Melbourne Metro Rail Project

MMR-AJM-PWAA-RP-NN-000827

Ground Movement and Land Stability Impact Assessment

Melbourne Metro Rail Authority

20 April 2016

Revision: C1

Reference: CMS332569

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in association with Grimshaw

AJM
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Document control record

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Report title		Ground Movement and Land Stability Impact Assessment				
Document ID		MMR-AJM-PWAA-RP-NN-000827			**Contract No.**	CMS332569
File path						
Client		Melbourne Metro Rail Authority		**Client contact**	Daniel Cullen	
Rev	**Date**	**Revision details/status**	**Prepared by**	**Author**	**Verifier**	**Approver**
C1	20/04/2016	Final issue for Exhibition	Antoinette Walshe Anthony Bennett	Antoinette Walshe	Mark Tansley	Lisa Ryan
Current revision		**C1**				

Approval			
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Glossary and Abbreviations

Term	Definition
AH Act	Aboriginal Heritage Act 2006
AHD	Australian Height Datum
AJM JV	Aurecon, Jacobs and Mott MacDonald Joint Venture
Alternative Design Option	Potential alternative solutions to the project's concept design or route alignment within the proposed project boundary which have been assessed as part of the EES process. These would provide for flexibility during project delivery whilst still allowing integrated assessment of the project.
Aquifer	Rock or sediment in a formation that is saturated and sufficiently permeable to transmit of water
Aquitard	A low permeability unit that can store groundwater and also transmit it slowly from one aquifer to another
Asset	An existing or proposed structure, pavement or utility whose stability, form or function may be impacted by ground movement
BCA	Building Code of Australia
BS	British Standard
BTS	British Tunnelling Society
CBD	Central Business District
CHMP	Cultural Heritage Management Plan
CIC	Commercial in confidence
Concept Design	The Concept Design demonstrates a feasible way for the project to achieve the Victorian Government's objectives and meet the performance requirements for the Melbourne Metro. It provides the basis for the EES to assess the expected potential environmental risks and impacts, and demonstrates that impacts can be managed. It recognises that the project could be configured differently, provided it meets the Government's objectives and performance requirements.
Condition Survey	A survey of an asset that is undertaken prior to construction works. A post construction condition survey may be undertaken, if required.
Consequence	A consequence is the outcome of an event and has an effect on objectives. A single event can generate a range of consequences which can have both positive and negative effects on objectives. Initial consequences can also escalate through knock-on effects.
CSR	Concept Summary Report
Damage	An impact adversely affecting structural integrity, serviceability, performance or aesthetics of an asset
DELWP	Department of Environment, Land, Water and Planning (formerly Department of Transport, Planning, Land and Infrastructure)
EMF	Environmental Management Framework

Term	Definition
EMP	Environmental Management Plan
EMS	Environmental Management System
Environmental Aspects	An environmental aspect is an element or characteristic of an activity that interacts or can interact with the environment. Environmental aspects can cause environmental impacts. They can have either beneficial impacts or adverse impacts and can have a direct and decisive impact on the environment or contribute only partially or directly to a larger environmental change.
Environmental Impact	An environmental impact is a change to the environment that is caused either partly or entirely by one or more environmental aspects. An environmental aspect can have either a direct and decisive impact on the environment or contribute only partially or indirectly to a larger environmental change. In addition, it can have either a beneficial environmental impact or an adverse environmental impact.
Environmental Issue	A detrimental effect due to the implementation of Melbourne Metro on the sustainability of the surrounding natural environment
EPA	Victorian Environment Protection Authority
EPB (TBM)	Earth Pressure Balance (Tunnel Boring Machine)
EPBC Act	*Environment Protection and Biodiversity Conservation Act 1999*
FFG Act	*Flora and Fauna Guarantee Act 1988*
GQRUZ	Groundwater Quality Restricted Use Zone
IAU	Impact Assessment Unit (within DELWP)
Likelihood	Likelihood is the chance that something might happen. Likelihood can be defined, determined, or measured objectively or subjectively and can be expressed either qualitatively or quantitatively (using mathematics).
m BGL	Metres below ground level
MF1	Siltstone with interbedded sandstone, slightly weathered to fresh. Joints relatively closely spaced. Blocky rock mass with joint spacing of approximately 200 mm. Some faults and shears. Dyke intrusions relatively unweathered
MF2	Siltstone and sandstone, generally moderately weathered. Blocky rock mass containing decomposed seams, shears and faults. Dykes where present are weathered to clay
MF3	Siltstone and sandstone, generally highly weathered. Siltstone beds may be extremely weathered whilst sandstone is less weathered. Closely spaced discontinuities. Contains decomposed seams. Dykes where present are weathered to clay
MF4	Generally extremely weathered siltstone and sandstone, predominantly hard clay. Discontinuities may present as fissures. Dykes where present are completely weathered to clay
MFB	Metropolitan Fire Brigade
MMRA	Melbourne Metro Rail Authority
MTM	Metro Trains Melbourne
MURL	Melbourne Underground Rail Loop (City Loop)

Term	Definition
Numerical Model	A computer model that is designed to simulate and reproduce the mechanisms of a particular system
Palaeovalley	Channel in a basal geological unit that is infilled with layered sediments and lava flows that are much younger in geological age than the basal unit
Project Boundary	The proposed project boundary established for the project defines the area in which the project components would be contained. The proposed project boundary encompasses all areas that would be used for permanent structures and temporary construction areas (both above and below ground). It provides the basis for the specialist assessments undertaken for the EES and would be refined through the EES process.
PTV	Public Transport Victoria
PZoI	Potential Zone of Influence
RL	Reduced Level, a level relative to a stated datum
RM Act	*Road Management Act 2004*
Sound	In good condition, not damaged
SLS	Serviceability Limit State
Tanked	An impermeable barrier incorporated into a final lining of an underground excavation to limit the inflow of water
TBM	Tunnel Boring Machine, circular in cross section, it is used to excavate tunnels through various geological conditions
TI Act	*Transport Integration Act 2010*
TRG	Technical Reference Group
UCS	Unconfined Compressive Strength
UDL	Uniform Distributed Load
ULS	Ultimate Limit State
VCCC	Victorian Comprehensive Cancer Centre
VHR	Victorian Heritage Register

Executive Summary

The proposed Melbourne Metro Rail Project (Melbourne Metro) comprises two rail tunnels from Kensington to South Yarra, travelling underneath Melbourne Central Business District (CBD), as part of a new Sunbury to Cranbourne/Pakenham line to form the new Sunshine-Dandenong Line. Five new underground rail stations would connect the new system with existing transport, business, and health and education hubs.

As in the case of any large tunnelling project, the potential for ground movement exists where excavations would be undertaken as part of Melbourne Metro works. Ground movements may occur above and adjacent to Melbourne Metro works due to underground excavations; open cut excavations; consolidation settlement of compressible soils due to groundwater drawdown and/or slope instability.

Buildings, utilities and civil infrastructure such as roads, rail lines and bridges may be subjected to the effects of ground movements (settlement) caused by construction of the tunnels, stations, shafts and portals and/or associated drawdown effects.

A Ground Movement and Land Stability Assessment was undertaken by Golder Associates and a summary report describing this work is appended to this report along with an Interpreted Geological Setting Summary Report. The Golder Associates assessment outputs are integrated through this report and informed the preliminary assessment, by AJM JV, of the potential ground movement effects on buildings, structures and services along the Melbourne Metro alignment.

The Potential Zone of Influence relating to ground movement is defined by the estimated 5 mm excavation induced ground surface settlement contours and the estimated 10 mm consolidation settlement contours. Experience from tunnelling projects over past decades has shown that structures subjected to smaller settlements than these have negligible or no effects from the movements. Structures and underground services located between the contours defined by the above zones are considered within the Potential Zone of Influence. The potential effects of ground movement have been assessed for a representative sample of buildings, utilities and key civil infrastructure along the alignment.

Based on the current interpreted geological models, the tunnels alignment would be predominantly located within favourable geological units for ground stability, while meeting the key requirement to achieve safe design gradients for Melbourne Metro rail operations.

This impact assessment reviewed the possible degree of damage to buildings and infrastructure that would be caused by the excavations associated with Melbourne Metro, considering the structural type, the current condition of the structure and the differential settlement across the structure. The impact assessment established the possible mechanisms leading to ground movement, estimated the settlements, and predicted the category of potential damage.

Generally, the potential impacts to property were found to be negligible or minor and within acceptable parameters. In some cases, mitigation would be required to achieve acceptable outcomes and limit the predicted effects so that there would be no impacts greater than minor. For such cases, the assessment includes descriptions of the mitigation measures that could be applied, and where management or mitigation would need to be applied. These measures are standard tunnelling construction practices that have already been included in the assessments and the derivation of the impacts. There remain a number of structures where the predicted damage level is moderate. These would require further investigation based upon additional data along with discussion with the relevant stakeholders to confirm that the potential impacts would be acceptable, or to identify any further mitigation measures necessary.

Prior to construction, detailed condition surveys of potentially affected structures would be conducted that may identify increased vulnerability of some structures. These structures could have higher susceptibility to adverse impacts from ground movements. In these cases, additional mitigation measures may be required.

A list of the anticipated ground movement risks is provided. Completion of further geotechnical and hydrogeological investigations and interpretation of these conditions prior to construction would assist in estimating identified risks. Effective implementation of the mitigation measures would be expected to reduce the majority of potential impacts to either negligible or minor.

With the mitigation measures applied, the estimated impacts associated with the described risk pathways for ground movement are considered to be acceptable. The identified mitigation measures are typical and proven tunnelling construction techniques, and would be applied effectively with the appropriate management in place.

1 Introduction

This report provides a preliminary assessment of potential ground movements that may result from Melbourne Metro works as well as potential subsequent ground movement impacts on existing structures, infrastructure and utilities as well as selected approved future developments within the estimated Potential Zone of Influence relating to ground movement, of the proposed Melbourne Metro. Some of the potential issues considered in this report overlap with considerations addressed within other assessments including but not limited to:

- Technical Appendix E *Land Use and Planning*
- Technical Appendix J *Historical Cultural Heritage*
- Technical Appendix O *Groundwater.*

1.1 Project Description

The proposed Melbourne Metro comprises two nine-kilometre long rail tunnels from Kensington to South Yarra, travelling underneath Swanston Street in the Central Business District (CBD), as part of a re-configured Sunbury to Cranbourne / Pakenham line.

The infrastructure proposed to be constructed as part of Melbourne Metro broadly comprises:

- Twin nine-kilometre rail tunnels from Kensington to South Yarra connecting the Sunbury and Cranbourne/ Pakenham railway lines (with the tunnels to be used by electric trains)
- Rail tunnel portals (entrances) at Kensington and South Yarra
- New underground stations at Arden, Parkville, CBD North, CBD South and Domain with longer platforms to accommodate longer High Capacity Metro Trains (HCMTs). The stations at CBD North and CBD South would feature direct interchange with the existing Melbourne Central and Flinders Street Stations respectively
- Train/tram interchange at Domain station.

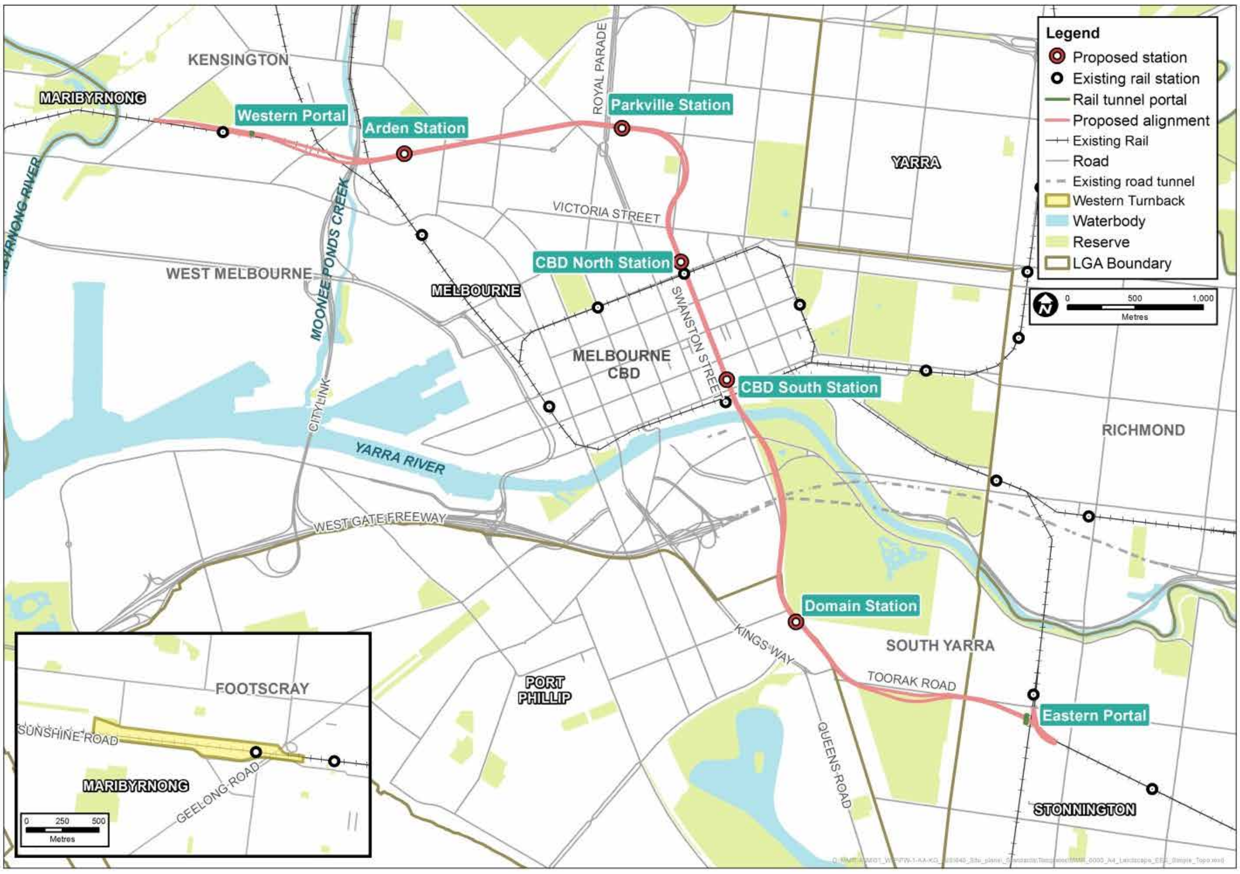

Figure 1-1 Map of the proposed Melbourne Metro alignment and five underground stations

1.2 Purpose of this Report

The purpose of this report is to assess the risk of ground movement and assess the potential for impacts to existing assets including buildings, infrastructure and utilities within the proposed project boundary. The impacts are evaluated based on potential damage that may arise from the movement of the ground resulting either directly from the excavation of underground and open cut structures, or indirectly through the response of the ground to changes in the groundwater levels.

Where these effects could lead to unacceptable damage, if not managed, the assessment describes potential risk mitigation measures that could be applied. The recommended Environmental Performance Requirements are framed to ensure appropriate mitigation and management measures would be adopted and implemented in the design and construction of the Melbourne Metro.

