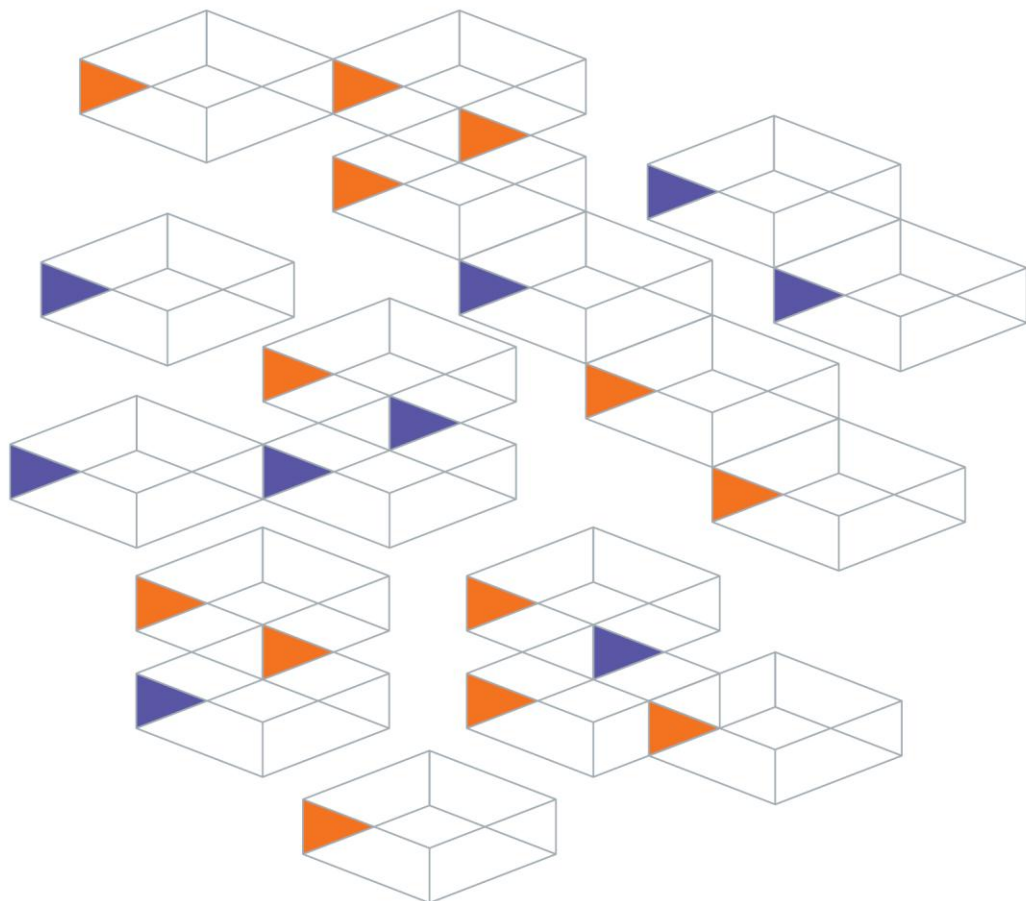


Mirvac

Harbourside Darling Harbour

Preliminary Geotechnical Assessment Report

27 September 2016



Boundaries
are set by those
who are afraid
to push them

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Harbourside Darling Harbour

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For and on behalf of Coffey



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Quality information

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1. Introduction

Coffey Geotechnics Pty Ltd (Coffey) has been engaged by Mirvac to prepare a preliminary geotechnical assessment for the proposed redevelopment of the Harbourside Shopping Centre, which is situated on the western foreshore of Darling Harbour. The location of the site is shown in Figure 1, attached.

The work was initially commissioned by Mr. Lachlan Attiwill on behalf of Mirvac and more recently by Steve McFarlane following the decision to adopt a two level basement. The commission was in response to a proposal submitted by Coffey dated 18th November 2015 (ref: GEOTLCOV25340AA-AB) and email dated 07 July 2016.

The scope of the preliminary geotechnical assessment includes general discussion and recommendations on the following geotechnical aspects:

- Preliminary geotechnical/ground model for the development site including drafted site plan showing inferred bedrock level contours for rock Classes IV and III.
- Identification and discussion of geotechnical issues and constraints for site redevelopment.
- Comment on project feasibility with respect to identified geotechnical issues.
- Initial geotechnical discussion on building footings, excavation works and retention.
- Indicative geotechnical pile capacities for various pile diameters and varying socket lengths
- Further geotechnical site investigation requirements and strategies to support detailed planning and engineering design.

2. Proposed development

This report supports a State Significant Development Application (SSDA) submitted to the Minister for Planning and Infrastructure pursuant to Part 4 of the Environmental Planning and Assessment Act 1979 (EP&A Act).

Mirvac Projects Pty Ltd (Mirvac) is seeking to secure approval to establish concept proposal details for the redevelopment of the Harbourside Shopping Centre (Harbourside), including a new retail shopping centre, residential apartment tower and substantial public domain improvements.

The project supports the realisation of the NSW State Government's vision for an expanded 'cultural ribbon' spanning from Barangaroo, around to Darling Harbour and Pyrmont. The project importantly will add further renewed diversity in tourism and entertainment facilities to reinforce Sydney's CBD being Australia's pre-eminent tourist destination.

2.1. Background

Mirvac acquired Harbourside, a key location within the Darling Harbour precinct, in November 2013. Harbourside, which was opened in 1988 as part of the Bicentennial Program, has played a key role to the success of Darling Harbour as Australia's premier gathering and entertainment precinct.

Despite its success, with an annual pedestrian visitation of around 13 million people, Harbourside is now outdated and in decline. The building lacks a quality interface to the Darling Harbour public domain and Cockle Bay and does not integrate well with the major transformation projects underway and planned for across Darling Harbour.

Harbourside is at risk of being left behind and undermining the significant investment being made in Darling Harbour that will see it return to the world stage as a destination for events and entertainment.

Accordingly, Mirvac are taking a carefully considered and staged approach to the complete revitalisation of the site and its surrounds.

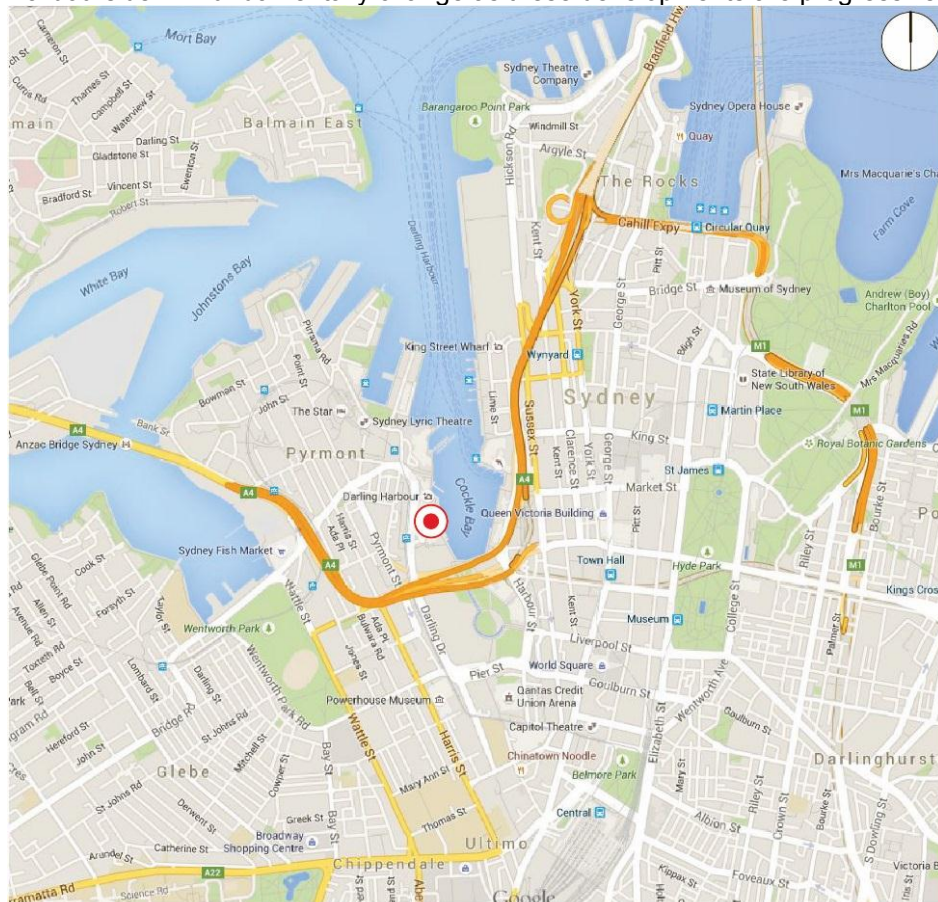
2.2. Site Description

The Site is located within Darling Harbour. Darling Harbour is a 60 hectare waterfront precinct on the south-western edge of the Sydney Central Business District that provides a mix of functions including recreational, tourist, entertainment and business.

More generally the site is bound by Pyrmont Bridge to the north, the Sydney International Convention, Exhibition and Entertainment Centre Precinct (SICEEP) to the south, Darling Drive and the alignment of the Light Rail to the west and Cockle Bay to the east.

A locational context area plan and location plan are provided at Sketch 1 below.

The Darling Harbour precinct is undergoing significant redevelopment as part of the SICEEP, Darling Square, and IMAX renewal projects. The urban, built form and public transport / pedestrian context for Harbourside will fundamentally change as these developments are progressively completed.



● The Site

Sketch 1 – Location Context Area Plan

2.3. Overview of Proposed Development

- The proposal relates to a staged development application and seeks to establish concept proposal details for the renewal and re-imagining of Harbourside.
- The concept proposal establishes the vision and planning and development framework which will be the basis for the consent authority to assess future detailed development proposals.
- The Harbourside site is to be developed for a mix of non-residential and residential uses, including retail and restaurants, residential apartments, and open space.
- The Concept Proposal seeks approval for the following key components and development parameters:
 - Demolition of existing site improvements, including the Harbourside Shopping Centre, pedestrian bridge links across Darling Drive, obsolete monorail infrastructure, and associated tree removal;
 - A network of open space areas and links generally as shown within the Public Domain Concept Proposal, to facilitate re-integration of the site into the wider urban context;
 - Building envelopes;
 - Land uses across the site, non-residential and residential uses;
 - A maximum total Gross Floor Area (GFA) across the Harbourside site of 87,000m² for mixed use development (non-residential and residential development);
 - Basement car parking;
 - Car parking rates to be utilised in subsequent detailed (Stage 2) Development Applications);
 - Urban Design and Public Realm Guidelines to guide future development and the public domain; and
 - Strategies for utilities and services provision, drainage and flooding, and ecological sustainable development.

A more detailed and comprehensive description of the proposal is contained in the Environmental Impact Statement (EIS) prepared by JBA.

2.4. Planning Approvals Strategy

The Site is located within the Darling Harbour precinct, which is identified as a State Significant Site in Schedule 2 of State Environmental Planning Policy (State and Regional Development) 2011. As the proposed development will have a capital investment exceeding \$10 million, it is declared to be State Significant Development (SSD) for the purposes of the Environmental Planning and Assessment Act 1979 (EP&A Act), with the Minister for Planning the consent authority for the project.

This State Significant Development Application (DA) is a staged development application made under section 83B of the EP&A Act. It seeks approval for the concept proposal for the entire site and its surrounds.

More specifically this staged DA includes establishing land uses, gross floor area, building envelopes, public domain concept, pedestrian and vehicle access and circulation arrangements and associated car parking provision.

Detailed development application/s (Stage 2 DAs) will accordingly follow seeking approval for the detailed design and construction of all or specific aspects of the proposal in accordance with the approved staged development application.

The Department of Planning and Environment provided the Secretary's Environmental Assessment Requirements (SEARs) to the applicant for the preparation of an Environmental Impact Statement for the proposed development on 30 August 2016. This report has been prepared having regard to the SEARs as relevant.

Information describing the proposed development is provided in Appendix A.

