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LAMINGTON MARKETS

612 LUTWYCHE RD, LUTWYCHE

STRUCTURAL DESIGN REPORT

NORTHERN BUSWAY IMPACTS REPORT

NOVEMBER 2019



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Lamington Markets
612 Lutwyche Rd, Lutwyche
STRUCTURAL DESIGN REPORT
NORTHERN BUSWAY IMPACTS REPORT

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1 GENERAL PROJECT INFORMATION

1.1 INTRODUCTION

The purpose of this Structural Engineering Impacts Report is to outline the current structural retention system & design criteria for the proposed development for the purposes of verifying there is no impact from the proposed development on Northern Busway Tunnel Structure.

The design criteria in this report is a combination of prescribed Australian Standards and good engineering practice. Our Engineering design brief is to provide an economical structure that is buildable, durable throughout it's design life and to best protect adjacent properties both in the temporary and permanent cases.

The design intent provides a structural scheme which will not impose any greater loads to which the Northern Busway Tunnel has been designed to resist.

Note that the project is still subject to final detailed design, and has not been subject to final building or operational works approval.

This brief is to be read in conjunction with the related documents prepared by the project team.

1.2 PROJECT DESCRIPTION

1.2.1 GENERAL

Generally, the structural framing consists of reinforced and post tensioned concrete slab systems supported by reinforced concrete columns.

Residential tower loads are generally transitioned out at upper floor levels so that the loads fall outside the busway tunnel zone to alleviate heavy vertical loading directly onto the tunnel.

Lift core and stair walls are generally reinforced concrete as the lateral stability elements which are also located outside the tunnel zone.

1.2.2 FOOTINGS

Generally, the primary footing system for the development consists of pad and strip footings. All footings shall be found in natural material. Footings have been designed using bore log information from geotechnical report 87424 by Douglas Partners dated February 2016.

We also referenced and compared this with the DTMR Northern busway geotechnical layout plan No.201/U30/B2 - 547798 & geotechnical long section dwg No. 201/U30/B2 -547801 which are in the location of the Lamington Markets development. The two reports generally align.

1.2.3 RETENTION

The site perimeter retention system consists of a conventional concrete soldier pile and infill shotcrete wall system. Temporary anchors will be installed in piles as excavation progresses. These anchors will be destressed and redundant in the final condition.

The retention system adjacent to the tunnel consists of non-anchored piles.

1.2.4 SITE CONDITIONS

The Geotechnical profile consists of a sloping bedrock layer of Brisbane Tuff, overlain by fill and insitu silty clays. The rock is found between 8 and 18m below the surface, and is identified as being high to very high strength at the base of the excavation.

1.3 DESIGN STANDARDS

The Structures have been designed and certified by a Registered Professional Engineer in Queensland (RPEQ).

The structures have been designed in accordance with the National Construction Code (NCC) as well as the relevant Australian Standards and other documents as follows:

National Construction Code

AS1170.0 Structural Design Actions Part 0 General principles

AS1170.1 Structural Design Actions Part 1 Permanent, Imposed and Other actions

AS3600 Concrete Structures

AS3798 Guidelines on Earthworks for Commercial and Residential Developments

AS2159 Piling Design and Installation

1.4 PREVIOUS MEETINGS / ADVICE

Initial concept sketches were issued to DTMR by KANE Constructions Pty Ltd in May 2015. Comments were received from DTMR which were responded to and then a follow up meeting held to discuss the queries with DTMR and Arup in early June 2015.

At that meeting, the proposed structural intent was generally accepted with any finer details to be closed out during detailed design.

The design information contained within this report provides further details to these initial 2015 concept stage meetings.

Refer Appendix D.

2 SHORING ADJACENT BUSWAY TUNNEL - DESIGN CRITERIA

2.1 KEY BUSWAY TUNNEL INFORMATION

Based on the information provided, key items to note are:

- The tunnel adjacent the site is of cut & cover construction (CC) consisting of bored pile walls propped at the top by the tunnel roof structure.
- Top of tunnel RL varies between approximately RL 24.00 at the South end and RL 25.200 at the north end along the length of the proposed development.
- At the southern end of the site, there is approximately 2.2m between the natural ground surface and the top of the tunnel.
- At the northern end of the site, there is approximately 2.3 m between the natural ground surface and the top of the tunnel

2.2 BUSWAY TUNNEL DESIGN PARAMETERS / RESTRICTIONS

The limitations of loading on the Northern Busway are defined in Airport Link | Northern Busway | EWAG Design manual BC-PBA-GLSTR115-RPT-0100-G-01. An extract of the design restrictions / limitations is below:

8.2 Future Development Load and Excavation Allowance

Except as noted below, the design of all CC-structures shall allow for a surcharge load of 25 kPa to provide for future building or other development as specified in Section 7.2.2 of the Design Requirements. This loading shall be in addition to loading from soil above the structure and shall be applied at the level of the top of the CC-structure roof.

2.3 BUILDING IMPORTANCE LEVEL AND DESIGN LIFE

The design life of the building is to be 50 years. It is classed as a building importance level 3 in accordance with the National Construction Code (NCC) Table B1.2a (Figure 5-1) as specified by the Building Certifier.

All temporary ground anchors used in the development are specified as a minimum design life of 2 years.

Any rock bolts used are to be hot-dipped galvanised or grouted full length to achieve the minimum required design life.

2.4 GEOTECHNICAL

Initial site investigations were initially undertaken by Douglas Partners, with their findings summarised in report 87424 dated February 2016.

Further site investigations and detailed geotechnical analysis of the shoring system will be undertaken in the form of a 2D Plaxis Analysis to check the impact of the proposed staged excavation on the Busway Tunnel.

2.5 DURABILITY

Concrete grade and cover to reinforcement shall be equal to or in excess of the requirements in the relevant standards for the relevant exposure conditions.

All concrete structures shall be designed for the following exposure classifications in accordance with AS 3600 Section 4.

LOCATION	EXPOSURE CLASSIFICATION	MIN F'C	MIN COVER
Surfaces of members in contact with the ground (No Damp Proof membrane)	A1	40 MPa	50mm
Base and Sides of footings	A1	40 MPa	75mm
Exposed surfaces of members	B1	40 MPa	30mm

2.6 WATERPROOFING

No waterproofing system is proposed to the shoring and retention wall, and it is expected that groundwater seepage may be visible on the exposed faces of the shoring wall.

Sub-soil drainage systems will be installed both behind the wall face and in front of the wall face on the floor slab to capture any groundwater that may seep through the wall. The shoring system is designed to be a “drained” system, and has not been designed as a watertight structure.

2.7 DEPTH OF EXCAVATION

Lowest basement excavated level adjacent the Northern Busway Tunnel is RL10.360 locally at the northern end of the site but is RL11.160 for the majority of the length, which also accounts for a drainage layer below the slab estimated to comprise 200mm thick gravel.

A half basement level 6 with excavation level RL8.360 is offset approximately 16m or greater away from the busway tunnel and is therefore not considered to be the critical excavation.

2.8 IN GROUND SERVICES

Several in-ground services are located adjacent to the proposed excavation works. The design of the shoring wall will consider the potential impact of soil displacements and anchor installation on these assets to ensure that no services are damaged due to the proposed excavation and shoring works.

Detailed site investigations have been undertaken to identify the existing services adjacent to the proposed excavation. Refer to Appendix B for more detail.

For permissible clearances for excavation equipment, deflections and permissible vibrations, please refer to specific advice from the service providers.

In the absence of specific advice, differential settlements behind the retention wall have been limited to 1 Vertical: 500.

Lateral Movements

Geotechnical design of the shoring wall will be designed to limit the potential impact of vertical and horizontal soil displacements caused by the excavation and shoring works on adjacent in ground services. Detailed geotechnical analysis will be undertaken to ascertain movements and confirm that there are no negative impacts on external services or adjacent structures.

Ground Anchors

Ground anchors are to be coordinated by the Contractor to maintain minimum clearances to all in ground service and existing structures. Minimum clearances are as follows;

Existing Footings, Piles and Structures: Min 1.5m clear

Electrical services: Minimum 1.5m clear

All other services: Minimum 1.5m clear

2.9 DESIGN LOADING

2.9.1 TEMPORARY LOADING

Temporary loading will occur on the tunnel roof during construction only. The load will be from the dead weight of wet concrete when constructing the structure spanning across the tunnel.

Excavation to construct this structure will result in the removal of 1.2m depth of soil (20 kN/m^3) = 24 kPa relieved pressure above the tunnel. Additional allowable loading on the tunnel roof is 25 kPa, Hence the allowable load from the development on the tunnel is $24 + 25 = 49 \text{ kPa}$.

The new structure concrete beam depths are 1.5m max ($24.5 \text{ kN/m}^3 \times 1.5$) = 37 kPa temporarily loading the tunnel roof until the concrete has cured. Void former is proposed beneath this structure to alleviate inducing any future loading onto the tunnel roof.

2.9.2 HORIZONTAL LOADING

Geotechnical

Horizontal soil loading will be determined by detailed geotechnical analysis with consideration given to site geology and soil strength parameters and determined in accordance with the Douglas Partners report 87424.

2.10 LATERAL MOVEMENT

Shoring Walls Displacement Tolerances

COMPONENT	HORIZONTAL TOLERANCE (PLAN)	OUT-OF-PLUMB / VERTICAL TOLERANCE	MAXIMUM PERMISSIBLE LATERAL DISPLACEMENT	REFERENCE/NOTES
Capping Beam and Shotcrete Facing – Typical walls including cantilevered walls adjacent to streets/footpaths	+/- 75mm	Wall height/75	+/-25mm	Tolerances are per AS2159 and MRTS63. Permissible lateral displacement is based upon limiting differential settlements behind the retention wall.
Capping beam and Shotcrete Facing including cantilevered walls - Adjacent Existing Structures (Northern Busway Tunnel)	+/- 75mm	Wall height/150	+/-25mm	The design of the retention wall adjacent to these structures is to be such that the displacement of the adjacent tunnel is less than the limits prescribed.

Shoring Walls Estimated Displacements

Lateral movement of the shoring wall will be calculated using the additional geotechnical analysis to be undertaken.

2.11 PROPOSED MONITORING REGIME

In order to measure the impact of the project on the adjacent Bus Tunnel, several different monitoring tools are proposed.

Structural monitoring is to be carried out for the duration of the proposed excavation and shoring works to confirm the performance of the wall system and to ensure that the adjacent Bus Tunnel are protected from damage due to differential settlement and vibrations.

Proposed monitoring equipment includes;

- Continuous Automated Vibration Monitoring with continuous recording throughout the monitoring period
- Geotechnical monitoring including in-ground Inclinometers and in-ground peizometers to measure groundwater movements behind the retention wall at key locations
- Total Station Survey with prisms on key assets and structures that are sensitive to movements.

2.11.1 VIBRATION MONITORING

Continuous vibration monitoring equipment is proposed to be installed on the existing tunnel structure to ensure that vibrations due to the proposed excavation and shoring works do not result in structural damage to the tunnel or services

Vibration Limits

Vibration limits to prevent damage to the adjacent Bus Tunnel are based upon review of both international and local standards, and are presented below;

Adjacent Bus Tunnel:

	Vibration Limits Relative to Frequency			
Asset	1 - 10Hz	11 - 49Hz	≥ 50 Hz	Notes
Adjacent Bus Tunnel Structure	≤ 10 mm/s	10mm/s - 19mm/s	20mm/s	Based upon guidelines in DIN4150

Other Assets:

Asset	Vibration Limit (1-80Hz)	Notes
In-Ground Services and Public Areas (Footpaths, Roads, etc.)	25mm/s	Based upon criteria in BS5228-4.

2.11.2 GEOTECHNICAL MONITORING

Several different monitoring tools are proposed for monitoring the geotechnical behaviour of the proposed excavation. These include;

In-Ground Inclinometers

An Inclinometer is a sensor that is lowered into a bore hole drilled into the ground and is used to measure small variations in ground movements over time. Inclinometers are very sensitive and are able to pick up minute displacements in the soil profile behind the retention wall. These measurements are then used to verify the stresses in the soil profile behind the retention wall and is a valuable correlation tool to verify that the retention wall is performing as expected.

Total Station Survey

Covered in more detail below, displacements of the retention wall and key assets adjacent to the proposed excavation will be tracked over time with a total station survey system. This measures the physical displacement of the structures.

Timing

Geotechnical monitoring is to commence prior to excavation commencing on site and continue until the temporary ground anchors are released.

2.11.3 TOTAL STATION SURVEY

A total station survey system measures the displacement of specific monitoring points and is used to track displacements over time. This monitoring is to be used throughout the duration of the excavation works to ensure movements of the retention system and adjacent Bus Tunnel Structure are within the prescribed limits.

Total station survey will be undertaken by a registered surveyor using an automatic system, with readings taken daily during excavation works. Readings are to be compiled in a concise report format by the project team and issued for analysis and review monthly.

Timing

Total station monitoring is to commence prior to excavation commencing on site and continue until temporary ground anchors are released.

Trigger Levels

Displacements of the retention wall will occur as the wall is progressively excavated and anchors are stressed. Preliminary geotechnical analysis has been undertaken to confirm the expected displacements, and these have formed the basis for the limits below.

Note that although deflections of the retention wall are related to movements of structures and services adjacent to the wall, they are not necessarily in direct proportion (ie, a 10mm lateral movement of the retention wall may not correlate directly to a 10mm displacement of the structures behind the wall). As such, the deflection limits below relate directly to the retention wall in isolation and WSP defers to the specific deflection limits for the adjacent structures separately.