Assessment of temporary works that would be needed to enable permanent works to be built are outside the scope of this assessment. Temporary works are so called as they would be removed after use.

The impacts of the Melbourne Metro works, once constructed, on future developments are addressed in the Technical Appendix E *Land Use and Planning*.

1.3 Project Precincts

Table 1-1 and Table 1-2 provide a summary of the components of the Concept Design and where present, alternative design options to the Concept Design. The components are shown on the plans contained in the EES Map Book.

For assessment purposes, the proposed project boundary has been divided into precincts as outlined below. The precincts have been defined based on the location of project components and required construction works, the potential impacts on local areas and the character of surrounding communities.

The construction methods to be adopted in the Concept Design are described in Section 1.5. Control measures inherent in the Concept Design and that would limit ground movement are described in Section 6.5.1.

The nine precincts are shown in Figure 1-2.

Table 1-1 Precinct Summary

Precinct	Section	Subdivision
Precinct 1	Tunnels	Twin tunnels from the western portal to Arden station
		Twin tunnels from Arden station to Parkville station
		Twin tunnels from Parkville station to CBD North station
		Twin tunnels from CBD North station to CBD South station
		Twin tunnels from CBD South station to Domain station
		Twin tunnels from Domain station to the Eastern Portal
Precinct 2	Western portal (Kensington)	-
Precinct 3	Arden station	-
Precinct 4	Parkville station	-
Precinct 5	CBD North station	-
Precinct 6	CBD South station	-
Precinct 7	Domain station	-
Precinct 8	Eastern portal (South Yarra)	-
Precinct 9	Western turnback (West Footscray).	-

1.4 Study Area

The ground movement study area is bounded by and includes the western portal and eastern portal, the full extent of the proposed tunnels plus the five new station precincts and western turnback precinct as listed in Table 1-1.

The Potential Zone of Influence for the purposes of this ground movement and land stability assessment is defined by the land area impacted by the following ground movement mechanisms:

- Underground excavation induced ground movement including tunnels, cross passages and cavern stations
- Open cut excavation induced movement at shafts, portals and stations
- Consolidation settlement of compressible soils due to groundwater drawdown or loading at the surface.

As the ground movements determined in this assessment include those generated by changes in the groundwater levels, a wider study area than the immediate vicinity of the tunnels has been considered. The extent of the consolidation settlement impacts assessment is based on the groundwater assessments and the groundwater drawdown contours documented in the Technical Appendix O *Groundwater*.

The vertical extent of the study area is based on the proposed concept design vertical alignment, up to 40 m below ground level.

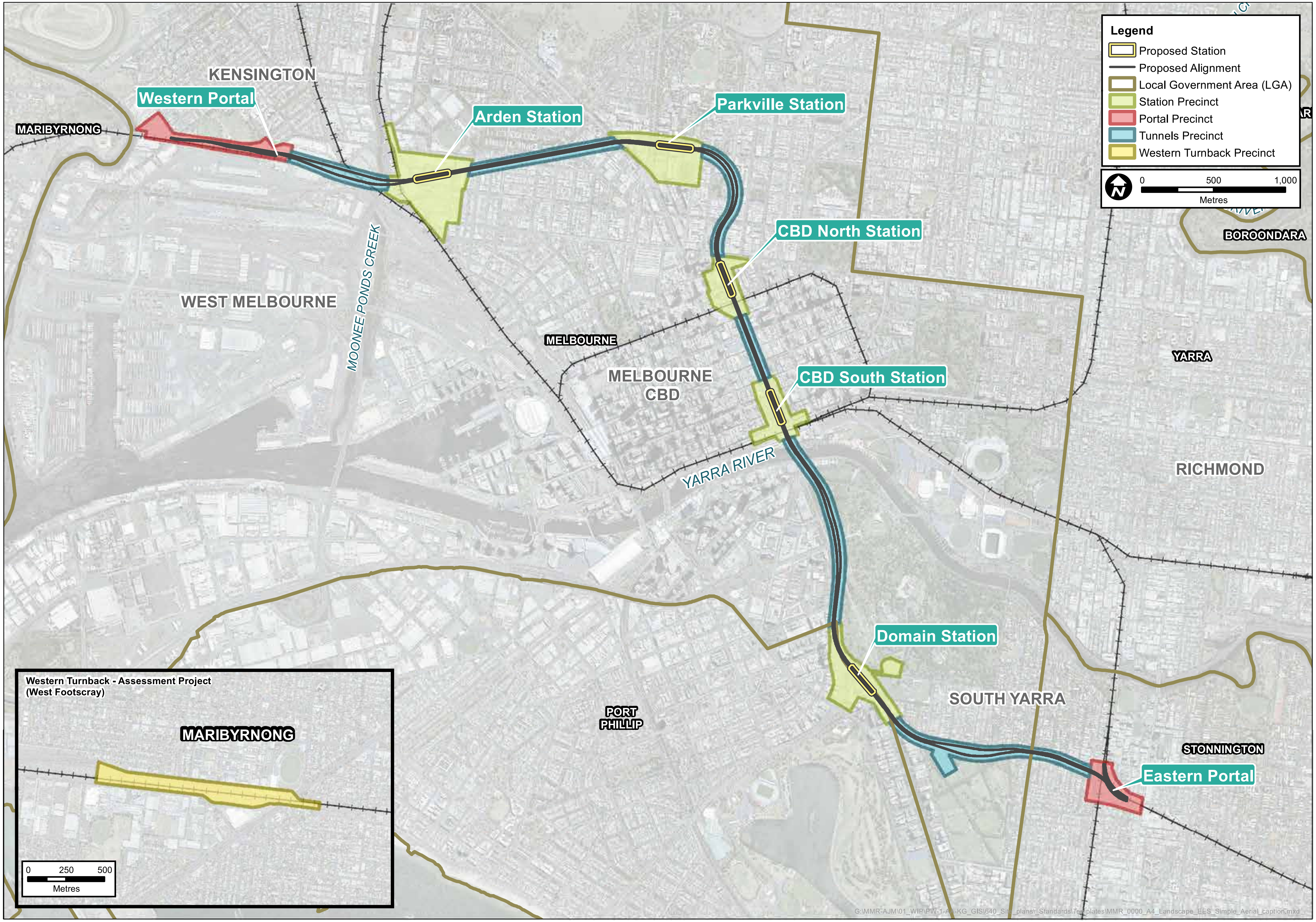

Figure 1-2 Melbourne Metro precincts

1.5 Proposed Construction Methods

Proposed construction methods would involve bored tunnels, mined tunnels, cut and cover construction of station boxes at Arden, Parkville and Domain, entrances and other shafts, and portals, and mined cavern construction at CBD North and South. The project would require planning and environmental related approvals to proceed.

A summary of the proposed construction elements included in each precinct scheme is provided in Table 1-2.

Table 1-2 Summary of potential Melbourne Metro under which this assessment was prepared

Precinct	Structure(s)	Potential construction scheme	Water Tightness	
			Construction Stage	Operational Stage
1. Tunnels	All tunnels, except the section between CBD North and CBD South stations.	Driven twin tunnels using TBM with precast reinforced concrete segmental lining. Hydrophilic gaskets to provide required water-tightness.	Tunnels: Undrained; Cross passages: Drained.	Undrained – assumed that all structures are “sealed” to a water tightness classification of Haack[1] Class 3 or equivalent for retaining wall systems for underground structures.
	Tunnels, CBD North station to CBD South station.	Mined technique for twin tunnels using road header or excavators.	Drained.	
	Linlithgow Avenue emergency access shaft.	Soldier Piles with shotcrete lagging or similar retaining walls, with segmental shaft lining at depth.	Drained, with control measures to limit groundwater inflows.	
	Fawkner Park emergency access shaft (both options).	Soldier Piles with shotcrete lagging or similar.	Drained.	
	Cross Passages.	Mined technique using road header or excavator.	Drained.	
2. Western portal	Embankment tie-in.	Earthen structure (above existing ground level). Precast reinforced concrete walls or reinforced soil wall.	Drained.	Drained.
	Dive structure.	Secant pile walls with precast reinforced concrete walls at shallow sections.	Undrained, incidental leakage to be managed.	Undrained – assumed that all structures are “sealed” to a water tightness classification of Haack[1] Class 2 or equivalent for retaining wall systems for underground structures.
	Cut and Cover Section.	Secant piles.		
	Western TBM retrieval point.			
3. Arden station	Station box.	Diaphragm walls.	Undrained, incidental leakage to be managed.	
4. Parkville station	Station box.	Soldier Piles or similar retaining walls.	Drained.	

Precinct	Structure(s)	Potential construction scheme	Water Tightness	
			Construction Stage	Operational Stage
5 & 6. CBD North and CBD South stations	Station caverns, adits and access shafts.	Mined cavern and adits constructed with primary rock support of rock bolts and shotcrete. Permanent lining is cast in place reinforced concrete. Access shafts soldier piles or similar retaining walls.	Drained, with control measures to limit groundwater inflows.	
7. Domain station	Station box.	Diaphragm walls.	Undrained, incidental leakage to be managed.	
8. Eastern portal	Eastern TBM retrieval point.	Secant piles.	Drained	
	Cut and Cover Section.			
	Dive structure.	Secant pile walls with precast reinforced concrete walls at shallow sections		
	Cutting retention system (existing rail corridor).	Soil nail walls, where appropriate and contiguous bored piles.		Drained.
9. Western Turnback	No underground or open excavations proposed in current scheme.	NA	NA	NA

1 See Section 1.5.1 for note on Haack Classifications

1.5.1 Water tightness of proposed Melbourne Metro structures

The water tightness is described using a classification system proposed by Haack 1991 to determine the degree of water tightness required of an underground structure. This allows a description of the allowable inflows and is also linked to the damage that would occur if the criterion was exceeded. The daily leakage limits for each class are defined for the flows in a short reference length, 10 m, considered a local peak flow, and the inflow over an extended reference length, 100 m, which would be more like the average inflow for that section of tunnel.

Haack Class 2 is characterised as substantially dry, suitable for station tunnels, i.e. the caverns, with slight isolated patches of moisture detectable on the surface of the constructed lining.

Haack Class 3 is characterised as capillary wetting, generally suitable for tunnels, with locally restricted patches of moisture occurring, but with no trickling water evident.

The criteria are expressed as the quantity of daily leakage of water per unit of area of the tunnel lining over the reference length.

The values of the Haack classes used in this assessment are listed in Table 1-3.

Table 1-3 Haack classification system for water tightness of underground structures

Tightness Class (Haack Class)	Moisture Characteristics	Permissible Daily Leakage Water Quantity (litre/sq. m) given a reference length of:	
		10 m	100 m
2	Substantially dry	0.1	0.05
3	Capillary wetting	0.2	0.1

Haack 2 or 3 would be adopted depending on detailed design stage assessment to confirm minimum requirements. The hydrogeological modelling completed to date and ongoing modelling considers various scenarios. Results of hydrogeological modelling completed to date are summarised in the Golder Associates Interpreted Hydrogeological Setting EES Summary Report appended to Technical Appendix O *Groundwater*.

These values readily suit hydrogeological modelling, allowing an analysis of the effects of the inflows on the ground water system. While the classification system is well known though the tunnelling industry, the tests at very low flows are somewhat subjective, and the Melbourne Metro might adopt an equivalent system for the detailed design.

For the purposes of the preliminary hydrogeological modelling of the cut and cover structures, the system has been used to describe the maximum groundwater inflows adopted in the Concept Design.

The water tightness described in Table 1-1 reflects that modelled in the hydrogeological modelling completed to date. The primary consolidation settlement assessment is based on these results.

1.6 Key Issues

Key project wide issues include:

- The potential to encounter unexpected geological conditions along the project alignment. The conceptual geological model is adequate to allow an assessment of the project for the purposes of the EES. The model's reliability is a function of the quality and quantity of ground information collected to date and the geological complexity at the location in which the boreholes have been drilled. Section 5.2 of Appendix A describes the level of reliability in the interpreted geological model along the Concept Design Alignment
- The potential for adverse impact to existing assets through ground movement associated with construction stage works, or associated with the project operational stage.

The key issues associated with the Concept Design are shown in Table 1-4.

Table 1-4 Key issues associated with the Concept Design

Concept Design	Issue
Vertical Alignment Project – Vertical Design	• Magnitude and profile of excavation induced ground settlement is related to vertical alignment, among other factors. • Underground excavation induced settlement impacts are related to tunnel and cavern cover. • Tunnel and structure excavation levels are generally below the groundwater table level.
Western portal	• Soft soils sensitive to additional loading and changes in groundwater levels.

Concept Design	Issue
Yarra River Crossing – TBM under the river	• Local uncertainties in a complex geological model. • Mixed conditions near the north bank of the Yarra River: TBM would pass through soft soils into very hard, potentially permeable rock. • Potential for mixed conditions for much of the alignment under the Yarra, if the upper basalt flow is deeper than expected or the lower basalt flow is intercepted. • Potential absence of basalt cover or very poor quality rock cover in the tunnel crown, resulting in high volume ground losses or instability of the excavation face without close TBM control and/or ground improvement. • Close proximity to Princes Bridge piers and abutments. • Aquifer connectivity with soft sediments in the region, potential for significant groundwater drawdown impacts some distance from the site.
CityLink tunnels crossing – Above CityLink tunnels	• Very low cover in Brighton Group leading to – Tunnelling induced settlement with narrow trough – Potential for sinkhole development without strict TBM control and/or ground treatment. • Potential to intercept Holocene aquifer near Alexandra Avenue creating damming effects or excessive drawdown resulting in local consolidation settlement, if difficult conditions are encountered.
TBM Southern launch site	
Fawkner Park open space and tennis courts	• Combined effects of tunnel induced settlement and shaft excavation retention system lateral deflection.
Domain Road road reserve and adjacent parklands	• Combined effects of tunnel induced settlement and station excavation retention system lateral deflection.
Emergency access shafts	
Fawkner Park north east shaft location	• Ground movement caused by retaining wall deflections in combination with adjacent tunnel induced settlement.
Queen Victoria Gardens, adjacent to Linlithgow Avenue	• Ground movement caused by retaining wall deflections in combination with adjacent tunnel induced settlement. • Temporary dewatering impacting local groundwater recharge schemes.

1.6.1 Alternative Design Options

The key issues associated with the alternative design options are identified in Table 1-5.

Table 1-5 Key issues associated with alternative design options

Alternative Design Option	Issue
CityLink tunnels crossing – Below CityLink tunnels	• Potential to intercept Holocene aquifer creating damming effects or excessive drawdown if the aquifer is higher than expected or the tunnelling allows hydraulic connection and de-pressurising.

