Waterloo Integrated
Station Development

A Joint Venture Project
JOHN HOLLAND mirvac

WATERLOO METRO QUARTER OVER STATION DEVELOPMENT

Environmental Impact Statement

Appendix HH – Geotechnical Interpretive Report SSD-10438 Basement Car Park

Detailed State Significant Development
Development Application

Prepared for **Waterloo Developer Pty Ltd**

30 September 2020

Reference	Description
Applicable SSD Applications	SSD-10437 Southern Precinct SSD-10438 Basement Carpark
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1. Glossary and abbreviations

Reference	Description
ACHAR	Aboriginal Cultural Heritage Assessment Report
ADG	Apartment Design Guide
AHD	Australian height datum
AQIA	Air Quality Impact Assessment
BC Act	Biodiversity Conservation Act 2016
BCA	Building Code of Australia
BC Reg	Biodiversity Conservation Regulation 2017
BDAR	Biodiversity Development Assessment Report
CEEC	critically endangered ecological community
CFA	Continuous Flight Auger Piles
CIV	capital investment value
CMP	Construction Management Plan
Concept DA	A concept DA is a staged application often referred to as a 'Stage 1' DA. The subject application constitutes a detailed subsequent stage application to an approved concept DA (SSD 9393) lodged under section 4.22 of the EP&A Act.
Council	City of Sydney Council
CPTED	Crime Prevention Through Environmental Design
CSSI approval	critical State significant infrastructure approval
CTMP	Construction Traffic Management Plan
DA	development application
DPIE	NSW Department of Planning, Industry and Environment
DRP	Design Review Panel
EP&A Act	Environmental Planning and Assessment Act 1979
EPA	NSW Environment Protection Authority
EPA Regulation	Environmental Planning and Assessment Regulation 2000
EPBC Act	Environment Protection and Biodiversity Conservation Act 1999

Reference	Description
ESD	ecologically sustainable design
GANSW	NSW Government Architect's Office
GFA	gross floor area
GIR	Geotechnical Interpretive Report
HIA	Heritage Impact Assessment
IAP	Interchange Access Plan
LGA	Local Government Area
NCC	National Construction Code
OSD	over station development
PIR	Preferred Infrastructure Report
PGA	Peak ground acceleration
POM	Plan of Management
PSI	Preliminary Site Investigation
RMS	Roads and Maritime Services
SEARs	Secretary's Environmental Assessment Requirements
SEPP	State Environmental Planning Policy
SEPP 55	State Environmental Planning Policy No 55—Remediation of Land
SEPP 65	State Environmental Planning Policy No. 65 – Design Quality of Residential Apartment Development
SLS	Serviceability Limit State
SRD SEPP	State Environmental Planning Policy (State and Regional Development) 2009
SREP Sydney Harbour	State Regional Environmental Plan (Sydney Harbour Catchment) 2005
SSD	State significant development
SSD DA	State significant development application
SLEP	Sydney Local Environmental Plan 2012
Transport for NSW	Transport for New South Wales

Reference	Description
TIA	Traffic Impact Assessment
The proposal	The proposed development which is the subject of the detailed SSD DA
The site	The site which is the subject of the detailed SSD DA
TSE	Tunnel and station excavation stage
ULS	Ultimate limit state
VIA	Visual Impact Assessment
WMQ	Waterloo Metro Quarter
WMP	Waste Management Plan
WSUD	water sensitive urban design

2. Executive summary

This ~~planning report~~ Geotechnical Interpretive Report (GIR) has been prepared by WSP Australia Pty Ltd to accompany a detailed State significant development (SSD) development application (DA) for the Southern Precinct and Basement Car Park over station development (OSD) at the Waterloo Metro Quarter site.

This report has been prepared to address the relevant conditions of the concept SSD DA (SSD 9393) and the Secretary's Environmental Assessment Requirements (SEARs) issued for the detailed SSD DA (SSD 10437 & SSD 10438).

This report concludes that the proposed Southern Precinct and Basement OSD is suitable and warrants approval subject to the implementation of the following recommendations:

- SSD 10438 Basement
  - Conventional earthmoving equipment should be suitable for the bulk excavation and no significant heavy ripping or rock breaking is anticipated during the bulk excavation.
  - A set of geotechnical design parameters have been provided to inform the temporary retention of the secant pile wall, which will likely comprise 600mm diameter CFA secant piles. One or two rows of anchors may be required along the perimeter, with a groundwater management system in place, which will be confirmed during a later design stage.
  - The impact of lateral loads from the excavation, anchor destressing, and/or eventual demolition of the basement on the station box is captured in a separate technical advice note.
  - The basement is to be designed as an undrained structure, and as such the pile foundations should be designed to withstand buoyancy uplift pressures, as well as superstructure loads, using the recommended pile design parameters.
  - A separate impact assessment from the basement excavation on the Waterloo Congregational Church will be undertaken at a later design stage.
- SSD-10437 Building 3 Southern Precinct
  - Recommended pile design parameters have been provided to inform structural design of the foundations, which will likely comprise CFA piles or cast in-situ reinforced bored piles with temporary casing.
  - The impact of pile loading on the station box is captured in a separate technical advice note.
- Structural elements related to the structural integrity of the Waterloo Station Box are to be designed to the same level of design life and importance level as the Waterloo Station Box.
- The required instrumentation and monitoring for ground movement and vibration will be captured within a separate Instrumentation and Monitoring Plan, which will be completed during a later design stage.

3. Introduction

This report has been prepared to accompany a detailed State significant development (SSD) development application (DA) for the Southern Precinct and Basement Car Park over station development (OSD) at the Waterloo Metro Quarter site. The detailed SSD DA is consistent with the concept approval (SSD 9393) granted for the maximum building envelope on the site, as proposed to be modified.

The Minister for Planning, or their delegate, is the consent authority for the SSD DA and this application is lodged with the NSW Department of Planning, Industry and Environment (DPIE) for assessment.

The detailed SSD DA seeks development consent for the design, construction and operation of:

Southern Precinct

- 25-storey residential building (Building 3) comprising student accommodation, to be delivered as a mixture of studio and twin apartments with approximate capacity of 474 students
- 9-storey residential building (Building 4) above the southern station box to accommodate 70 social housing dwellings
- ground level retail tenancies including Makerspace and gymnasium lobby, and loading facilities
- level 1 and level 2 gymnasium and student accommodation communal facilities
- landscaping and private and communal open space at podium and roof top levels to support the residential accommodation
- new public open space including the delivery of the Cope Street Plaza, including vehicle access to the site via a shared way from Cope Street, expanded footpaths on Botany and Wellington streets and public domain upgrades
- signage zone locations
- utilities and service provision
- stratum subdivision (staged).

Basement Car Park

- 2-storey shared basement car park and associated excavation
- Ground level structure
- carparking for the commercial Building 1, residential Building 2, social housing Building 4, Waterloo Congregational Church and Sydney Metro
- service vehicle spaces
- commercial end-of-trip and bicycle storage facilities
- retail end-of-trip and bicycle storage facilities
- residential storage facilities
- shared plant and services
- in ground OSD tank for Building 2 located in Church Square.

This report has been prepared in response to the requirements contained within the Secretary's Environmental Assessment Requirements (SEARs) dated 8 April 2020 and 9 April 2020, and issued for the detailed SSD DA. Specifically, this report has been prepared to respond to the SEARs requirements summarised below.

Item	Description of requirement	Section reference (this report)
10. Construction Impacts	An assessment of potential impacts of the construction on surrounding buildings and the public domain, including air quality and odour impacts, dust emissions, water quality, stormwater runoff, groundwater seepage, soil pollution and construction and demolition waste, and proposed measures to mitigate any impacts.	Section 10
Plans and documents	Geotechnical assessment	Section 10

Table 1 - SEARS requirements

This report has also been prepared in response to the following conditions of consent issued for the concept SSD DA (SSD 9393) for the OSD as summarised in the table below.

Item	Description of requirement	Section reference (this report)
	Future development applications shall provide analysis and assessment of the impacts of construction works.	Section 10

Table 2 - Conditions of Concept Approval

4. The site

The site is located within the City of Sydney Local Government Area (LGA). The site is situated about 3.3 kilometres south of Sydney CBD and eight kilometres northeast of Sydney International Airport within the suburb of Waterloo.

The Waterloo Metro Quarter site comprises land to the west of Cope Street, east of Botany Road, south of Raglan Street and north of Wellington Street (refer to Figure 1). The heritage-listed Waterloo Congregational Church at 103–105 Botany Road is within this street block but does not form a part of the Waterloo Metro Quarter site boundaries.

The Waterloo Metro Quarter site is a rectangular shaped allotment with an overall site area of approximately 1.287 hectares.

The Waterloo Metro Quarter site comprises the following allotments and legal description at the date of this report. Following consolidation by Sydney Metro (the Principal) the land will be set out in deposited plan DP1257150.

- 1368 Raglan Street (Lot 4 DP 215751)
- 59 Botany Road (Lot 5 DP 215751)
- 65 Botany Road (Lot 1 DP 814205)
- 67 Botany Road (Lot 1 DP 228641)
- 124-128 Cope Street (Lot 2 DP 228641)
- 69-83 Botany Road (Lot 1, DP 1084919)
- 130-134 Cope Street (Lot 12 DP 399757)
- 136-144 Cope Street (Lots A-E DP 108312)
- 85 Botany Road (Lot 1 DP 27454)
- 87 Botany Road (Lot 2 DP 27454)
- 89-91 Botany Road (Lot 1 DP 996765)
- 93-101 Botany Road (Lot 1 DP 433969 and Lot 1 DP 738891)
- 119 Botany Road (Lot 1 DP 205942 and Lot 1 DP 436831)
- 156-160 Cope Street (Lot 31 DP 805384)
- 107-117A Botany Road (Lot 32 DP 805384 and Lot A DP 408116)
- 170-174 Cope Street (Lot 2 DP 205942).

The detailed SSD DA applies to the Southern Precinct and Basement Car Park (the site) of the Waterloo Metro Quarter site. The site has an area of approximately 4830sqm and 5,700sqm respectively. The subject site comprises the following allotments and legal description at the date of this report.

Southern Precinct DA

- 130–134 Cope Street (Lot 12 DP 399757) (Part)
- 136–144 Cope Street (Lots A-E DP 108312) (Part)
- 93–101 Botany Road (Lot 1 DP 433969 and Lot 1 DP 738891) (Part)
- 156–160 Cope Street (Lot 31 DP 805384)

- 107–117A Botany Road (Lot 32 DP 805384 and Lot A DP 408116)
- 119 Botany Road (Lot 1 DP 205942 and Lot 1 DP 436831)
- 170–174 Cope Street (Lot 2 DP 205942).

Basement Car Park DA

- 1368 Raglan Street (Lot 4 DP 215751) (Part)
- 59 Botany Road (Lot 5 DP 215751) (Part)
- 65 Botany Road (Lot 1 DP 814205) (Part)
- 67 Botany Road (Lot 1 DP 228641) (Part)
- 124–128 Cope Street (Lot 2 DP 228641) (Part)
- 69–83 Botany Road (Lot 1, DP 1084919)
- 130–134 Cope Street (Lot 12 DP 399757) (Part)
- 136–144 Cope Street (Lots A-E DP 108312) (Part)
- 85 Botany Road (Lot 1 DP 27454)
- 87 Botany Road (Lot 2 DP 27454)
- 89–91 Botany Road (Lot 1 DP 996765)
- 93–101 Botany Road (Lot 1 DP 433969 and Lot 1 DP 738891) (Part).

The boundaries of the overall site are identified at Figure 1, and the subject site of the detailed SSD DA is identified at Figures 2 and 3. The site is reasonably flat with a slight fall to the south.

The site previously included three to five storey commercial, light industrial and shop top housing buildings. All previous structures except for an office building at the corner of Botany Road and Wellington Street have been demolished to facilitate construction of the new Sydney Metro Waterloo station. As such the existing site is predominately vacant and being used as a construction site. Construction of the Sydney metro is currently underway on site in accordance with critical State significant infrastructure approval (CSSI 7400).

Figure 1 - Aerial image of the site
Source: Urbis

The area surrounding the site consists of commercial premises to the north, light industrial and mixed-use development to the south, residential development to the east and predominantly commercial and light industry uses to the west.