3. Desk study information

Coffey has existing geotechnical information in close proximity to the site and immediate environs. This includes site investigations carried out by Coffey and information gathered by others for previous projects. Borehole data from the references listed below were used to assess the geotechnical/geological conditions of the site:

- 1) Coffey & Partners Pty Ltd, "Darling Harbour Development Maritime Structures Geotechnical Investigation Zones 1 to 6" May 1985 (S7559/1-AE)
- 2) Coffey & Partners Pty Ltd, "Darling Harbour Development Project Convention Centre – Geotechnical Investigation" June 1985 (S7559/3-AD)
- 3) Arup Geotechnics, "Darling Harbour Development Western Boulevard – Site Investigation Report", December 1985
- 4) Coffey & Partners Pty Ltd, "Darling Harbour Light Monorail Geotechnical Investigation", May 1986 (S7769/1-AG)
- 5) Coffey Geosciences Pty Ltd, "Proposed Convention and Exhibition Centre", May 2003
- 6) Coffey Geotechnics Pty Ltd, "Proposed Sydney International Convention, Exhibition and Entertainment Precinct (SICEEP)", 25 May 2013 (GEOTLCOV24303AC-AD)
- 7) Coffey Geotechnics Pty Ltd, "Geotechnical Investigation Report for SSDA6, Sydney International Exhibition and Entertainment Precinct - ICC Hotel", 26 August 2013 (GEOTLCOV24303AH-AH Rev 1)

4. Geotechnical overview

4.1. Geological setting

A review of the Sydney 1:100,000 Geological Sheet (Sheet No. 9130; dated 1983) indicates the site is underlain at depth by the Hawkesbury Sandstone Formation, which is generally described as a medium to coarse grained sandstone with very minor shale and laminite lenses. Quaternary alluvial sediments comprising silty to peaty quartz sand, silt and clay and man-made fill materials are shown overlying the sandstone in the vicinity of the site.

The site is located on the western side of what was originally known as Cockle Bay, as shown on historic maps of Sydney. The former bay and its tributaries extended almost 1 km inland from the southern boundary of the existing harbour. The present-day shoreline has been formed progressively through infilling of the inland bay since the 1820s. For reference, a 1970 Parish of St Andrew survey plan is presented in Figure 2 in Appendix A, which shows the former natural coastline at Cockle Bay relative to the more recent man-made coastline (this 1970 coastline is close to the present day coastline).

Below the fill, younger back-swamp and estuarine sediments (likely Holocene in age) overlying older alluvial deposits (likely Pleistocene in age) would be anticipated, likely deposited predominantly in a south-north direction consistent with the shape of the bay, defined by an ancient palaeochannel. The thickness of the alluvial deposits is expected to increase to the north and centre of the palaeochannel, with its axis along the centre of Cockle Bay.

Due to their age difference and depositional history, Holocene and Pleistocene sediments often exhibit very different characteristics. The lower Pleistocene sediments tend to be more stiff and dense in nature and exhibit orange and brown hues owing to exposure and oxidation during falls in sea-level. The upper Holocene sediments, however, tend to be softer and looser in nature, and are typically dark grey in colour, often with organics and shell fragments. Holocene sediments were deposited within the last 10,000 years following a dramatic rise in sea-level; consequently they are typically unoxidised and often contain acid sulphate soils.

Due to the position of the site on the western side of Cockle Bay, the rock level is anticipated to step down to the east, towards the centre of Cockle Bay. Correspondingly, the overlying alluvial and estuarine sediments (and overlying man-made fill) would be anticipated to thicken to the east where bedrock levels are deeper.

The existing site is approximately on level ground. The surface elevation along Darling Drive on the west side is approximately RL 3.5 m AHD and along the wharfside pedestrian way on the east side it is at approximately RL 3 m AHD.

4.2. Preliminary ground model

Based upon the Coffey archival site investigation data, the geotechnical conditions at the proposed Harbourside redevelopment are expected to comprise the following:

- Pavement and heterogeneous fill (Unit 1), overlying
- Estuarine and alluvial sediments (Unit 2) of variable thickness, overlying
- Slopewash and residual soil (Unit 3), overlying
- Hawkesbury Sandstone bedrock (Unit 4).

The available borehole data within and immediately adjacent to the site indicate a top of bedrock level that steps down to the east, towards Darling Harbour. Bedrock is anticipated to be closest to surface adjacent to Darling Drive along the western site boundary where rock levels are anticipated at approximately RL 1 m AHD to RL 2 m AHD at the northern and southern ends of the site, and possibly deeper in the central portion of the western boundary (approximately RL -2 m AHD to RL -3 m AHD). Along the eastern site boundary, rock levels are anticipated to deepen towards the south, from approximately RL -2 m AHD in the north to possibly as deep as RL -12 m AHD to RL -13 m AHD in the far south-eastern site corner. More generally, rock levels of RL -6 m AHD to RL -8 m AHD are anticipated along the eastern site boundary.

Experience within this area suggests that the sub-horizontally bedded sandstone is likely to step down towards Darling Harbour in a series of buried cliffs and possible overhangs paralleling the natural shoreline geometry. Due to the relative lack of data, the modelled top of rock does not include possible cliff line locations. The presence of cliff lines would however be anticipated and further investigation is recommended. Furthermore, the possible presence of minor, incised palaeochannels aligned west to east cannot be discounted.

Top of bedrock contour plans have been developed for sandstone Classes IV and III and are presented in Figures 3 and 4 respectively in Appendix B. Irregularities and tightening observed in the inferred contours may be indicative of possible cliff lines, although this would need to be substantiated with further investigation. Sketch geotechnical cross sections through the site presenting the inferred sub-surface conditions are presented within Appendix B.

Reference to the 1970 Parish of St Andrew survey plan (Figure 2) indicates a former promontory from the natural coastline on the western side of Cockle Bay that would appear to coincide with the shallow rock levels observed at the south-western site corner, in the vicinity of the ICC Hotel. Embayments are shown on the 1822 map to the north and south of this promontory, which also coincide with the deeper rock levels that have been inferred in these areas.

Natural estuarine and alluvial sediments are anticipated to be present immediately above the bedrock (and associated residual soil or slopewash deposits). These natural sediments are anticipated to thicken to the east. It is possible that these sediments are absent, or at least of negligible thickness, below the northern portion of the site. Estuarine and alluvial sediments may only be present below the south-eastern portion of the site where rock levels are deepest.

Upper fill materials are assessed to be present below the majority of the site. The fill material is anticipated to increase in thickness to the east, following the natural fall of the former near shore sediments. The maximum thickness of fill is assessed to be approximately 8 m at the south-eastern boundary. Where the fill overlies natural sediments, the basal contact is likely to be highly irregular and possibly mixed with the upper surface of the underlying natural soil during placement.

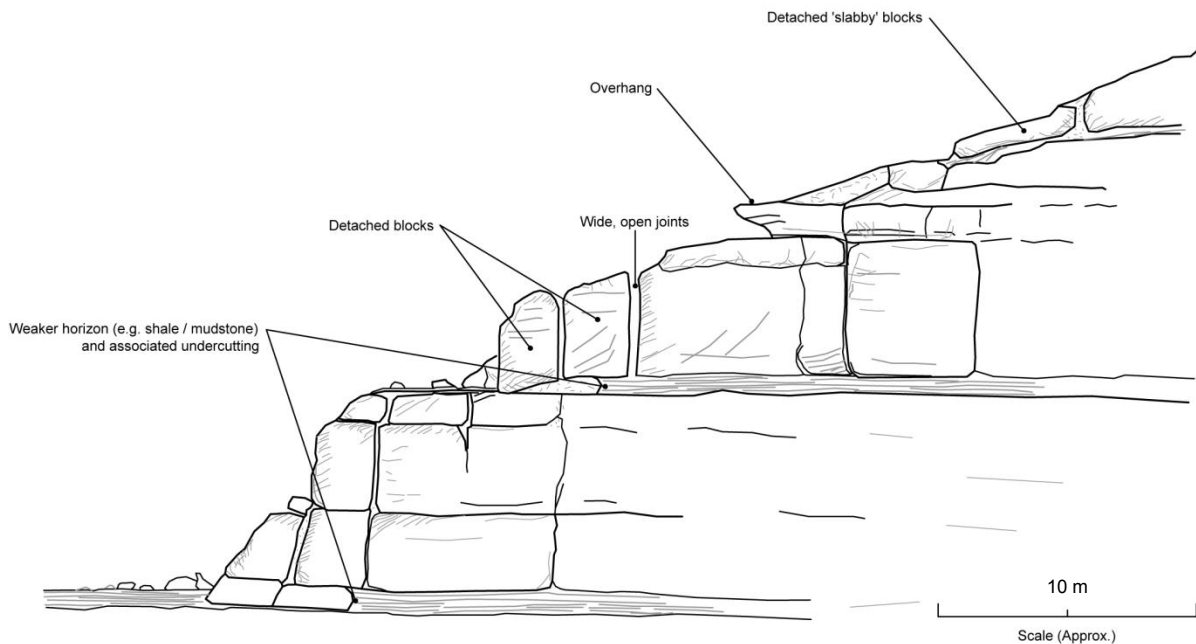
A summary preliminary ground model for the site is presented in Table 1.

Table 1 – Summary of subsurface conditions and geotechnical units

Unit	Thickness (m)	Description
1. Fill	Up to 8	<p>Surface conditions may consist of a concrete slab overlying a mixture of variably clayey sand and gravel. The fill is observed to contain sandstone and shale cobbles, concrete, coal, brick and timber fragments. The fill may contain large boulders, and timber piles.</p> <p>The base of the fill is likely to be highly irregular and has often mixed with the upper surface of the underlying natural soil (where present) during placement.</p>
2. Estuarine and Alluvial Sediments	Absent to 7	<p>Estuarine Sediments are typically dark grey to black silts and clays with sub-ordinate clayey sands. They typically contain shells, shell fragments and organic material. Organic/peaty clay horizons may be present, possibly corresponding to an area where mangrove swamps once existed.</p> <p>Alluvial Sediments comprise clayey sand with subordinate and interbedded silty clays and sandy clays. Inferred to be derived from weathered sandstone from neighbouring sandstone 'highland'.</p>
3. Residual Soil	Generally absent to less than 1	<p>Due to the erosional nature of the overlying alluvial deposits, residual soil is generally absent and where present is typically limited to less than 1 m.</p>
4. Sandstone	Insufficient data to assess thicknesses of Sandstone sub-units	<p>Close to the top of the unit, the sandstone is often of extremely low strength and is extremely to highly weathered. The unit grades to medium to high strength and fresh at greater depth. The sandstone bedrock has been sub-divided into a number of separate units based on the Pells <i>et al.</i> (1998) rock classification system as follows:</p> <p>Unit 4A – Class V Sandstone</p> <p>Unit 4B – Class IV Sandstone</p> <p>Unit 4C – Class III Sandstone</p> <p>Unit 4D – Class II or better Sandstone</p> <p>Class V Sandstone is extremely to highly weathered with extremely low to low strength, frequent zones of clay seams, highly fractured or fragmented.</p> <p>Class II Sandstone or better is generally fresh to slightly weather with medium to very high strength, slightly fractured to unbroken.</p>

4.3. Buried sandstone cliffs

Experience suggests that the sub-horizontally bedded sandstone is likely to step down towards Darling Harbour in a series of buried cliffs and possible overhangs paralleling the natural shoreline geometry (refer to Sketch 2 below).



Sketch 2 - Key characteristics of sandstone cliffs encountered in the Sydney region

The key features to note are the sub-vertical faces formed by relatively massive, more homogeneous sandstone which is separated by weaker beds (such as shales, siltstones, mudstones and weaker sandstones). These weaker beds provide breaks in the cliff faces often forming the sub-horizontal areas of the overall cliff profile. These weaker beds can also promote undercutting of the sandstone above; weathering more readily than their more resistant sandstone counterparts. The presence of these weaker horizons in boreholes within the upper bedrock profile may indicate the presence of a nearby cliff line.

The other key features of sandstone cliffs are the wide, open, sub-vertical joints at the cliff margins which may or may not be infilled with soil strength material. These joints will eventually propagate the formation of large detached sandstone blocks. The presence of these types of defects within boreholes may be an indication of a nearby cliff line.

4.4. Groundwater conditions

Data from which to develop a groundwater model for the site is extremely limited. However, groundwater levels in the fill, sediments and rock would be anticipated to be generally within the range of harbour water levels (i.e. close to RL 1 m AHD). Groundwater levels within the fill would be anticipated to vary with the tide and potentially illustrate strong hydraulic connection with the waters of Darling Harbour. Excavation below the pre-development groundwater level will require dewatering and in most cases construction of a perimeter diaphragm wall to allow construction of basements.