Trigger Level	Green (Normal)	Amber (Proceed with Caution)	Red (Stop work and rectify)
Displacement	+/-15mm	+/-16mm – 25mm	>26mm

2.11.4 TRIGGER LEVELS AND PERMISSIBLE MOVEMENTS

Trigger Level	Green (Normal)	Amber (Proceed with Caution)	Red (Stop work and rectify)
Description	Structural displacements and/or vibrations are within the expected range of permissible movements.	Structural displacements and/or vibrations have exceeded normal operating range, but are less than the limits nominated in the “Red” trigger level. Adjacent structures may have been affected and should be inspected	Structural displacements and/or vibrations have exceeded the safe limits and works are to stop immediately until the source of the vibration/displacement is controlled.
Action required	Construction progresses with ongoing monitoring continuing.	Temporarily halt construction works within 20m of identified alert area. Conduct visual inspection of area identified by alert and confirm if any damage has occurred. Works to commence once source of displacement/vibration is identified and mitigated as required	Halt construction works within 50m of identified alert until source of movement/vibration is isolated and controlled. Provide temporary stability measures as required to prevent further displacement or damage occurring. Conduct visual inspection of area identified by alert and arrange for inspections by asset owner and/or project engineer to confirm extent of damage. Works to recommence only once asset owner or site engineer have given approval.

3 STRUCTURAL RETENTION DESIGN ADJACENT NORTHERN BUSWAY TUNNEL

3.1 STRUCTURAL CONCEPT

A key component of this project is the construction of underground carparking adjacent to the Northern Busway tunnel to service the proposed development requiring excavation of the basement structure to critical case of RL11.160.

The proposed structural concept and excavation methodology is designed to be completely independent of and protect the existing busway structure, and maintain operation of the tunnel throughout construction. There is no requirement for any new structure to interfere or be located within the Busway Tunnel structure.

It is proposed to install a piled retention wall offset along the length of either side of the tunnel to support a new structure spanning across the tunnel width thereby surrounding and creating an independent 'gantry' to the tunnel.

In the final state, all vertical load from the new development is directed and supported by the new structure. Surcharge loading is supported by the new piled wall adjacent to each side of the tunnel thereby alleviating any new load on the existing tunnel structure.

The 'gantry' is independent of the existing tunnel and the design allows for future modification to be carried out on the tunnel without reliance of the new development and vice versa.

The new structural system consists of reinforced concrete systems that maximises the life of the structure to a design life exceeding 50 years.

Reinforced concrete elements ensure a low maintenance finish. Maintenance (if required) can be done within the new development.

3.2 RETENTION SYSTEM SELECTED

The most appropriate retention system selected for this site consists of:

The Site Perimeter

Conventional temporary anchored soldier pile and shotcrete infill panel system.

Adjacent the Busway Tunnel

Non anchored free spanning soldier pile system using the anchored soldier pile wall along Lutwyche Rd as a 'deadman' anchor wall.

Permanent Restraint

Permanent lateral restraint of the shoring wall is to be provided by the wall facing spanning vertically to the basement floor slabs.

Temporary Restraint

If it is envisaged that permanent wall restraint via the new structural floor slabs takes longer than expected, (exceeding the temporary anchor design life), the anchor sub-contractor is to visually inspect each ground anchor head and carry out necessary testing (such as lift-off testing) of all anchors to confirm the condition of the anchors are satisfactory and the design loads are still being achieved. Depending on the results of the inspection and testing, re-stressing of various anchors may be required or additional anchors be installed to ensure the wall continues to perform as intended.

The anchor sub-contractor is to implement an agreed inspection and maintenance scheme to regularly check the anchors are performing as intended.

Locking off floor plates

In order to limit the impact of shrinkage of the floor slabs on the wall movement, floor plates are to be structurally connected to the shoring wall a minimum of 180 days after pouring. Expected additional long-term lateral movement of the floor slabs after locking off the temporary movement joint is 5-10mm.

Connection of the floor plate to the wall is to be provided by temporary movement joints. Reinforcing starter bars for the future connection of the structural floor slabs are to be installed to the face of the retention wall by the shoring contractor during construction of the retention wall.

Destressing Temporary Anchors

Anchors will be fully destressed once the permanent restraining basement floors have been cured and locked off.

3.3 CONSTRUCTION CONSTRAINTS

3.3.1 BASEMENT EXCAVATION

The proposed site is underlain by fill and insitu soils over weathered Brisbane Tuff.

The insitu soils and moderately weathered layers of this material are expected to be excavated via conventional excavators with rockbreakers and heavy dozers with ripping required to help break up the insitu rock.

Between 8-18m below existing ground (Approx RL 18 to RL 10) is identified as being slightly weathered, very high strength, and may require controlled blasting to allowed for efficient excavation rates.

This process is common for deep basements in the Brisbane, and has most recently been undertaken on the Queens Wharf project adjacent to several significant heritage structures without incident. The blasting operation is designed with pre-drilled holes around the perimeter of the excavation to limit vibrations on adjacent structures and over break of the rock, preserving the quality of the adjacent rock mass and so ensuring that adjacent properties are protected. Vibrations during the blasting are carefully monitored and the blast controlled such that vibrations remain below the pre-agreed limits identified in section 2.11 of this report.

3.3.2 NORTHERN BUSWAY TUNNEL

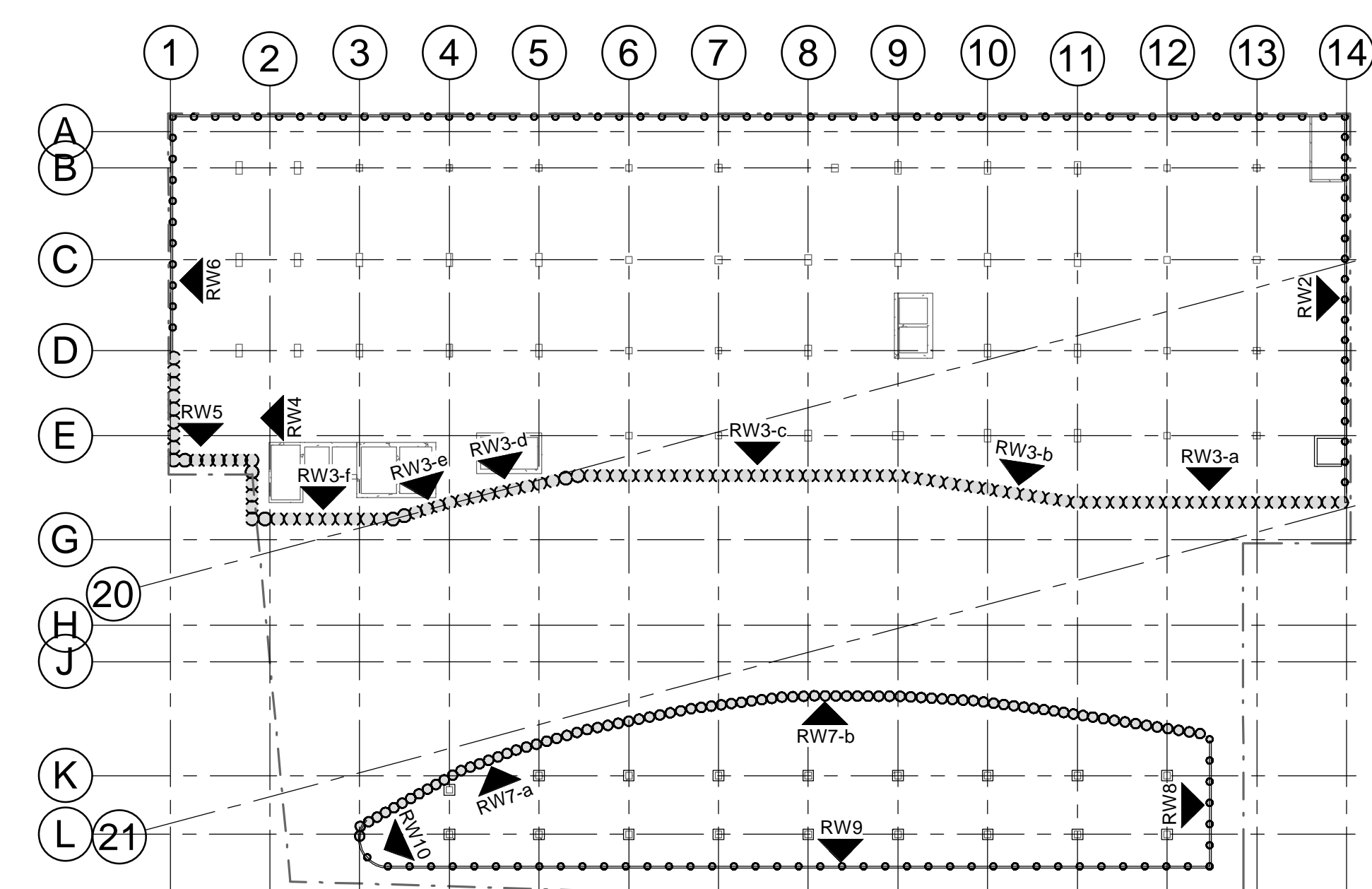
The Northern Busway Tunnel runs through the proposed site and the basement shoring system has been designed to enclose and confine the existing rock and soils around the existing tunnel structure to ensure that the tunnel is not impacted by the proposed excavation works.

Refer to the attached proposed construction staging included in Appendix A of this report for the proposed excavation staging adjacent to the tunnel.

APPENDIX A –

SUPPORTING STRUCTURAL DRAWINGS





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Ground Floor

Basement 1

Basement 2

Basement 3

Basement 4

Basement 5

Basement 6

1800W x 900D CAPPING BEAM

1800W x 900D CAPPING BEAM

1800W x 900D CAPPING BEAM

1800W x 900D CAPPING BEAM

1800W x 900D CAPPING BEAM

1200mm DIA BORED PIER CONTIGUOUS PILE WALL
N40 CONCRETE, 250kg/m³ REO RATE

1200mm DIA BORED PIER CONTIGUOUS PILE WALL
N40 CONCRETE, 250kg/m³ REO RATE

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N40 CONCRETE, 250kg/m³ REO RATE

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N40 CONCRETE, 250kg/m³ REO RATE

1200mm DIA BORED PIER CONTIGUOUS PILE WALL
N40 CONCRETE, 250kg/m³ REO RATE

RW3-a
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RW3-b
SCALE 1:100

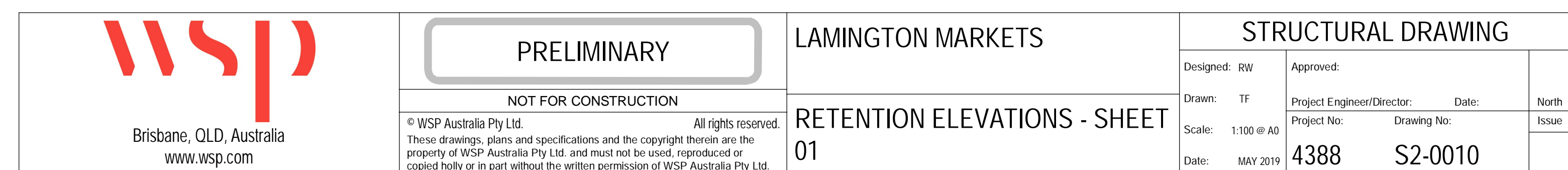
RW3-c
SCALE 1:100

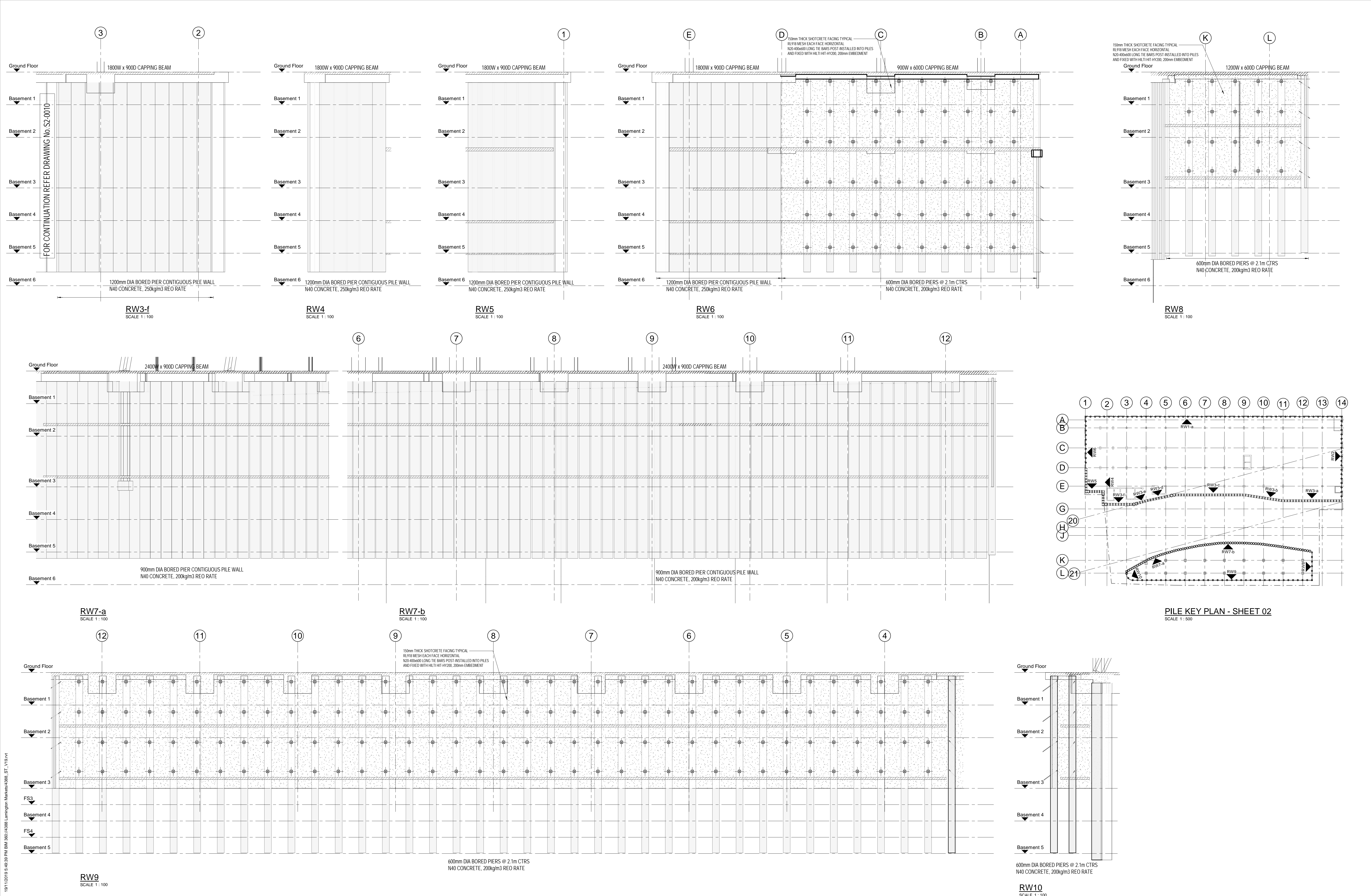
RW3-d
SCALE 1:100

RW3-e
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FOR CONTINUATION REFER DRAWING No. S2.0011