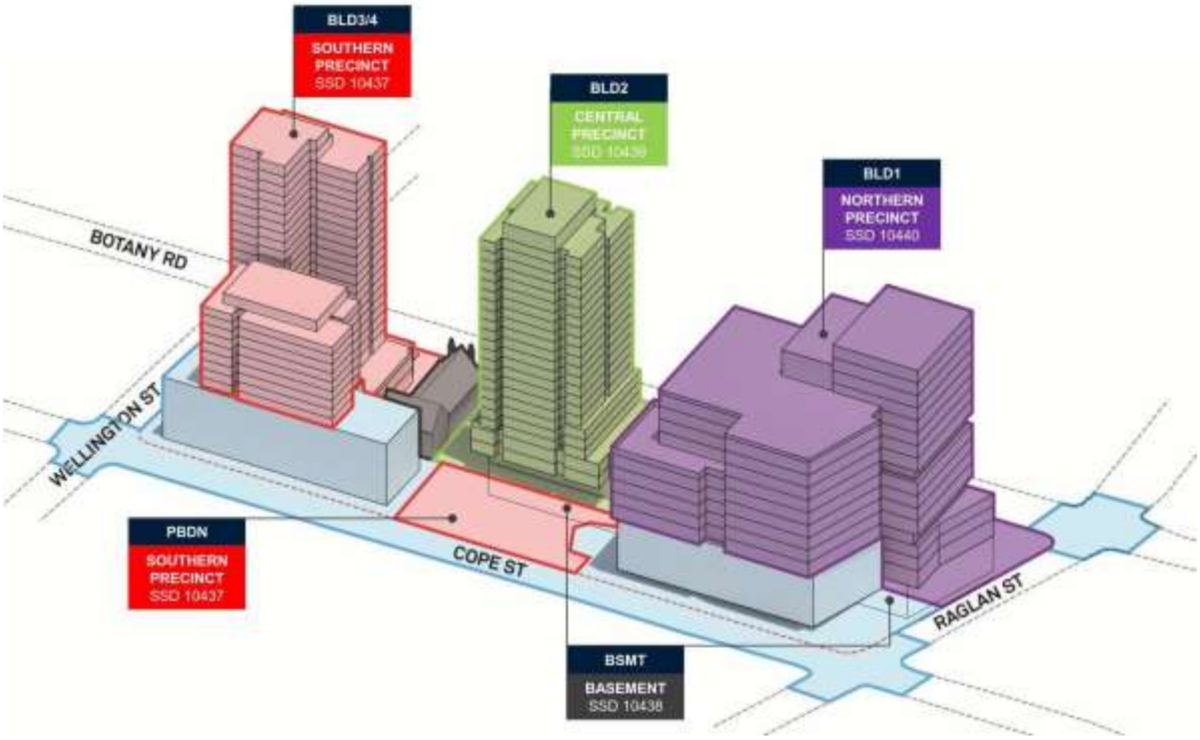

Figure 2 - Waterloo Metro Quarter site, with sub-precincts identified

Source: HASSELL

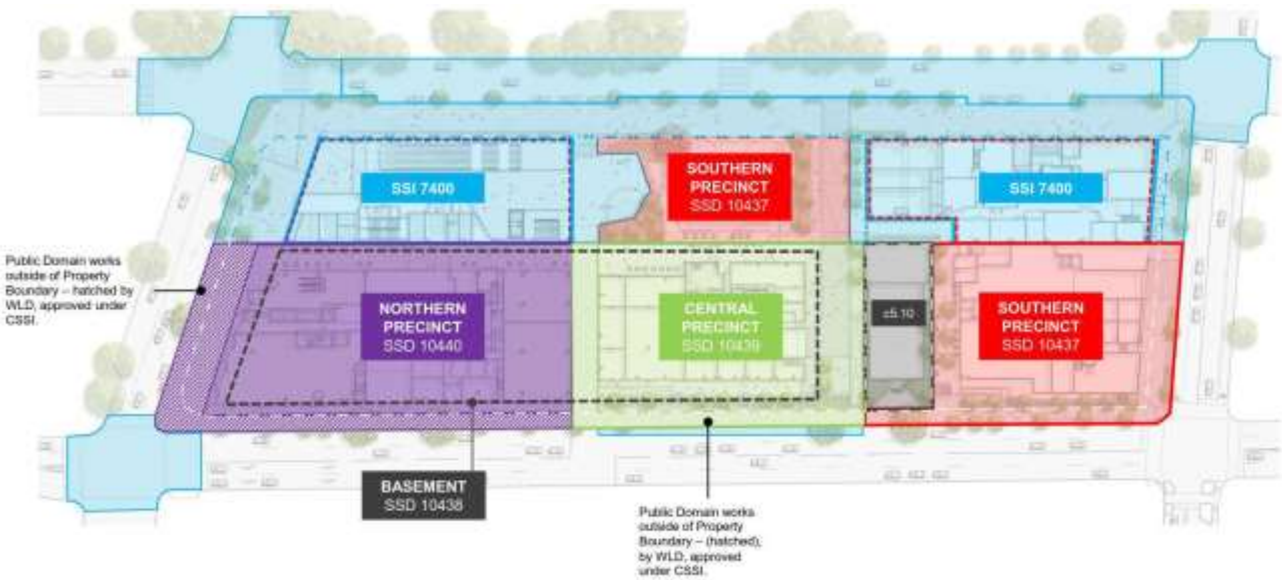

Figure 3 - Waterloo Metro Quarter site, with sub-precincts identified
Source: Waterloo Developer Pty Ltd

5. Background

5.1 About Sydney Metro

Sydney Metro is Australia's biggest public transport project. Services started in May 2019 in the city's North West with a train every four minutes in the peak. A new standalone railway, this 21st century network will revolutionise the way Sydney travels.

There are four core components:

5.1.1 Sydney Metro North West

This project is now complete and passenger services commenced in May 2019 between Rouse Hill and Chatswood, with a metro train every four minutes in the peak. The project was delivered on time and $1 billion under budget.

5.1.2 Sydney Metro City & Southwest

Sydney Metro City & Southwest project includes a new 30km metro line extending metro rail from the end of Metro Northwest at Chatswood, under Sydney Harbour, through new CBD stations and southwest to Bankstown. It is due to open in 2024 with the ultimate capacity to run a metro train every two minutes each way through the centre of Sydney.

Sydney Metro City & Southwest will deliver new metro stations at Crows Nest, Victoria Cross, Barangaroo, Martin Place, Pitt Street, Waterloo and new underground metro platforms at Central Station. In addition, it will upgrade and convert all 11 stations between Sydenham and Bankstown to metro standards.

5.1.3 Sydney Metro West

Sydney Metro West is a new underground railway connecting Greater Parramatta and the Sydney CBD. This once-in-a-century infrastructure investment will transform Sydney for generations to come, doubling rail capacity between these two areas, linking new communities to rail services and supporting employment growth and housing supply between the two CBDs.

The locations of seven proposed metro stations have been confirmed at Westmead, Parramatta, Sydney Olympic Park, North Strathfield, Burwood North, Five Dock and The Bays.

The NSW Government is assessing an optional station at Pyrmont and further planning is underway to determine the location of a new metro station in the Sydney CBD.

5.1.4 Sydney Metro Greater West

Metro rail will also service Greater Western Sydney and the new Western Sydney International (Nancy Bird Walton) Airport. The new railway line will become the transport spine for the Western Parkland City's growth for generations to come, connecting communities and travellers with the rest of Sydney's public transport system with a fast, safe and easy metro service.

The Australian and NSW governments are equal partners in the delivery of this new railway.

The Sydney Metro project is illustrated below.

Figure 4 - Sydney Metro alignment map
Source: Sydney Metro

5.2 Sydney Metro CSSI Approval (SSI 7400)

On 9 January 2017, the Minister for Planning approved the Sydney Metro City & Southwest - Chatswood to Sydenham project as a critical State significant infrastructure (CSSI) project (reference SSI 7400) (CSSI approval). The terms of the CSSI approval includes all works required to construct the Sydney Metro Waterloo Station. The CSSI approval also includes the construction of below and above ground works within the metro station structure for appropriate integration with the OSD.

With regards to CSSI related works, any changes to the ‘metro station box’ envelope and public domain will be pursued in satisfaction of the CSSI conditions of approval and do not form part of the scope of the concept SSD DA or detailed SSD DA for the OSD.

Except to the extent described in the EIS or Preferred Infrastructure Report (PIR) submitted with the CSSI application, any OSD buildings and uses do not form part of the CSSI approval and will be subject to the relevant assessment pathway prescribed by the EP&A Act.

The delineation between the approved Sydney Metro works, generally described as within the two ‘metro station boxes’ and surrounding public domain works, and the OSD elements are illustrated in Figure 5.

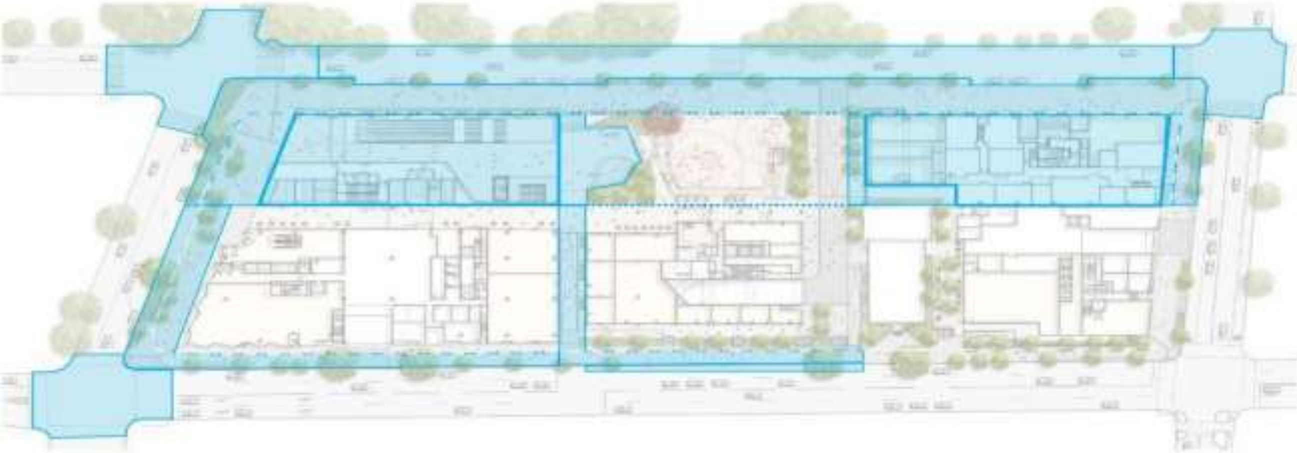

Figure 5 - CSSI Approval scope of works
Source: WL Developer Pty Ltd

5.3 Concept Approval (SSD 9393)

As per the requirements of clause 7.20 of the *Sydney Local Environmental Plan 2012* (SLEP), as the OSD exceeds a height of 25 metres above ground level (among other triggers), development consent is first required to be issued in a concept DA (formerly known as Stage 1 DA).

Development consent was granted on 10 December 2019 for the concept SSD DA (SSD 9393) for the Waterloo Metro Quarter OSD including:

- a maximum building envelope for podium, mid-rise and tower buildings
- a maximum gross floor area of 68,750sqm, excluding station floor space
- conceptual land use for non-residential and residential floor space
- minimum 12,000sqm of non-residential gross floor area including a minimum of 2,000sqm of community facilities
- minimum 5% residential gross floor area as affordable housing dwellings
- 70 social housing dwellings
- basement car parking, motorcycle parking, bicycle parking, and service vehicle spaces.

The detailed SSD DA seeks development consent for the OSD located within the Southern Precinct and Basement Car Park of the site, consistent with the parameters of this concept approval. A combined SSD DA has been prepared and will be submitted for the Southern Precinct and Basement proposed across the Waterloo Metro Quarter site.

A concurrent amending concept SSD DA has been prepared and submitted to the DPIE which proposed to make modifications to the approved building envelopes at the northern precinct and central building. This amending concept SSD DA does not impact the proposed development within the southern precinct.

6. Proposed development

6.1 Waterloo Metro Quarter Development

The Waterloo Metro Quarter OSD comprises four separate buildings, a basement carpark and public domain works adjacent to the Waterloo Metro station.

Separate SSD DAs will be submitted concurrently for the design, construction and operation of each building in the precinct;

- Southern precinct SSD-10437,
- Basement Car Park SSD-10438,
- Central precinct SSD-10439, and
- Northern precinct-SSD-10440.

An overview of the Development is included below for context. This detailed SSD DA seeks development consent for the design, construction and operation of the Southern Precinct and Basement Car Park:

6.1.1 Southern Precinct [Subject DA]

The Southern Precinct comprises:

- 25-storey residential building (Building 3) comprising student accommodation, to be delivered as a mixture of studio and twin apartments with approximate capacity of 474 students
- 9 storey residential building (Building 4) above the southern station box to accommodate 70 social housing dwellings
- ground level retail tenancies including Makerspace and gymnasium lobby, and loading facilities
- level 1 and level 2 gymnasium and student accommodation communal facilities
- landscaping and private and communal open space at podium and roof top levels to support the residential accommodation
- new public open space including the delivery of the Cope Street Plaza, including vehicle access to the site via a shared way from Cope Street, expanded footpaths on Botany and Wellington Streets and public domain upgrades
- signage zone locations
- utilities and service provision
- stratum subdivision (staged).

6.1.2 Basement Car Park [Subject DA]

The Basement Car Park comprises:

- 2-storey shared basement car park and associated excavation comprising
- Ground level structure
- Carparking for the Commercial Building 1, Residential Building 2, social housing Building 4, Waterloo Congregational Church and Sydney Metro
- Service vehicle bays

- commercial end of trip and bicycle storage facilities
- Retail end of trip and bicycle storage facilities
- residential storage facilities
- shared plant and services
- in ground OSD tank for building 2 located in Church Square.

6.1.3 Central Precinct

The Central Precinct comprises:

- 24-storey residential building (Building 2) comprising approximately 126 market residential and 24 affordable housing apartments, to be delivered as a mixture of 1 bedroom, 2 bedroom and 3 bedroom apartments
- Ground level retail tenancies, community hub, precinct retail amenities and basement car park entry
- level 1 and level 2 community facilities (as defined in the SLEP) intended to be operated as a childcare centre
- landscaping and private and communal open space at roof top levels to support the residential accommodation
- new public open space including the delivery of the Church Square, including vehicle access to the basement via a shared way from Cope Street, expanded footpaths and public domain upgrades on Botany Road
- external licensed seating areas
- signage zone locations
- utilities and service provision
- stratum subdivision (staged).

6.1.4 Northern Precinct

The Northern Precinct comprises:

- 17-storey commercial building (Building 1) comprising Commercial floor space, with an approximate capacity of 4000 workers
- ground level retail tenancies, loading dock facilities serving the northern and central precinct including Waterloo metro station
- landscaping and private open space at podium and roof top levels to support the commercial tenants
- new public open space including the delivery of the Raglan Street Plaza, Raglan Walk and expanded footpaths on Raglan Street and Botany Road and public domain upgrades
- external licensed seating areas
- signage zone locations
- utilities and service provision
- stratum subdivision (staged).

