between each caisson unit. This seepage could be controlled through grouting beneath the retaining wall units and sealing of the joints between adjacent units. Methods of achieving this would require investigation.

Grouting of the marine/estuarine sediments underlying the caisson wall would limit groundwater flow beneath the units. However it is unlikely that such a grouting exercise would be 100% effective resulting in some seepage occurring beneath the wall. The design of temporary and permanent works would need to accommodate some seepage as a result.

6.3.2 Design considerations

Pile lengths will vary significantly around the basement due to the rock level contours generally falling towards the west of the site. Design rock class can be specified but during construction a geotechnical engineer could be required to assess and verify the founding material.

Jet grout infill may result in large breakouts of grout within the loose fill material and poor penetration (insufficient column diameter) in dense or stiffer materials. Validation of the column diameter and hence the watertightness of the wall is not possible during installation. This was experienced on the recent Darling Walk project and remedial work involving the application of shotcrete was required during excavation. Remedial works of this nature are unlikely to be 100% watertight especially where significant hydraulic heads exist.

Rock anchors are considered a feasible method of providing lateral restraint to retaining walls in the temporary condition. Permanent lateral support for the retaining walls should be derived from the structure. Where rock anchors are to be used, allowable bond stresses may be taken as the allowable shaft frictions provided in Section 6.2.4. The bond length for the rock anchors should generally be limited to 10m. The allowable bond stresses given in Section 6.2.4 should only be used as a guide for design purposes, and should be reviewed on the basis of in-situ load tests undertaken on anchors installed on site.

Due to the significant thickness of fill and marine/estuarine sediments to the west of the site, anchors may be required to extend through these materials and anchor into the rock. However, this may result in excessive anchor lengths. Depending on the final geometry of the basement, consideration could be given to anchoring to the existing caisson sea wall to act as a dead man anchor.

6.4 Foundations

6.4.1 General

Foundation conditions will vary considerably across the site depending on building location, presence of a basement and proposed basement depth. Dependent upon the final building / basement geometry and the foundation loads, the following foundation options are considered feasible.

6.4.2 Shallow Foundations

Where rock is present at the basement level, spread footings are considered a feasible foundation option. The recommended foundation design parameters for the various rock classes are presented in Table 7. Settlements in the order of 1% of the minimum footing dimension can be anticipated for footings founded on rock using the design parameters provided above.

All spread footings should be socketed a minimum of 0.5m into the designated rock class. A geotechnical engineer should be engaged during construction to log the proof cores (where required) and to confirm the exposed foundation material satisfies the design requirements.

6.4.3 Piled foundations

Where fill or marine/estuarine sediments are present at basement level or where foundation loads yield shallow foundations unfeasible, piled foundations to rock will be required. Bored piles may need to be cased over the depth of fill and marine/estuarine sediments to prevent collapse of this material into the shaft during construction. Alternatively, the pile bores could be constructed under support fluid such as bentonite.

Preliminary design of piled foundations should be undertaken using the foundation design parameters presented in Table 7. Piles should be designed with a nominal embedment into the design class of rock of 0.5m to develop the full design end bearing.

Rock socket design should utilise the parameters presented in Table 7 in conjunction with a design method that allows for stress distribution between the socket and the base. The geotechnical reduction factor (ϕ_g) should be adopted from AS2159 in accordance with the design case being considered and the quantum of available ground investigation information. Given the current level of ground investigation information, including limited information for the sandstone, a geotechnical reduction factor of ϕ_g = 0.40 is considered appropriate.

The information provided above is appropriate for preliminary design. The recommendations, including refinement of the geotechnical reduction factor (ϕ_g) will be provided following completion of the detailed investigation.

6.5 Hydrogeology

6.5.1 Material permeability

Hydraulic conductivities for the fill material and marine/estuarine sediments have been provided in Sections 5.3 and 5.4. The fill material is considered to have a variable permeability, but due to the granular nature of the fill the permeability would be expected to be high.

Potentially reduced permeability may be associated with the marine/estuarine sediments due to the presence of clay and sand beds. It is recommended that further in-situ and laboratory testing is conducted within the marine/estuarine sediments. The evaluation of the permeability of the material will inform design decisions relating to seepage paths and cut off requirements for the basement construction and the use of drained verses tanked basement structures.

In-situ permeability tests conducted in the sandstone as part of the CBD Metro investigation in close proximity to the Barangaroo South development recorded average Lugeon values of under 10uL but values as high as 90uL were recorded. The topography surrounding the site may suggest groundwater flow towards the west from the elevated areas of the CBD to the east of the site. It is anticipated that groundwater flow within the sandstone would occur along bedding planes and larger continuous defects within the rock mass.