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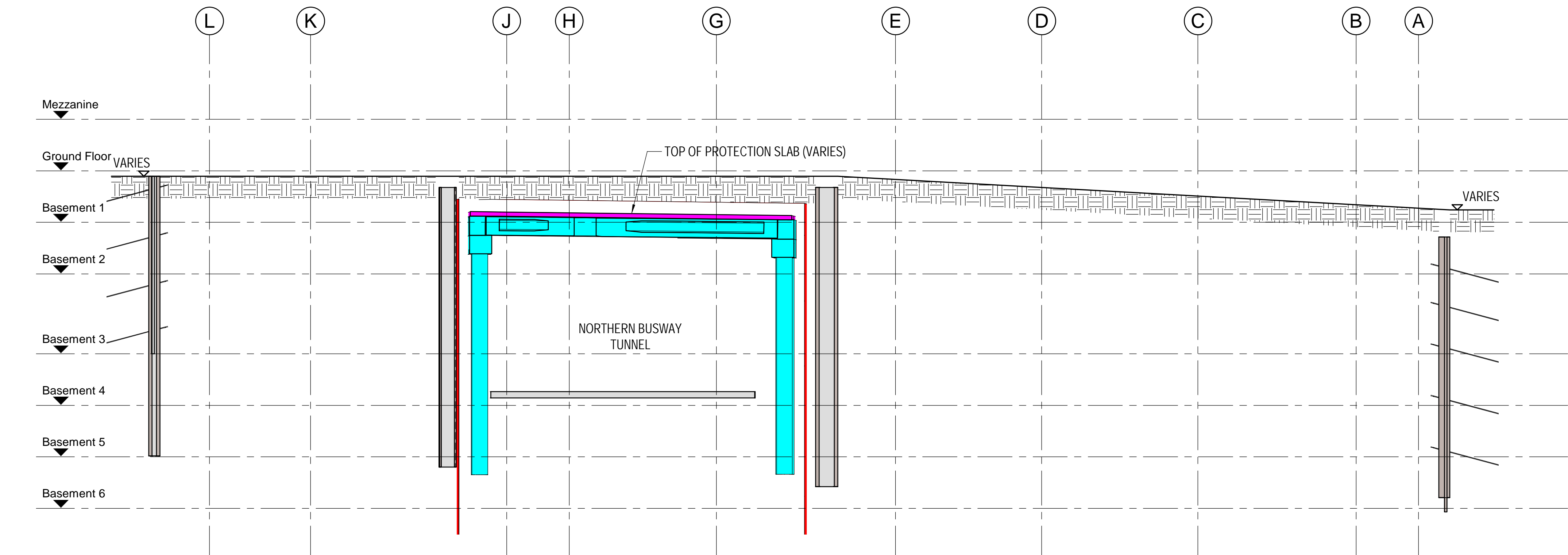
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RETENTION ELEVATIONS - SHEET 02

STRUCTURAL DRAWING

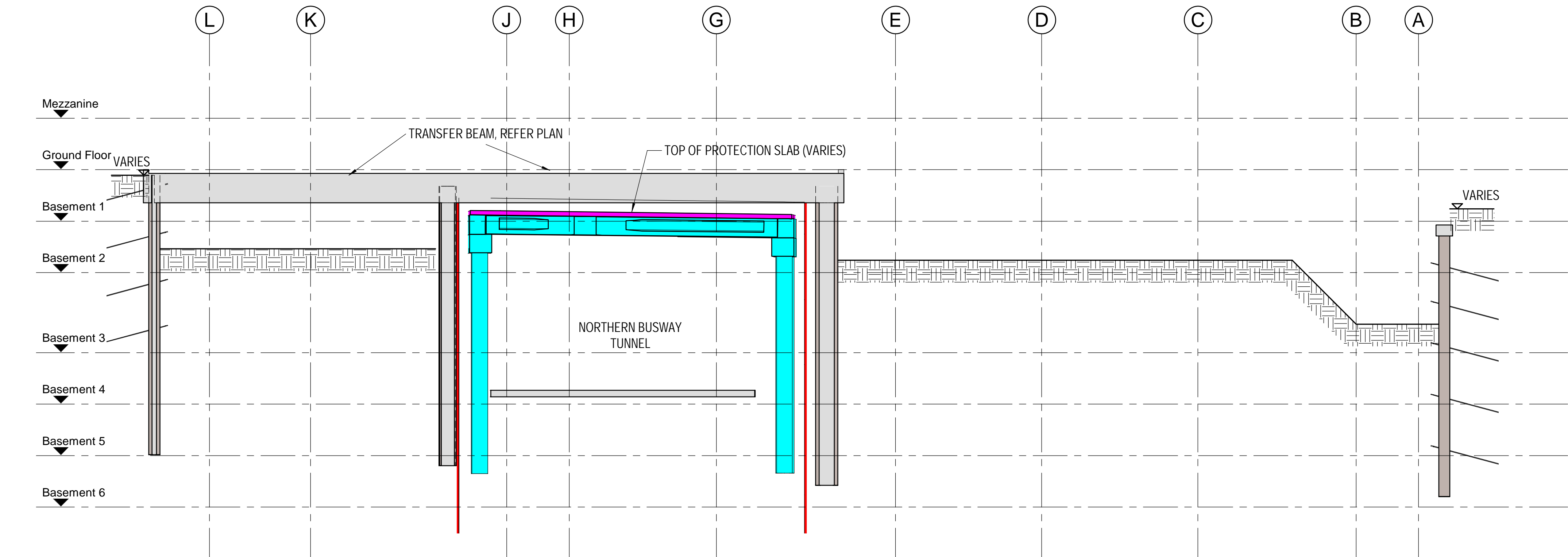
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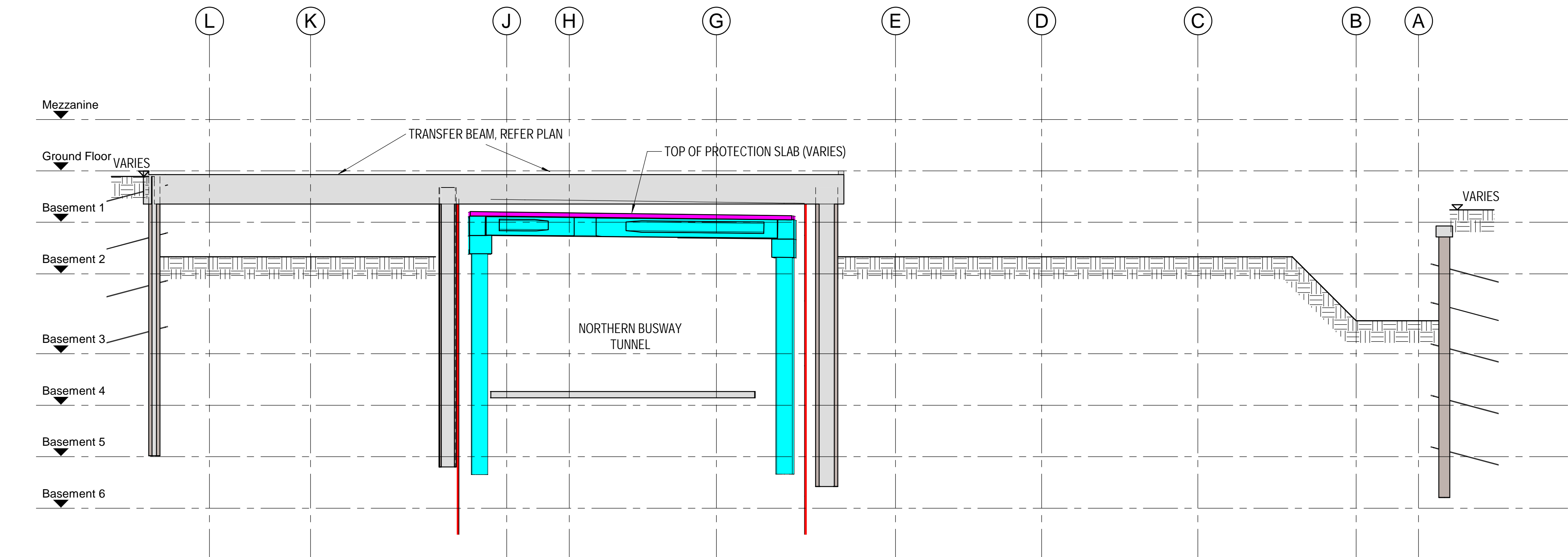
STAGE1
SCALE 1 : 150

STAGE 1
1. INSTALL SOLDIER PILES AND CAPPING BEAMS FROM EXISTING SURFACE



STAGE2
SCALE 1 : 150


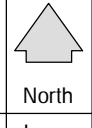
STAGE 2
2. CONSTRUCT TRANSFER BEAM ABOVE TUNNEL ROOF SLAB. POUR ON VOID FORMER IN STAGES, LIMITING LOADS ON TUNNEL ROOF TO 20kPa MAXIMUM.
3. START EXCAVATION OF BULK EARTHWORKS. EXCAVATE IN LAYERS EVENLY BOTH SIDES OF EXISTING TUNNEL.
4. INSTALL GROUND ANCHORS AND STRESS TO PRESCRIBED FORCES PER GEOTECHNICAL ENGINEERS SPECIFICATION. WHERE REQUIRED, PROVIDE BATTER SLOPES PER ADVICE IN GEOTECHNICAL REPORT.