Natural groundwater flow would be anticipated to be eastward toward Darling Harbour. Seepage rates would be expected to be driven by rainfall infiltration and possibly leakage from stormwater services. Superimposed on this natural eastward seepage would be the anticipated variations driven by tidal movement. This may result in a back and forth movement of groundwater within the site. The

effect of the development on groundwater flow will be to reduce the rate of eastward groundwater seepage due to the restriction of lateral groundwater flow where cut off walls are installed.

5. Geotechnical considerations

We identify the following key geotechnical considerations in relation to the proposed Harbourside development based on the findings of our preliminary geotechnical assessment and experience within this area.

5.1. Excavation conditions

Review of the supplied drawings for the development indicates that construction would involve excavation of a two level basement, with B2 floor level of -3.1 m AHD indicated on the drawings. It is reasonable to expect the bulk excavation level to extend to approximately RL -4.0 m. Based on the preliminary ground model and top of rock contours (Figure 3), an excavation of this depth may encounter up to approximately 5 m to 6 m of sandstone bedrock (Unit 4) at discrete areas along the western boundary. Highly weathered to fresh sandstone (Units 4B, 4C and 4D) would likely require excavation by an excavator with rock hammers (for example, a Caterpillar 330D excavator equipped with a rock hammer). Trimming of cut faces in the lower Hawkesbury Sandstone could also be performed with rock saws.

Otherwise the basement excavation would be anticipated to be predominantly within the upper fill materials (Unit 1). It is expected that the fill materials will be able to be excavated using a hydraulic excavator or bulldozer blade and bucket. Saturated soil layers are expected to exist within the excavation, with wet and poor trafficability expected for machinery.

The use of impact hammers within the bedrock may result in vibrations that could damage adjacent structures. A rock saw could be used to reduce the lateral transfer of vibrations. Dilapidation surveys and vibration monitoring should be carried out if vibration sensitive structures lie within close proximity to excavations.

Contractors should be required to examine the engineering logs and rock cores to make their own assessment of excavation plant and production rates.

5.2. Bedrock profile

Possible cliff lines have been inferred parallel to the natural shoreline geometry. The possible presence of buried cliff lines within the upper sandstone bedrock surface requires careful consideration. The cliff lines are likely to impact on the following:

- The type of retention system and anchoring / bracing system adopted;
- Requirements for grouting; and
- Downgrading of footing end bearing pressures due to the proximity to the cliff lines.

5.3. Quality of fill and natural sediments

The fill within the site is anticipated to be variable and may contain a number of inclusions such as wood, timber, steel, sandstone boulders and other building rubble. Voids may also be present. The presence of such inclusions, and their associated voids, will have a significant impact on the constructability and choice in foundation type and water management programme.

In addition, the consistency of the fill is likely to be highly variable and the soft/loose parts may undergo settlement both during dewatering and over time. Similarly, the alluvial/estuarine sediments may contain compressible clays and organic/peaty horizons.

The alluvial/estuarine sediments may also prove to be acid sulphate bearing owing to their assumed depositional environment. Acid sulphate soils are only a problem if they are allowed to oxidise following sub-aerial exposure. Should acid sulphate soils be present, the treatment of natural sediment spoil would need to be considered.

An assessment of soil aggressivity with respect to buried steel and concrete elements for both the fill material and natural sediments should also be carried out.

5.4. Retention systems

Excavation for the proposed two level basement would certainly extend below the groundwater table. The retention system for excavation will therefore need to provide watertightness, and shall need to account for tidal variation in groundwater levels. It would need to provide a cut-off to groundwater flow through the highly permeable fill and estuarine/alluvial deposits, and to be keyed into sandstone bedrock. The following retention/cut-off options could be considered depending upon the depth to rock:

- Secant pile wall
- Diaphragm wall

Secant piles may be adopted where the rock elevation is relatively higher, for example along the western side of the development. Based on experience total pile lengths should be limited to 10 m to reduce the possibility of loss of overlapping between hard and soft piles. At this stage, in the absence of detailed site investigation information, we would propose a 1.5 m socket into Class III or better sandstone. It may be possible to re-visit the socket requirements following the site investigation works. The secant wall would require temporary lateral support at two levels in the form of anchors. Depending on the rock elevations the bottom anchor could be replaced by rock bolts as opposed to conventional strand anchors.

For areas with deeper rock elevations i.e. mostly along the eastern and southern parts of the proposed development a diaphragm wall (D Wall) solution could be developed. Due to the presence of the Sea wall and high groundwater level it is proposed to develop the D wall design with a single row of temporary anchors positioned above RL 1 m. At this stage, in the absence of detailed site investigation information, we would propose a 1.5 m socket into Class III or better sandstone. It may be possible to re-visit the socket requirements following the site investigation works.

The fill material on site is likely to be highly variable in nature and may contain sandstone cobbles and boulders. Therefore, it is recommended that specialist contractor advice is sought to confirm the feasibility of the proposed cut off wall options as well as information about the appropriate construction plant.

Appendix C provides sketches of the two options discussed above.

Where the edge of buried cliff lines are present close to the toe of the retention wall there may be a risk of reduced toe support, together with a local increase in water inflow. Potential methods of managing the impacts of the buried cliff lines within the deep cut slot area include:

- Carry out pre boring along the proposed alignment of the cut off wall to assess the depths to the top of rock and the variation in rock levels.
- Inclusion of vertical steel pipes within the wall reinforcement to provide access to the toe of the retaining wall, should subsequent grouting or toe pins be required.

- Extension of the wall units to a level below possible cliff lines. Depending upon the construction methods employed, this may have practical challenges.

5.5. Design parameters

5.5.1. Retaining wall design parameters

For the preliminary design of retention systems, the parameters in Table 2 may be adopted:

Table 2 – Preliminary retaining wall design parameters

Unit	Description	Bulk Unit Weight (kN/m ³)	Effective Cohesion (kPa)	Effective Friction Angle (degrees)	Undrained Shear Strength (kPa)	Elastic Modulus (MPa)	Poisson's Ratio
1	Fill	18 to 20	0	30	-	5 to 15	0.35
2	Alluvium	18	0	25	25	5 to 10	0.35
3	Residual Soil	20	5	25	75	60	0.35
4A	Class V Sandstone	23	20	35	250	80	0.3

Retaining walls should be designed for appropriate hydrostatic and surcharge loads.

The design of retention systems is geotechnically complex, and best carried out using soil-structure interaction analysis methods. For preliminary design purposes, we have provided some recommended earth pressure coefficients for the various units in Table 3. It must be pointed out that these values are based on empirical methods and may not provide satisfactory solutions in some cases.

Table 3 – Recommended earth pressure coefficients for preliminary design

Unit	Description	'Active' Earth Pressure Coefficient, K_a	'At Rest' Earth Pressure Coefficient, $K_0^{(1)}$
1	Fill	0.35	0.5
2	Alluvium	0.35	0.5
3	Residual Soil	0.35	0.5
4A	Class V Sandstone	0.27	0.5

⁽¹⁾Values provided assume a lateral movement of the wall of about 0.2% of the wall height is allowed to occur. *In situ* "at rest" earth pressure coefficient may be significantly higher for Units 3 and 4A.

Active earth pressure coefficients should be adopted where wall movements of about 1% of the wall height can be tolerated. At-rest pressure coefficients should be adopted where less movement can be tolerated. A well-constructed wall will still undergo movements of the order of 0.1% to 0.3% of the wall height where at-rest pressures are adopted.

Retaining walls should be designed for hydrostatic pressures unless permanent and effective drainage can be provided. Applicable surcharge loads should be added to earth pressures.

The earth pressure coefficients for Class IV or better sandstone are variable and dependent on global effects of defects. Variability also occurs due to the in situ stress environment and geometry of the excavation.

5.5.2. Foundation design parameters

Based on the site conditions, foundation options for the development will likely include piles. Due to the heterogeneity of the fill, it is considered that foundations should be extended into the weathered sandstone units.

The piles shall require casing through the fill and water charged soil profile during installation. Alternatively, Continuous Flight Auger (CFA) may be suitable and have the benefit not requiring temporary casings or support fluids.

Lightly loaded, or temporary structures with appropriate mechanisms for accommodating differential settlement, may be placed on pad footings on the fill or concrete slabs at ground surface. However, additional geotechnical investigations should be undertaken during the design of such structures. Due to the possible presence of voids beneath the site, particular attention should be made to the placement of structures and the use of cranes and heavy plant during construction.

As a guide, footings founded on sandstone may be designed in accordance with the limit state design parameters presented in the Table 4.

Table 4 – Indicative foundation design parameters

Unit ⁽¹⁾	Limit State Design Values		
	Ultimate End Bearing (MPa)	Ultimate Shaft Adhesion (kPa)	Young's Modulus (MPa) ⁽³⁾
4A Class V Sandstone	3	150	100
4B Class IV Sandstone	10	500	500
4C Class III Sandstone	50	800	800
4D Class II or better sandstone	80 ⁽²⁾	1500	1600

(1) Rock classification according to Pells et al (1998). These are generalised rock classification for the purpose of foundation design only and should not be used for assessment of rock excavatability or pile drillability.

(2) Maximum limit state design stress level of concrete likely to govern

(3) The modulus value is for long term vertical loading

The bearing pressure values given in Table 4 assume a minimum embedment of 0.3 m into the relevant bearing stratum. Bearing pressure values are dependent on the level of assessment carried out during construction.

Unit 4A Sandstone would be anticipated to be relatively thin across the site. For heavily loaded structures we recommend that footings be taken to the better quality Unit 4C or 4D Sandstone to obtain the benefit of higher end bearing and shaft adhesion values. The design values provided may be adopted for concept design purposes, however, these values may be found to be somewhat conservative during detailed design. Where specific design of footings is required, further assessment of the nature of the bedrock at specific locations would prove worthwhile.

To achieve the parameters presented within Table 4 the base of piles or shallow footings should be cleaned of debris prior to concrete pouring. A clean pile socket of roughness category R2 (defined as grooves of depth 1 mm to 4 mm and width greater than 2 mm at spacing of 50 mm to 200 mm) or better is required to achieve the values provided above. Shaft adhesion values may have to be reduced if rock sockets are smeared or polish is present.

For Limit State Design, the selection of geotechnical strength reduction factors is dependent on the confidence in the selected design parameters and analysis model and results. For rock sockets comprising both side shear and end bearing resistance, a geotechnical strength reduction factor, ϕ_g , for Hawkesbury Sandstone of 0.6 or even higher can be obtained if appropriate investigation and construction testing is carried out.

For uplift capacity the shaft adhesion values in Table 4 should be multiplied by 0.6, in addition to the geotechnical strength reduction. In addition to shaft adhesion, the uplift capacity should be checked for a cone pull-out failure mode assuming a cone angle of 90° considering the submerged weight of the soil or rock and adopting a factor of safety of 1 against pull-out. Uplift capacity for groups of piles will need to take into account interaction between piles which will lead to lower individual pile capacities.

For footings on rock the settlement characteristics of the footing rather than ultimate load capacity often governs the design. The serviceability limit state should be assessed using the elastic modulus values presented in Table 4.

Within areas of potential buried cliff features, bearing pressures may need to be downgraded unless the entire base of the footing should lie outside a line projected upwards at 45° from the base of the cliff. Due to the uncertain nature of the buried features it is recommended that further investigations be undertaken during or preferably prior to undertaking the detailed design.

Prior to concreting, all shallow footings should be observed by a Geotechnical Engineer or Engineering Geologist to assess the exposed sandstone. Where the required ultimate bearing pressure for shallow, pad or strip footings is greater than 3 MPa, an assessment should also include on-site testing to assess whether defects below the base of the footing are within tolerable limits. For piles, a geotechnical engineer should be engaged to observe piling and depending on the adopted design parameters, proof coring may be required to confirm rock class at individual pile locations. About 50% of the piles should be proof cored if founding sandstone Class III or better.

5.6. Groundwater management

Please refer to the separate Coffey report ref: GEOTLCOV25340AA-AE Rev1 - Preliminary Groundwater Assessment, dated 12 July 2016.

5.7. Excavation induced ground movements

Walls retaining soil strength material may laterally deflect up to 1% of the retained height, depending on the stiffness of the retaining wall system but can be walls can be engineered to maintain the lateral deflections within 0.3% to 0.5%.