7. Methodology

7.1 Available reports

The following data has been reviewed in preparation of this Geotechnical Interpretive Report (GIR):

- Waterloo TAN WSP 013 / Rev 1, dated 13/02/2019 by WSP Australia Pty Limited.
- Waterloo TAN WSP 016 / Rev 0, dated 5/03/2019 by WSP Australia Pty Limited.
- Sydney Metro – City & Southwest Geotechnical Interpretive Report – City, Reference Design, NWRLSRT-PBA-SRT-GE-REP-000004, dated 29/11/2016 by AECOM Australia Pty Limited and Parsons Brinckerhoff Australia Pty Limited.
- Geotechnical Interpretive Report System Wide – Stage 1 Design, NWRLSRT-MET-SRT-GE-REP-000001, dated 31/01/2018 by Metron.
- Geotechnical Interpretive Report Waterloo Station, PS117919-GEO-REP-668A Rev C, dated 19/02/2020 by WSP Australia Pty Ltd.
- MQD Enabling Works Basis of Design report, SMCSW-RBG-SWL-ST-REP-120003 Rev C, dated 22/06/2020 by RBG.

7.2 Scope and objective

The purpose of this geotechnical interpretive report is to summarise the existing geotechnical data pertaining to the Waterloo Metro Quarter Development, specifically the Southern Precinct and the Basement, and to provide information on the ground model and geotechnical design parameters to inform the structural design of Metro Quarter Development.

The interpretation contained within this report is based on existing geotechnical investigation data from the Waterloo Station site, provided information by the Tunnels Station and Excavation (TSE) contractor of the Waterloo Station and a site visit undertaken on 18 December 2019 of the base of the excavation. No additional site investigations or tests have been undertaken.

8. Desk study and site walkover

8.1 Existing geotechnical data

WSP has extensive geotechnical information within the site and surrounding areas, which include multiple boreholes, cone penetration tests and laboratory tests, from investigations carried out by WSP and others. The figure below shows the locations of the undertaken geotechnical investigations, of which the data has been captured in the previous reports mentioned in Section 8.1.

Figure 6 - Existing geotechnical data available across the Waterloo Metro Quarter Development site

8.2 Geology

The 1:100,000 Geological map of Sydney indicates that the site is underlain by a layer of Quaternary sand deposits of fine to medium grain size, which are interpreted to be wind-blown (aeolian) deposits. This aeolian sand layer is underlain by residually weathered material of the Ashfield Shale, Mittagong Formation and basal Hawkesbury Sandstone. The Ashfield Shale is described as black to dark grey shale and laminate, whilst the Hawkesbury Sandstone is described as medium to coarse grained quartz sandstone with very minor shale and laminate lenses. At the interface between Ashfield shale and the Hawkesbury Sandstone, the Mittagong Formation is also encountered and is characterised by interbedded sandstones and shales.

8.3 Site walkover

A site walkover was undertaken by a senior principal engineering geologist and a technical executive geotechnical engineer on 18 December 2019 at the base of the station box excavation, adjacent to the proposed Waterloo Metro Quarter Development. There was no rain recorded in Sydney in the three weeks preceding the site visit, and weather condition on the day was noted to be fine.

The purpose of the site walkover was to confirm the following:

- Ground conditions at the base of the excavation.
- Confirmation of the Woolloomooloo Fault Zone along the southern zone of the project site, i.e. BLD 3.
- Observations of groundwater.

During the site walkover, shotcrete panels obscured the view of the rock behind the temporary shoring, and only the lower sections of the walls and the base of the excavations were exposed, which revealed Class I Hawkesbury Sandstone. The ground conditions overlying the Class I sandstone were completely obscured by the reinforced shotcrete facing, and the ground conditions overlying were unable to be confirmed. However, data from the TSE secant pile drilling and geological mapping undertaken during the TSE excavation would be able to supplement this. Towards the southern zone of the station box excavation, the Woolloomooloo Fault Zone was only noted to be present in the form of a few localised joints and was not as widely spread and weathered as initially assumed.

Groundwater stains were noted at several locations along the anchor heads from the second and third rows of anchors, as well as more significantly from below the shotcrete facing.

9. Ground conditions

9.1 Subsurface conditions

A variable thin layer of fill comprising a mixture of sand and gravel occurs across the Waterloo Metro Quarter Development site, underlain by Quaternary deposits which are interpreted as wind-blown (aeolian) sands. Underlying the sands are residual soils comprising silty clay which forms part of the weathered Ashfield Shale horizon. Localised thickening may be associated with fault/joint structures that have been identified within the region.

The Ashfield Shale is a highly to slightly weathered siltstone, below which the Mittagong Formation is encountered which comprises siltstone with variably thick laminations of fine grained sandstone. It grades sharply into the Hawkesbury Sandstone, which can be described as a fine to medium grained moderately cross bedded quartzose sandstone, with some light carbonaceous laminations.

The interpreted ground conditions across the Waterloo Metro Quarter Development are summarised in the geotechnical longitudinal and cross sections presented in Appendix A. The geotechnical model has been developed based on the provided information from the Sydney Metro geotechnical investigations (SRT series), including 4 boreholes completed for the Waterloo Metro Quarter Development works that were provided in February 2019, observations from the recently undertaken site visit of the excavation and geological mapping sheets provided by the TSE contractor during the excavation of the station box. The anticipated sub-surface profile across the project site is presented in the table below. The depth to top of rock generally dips towards the north-west, with soil thicknesses increasing from approximately 7 m to 13 m.

Geotechnical Unit	Description	Variability of Elevation at Top of Unit (RL M AHD)	Thickness Variability (M)
Fill	Sand and gravel	15 to 17	1 to 2
Quaternary Sediments	Sand, loose to medium dense	14 to 16	3 to 7
Residual soil	Silty clay, stiff to very stiff	8.5 to 12.5	4 to 8
Ashfield Shale	Shale (Class IV and V)	3 to 7	1 to 3
	Shale (Class III or better)	1 to 6	2 to 7
Mittagong Formation	Sandstone (Class I/II)	0.5 to -2.5	2
Hawkesbury Sandstone	Sandstone (Class I/II)	-0.8 to -4.8	N/A

Table 3 - Summary of ground conditions

The following geotechnical units are described as below.

9.1.1 Fill, quaternary sediments and residual soils

A variable thin (typically 0.5 m to 1.5m in depth) layer of fill comprising a mixture of sand and gravel occurs across the station box. Below this fill are quaternary sand deposits which are interpreted as Aeolian sand deposits and are intersected between approximately RL 8.5 m to 12.0 m AHD. Underlying the sands are residual soils comprising silty clay that persist to between RL 3 m to 7 m AHD. The residual layer forms part of the weathered Ashfield Shale rock.

9.1.2 Ashfield Shale

The Ashfield Shale was encountered within the deeper boreholes undertaken, below the residual layer, and was recorded as a highly to slightly weathered siltstone of the Rouse Hill Member, interpreted as a Class IV/V shale to RL 1.5 m to 6.0 m AHD. Below this layer lies a variable Class III to Class I shale that persists down to approximately RL -2.5 m to 1.0 m AHD, where the Mittagong Formation is encountered.

9.1.3 Mittagong Formation

The Mittagong Formation has been encountered as sandstone with variably thick laminations of siltstone and is generally thin (about 2 m in thickness). It grades sharply into the Hawkesbury Sandstone at approximately RL 1.0 m AHD at the north end of the project site (BLD 1) and RL5.0 m AHD at the south of the project site (BLD 3).

9.1.4 Hawkesbury Sandstone

Underlying the Mittagong Formation is the Hawkesbury Sandstone, which can be described as a fine to medium grained, moderately cross bedded, quartzose sandstone, with some light carbonaceous laminations. The sandstone that was encountered within the deeper boreholes were logged as fresh, cross bedded sandstone with no obvious geological structure. During the site walkover of the excavation, the formation was visible at the exposed faces near the base of the excavation, as shown in Figure 7.

Figure 7 - Exposed face of the excavation, depicting the Hawkesbury Sandstone formation

9.2 Geological structures

Regional geological mapping (Och et al, 2009) initially indicated that a projection of the Woolloomooloo Fault Zone extended across the southern end of the project site. The inclined borehole undertaken along Cope St, adjacent to the southern zone of the station box excavation (SRT_BH605), encountered discrete low angle structures (shears and joints) mainly within the Ashfield Shale, which had been interpreted to be associated with this fault zone. However, observations from the site walkover of the station box excavation did not reveal any significant geological structures associated with a typical fault zone. Only localised, discrete joints were present in the exposed Hawkesbury Class I/II Sandstone, at the southern zone of the excavation, where the Woolloomooloo Fault Zone was predicted to be present as shown in Figure 8. The rock mass in that area was typically observed to be slightly to unweathered and inferred to be of a high Geological Strength Index (GSI).

Figure 8 - Section of exposed Hawkesbury Sandstone underneath the proposed BLD 3

9.3 Groundwater

Piezometers for groundwater monitoring have previously been installed at 9 locations in the project site vicinity, with observed groundwater levels summarised in Table 4. The monitoring results indicate that the groundwater levels are typically between 3m to 5m below ground level (RL of 10 to 12 m AHD) within the Quaternary sands. It is possible that the groundwater table in the sand is perched at some locations. Figure 9 contains a hydrograph showing recorded groundwater levels and rainfall over the period September 2015 to September 2017. An additional 3 standpipes were installed in October 2018 at SRT_BH409, SRT_BH419 and SRT_BH420. Groundwater inflows were recorded at approximately 4m below ground level (RL 12 m AHD) during drilling.

The highest level of groundwater seepage stains along the western boundary of the station box interface observed during the site walkover was noted to be along the second row of anchors, which were installed between RL 8.5 m to 9.5 m AHD. Most the top level of ground anchors, which were installed between RL12.5 m to 13.42 m AHD did not exhibit groundwater seepage stains. However, it is noted that below average levels of rainfall were recorded in Sydney in the months preceding the site visit and has caused the groundwater table to be depressed below original design groundwater levels. Notwithstanding this, due to the high permeability of the sands, the serviceable design (permanent) groundwater level for the station box design was taken at the ground level, as there is potential for groundwater level to rise very quickly during flood events, as per clause 2.3.6 of Appendix B2 of the SWTC. For ultimate limit state design, the groundwater level is understood to be set at the Probable Maximum Flood (PMF) levels, to be confirmed within the SWTC requirements. It is understood that the Waterloo Metro Quarter Development will follow the same design standards as the station box.

Monitoring Bore	Date of construction	Date of last observation	Screened unit	Average groundwater depth (MBGL)	Average groundwater level (M AHD)
SRT_BH403	18/06/2015	June 2016	Sandstone Class I/II	3.4	11.6
SRT_BH404	26/06/2015	June 2016	Sandstone Class I/II	6.0	9.3
SRT_BH405	2/08/2016	September 2017	Sandstone Class I/II	6.7	9.9
SRT_BH406	2/08/2016	September 2017	Sand / Residual Soil	3.1	12.3
SRT_BH605	2/11/2016	May 2017	Shale Class III	4.1	10.8
JCG_BH1120	7/08/2017	September 2017	Shale Class V	5.0	10.4
			Sandstone Class I	9.9	5.5
JCG_BH1121	26/10/2016	September 2017	Sand	3.2	12.2
R469_BH101M	19/10/2015	October 2015	Sand	3.3	12.8
R469_BH102M	19/10/2015	October 2015	Sand	3.0	13.1

Table 4 - Summary of groundwater monitoring locations and observations

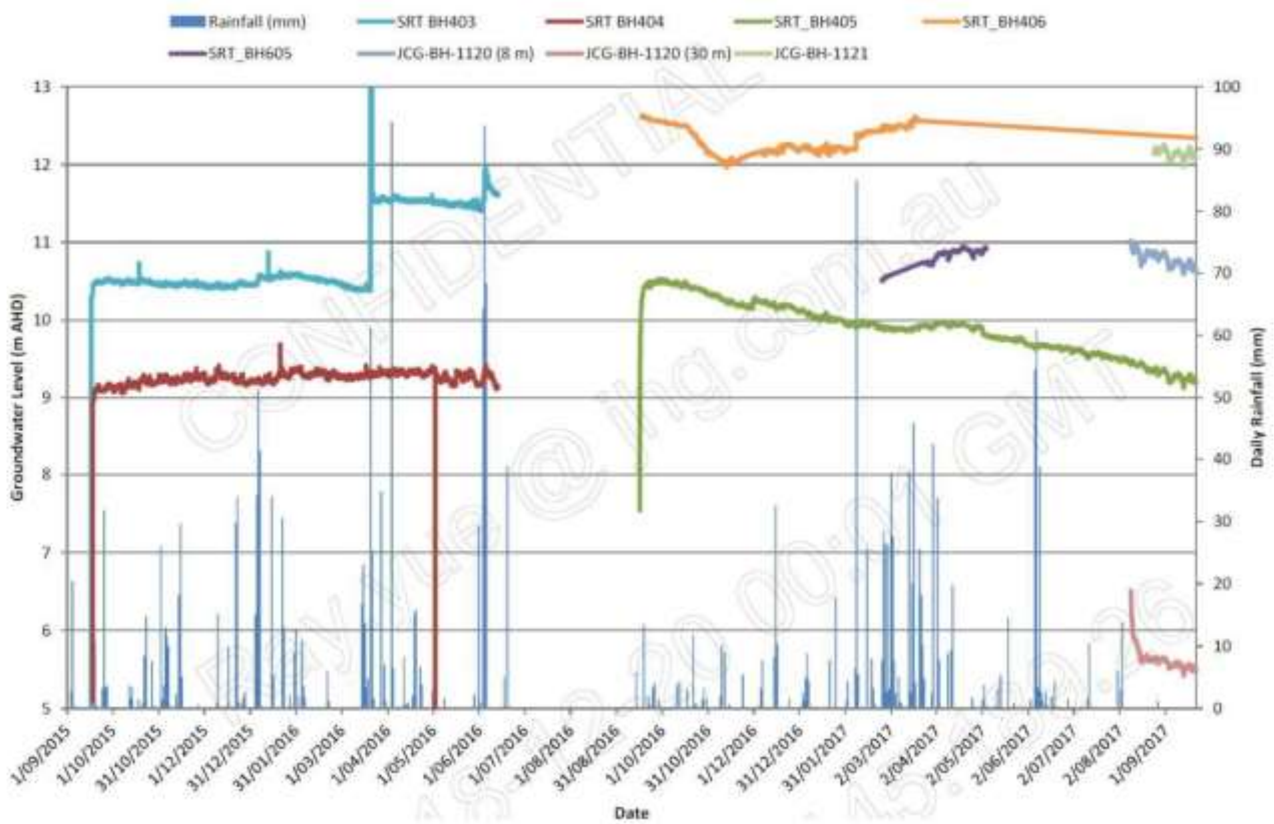

Figure 9 - Groundwater levels and recorded rainfall (sourced from TSE Hydrogeological Interpretive Report, SMCSWTSE-JPS-TPW-GE-RPT-110003)

10. Geotechnical assessment

The portions of the Waterloo Metro Quarter Development project requiring significant geotechnical consideration are the basement underneath BLD 1 and BLD 2, and the foundation and subsurface systems of BLD 3. The other components of the Waterloo Metro Quarter Development, i.e. superstructure of BLD 1, BLD 2 and BLD 4 only interact with the ground via the basement or the existing station box structure.