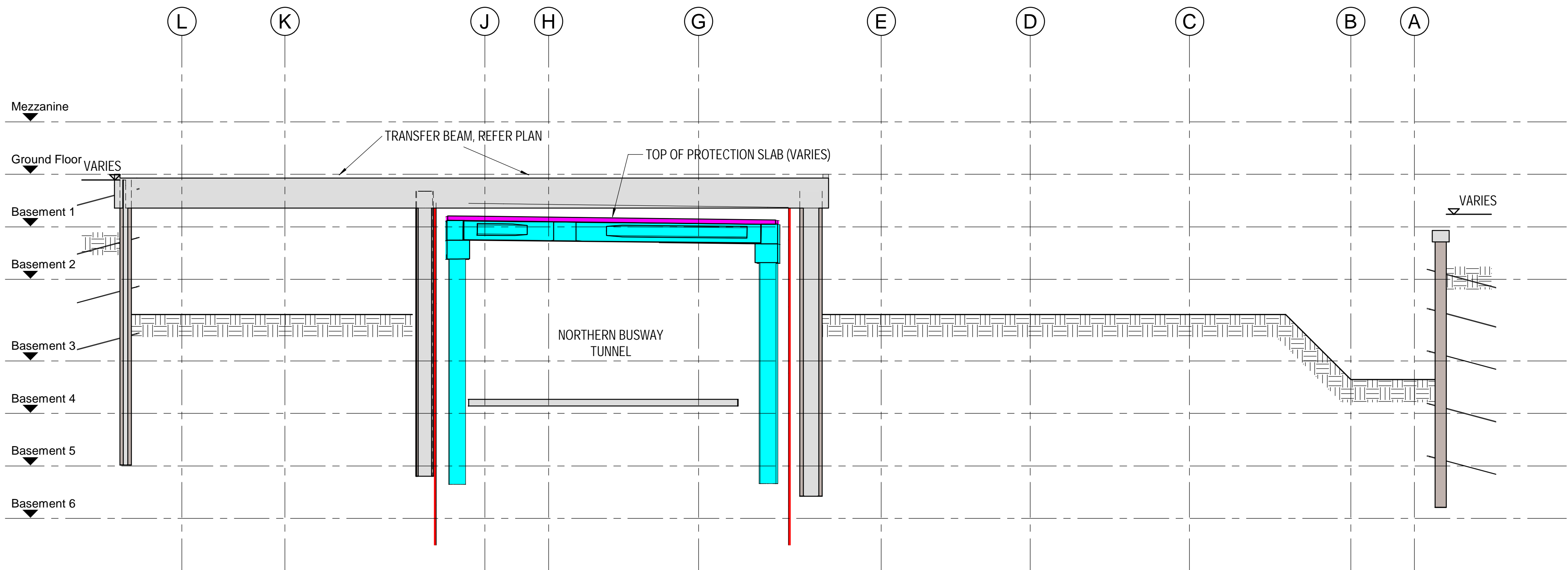


STAGE3
SCALE 1 : 150

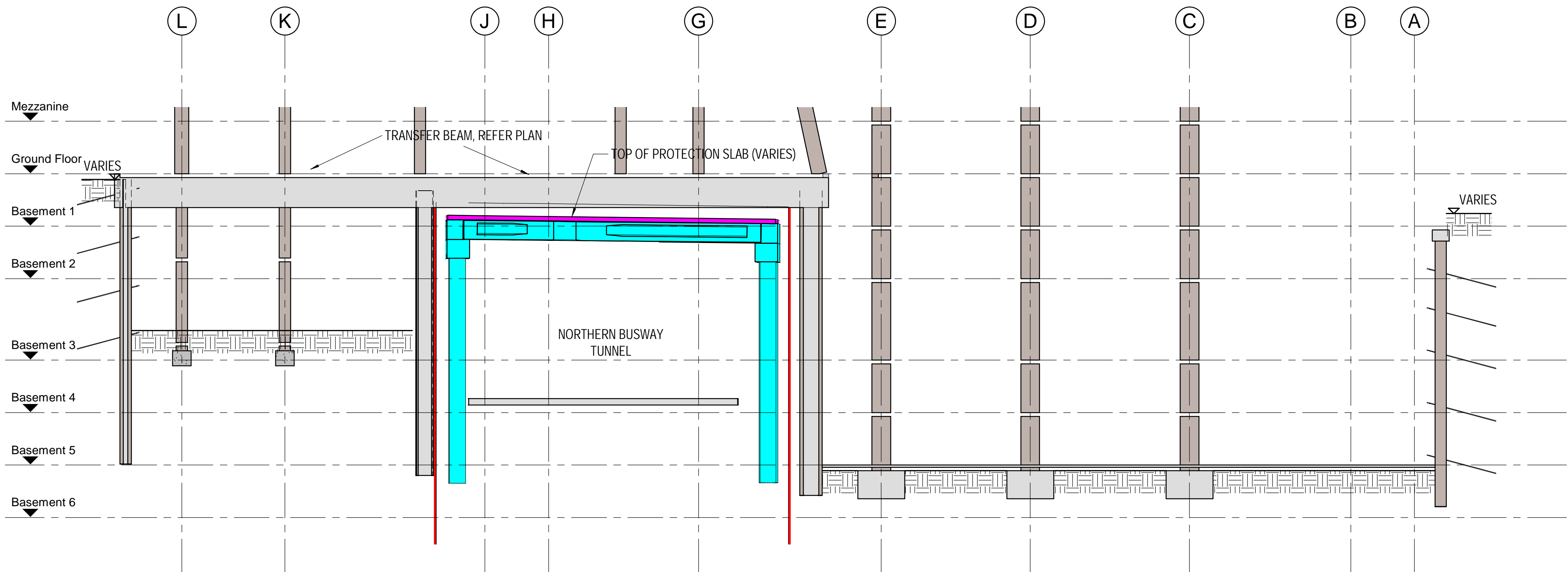
STAGE 3
5. CONTINUE EXCAVATING EVENLY / CONCURRENTLY ON BOTH SIDES OF TUNNEL TO MAINTAIN EQUILIBRIUM ON TUNNEL WALLS. BATTER SLOPES TO SUIT GEOTECHNICAL REPORT.
6. INSTALL GROUND ANCHORS TO SOLDIER PILES

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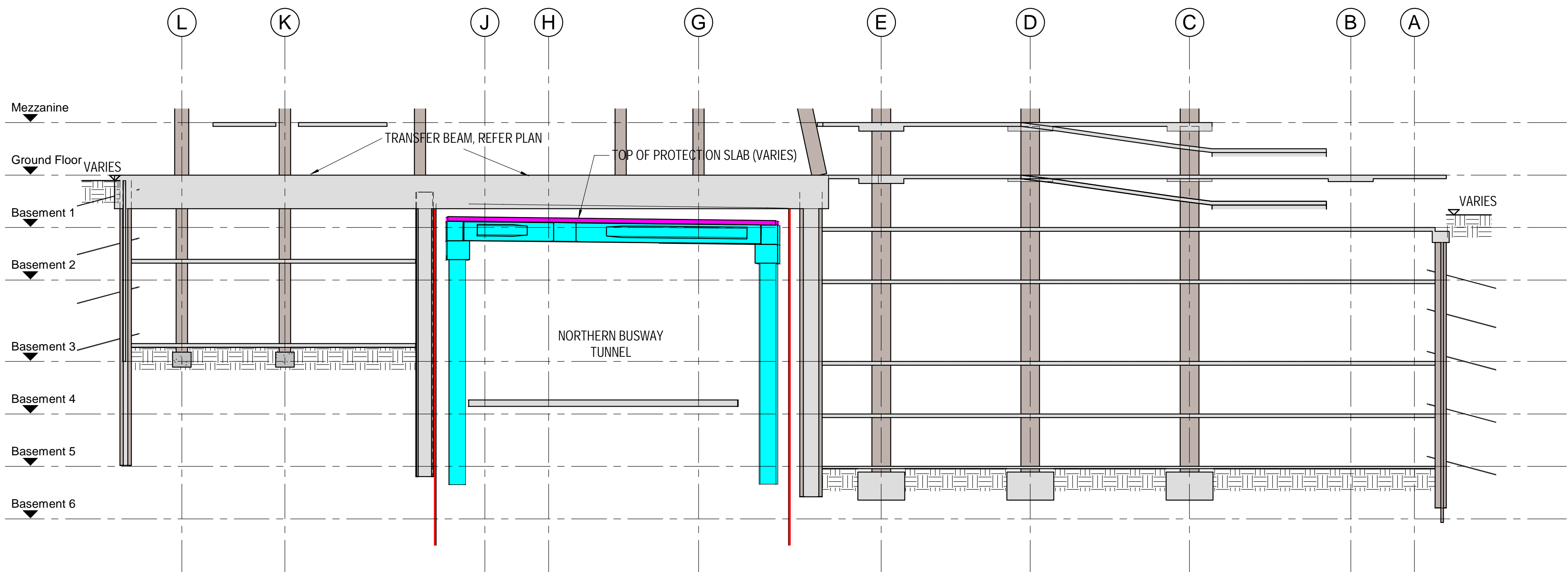
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STAGE4
SCALE 1 : 150



STAGE5
SCALE 1 : 150





STAGE6
SCALE 1 : 150

STAGE 4
7. CONTINUE EXCAVATING EVENLY / CONCURRENTLY ON BOTH SIDES OF TUNNEL TO MAINTAIN EQUILIBRIUM ON TUNNEL WALLS. BATTER SLOPES TO SUIT GEOTECHNICAL REPORT.
8. CONTINUE TO PROGRESSIVLEY INSTALL GROUND ANCHORS TO SOLDIER PILES

STAGE 5
9. CONTINUE EXCAVATING EVENLY / CONCURRENTLY ON BOTH SIDES OF TUNNEL TO MAINTAIN EQUILIBRIUM ON TUNNEL WALLS. BATTER SLOPES TO SUIT GEOTECHNICAL REPORT.
10. INSTALL GROUND ANCHORS TO SOLDIER PILES.
11. INSTALL SOLDIER PILES AND STRIP FOOTINGS OVER TO SUPPORT NEW COLUMNS AND INFILL WALL OVER

STAGE 6
BUILD NEW FOOTINGS, COLS, WALLS, SLABS AND TRANSFER BEAM OVER TUNNEL AS PER NORMAL CONSTRUCTION SEQUENCING

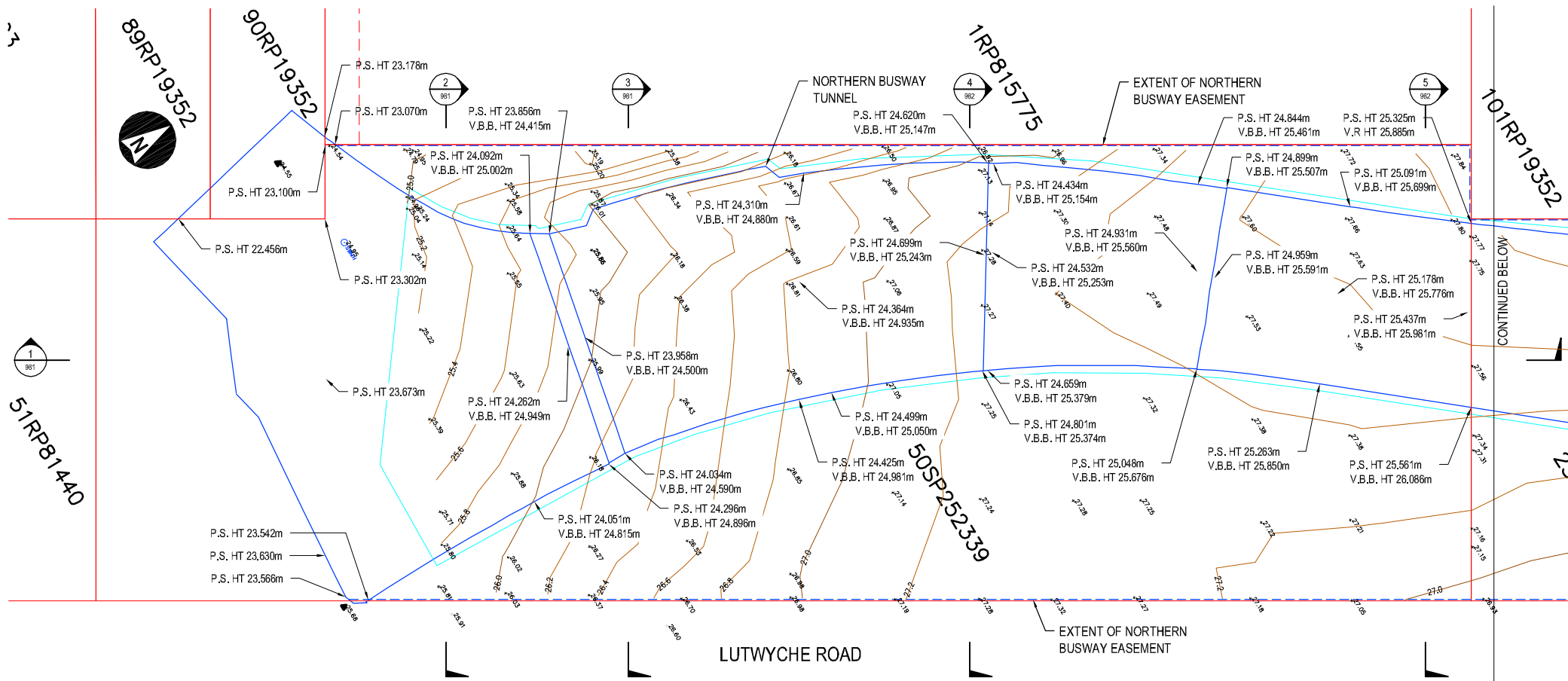
ISSUE	REVISION	DATE	INITIAL

<div></div> <div>Brisbane, QLD, Australia</div> <div>www.wsp.com</div>	<div>INFORMATION</div>	LAMINGTON MARKETS	STRUCTURAL DRAWING			
	<div>NOT FOR CONSTRUCTION</div> <div>© WSP Australia Pty Ltd. All rights reserved.</div> <div>These drawings, plans and specifications and the copyright therein are the property of WSP Australia Pty Ltd. and must not be used, reproduced or copied fully or in part without the written permission of WSP Australia Pty Ltd.</div>	<div>PROPOSED CONSTRUCTION</div> <div>SEQUENCE OVER NORTHERN</div> <div>BUSWAY - SHEET 02</div>	Designed: RW	Approved: Approver	<div></div> <div>North</div> <div>Issue</div>	
			Drawn: TF	Project Engineer/Director:		Date:
			Scale: 1:150 @ A0	Project No:		Drawing No:
			Date: MAY 2019	4388		S2-0051