Alternative Design Option	Issue
Emergency access shafts	
Using the location of the Fawkner Park TBM launch site	• Ground movement caused by retaining wall deflections in combination with adjacent tunnel induced settlement.
Shaft located in Tom's Block	• Ground movement caused by retaining wall deflections in combination with adjacent tunnel induced settlement. • Temporary dewatering impacting local groundwater recharge schemes.
Western portal	• Similar to base option

2 Scoping Requirements

2.1 EES Objectives

The following draft evaluation objectives (Table 2-1) are relevant to ground movement and identify the desired outcomes in the context of potential project effects. The draft evaluation objectives provide a framework to guide an integrated assessment of environmental effects of the project, in accordance with the *Ministerial guidelines for assessment of environmental effects under the Environment Effects Act 1978.*

Table 2-1 EES Evaluation Objective: Land stability

Draft EES Evaluation Objective	Key Legislation
Land Stability – To avoid or minimise adverse effects on land stability that might arise directly or indirectly from project works.	*Planning and Environment Act 1987.*

This report does not deal with individual impacts of settlement on all structures, utilities and infrastructure. Settlement assessments of all structures, utilities and infrastructure would be undertaken during detailed design prior to construction and the results reported separately. Potential impacts of detailed design construction schemes and construction methodology, using refined structural and geotechnical models, would be assessed to confirm consistency of assessment outputs with preliminary assessments conducted to date.

This report does, however, provide an assessment of the extent of likely ground movement impacts along the Melbourne Metro alignment, and identifies a series of performance requirements to ensure that any adverse effects on land stability are controlled within acceptable limits.

2.2 EES Scoping Requirements

The following extracts from the Scoping Requirements, issued by the Minister for Planning, are relevant to the land stability draft evaluation objectives (Table 2-2).

Table 2-2 Scoping Requirements for Land Stability

Aspect	Relevant Response
Key Issues	• Potential for project works to cause or lead to reduced ground stability, which could adversely affect properties, structures or other values.
Priorities for characterising the existing environment	• Identify and map ground conditions along and in the vicinity of the project alignment. • Identify ground conditions which might be susceptible to instability, in particular if subjected to tunnelling, deep excavation or dewatering.
Design and mitigation measures	• Identify design and management measures to maintain ground stability where risks of potential instability have been identified.
Assessment of likely effects	• Assess potential for project works to lead to immediate or incremental reduction of ground stability.
Approach to manage performance	• Describe principles to inform a monitoring program to detect ground instability, if it occurs after project works commence, including after construction. • Describe principles to be adopted to formulate contingency actions which might be implemented if potential ground instability resulting from the project is identified.

This report responds to each requirement in the context of an evaluation of the environmental impacts associated with ground movements covering the construction and operation of the Melbourne Metro.

3 Legislation, Policy and Guidelines

The *Environment Effects Act 1978* provides for assessment of proposed projects (works) that are capable of having a significant effect on the environment.

There are no specific Commonwealth or Victorian laws and policies directly relating to ground movement. However, some laws and policies that apply to groundwater (including those relevant to dewatering and recharging through bores) are applicable to the assessment and mitigation of ground movement and land stability.

Table 3-1 summaries the relevant primary legislation that applies to Melbourne Metro as well as the implications, required approvals and interdependencies and information requirements associated with obtaining approvals. In addition, the objectives of the *Planning and Environment Act 1978* require that planning authorities consider the environmental, economic and social effects of a planning scheme amendment (including in respect of ground movement). Additional legislation would be applicable to heritage structures.

Table 3-1 Primary legislation relating to potential ground movement and land stability

Legislation / policy	Key policies / strategies	Implications for this project	Approvals required	Timing / Interdependencies
Commonwealth				
National Environment Protection Council Act 1994	• NEPC 1999. The National Environment Protection (Assessment of Site Contamination) Amendment Measure 2013 (No. 1) Amendment of the National Environment Protection (Assessment of Site Contamination) Measure 1999. • National Health and Medical Research Council (NHMRC) 2008. Guidelines for Managing Risks in Recreational Water. • Australian and New Zealand Guidelines for Fresh and Marine Water Quality (ANZECC/ARMCANZ (2000). • Australian Drinking Water Guidelines (NHMRC/NRMMC (2011)), National Water Quality Management Strategy. • Minimum Construction Requirements for Water Bores in Australia (NUDLC, 2012).	Project wide	NA	NA
State				
Water Act 1989	Allocating surface water and groundwater throughout Victoria – including for dewatering. Sections 67 and 72 – issuing bore licences These licensing systems are administered by the rural water authorities (Southern	Groundwater dewatering and recharge through bores requires a licence from Southern Rural	Southern Rural Water – licence to construct bores for dewatering or recharge.	These licences require a hydrogeological assessment to be undertaken.

Legislation / policy	Key policies / strategies	Implications for this project	Approvals required	Timing / Interdependencies
	Rural Water in southern Victoria).	Water (for construction of bores and for pumping from/to bores).	Southern Rural Water – licence to pump from or inject to groundwater.	
Planning and Environment Act 1978	No specific requirements but contains a general requirement on the part of the planning authorities to consider the social, economic and environmental effects of a planning scheme amendment.	Should consider impacts of ground movement on environment as well as social and economic effects.	No specific requirements but relevant to planning scheme amendment.	NA
Environment Protection Act 1970	State Environment Protection Policy Groundwaters of Victoria, 1997 State Environment Protection Policy Waters of Victoria, 2003.	Must consider the impacts on groundwater and surface water quality.	EPA Discharge licenses for disposal to surface waters or sewers.	NA

4 Background

4.1 Regional Context

The proposed Melbourne Metro alignment runs through a number of distinctly different areas of Melbourne, distinguished by differing building types, age, condition and sensitivity of infrastructure, land uses and geological conditions.

A surface geological map sheet set of the alignment is attached in Appendix A of this report and demonstrates the subsurface geological and hydrogeological framework adopted in the preliminary assessments. Ground surface level relative to the tunnels elevation is indicated in the geological profile drawing found in the Appendix A of this report.

Melbourne Metro is proposed to tie in to the existing Sunbury rail tracks at the proposed western portal on the existing embankment above the river flats of the Maribyrnong River. The railway tracks form a boundary between the largely recreational and residential properties to the north and the industrial and railways goods yards to the south. The river flats are typically at approximately RL 4 m AHD. Heading to the east past South Kensington station, Melbourne Metro dives underground below higher ground, up to RL 10 m, formed by a residual basalt flow. Approaching Lloyd Street in Kensington, the buildings either side of the railway are industrial. Crossing Lloyd Street and the North Yarra Main Sewer alignment, the proposed alignment enters the flood plain of the Moonee Ponds Creek, passing under the West Melbourne Terminal Station. Near the Moonee Ponds Creek, the proposed alignment passes under multiple railway tracks, railway bridges and the viaduct of CityLink. Through this area, once away from the portal, the depth to the top of the proposed tunnels varies from 10 m to 20 m below ground level.

Arden station is proposed to be constructed on the eastern side of the Moonee Ponds Creek, in an area of former railway sidings, now used for industrial and commercial purposes. From Dryburgh Street in North Melbourne, the buildings are predominantly residential, both Victorian era and modern buildings, becoming more commercial approaching Flemington Road. Along Grattan Street, where the Parkville station is proposed to be constructed, the alignment passes between hospital and medical buildings and then the University of Melbourne. From the east of the proposed Arden station to the proposed station at Parkville, the ground surface level and the alignment in the bedrock material both rise in elevation. Along this section of the alignment, the depth to the crown of the proposed tunnels is typically 15 m, increasing to 20 m under local high points in the topography.

From Parkville to the northern end of the CBD at Victoria Street, the surface structures include buildings associated with the University of Melbourne, merging into medium rise commercial buildings near Swanston Street, interspersed with more recent residential developments. The surface level at the high point near Barry Street in Carlton is RL 37 m AHD with depth to top of the proposed tunnels of 20 m. The surface levels fall to approximately RL 22 m, while the depth to the proposed tunnels increases to over 30 m.

Through the CBD, the proposed tunnels pass between a mixture of heritage buildings and more recent buildings with a variety of commercial, academic and residential uses. The proposed alignment runs beneath the tram tracks in Swanston Street and the Melbourne Main Sewer under Flinders Street, in addition to numerous major drainage and communications utilities. The proposed tunnel depth varies from 35 m to 25 m, as the surface levels fall to just below RL 10 m. The cover over the proposed CBD North station cavern would be approximately 25 m, while at the proposed CBD South station, the cover would be approximately 15 m.

The proposed alignment passes beneath the rail tracks approaching Flinders Street Station before crossing under the Yarra River and Princes Bridge, and the parklands and roadways to the south. The river flats on the south bank of the Yarra are typically at RL 4 m to RL 7 m and are formed by the mixed soft and hard materials of the Yarra Delta infilling old valleys.

Approaching Linlithgow Avenue, the surface rises on the next zone of bedrock, with the proposed tunnels being under parklands. The tunnels rise to pass over the CityLink tunnels, reducing the depth to the tunnels to less than 4 m.

In the parklands and along the service road of St Kilda Road, south of the CityLink crossing, the ground surface level remains near RL 10 m. While the proposed alignment increases its cover to 10 m at the proposed Domain station, which lies between Melbourne Grammar School to the east and commercial and residential properties to the west.

As the proposed alignment turns eastwards from St Kilda Road near Toorak Road, it runs between the residential and commercial buildings near the junction with Toorak Road before moving under the northern end of Fawkner Park with at least 15 m cover. The ground rises with a high point just west of Punt Road at RL 27 m. The depth to the proposed tunnels increases to 25 m under this high point formed by the bedrock. To the east of Punt Road, the proposed alignment rises towards the eastern portal and the level of the land falls. As they would pass beneath a mix of residential and commercial buildings, predominantly Victorian era or early twentieth century, the depth of the proposed tunnels decreases to a minimum value of 10 m near Osborne Street, South Yarra.

The proposed tie-in to the Dandenong tracks is in the South Yarra railway cutting to the west of Chapel Street. This area is surrounded by residential buildings, with some of the original housing stock being replaced by more recent blocks of flats.

4.2 Geological Setting

The proposed project alignment traverses predominantly bedded and folded sedimentary rock known as the Melbourne Formation, which forms the basement rock through Melbourne. The tunnels would be located within Melbourne Formation between the Arden station precinct and the Yarra River crossing. Layered soils of varying composition and consistency interbedded with tongues of basalt are encountered from the Maribyrnong River to the Moonee Ponds Creek valleys (western portal to Arden station precincts), as well as at the Yarra River crossing. A layer of generally very stiff sedimentary soil is found overlying the Melbourne Formation from Kings Domain to the eastern portal and the tunnel passes through these materials along this eastern section of the project.

Appendix A of this report describes the interpreted geological and estimated engineering properties of the soils and rock that are present across the proposed alignment. Section 5.2 of Appendix A describes the level of reliability in the interpreted geological model along with the implications of the respective Ground Model Reliability Scores. The current levels of confidence in the geological model are adequate to allow an assessment of the project for the purposes of the EES. The current model would reliably inform the ongoing Procurement Stage Site Investigations program as well as this assessment.

Figure 4-1 presents the interpretive geological profile along the proposed alignment. More detailed sections are found in the Appendix A of this report.

The interpreted geological profile drawings provided in Appendix A to this report have been prepared based on geotechnical information collected up to September 2015. It should be expected that the model would be refined and updated following ongoing future geotechnical investigations undertaken for Melbourne Metro which would inform ongoing design and construction phases of the project.

4.3 Hydrogeological Setting

As documented in Technical Appendix O *Groundwater*, the highest groundwater elevations along the proposed alignment occur in the Parkville area and the lowest groundwater levels occur in the area of the CityLink tunnels. Based on the vertical alignment of tunnels and stations adopted for the Concept Design and measured groundwater elevations, the maximum height of groundwater above the proposed tunnel invert is 34 m. In some areas, the measured groundwater level is below the proposed tunnel invert. Regionally, the highest groundwater levels are associated with higher topographic areas and areas of groundwater recharge such as the Royal Botanic Gardens. The lowest measured groundwater elevations coincide with groundwater sinks such as the North and South Yarra Main Sewers, the City Loop tunnels and the CityLink tunnels, as well as deep basements within Parkville, the CBD and Southbank.

The Interpreted Hydrogeological Setting EES Summary Report is appended to Technical Appendix O *Groundwater*.

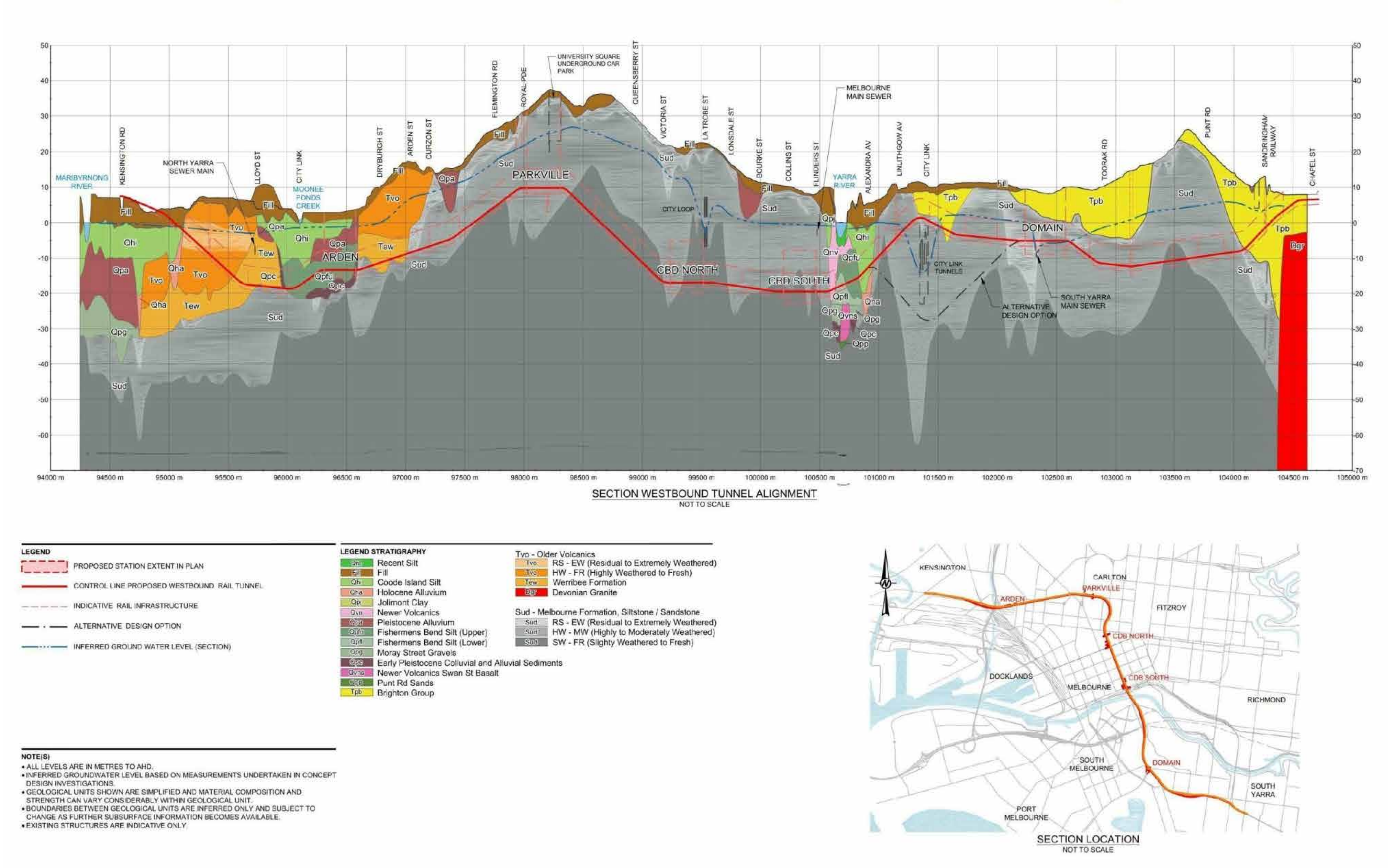

Figure 4-1 Long Section - Interpreted Geological Profile

4.4 Ground Movement Mechanisms

As in the case of any large tunnelling project, the potential for ground movement exists where excavations would be undertaken as part of Melbourne Metro works. Ground movements may occur above and adjacent to Melbourne Metro works due to the following mechanisms:

- Underground excavation induced ground movement
- Open cut excavation induced ground movement
- Primary consolidation settlement of soft soils, primarily Coode Island Silt
- Slope instability.