The potentially damaging effects of stress redistribution in the vicinity of excavations should be assessed as part of the detailed design. Lateral displacements of retaining walls due to stress redistribution may also result in settlement. For preliminary assessment of impacts we recommend that potential settlement be assumed to be equal to predicted lateral displacements. Typically, ground movements (lateral displacement and settlement) are greatest at the excavation face and decrease with increasing offsets from the face of the excavations.

For preliminary impact assessment purposes the above guidelines on displacements may be used. If such movements cannot be tolerated for sensitive features, then retaining walls should be designed

for higher earth pressures. Depending on the specific retention system, basement excavation details and the nature of adjacent structures, a more detailed analysis will be required.

5.8. Acid sulphate soils

With reference to the Acid Sulfate Soil (ASS) Risk Map available in the Australian Soil Resource Information System (ASRIS), the site is noted as 'Disturbed Terrain' which relates to the historic reclamation of low lying areas along the Darling Harbour foreshore for urban development.

Analysis of alluvial soils collected from land immediately to the southeast provided a strong indication that these soils are classified as PASS (Coffey, Aug 2013).

The ASRIS map indicated that there was a high probability of ASS or Potential Acid Sulfate Soils (PASS) in sediments in Darling Harbour and Sydney Harbour. There is evidence that the site and surrounding area has been reclaimed using harbour sediments, possibly along with other sources of fill material. Therefore it is possible that the some of the fill material at the site could contain ASS or PASS.

Reference should also be made to the Coffey report ref: GEOTLCOV25340AA-AC v2, dated 12 July 2016.

5.9. Seismic design considerations

Selection of seismic parameters for design of soil retention and underground structures should follow the requirements of:

- AS 4678-2002 Earth Retaining Structures
- AS 1170.4-2007 Structural Design Actions – Part 4: Earthquake Action in Australia

The probability factor (k_p) for the annual probability of exceedance shall be selected in accordance with AS1170.4 as appropriate for the limit state under consideration. A hazard factor (Z) of 0.08 is appropriate for the site. Based on the limited data available, ground conditions are considered to most closely resemble the site sub-soil class of Class C_e , as defined in Section 4 of AS 1170.4.

The presence of loose sands within the fill deposits poses the potential risk of liquefaction. Therefore the potential for the site soils at Harbourside to liquefy under design earthquake events should be assessed. Liquefaction is the temporary loss of shear strength in granular soils that results from increased pore water pressure during an earthquake event. The effects of liquefaction can be significant and should therefore be considered in the detailed design of in-ground structures.

6. Further geotechnical site investigation requirements

Geotechnical data within the site is limited. A broad characterisation of the site has been possible based upon the available data and our experience within this part of Sydney, however, as design progresses more specific geotechnical information is recommended. Further work that may be anticipated would include:

- A series of targeted boreholes across the site to assess the thickness and nature of the fill and natural soils, improve the top of bedrock model and better understand the weathered bedrock profile.
- Geotechnical laboratory testing to help refine geotechnical design parameters.

- Installation of groundwater piezometers for water level monitoring and permeability testing.
- Assessment of the potential for acid sulphate soils within the alluvial/estuarine sediments (and possibly fill where this comprises reworked natural sediments), and whether treatment of natural sediment spoil will be required.
- Assessment of the potential aggressivity of soils and groundwater on subsurface steel and concrete elements.

7. Conclusions and recommendations

We have assessed the elevation of geological strata (fill, sediments and rock) at the Harbourside Darling Harbour site. Due to the relative lack of data within the site, the modelled top of rock does not include possible cliff line locations. The presence of cliff lines is anticipated and further investigation is recommended.

Development drawings indicate that construction shall involve excavation of a two level basement, with B2 floor level of -3.1 m AHD indicated on the drawings. It is reasonable to expect the bulk excavation level to extend to approximately RL -4.0 m (or slightly higher). The excavation shall extend through highly variable fill material as well as sandstone bedrock.

Retention of the excavation will be required. Excavation shall extend below the groundwater table and will therefore need to provide cut-off to groundwater flow through the permeable fill and estuarine/alluvial deposits. Retention/cut-off options may consider D wall and secant pile wall options.

Geotechnical design parameters have been provided for retention systems and for foundation systems (piles and footings).

The presence of loose sands within the fill deposits poses the potential risk of liquefaction. The effects of liquefaction should be considered in the detailed design of in-ground structures.

We recommend the following:

- Review of geotechnical and groundwater constraints and excavation retention requirements following concept design of the proposed basement and any other proposed in-ground structures.
- Analysis of potential groundwater seepage below the basement perimeter wall, and measures which could be employed to reduce the magnitude of seepage inflows or to otherwise mitigate impacts.
- Assessment of excavation-induced ground movements.

Based on our site observations, preliminary geotechnical model, and experience on similar projects, the proposed development is considered feasible from a geotechnical perspective. The proposed development presents a low risk to surrounding structures and the groundwater environment, provided appropriate additional site investigation, design assessments, and construction monitoring normally associated with this type of development are carried out.

8. Limitations

The inferred geotechnical model presented in this desk study has been based predominantly upon limited data. The assessments provided herein are based on discrete/specific investigation

methodologies used in accordance with normal practices and standards. As more information becomes available during construction, the geotechnical interpretation and recommendations should be reviewed in the light of this information.

In addition, subsurface conditions can change over relatively short distances and the subsurface conditions revealed at the test locations may not be representative of subsurface conditions across the site. We recommend that a geotechnical engineer/engineering geologist be engaged during construction to confirm that subsurface conditions are consistent with design assumptions.

The attached document entitled "Important Information About Your Coffey Report" presents additional information on the uses and limitations of this report.

9. References

Pells, P.J.N, G. Mostyn, and B.F. Walker (1998), Foundations on Sandstone and Shale in the Sydney Region, Australian Geomechanics Journal, December 1998.

New South Wales Department of Mineral Resources, Geological Series Sheet 9130 (Edition 1), Sydney 1:100,000 Scale, 1983

Important information about your Coffey Report

As a client of Coffey you should know that site subsurface conditions cause more construction problems than any other factor. These notes have been prepared by Coffey to help you interpret and understand the limitations of your report.

Your report is based on project specific criteria

Your report has been developed on the basis of your unique project specific requirements as understood by Coffey and applies only to the site investigated. Project criteria typically include the general nature of the project; its size and configuration; the location of any structures on the site; other site improvements; the presence of underground utilities; and the additional risk imposed by scope-of-service limitations imposed by the client. Your report should not be used if there are any changes to the project without first asking Coffey to assess how factors that changed subsequent to the date of the report affect the report's recommendations. Coffey cannot accept responsibility for problems that may occur due to changed factors if they are not consulted.

Subsurface conditions can change

Subsurface conditions are created by natural processes and the activity of man. For example, water levels can vary with time, fill may be placed on a site and pollutants may migrate with time. Because a report is based on conditions which existed at the time of subsurface exploration, decisions should not be based on a report whose adequacy may have been affected by time. Consult Coffey to be advised how time may have impacted on the project.

Interpretation of factual data

Site assessment identifies actual subsurface conditions only at those points where samples are taken and when they are taken. Data derived from literature and external data source review, sampling and subsequent laboratory testing are interpreted by geologists, engineers or scientists to provide an opinion about overall site conditions, their likely impact on the proposed development and recommended actions. Actual conditions may differ from those inferred to exist, because no professional, no matter how qualified, can reveal what is hidden by earth, rock and time. The actual interface between materials may be far more gradual or abrupt than assumed based on the facts obtained. Nothing can be done to change the actual site conditions which exist, but steps can be taken to reduce the impact of unexpected conditions.

For this reason, owners should retain the services of Coffey through the development stage, to identify variances, conduct additional tests if required, and recommend solutions to problems encountered on site.

Your report will only give preliminary recommendations

Your report is based on the assumption that the site conditions as revealed through selective point sampling are indicative of actual conditions throughout an area. This assumption cannot be substantiated until project implementation has commenced and therefore your report recommendations can only be regarded as preliminary. Only Coffey, who prepared the report, is fully familiar with the background information needed to assess whether or not the report's recommendations are valid and whether or not changes should be considered as the project develops. If another party undertakes the implementation of the recommendations of this report there is a risk that the report will be misinterpreted and Coffey cannot be held responsible for such misinterpretation.

Your report is prepared for specific purposes and persons

To avoid misuse of the information contained in your report it is recommended that you confer with Coffey before passing your report on to another party who may not be familiar with the background and the purpose of the report. Your report should not be applied to any project other than that originally specified at the time the report was issued.

Interpretation by other design professionals

Costly problems can occur when other design professionals develop their plans based on misinterpretations of a report. To help avoid misinterpretations, retain Coffey to work with other project design professionals who are affected by the report. Have Coffey explain the report implications to design professionals affected by them and then review plans and specifications produced to see how they incorporate the report findings.

Data should not be separated from the report*

The report as a whole presents the findings of the site assessment and the report should not be copied in part or altered in any way.

Logs, figures, drawings, etc. are customarily included in our reports and are developed by scientists, engineers or geologists based on their interpretation of field logs (assembled by field personnel) and laboratory evaluation of field samples.

These logs etc. should not under any circumstances be redrawn for inclusion in other documents or separated from the report in any way.

Geoenvironmental concerns are not at issue

Your report is not likely to relate any findings, conclusions, or recommendations about the potential for hazardous materials existing at the site unless specifically required to do so by the client. Specialist equipment, techniques, and personnel are used to perform a geoenvironmental assessment. Contamination can create major health, safety and environmental risks. If you have no information about the potential for your site to be contaminated or create an environmental hazard, you are advised to contact Coffey for information relating to geoenvironmental issues.

Rely on Coffey for additional assistance

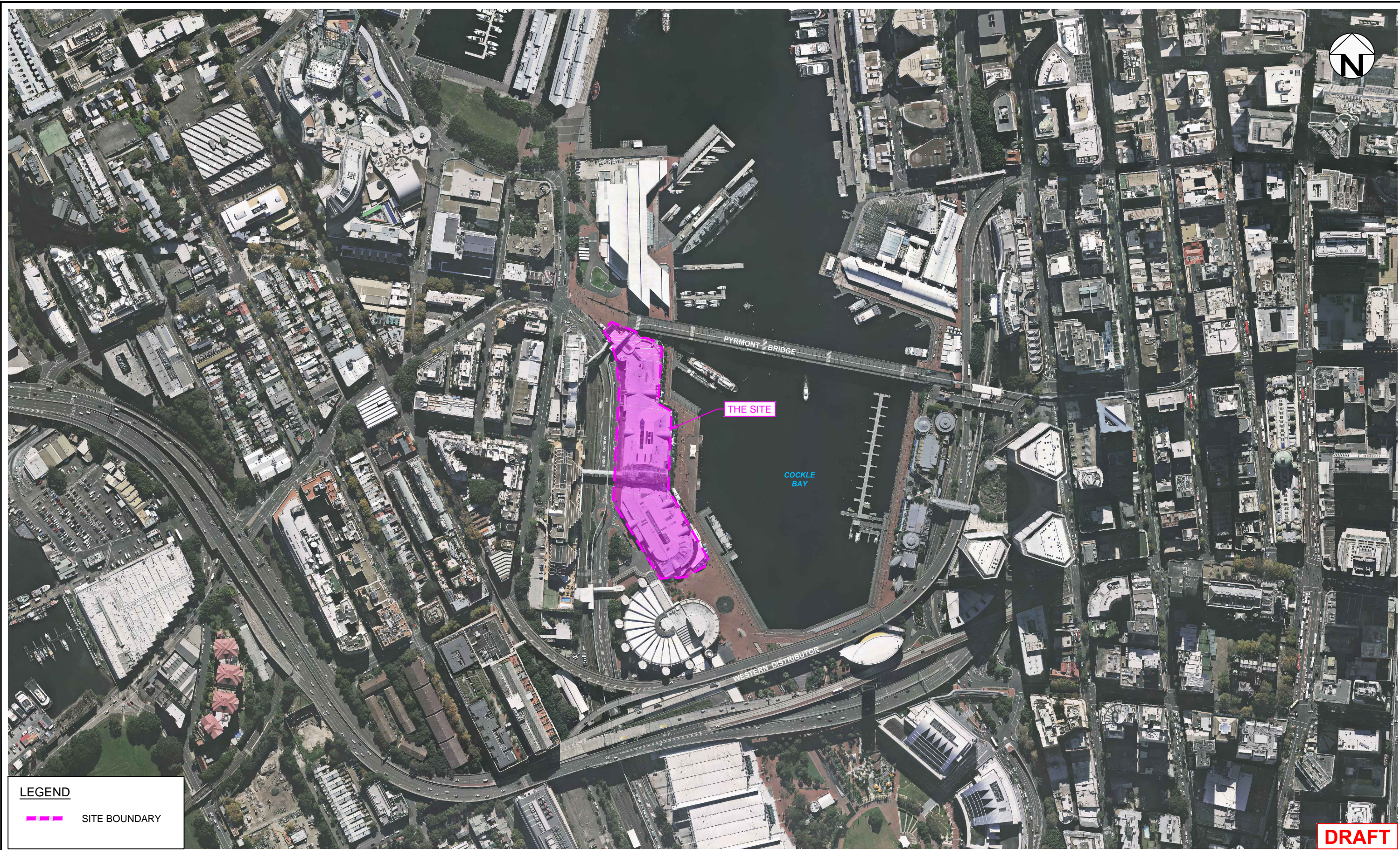
Coffey is familiar with a variety of techniques and approaches that can be used to help reduce risks for all parties to a project, from design to construction. It is common that not all approaches will be necessarily dealt with in your site assessment report due to concepts proposed at that time. As the project progresses through design towards construction, speak with Coffey to develop alternative approaches to problems that may be of genuine benefit both in time and cost.