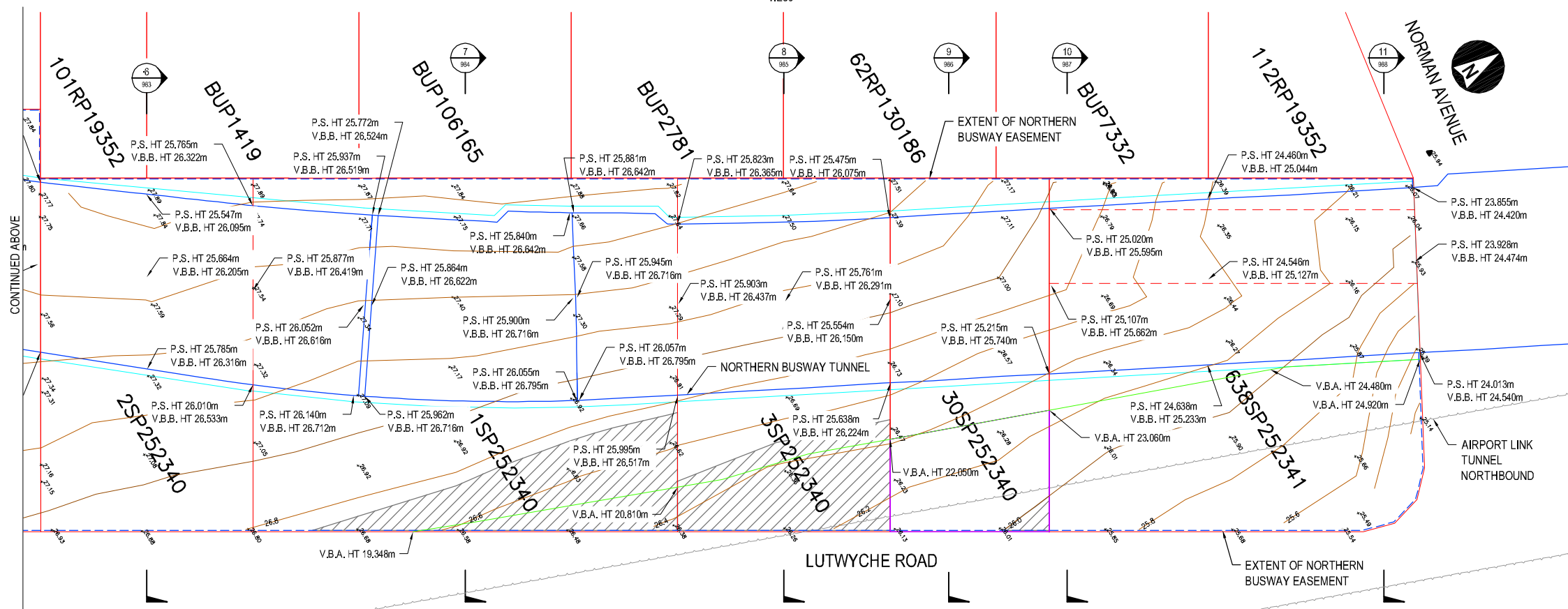
APPENDIX B –

SITE SURVEY AND EXISTING
SERVICES DRAWINGS





PLAN
1:250



PLAN - CONTINUED
1:250

NOTES

1. ALL HEIGHTS ARE TO AHD.
2. HEIGHTS OF THE EXISTING SURFACE, VOLUMETRIC BOUNDARY AND TUNNEL STRUCTURES VARY OVER THE EXTENT OF THE PROPERTY.
3. THE PROFILE AND LOCATION OF THE AIRPORT LINK TUNNELS ARE BASED ON DESIGN INFORMATION ONLY. REFER TO AS-BUILT DRAWINGS FOR CONSTRUCTED LOCATION AND PROFILE.
4. THIS SKETCH MUST BE READ IN CONJUNCTION WITH THE 'ENGINEERING SPECIFICATIONS (FOR AN EASEMENT LOCATED ABOVE AND ADJACENT TO THE NORTHERN BUSWAY CUT AND COVER TUNNEL)' FOR THE APPLICABLE LOT.
5. DIMENSIONS SHALL NOT BE DETERMINED FROM THE SKETCH BY SCALING.
6. DCDB LOT BOUNDARIES AND NUMBERS AS SUPPLIED BY DTMR.
7. SURFACE CONTOURS ARE AS PER DRAWING NUMBER 6797 S 01 DT A.dwg AS SUPPLIED BY DTMR.

LEGEND

P.S. - CUT AND COVER PROTECTION SLAB
V.B.B. - BUSWAY VOLUMETRIC BOUNDARY
V.B.A. - AIRPORT LINK TUNNELS
VOLUMETRIC BOUNDARY

- APPROXIMATE AREA WHERE ALLOWABLE EXCAVATION IS ABOVE EXISTING SURFACE. REFER SECTIONS.

- EXISTING SURFACE SPOT HEIGHTS

- EXISTING SURFACE HEIGHT CONTOURS

- EASEMENT BOUNDARY

- DCDB PROPERTY BOUNDARIES

- NORTHERN BUSWAY CUT AND COVER STRUCTURE

- NORTHERN BUSWAY VOLUMETRIC BOUNDARIES

- AIRPORT LINK TUNNELS VOLUMETRIC BOUNDARIES

FOR INFORMATION ONLY
NOT FOR CONSTRUCTION

Rev.	Date	Revision Details	By	Ver.	App.
B	2/08/13	AIRPORT LINK VOLUMETRIC ADDED	JLP		
A	1/08/13	ISSUED FOR INFORMATION	JLP		

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**NORTHERN BUSWAY PROJECT
(WINDSOR TO KEDRON)**

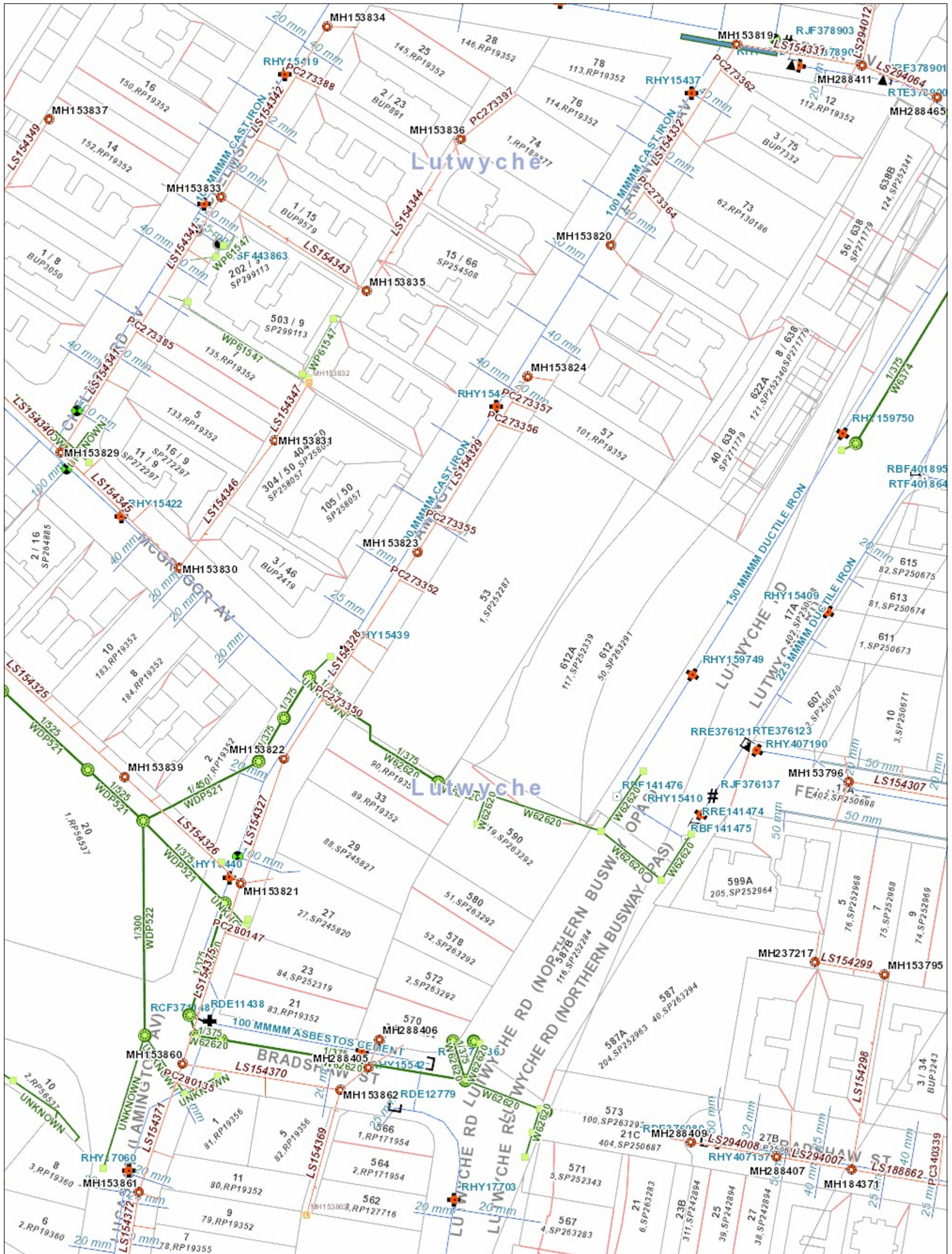
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Drawing Title:

**LUTWYCHE WEST
LOADING AND UNLOADING CONDITIONS
PLAN**

Project No.	22904
Scale	1:250
Sheet Size	A1
Drawing No.	SK980
Rev.	B

05.08.2013@06:42 P:\CIVIL\22904\cad\sketch\22904-SK980.dwg XREF: c:\aha_scs_x-4735 S 01 DT A.dwg X-PPARCH-SURV-LUTWYCHE_22904-NB_TUNNELPROTECTION-StructuralPlan-SCSPLAH-LutwycheWest101_L2_L5_x-4904 Lutwyche West (Northern Busway) Version 1 - Copy LOGIN NAME: jason.podlich



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50.0 0 25.0 37.5 50.0

Metres

Scale: 1: 1,000

Legend

<div><div></div></div> <div>Local Government Areas</div> <div><div><div></div></div>PRESSURE GAUGE</div> <div><div><div></div></div>PRESSURE GAUGE - OFFLINE</div> <div>Recycled Water Fitting</div> <div><div><div></div></div>END CAP</div> <div><div><div></div></div>GIBAULT JOINT</div> <div><div><div></div></div>REDUCER</div> <div><div><div></div></div>RESERVOIR INLET</div> <div><div><div></div></div>SAMPLING STATION</div> <div><div><div></div></div><all other values></div> <div><div><div></div></div>HEAD WALL</div> <div><div><div></div></div></div> <div>Recycled Water Chamber</div> <div><div><div></div></div>INGROUND HYDRANT</div> <div>Recycled Water Control Valve</div> <div><div><div></div></div>AIR</div> <div><div><div></div></div>ALTITUDE</div> <div><div><div></div></div>SCOUR - OFFLINE</div> <div><div><div></div></div>PRESSURE REDUCING - OFFLINE</div> <div>Recycled Water System Valve</div> <div><div><div></div></div>GATE, OPEN</div> <div><div><div></div></div>BALL, OPEN</div> <div><div><div></div></div>SERVICE VALVE CLOSED</div> <div><div><div></div></div>RESERVOIR</div> <div><div><div></div></div>TREATMENT PLANT</div> <div><div><div></div></div>PUMP STATION</div> <div>Recycled Water Pumps</div> <div><div><div></div></div>BOOSTER PUMP - 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APPENDIX C –

GEOTECHNICAL INFORMATION





Douglas Partners
Geotechnics | Environment | Groundwater

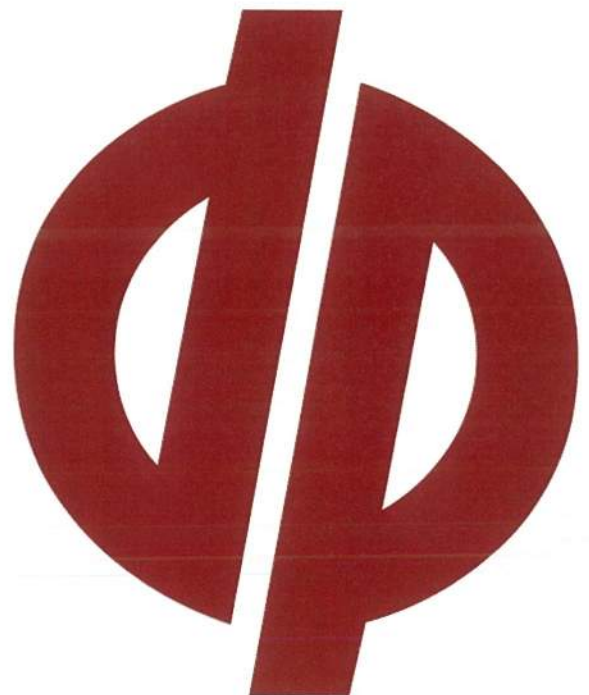
Report on
Geotechnical Investigation

Proposed Mixed Use Development
33 to 57 Lamington Avenue and 612 Lutwyche Road,
Lutwyche

Prepared for
Kane Constructions Pty Ltd

Project 87424.00
February 2016

Integrated Practical Solutions





Douglas Partners

Geotechnics | Environment | Groundwater

Document History

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Report prepared for	Kane Constructions Pty Ltd		
File name	P:\87424.00 - LUTWYCHE, Proposed Mixed Use Development\8.0 Documents\87424.00.R.001.doc		


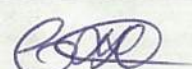
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0	1	0	D Laycock, Kane Constructions Pty Ltd

The undersigned, on behalf of Douglas Partners Pty Ltd, confirm that this report, but excluding any information provided by others, has been checked and reviewed for errors, omissions and inaccuracies.

Signature	Date
Author 	8 February 2016
Reviewer 	8 February 2016



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Report on Geotechnical Investigation

Proposed Mixed Use Development

33 to 57 Lamington Avenue and 612 Lutwyche Road, Lutwyche

1. Introduction

This report presents the results of a geotechnical investigation carried out for a proposed mixed use development to be located at 33 to 57 Lamington Avenue and 612 Lutwyche Road, Lutwyche. The investigation was undertaken at the request of Mr David Laycock of Kane Constructions Pty Ltd, the project builder.

It is understood that the proposed development will comprise a six storey mixed building over a two to three level in-ground basement car park. The aim of this report, as outlined in DP's Proposal BNE150645 dated 25 June 2015, was to assess the conditions at the site in order to provide comments on:

- subsurface conditions, including groundwater (if encountered);
- excavation conditions and suitable excavation methods;
- earthworks requirements, re-use of excavated materials and temporary batter slopes;
- basement retention options and geotechnical basement retaining wall design parameters (comprising unit weight, active, passive and at rest earth pressure coefficients, ultimate passive pressures in rock),
- soil and rock anchor design parameters, comprising preliminary bond stresses and pull-out profiles;
- groundwater conditions and design requirements to address same;
- suitable spread footing options, maximum allowable bearing pressures and estimated settlements (if appropriate);
- suitable pile footing options, ultimate end bearing and shaft adhesion pressures for axially loaded piles in compression;
- assessment of site sub-soil class to AS1170.4-2007 Part 4 (Ref. 1) within the depths drilled; and
- site erosion potential.

The investigation comprised the drilling and sampling of five bores, laboratory testing, engineering analysis and reporting. Details of the field work and laboratory testing are presented in this report together with comments and recommendations on the items listed above.

This report must be read in conjunction with the notes entitled 'About This Report' in Appendix A and other explanatory notes, and should be kept in its entirety without separation of individual pages or sections.