Movements can occur as a result of one or a combination of these mechanisms.

The magnitude and profile of ground surface impacts relate to:

- Site specific geological conditions
- Cover to underground excavation
- Proposed twin tunnel horizontal separation distance, where applicable
- Excavation methods and support installation sequence
- The type of ground support adopted
- Combination of effects with adjacent open excavations or consolidation settlement from changes in the water table levels, if applicable.

Localised geological and topographical variations within the Melbourne Metro alignment are influential in the degree of ground movement that would occur due to underground excavations. The tunnels alignment is located within predominantly favourable geological units for ground stability, where possible, while meeting the key requirement to achieve acceptable design gradients for rail operations.

Excavation of underground and open cut structures would produce ground movements above and adjacent to the excavation. The movements are usually downwards (settlements) with a horizontal component of displacement towards the excavation. These movements have potential effects on buildings, structures and utilities above or near to the new tunnels or excavations only where the ground movements at one part of the structure or utility are different from those at another part.

4.4.1 Underground Excavation Induced Ground Movement

Underground excavations with minimal surface footprint available for access, in contrast to excavation from the surface, would be used in the construction of many Melbourne Metro structures, including the tunnels, cross passages, adits and station caverns.

4.4.1.1 Tunnels, Cross Passages and Adits

The tunnels would be excavated by Tunnel Boring Machines (TBMs) or mined using road headers. Cross passages and adits (connection tunnels) would be excavated by road headers or other mobile plant with rock excavating tools. The majority of the tunnels, cross passages and adits are in ground suitable for excavating a full face advance of defined length per round, followed by the installation of the primary ground support. It is noted that some large adits would require staged excavations.

As tunnel excavations progress, the ground mass deforms ahead of and towards the tunnel excavation face. The ground may experience a slight heave or rise ahead of the TBM due to the outwards pressure imposed on the ground by the TBM.

Figure 4-2: Road header excavation

Above and behind the advancing excavation face the ground would experience deformation towards the excavation. There might be some further deformation over the length of unsupported tunnel between the face and the nearest installed ground support. The combined effects are typically quantified as a "volume loss" parameter. This is defined as a percentage of the tunnel cross sectional area that would manifest as settlement at ground surface level and is idealised to occur in the shape of a trough immediately above and extending outward from the underground excavation.

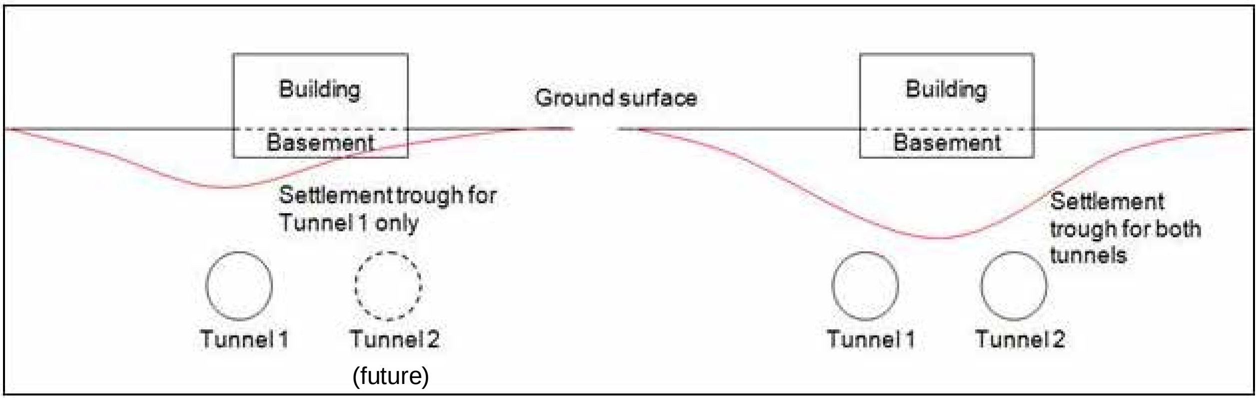

Figure 4-3 Exaggerated ground settlement troughs for single and twin tunnels

Figure 4-3 above shows the approximated inverse Gaussian settlement trough for a single tunnel excavation and twin tunnel excavation. Studies on previous tunnelling projects have shown that the settlement tough induced by tunnel excavation is well approximated by the Gaussian mathematical function.

Each tunnel excavation induces its own Gaussian settlement trough at the time of its construction. When the second tunnel is constructed, the settlement troughs from Tunnel 1 and Tunnel 2 are superimposed or combined to form a single settlement trough. The shape of the combined settlement trough would depend on the separation and relative depths of the two tunnels.

It can be seen from Figure 4-3 that the critical case indicating potential for adverse impacts to a structure could be during the construction stage as the building position along the ground settlement trough may vary through the construction stage.

4.4.1.2 Mined Caverns

The caverns would be mined principally using road headers but possibly also using other mobile plant with rock excavating tools to complete the profiles.

While the two caverns are in rock, the need to limit ground movement, and, incidentally, the size of the cavern in relation to the available equipment, would preclude excavation of the full cross section in a single pass. Cavern excavations would be conducted in stages of limited cross-sectional area, achieving the final excavation profile by means of a number of smaller excavations. An example is shown in Figure 4-4. The open area of the interim excavation face is effectively limited and supported with temporary support to minimise rock mass relaxation in the cavern crown, sidewalls and face, if required. In this case, assuming that steel cables and rock bolts are used to reinforce the rock, it can be seen that a substantial part of the ground support is installed before the full span of the rock is opened up. In order to limit the interaction between adjacent headings, the excavation faces would be separated by distance determined during detailed design, but typically greater than 15 m or 20 m along the cavern.

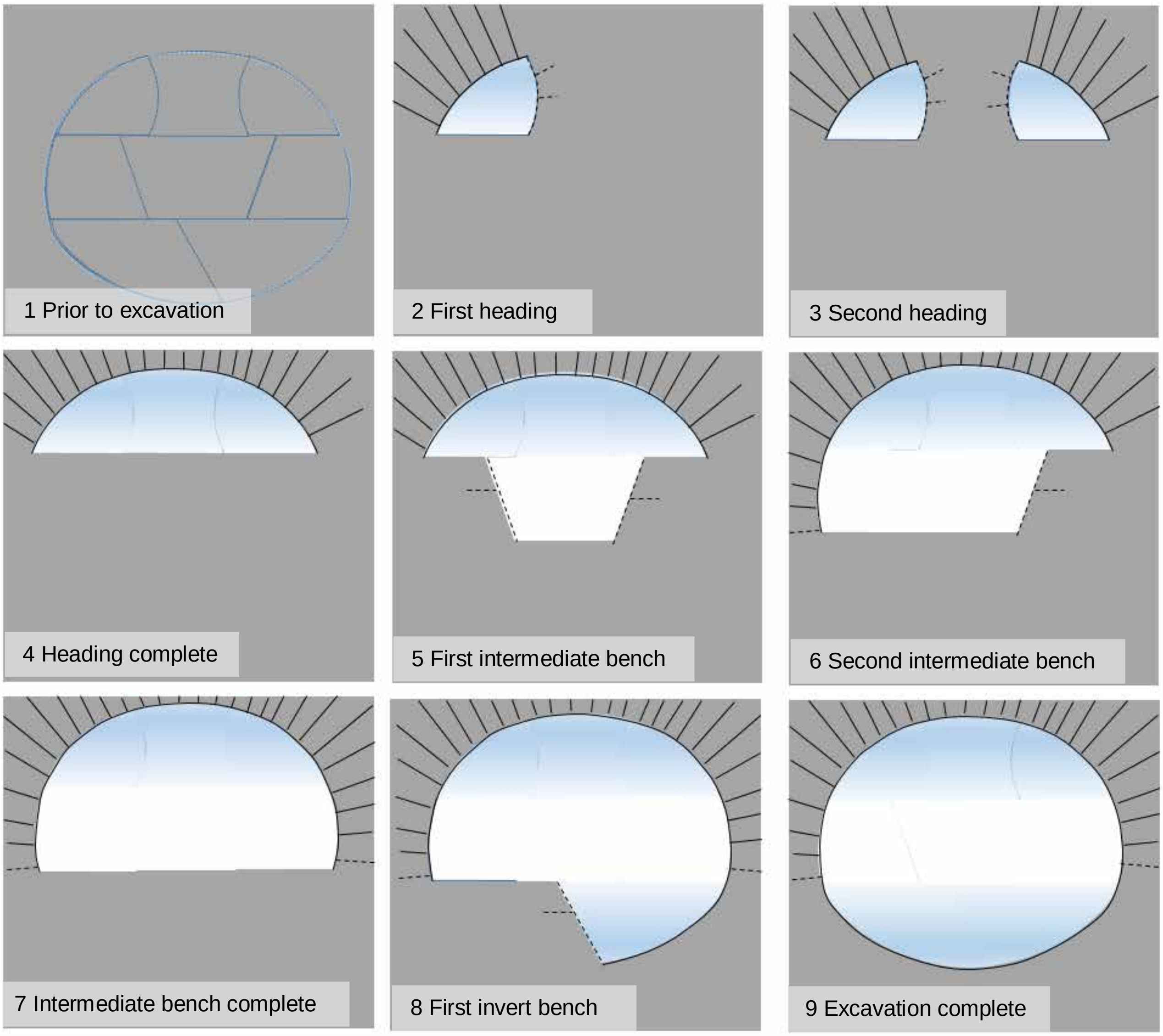

Figure 4-4 Example of an excavation and support sequence for a cavern

Maintaining the rock mass integrity, by minimising relaxation as the stress regime is altered, helps the rock mass to remain partially self-supporting and provides a safe working environment until permanent support is installed and reaches its design strength. With regard to ground movement, the early installation of the initial ground support and limited relaxation of the rock both assist in limiting the ground movement during excavation.

Similar to the ground response described for the tunnel excavation in Section 4.4.1.1, as each heading excavation progresses, the ground mass deforms ahead of and surrounding the excavation face. There might be some further deformation over the length of unsupported excavation between the face and the nearest installed ground support. In a similar way to the tunnels, the "volume loss" would manifest as a settlement trough at ground surface.

The settlement effects of each stage of excavation add to the previous settlements. However, the use of smaller headings allows tighter control of the ground, meaning that the sum of all the smaller excavation effects is less than if the cavern was excavated in larger sections.

Special design would be required at the intersections of adits with the caverns. As well as overall stability, the design would include a sequence of excavation and support installation that limits ground movement to levels that limit surface settlement to the acceptable limits.

4.4.2 Open Cut Excavation Ground Movement

Vertical ground movement can occur as a result of the lateral deflection of retaining walls at the shafts, station boxes, decline structures and cut and cover tunnel sections.

When retaining walls are constructed, some lateral relaxation of the retained ground mass can occur as the wall deflects under load. This can lead to small surface settlements up to a distance of 1.5 times the vertical height of the wall, from the wall. Buildings or other structures within this potential influence zone need to be assessed, as they might be adversely impacted by the ground movement.

Estimates of surface displacements resulting from open cut excavations were obtained from computer modelling and analysis of the ground mass-structure interaction undertaken during the concept design process.

Figure 4-5 shows exaggerated retaining wall deflection due to ground loading on one side and the corresponding vertical ground movement profile behind the wall.

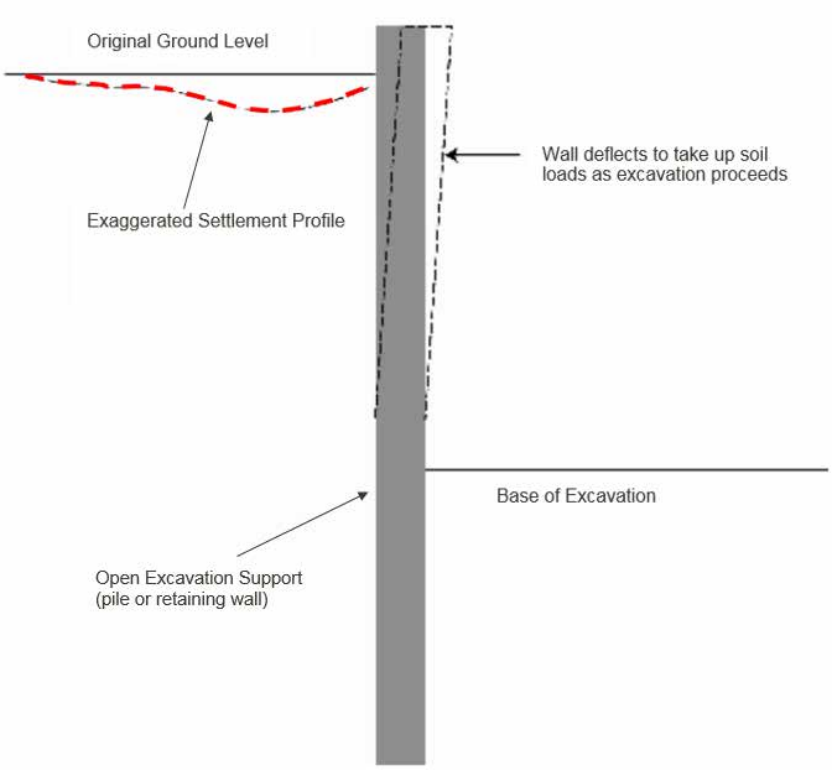

Figure 4-5 Open cut excavation induced ground movement

4.4.3 Consolidation Settlement

The volume of groundwater inflows and the extent of drawdown (both lateral extent and vertical magnitude) around Melbourne Metro works would depend on the depth and size of the excavation below the groundwater level and the hydrogeological characteristics of the aquifers and aquitards surrounding the works. Inflows (and therefore drawdown) can occur during both the construction and operational phases of the project.

Lowering of the water table in soft soils increases the stress between the soil particles as the buoyant effects on the particles are removed.

Effective stress increases as the pore water pressures dissipate.

As the water filled voids are drained, there is a change in volume due to compression in the soil matrix. This change in volume would be measurable at the ground surface as vertical settlement, namely primary consolidation settlement.