The elements of the basement structure underneath BLD 1 and BLD 2 which require significant geotechnical consideration include:

- Temporary and permanent retention of the basement walls
- The effect of bulk excavation on the adjacent heritage Waterloo Congregational Church
- Groundwater pressure assessment due to the necessity for the basement structure to be undrained to limit withdrawal of potentially contaminated groundwater from the quaternary sands
- Piled foundations which will be used to support a range of design actions including the overlying superstructures of BLD 1 and BLD 2, as well as any potential uplift forces from the buoyancy of the undrained basement structure
- Construction of subsurface structures along the eastern perimeter of the BLD 1 and BLD2 site, and the impact of construction on the adjacent station box structure
- The Waterloo Congregational Church (heritage item 2069) located at 103-105 Botany Rd, to the south of the Basement Car Park

The elements of BLD 3 requiring significant geotechnical consideration include:

- Piled foundations which will be used to support the superstructure of BLD 3
- Construction of subsurface structures along the eastern perimeter of the BLD 3 site, and the impact of construction on the adjacent station box structure
- The Waterloo Congregational Church (heritage item 2069) located at 103-105 Botany Rd, to the north of the Southern Precinct BLD 3

These elements are discussed further below.

10.1 Basement structure

10.1.1 Bulk excavation

The basement of BLD 1 and BLD 2 will be excavated to approximately 9.5mRL AHD according to WMQ-BLD1-WBG-AR-DRG-A1062 Rev E and is inferred to be within the fill, Botany Sand and residual soil layers, with groundwater expected to be encountered at around 12mRL AHD. As such, conventional earthmoving equipment should be suitable for the bulk excavation and no heavy ripping or rock breaking equipment should be required during excavation.

10.1.2 Site retention

The depth of the excavation is to extend around 8m below the surface of the existing ground level, with the finished surface level of the B2 slab to be at 10.2mRL AHD. The extent of the basement excavation is expected to be built up to the boundary of the station box wall on the east, within 10m of the Waterloo Congregational Church to the south, Botany Road to the west and within 5m of Raglan Street to the north. As such, the retention of the basement excavation will directly affect adjacent properties and buildings. The below table presents the recommended geotechnical design parameters for soil and weathered rock materials that can be adopted for structural design of the basement retention. The parameters are based on information available from the Sydney Metro (SRT) investigations, and experience of similar projects within the Sydney area. A range of parameters is provided in some cases, with the best estimate values shown in brackets.

Material type	Unit weight	Undrained shear strength (Cu)	Effective cohesion (c')	Effective friction angle (ɸ')	Young's modulus (E)	At Rest Earth Pressure (Ko)	Active Earth Pressure (Ka)	Passive Earth Pressure (Kp)
	(kN/M^3)	(kPa)	(kPa)	(°)	(MPa)			
Fill (sandy)	16 – 18 (18)	-	0	30 – 35 (33)	10 – 30 (15)	0.45	0.29	3.39
Quaternary sand (loose to medium dense)	16 – 20 (18)	-	0	30 – 35 (33)	15 – 30 (20)	0.45	0.29	3.39
Residual clay (stiff to very stiff)	20	100 – 200 (150)	10	28	30 – 50 (40)	0.53 (5)	0.36 (1)	2.77 (1)
Shale (Class V)	21	100 – 300 (200)	10 – 20 (15)	27 – 30 (28)	60 – 200 (100)	0.53 (5)	0.36 (1)	2.77 (1)
Shale (Class IV)	22	-	20 – 40 (30)	28 – 32 (30)	100 – 500 (250)	0.53 (5)	0.36 (1)	2.77 (1)

Table 5 - Geotechnical design parameters for retention system design

1. Short term (undrained) Ka and Kp = 1.0 for Residual Clay and Class IV/V Shale.
2. All K values assume level ground conditions above the wall. Higher coefficients would apply where the ground surface slopes above the wall. Lower coefficients may apply with assumed wall friction.
3. Appropriate water pressures should be adopted unless effective drainage at the rear of the wall is provided.
4. Surcharge pressures should be added to earth pressure, where appropriate.
5. At rest (Ko) value is based on an expectation that the excavation of the adjacent station box has reduced the lateral pressure which approaches an active (Ka) value.
6. Soil-structure interaction analyses (finite element or other) is more appropriate to quantify lateral earth pressures which would be effected due to the destressing of the temporary anchors associated with the TSE excavation walls

Groundwater pressures and surcharge from the buildings and infrastructure need to be considered when designing the temporary and permanent retention of the basement. In addition, the ground deformation limits for adjacent buildings, structures and utilities need to be accounted for and checked with anticipated deformation predictions. Specific attention needs to be provided within the vicinity of the Waterloo Congregational Church, which have more stringent deformation limits. These limits are to be confirmed with the structural engineers.

At this stage of the project, the indicative design of the basement retention is 600mm diameter secant piles, which will be constructed via the continuous flight auger (CFA) methodology due to a relatively thick layer of sand underlying the project site. To minimise the inflow of potentially contaminated groundwater into the construction site during excavation and to assist with the subsequent construction of the undrained tanked structure, the piles should either be embedded a minimum of two pile diameters into Class III Shale, i.e. approximately 1mRL AHD, or other groundwater management process be adopted. One or two rows of ground anchors may also be required to be used, depending on the depth of the excavation at each location, to optimise the required thickness of the secant piles and distribute the bending moments across the secant pile wall. If anchors are used, they will likely be required to be destressed or removed prior to completion of construction as they will encroach under the neighbouring properties, i.e. Waterloo Congregational Church, Botany Road or Raglan Street.

Following destressing of the anchors, the lateral loads from the soils which were shored and supported by the temporary anchors will still be required to be propped. As such, the basement slabs or otherwise would be required to withstand these loads. An estimate of the magnitude of these loads can be calculated using the design parameters from Table 5, but finite element analysis should be used to accurately determine the magnitude and distribution of the lateral earth pressures, which are likely to be more pronounced at the locations where the anchors will be destressed. While the station box bounds the basement along its eastern perimeter and no lateral earth pressure will be present here, when the temporary anchors along the eastern perimeter of the station box are destressed (i.e. along Cope Street), the release of lock off loads will be transmitted across the station box on to the walls of the basement structure. As such, these loads will be required to be resisted with minimal resultant deflection to minimise out of balance forces on the station box. To accurately quantify these loads, a staged finite element analysis is recommended to be carried out during detailed design.

At this stage of design, it is understood that two design options are being considered to resist the lateral pressures from the adjacent ground or the lock off anchor loads from the station box:

1. The use of the basement floor slabs and/or beams to act as a prop to transmit the loads across the basement
2. The construction of additional buttresses, which will be piled into the bedrock. The piles may be required to be sleeved until bedrock layer to prevent transmission of stress from the shafts of the pile to the soil and subsequently walls of the station box.

10.1.3 Basement hydrostatic slab

The geological sections indicate that the floor of the basement level will likely be founded on very stiff to hard residual clay material. As such, trafficking of construction plant and machinery over this material will likely reduce the quality of the subgrade over time. To mitigate this, the basement should be slightly over excavated and brought back up to basement soffit level with a layer of blinding concrete. Compacted imported granular fill may not be suitable as a subgrade material for the ground slab as the basement will require a waterproof membrane to be designed as undrained.

Subsequently, as the entire basement will be tanked, the uplift pressure on the basement from hydrostatic pressure should be designed to be withstood by either the weight of the super structure or nett tension action on the foundation piles. The hydrostatic pressure will be present across the base of the ground slab, and should be taken from the appropriate groundwater levels. As the basement is undrained, no drainage will be necessary on the underside of the slab.

10.1.4 Pile foundations

Due to the anticipated magnitude of buildings loads and/or hydrostatic pressures, piled foundations founded within competent bedrock is likely required. The table below presents the recommended geotechnical design parameters for soil and rock materials that can be adopted for design of piles between 450mm to 1500mm in diameter.

Material type / Class		Description	Young's modulus (E)	End bearing capacity		Ultimate shaft adhesion (4)
				Ultimate (2)	**Allowable(3)**	
			(MPa)	**(MPa)**	**(MPa)**	**(kPa)**
Shale	V	Highly fractured, extremely low strength	100	3	0.7	100
	IV	Highly fractured, very low strength	250	5	1	150
	III	Fractured to highly fractured, low strength	500	10	3	500
	II	Fractured, medium strength	1200	70	5	800
	I	Slightly fractured to fractured, medium to high strength	2000	120	8	1000
Sandstone	V	Highly fractured, very low strength	100	3	1	150
	IV	Fractured, low strength	500	10	2	500
	III	Fractured, medium to high strength	1000	30	5	1000
	II	Slightly fractured, medium strength	2000	80	8	2000
	I	Slightly fractured or unbroken, high strength	3000	120	12	3000

Table 6 - Geotechnical design parameters for pile foundations

1 Piles are recommended to be socketed within Class III rock or better

2 Ultimate end bearing pressures occur at large settlement, generally larger than 5% of minimum footing dimensions.

3 Allowable values assume settlement magnitudes of less than about 1% of the foundation width. Parameters are provided as guidance only; detailed analysis shall be calculated for specific structure and layered subsurface ground condition.

4 Assuming clean rock socket of roughness category R2 or better. Shaft adhesion values apply for both axial compression and tension loading.

5 Material units shaded in grey are not anticipated to be encountered within the excavation of the basement or foundations

Adequate shaft resistance to carry ultimate pile loading is anticipated to be mobilised at relatively small displacements (typically less than 1% of pile diameter). For the piles located within the vicinity of the station box, the shafts should be sleeved to prevent transfer of load from the pile to the adjacent soil, which may subsequently transfer load on to the station box, and should be founded at the same level as the base of the station box (approximately RL -10.4m).
For piles loaded in nett tension, a 30% reduction of shaft resistance should be applied, based on a sensitivity study recommended by GEO Publication No.1/2006 (Foundation Design and Constriction, published by the Hong Kong Government). Pells et al (1998 & 2019) also defines the following potential failure mechanisms, in accordance with AS 2159 – 2009 Clause 4.4.2:

1. "Piston pull-out" – uplift resistance is provided by the ultimate shaft adhesion between the pile and rock.
2. "Cone lift-out" – uplift of a mass of rock around the pile socket. Depending on the quality of the rock mass, the angle of cone apex to be adopted can vary from 30° to 60°.

Following confirmation of design loads on each pile, piles loaded in nett tension should be assessed against each of the aforementioned failure mechanisms.

A geotechnical strength reduction factor (Φg) of 0.52 may be adopted for the design of bored piles, assuming no strength testing will take place. This value is based on an average risk rating (ARR) of 2.79 and a low redundancy system, in accordance with AS 2159 – 2009, as attached in Appendix B. An increased geotechnical strength reduction factor of 0.76 may be adopted by performing static load pile testing to confirm pull-out capacity, allowing 3% testing of all piles. Consideration would need to be given to the size of the reaction beam and amount of steel required within the pile to couple the beam to the pile. Alternatively, high strain dynamic testing (PDA) may also be performed to increase the geotechnical strength reduction factor, and this method is anticipated to be quicker and cheaper than static load testing. The choice of hammer would need to have capacity to apply sufficient energy to mobilise shaft friction.

During construction, there is potential for the ground near the base of the excavation to be disturbed by construction plant and machinery, which may impact the full mobilisation of the shaft adhesion in this area. Other factors which may affect the mobilisation of the shaft adhesion near the pile head include the piling methodology adopted, potential for lateral loads, track record and experience of the piling contractor, final ground conditions at the base level of the slab, exposure of the excavation to weathering elements in between completion of excavation and construction of piles, etc. Considering these risks, the surface area from ground surface level to 1.5 pile diameters depth shall be assumed to be ineffective in providing shaft resistance, as per cl 4.4.1 of AS2159 (2009).

Care should be taken during the drilling of the piles and operation of plant and machinery in the vicinity of the proposed piles. A minimum sidewall roughness class of R2 should be obtained along the sidewalls of the rock socket for the geotechnical resistance to be mobilised along the shaft. This is described as grooves of a depth of 1 to 4 mm, widths greater than 5 mm, at spacings of 50mm to 200 mm (Walker and Pells, 1998). Following completion of drilling of the socket, the socket sidewalls shall be cleaned and be free of soil and/or crushed rock to the extent that the natural rock is exposed over at least 80% of the socket sidewall. This should be inspected and confirmed by a suitably qualified geotechnical engineer prior to insertion of steel reinforcement. Other construction clauses contained within Walker and Pells (1998) should be adhered to, unless specifically stated otherwise by the engineer.