6.5.2 Basement inflows

Basement groundwater inflows will be dependent on:

- The groundwater cut off solution adopted;
- Depth of basement excavation;
- Groundwater flow regime through the soils and sandstone;
- Number of defects within the exposed rock mass.

Detailed estimates of groundwater inflows will be developed as the design progresses.

The need for tanking of the basement structure needs to be decided in parallel with the wider scheme for groundwater control. If a cutoff wall is installed for example, drained basement slabs may be appropriate. Where a cutoff is not provided, tanked structures will most probably be needed, at least to some distance into the rock.

6.5.3 Impact on hydrogeological regime

The overall long term impact on the hydrogeological regime is expected to be minimal. It is likely that the basement will be formed using either a cutoff wall to control flow from the harbour or using tanked (watertight) basement walls. Either option will result in limited flow of water into the basement from the fill and soil layers.

In the areas of deep basement, excavation in the rock is likely to be undrained below the base of any retention structure (typically socketed into Class III Sandstone) resulting in minor ongoing seepage into the basement accompanied by possible treatment and disposal. Given the general low mass permeability of the Hawkesbury Sandstone ($2x10^{-9}$ to 2×10^{-10}m/s) the inflow volumes can be expected to be small and to have only a minor impact on the wider groundwater regime. This option of drained deeper basements is typical of construction practice in Sydney, including many basements in close proximity to Sydney Harbour.

6.6 Seismicity

6.6.1 Site seismicity

The Earthquake Design Category (EDC) for each of the buildings should be determined in accordance with AS/NZS 1170.0–2002 and 1170.4–2007.The determination of the EDC is related to the following components.

The Hazard Factor (*Z*) and the Probability Factor (k_p) are specific to the location and type of building being designed and are not influenced by geotechnical conditions. Empirical data is presented in AS/NZS 1170.4–2007 for annual probability of exceedance and hazard factors determined by area (for example Sydney is given a hazard factor of 0.08).

The Site Soil Sub Class is related to the stratigraphy and nature of materials beneath a site. From this perspective, each of the individual buildings should be assessed individually due to the varying soil profile underlying the site. The sub soil class is anticipated to vary depending on the position of the structure within the site. For example, structures located towards the north east of the site would be underlain by shallow rock and a Site Sub-Soil Class of B_e or C_e may be appropriate depending on the rock level relative to the foundation level. In contrast, structures located outside of the basement may be underlain by significant depths of soft or loose materials resulting in a Site Sub-Soil Class of D_e or E_e.

The structure height also forms part of the EDC evaluation, and based on the structure location and type within the site, a varying EDC could be calculated.

It is recommended that each structure is evaluated individually once its form and position is finalised rather than adoption of a site wide seismic classification.

6.6.2 Liquefaction

Based on the limited in situ testing completed to date and material observations there are zones within the fill and marine / estuarine deposits that are described as granular, loose and saturated. It is considered that liquefaction is a potential risk in these zones during a seismic event. Further testing is recommended to assess density and classification of these materials as the drilling methods utilised during the existing investigations may have loosened the material leading to inaccurate assessments. Investigation using CPT methods will provide the information required for this assessment. It is recognised that investigations using CPT methods may be problematic due to the variable nature of the fill and presence of obstructions.

The impact of potential liquefaction on temporary and permanent structures will require detailed assessment following completion of the detailed ground investigation.

6.7 Soil and groundwater aggressivity

6.7.1 Soil aggressivity

Soil and groundwater aggressivity assessment for concrete and steel piles is generally carried out in accordance with AS3600-2001 'Concrete Structure' and AS2159-2009 'Piling Design and Installation'. Both standards require evaluation against soil pH, sulphate and chloride content in addition to location relative to sea water. Sulphate screening was undertaken by ERM,(2007)[14] with concentrations in soil varying between no trace and an isolated very high concentration of 31,500mg/kg. It is noted that no reference was provided as to the test method so direct comparison with AS 2159 and AS 3600 should be undertaken with caution. No reference to testing for Chloride or pH of the soil or soil/water extract is made.

A high potential of acid sulphate soil is associated with the estuarine sediments in Sydney Harbour and it is anticipated that the estuarine sediments underlying the site could have acid sulphate soil potential. Actual acid sulphate conditions only occur when the materials are exposed and allowed to oxidise. Where these sediments are to be exposed in excavations, aggressive conditions are likely to result. Durability design will need to assess this issue carefully.