4.4.3.1 Primary Consolidation

Primary consolidation settlement may occur in softer soils due to groundwater drawdown, or new embankment loading. Drawdown describes lowering of the water table which may occur due to drainage or groundwater extraction. Large groundwater inflows could occur, if not controlled, during construction of some proposed Melbourne Metro structures, resulting in significant drawdown in their vicinity. Based on the information in Appendix A of this report, the zones along the alignment that are of interest in relation to potential consolidation settlement are those where compressible soils are found locally to the proposed alignment. Further, based on the Interpreted Geological Setting Report Geological Plan drawings in Appendix A of this report, these zones are:

- Western portal up to and including Arden station
- Yarra River crossing to Alexandra Gardens

- South Melbourne areas in the potential influence zone of Domain station.

The potential zones of influence do not necessarily overlie the project alignment but are located within the zones of compressible soils that could be affected by groundwater drawdown resulting from inflow to Melbourne Metro excavations. The extents of the compressible soils, namely the Coode Island Silt, relative to the proposed alignment is found in Appendix A of this report. A description of the geological characteristics and engineering properties of this unit is also contained in Appendix A of this report.

As measures to limit groundwater inflow would be adopted during construction, groundwater drawdown during the Melbourne Metro's operation phase would be insignificant and subsequently, primary consolidation settlement associated with this drawdown would be negligible.

If Melbourne Metro works reduce the groundwater pressures in nearby aquifers, in strata of high permeability, the effects can extend across a greater geographical area. This drawdown is short-term, and groundwater levels recover after the structures have been tanked. Construction stage measures to reduce groundwater inflows are likely to minimise the lateral extent of groundwater drawdown. However, anticipated recovery of the groundwater table during the operational stage and post construction stage would not reverse any settlement which might have already occurred.

4.4.3.2 Secondary Compression

As described in the Appendix B of this report, *Golder Associates Ground Movement Assessment EES Summary Report,* secondary compression is ongoing movement which occurs after primary consolidation of compressible sediments. It occurs both as a natural process, due to the consolidation occurring from the self-weight of the soil, as well as due to historical activities such as fill placement.

Historical records indicate that settlement in Coode Island Silt is occurring with no apparent change in loading. Historical measurements of secondary compression over the last century suggest an apparent linear trend rather than a diminishing rate, as described by conventional soil mechanics theory. It is possible that construction activities in the past century may have caused small settlements, even in some cases where the activities might not have been in the vicinity of the proposed Melbourne Metro corridor.

The current rate of secondary compression of Coode Island Silt within the project area is reported to be up to 10 mm per year (Ervin 1992), depending on the thickness of the deposit. As described in Appendix B of this report, the effects of secondary compression have not been assessed as the current rate is not expected to be exacerbated by Melbourne Metro activities or other environmental effects of Melbourne Metro. The potential influence of any long term background secondary compression which is currently occurring would be assessed once a settlement monitoring network has been installed and background baseline settlement rates established. Based on the local experience and the records of long term Coode Island Silt background consolidation available in the Melbourne area, the values of creep rates presented in Table 13 of the Golder Associates Appendix B report, may be considered for the purposes of the development of the Melbourne Metro Concept Design. These are suggested to inform the EES and are considered indicative. The creep settlement can also result in differential settlements within the footprints of existing structures. Differential settlement due to creep could be up to 30% of the total creep settlement over a 20 m distance.

4.4.4 Slope Instability

The existing rail cutting batters adjacent to the proposed eastern portal would be extended or re-cut as the existing rails are reconfigured to accommodate the Melbourne Metro dive structure and realigned surface railway tracks.

Retaining walls or soil reinforcement systems would be required at various locations to maintain batter stability where there is inadequate space to excavate the batter to a long-term stable angle. Widening of the existing corridor would result in retaining systems or batter crests being located in closer proximity to existing properties situated beside the rail corridor. Potential impacts to these properties are considered in Section 8.

Figure 4-6 shows a cut batter model with soil nail support.

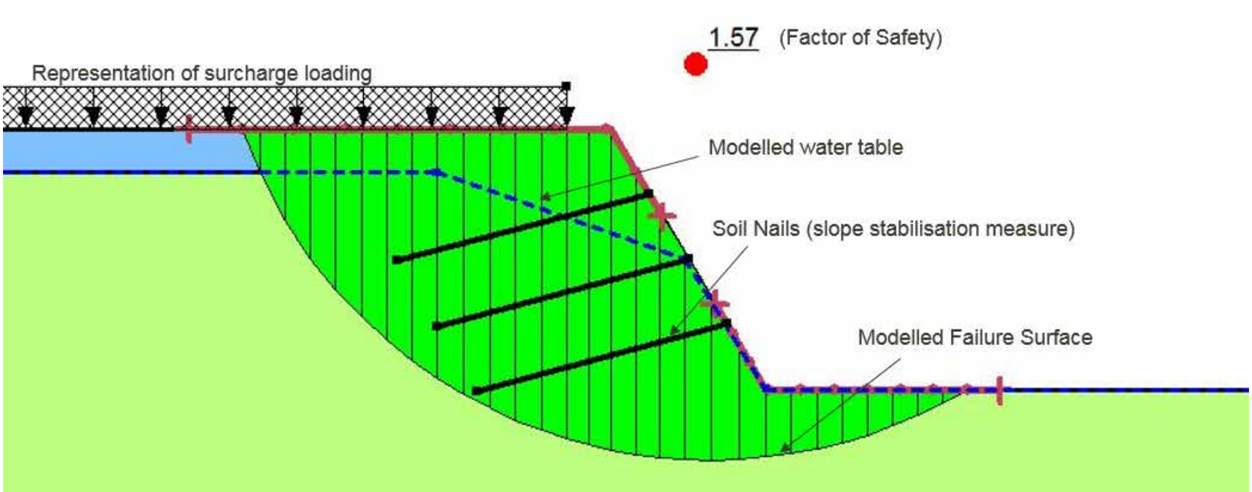

Figure 4-6 Example slope stability model with soil nail support

4.4.5 Seasonal Movements

Seasonal ground movement magnitudes vary according to geology type, groundwater level and soil moisture variations, temperature and the thickness, construction type and function of existing pavements. With the exception of secondary compression, ongoing seasonal movements across the Melbourne Metro alignment are estimated to vary between approximately 0 mm and 25 mm. Seasonal ground movement can only be established through site specific baseline monitoring. Similarly, existing structures may experience ongoing seasonal movements in response to seasonal ground movements in addition to other structure movements that might be attributed to natural foundation settlement, thermal effects and/or shrinkage. Existing structures would typically be designed to tolerate these movements attributable to natural phenomena without impacting the structural stability or serviceability.

4.5 Summary of Current Land Use

The Melbourne Metro alignment traverses and intercepts land with existing mixed use functions. A summary of some of the current land use features that are considered in the ground movement assessment are summarised in the matrix provided in Table 4-1.

Table 4-1 Matrix summarising predominant land use functions by precinct

Precinct	Commercial / Industrial Property	Residential Property	Heritage buildings / structures	Major Health Infrastructure	Educational Centres	Parklands	Rail Lines	Road Pavement	Tram Lines	Existing Utilities
1. Tunnels	X	X	X	X	X	X	X	X	X	X
2. Western portal	X	X				X	X	X		X
3. Arden station	X						X	X		X
4. Parkville station	X	X		X	X			X	X	X
5. CBD North station	X	X	X		X		X	X	X	X
6. CBD South station	X	X	X		X		X	X	X	X

Precinct	Commercial / Industrial Property	Residential Property	Heritage buildings / structures	Major Health Infrastructure	Educational Centres	Parklands	Rail Lines	Road Pavement	Tram Lines	Existing Utilities
7. Domain station	X	X	X		X	X		X	X	X
8. Eastern portal		X	X			X	X	X		X
9. Western Turnback (West Footscray).							X			

There are three rivers or creeks within the Potential Zone of Influence as described in the Technical Appendix O *Groundwater.* The Maribyrnong River is situated approximately 500 m west of the proposed western portal embankment tie-in to the existing rail lines. The TBM driven tunnels would pass beneath Moonee Ponds Creek immediately west of the Arden station precinct and beneath the Yarra River, immediately south of the CBD South station precinct.

Hydrogeological considerations around these major waterways in a regional context are described in Technical Appendix O *Groundwater*.

Technical Appendix J *Historical Cultural Heritage* identifies sites with potential to be affected by the works and are on the Victorian Heritage Register, within Heritage Overlays and those that are otherwise documented as being of heritage significance. The report describes the heritage values associated with these places and any risks and potential impacts on these values. Detailed design stage assessments would investigate the heritage significance of buildings with potential to be affected by the works.

The project geological and hydrogeological settings are described in Section 5 of this report.

4.6 Existing Structural Conditions

The Melbourne Metro alignment passes beneath and adjacent to many different buildings, infrastructure, and utilities as it crosses through the inner city areas of Melbourne, as outlined in Section 4 of this report.

It is unlikely that any of these structures have been subject to the effects of large diameter tunnels, aside from those in the vicinity of the City Loop tunnels, and possibly some of the sewer mains. However, many would have already experienced some form of ground movement generated by other sources.

Parts of the alignment pass through less stable conditions. An example is where the soils, such as those derived from underlying basalt, are sensitive to seasonal changes in moisture content. These would be encountered between the proposed western portal and Lloyd Street in Kensington. Similarly, movements from settlement might be experienced over some of the softer soils in the areas around the current and former creeks and rivers. Some structures might have been affected by inadequate foundation design or construction or previous adjacent excavations. Pre-construction baseline survey measurements, conducted over a number of seasons, would assist in determining where the ground and structures are already experiencing movement from one or a combination of these sources.

The manner in which a building or other structure responds to ground movements would depend upon its size and materials. A modern steel or reinforced concrete structure can be flexible, deflecting as the ground moves. In contrast, a masonry building, subject to similar displacements, could behave as a relatively brittle structure and respond by cracking. The interaction between a structure and ground movement is also influenced by the foundation type. Deep foundations might support a structure from outside the zone of movement, isolating the structure from the adjacent surface level changes.

5 Methodology

Four of the fundamental inputs to the ground movement and impacts assessment are:

- The Concept Design, as described in Section 1.5, comprising a horizontal and vertical alignment and proposed construction schemes
- The conceptual ground and geological models
- The results of the regional groundwater modelling and
- The location, types and condition of the existing assets that overlie or are located close to the proposed works.

5.1 A Collaborative Approach

As background to the method of assessment, the relationship between the EES Specialist Reports and the supporting Golder Associates EES Summary Reports is shown in Table 5-1.

Table 5-1 Relationship between EES reports

Relationship between EES Specialist Reports and the supporting Golder Associates EES Summary Reports		EES Specialist Reports			
		Ground Movement (this report)	Future Development Loading	Groundwater	Contaminated Land and Spoil Management
Golder Associates EES Summary Report	Interpreted Geological Setting	Appendix A of this report			
	Ground Movement Assessment	Appendix B of this report			
	Interpreted Hydrogeological Setting				
	Regional Groundwater Numerical Modelling				
	Contaminated Land Assessment				

This impact assessment relies on Technical Appendix O *Groundwater* and the content of the reports attached to this report as follows:

- Appendix A Golder Associates Interpreted Geological Setting EES Summary Report
- Appendix B Golder Associates Ground Movement Assessment EES Summary Report.

Various geotechnical, hydrogeological and contamination investigations have been undertaken through each previous stage of the project. Interpretation of the investigation results are presented in the Appendix A and B of this report.

The method adopted in respect of estimation of ground movement due to proposed Melbourne Metro works and potential impacts relating to the estimated ground movement undertaken by Golder Associates and AJM JV in the above listed reports is summarised in Table 5-2.

Table 5-2 Summary of Ground Movement Assessment Components

Task	Relevant Report	Detail
Development of Preliminary Assessment Inputs		
Derivation of the interpreted geological and geotechnical model including recommendations for the parameters to be used in the analyses	Appendix A *Golder Associates* Interpreted Geological Setting *EES Summary Report*	Conceptual ground model Preliminary parameters for numerical modelling
Estimation of the parameters to relate the surface settlement to the tunnelling techniques and ground conditions	Appendix B to this report, *Golder Associates Ground Movement Assessment EES Summary Report*	Numerical modelling of alignment cross sections to estimate the surface settlement profile for face loss values of 0.5 percent, 1 percent, and 1.5 percent
	Technical Appendix P *Ground Movement and Land Stability* (this report)	Using the settlement profiles from the numerical modelling, back-calculation of the corresponding tunnelling induced settlement parameters Selection of the appropriate face loss values to reflect the proposed tunnelling techniques and ground conditions (AJM JV with Golder Associates)
Estimation of the surface settlement profiles resulting from the excavation for open cut excavations (station boxes and cut and cover tunnels)		Numerical modelling was undertaken using preliminary geotechnical parameters for numerical modelling as recommended by Golder Associates
Estimation of the surface settlement profile resulting from cavern station excavations	Appendix B to this report, *Golder Associates Ground Movement Assessment EES Summary Report*	Numerical modelling was undertaken by Golder Associates using preliminary geotechnical parameters for numerical modelling as recommended by Golder Associates
Estimation of primary consolidation settlement induced by drawdown of ground water, resulting from Melbourne Metro works	*Golder Associates Regional Groundwater Numerical Modelling Report EES Summary Report*	Hydrogeological modelling to predict ground water drawdown
	Appendix B to this report, *Golder Associates Ground Movement Assessment EES Summary Report*	Estimation of the areas potentially experiencing consolidation settlement and the magnitudes of the settlements
Determination of the Potential Zone of Influence[1]		
Combination and plotting of the surface settlements resulting from underground and open cut excavations	Technical Appendix P *Ground Movement and Land Stability* (this report)	Calculation of surface settlement from underground excavations including tunnels and combination with the settlements from cut and cover structures conducted by AJM JV using software XDisp
	Appendix B to this report, Golder	Plotting of the excavation induced settlement Xdisp outputs as contours

[1] See Section 5.3 Levels of Impact Assessment for additional details.

Task	Relevant Report	Detail
	Associates Ground Movement Assessment EES Summary Report	Plotting of estimated primary consolidation settlement contours and summarised with respect to ground movement
Review of settlement contours and determination of the extent of influence of the excavation induced surface settlements	Technical Appendix P *Ground Movement and Land Stability* (this report)	Conducted by AJM JV in consultation with Golder Associates
Impact Assessment[1]		
First level of interpretation of settlement effects on buildings and other infrastructure including utilities	Technical Appendix P *Ground Movement and Land Stability* (this report)	Interpretation of potential settlement effects on buildings and other infrastructure
		Review of the infrastructure and buildings for their position and type in relation to the plotted ground surface contours and selection of assets for further assessment
		Calculation of strains and distortion of the buildings and infrastructure using the software XDisp, with output in terms of potential consequences
		Interpretation of the combined effects of excavation and consolidation induced settlement on buildings and other infrastructure
Site Specific Assessment[2]		
Interpretation of settlement effects on buildings and other infrastructure selected for site specific assessment	Technical Appendix P *Ground Movement and Land Stability* (this report)	Review of the infrastructure and buildings that require more specific numerical modelling because of their height or complexity
		Detailed two dimensional numerical modelling of the excavation induced settlements at selected structures
		Structural engineer review of the assessment outputs to interpret potential consequences to the respective assets
	Golder Associates Ground Movement Assessment EES Summary Report	Two dimensional and three dimensional numerical modelling of the excavation induced settlements at selected structures
Other Considerations		
Interpretation of the effects of Melbourne Metro on the historically recorded and continuing creep settlement	Appendix B to this report, *Golder Associates Ground Movement Assessment EES Summary Report*	Review of the history and rate of the creep settlement, with interpretation the influence of Melbourne Metro on it
	Technical Appendix P *Ground Movement and Land Stability* (this report)	Review of the addition of these effects to the settlements induced by Melbourne Metro

See Section 5.3 Levels of Impact Assessment for additional details.