10.1.5 Impact assessment on the church

An impact assessment on the Waterloo Congregational Church, currently located to the south of the Basement, is recommended to be undertaken considering the proximity of the basement structure, as well as the station works which will have been undertaken by the time of the basement excavation. This is anticipated to be undertaken during the detailed design stage and is required to ensure that ground deformation is engineered to be within tolerance limits. Two - dimensional finite element analyses should be sufficient to address the anticipated movement associated with the construction of the basement.

It is understood that the edge of the Basement will be offset from the church by at least 11m. A total displacement limit of 15mm for the church has previously been assumed during the TSE stage, but this limit should be confirmed by structural engineers to ensure non-damage to the structure of the church. Further acceptance criteria should be procured following a building assessment of the church structure, i.e. total, lateral, differential displacement limits. Displacement which has occurred during the TSE and station box construction stages should also be considered within this assessment.

The critical stages of the impact assessment will likely occur during the bulk excavation of the basement and during the destressing stage of any temporary anchors. As such, instrumentation and monitoring will be required during excavation and throughout the construction stage to monitor displacement, vibration and groundwater, and further ensure non-exceedance of the displacement thresholds. In the event of exceedance of displacement thresholds, excavation and/or construction works in the vicinity should stop immediately and the cause of the movement is to be identified and managed. Further details on the identification and mitigation of excessive ground movement are to be captured within a separate Instrumentation & Monitoring Plan.

10.2 BLD 3 Foundation system

10.2.1 Pile foundations

Similarly, with the basement structure underneath BLD 1 and BLD 2, pile foundations will be required to support the superstructure of BLD 3. The geotechnical design parameters for pile foundations contained within Table 6 are suitable for this site for design of piles between 450mm to 1500mm in diameter. Piles located near the station box may need to be sleeved to prevent transfer of load on to the station box.

A geotechnical strength reduction factor (Φg) of 0.52 may again be adopted for the design of bored piles, assuming no strength testing will take place, with further details available in section 10.1.4 and Appendix B. As BLD 3 is not anticipated to contain any significant basement structures, the pile foundations will only be loaded in compression and the 30% reduction factor for piles in tension does not need to be applied. However, the other recommendations contained within section 10.1.4 would apply for these foundations as well. In addition, the piles underneath BLD 3 are expected to be installed through the Botany Sand layer, and at this stage, suitable piling options would include the following:

- Cast in-situ reinforced bored piles with temporary casing – due to the presence of groundwater and sand profiles, bored piles would require casing over the sand length within the sand layer to prevent collapse of saturated sands during pile installation. Should groundwater flow, seepage or surface runoff be encountered within the pile excavation, the hole should be dewatered and debris removed from within the hole prior to concrete pour.
- Continuous Flight Auger (CFA) piles – CFA piles can typically be installed quickly with lower noise and vibration compared to bored and driven piles. However, construction of

CFA piles is usually associated with deviation in verticality, with potential for pile necking and honeycombing and requires strict quality controls during the construction stage

Various substructures, such as the lift core pit and other plant room are anticipated to be located directly adjacent to the station box. The pile foundations supporting these substructures need to be designed such that they do not allow any transfer of load to the subgrade, as this would exert additional surcharge pressures on the station box. Pells et al. (2019) provides recommended pressures and methodology which should be adopted to minimise settlement to <1% of the foundation width.

10.2.2 Ground slab

The ground slab of BLD 3 is at 16.4mRL AHD with the surrounding footpath at approximately 15mRL AHD, and thus, approximately 1m of fill is required. Again, trafficking of construction plant and machinery over this material will likely reduce the quality of the subgrade over time. To mitigate this, the subgrade should be slightly over excavated and brought back up to final subgrade level with a layer of blinding concrete or compacted granular fill, with the appropriate drainage installed underneath.

10.2.3 Impact on church

No significant excavations are proposed underneath BLD 3, and as such excessive deformation of the ground is not expected. However, construction of piles and tracking of heavy machinery close to the church may cause excessive noise and vibration to the structure. An appropriate noise and vibration mitigation strategy should be adopted and contained within a Construction Methodology Statement to ensure that the church structure is not subject to excessive vibration and noise. In addition, noise and vibration instrumentation along the church boundary should be installed and monitored during the construction stage, as further detailed in section 10.5.

10.3 Seismic design

The Waterloo Metro Quarter Development is located within the Sydney Metropolitan area of the Sydney Basin, which is known to experience infrequent and minor levels of seismicity compared to other regions around the world. The area where the project site is located, in particular, is known to experience lower levels of earthquake activity compared to the southern and western regions of the basin, closer towards the Blue Mountains. This is supported by empirical data, which record that no earthquakes with a magnitude greater than M_L 3.0 have occurred within a 20km radius of the Waterloo project site. Furthermore, a paleoseismological study by Clark (2010) estimates that earthquake magnitudes in the region of M_L 7.0 along the west and south of Sydney typically have average recurrence period of between 1 to 2 million years. While there are faults located near the project site, there is no known evidence of activity within these faults in recent history.

10.3.1 Site subsoil class

Based on the review of the geotechnical data available, the ground underlying BLD 3 and the basement of BLD 1 and BLD 2 can be classified as Class C_e according to AS1170.4. This is based on the layer of residual soil or highly weathered rock overlying competent rock to be greater than 3m thick at the basement of BLD 1 and BLD 2, while the ground slab level of BLD 3 is at approximately the ground surface.

10.3.2 Geotechnical seismic loading

Seismic design is commonly approached for two different levels of severity: The Maximum Design Earthquake (MDE) and the Operating Basis Earthquake (OBE), otherwise known as the Serviceability Limit State (SLS) or Ultimate Limit State (ULS). Australia does not currently have an individual standard for underground structures. AS1170.4 - 2007 Earthquake Actions in Australia and AS 4678 – 2002 Earth-Retaining Structures are therefore used for design guidance but there is no specific definition of MDE and OBE for underground structures.

Hashash et al (2011) and Wang (1993) define the MDE as the event with a small probability of exceedance during the life of the facility (for example 3 to 5 percent) and the OBE as the earthquake that can be expected to occur at least once during the design life of the facility with probability of exceedance between 40 and 50 percent. These definitions are adopted for this preliminary assessment of design earthquakes.

According to SMCSW-RBG-SWL-ST-REP-120003, the buttress system of the BLD 1/2 basement have to be designed to the same standards as the adjacent Waterloo Station box, i.e. a 100-year design life and Importance Level of 4 (IL4), as per clause 2.2.1 of Appendix B2 of the SWTC. However, other components of the Waterloo Metro Quarter Development are to be designed for a 50 year design life and an Importance Level of 3.

For an IL4 structure, the MDE can be assumed as the earthquake with a return period of 2500 years, equivalent to an earthquake with approximately 4 percent probability of exceedance in 100 years. IL4 structures shall also remain serviceable for immediate use following the design event associated with IL2 structures, as per section 2.2 of AS1170.4. As such, the OBE can be assumed as the earthquake with a return period of 500 years. IL3 structures are to be design to an earthquake with a reduced return period of 1000 years, but both IL3 and IL4 are to be designed according to EDCIII as per AS1170.

Despite the lack of specific definitions for the MDE and the OBE in AS1170.4 and AS4672, design earthquakes of different return period in terms of horizontal peak ground acceleration (PGA) on bedrock can be implicitly estimated by factoring the hazard factor (Z) by the probability factor (kp, equivalent to earthquake return periods) and by considering the site sub-soil classification.

Seismic loading for geotechnical design has been estimated and the following inputs are to be used to obtain the design geotechnical seismic loads:

Parameter	Input	
Site subsoil class	C_e	
Importance Level (IL)	4	3
Design life	100 years	50 years
Annual probability of exceedance (OBE)	1/500	1/25
Annual probability of exceedance (MDE)	1/2500	1/1000
Spectral shape factor ($C_h(T)$)	1.3	1.3
Hazard Factor (Z, Sydney)	0.08	0.08
Probability factor (k_p) (OBE)	1.0	0.25
Probability factor (k_p) (MDE)	1.8	1.3
Unweighted design PGA (OBE)	0.10	0.03
Unweighted design PGA (MDE)	0.19	0.14

Table 7 Geotechnical seismic loading inputs

Detailed seismic loads on the basement structure should be deduced via finite element modelling, as summarised in section 10.1.

10.4 Vibration

Demolition works, basement excavation and construction of ground structures such as the installation of ground retention structures and construction of pile foundations are not expected to result in excessive levels of vibrations, i.e. a Peak Particle Velocity (PPV) at the site boundaries of 10mm/s or less. Vibration would need to be monitored as per the recommendations set out in section 10.5.

10.5 Instrumentation and monitoring

Geotechnical instrumentation has been installed by the TSE contractor around the Waterloo Station site during station box excavation, as per the SMCSWTSE-JCG-TPW-GE-DRG-048716 drawing. These include:

- Open standpipe piezometers in the vicinity.
- Vibrating wire-tip piezometer adjacent to the heritage church and Botany Road.
- Surface settlement monitoring array along Cope Street, Wellington Street and Raglan Street.
- Extensometers along Cooper Street.
- Vibration meter on the north-east corner of the heritage church.
- Electronic tilt meters installed on the heritage church.
- Additional prisms installed on the heritage church.

It is recommended that as-built installation reports, baseline reports and all the monitoring data at these monitoring locations be requested. If possible, these existing instrumentation and monitoring could be reused to monitor displacement, vibration and groundwater during the bulk excavation of the basement. A gap analysis is then recommended to deduce whether additional instrumentation and monitoring is required for the excavation of the basement, and during the piling of BLD 3 in the vicinity of the heritage church and station box. Use of telemetry and in-place instruments may be considered to minimise impacts on construction operations and in-field labour for data collection. In addition to providing monitoring information, the instrumentation will give assurance that the actual behaviour is as predicted and will provide an early warning system if the measurements exceed pre-determined levels.

The instrumentation and monitoring requirements for the construction of the Basement and BLD 3 are to be further detailed within a separate Instrumentation & Monitoring Plan.

11. Authority approvals

Roads and Maritime Services approval is required as the proposed development is adjacent to their road infrastructure. As advised by Roads and Maritime, the following geotechnical information documentation are required for this approval process:

- Geotechnical investigation report.
- Geotechnical assessment report.
- Geotechnical monitoring plan. (to be completed within detailed design)

Refer to following Roads and Maritime's technical direction for more detail:

- RMS Technical Direction GTD 2012_001 - 27 April 2012 (http://www.rms.nsw.gov.au/business-industry/partners-suppliers/documents/technical-directions/gtd_2012-01.pdf) for details.

As there is potential for the groundwater to be contaminated, appropriate disposal according to Sydney Water is required. This is generally as per section 105 of the Contaminated Land Management Act 1997, and is detailed further within *Guidelines for the assessment and management of groundwater contamination.*

It is also understood that there are strict displacement and vibration limits in place regarding the adjacent heritage Waterloo Congregational Church on Botany Road. Compliance requirements and threshold limits should be consulted with the local authorities and compared with received instrumentation and monitoring data from the construction of the station box to ensure non-exceedance of these limits.

12. Conclusion

This geotechnical interpretive report has summarised the existing geotechnical data pertaining to the Waterloo Metro Quarter Development, specifically the Southern Precinct and the Basement, and to provide information on the ground model and geotechnical design parameters to inform the structural design of Metro Quarter Development. This was undertaken based off existing geotechnical data available and a previous site walkover of the TSE excavation. The other Precincts of the Waterloo Metro Quarter Development do not require geotechnical input due to the minimal interaction with the ground.

A series of geotechnical recommendations have been provided for the Basement and BLD 3, summarised as below:

- Basement
  - Conventional earthmoving equipment should be suitable for the bulk excavation and no significant heavy ripping or rock breaking is anticipated during the bulk excavation.
  - A set of geotechnical design parameters have been provided to inform the temporary retention of the secant pile wall, which will likely comprise 600mm diameter CFA secant piles. One or two rows of anchors may be required along the perimeter, with a groundwater management system in place, which will be confirmed during a later design stage.
  - The impact of lateral loads from the excavation, anchor destressing, and/or eventual demolition of the basement on the station box is captured in a separate technical advice note to ensure that loads are within the safe working limits of the station box structure design.
  - The basement is to be designed as an undrained structure, and as such the pile foundations should be designed to withstand buoyancy uplift pressures, as well as superstructure loads, using the recommended pile design parameters.
  - A separate impact assessment from the basement excavation on the Waterloo Congregational Church will be undertaken at a later design stage to ensure any ground deformation which occurs is within tolerance limits.
- BLD 3 Southern Precinct
  - Recommended pile design parameters have been provided to inform structural design of the foundations, which will likely comprise CFA piles or cast in-situ reinforced bored piles with temporary casing.
  - The impact of pile loading on the station box is captured in a separate technical advice note to ensure that loads are within the safe working limits of the station box structure design.
  - Significant ground deformation is not anticipated from the construction of BLD 3, but noise and vibration needs to be monitored, especially when working within the vicinity of the Waterloo Congregational Church.
- Structural elements related to the structural integrity of the Waterloo Station Box are to be designed to the same level of design life and importance level as the Waterloo Station Box.
- The required instrumentation and monitoring for ground movement and vibration will be captured within a separate Instrumentation and Monitoring Plan, which will be completed during a later design stage.

13. Limitations

The geotechnical interpretation presented in this report is based on geotechnical investigation data provided by external third party sources and is at a stage where the specific structural details of the proposed structures are still being confirmed. Once specific development details are confirmed, a geotechnical review should be undertaken and, if necessary, additional investigations commissioned to provide the level of information required for assessing design parameters. The report is provided as a basis to inform design of the structural elements of the proposed structure.