Responsibility

Reporting relies on interpretation of factual information based on judgement and opinion and has a level of uncertainty attached to it, which is far less exact than the design disciplines. This has often resulted in claims being lodged against consultants, which are unfounded. To help prevent this problem, a number of clauses have been developed for use in contracts, reports and other documents. Responsibility clauses do not transfer appropriate liabilities from Coffey to other parties but are included to identify where Coffey's responsibilities begin and end. Their use is intended to help all parties involved to recognise their individual responsibilities. Read all documents from Coffey closely and do not hesitate to ask any questions you may have.

* For further information on this aspect reference should be made to "Guidelines for the Provision of Geotechnical information in Construction Contracts" published by the Institution of Engineers Australia, National headquarters, Canberra, 1987.

Appendix A - Proposed development information

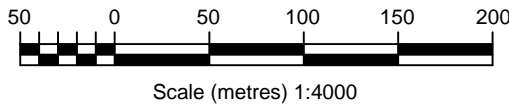


LEGEND

— SITE BOUNDARY

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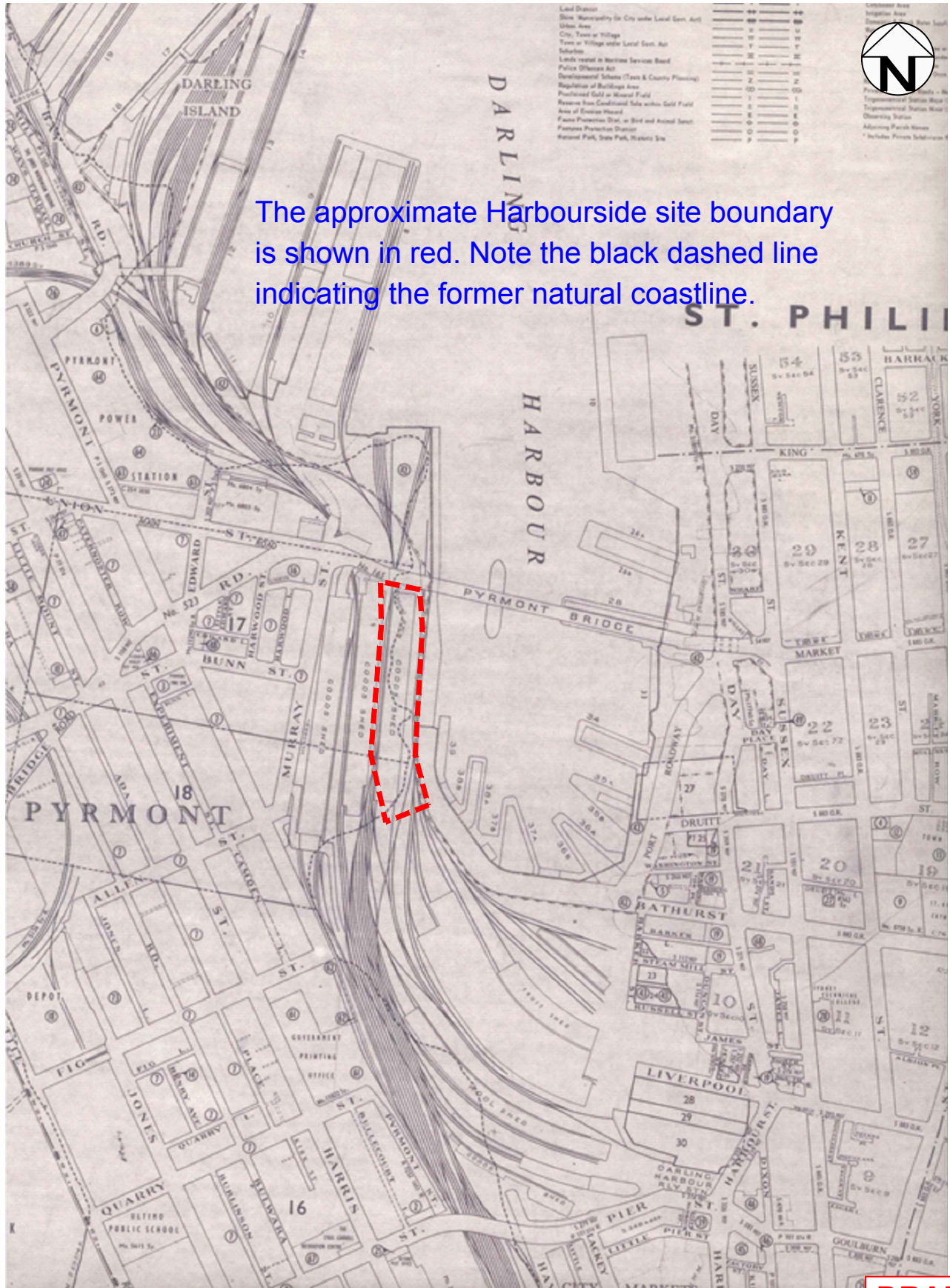
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	A	ORIGINAL ISSUE		MF	STP	18/02/16



drawn	PS / AW
approved	-
date	13 / 07 / 16
scale	AS SHOWN
original size	A3



client: MIRVAC	
project: PRELIMINARY GEOTECHNICAL ASSESSMENT REPORT HARBOURSIDE DARLING HARBOUR, SYDNEY, NSW	
title: SITE LOCATION PLAN	
project no: GEOTLCOV25340AA-AD	figure no: FIGURE 1
rev: A	

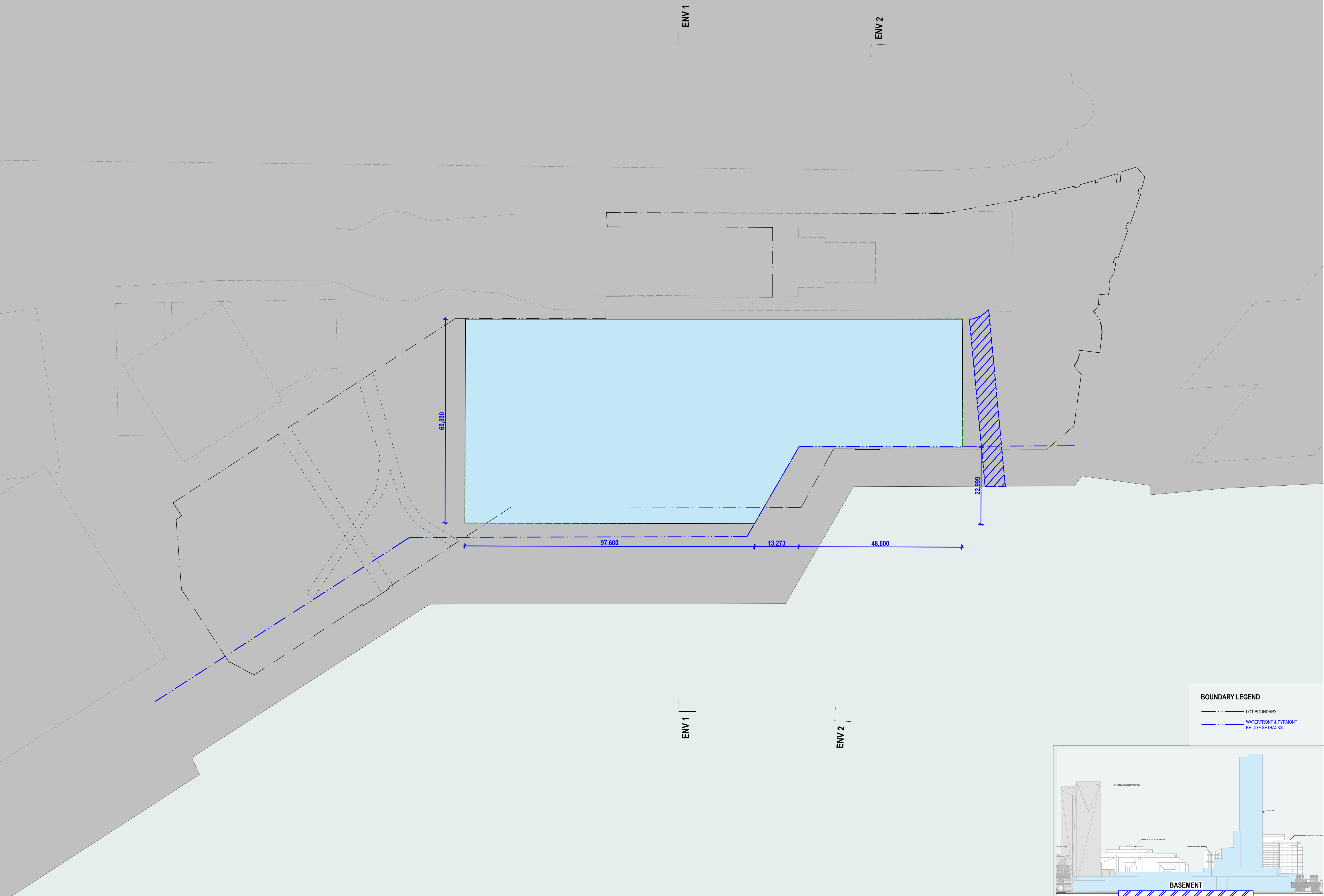


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drawn	PS / AW
approved	-
date	13 / 07 / 16
scale	AS SHOWN
original size	A4

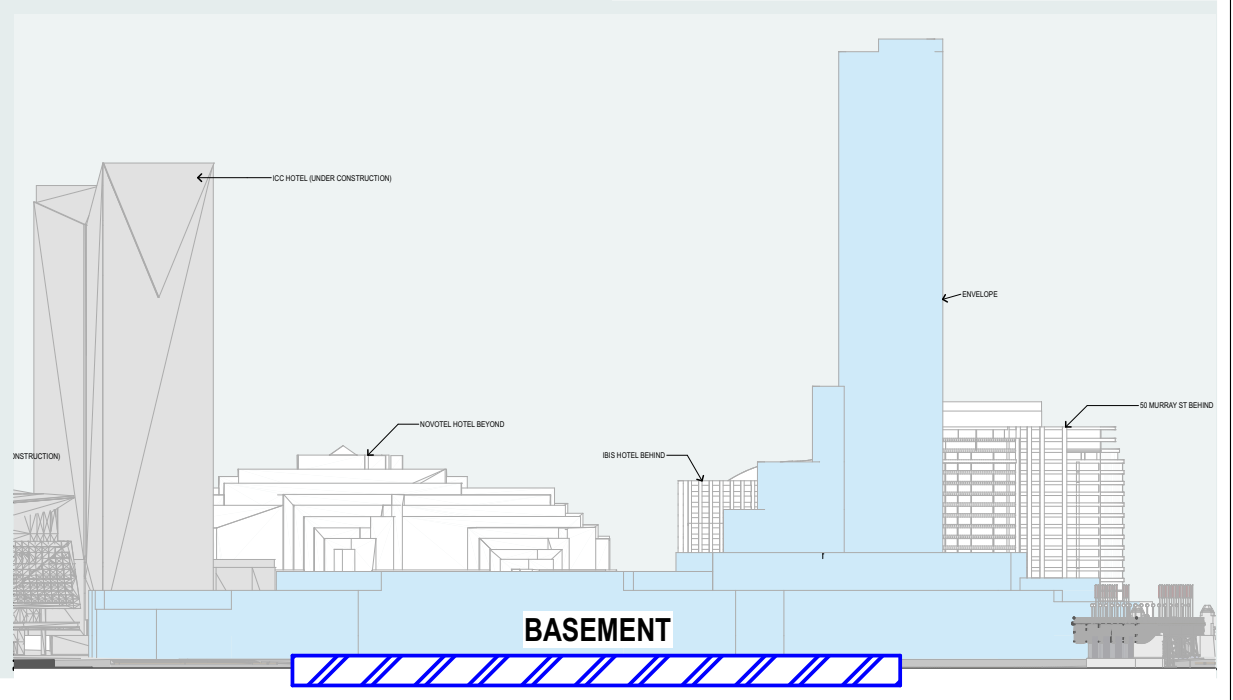


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project:	PRELIMINARY GEOTECHNICAL ASSESSMENT REPORT HARBOURSIDE DARLING HARBOUR, SYDNEY, NSW		
title:	1970 PARISH OF ST ANDREW, COUNTRY OF CUMBERLAND SURVEY PLAN		
project no:	GEOTLCOV25340AA-AD	figure no:	FIGURE 2
		rev:	A