5.2 Estimation of Potential Ground Movement

A Ground Movement Assessment was undertaken by Golder Associates and a summary report describing this work is provided in Appendix B of this report. The objective of the report was to inform preliminary assessment of the potential ground movement effects on buildings, structures and services along the Melbourne Metro alignment.

The excavation induced settlement (due to underground and open cut works) at ground surface level is the result at the surface of the settlement caused by the ground movement mechanisms summarised in Section 4.4 that has propagated from the excavation works. Excavation induced settlement contours are provided in Appendix C of this Report. The extents and estimated potential magnitudes of primary consolidation settlement contours are provided in Appendix D of this report.

5.2.1 Embankment Tie-In

Previous project experience formed the basis for the preliminary estimation of embankment deformation. Observations made at the Regional Rail Link construction stage for a comparable embankment height to that proposed at the western portal tie-in and in similar geological conditions observed settlement of up to 30 mm adjacent to excavations for Regional Rail Link works. A linear deformation profile was assumed in the settlement contour drawings.

The proposed Melbourne Metro works would incorporate embankment widening using precast retaining system and foundation preparation which may include ground improvement to limit consolidation settlement of the underlying Coode Island Silt and deep alluvial soils. Additional consolidation settlement of the Coode Island Silt may result from potential groundwater drawdown during and after construction stage.

Further work would be required at detailed design stage to refine the estimated settlement that may impact the operating rail on the existing embankment.

5.2.2 Underground Excavation Induced Movement

Estimates of surface displacements resulting from underground excavations were obtained from modelling and assessment of the sub-surface geology and geotechnical properties as described in Appendix A of this report.

The adopted volume loss parameters were supported by the results of numerical analyses as documented in Appendix B of this report. These values were an input into the assessment of potential ground movement due to tunnelling in Xdisp, the program used to generate the estimated settlement trough(s) overlying the proposed twin Melbourne Metro tunnels. The trough width parameter, K is an empirical constant based on ground properties and is typically in the range of 0.25 and 0.7. A settlement trough with a large K value would have a wider trough than that for a tunnel in similar ground conditions with equivalent tunnel geometry and depth below surface but a lower K value. Lower K value trough would be narrower and have steeper gradients in the ground surface deformation profile.

Table 5-3 summarises the adopted tunnelling induced ground movement parameters for various ground conditions.

Table 5-3 Adopted Tunnelling Induced Settlement Parameters

Type	Volume Loss (%)	Trough Width Parameter
Soil (all types)	1	0.4
Yarra River (South Bank)	1 and 1.5*	0.3
Rock	0.5	0.6
Rock (greater than two tunnel diameters of cover)	0.5	0.7

Sensitivity assessment of varying volume loss values has been completed and are reported in Appendix B of this report to examine the potential effects of a higher volume loss parameter on the estimated deformation results.

In soft ground conditions the analysis assumes appropriate ground improvement is undertaken in conjunction with an appropriate ground movement monitoring program.

PLAXIS modelling and ground deformation assessments for the cavern stations are described in Appendix B of this report. Sensitivity assessment of in-situ (Ko) values was undertaken based on the pressuremeter and in situ stress test results completed to date. Some discussion on the preliminary geotechnical parameters is found in Appendix A of this report.

5.2.3 Open Cut Excavations

Numerical software package PLAXIS was used for the assessment of retaining wall deflections and at some locations, included seepage analyses associated with excavations and support systems. Analyses documented in the Technical Appendix O *Groundwater* took into consideration the effects of dewatering. Mitigating measures and recharging measures are described in this report. Correlation of analytical studies to local case studies of excavations similar to those proposed in this project are not readily available. In the station box and shaft design, ground movements from temporary works and permanent works are both considered and reflected in the retention system ground movement estimations represented in the settlement contour drawings.

Estimates of vertical ground movement corresponding to modelled retaining wall lateral movements are incorporated into the settlement contour drawings.

The existing cutting at the eastern portal would be widened to accommodate the realignment of existing tracks. Slope stability assessments were undertaken in SLOPE/W, a slope stability software program for computing the factor of safety of earth and rock slopes. SLOPE/W can effectively analyse both simple and complex problems for a variety of slip surface shapes, pore-water pressure conditions, soil properties, analysis methods and loading conditions. The SLOPE/W results demonstrated that soil nail support with shotcrete facing could be a feasible batter stabilisation measure in the short term as well as the long term for various slope geometries and potential construction schemes. Estimates of Soil Nail Wall Deformation were based on Soil nailing recommendations--1991 for designing, calculating, constructing and inspecting earth support systems using soil nailing, Clouterre, as referenced in CIRIA Report C637 Soil Nailing – Best Practice Guidance.

5.2.4 Excavation Induced Settlement Contours

XDisp is a program that predicts ground movements due to all kinds of excavations, including tunnels, basements, mines and embedded walls. It then uses these predicted ground movements to assess building damage, eliminating the need for separate analysis programs or multiple spreadsheets. 3-D graphics make it easy to check and interpret the data, as well as helping to make things clear for designers, contractors and clients. The program is used regularly for ground movement assessments on tunnelling projects worldwide.

The program was developed based on experience of complex urban tunnelling projects in London, New York and Hong Kong, Xdisp has been used on Crossrail and King’s Cross underground station in London as well as numerous underground railway and building projects around the world.

The proposed alignment with proposed excavation depths, dimensions and separation, the project boundary topography, a simplified geological profile and tunnelling induced ground movement parameters are input into the Xdisp to generate the tunnelling induced settlement contours. The estimated ground movement data surrounding open cut structures from the various shaft, portal and station box preliminary design models and the preliminary cavern modelling are also imported into the program. The program combines the ground movement effects at different structure interfaces.

5.2.5 Consolidation Settlement

The conceptual hydrogeological model prepared by Golder Associates forms the basis of the completed preliminary hydrogeological modelling and subsequent consolidation settlement assessment which is described in Appendix B of this report. It is assumed that consolidation settlements are generally relatively uniform when the depth and compressibility of the soft soils are uniform. Where uniform, consolidation settlements do not cause tensile strain and typically do not cause damage.

5.2.6 Cumulative Effects

Effects due to movement induced by underground excavation and lateral deflection of retaining walls are summed within the program Xdisp where these structures interact, such as where a station entrance shaft is adjacent to a station cavern. Effects due to consolidation settlement are summed with underground and open cut excavation movement on a site by site basis.

Settlements for each of the mechanisms described in Section 4.4 above were calculated separately. However, where the influences of these mechanisms overlap, the results are typically combined when assessing their effects.

Settlements due to consolidation are assumed to be relatively uniform, and do not induce bending or horizontal strain except where there might be significant differences in soft soil thicknesses across a short horizontal distance or at the edge of a zone of soft soil.

5.2.7 Unexpected Ground Movement

The estimation of the magnitudes of ground movement and the assessment of their effect on buildings and infrastructure relies on the preliminary conceptual geological model. The selection of the appropriate construction methods and equipment and experience of the operators is also a factor that influences the magnitudes of ground movements that may occur. The assessments and the risk ratings developed are based upon the existing controls which are described in Section 6.4, with appropriately conservative use of the parameters derived from the geotechnical information and the proposed construction methods.

The risk of ground movements being greater than estimated arises from a number of hazards principally related to the ground but also the construction methods and practices. Some of the main examples of factors leading to greater ground movements than predicted are listed below.

- Greater face loss at the excavation because of encountering weaker or less stiff material leading to greater than predicted settlement, and thus leading to greater adverse effects on buildings or other infrastructure
- A different response of the ground to the face loss leading to more localised settlement than predicted, leading thus to greater strains in effected structures, and thus more adverse effects
- Unexpected ground creating conditions more difficult to control or less suited to the adopted construction methods, leading to greater than predicted settlement, and thus leading to greater adverse effects on buildings or other infrastructure
- Unexpected permeability of the ground, either locally at a features such as a zone of fractured rock or more generally through the ground in general, leads to faster or greater quantities of ground water flows into the excavations, causing more drawdown of the ground water than predicted, leading to greater than predicted consolidation settlement, and thus leading to greater adverse effects on buildings or other infrastructure
- Unexpected actions taken during construction, increasing face loss at the excavation leading to greater than predicted settlement, and thus leading to greater adverse effects on buildings or other infrastructure.

The reliability of the ground model has been considered by Golder Associates using a combination of the complexity of the ground and the quality and spacing of the available investigation boreholes, and is summarised in Appendix A to this report. The reliability rating has been derived from considering the quality and spacing of the ground investigations sites with the expected complexity of the ground conditions. The rating assists in determining whether there are likely to be conditions in the ground encountered unpredictably as the tunnelling advances.

The risk presented by these uncertainties are somewhat mitigated by the information available from the currently available geotechnical data and the knowledge on these areas of Melbourne that has built up from previous projects. Therefore, it is unlikely that the tunnelling would encounter a ground condition that has not been found within the geological strata before. However, what is difficult to predict is the position of such conditions ahead or around the face being excavated, and the relationship between the different ground strata. This uncertainty would be reduced by the final suite of geotechnical investigations that would be completed during the procurement phase and any conducted by the project proponents. However, there would always some degree of uncertainty remaining, however comprehensive the investigation.

The current control measures that have been assumed include the use of construction methods that are appropriate for the range of conditions expected within the ground. These would be augmented by the use of probing ahead, and if necessary, around the excavation faces to provide information on the conditions about to be encountered. Additional risk mitigation measures include monitoring the behaviour of the tunnel, and the movement of the ground at depth and at the surface, and the effects on buildings and infrastructure. These final effects, indicate the effectiveness of the design and construction methods, for the next sections of drive in similar materials.

Both the probing and monitoring would be linked to a construction risk management process that would initiate different responses depending upon how different from the predictions the measurements or detected conditions are.

The initial response could range from an increase in the frequency of monitoring for minor variations from predicted trends, to stopping work and stabilising the ground if measured ground movements are approaching values that might lead to unacceptable damage.

In the event that interim ground movements appear to be greater than predicted for the current stage of construction, the conditions would be assessed, and additional control measures applied. These could include adjustments in the construction methods, such as increasing or reducing face pressure in a TBM, applying ground improvement, protecting a structure, or in an extreme case, changing the construction methodology.

As the possible scenarios that could result from the failure of the current project controls are numerous, with multiple combinations of causes, the consequences have not been assessed for individual structures. However, indicative assessments show that the consequences remain a function of both the depth of the tunnel and the ground conditions. For example, the effects of a ground loss equivalent to the volume of the head of a TBM has little effect on a building if the tunnel is at 25 m depth in rock, possibly increasing from negligible to very slight, at worst. However, the same loss in soils with a cover of 10 m could lift the damage category to severe.

This shows the importance of the control measures, including construction methodology, monitoring programs and construction management that have been assumed in the existing controls for the risk assessment and would be expected of a competent and experienced contractor.

5.3 Levels of Impact Assessment

Potential impacts resulting from the estimated ground movement, to the following broad structure types and assets were considered:

- Buildings
- Civil infrastructure, including road and rail assets
- Utilities
- Parklands.

Individual impacts of settlement on all structures, utilities and infrastructure have not been assessed. Settlement assessments of all structures, utilities and infrastructure would be undertaken during detailed design prior to construction and the results reported separately. Potential impacts of detailed design construction schemes and construction methodology, using refined structural and geotechnical models would be assessed to confirm consistency of assessment outputs with preliminary assessments conducted to date.

While excavation induced ground settlements are generally expected to be small and unlikely to give rise to material impacts to nearby surface and underground structures along the route, appropriate design requirements and management measures are required to avoid unacceptable impacts.

5.3.1 Level 1 Assessment

A Level 1 Assessment is the process of identifying structures and civil infrastructure that are within the Potential Zone of Influence relating to Ground Movement. The extents of the Potential Zone of Influence is derived from the Xdisp excavation induced settlement contour predictions and the primary consolidation settlement assessment. Further detail on the estimated Potential Zone of Influence relating to potential ground movement is described in Section 6.3 of this report.

The potential ground surface deformation profile is provided as settlement contours in the attached Appendix C and Appendix D of this report.

The appropriate level of further assessment required depends on the magnitude of settlement beneath a structure, the predicted ground slope and the structural vulnerability.

Experience from previous tunnelling projects has shown that the effects on buildings subjected to ground movements less than 10 mm and with maximum slope of 1 in 500 are negligible (Rankin 1988).

5.3.2 Level 2 Assessment

In a Level 2 Assessment, it is assumed that building and structure foundations behave flexibly and follow the estimated ground settlement profile. In reality, the inherent stiffness of the structures and structure foundations would tend to reduce deflections and strains induced by ground movement. For this assessment, a representative sample of different building types, utilities and key civil infrastructure were assessed that are founded in varying geological settings, have varying construction types and overlie or are situated close to project works with varying tunnel or underground structure arrangements.

The influences of existing foundations, existing structures or existing underground openings are not included in a Level 2 assessment.

The building assessment results were assessed against industry accepted potential building damage classifications (Mair, Taylor and Burland, 1996) which correlate maximum tensile strain against typical degree of damage for buildings. This system is generally only applicable for buildings with relatively shallow foundations and is not strictly appropriate for assessment of structures with deep foundations nor tall buildings which would typically be assessed in a Level 3 assessment.

Assessment results for utilities and infrastructure were compared against preliminary impacts evaluation criteria as documented in Section 5.4.4.

Buildings, structures or utilities where potential impacts were considered acceptable, being negligible or minor, were not subject to further assessment.

5.3.3 Level 3 Assessment

Level 3 Assessment is conducted for:

- Structures or utilities with potential for unacceptable damage (moderate or worse impacts)
- Structures on shallow foundations and within a distance from an open excavation equal to the excavated depth of soils or extremely weathered rock or 50% of the total excavation depth

- Structures with piled foundations
- Tall buildings
- Structures where protective measures might be required.

In a Level 3 assessment, each building is modelled to incorporate stiffness of the building, influence of existing structures and traffic or other conditions as required. The strain developing within the structure and the applicability of the standard evaluation criteria is reappraised using refined models. A structural appraisal by a qualified structural engineer is carried out for the purpose of confirming the likely structural behaviour and determining whether a structural survey is necessary.

The Level 3 assessment considers:

- The sensitivity of the asset to ground movements and its ability to tolerate movement without significant distress
- The potential for interaction with adjacent buildings / structures
- The sensitivity to movement of particular features within structures and how they might respond to ground movements
- The current condition of the structure or utility which might increase the vulnerability of the asset. Therefore, a structural survey is required to determine the structural condition of the buildings or assets and, also, to confirm as built details which are typically an analysis input.

Selected preliminary Level 3 assessments were undertaken by both AJM JV and Golder Associates. Appendix B Potential Zone of Influence includes detail of preliminary Level 3 Assessments undertaken by Golder Associates.