Scope of services

This geotechnical site assessment report (the report) has been prepared in accordance with the scope of services set out in the contract, or as otherwise agreed, between the client and WSP (scope of services). In some circumstances the scope of services may have been limited by a range of factors such as time, budget, access and/or site disturbance constraints.

Reliance on data

In preparing the report, WSP has relied upon data, surveys, analyses, designs, plans and other information provided by the client and other individuals and organisations, most of which are referred to in the report (the data). Except as otherwise stated in the report, WSP has not verified the accuracy or completeness of the data. To the extent that the statements, opinions, facts, information, conclusions and/or recommendations in the report (conclusions) are based in whole or part on the data, those conclusions are contingent upon the accuracy and completeness of the data. WSP will not be liable in relation to incorrect conclusions should any data, information or condition be incorrect or have been concealed, withheld, misrepresented or otherwise not fully disclosed to WSP.

Geotechnical investigation

Geotechnical engineering is based extensively on judgment and opinion. It is far less exact than other engineering disciplines. Geotechnical engineering reports are prepared to meet the specific needs of individuals. A report prepared for a consulting civil engineer may not be adequate for a construction contractor or even some other consulting civil engineer. This report was prepared expressly for the client and expressly for purposes indicated by the client or his representative. Use by any other persons for any purpose, or by the client for a different purpose, might result in problems. The client should not use this report for other than its intended purpose without seeking additional geotechnical advice.

This geotechnical report is based on project-specific factors

This geotechnical engineering report is based on a subsurface investigation which was designed for project-specification factors, including the nature of any development, its size and configuration, the location of any development on the site and its orientation, and the location of access roads and parking areas. Unless further geotechnical advice is obtained, this geotechnical engineering report cannot be used:

- When the nature of any proposed development is changed.
- When the size, configuration location or orientation of any proposed development is modified.

This geotechnical engineering report cannot be applied to an adjacent site.

The limitations of site investigation

In making an assessment of a site from a limited number of boreholes or test pits there is the possibility that variations may occur between test locations. Site exploration identifies specific subsurface conditions only at those points from which samples have been taken. The risk that variations will not be detected can be reduced by increasing the frequency of test locations; however, this often does not result in any overall cost savings for the project. The investigation program undertaken is a professional estimate of the scope of investigation required to provide a general profile of the subsurface conditions. The data derived from the site investigation program and subsequent laboratory testing are extrapolated across the site to form an inferred geological model and an engineering opinion is rendered about overall subsurface conditions and their likely behaviour with regard to the proposed development. Despite investigation the actual conditions at the site might differ from those inferred to exist, since no subsurface exploration program, no matter how comprehensive, can reveal all subsurface details and anomalies.

The borehole logs are the subjective interpretation of subsurface conditions at a particular location, made by trained personnel. The interpretation may be limited by the method of investigation and cannot always be definitive. For example, inspection of an excavation or test pit allows a greater area of the subsurface profile to be inspected than borehole investigation, however, such methods are limited by depth and site disturbance restrictions. In borehole investigation, the actual interface between materials may be more gradual or abrupt than a report indicates.

Subsurface conditions are time dependent

Subsurface conditions may be modified by changing natural forces or man-made influences. A geotechnical engineering report is based on conditions which existed at the time of subsurface exploration.

Construction operations at or adjacent to the site, and natural events such as floods, or groundwater fluctuations, may also affect subsurface conditions, and thus the continuing adequacy of a geotechnical report. The geotechnical engineer should be kept appraised of any such events and should be consulted to determine if additional tests are necessary.

Avoid misinterpretation

A geotechnical engineer should be retained to work with other appropriate design professionals explaining relevant geotechnical findings and in reviewing the adequacy of their plans and specifications relative to geotechnical issues.

Bore/profile logs should not be separated from the engineering report

Final bore/profile logs are developed by geotechnical engineers based upon their interpretation of field logs and laboratory evaluation of field samples. Customarily, only the final bore/profile logs are included in geotechnical engineering reports. These logs should not under any circumstances be redrawn for inclusion in architectural or other design drawings. To minimise the likelihood of bore/profile log misinterpretation, contractors should be given access to the complete geotechnical engineering report prepared or authorised for their use. Providing the best available information to contractors helps prevent costly construction problems. For further information on this matter 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.

Geotechnical involvement during construction

During construction, excavation is frequently undertaken which exposes the actual subsurface conditions. For this reason, geotechnical consultants should be retained through the construction stage, to identify variations if they are exposed and to conduct additional tests which may be required and to deal quickly with geotechnical problems if they arise.

Report for benefit of client

The report has been prepared for the benefit of the client and no other party. WSP assumes no responsibility and will not be liable to any other person or organisation for or in relation to any matter dealt with or conclusions expressed in the report, or for any loss or damage suffered by any other person or organisation arising from matters dealt with or conclusions expressed in the report (including without limitation matters arising from any negligent act or omission of WSP or for any loss or damage suffered by any other party relying upon the matters dealt with or conclusions expressed in the report). Other parties should not rely upon the report or the accuracy or completeness of any conclusions and should make their own enquiries and obtain independent advice in relation to such matters.

Other limitations

WSP will not be liable to update or revise the report to take into account any events or emergent circumstances or facts occurring or becoming apparent after the date of the report.

14. References

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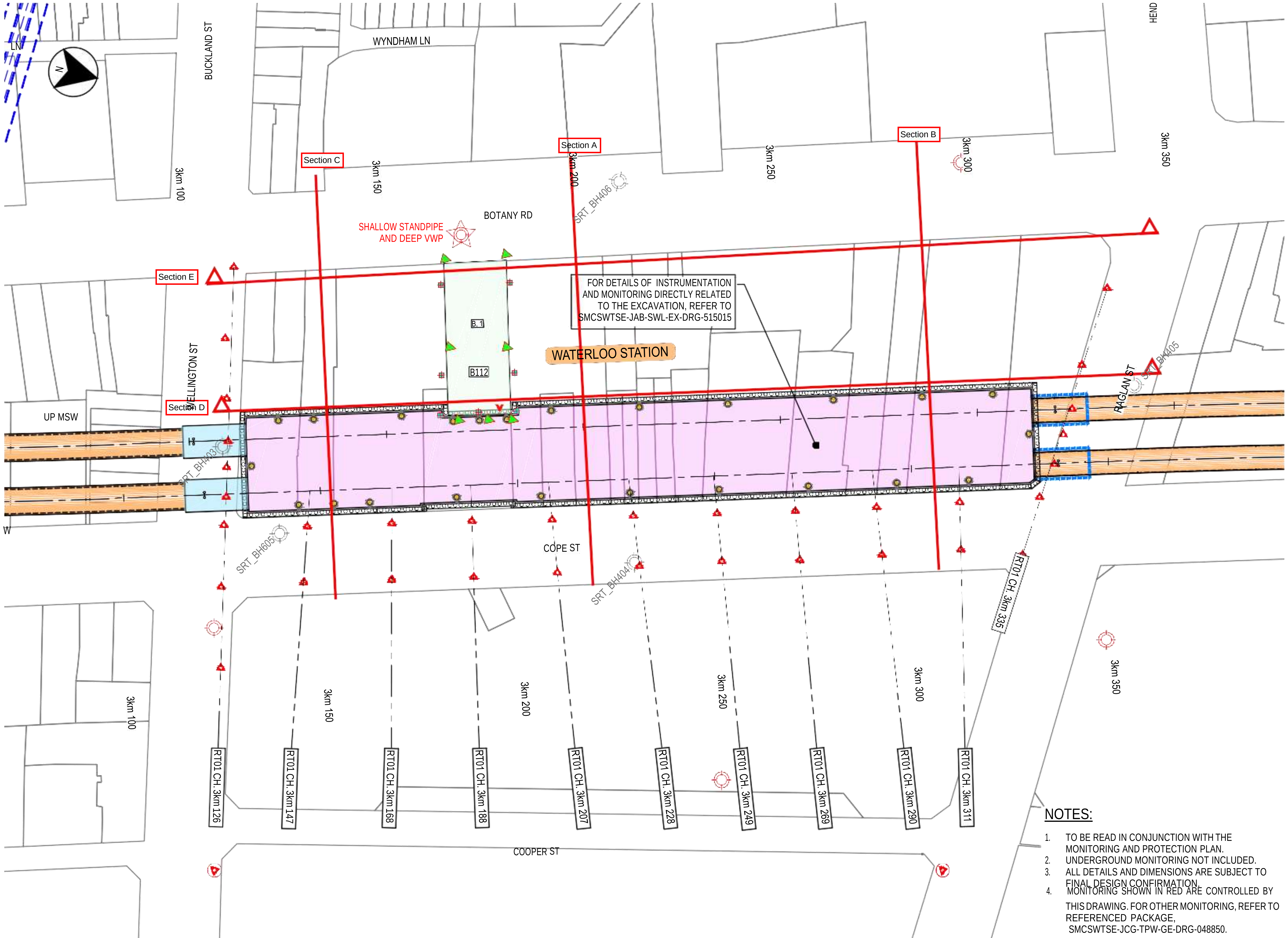
15. Appendices

15.1 Appendix A – Geological cross sections



LEGEND

SRT_BH001

- EXISTING GROUNDWATER MONITORING PIEZOMETER (OPEN STANDPIPE)
- NEW GROUNDWATER MONITORING PIEZOMETER (OPEN STANDPIPE)
- NEW GROUNDWATER MONITORING PIEZOMETER (VIBRATING WIRE PIEZOMETER)
- SURFACE SETTLEMENT MONITORING ARRAY (SURVEY MARKERS) 1D
- BENCH MARK / SINGLE ROD EXTENSOMETER
- VIBRATION METER
- OPTICAL SURVEY TARGET
- BOREHOLE INCLINOMETER
- MULTI-POINT ROD EXTENSOMETER (INSTALLED FROM SURFACE)
- REFER NOTE 4
- TYPE 1 SURFACE RAIL MONITORING
- TYPE 1 RAIL TUNNEL MONITORING
- TYPE 2 RAIL MONITORING
- ROAD TUNNEL MONITORING
- 01 BUILDING MONITORING
- 01 BUILDING MONITORING TO BE CONFIRMED BY THE AFC BUILDING EFFECTS REPORT

BUILDING MONITORING

- ELECTRONIC TILT METER
- TYPE A PRISM - 1 LEVEL AT ~3m ABOVE GROUND LEVEL
- TYPE B PRISM - 2 LEVELS, FIRST AT ~3m ABOVE GROUND LEVEL, SECOND ~15m ABOVE GROUND LEVEL
- TYPE D MONITORING - SPECIFIC OPTICAL PRISM MARKER IN THE BASEMENT

BUILDING NOTATION

A, 1

DENOTES BUILDING MONITORING TYPES. REFER TO DRAWING No. 600051 FOR DESCRIPTION OF MONITORING LOCATIONS

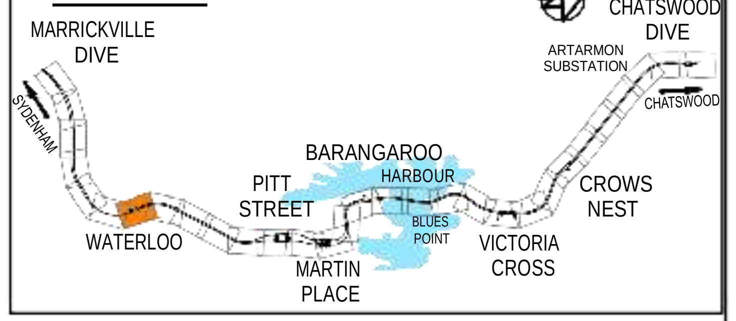
NOTE

1) BUILDING MONITORING POINTS ARE INDICATIVE ONLY FINAL POSITION WILL BE SHOWN ON ASBUILT DRAWINGS.

NOTES:

1. TO BE READ IN CONJUNCTION WITH THE MONITORING AND PROTECTION PLAN.
2. UNDERGROUND MONITORING NOT INCLUDED.
3. ALL DETAILS AND DIMENSIONS ARE SUBJECT TO FINAL DESIGN CONFIRMATION.
4. MONITORING SHOWN IN RED ARE CONTROLLED BY THIS DRAWING. FOR OTHER MONITORING, REFER TO REFERENCED PACKAGE, SMCSWTSE-JCG-TPW-GE-DRG-048850.