BOUNDARY LEGEND

- LOT BOUNDARY
- WATERFRONT & PYRMONT BRIDGE SETBACKS



Appendix B – Sketches for geotechnical sections and Rock contours for Classes IV and III

client: MURVAL

office: CHATS

principal:

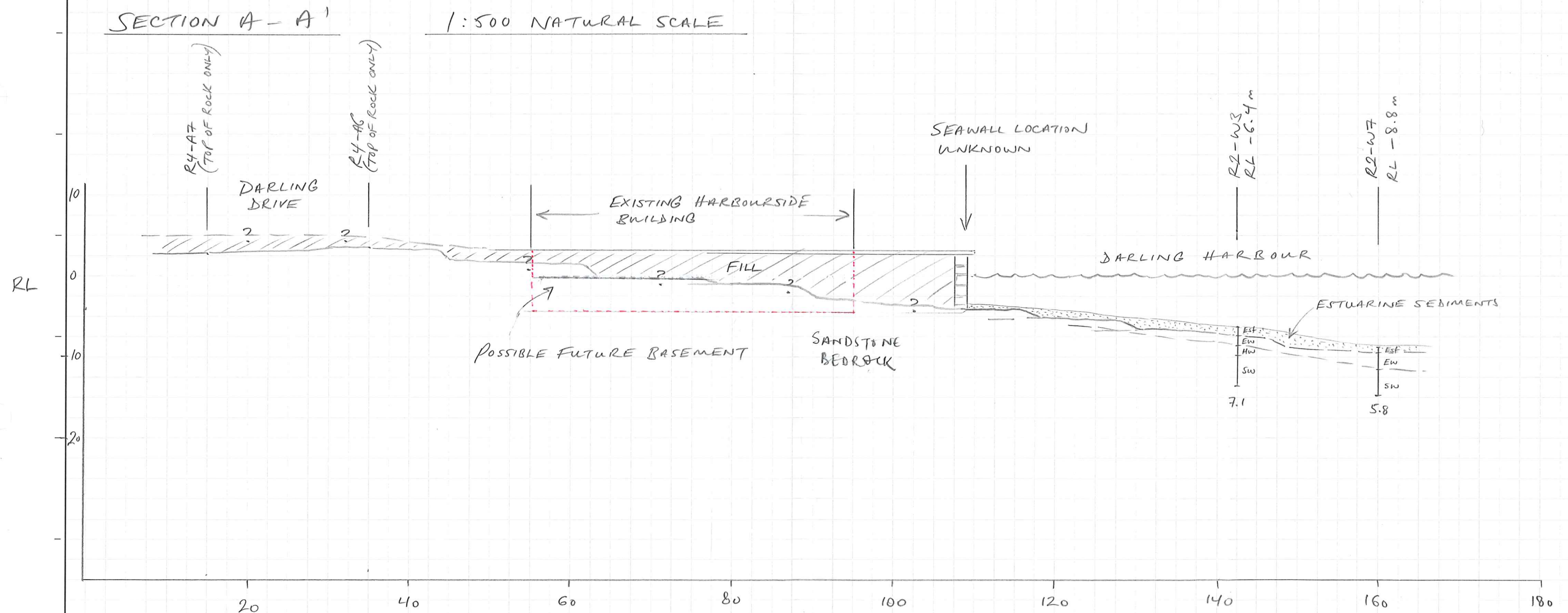
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project: HARBOURSIDE

by: MF/PS

location:

checked:



client: MIRVA

principal:

project: HARBOURSIDE

location:

project no: GEOTLCOV2524044

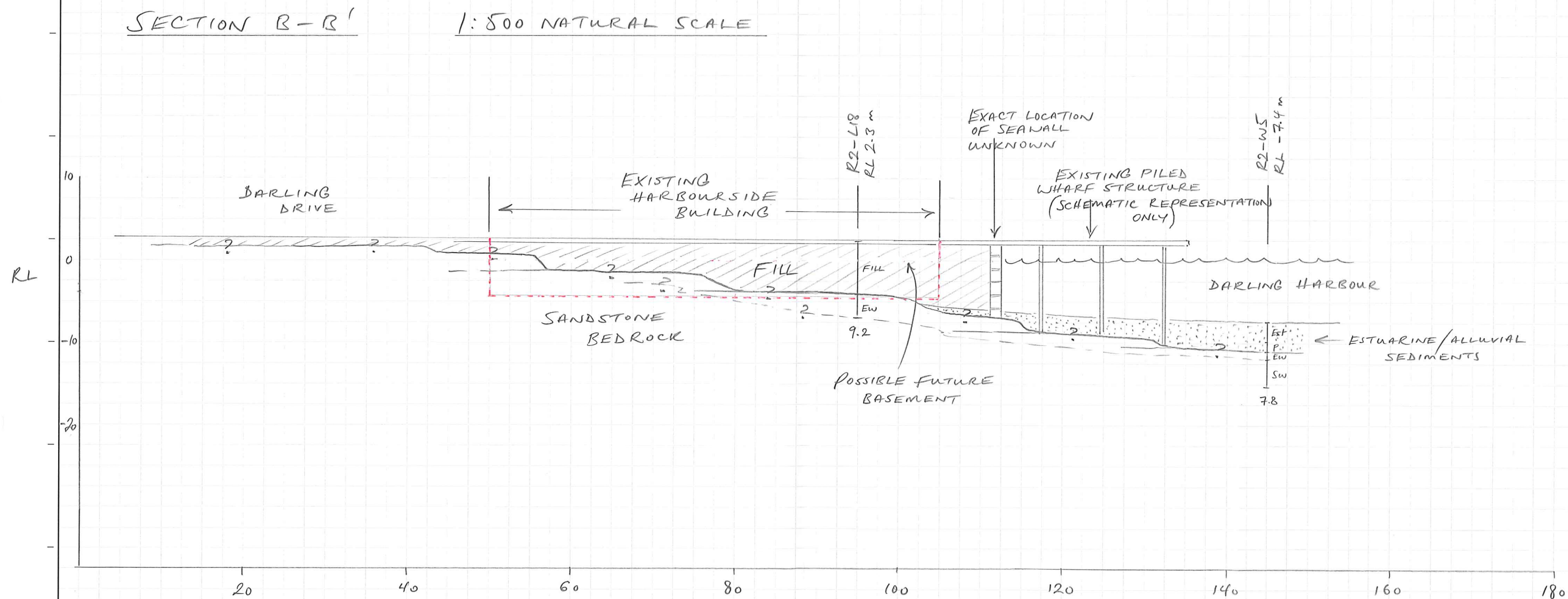
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office: CHATS

date: 18/02/2016

by: MF/PS

checked:



client: MRVAL

office: CHATS

principal:

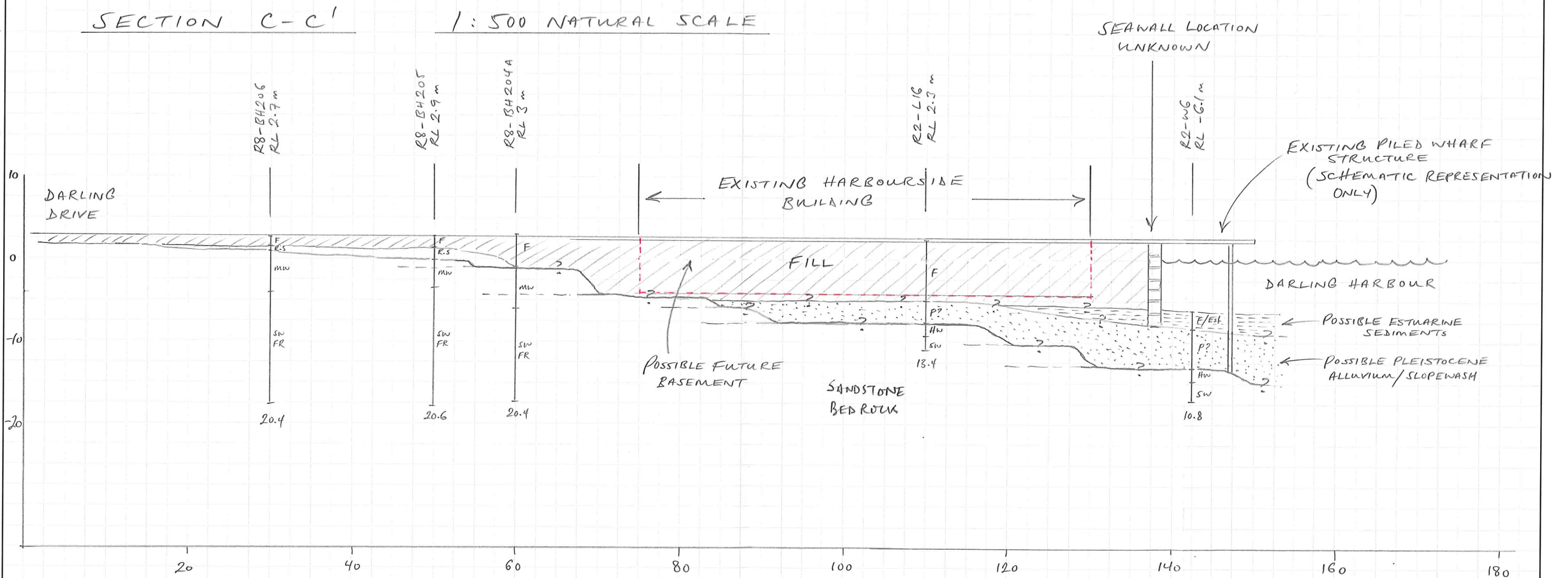
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project: HARBOURSIDE