Preliminary assessment of potential impacts to existing assets is based on this preliminary work. It is noted that as built information was largely incomplete for many completed assessments but the assessment results, although preliminary in nature are considered representative of the likely effects of Melbourne Metro excavations and are adequate to inform preliminary mitigations planning and the Environmental Performance Requirements.

On completion of the Level 1 Assessments and Level 2 Assessments on the selected structures and utilities, the site specific assessments (Level 3) include the following steps:

- Review as-built information for the existing structure (which was collated for the Level 2 Assessment)
- Collate or determine design inputs and engineering parameters such as initial stress conditions, soil strength or structure loads, that allow the ground-building interactions to be modelled
- Establish anticipated excavation and construction sequence based on the proposed structural scheme
- Undertake analyses based on numerical modelling which is used to represent a complex ground-structure interaction scenario
- Undertake independent calculations using empirical classical equations as a validation check, where appropriate
- Document potential impacts, required mitigations and/or further required work.

5.3.4 As-built Information

Only very general information is available on many buildings, utilities and infrastructure. Therefore, in these cases, the current assessments are based upon visual assessment of the building infrastructure and engineering judgment and experience used to assign likely foundation types. In the absence of structure or building specific preliminary condition assessments, the impact assessments to date have assumed that the current structural condition and serviceability of buildings and structures are sound. A comprehensive pre-construction stage condition assessment program would inform detailed design assessments and particular structure vulnerabilities might be realised that would be incorporated into the detailed design stage assessments.

5.4 Risk and Impact Assessment

5.4.1 Overview

An Environmental Risk Assessment has been completed for impacts of Melbourne Metro in relation to ground movement. The risk based approach is integral to the EES. Importantly, an environmental risk is different from an environmental impact. Risk is a function of the likelihood of an adverse event occurring and the consequence of the event. Impact relates to the outcome of an action in relation to values of a resource or sensitivity of a receptor. Benefits are considered in impact assessment but not in risk assessment. The impact assessment must be informed by the risk assessment so that the level of action to manage an impact relates to the likelihood of an adverse impact occurring and its severity.

The overall risk assessment process adopted was based on AS/NZS ISO 31000:2009, as illustrated in Figure 5-1.

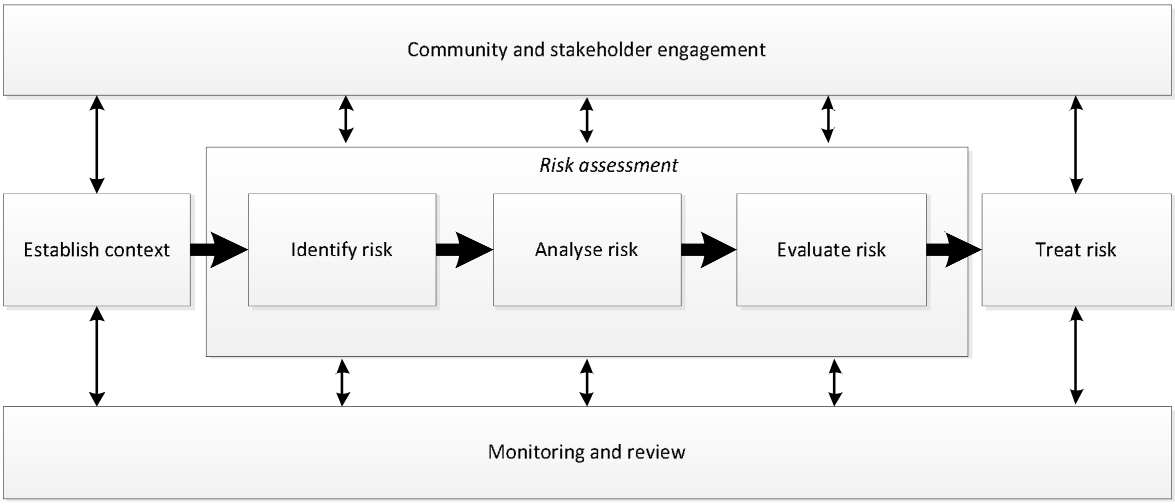

Figure 5-1 Overview of AS/NZS ISO 31000-2009 Risk Process

The following tasks were undertaken to determine the impact pathways and assess the risks:

- Setting of the context for the environmental risk assessment
- Development of consequence and likelihood frameworks and the risk assessment matrix
- Review of project description and identification of impact assessment pathways by specialists in each relevant discipline area
- Allocation of consequence and likelihood categories and determination of preliminary initial risks
- Workshops with specialist team members from different yet related discipline areas and focussing on very high, high and moderate initial risks to ensure a consistent approach to risk assessment and to identify possible interactions between discipline areas
- Follow-up liaison with specialist team members and consolidation of the risk register.

A more detailed description of each step in the risk assessment process is provided in the Technical Appendix B *Environmental Risk Assessment Report.*

5.4.2 Context

The overall context for the risk assessment and a specific context for each specialist study is described in the Technical Appendix B *Environmental Risk Assessment Report*. The context describes the setting for evaluation of risks arising from the Melbourne Metro.

The likelihood rating criteria used in the risk assessment by all specialists is shown in the table below.

Table 5-4 Melbourne Metro Likelihood rating criteria

Level	Description
Rare	The event is very unlikely to occur but may occur in exceptional circumstances.
Unlikely	The event may occur under unusual circumstances but is not expected.
Possible	The event may occur once within a 5 year timeframe.
Likely	The event is likely to occur several times within a 5 year timeframe.
Almost Certain	The event is almost certain to occur one or more times a year.

The consequence criteria framework used in the risk assessment follows.

Table 5-5 Melbourne Metro Ground Movement and Land Stability Consequence Framework

Level	Qualitative description of biophysical/ environmental consequence	Qualitative description of socio-economic consequence
Negligible	No detectable change in a local environmental setting.	No detectable impact on economic, cultural, recreational, aesthetic or social values.
Minor	Short term, reversible changes, within natural variability range, in a local environmental setting.	Short term, localised impact on economic, cultural, recreational, aesthetic or social values.
Moderate	Long term but limited changes to local environmental setting that are able to be managed.	Significant and/or long term change in quality of economic, cultural, recreational, aesthetic or social values in local setting. Limited impacts at regional level.
Major	Long term, significant changes resulting in risks to human health and/or the environment beyond the local environmental setting.	Significant, long term change in quality of economic, cultural, recreational, aesthetic or social values at local, regional and State levels. Limited impacts at national level.
Severe	Irreversible, significant changes resulting in widespread risks to human health and/or the environment at a regional scale or broader.	Significant, permanent impact on regional economy and/or irreversible changes to cultural, recreational, aesthetic or social values at regional, State and national levels.

A potential risk of the Melbourne Metro is that ground movement may occur resulting from the proposed excavations or as secondary effect of groundwater drawdown caused by the proposed works. To assess this risk, assessments were undertaken to estimate the magnitudes and extent of potential ground movements caused by Melbourne Metro works. Any ground movements that occur could subsequently pose a risk to existing buildings, infrastructure and/or utilities that are located within the potential zone of influence of the project works. To assess the risk of adverse impacts to existing assets, impact assessments were conducted on a representative sample of structures and utilities.

To evaluate the risk of adverse impacts to existing assets due to ground movement, the completed preliminary impact assessment results were used to determine consequence in the initial risk rating. Where the initial risk rating was found to be potentially moderate or worse, potential risk mitigation measures were developed that informed selection of the residual risk rating.

5.4.3 Risk Assessment Matrix

The environmental risk assessment matrix used by all specialists to determine levels of risk from the likelihood and consequence ratings is shown.

Table 5-6 Risk Assessment Matrix

		Consequence ratings				
		Negligible	**Minor**	**Moderate**	**Major**	**Severe**
Likelihood rating	**Rare**	Very Low	Very Low	Low	Medium	Medium
	Unlikely	Very Low	Low	Low	Medium	High
	Possible	Low	Low	Medium	High	High
	Likely	Low	Medium	Medium	High	Very High
	Almost Certain	Low	Medium	High	Very High	Very High

5.4.4 Ground Movement Impacts Evaluation Criteria

Evaluation of the effects of ground movement is based on established potential damage classifications for buildings and utilities. The adoption of these impacts evaluation criteria is considered adequate for preliminary assessment but discussion would be required with the respective asset owners and operators to determine appropriate Melbourne Metro specific acceptability criteria for the various types of existing assets.

The three categories of potential damage to a structure or underground asset are broadly:

- Aesthetic: affecting the appearance of an asset only
- Serviceability: cracking and distortion which might impair the weather tightness of an asset (durability) or other functions such as ease of operation and potentially resulting in increased maintenance without adoption of suitable preventative measures or timely repairs
- Stability: There is an unacceptable risk of instability or loss of function without adoption of preventative measures.

A potential damage category of minor is considered acceptable notwithstanding that cosmetic damage may need to be addressed post-construction. A damage category greater than minor is considered significant, probably requiring additional measures to mitigate the potential magnitudes of ground movement or protective measures to achieve acceptable outcomes.

The impact evaluation criteria used in the Ground Movement risk assessment study are shown in the following sections, and are directly related to the consequences that informed the initial risk ratings. These criteria are consistent with industry practice for projects of this type and are based on criteria adopted in previous tunnelling projects.

5.4.4.1 Buildings

The manner in which a building or other structure responds to differences in ground movements depends upon its size, design (foundation and building superstructure) and materials. A modern steel or reinforced concrete structure can be flexible, deflecting as the ground moves. In contrast, a masonry building, subject to similar displacements, could behave as a relatively brittle structure and respond by cracking. The interaction between a structure and ground movement is also influenced by the foundation type. Deep foundations might support a structure from outside the zone of movement, isolating the structure from the adjacent surface level changes.

Unless identified as particularly sensitive to potential movements, buildings that are found to be subject to less than 10 mm potential settlement and less than 1 in 500 slope are not subject to further assessment as the risk of damage is considered negligible and superficial damage is unlikely (Rankin 1988).

The methodology for the Level 2 building damage assessments is based on limiting tensile strains using the approaches of Mair et al. (1996). Burland & Wroth (1974) showed that the onset of visible cracking is associated with a well-defined value of average tensile strain and this value was found not to be sensitive to the mode of deformation.

The classification of building strains in accordance with *Relationship between Category of Damage and Limiting Tensile Strain* (After Burland (1995), and Mair et al (1996)) and *Classification of Visible Damage to Walls with Particular Reference to Ease of Repair of Plaster and Brickwork* (Mair, Taylor and Burland, 1996) is provided in Table 5-7.

Table 5-7 Impact ratings for buildings (after Burland, 1995 and Boscardin & Cording 1989)

Potential Impact	Category of damage and Normal degree of severity**	Description of typical damage*	Limiting tensile strain** %	Broad category grouping
Negligible	0 – Negligible	Hairline cracks less than about 0.1 mm wide.	Less than 0.05	Aesthetic Damage
Minor	1 – Very Slight	Fine cracks that are easily treated during normal decoration. Damage generally restricted to internal wall finishes. Close inspection may reveal some cracks in external brickwork or masonry. Typical crack widths up to 1 mm.	0.05 to 0.075	
	2 – Slight	Cracks easily filled. Redecoration probably required. Recurrent cracks can be masked by suitable linings. Cracks may be visible externally and some repointing may be required to ensure weather-tightness. Doors and windows may stick slightly. Typical crack widths up to 5 mm.	0.075 to 0.15	
Moderate	3 – Moderate	The cracks require some opening up and can be patched by a mason. Repointing of external brickwork and possibly a small amount of brickwork to be replaced. Doors and windows sticking. Service pipes may fracture. Weather-tightness often impaired. Typical crack widths are 5–15 mm or several >3 mm.	0.15 to 0.3	Serviceability Damage
Major	4 – Severe	Extensive repair work involving breaking out and replacing sections of walls, especially over doors and windows. Windows and door frames distorted, floor sloping noticeably. Walls leaning or bulging noticeably, some loss of bearing beams. Service pipes disrupted. Typical crack widths are 15–25 mm, but it also depends on the number of cracks.	Greater than 0.3	

Potential Impact	Category of damage and Normal degree of severity**	Description of typical damage*	Limiting tensile strain** %	Broad category grouping
Severe	5 – Very Severe	Irreversible, significant changes resulting in widespread risks to human health and/or the functioning of the building. This requires a major repair job involving partial or complete rebuilding. Beams lose bearing; walls lean badly and require shoring. Windows broken with distortion. Danger of instability. Typical crack widths are greater than 25 mm, but it also depends on the number of cracks.	Greater than 0.3	Stability Damage

* Note: Crack width is only one factor in assessing category of building damage and is not used as a direct measure of damage. Ease of repair is the key factor in development of this table, based on a large number of other studies

**Relationship between Category of Damage and Limiting Tensile Strain for Buildings (After Burland (1995), and Mair et al (1996))

The categorisation above is strictly applicable to buildings founded on shallow foundations, for which tensile strains induced in a building and associated ground slopes are derived and compared against limiting values to assess the risk category and degree of damage. Detailed design would require assessment of impacts of potential ground settlement on any slab on grade structures, in addition to a review of the capacity of the existing piles for the increased pile loads associated with ground settlement.

On the basis of the above damage classifications, negligible or minor categories would be typically considered to be acceptable levels. It should be noted that the prediction of small settlements, such as negligible or very slight, does not guarantee that no damage to buildings would occur. While damage should be unlikely, it is still possible, and would be expected to occur in the form of readily repairable damage to finishes.

5.4.4.1.1 Deep Foundations

It is not always appropriate to adopt the Mair, Taylor and Burland (1996) criteria for potential building impacts for a building founded on piles as the building settlement profile may not feasibly mirror the predicted ground settlement trough. Ground movements may result in both lateral deformation and settlement of piles within the Potential Zone of Influence. The ground movement can induce both bending moments and axial down drag forces on the pile. The magnitude of these additional loads on the piles is largely dependent on a number of factors including the amount of horizontal and vertical ground movement, the distance of the piles from the proposed Melbourne Metro works and the relative position of the pile tip with respect to the depth of the excavation and/or tunnel axis.

5.4.4.2 Infrastructure

The following sections describe indicative Ground Movement impacts evaluation criteria for a number of civil infrastructure and utility asset types. It is noted that individual impacts evaluation criteria for significant assets should be developed in consultation with infrastructure and utility stakeholders on a case by case basis.

Limiting values of structural deformation and foundation movement depends on structure type, condition and ground conditions.

Further discussion is required to confirm appropriate acceptability criteria with the relevant stakeholders.

5.4.4.2.1 Tram Lines and Road pavements

Table 5-8 Impact ratings for road pavements*, tram ways, kerbs and footpaths (Hudson-Smith & Grinceri)

Potential Impact	Max Slope And Settlement Induced	Maximum Induced Slip (Mm) Or Strain (Mm/M)	Description Of Potential Damage
Negligible	< 1/750 Settlement 10 mm	5 mm/m	Negligible effects, superficial damage unlikely
Minor	1/500 to 1/750 Settlement 10 mm	5 mm/m	Negligible effects, superficial damage unlikely
Moderate	1/500 to 1/150 Settlement 15 mm	10 mm/m	Possible superficial damage, which is unlikely to have significant effect to the structure
Major	1/150 to 1/50 Settlement 25 mm	20 mm/m	Expected superficial damage to structures, possible structural damage to structures
Severe	>1/50 Settlement 50 mm	30 mm/m	Expected structural damage to structure

***For roads identified as potentially exceeding the serviceability limiting criteria or locations considered sensitive to ground movements, a risk assessment would be developed that takes into account features such as the road surfacing material, the existing road condition and traffic levels.**

5.4.4.2.2 Rail Lines

There are numerous locations along Melbourne Metro alignment where ground movement induced by project works would affect operating rail resulting in settlement under or adjacent to operating lines.