KEY PLAN



DRAWING COLOUR CODED - PRINT ALL COPIES IN COLOUR

FOR CONSTRUCTION

AMD	DESCRIPTION	DESIGNER SIGN./DATE	VERIFIED SIGN./DATE	APPROVED SIGN./DATE
01	ASSURED FOR CONSTRUCTION	DW/10/10/18	DW/10/10/18	DW/10/10/18
00	ASSURED FOR CONSTRUCTION	DW/26/04/18	DW/26/04/18	DW/26/04/18

CO-ORDINATE SYSTEM: MGA | HEIGHT DATUM: AHD | SCALE: 1:500

SCALES
0 5 10 15 20 25 30m
SCALE 1:500 AT A1 SIZE

IC CERTIFIED - IC CERTIFICATE

CLIENT

Transport for NSW

SERVICE PROVIDERS

DRAWN	N.PHAM	08/10/18
DESIGNED	D.WORSLEY	10/10/18
DRG CHECK	E. NOCKOLDS	10/10/18
DESIGN CHECK	D. WORSLEY	10/10/18
APPROVED	D.WORSLEY	10/10/18

SYDNEY METRO CITY & SOUTHWEST

TSE - PROJECT WIDE

GEOTECHNICAL & HYDROGEOLOGY INSTRUMENTATION & MONITORING

WATERLOO STATION - ENLARGED PLAN

FILE No. | SHEET: 1 OF 1 | A1

STATUS: ASSURED FOR CONSTRUCTION

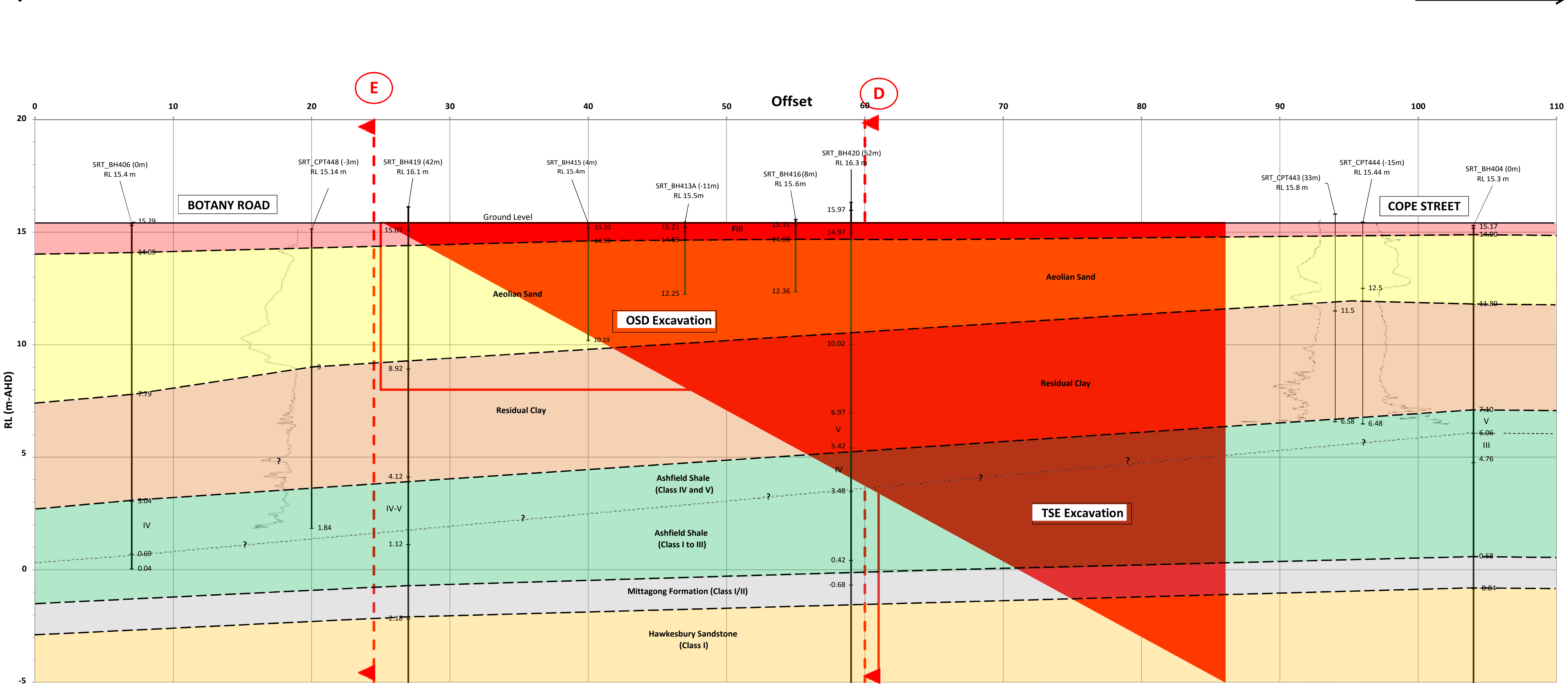
DRG No. SMCSWTSE-JCG-TPW-GE-DRG-048750 | 01 | EDMS No.

DF 801*554

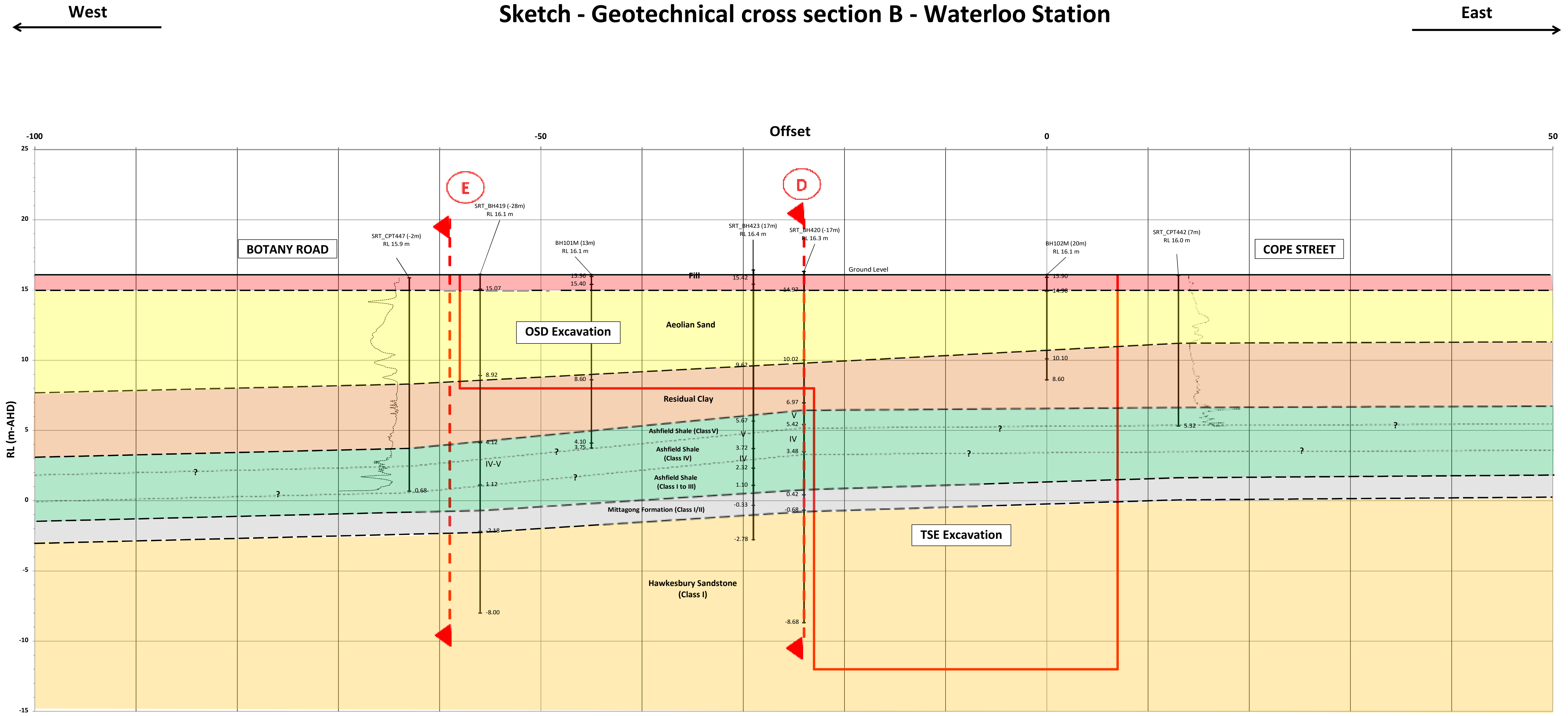
Plotted by LYOGASUTHAN

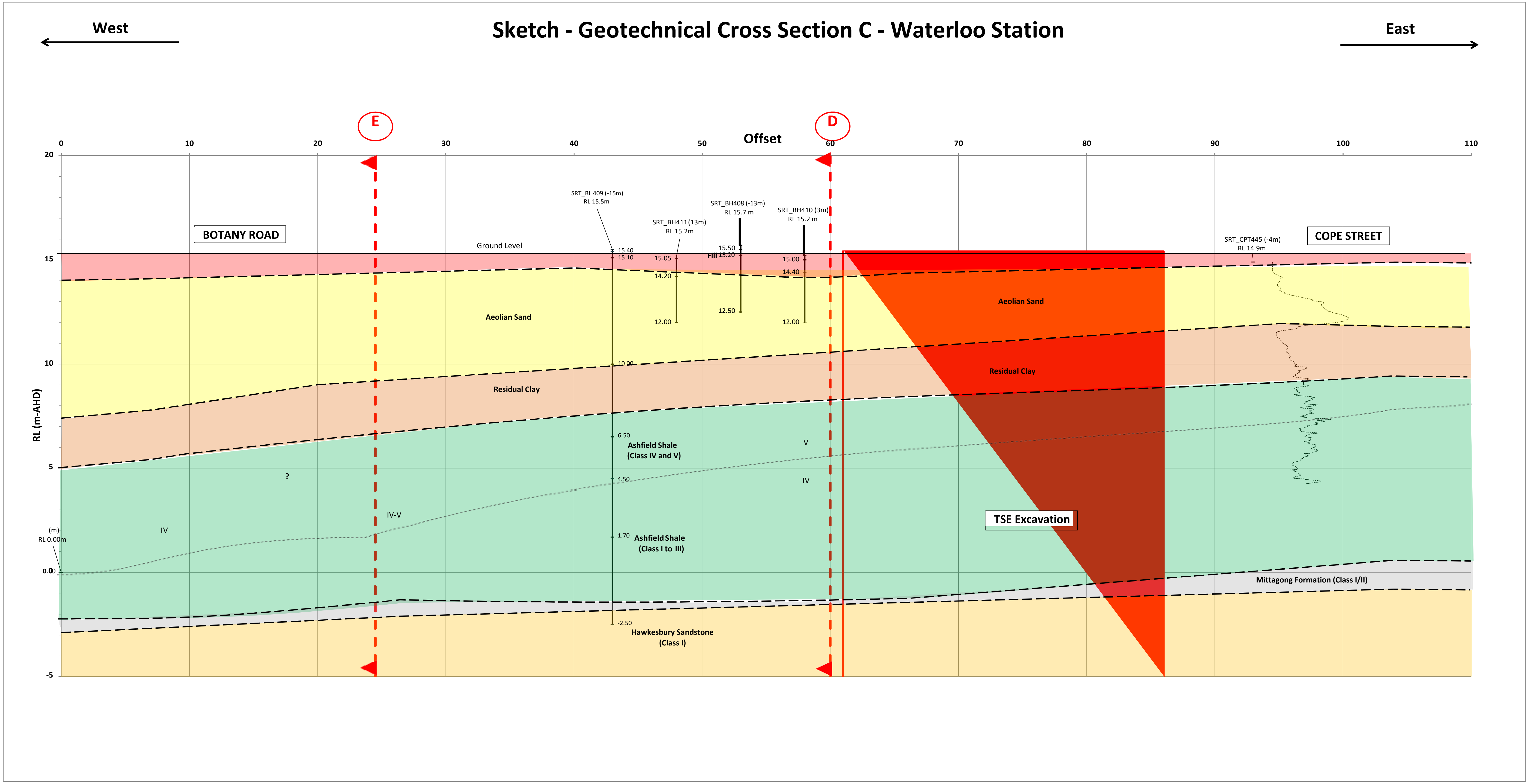
Plot Date & Time 11/2/2018 7:08 PM

Sketch - Geotechnical Cross Section A - Waterloo Station



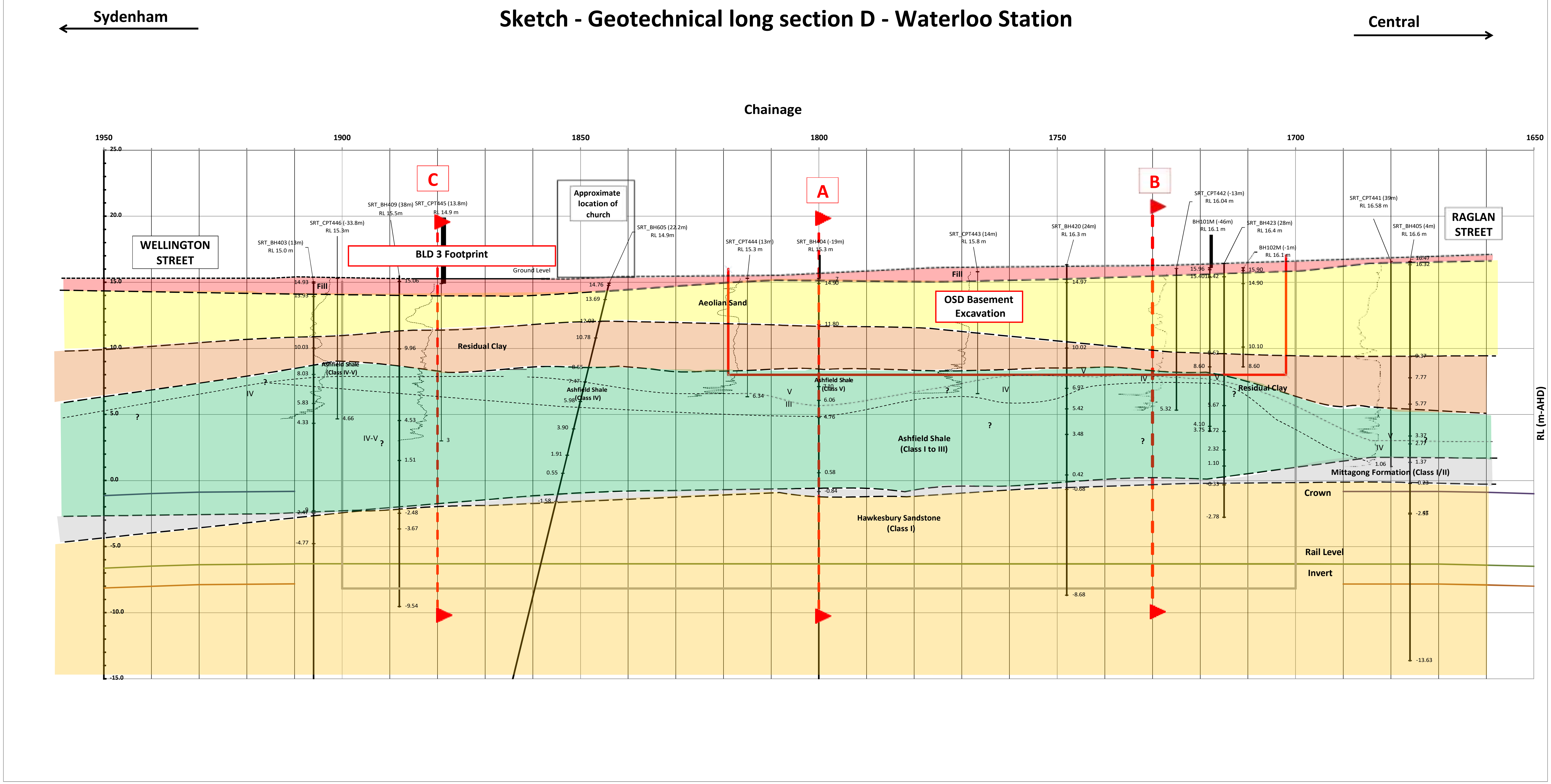
Sketch - Geotechnical cross section B - Waterloo Station