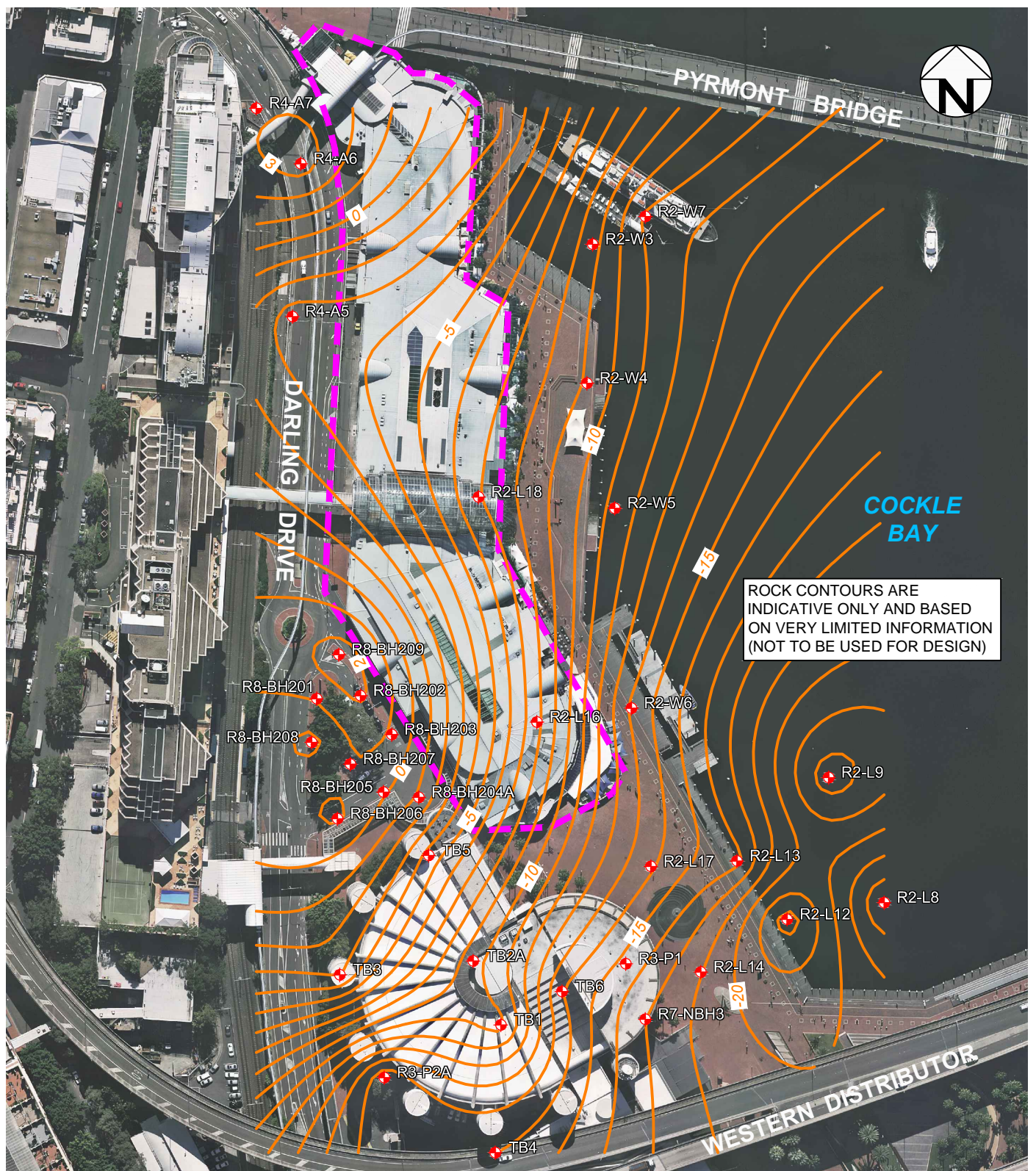
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location:

checked:






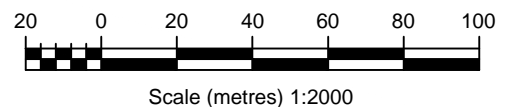
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ROCK CONTOURS ARE INDICATIVE ONLY AND BASED ON VERY LIMITED INFORMATION (NOT TO BE USED FOR DESIGN)

LEGEND

-  PREVIOUS BOREHOLE LOCATION
-  SITE BOUNDARY
-  TOP OF ROCK CONTOURS (m)



Scale (metres) 1:2000

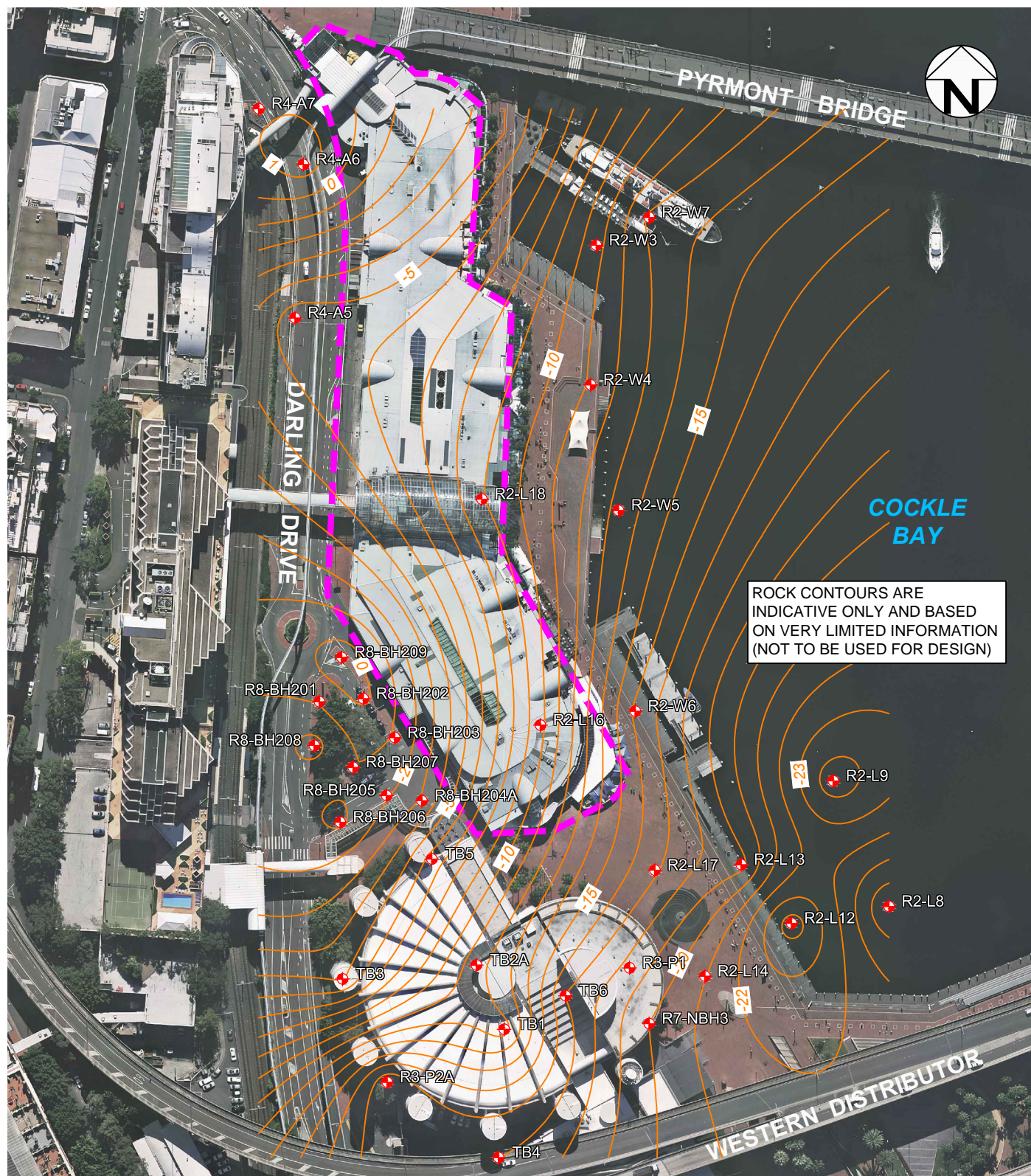
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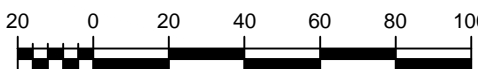

drawn	PS / AW
approved	-
date	13 / 07 / 16
scale	AS SHOWN
original size	A4

coffey
A TETRA TECH COMPANY

client:	MIRVAC		
project:	PRELIMINARY GEOTECHNICAL ASSESSMENT REPORT HARBOURSIDE DARLING HARBOUR, SYDNEY, NSW		
title:	TOP OF CLASS IV		
project no:	GEOTLCOV25340AA-AD	figure no:	FIGURE 3
		rev:	A

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LEGEND		 Scale (metres) 1:2000	
	PREVIOUS BOREHOLE LOCATION		
	SITE BOUNDARY		
	CLASS III SANDSTONE CONTOURS (m)		

drawn	PS / AW
approved	-
date	13 / 07 / 16
scale	AS SHOWN
original size	A4



A TETRA TECH COMPANY

client:	MIRVAC		
project:	PRELIMINARY GEOTECHNICAL ASSESSMENT REPORT HARBOURSIDE DARLING HARBOUR, SYDNEY, NSW		
title:	TOP OF CLASS III		
project no:	GEOTLCOV25340AA-AD	figure no:	FIGURE 4
		rev:	A

Appendix C - Cut off wall sketeches and Preliminary geotechnical pile capacities

Client: MIPVAC

Office: CHATEWOOD

Principal:

Date: 13/7/2016

Project: HARBOURSIDE

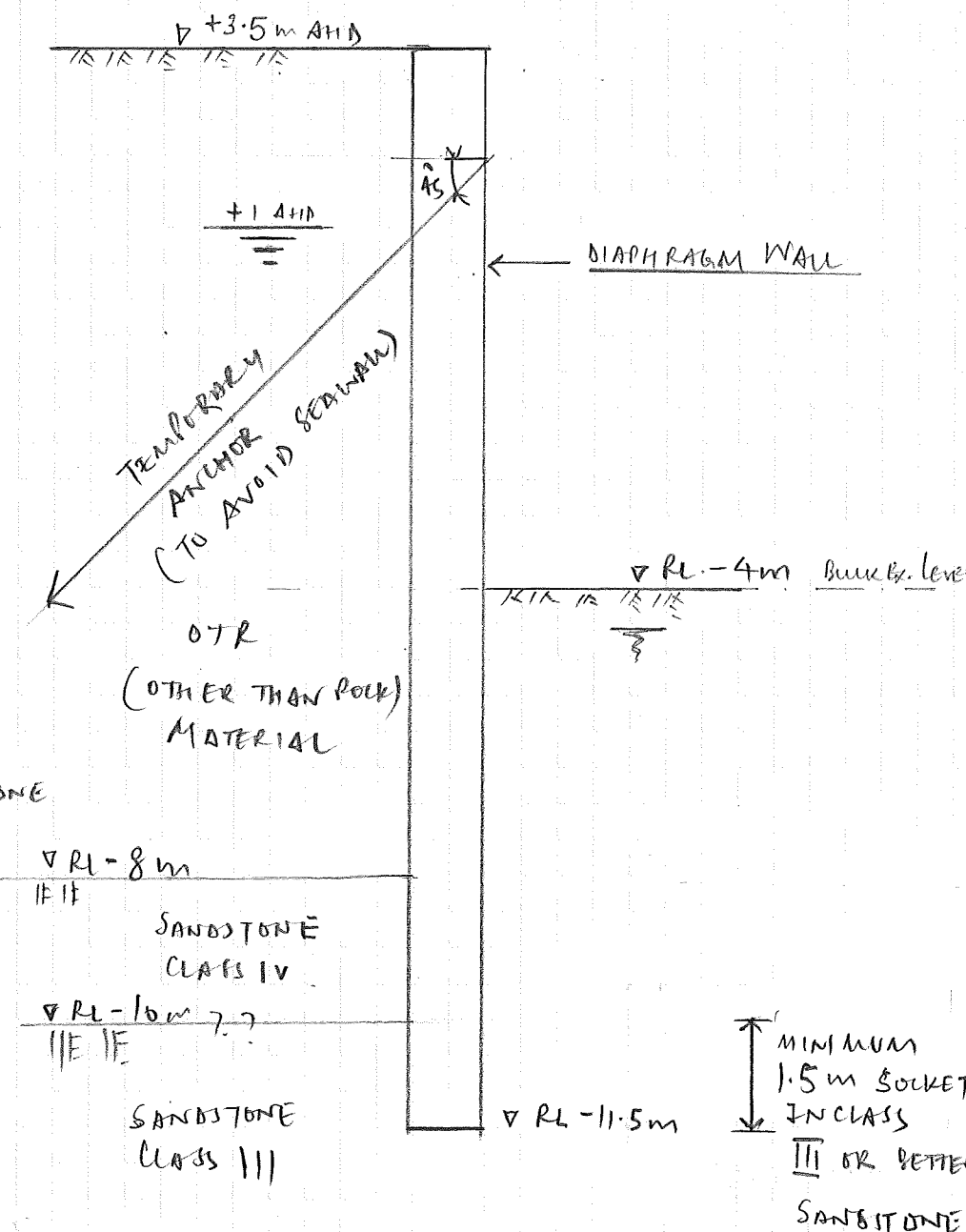
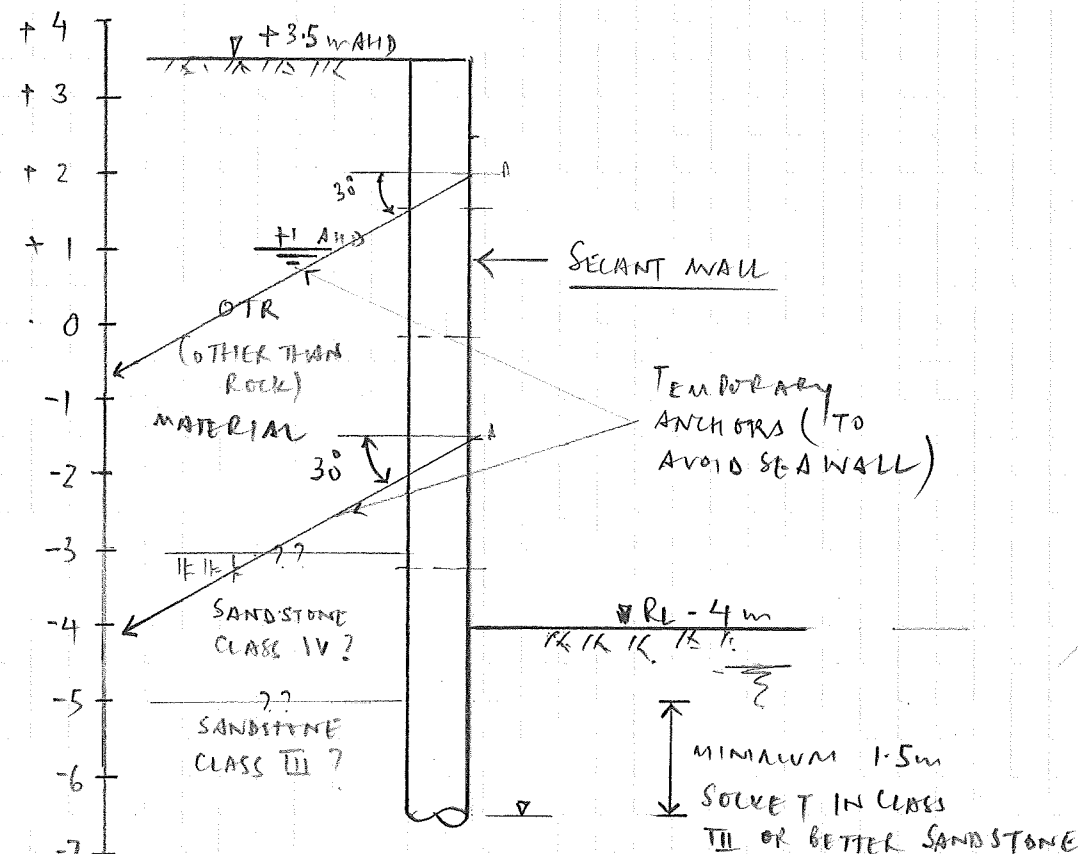
By: LU