2. Site Description

The development site is located at 33 to 57 Lamington Avenue and 612 Lutwyche Road, Lutwyche, as indicated on Drawing 1 in Appendix B. The site is an irregular shaped area measuring approximately 73 m by 120 m and is dissected by the Northern Busway Tunnel. The Lutwyche Bus Station is located on the southern side of the site, and residential properties adjoin the northern side.

At the time of the investigation, the eastern half of the site was vacant, and has remained so since the construction of the Northern Busway Tunnel was completed beneath it in late 2011. The western half of the site is mostly occupied by two blocks of residential units which are three storeys high and of brick construction with concrete car parking along the eastern side. An elevated weatherboard cottage and surrounding gardens is located at 57 Lamington Avenue, and 33 and 35 Lamington Avenue were lawn areas adjacent to the busway portal.

The eastern half of the site slopes down towards the south-east from approximately RL 27.5 m to RL 25.5 m along the north-eastern boundary. The western half of the site has slightly more cross fall, falling from about RL 26.5 m to RL 18.0 m. A 1.5 m high retaining wall separates the southernmost unit block building from the vacant portion of the site.

Some photographs of the site at the time of the investigation are presented below as Figures 1 and 2.



Figure 1: Drilling rig on Bore 2 in southwestern corner of site (bus station in background)



Figure 2: Drilling rig on Bore 3 in northern portion of the site (northern unit block behind fence)

A series of historical aerial photographs for the site (from Nearmap) are presented in Figures 2 to 7 below. The eastern portion of the site was resumed from a previous commercial usage prior to 2008 and the Northern Busway was constructed as part of the Airport Link project. It appears that in late 2010 the tunnel was excavated after two rows of contiguous piles were installed to form the walls, then a roof structure was constructed. From the photographs shown below, it doesn't appear that the site was excavated substantially beyond the extent of the tunnel.



Figure 3: Aerial photograph on 21 November 2009 (from Nearmap, site shown in red outline)



Figure 4: Aerial photograph on 12 September 2010 (from Nearmap)



Figure 5: Aerial photograph on 14 January 2011 (from Nearmap)



Figure 6: Aerial photograph on 2 March 2012 (from Nearmap)



Figure 7: Aerial photograph on 12 January 2016 (from Nearmap)

3. Regional Geology

The Geological Survey of Queensland's 1:100,000 series 'Brisbane Sheet' indicates that the site is underlain by the Late Triassic aged Brisbane Tuff, typically comprising "ignimbrite, stratified and massive rhyolitic tuff, conglomerate, sandstone, scree breccia".

The residual soil and tuff encountered during the field work are generally consistent with the published geology.

4. Field Work Methods

The field work was undertaken between 19 and 25 November 2015 and comprised the drilling of five bores (designated Bores 2 to 6). Bore 1 was abandoned due to a lack of access for the drilling rig and will be drilled at a later date. The bores were drilled to between 18.0 m and 25.25 m depth using a truck-mounted Hydrapower Scout drilling rig, initially by continuous solid flight auger techniques to 2.5 m depth. Temporary steel casing was then installed and the bores were advanced by rotary washbore and NMLC rock coring techniques to the termination depths.

Standard penetration tests (SPTs) were carried out at 1.5 m intervals from 1.0 m depth to provide an indication of soil strength consistency/relative density and to collect samples for visual identification and laboratory testing. On completion of drilling, and after checking for groundwater, Bores 3 to 6 were backfilled with drill spoil and capped with rapid set concrete. Bore 2 was plugged at 6 m depth then a hand-slotted 50 mm DWV PVC standpipe piezometer was installed for future groundwater monitoring. The piezometer was bailed dry at the time of the field work.

The test locations were determined with reference to existing site features, and the approximate locations are indicated on Drawing 1 in Appendix B. Ground surface levels were interpolated from a client supplied survey drawing (Lawson Surveys Pty Ltd Drawing Reference 18439 dated 23 October 2015).

The field work was supervised by a geotechnical engineer who positioned and logged the bores, and also collected samples for visual and tactile assessment and for laboratory testing purposes.

5. Field Work Results

The subsurface conditions encountered in the bores are described in detail on the borehole logs in Appendix C. These should be read in conjunction with the notes entitled 'About This Report' and other explanatory notes in Appendix A which describe sampling methods, soil and rock descriptions, symbols and abbreviations used in their preparation.

The subsurface conditions encountered in the bores can be summarised as follows:

- **Filling:** Filling comprising medium dense to dense silty sand and stiff gravelly clay was encountered in Bores 2, 4, 5 and 6 to between 0.5 m and 1.5 m depth.
In the absence of documentation to prove otherwise the filling is assumed to be 'uncontrolled'.
- **Residual Soils:** Stiff to very stiff then very stiff and hard silty clay, sandy clay, clayey silt, silty sandy clay and gravelly sandy clay, and medium dense clayey sand residual soils were encountered below the filling to between 3.9 m and 16.5 m depth.

- **Tuff:** Extremely low strength or very low strength tuff was then encountered and increased to predominantly high and very high strength within 1.0 m to 3.1 m penetration. The rock increased to very high strength below 9.5 m and 4.3 m depth in Bores 1 and 2 respectively. The rock was generally fresh stained, with some slightly weathered and fresh bands, and slightly fractured with some relatively extensive unbroken zones.

Free groundwater was not encountered during auger drilling in the bores and the use of water as a drilling fluid generally precluded observation of groundwater thereafter. The standpipe piezometer installed in Bore 2 was dipped for groundwater on 26 November 2015 and groundwater was observed at 12.4 m depth (RL 7.6 mAHD). The drilling fluid was also purged from Bore 5 at the completion of drilling and groundwater recovered to 10.8 m depth (RL 14.4mAHD). It should be noted that groundwater depths are affected by climatic conditions and soil and rock permeability and will therefore vary with time.

6. Laboratory Testing

Geotechnical laboratory testing was conducted on samples collected from the bores and comprised:

- Atterberg limits and linear shrinkage testing was carried out on samples of silty clay recovered from Bores 2 and 3; and
- Emerson class number tests for dispersion and soil pH were undertaken on a disturbed samples from Bores 2 and 3 to determine the soils' potential for erosion as detailed in Brisbane City Council's Erosion Hazard Assessment Technical Notes (Ref. 2).

The results of the laboratory tests are given in Appendix D and are summarised below.

Table 1: Results of Plasticity Testing

Bore	Depth (m)	Description	Moisture Content (%)	Liquid Limit (%)	Plastic Limit (%)	Plasticity Index (%)	Linear Shrinkage (%)
2	1.0-1.45	Silty clay	15.5	49	20	29	13.5
3	1.0-1.45	Silty clay/sandy clay	12.5	43	17	26	11.5

Table 2: Summary of Dispersion and pH Test Results

Bore	Depth (m)	Description	Emerson Class No.	Potential for Erosion (from Ref. 2)	pH
2	1.0-1.45	Silty clay	4	'Moderate'	5.1
3	1.0-1.45	Silty clay/sandy clay	4	'Moderate'	3.6

Selected lengths of rock core recovered from the bores were tested in the laboratory for point load index (I_s), both in axial and diametral orientations, to assess intact rock strength. The results [corrected to I_{s50}] are given on the borehole log report sheets and are in the range 1 MPa to 10 MPa, generally indicating high to very high strength rock. Some slightly lower and higher results were obtained which are considered to be slight variations on the above, and visual and tactile assessment was used in connection with the testing to further assess the validity of such results and descriptions were adopted as considered appropriate.

7. Proposed Development

It is understood that the proposed development will comprise three buildings generally constructed from a two to three storey podium level. The main building will be mixed commercial and residential and up to 11 storeys high (including the podium) over up to five levels of basement car parking. Two smaller residential buildings will be constructed adjacent to the western and northern boundary of the site, west of the busway tunnel, and be either four or five storeys high above the podium level.

It is anticipated that the buildings will be reinforced concrete framed structures with suspended concrete floors and walls. From similar type developments, column loads for the main mixed commercial and residential part of the development are anticipated to be in the order of 8000 kN to 10 000 kN (working) at lowest basement level.

The most significant basement will be limited to the western side of the busway, with between 12 m and 18 m depth of excavation required below existing site levels to achieve a lowest basement floor level of RL 10.2 mAHD (anticipated bulk excavation level (BEL) of RL 9.9 mAHD). Apart from the lowest basement level, it is understood that the basement excavation will extend close to the boundaries on the western side of the busway tunnel. A smaller, more localised basement excavation will also be constructed on the eastern side of the tunnel for cinemas, with a proposed lower floor level of RL 21.4 mAHD resulting in an approximately 7 m depth of excavation. Locally deeper excavations are expected to be necessary for confined excavations (i.e. lift shaft, pad footings, etc.).

Supplied cross-sections of the proposed development indicate that the lowest basement level is approximately 4.5 m below the tunnel invert level, although from the supplied plans the location of this cross-section is not clear.

The highest part of the building will be on the eastern side of the busway tunnel, where construction will be from existing ground level.

8. Comments

8.1 Appreciation of Ground Conditions

The subsurface conditions encountered in the bores generally comprised variable depths of filling then residual soils comprising stiff to very stiff then very stiff and hard clays and medium dense clayey sands over extremely low strength tuff, which grades to predominantly high and very high strength within about 3 m of penetration.

Monitoring of the standpipe in Bore 2 and in Bore 5 while it remained open indicates that free groundwater at the site is between RL 7.6 m and RL 14.4mAHD, which is within the high and very high strength tuff. Groundwater inflow is generally expected to be minor in the longer term, although slightly higher inflows can occur through open joints initially and therefore will need to be considered in the design of basement walls and on-ground floor slabs.

Due to the subsurface conditions encountered and the need to excavate close to the boundaries on the western side of the busway site, there will be implications for the design and construction of the building basement on this site, as follows:

- excavatability of rock;
- control of vibrations and movement;
- stability of excavated faces during construction; and
- stability of adjoining building (to the north) and the busway tunnel during construction.

8.2 Site Classification

The site classification in accordance with AS 2870 (Ref. 3) would be 'Class P', due to the presence of 'uncontrolled' filling. As a general guide for information purposes, it is anticipated that characteristic surface movements at the site would probably be about 20 mm, which would be consistent with a 'Class M' site if the 'uncontrolled' filling did not exist.

8.3 Basement Construction

Excavation in the order of 12 m to 18 m generally will be required below existing site levels to achieve the BEL of RL 9.9 mAHD. It is understood that the line of the excavation will extend close to the site boundaries for all but the lowest basement level. The need to maintain stability of the excavation faces and of the adjacent tunnel, building to the north, and footpaths and in-ground services will impact on the method of construction adopted.

The close proximity of the basement walls to the site boundaries effectively negates the option of battering to ensure short term stability of excavation faces.

The upper soil and extremely low strength rock was indicated to extend to between 7.0 m and 16.0 m depth on the western side of the tunnel which will generally need to be retained by an anchored or propped in-situ retaining wall prior to excavation of the site.

8.3.1 Positive Support

Positive excavation support will be required prior to excavation of the site to minimise ground movements behind the excavation faces and to ensure the risk of damage to adjoining structures, roadways and in-ground services is minimised as a result of basement construction.

Contiguous pile walls are considered to be the most suitable means of providing positive excavation support for the excavation on the western side of the busway tunnel. Deadman anchors will probably be required on the eastern side of the busway tunnel to restrain piles along this face and rock anchors installed as excavation progresses are suggested on the remaining sides to minimise movements in retaining walls. Further comments on these systems are given below.

8.3.1.1 Pile Walls

The bored piles should socket into high to very high strength tuff below the base of the excavation to provide toe restraint and some overturning moment fixity, although consideration may be given to terminating the piles above the base of the excavation on the sides that don't adjoin the tunnel. This would require penetration of the high to very high strength tuff, with additional anchoring provided for toe restraint and moment fixity. Below the toe of these piles, a regular grid of prestressed cable-type rock anchors and/or spot bolting would be necessary to minimise excavation movements. Locally pinned/anchored shotcrete and mesh will still be required over any clay seams and highly fractured zones in the rock, to reduce the likelihood of local instability of the faces between the anchors. Excavation should only proceed in maximum 1.5 m lifts, so adversely dipping joints, highly fractured zones and clay seams are identified and appropriate support measures may be advised by a geotechnical professional.

Soldier piles may also be considered as an alternative to contiguous piles where the risk of damage associated with movement or loss of retained soil from between the piles is not as critical, although it should be noted that soldier piles are considered unacceptable adjacent to the tunnel. Soldier piles can be typically spaced at up to about three pile diameter centres.

It should be noted that the ability to drill bored piles in rock is not only dependent on the characteristics of rock (strength, fracture spacing etc.) but also the type of drilling rig (especially rotation torque and weight or crowd force) and the diameter of piles. Bored pile installation in high strength or stronger tuff will require the use of a heavy hydraulic rotary piling rig with torque in excess of 220 kNm such as those operated by experienced piling contractors. Slow drilling penetration rates and high bit wear should be allowed for in piling tenders. Specialist techniques such as rock coring buckets, rock augers or pilot hammer holes through a base template may be required to drill sockets in the very high strength tuff. It is recommended that the drilling contractors assess the size of equipment required on the basis of the logs and inspection of the rock core samples.

Once the piles are installed, excavation should proceed in 1.5 m to 2 m vertical height lifts, with anchors (and also strip drains mesh and shotcrete for soldier piles) installed prior to proceeding with the next lift.

8.3.1.2 Rock Anchors

Typically anchors would be spaced on a regular grid of 2 m to 3 m both horizontally and vertically to support piles and excavated faces in such an excavation. It is assessed that on this site spot bolting may be able to replace the regular grid of anchors where rock strength and fracturing are favourable below the upper piles (subject to further assessment at the time of construction).