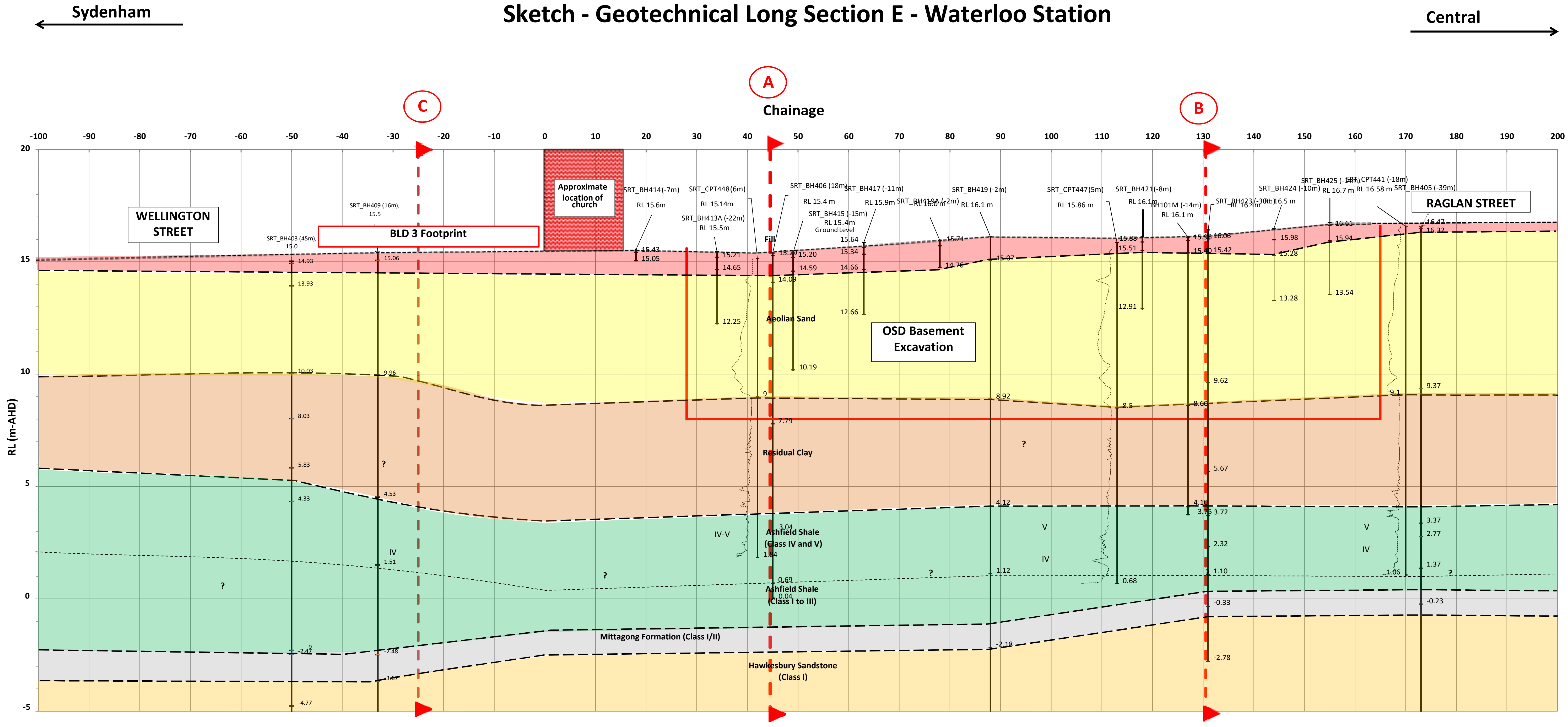


West
Sketch - Geotechnical Cross Section C - Waterloo Station
East
Offset
RL (m-AHD)
E
D
BOTANY ROAD
COPE STREET
Ground Level
Fill
SRT_BH409 (-15m) RL 15.5m
SRT_BH411(13m) RL 15.2m
SRT_BH408 (-13m) RL 15.7 m
SRT_BH410 (3m) RL 15.2 m
SRT_CPT445 (-4m) RL 14.9m
Aeolian Sand
Residual Clay
Ashfield Shale (Class IV and V)
Ashfield Shale (Class I to III)
Mittagong Formation (Class I/II)
Hawkesbury Sandstone (Class I)
TSE Excavation
IV
IV-V
V
?
(m) RL 0.00m

Sketch - Geotechnical long section D - Waterloo Station



Sketch - Geotechnical Long Section E - Waterloo Station



15.2 Appendix B – Average Risk Rating for Pile Design (AS 2159)

4.3.1 Design geotechnical strength

A pile shall be proportioned such that the design geotechnical strength ($R_{d,g}$) is not less than the design action effect (E_d) as detailed in Clause 3.2.2, that is—

$$R_{d,g} \geq E_d \qquad \ldots 4.3.1(1)$$

The design geotechnical strength ($R_{d,g}$) shall be calculated as the design ultimate geotechnical strength ($R_{d,ug}$) multiplied by a geotechnical strength reduction factor (ϕ_g), according to the following equation:

$$R_{d,g} = \phi_g R_{d,ug} \qquad \ldots 4.3.1(2)$$

ϕ_g Calculation Info

The geotechnical strength reduction factor (ϕ_g) shall be determined as follows:

$$\phi_g = \phi_{gb} + (\phi_{tf} - \phi_{gb})K \geq \phi_{gb}$$

where

ϕ_{gb} = basic geotechnical strength reduction factor as given in Clause 4.3.2

ϕ_{tf} = intrinsic test factor

= 0.9, for static load testing (see Section 8)

= 0.75, for rapid load testing (see Section 8)

= 0.8, for dynamic load testing of preformed piles (see Section 8)

= 0.75, for dynamic load testing of other than preformed piles (see Section 8)

= 0.85, for bi-directional load testing (see Section 8)

= ϕ_{gb}, for no testing

K = testing benefit factor

= $1.33p/(p + 3.3) \leq 1$, for static or rapid load testing

= $1.13p/(p + 3.3) \leq 1$, for dynamic load testing

p = percentage of the total piles that are tested and meet the specified acceptance criteria

2 Where there is a satisfactory correlation between static and dynamic tests, ϕ_{tf} may be increased by 0.05.

ϕ_{gb} Information

AS 2159—2009 24

TABLE 4.3.2(C)

BASIC GEOTECHNICAL STRENGTH REDUCTION FACTOR (ϕ_{gb}) FOR AVERAGE RISK RATING

Range of average risk rating (ARR)	Overall risk category	ϕ_{gb} for low redundancy systems	ϕ_{gb} for high redundancy systems
ARR ≤1.5	Very low	0.67	0.76
1.5 < ARR ≤2.0	Very low to low	0.61	0.70
2.0 < ARR ≤2.5	Low	0.56	0.64
2.5 < ARR ≤3.0	Low to moderate	0.52	0.60
3.0 < ARR ≤3.5	Moderate	0.48	0.56
3.5 < ARR ≤4.0	Moderate to high	0.45	0.53
4.0 < ARR ≤4.5	High	0.42	0.50
>4.5	Very high	0.40	0.47

(c) Determine the basic geotechnical strength reduction factor (ϕ_{gb}) from Table 4.3.2(C) depending on the level of redundancy in the piling system. Systems with a high degree of redundancy would include large pile groups under large caps, piled rafts and pile groups with more than 4 piles. Systems with a low level of redundancy would include isolated heavily loaded piles and piles set out at large spacings.

CALCULATION SHEET

Waterloo Metro Quarter Development

Project No. PS119449
Client Ref.
Date: 03-07-20

ϕ_g Calculation according to AS2159-2009, Section 4.

(a) Rate each risk factor in Table 4.3.2(A) on a scale from 1 to 5 for the nature of the site, the available site information and the pile design and installation procedures adopted. This will produce an individual risk rating (IRR) according to the assessed level of risk, as set out in Table 4.3.2(B)

(b) Determine the overall design average risk rating (ARR) using the weighted average of the product of all of the risk weighting factors (w_i) shown in column 2 of Table 4.3.2(A) times the relevant individual risk rating (IRR), as follows:

$$ARR = \Sigma(w_i \mathrm{IRR}_i)/\Sigma w_i \qquad \ldots 4.3.2$$

TABLE 4.3.2(B)

INDIVIDUAL RISK RATING (IRR)

Risk level	Individual risk rating (IRR)
Very low	1
Low	2
Moderate	3
High	4
Very high	5

ARR Calculation

TABLE 4.3.2(A)

WEIGHTING FACTORS AND INDIVIDUAL RISK RATINGS FOR RISK FACTORS

Risk factor	Weighting factor (w_i)	Typical description of risk circumstances for individual risk rating (IRR): 1 (Very low risk)	3 (Moderate)	5 (Very high risk)	w_i	IRR		w_iIRR
Site								
Geological complexity of site	2	Horizontal strata, well-defined soil and rock characteristics	Some variability over site, but without abrupt changes in stratigraphy	Highly variable profile or presence of karstic features or steeply dipping rock levels or faults present on site, or combinations of these	2	3	=	6
Extent of ground investigation	2	Extensive drilling investigation covering whole site to an adequate depth	Some boreholes extending at least 5 pile diameters below the base of the proposed pile foundation level	Very limited investigation with few shallow boreholes	2	3	=	6
Amount and quality of geotechnical data	2	Detailed information on strength compressibility of the main strata	CPT probes over full depth of proposed piles or boreholes confirming rock as proposed founding level for piles	Limited amount of simple in situ testing (e.g., SPT) or index tests only	2	3	=	6
Design								
Experience with similar foundations in similar geological conditions	1	Extensive	Limited	None	1	2	=	2
Method of assessment of geotechnical parameters for design	2	Based on appropriate laboratory or in situ tests or relevant existing pile load test data	Based on site-specific correlations or on conventional laboratory or in situ testing	Based on non-site-specific correlations with (for example) SPT data	2	3	=	6
Design method adopted	1	Well-established and soundly based method or methods	Simplified methods with well-established basis	Simple empirical methods or sophisticated methods that are not well established	1	1	=	1
Method of utilizing results of in situ test data and installation data	2	Design values based on minimum measured values on piles loaded to failure	Design methods based on average values	Design values based on maximum measured values on test piles loaded up only to working load, or indirect measurements used during installation, and not calibrated to static loading tests	2	3	=	6

CALCULATION SHEET

Waterloo Metro Quarter Development

Project No. PS119449
Client Ref.
Date: 03-07-20

ϕ_g Calculation according to AS2159-2009, Section 4.

ARR Calculation (Cont')

Risk factor	Weighting factor (w_i)	Typical description of risk circumstances for individual risk rating (IRR): 1 (Very low risk)	3 (Moderate)	5 (Very high risk)	w_i	IRR		w_iIRR
Installation								
Level of construction control	2	Detailed with professional geotechnical supervision, construction processes that are well established and relatively straightforward	Limited degree of professional geotechnical involvement in supervision, conventional construction procedures	Very limited or no involvement by designer, construction processes that are not well established or complex	2	3	=	6
Level of performance monitoring of the supported structure during and after construction	0.5	Detailed measurements of movements and pile loads	Correlation of installed parameters with on-site static load tests carried out in accordance with this Standard	No monitoring	0.5	3	=	1.5
					14.5			40.5

NOTE: The pile design shall include the risk circumstances for each individual risk category and consideration of all of the relevant site and construction factors.

ϕ_g Calculation

ARR = 40.5 ÷ 14.5 = 2.793

ϕ_{gb} = 0.52 for low to moderate Category low redundancy
{refer to Table 4.3.2(C)}

ϕ_{tf} = 0.9 for Static Testing

= 0.9, for static load testing (see Section 8)

= 0.75, for rapid load testing (see Section 8)

= 0.8, for dynamic load testing of preformed piles (see Section 8)

= 0.75, for dynamic load testing of other than preformed piles (see Section 8)

= 0.85, for bi-directional load testing (see Section 8)

= ϕ_{gb}, for no testing

K = $1.33p/(p + 3.3) \leq 1$, for static or rapid load testing

= $1.13p/(p + 3.3) \leq 1$, for dynamic load testing

p = percentage of the total piles that are tested and meet the specified acceptance criteria

K = 1.33 * 3 / (3 + 3.3) = 0.63333 ≤ 1

p = 3.00

	ϕ_{gb}	+ (	ϕ_{tf}	-	ϕ_{gb}	)	*	K	≥	ϕ_{gb}
ϕ_g =	0.52	+ (	0.9	-	0.52	)	*	0.6333	=	**0.76**

Where the basic geotechnical strength reduction factor is 0.4 or less, no testing is required unless otherwise specified, e.g., for providing the adequacy of construction practices.