Based on the variable nature of the soils beneath the site and the known presence of contaminants it is anticipated that aggressive conditions are likely to occur. In addition, given the proximity to the Sydney Harbour and the known high hydraulic conductivity of the granular fill materials, it is anticipated that exposure categories of Moderate to Severe in accordance with AS 2159 are appropriate and B2 and C in accordance AS 3600 are appropriate.

6.7.2 Groundwater aggressivity

Sulphate concentrations of between 26mg/L to 3240mg/L, and pH ranging from 6.2 to 9.9 were recorded during the ERM investigation.

When compared against the assessment criteria in AS3600-2001, the surface and exposure environment of B2 and C is appropriate depending on proximity to the harbour edge.

When compared assessed in accordance with AS2159-2009, considering the highly permeable soil condition and proximity to the harbour an exposure classification of moderate to severe is appropriate. Classification against pH suggests a mild exposure classification in accordance with AS2159-2009 which is considered un-conservative in this environment.

6.7.3 Further testing

Further detailed testing in accordance with the requirements of AS2159 and AS3600 will be required during the detailed ground investigation to confirm the exposure classification and durability design requirements.

6.8 Acid Sulphate Soils

A total of five samples were analysed for Suspension Peroxide Oxidation Combined Acidity & Sulfur (sPOCAS) by AECOM to evaluate the potential for potential or actual acid sulfate soils to be present. The reported results indicate that Actual Acid Sulfate (AAS) soils were not present in the samples analysed, however potential acid sulfate soils (PASS) may be present at depth within the marine / estuarine deposits.

Based on Table 4.4 of the ASSMAC Assessment Guidelines, the reported sulfur trail (% sulfur oxidisable) and Acid trail (mol H^+/tonne) exceeds the Action criteria (0.03% and 18 mol H^+/tonne respectively) if more than 1000 tonnes of soils are to be disturbed. This triggers the need for an Acid Sulfate Soils Management Plan.

6.9 Landmark building

6.9.1 Maritime foundations

A Landmark building is proposed to be constructed within the Sydney Harbour to the west of the existing caisson seawall. As such it will require foundations constructed within the harbour. The Landmark Building is proposed to accommodate 33,000m^2 of GFA and will

[14] *ERM 2007 Environmental Site Assessment East Darling Harbour, Sydney, NSW. Final Report – Rev 1. Report Ref 0044432RP02 Rev01 Final.*

rise up to 45 levels above the Public Pier. A submerged basement structure beneath the Public Pier is proposed to service the Landmark Building. The submerged basement will be supported by the Landmark Building tower and Public Pier foundations as such that the basement structure does not at any point rest upon the existing harbour sea bed.

Considering the depth of harbour and the underlying stratigraphy, the foundation system is likely to be limited to bored cast in place piles, installed through steel or other liners or casings driven (or screwed) through the harbour bed into underlying bedrock. The following pile construction techniques are considered feasible:

- The construction of bored piles through permanent casings that are placed in the water and driven to rock. These could either be constructed from the existing wharf using a long reach piling rig with an incrementally constructed deck placed progressively out over the water or from a barge or jack-up platform.
- The installation of a cofferdam around the perimeter of the proposed structure that is dewatered with construction of foundations and structure taking place from a temporary platform constructed on the existing sea bed. The cofferdam could either be designed as a double skin self supporting structure or it could be strutted internally. Internal strutting will have implications on accessibility and working room.
- Where a basement is proposed that does not reach the existing seabed, piles could be constructed as described above and the casings cut down to the required level below water level. A basement structure could then be lifted in sections or floated over the foundations and sunk with an in-situ connection formed to the piles. A further option would be to construct the basement structure in situ using conventional concrete construction over floating formwork in a manner similar to the recent development of Walsh Bay. The cast in situ basement structure would be progressively sunk into final position in conjunction with the extension of walls above the waterline using conventional in situ construction techniques. Upon completion of the basement and subsequent construction of sufficient tower structure above (to act against the buoyant force of the basement), the basement would be pumped clear of water and sealed. The basement below the public pier could then be connected landside through the existing caisson seawall and sealed off using conventional submerged construction techniques.

In all cases, it is likely that bored, cast in place piles, through driven liners will be the preferred foundation option for the Landmark building. The preliminary design of these piled foundations should follow the procedure outlined in Section 6.4.3. Appropriate allowances will need to be made for exceptional lateral loading on the foundation system such as ship impact loading where required by Sydney Ports or other appropriate Authority.

6.9.2 Impact of foundation construction on the seabed

As discussed above the construction of the Landmark building will require the construction of bored piles, either through the water via driven liners or from within a cofferdam.