An ultimate bond stress of 3000 kPa is suggested for the design of rock anchors and rock bolts bonding in high to very high strength tuff. This bond stress should be divided by a factor of safety of 2 to assess suitable working bond stress in the design of fixed anchor lengths. Higher bond stresses may be feasible in the very high strength tuff, however the extent of this material was slightly variable. It would be more appropriate for anchoring contractors to assess the strength of rock during drilling and select design bond stresses on the basis, provided that appropriate load testing is carried out to verify the selected parameters.

For the conditions indicated by the investigation, the preparation of all temporary anchors at the site should also include:

- a free length equal to their height above the base of the excavation (this includes anchors for lower wall panels);
- a minimum bond length of 3 m;
- a maximum bond length of 10 m (unless specialist single bore multi-anchored systems are adopted);
- test loading up to 130% of the working load with checks for any creep or loss of load and the resistance to uplift on an inverted cone of rock with a base angle of 90° as indicated in Figure B3, Appendix B of AS 4678 (Ref. 4), with lock-off at 90% of the working load.

Determination of anchor spacing and lengths is a matter for detailed design, DP can assist with this design if required.

8.3.1.3 Wall Design Pressures

Walls supported by multiple rows of anchors or props could be designed using a rectangular earth pressure distribution of $0.3\gamma H$ kPa over the full height of the wall, where H is the total vertical height of the wall in metres and γ is the bulk unit weight of the soil and rock retained. Suggested bulk unit weight values for soil and tuff are 20 kN/m³ and 24 kN/m³ respectively. It is probable that the actual earth pressures will be lower at the top and toward the base of the walls. DP should be contacted if further details are required.

Surcharge loadings bearing in soil and rock should be multiplied by lateral coefficients of 0.3 and 0.15 respectively to calculate the resulting additional lateral pressure on the walls.

Adequate drainage will need to be provided behind temporary shoring and/or permanent walls (for 'drained' basement) holes to ensure water pressure does not build up behind these walls.

8.3.2 Excavatability

Excavation of filling, residual soils and extremely low to very low strength tuff should be readily achieved using conventional earthmoving plant (i.e. 30 tonne hydraulic excavator, drott etc.). Where high strength or stronger, slightly fractured rock is encountered large excavators with heavy rock breakers (i.e. 75 to 80 tonne excavators with 5 tonne hammers) will be required, however slow production rates would be expected. Blasting would increase production rates, however on this site blasting is unlikely to be deemed acceptable by the adjoining tunnel asset owner due to the potential for vibration affecting this structure.

Diamond blade rock saws will probably be suitable for detailed excavation for (say) pad footings or lift over-run pits at the base of the bulk excavation to control excavation overbreak and also for the corners of the excavation. Alternatively heavy rock breaking will be required which will likely give rise to overbreak in places.

Inspection of core samples by intending contractors is considered essential prior to the selection of the most suitable excavation technique and finalising of tender prices.

It should be recognised that the above excavatability estimates are based on materials encountered at the test locations only and that conditions may prove more difficult (or easier) for excavatability between and beyond these test locations. Site trials are recommended to confirm the above estimates.

8.3.3 Construction Vibration, Noise and Movements

Vibration will result from demolition, excavation and construction work on this site. There is significant debate as to the maximum amount of vibration that buildings can accommodate; however, vibration restrictions must be set with a realistic appreciation for the normal operational environment of the site. Tolerance to vibration will also depend upon the nature of the materials used in construction (ductile or brittle), the age of the buildings, and whether or not the buildings are already cracked or in disrepair.

From current information, it is considered likely that the structures adjacent to the site can withstand vibration levels higher than those required to maintain the comfort of their occupants. A human comfort criterion is therefore suggested and the vector sum peak particle velocity (VSPPV) is proposed as the control parameter. It is recommended that a Provisional Allowed Vibration Limit of 8.0 mm/sec (VSPPV) be set during normal working hours, at foundation level of the potentially affected building/s.

A properly designed vibration and movement monitoring program will need to be implemented during the excavation and construction phases of the project. During basement excavation and construction it is recommended that regular survey monitoring points (survey target, survey spigot or similar) be installed on the top of wall capping beams. A second row of survey monitoring points should also be set up parallel to the capping beams and be offset at least 6 m outside the basement excavation. The site survey points should be referenced to an off-site benchmark situated beyond the influence of any site ground movements.

Survey movement monitoring of the basement walls will be required as a minimum to the following monitoring frequency:

- once before excavation commences;
- twice weekly until the basement excavation is complete;
- every month for three months following completion of the basement.

The movement monitoring results should be provided to the project structural and geotechnical engineers for review throughout the project duration until project completion.

Excavation and construction noise and its impact upon nearby tenants and residents will also need to be considered. Dilapidation/building condition surveys of the adjacent buildings, prior to commencing site work, coupled with vibration, noise and movement monitoring, is also suggested. DP can undertake the vibration monitoring works if required.

8.3.4 Temporary Slope Batters

For construction outside the basement excavations, temporary batters of 2H:1V or flatter are suggested for temporary slopes up to 3 m vertical height in soils. If construction is carried out during the wetter months of the year and groundwater seeps from the face, then flatter batters may be necessary.

Vertical batters would be suitable in high strength or stronger rock up to 3 m height, if inspection during excavation confirms that there are no adversely orientated joints or shear planes, which might lead to wedge or block failure. Apart from wall panels, these batters should be kept well inside the line of excavation and are only suggested as temporary side slopes within the perimeter anchored basement walls. It is possible that water may seep from the faces of confined excavations and these excavations will need to be temporarily dewatered.

8.3.5 Site Preparation – Excavation Base

The exposed subgrade at the BEL of RL 9.9 m is expected to comprise high to very high strength tuff. The subgrade at the base of the deeper excavation should be air blasted to remove all loose material and then inspected. On the intermediate bench where the lowest western basement level is setback from the tunnel and also for the excavation on the eastern side of the tunnel, the base subgrade should comprise hard gravelly silty clay and very stiff to hard clayey silt. Additional preparation would probably be limited to the removal and replacement of loose spoil in any over-excavated zones with a suitable approved crushed rock product or similar. It may also be prudent to undertake a test roll with a 12 tonne or larger smooth drum roller to detect any loose or uncompacted zones also requiring removal.

Any new filling if required to achieve design levels or beyond the perimeter of the excavation, should be undertaken under 'Level 1' supervision and testing as detailed in AS 3798–2007 (Ref. 5). Filling should be placed in layers not exceeding 0.2 m 'loose' thickness, with each layer compacted to a minimum dry density ratio of 100% Standard compaction within 2% of optimum moisture content.

8.4 Foundations

It is anticipated that column loads at lowest basement level for the new building will be in the order of 8000 kN to 10 000 kN (working).

Based on the high to very high strength tuff conditions anticipated at lowest basement level, suitable foundations for most of the building are expected to comprise shallow pad and strip footings. For the intermediate zone where the lowest basement level is setback from the tunnel on the western side of the site and all of the eastern side of the tunnel, piled footings will also be necessary be expensive to construct and probably unnecessary.

Pad or strip footings founding in rock and bored piles founded a minimum of three pile diameters into natural soil or a minimum one pile diameter into rock, could be sized using the maximum allowable values given in Table 3 below.

Table 3: Allowable Foundation Design Pressures

Material	Maximum Allowable Pressure (kPa)	
	Shaft Adhesion	End Bearing
Very stiff (or stronger) residual clay	20	Not applicable
Extremely low strength tuff	70	Not applicable
High to very high strength (or stronger) tuff	1200	12 000

Total and differential settlements for footings up to 1 m wide, designed and constructed on the above basis, are not expected to exceed 10 mm and 5 mm respectively. The base of all footings should be clean and dry at the time of casting.

Where limit state methods are used to design the foundations, the ultimate unfactored geotechnical strength ($R_{d,ug}$) can be estimated by multiplying the allowable values given above by a factor of safety of 2.5, and then multiplying by a suitable geotechnical strength reduction factor (ϕ_g) to obtain the design geotechnical strength ($R_{d,g}$). As a guide, where the average risk rating is assessed to be high and there is low redundancy in foundations, a ϕ_g value of 0.45 would apply. Guidance on the choice of ϕ_g is provided in Section 4 of AS 2159 (Ref. 6).

All footing excavations will need to be inspected by a qualified geotechnical engineer/engineering geologist prior to casting of concrete to confirm rock quality. As part of the inspection process, probe drilling and spoon testing will need to be carried out on strip and pad footings proportioned for the above pressure. This is required to ensure a suitable thickness (i.e. 1.5 to two times the footing width) of intact rock (free of clay seams and highly fractured zones) is present beneath the footings, suitable for the support of the suggested bearing pressure.

8.5 Earthquake Site Factor

In accordance with AS1170.4, it is recommended that a site sub-soil classification of "Class C_e – Shallow Soil Site" be adopted, in accordance with the definitions presented in *Section 4.2 – Class Definitions*. This classification is based on no more than 40 m depth of stiff soil.

8.6 On-Ground Floor Slabs

Following basement excavation, most of the exposed subgrade is expected to comprise high to very high strength tuff with some localised very stiff to hard clays in the western basement excavation and across the base of the eastern basement excavation.

Provided subgrade preparation is carried out in accordance with Section 8.3.5 above, moduli of subgrade reaction (k) of 25 kPa/mm and 80 kPa/mm are suggested for the design of floor slabs subjected to standard wheel loads (i.e. car traffic) where very stiff to hard clays/silts and high to very high strength tuff rock respectively are exposed. These are based on a soaked California bearing ratio (CBR) values of 3% and 15% where the subgrade comprises very stiff to hard clays/silts and high to very high strength tuff rock respectively. For loaded areas of different proportion or different load intensity to standard wheel loads, Douglas Partners should be contacted for further advice.

It is envisaged that the basement will be 'drained' and therefore on-ground basement floor slabs will need to be designed with appropriate hydrostatic pressure relief such as a gravel drainage layer with a grid of 'ag' pipes linked to sumps for removal by pumps.

8.7 Erosion

The results of Emerson Class dispersion tests indicate that the silty clay had a moderate potential for erosion referred from Brisbane City Council's Erosion Hazard Assessment (Ref. 2). Given the expected depth of excavation required for the basement proposed at the site, it is considered that there is a relatively low erosion risk associated with surface water flow, as the soils exposed to water runoff will largely be contained within the basement excavation.

Any erosion control measures at the ground surface will require detailed design. It is expected that, as a minimum, such measures will include silt fences, hay bales and other precautions to limit water runoff velocity.

9. Limitations

DP has prepared this report for the proposed mixed use development at 33 to 57 Lamington Avenue and 612 Lutwyche Road, Lutwyche, in accordance with DP's Proposal BNE150645 dated 25 June 2015 and acceptance received from Mr David Laycock of Kane Constructions Pty Ltd. The work was carried out under a DP's *Conditions of Engagement*. This report is provided for the exclusive use of Kane Constructions Pty Ltd for this project only and for the purposes as described in the report. It should not be used by or relied upon for other projects or purposes on the same or other site or by a third party. Any party so relying upon this report beyond its exclusive use and purpose as stated above, and without the express written consent of DP, does so entirely at its own risk and without recourse to DP for any loss or damage. In preparing this report, DP has necessarily relied upon information provided by the client and/or their agents.

The results provided in the report are indicative of the subsurface conditions only at the specific sampling and/or testing locations, and then only to the depths investigated and at the time the work was carried out. Subsurface conditions can change abruptly due to variable geological processes and also as a result of human influences. Such changes may occur after DP's field testing has been completed.

DP's advice is based upon the conditions encountered during this investigation. The accuracy of the advice provided by DP in this report may be limited by undetected variations in ground conditions across the site and between sampling and/or testing locations. The advice may also be limited by budget constraints imposed by others or by site accessibility.

This report must be read in conjunction with all of the attached and should be kept in its entirety without separation of individual pages or sections. DP cannot be held responsible for interpretations or conclusions made by others unless they are supported by an expressed statement, interpretation, outcome or conclusion stated in this report.

This report, or sections from this report, should not be used as part of a specification for a project, without review and agreement by DP, as this report has been written as advice and opinion rather than instructions for construction.

The contents of this report do not constitute formal design components such as are required by the Health and Safety Legislation and Regulations, to be included in a safety report specifying the hazards likely to be encountered during construction and the controls required to mitigate risk. This design process requires risk assessment to be undertaken, with such assessment being dependent upon factors relating to likelihood of occurrence and consequences of damage to property and to life. This, in turn, requires project data and analysis presently beyond the knowledge and project role respectively of DP. DP may be able, however, to assist the client in carrying out a risk assessment of potential hazards contained in the Comments section of this report, as an extension to the current scope of works, if so requested, and provided that suitable additional information is made available to DP. Any such risk assessment would, however, be necessarily restricted to the geotechnical components set out in this report and to their application by the project designers to project design, construction, maintenance and demolition.

10. References

1. Australian Standard AS 1170.4–2007, "Structural Design Actions, Part 4: Earthquake actions in Australia" Standards Australia.
2. Erosion Hazard Assessment Supporting Technical Notes–February 2010, Brisbane City Council.
3. Australian Standard AS 2870–2011 "Residential Slabs and Footings", Standards Australia.
4. Australian Standard AS 4678–2002 "Earth-Retaining Structures", Standards Australia.
5. Australian Standard AS 3798–2007 "Guidelines on earthworks for commercial and residential developments", Standards Australia.
6. Australian Standard AS 2159–2009 "Piling – design and installation", Standards Association of Australia.

Douglas Partners Pty Ltd

Appendix A

About This Report
Sampling Methods
Soil Descriptions
Rock Descriptions
Symbols and Abbreviations

About this Report

Douglas Partners



Introduction

These notes have been provided to amplify DP's report in regard to classification methods, field procedures and the comments section. Not all are necessarily relevant to all reports.

DP's reports are based on information gained from limited subsurface excavations and sampling, supplemented by knowledge of local geology and experience. For this reason, they must be regarded as interpretive rather than factual documents, limited to some extent by the scope of information on which they rely.