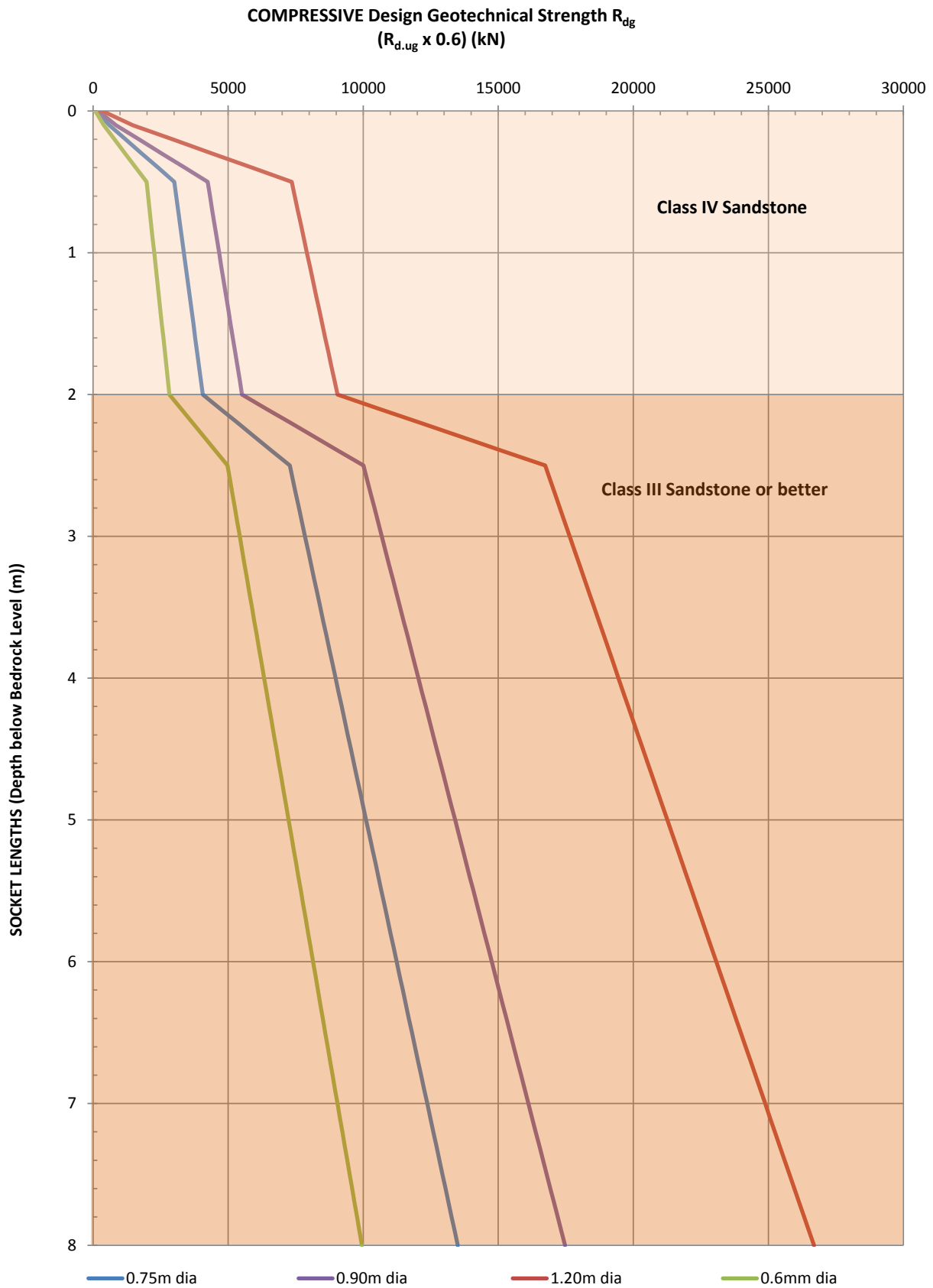
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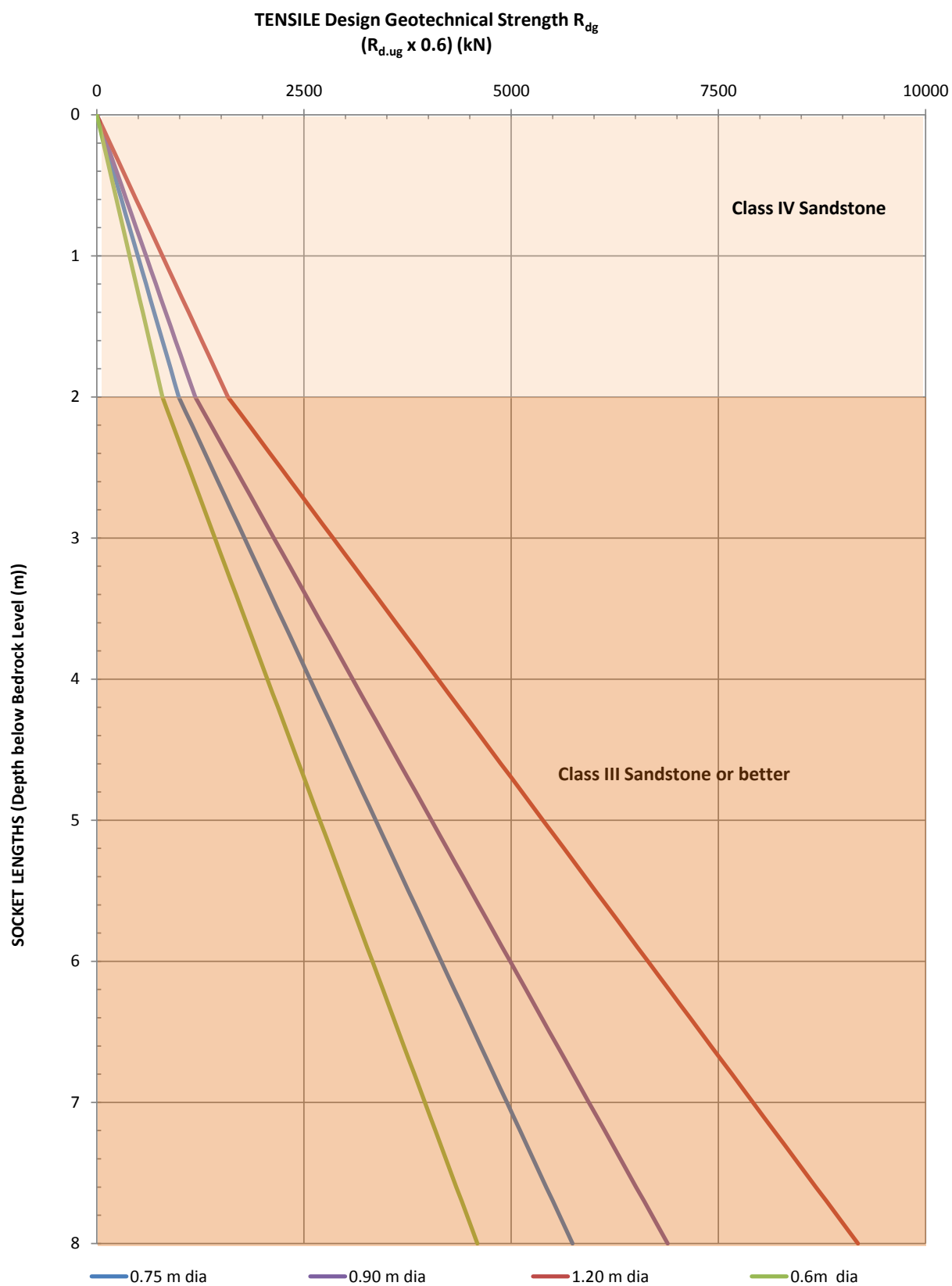
NOTES


1. ASSUMED GEOTECHNICAL PROFILE TO BE CONFIRMED BY FURTHER SITE INVESTIGATION
2. SECANT WALL FOR ROCK ELEVATION UP TO RL - 3.0 m
3. DIAPHRAGM WALL FOR ROCK ELEVATIONS LOWER THAN RL - 3 m
4. WALL CONFIGURATIONS ARE FOR GUIDANCE ONLY - NO WALL ANALYSIS HAS BEEN CARRIED OUT AT THIS STAGE
5. SECANT WALL - FOR AREAS WITH SHALLOW ROCK LEVELS I.E. RL - 1 m THE BOTTOM ANCHORS COULD BE REPLACED WITH TOE BOLTS/PINS AND THE ROCK SOCKET SHORTENED
6. THE TEMPORARY ANCHORS HAVE BEEN POSITIONED TO BE ABOVE WATER LEVEL (UPPER ROW).
7. TEMPORARY ANCHORS WILL NEED TO AVOID THE SEAWALL ON THE EASTERN SIDE





drawn	PS	 A TETRA TECH COMPANY	client:	MIRVAC	
approved			project:	Harbourside Development	
date	8/07/2016		title:	PILE (Single) DESIGN GEOTECHNICAL STRENGTH PLOT (Typical assumed Profile)	
scale	NTS		project no:	GEOTLCOV25340AA-AD	figure no: 5
original size	A4				



drawn	PS	 A TETRA TECH COMPANY	client:	MIRVAC	
approved			project:	Harbourside Development	
date	8/07/2016		title:	TENSILE PILE (Single) DESIGN GEOTECHNICAL STRENGTH PLOT (Typical assumed Profile)	
scale	NTS		project no:	GEOTLCOV25340AA-AD	figure no: 6
original size	A4				

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