Construction of bored pile foundations (through driven liners) or a cofferdam formed using driven sheet piles will cause minimal local disturbance to the seabed materials during placement. Following placement of the sheet piles or casing, all excavation works would be undertaken within the casing or cofferdam and any associated disturbance to existing marine sediments or underlying softer strata would be effectively contained. Placement of pile casings is generally undertaken by craning them into position and either driving or screwing them into the softer overburden materials.

Installation of sheet piles in this environment would be typically undertaken using a vibratory rig, however, driving of sheets may also be considered necessary in some circumstances. Heavy driving of the sheets would be required if penetration was required into the underlying sandstone to aid toe support. If a cofferdam was used, the sheet piles would be extracted following completion of the structure. Again, the process of extraction causes little disturbance to the existing seabed.

It is noted that both of these techniques have been used recently in the Sydney Harbour. Sheet piles were installed around the Wharves 8 & 9 site in Pyrmont to facilitate below ground / water works. The sheet piles were installed to rock through the seabed using a vibratory rig and were extracted following completion of the works. Pile construction through casings was used to construct the foundations for the recent expansion of the Sydney Aquarium.

Silt control in the waters surrounding the site can be readily undertaken using silt boom around the construction site.

Our preliminary assessment has determined that the geotechnical challenges associated with the proposed site of the Landmark building are able to be addressed utilising proven and industry standard engineering design and construction techniques and practices and that the site of the proposed Landmark building can be made suitable for the proposed development.

6.10 Future Metro

Proposed structures located above the future tunnels shall typically be founded below the invert of the tunnels utilising bridging structures. Where required, it is proposed the foundation piles shall be sleeved above the tunnel invert level to prevent load transfer between the tunnels and the piles and to potentially isolate the structure from noise and vibration during construction of the Metro. Where appropriate, the bridging structure shall be designed to accommodate construction of the tunnels following construction of the overlying buildings.

Design considerations will include stress redistribution, changes in groundwater regime, ground deformations etc.

The general approach to the co ordination of the Sydney Metro with the LLMP proposed Concept Plan Amendment scheme is as discussed below.

- The ultimate LLMP basement design, is proposed to incorporate a continuous retention and groundwater control wall to the final perimeter of the basement (across the Barangaroo South site) in its entirety. The design detail and depth of the final arrangements of the basement perimeter retention systems will be the subject of further design development;
- In the proximity of the Barangaroo Metro Station and Service Building Box (both to be constructed within Barangaroo South by others under a separate approval), it is proposed that the basement slabs, podium ground plane slab and the above ground structures co ordinate with and will be supported by the Metro Station and Service Building Box structure prior to the LLMP development works under a separate approval;
- To the west of the Metro Station and Service Building Box, the vertical transitions in the LLMP basement structure adjacent and over the metro tunnels are proposed to be undertaken typically with conventional concrete frame construction supported on local piled foundations;
- Columns supporting the future tower closest to and above the rail corridor are proposed to pass between and be founded at a depth below the Metro tunnels where applicable. Lower rise buildings such as the future proposed tower podiums will typically be founded on piled foundations bridging the metro tunnels or passing between the tunnels as proposed above in the case of the tower. The proposed commercial building adjacent to Hickson Road, in part directly over the Station and Service Building Box, will be designed to co ordinate vertical loads to pass onto the walls of the Box (or transfers) with the approval of Sydney Metro;

- The structural arrangements of the southern tower and podium and the commercial building adjacent to Hickson Road will be the subject of further consultation with Sydney Metro Authority and be the subject of future Project Applications;
- The proposed entry and exit to the commercial and loading dock vehicular ramp structure at the intersection of the proposed extension of Margaret Street West and Shelley Street which passes over the metro rail tunnels, is proposed to be supported on piled foundations (or other suitable foundations) either side of the metro tunnels; and
- The requirements of the document Sydney Metro, CBD Metro Development Guidelines will be considered in demonstrating that the proposed works do not have adverse structural or operational impacts on the SMN-1 Metro.

On the basis of this approach, it is not anticipated that the proposed Concept Plan Amendment would impede the metro rail corridor or affect the future operations of the metro project.

7 Conclusion

This report identifies and discusses geotechnical issues relating to the Concept Plan Amendment development scheme proposed for the Barangaroo South site. The purpose of the assessment is to inform and accompany the Concept Plan Amendment Application MP06_0162 MOD 4.

The geotechnical considerations highlighted in this report are appropriate for preliminary design purposes and will require validation and refinement during further geotechnical investigation to input into detailed design. Reference should be made to Section 1.6 of this report regarding the limitations of this assessment.

Based on the understanding of the ground conditions presented herein, our conclusion is that the project presented in the proposed Concept Plan Amendment (MP06_0162 MOD4) can be designed and constructed utilising industry standard and proven design and construction techniques.