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This report is the property of Douglas Partners Pty Ltd. The report may only be used for the purpose for which it was commissioned and in accordance with the Conditions of Engagement for the commission supplied at the time of proposal. Unauthorised use of this report in any form whatsoever is prohibited.

Borehole and Test Pit Logs

The borehole and test pit logs presented in this report are an engineering and/or geological interpretation of the subsurface conditions, and their reliability will depend to some extent on frequency of sampling and the method of drilling or excavation. Ideally, continuous undisturbed sampling or core drilling will provide the most reliable assessment, but this is not always practicable or possible to justify on economic grounds. In any case the boreholes and test pits represent only a very small sample of the total subsurface profile.

Interpretation of the information and its application to design and construction should therefore take into account the spacing of boreholes or pits, the frequency of sampling, and the possibility of other than 'straight line' variations between the test locations.

Groundwater

Where groundwater levels are measured in boreholes there are several potential problems, namely:

- In low permeability soils groundwater may enter the hole very slowly or perhaps not at all during the time the hole is left open;

- A localised, perched water table may lead to an erroneous indication of the true water table;
- Water table levels will vary from time to time with seasons or recent weather changes. They may not be the same at the time of construction as are indicated in the report; and
- The use of water or mud as a drilling fluid will mask any groundwater inflow. Water has to be blown out of the hole and drilling mud must first be washed out of the hole if water measurements are to be made.

More reliable measurements can be made by installing standpipes which are read at intervals over several days, or perhaps weeks for low permeability soils. Piezometers, sealed in a particular stratum, may be advisable in low permeability soils or where there may be interference from a perched water table.

Reports

The report has been prepared by qualified personnel, is based on the information obtained from field and laboratory testing, and has been undertaken to current engineering standards of interpretation and analysis. Where the report has been prepared for a specific design proposal, the information and interpretation may not be relevant if the design proposal is changed. If this happens, DP will be pleased to review the report and the sufficiency of the investigation work.

Every care is taken with the report as it relates to interpretation of subsurface conditions, discussion of geotechnical and environmental aspects, and recommendations or suggestions for design and construction. However, DP cannot always anticipate or assume responsibility for:

- Unexpected variations in ground conditions. The potential for this will depend partly on borehole or pit spacing and sampling frequency;
- Changes in policy or interpretations of policy by statutory authorities; or
- The actions of contractors responding to commercial pressures.

If these occur, DP will be pleased to assist with investigations or advice to resolve the matter.

About this Report

Site Anomalies

In the event that conditions encountered on site during construction appear to vary from those which were expected from the information contained in the report, DP requests that it be immediately notified. Most problems are much more readily resolved when conditions are exposed rather than at some later stage, well after the event.

Information for Contractual Purposes

Where information obtained from this report is provided for tendering purposes, it is recommended that all information, including the written report and discussion, be made available. In circumstances where the discussion or comments section is not relevant to the contractual situation, it may be appropriate to prepare a specially edited document. DP would be pleased to assist in this regard and/or to make additional report copies available for contract purposes at a nominal charge.

Site Inspection

The company will always be pleased to provide engineering inspection services for geotechnical and environmental aspects of work to which this report is related. This could range from a site visit to confirm that conditions exposed are as expected, to full time engineering presence on site.

Sampling Methods

Douglas Partners



Sampling

Sampling is carried out during drilling or test pitting to allow engineering examination (and laboratory testing where required) of the soil or rock.

Disturbed samples taken during drilling provide information on colour, type, inclusions and, depending upon the degree of disturbance, some information on strength and structure.

Undisturbed samples are taken by pushing a thin-walled sample tube into the soil and withdrawing it to obtain a sample of the soil in a relatively undisturbed state. Such samples yield information on structure and strength, and are necessary for laboratory determination of shear strength and compressibility. Undisturbed sampling is generally effective only in cohesive soils.

Test Pits

Test pits are usually excavated with a backhoe or an excavator, allowing close examination of the in-situ soil if it is safe to enter into the pit. The depth of excavation is limited to about 3 m for a backhoe and up to 6 m for a large excavator. A potential disadvantage of this investigation method is the larger area of disturbance to the site.

Large Diameter Augers

Boreholes can be drilled using a rotating plate or short spiral auger, generally 300 mm or larger in diameter commonly mounted on a standard piling rig. The cuttings are returned to the surface at intervals (generally not more than 0.5 m) and are disturbed but usually unchanged in moisture content. Identification of soil strata is generally much more reliable than with continuous spiral flight augers, and is usually supplemented by occasional undisturbed tube samples.

Continuous Spiral Flight Augers

The borehole is advanced using 90-115 mm diameter continuous spiral flight augers which are withdrawn at intervals to allow sampling or in-situ testing. This is a relatively economical means of drilling in clays and sands above the water table. Samples are returned to the surface, or may be collected after withdrawal of the auger flights, but they are disturbed and may be mixed with soils from the sides of the hole. Information from the drilling (as distinct from specific sampling by SPTs or undisturbed samples) is of relatively low

reliability, due to the remoulding, possible mixing or softening of samples by groundwater.

Non-core Rotary Drilling

The borehole is advanced using a rotary bit, with water or drilling mud being pumped down the drill rods and returned up the annulus, carrying the drill cuttings. Only major changes in stratification can be determined from the cuttings, together with some information from the rate of penetration. Where drilling mud is used this can mask the cuttings and reliable identification is only possible from separate sampling such as SPTs.

Continuous Core Drilling

A continuous core sample can be obtained using a diamond tipped core barrel, usually with a 50 mm internal diameter. Provided full core recovery is achieved (which is not always possible in weak rocks and granular soils), this technique provides a very reliable method of investigation.

Standard Penetration Tests

Standard penetration tests (SPT) are used as a means of estimating the density or strength of soils and also of obtaining a relatively undisturbed sample. The test procedure is described in Australian Standard 1289, Methods of Testing Soils for Engineering Purposes - Test 6.3.1.

The test is carried out in a borehole by driving a 50 mm diameter split sample tube under the impact of a 63 kg hammer with a free fall of 760 mm. It is normal for the tube to be driven in three successive 150 mm increments and the 'N' value is taken as the number of blows for the last 300 mm. In dense sands, very hard clays or weak rock, the full 450 mm penetration may not be practicable and the test is discontinued.

The test results are reported in the following form.

- In the case where full penetration is obtained with successive blow counts for each 150 mm of, say, 4, 6 and 7 as:
4,6,7
N=13
- In the case where the test is discontinued before the full penetration depth, say after 15 blows for the first 150 mm and 30 blows for the next 40 mm as:
15, 30/40 mm

Sampling Methods

The results of the SPT tests can be related empirically to the engineering properties of the soils.

Dynamic Cone Penetrometer Tests / Perth Sand Penetrometer Tests

Dynamic penetrometer tests (DCP or PSP) are carried out by driving a steel rod into the ground using a standard weight of hammer falling a specified distance. As the rod penetrates the soil the number of blows required to penetrate each successive 150 mm depth are recorded. Normally there is a depth limitation of 1.2 m, but this may be extended in certain conditions by the use of extension rods. Two types of penetrometer are commonly used.

- Perth sand penetrometer - a 16 mm diameter flat ended rod is driven using a 9 kg hammer dropping 600 mm (AS 1289, Test 6.3.3). This test was developed for testing the density of sands and is mainly used in granular soils and filling.
- Cone penetrometer - a 16 mm diameter rod with a 20 mm diameter cone end is driven using a 9 kg hammer dropping 510 mm (AS 1289, Test 6.3.2). This test was developed initially for pavement subgrade investigations, and correlations of the test results with California Bearing Ratio have been published by various road authorities.

Soil Descriptions

Douglas Partners



Description and Classification Methods

The methods of description and classification of soils and rocks used in this report are based on Australian Standard AS 1726, Geotechnical Site Investigations Code. In general, the descriptions include strength or density, colour, structure, soil or rock type and inclusions.

Soil Types

Soil types are described according to the predominant particle size, qualified by the grading of other particles present:

Type	Particle size (mm)
Boulder	>200
Cobble	63 - 200
Gravel	2.36 - 63
Sand	0.075 - 2.36
Silt	0.002 - 0.075
Clay	<0.002

The sand and gravel sizes can be further subdivided as follows:

Type	Particle size (mm)
Coarse gravel	20 - 63
Medium gravel	6 - 20
Fine gravel	2.36 - 6
Coarse sand	0.6 - 2.36
Medium sand	0.2 - 0.6
Fine sand	0.075 - 0.2

The proportions of secondary constituents of soils are described as:

Term	Proportion	Example
And	Specify	Clay (60%) and Sand (40%)
Adjective	20 - 35%	Sandy Clay
Slightly	12 - 20%	Slightly Sandy Clay
With some	5 - 12%	Clay with some sand
With a trace of	0 - 5%	Clay with a trace of sand

Definitions of grading terms used are:

- Well graded - a good representation of all particle sizes
- Poorly graded - an excess or deficiency of particular sizes within the specified range
- Uniformly graded - an excess of a particular particle size
- Gap graded - a deficiency of a particular particle size with the range

Cohesive Soils

Cohesive soils, such as clays, are classified on the basis of undrained shear strength. The strength may be measured by laboratory testing, or estimated by field tests or engineering examination. The strength terms are defined as follows:

Description	Abbreviation	Undrained shear strength (kPa)
Very soft	vs	<12
Soft	s	12 - 25
Firm	f	25 - 50
Stiff	st	50 - 100
Very stiff	vst	100 - 200
Hard	h	>200

Cohesionless Soils

Cohesionless soils, such as clean sands, are classified on the basis of relative density, generally from the results of standard penetration tests (SPT), cone penetration tests (CPT) or dynamic penetrometers (PSP). The relative density terms are given below:

Relative Density	Abbreviation	SPT N value	CPT qc value (MPa)
Very loose	vl	<4	<2
Loose	l	4 - 10	2 - 5
Medium dense	md	10 - 30	5 - 15
Dense	d	30 - 50	15 - 25
Very dense	vd	>50	>25

Soil Descriptions

Soil Origin

It is often difficult to accurately determine the origin of a soil. Soils can generally be classified as:

- Residual soil - derived from in-situ weathering of the underlying rock;
- Transported soils - formed somewhere else and transported by nature to the site; or
- Filling - moved by man.

Transported soils may be further subdivided into:

- Alluvium - river deposits
- Lacustrine - lake deposits
- Aeolian - wind deposits
- Littoral - beach deposits
- Estuarine - tidal river deposits
- Talus - scree or coarse colluvium
- Slopewash or Colluvium - transported downslope by gravity assisted by water. Often includes angular rock fragments and boulders.

Rock Descriptions

Douglas Partners



Rock Strength

Rock strength is defined by the Point Load Strength Index ($Is_{(50)}$) and refers to the strength of the rock substance and not the strength of the overall rock mass, which may be considerably weaker due to defects. The test procedure is described by Australian Standard 4133.4.1 - 1993. The terms used to describe rock strength are as follows:

Term	Abbreviation	Point Load Index $Is_{(50)}$ MPa	Approx Unconfined Compressive Strength MPa*
Extremely low	EL	<0.03	<0.6
Very low	VL	0.03 - 0.1	0.6 - 2
Low	L	0.1 - 0.3	2 - 6
Medium	M	0.3 - 1.0	6 - 20
High	H	1 - 3	20 - 60
Very high	VH	3 - 10	60 - 200
Extremely high	EH	>10	>200

* Assumes a ratio of 20:1 for UCS to $Is_{(50)}$

Degree of Weathering

The degree of weathering of rock is classified as follows:

Term	Abbreviation	Description
Extremely weathered	EW	Rock substance has soil properties, i.e. it can be remoulded and classified as a soil but the texture of the original rock is still evident.
Highly weathered	HW	Limonite staining or bleaching affects whole of rock substance and other signs of decomposition are evident. Porosity and strength may be altered as a result of iron leaching or deposition. Colour and strength of original fresh rock is not recognisable
Moderately weathered	MW	Staining and discolouration of rock substance has taken place
Slightly weathered	SW	Rock substance is slightly discoloured but shows little or no change of strength from fresh rock
Fresh stained	Fs	Rock substance unaffected by weathering but staining visible along defects
Fresh	Fr	No signs of decomposition or staining

Degree of Fracturing

The following classification applies to the spacing of natural fractures in diamond drill cores. It includes bedding plane partings, joints and other defects, but excludes drilling breaks.

Term	Description
Fragmented	Fragments of <20 mm
Highly Fractured	Core lengths of 20-40 mm with some fragments
Fractured	Core lengths of 40-200 mm with some shorter and longer sections
Slightly Fractured	Core lengths of 200-1000 mm with some shorter and longer sections
Unbroken	Core lengths mostly > 1000 mm

Rock Descriptions

Rock Quality Designation

The quality of the cored rock can be measured using the Rock Quality Designation (RQD) index, defined as:

$$\text{RQD \%} = \frac{\text{cumulative length of 'sound' core sections} \geq 100 \text{ mm long}}{\text{total drilled length of section being assessed}}$$

where 'sound' rock is assessed to be rock of low strength or better. The RQD applies only to natural fractures. If the core is broken by drilling or handling (i.e. drilling breaks) then the broken pieces are fitted back together and are not included in the calculation of RQD.

Stratification Spacing

For sedimentary rocks the following terms may be used to describe the spacing of bedding partings:

Term	Separation of Stratification Planes
Thinly laminated	< 6 mm
Laminated	6 mm to 20 mm
Very thinly bedded	20 mm to 60 mm
Thinly bedded	60 mm to 0.2 m
Medium bedded	0.2 m to 0.6 m
Thickly bedded	0.6 m to 2 m
Very thickly bedded	> 2 m