The evaluation criteria for the rail lines would depend on the track classification which relate to varying rail speeds and the current condition of the tracks. The adopted evaluation criteria may be related to the existing maintenance intervention levels for the potentially impacted tracks.

Track monitoring and mitigation of track movement would be a major component of the ground movement and infrastructure monitoring program. Track bed movements are typically measured over short distance of 2 m (short twist) and over 8 m (long twist) along critical zones on the existing railway lines. Monitoring points on the track may be positioned along the track at 2 m intervals on both rails of a single track.

Utilities The assessment of potential damage to services and utilities was based on calculated ground displacements and, for rigid utilities such as pipes, on the sectional forces derived from the respective Level 2 Assessment profile. The methods of Attewell et al. (1986) and Bracegirdle et al. (1996) were used. For damage categories greater than minor, services protection measures (or services relocation) might be required to reduce construction impacts to minor or lesser damage category.

Table 5-9 Impact ratings for Utilities (Attewell et al. (1986) and Bracegirdle et al. (1996))

Potential Impact	Max slope induced	Maximum induced slip (mm) or strain (mm/m)	Description of potential damage
Negligible	< 1/750	Concrete pipe/culvert: 10 mm Water Steel & Iron: 10 mm Cable in PCV duct: 2 mm/m Cable buried in the ground: 1 mm/m Gas Pipes PVC: 5 mm Gas Steel & Iron: 5 mm	Negligible effects, superficial damage unlikely

Potential Impact	Max slope induced	Maximum induced slip (mm) or strain (mm/m)	Description of potential damage
Minor	1/500 to 1/750	Concrete pipe/culvert: 10 mm Water Steel & Iron: 10 mm Cable in PVC duct: 2 mm/m Cable buried in the ground: 1 mm/m Gas Pipes PVC: 5 mm Gas Steel & Iron: 5 mm	Negligible effects, superficial damage unlikely
Moderate	1/500 to 1/150	Concrete pipe/culvert: 15 mm Cable in PVC duct 4 mm/m Cable buried in the ground: 2 mm/m Gas Pipes PVC: 10 mm	Possible superficial damage, which is unlikely to have significant effect to the structure or function of the utility
	1/500 to 1/250	Water Steel & Iron: 15 mm Gas Steel & Iron: 10 mm	
Major	1/150 to 1/50	Concrete pipe/culvert: 25 mm Cable in PVC duct 6 mm/m Cable buried in the ground: 3 mm/m Gas Pipes PVC: 20 mm	Expected superficial damage to structures, possible damage to structures, possible damage to rigid utilities
	1/250 to 1/130	Water Steel & Iron: 25 mm Gas Steel & Iron: 15 mm	
Severe	>1/50	Concrete pipe/culvert: 30 mm Cable in PVC duct 8 mm/m Cable buried in the ground: 4 mm/m Gas Pipes PVC: 25 mm	Expected structural damage to structure and function of utility
	>1/130	Water Steel & Iron: 30 mm Gas Steel & Iron: 20 mm	

*Consequence ratings depend on utility construction type, construction method, function, and condition and asset owner requirements, further discussion required to confirm appropriate criteria with relevant stakeholder(s)

5.5 Stakeholder Engagement

As part of this assessment, the following specific engagement with stakeholders was undertaken.

Table 5-10 Summary of stakeholder engagement

Activity	When	Matters discussed/ issues raised	Consultation outcomes/Future Work
PTV **MTM** **VicTrack**	October 2015	Presentation by AJM JV to describe the project, ground movement impacts assessment methodology and discuss process of developing acceptability criteria.	Further consultation required to present preliminary assessment results, develop appropriate acceptability criteria that incorporate operating rail considerations and existing maintenance intervention levels.
Telstra	December 2015	Walk through site inspection of tunnels parallel to and perpendicular to Swanston Street between the proposed City station precincts by AJMJV and MMRA.	Further consultation required to present preliminary assessment results, develop appropriate acceptability criteria that incorporate maintenance of critical infrastructure considerations and discuss suitable mitigation or protective works that might be adopted, if required.

Activity	When	Matters discussed/ issues raised	Consultation outcomes/Future Work
Yarra Trams	4 December 2015	Surface movements that would be tolerated by tram operations and tram infrastructure.	Yarra Trams to consult internally. Preliminary potential impact assessment results to be presented.
VicRoads	-	-	Consultation required to present preliminary assessment results, develop appropriate acceptability criteria that incorporate operating road considerations and existing maintenance intervention levels.
Transurban	June 2015	Meeting with AJM JV and MMRA to introduce the proposed project scheme, Melbourne Metro structure proximity to TransUrban infrastructure, the CityLink viaduct and CityLink tunnels.	Further consultation required to present preliminary assessment results, develop appropriate stakeholder acceptability criteria and appropriate mitigation and monitoring works that may be adopted, where required.
City of Stonnington	2015	As-Built information.	Further consultation required to present preliminary assessment results in relation to Council owned/managed assets, potential impacts to residents and business owners, develop appropriate acceptability criteria and discuss community consultation strategy in relation to ground movement impacts.
City of Melbourne	2015		
City of Port Phillip	2015		

In addition to the specific agency and TRG engagement and the engagement listed in the table above, general engagement and consultation with the community was also conducted as part of this assessment. Written feedback was obtained through feedback forms and the online engagement platform, and face-to-face consultation occurred at the drop-in sessions (refer to Technical Appendix C *Community and Stakeholder Feedback Summary Report* for further information) ,Although the community was given the opportunity to offer feedback in regards to ground movement, no comments were provided or concerns identified.

5.6 Assumptions

The following table outlines the basis and assumptions of the methodology.

Table 5-11 Basis and Assumptions of Methodology

Element/Interface	Basis and Assumptions of methodology
Project description	Infrastructure design and design requirements have been taken from the Concept Design. Any changes made after this may alter the impacts discussed in this report.
Melbourne Metro alignment	This impact assessment is based on Concept Design. Any subsequent alignment changes have not been incorporated in this impact assessment.
Geology	The geological profile drawings are based on data collated to 30 September 2015. Any changes after this date not been incorporated in this impact assessment.
Hydrogeological Modelling	The completed consolidation settlement assessment relies on the hydrogeological modelling completed to date and described in the Golder Associates Interpreted Hydrogeological Setting EES Summary Report.

Element/Interface	Basis and Assumptions of methodology
Chainages	The chainages used in this report were taken from the horizontal and vertical alignment revision P2.3 drawings. Any changes subsequent to this issue have not been incorporated in this impact assessment.
Referenced reports	The scope does not include verifying the accuracy or completeness of work of others and therefore in performing this task we rely upon, and presume accurate, any information provided by the client and/or other sources on behalf of the client.
Future Developments	Future developments are covered in the Technical Appendix E *Land Use and Planning*.
Volume loss	For purposes of settlement analysis the following volume losses has been adopted: • 0.5 per cent in rock • 1 per cent for in soft ground conditions with Closed Face TBM (EPB/Slurry) These are expected to be achieved by the project using competent but typical construction techniques and management.
Settlement Trough width	For the purposes of settlement analysis, the settlement trough, expressed as the width to the point of contra flexure, is the depth to the centre of the tunnel multiplied by 0.4 in soil, 0.6 in rock, except where the tunnel would be more than two diameters, where the multiplier is 0.7
Settlement analysis method	The empirical method adopted for settlement analysis, as confirmed at certain sections using FE modelling, is applicable for all ground conditions where it is used.
Water tightness of tunnels	The water tightness of tunnels under Moonee Ponds Creek and Yarra River areas would be to be constructed to specified level.
Consolidation Settlements	• Consolidation settlements are generally relatively uniform if the depth and compressibility of the underlying soft soils are uniform. • If uniform, consolidation settlements do not cause tensile strain in near surface structures and do not impact assets adversely. • Consolidation settlements have negligible effect because of their flat nature away from any geological variations
Further heritage advice	The design of permanent infrastructure where this interfaces with heritage places would be developed with further heritage advice and input (consistent with Environmental Performance Requirement CH2).
Construction methodology	The assessment also assumes that ground movements and associated potential impacts would be minimised by adopting sound engineering practices which would include the engagement of contractors with the appropriate levels of skill and experience, using the proposed, or equivalent construction methodologies and managing the excavation sequencing and appropriate controls on TBM operation. In addition, appropriate real time monitoring programs would be implemented from the onset of construction.

6 Estimated Ground Movement

6.1 Preliminary Ground Movement Estimates

Due to the preliminary stage of design development when the preliminary ground movement assessment was completed, the calculated settlement values are approximate and based on a number of prudently conservative assumptions, including simplified ground profiles and preliminary design geotechnical parameters.

The predictions of potential settlements are considered to be reasonable initial estimates and are based on Golder Associates and the AJM JV past deep excavation and tunnelling experience in both Melbourne and worldwide; however further analyses would be required to refine them going forward as the project scheme is developed.

Melbourne Metro structures and utility connections would be designed to accommodate potential differential settlement that might occur between a zone undergoing consolidation settlement and stiffer components of proposed structures which could be founded in deeper strata.

6.1.1 Excavation Induced Ground Movement

Construction methods and sequences have a significant influence on ground surface settlements for both bored tunnels and cut and cover excavations such as tunnels and stations. If construction methods or sequences change, wall deflections would probably change from the design estimates. Potential impacts of potential changes would be assessed through additional ground movement and impacts assessments.

The effects of portals, tunnels and station boxes and cavern excavations and groundwater drawdown induced ground movements for the Concept Design are presented as indicative excavation induced ground settlement contours provided in Appendix C of this report.

6.1.1.1 Tunnelling Induced Ground Movement

The use of semi-empirical methods such as the Gaussian curve to approximate the shape of ground surface deformation is considered applicable, although much of the underground excavations would be through Melbourne Formation, a weak rock mass. The 2D PLAXIS assessments completed by Golder Associates were found to validate the assumed tunnelling induced settlement parameters.

The results typically demonstrate the requirement for tight control on TBM operation, particularly through zones of alluvial sediments along with the requirement to monitor excavated volumes against tunnelling advance rates and surface level monitoring. Further discussion is found in Appendix B of this report.

As reported in Appendix B to this report, *Golder Associates Ground Movement Assessment EES Summary Report*, the key findings of the preliminary settlement assessments indicate that settlements at the surface due to tunnel excavations can be summarised as follows:

- A maximum settlement of up to 60 mm is predicted at sections underlain by soft soils
- Where the ground conditions comprise mostly fill and residual/alluvial soil, estimated ground settlements typically range between about 10 mm and 40 mm
- Less than about 5 mm for deep tunnels to a maximum of about 20 mm in shallower sections in weathered rock
- The influence of excavation induced ground movement along the proposed Melbourne Metro alignment, varies between a maximum of 80 m around the section between the western portal and Arden station and a maximum of about 50 m between Alexandra Avenue and City Link tunnels area

- The cross passage settlement has been found to be minimal in rock ground conditions. Magnitude of settlement typically ranges from 2 mm to 5 mm, depending on the depth of ground cover and the horizontal tunnel separation distance.

6.1.1.2 Open Cut Excavations

Key findings of the preliminary settlement assessment indicate that settlements at the surface due to open cut excavations can be summarised as follows:

- Estimated ground settlements due to the station box excavations vary significantly across the site, with the largest predicted ground settlement of about 40 mm occurring at the Arden station area, adjacent to the excavation
- A maximum settlement and horizontal displacement of less than about 20 mm was predicted at the critical section within the weathered rock at the Parkville station area, adjacent to the excavation
- The potential zone of influence relating to excavation induced ground movement, around the proposed Melbourne Metro stations excavations, is predicted to range typically between about 20 m and 30 m.

6.1.2 Primary Consolidation Settlement

The preliminary predictions of consolidation settlement presented in Appendix D of this report take account of the results of the preliminary regional groundwater numerical modelling as well as the predictions of potential groundwater drawdowns during construction and operational phases of selected elements of the project, described in the Technical Appendix O *Groundwater*.

It is assumed the construction activities would manage groundwater inflows such that the drawdowns would not be greater than those shown in Appendix D of this report. It should be noted that, in the event that higher drawdowns beneath the Coode Island Silt are induced during construction, larger settlements than those currently predicted would occur.

The estimated consolidation settlement contour drawings are provided in Appendix D of this report. The Potential Zone of Influence due to groundwater drawdown is much greater than that estimated for excavation induced settlement. Technical Appendix O *Groundwater* describes the estimated construction and operational stage drawdown and inflows; assumptions and limitations around the completed hydrogeological assessments and cumulative drawdown impacts.

The impacts of potential consolidation settlement are localised and confined to inferred zones of soft soils, namely Coode Island Silt. The contours show that groundwater drawdown during the construction stage has the potential to cause consolidation that might continue into the project operational stage, with control measures to limit inflow of groundwater into Melbourne Metro excavations, as summarised in Table 6-1.

Potential consolidation settlement elsewhere on the project, resulting from construction stage inflows or operational stage groundwater drawdown, is estimated to be less than 10 mm. Potential impacts related to settlement of this magnitude are considered negligible (Rankin 1988).

As reported in Appendix B to this report, *Golder Associates Ground Movement Assessment EES Summary Report,* consolidation settlement of the Coode Island Silt due to groundwater drawdown was estimated to be typically less than about 10 mm, except in the areas of western portal precinct near Maribyrnong River and Arden station precinct near the Moonee Ponds Creek. Consolidation settlements up to 50 mm and 100 mm were estimated in the areas of western portal and Arden Station, respectively. The higher settlements are confined to areas of higher predicted drawdowns close to the Melbourne Metro alignment, where Coode Island Silt is thicker.

At locations where substantial thickness (greater than 5 m) of Coode Island Silt exists, there is a potential for ongoing creep settlement (secondary compression) which currently has not been taken into consideration. Creep settlement has not been assessed and is not included in the settlement contour drawings provided in Appendix D to this report. It should be noted that creep settlement is a natural background settlement that would occur regardless of whether Melbourne Metro is constructed or not.

6.1.3 Combined Effects

Figure 6-1 to Figure 6-4 present extracts of the settlement contour drawings found in Appendix C and Appendix D of this report. The extracts show estimated primary consolidation settlement alongside the corresponding estimated excavation induced settlement at selected locations along the alignment where combined ground movement (consolidation and excavation induced) effects have been identified as a potential risk to existing assets.

The consolidation contours show that the maximum settlement induced by the drawdown of groundwater under buildings could potentially be found immediately north of the proposed Arden station.

The boundaries and contours of the estimated primary consolidation settlement are related to the inferred thickness of the soft soils and the estimated groundwater drawdown contours, an output from the regional hydrogeological model.

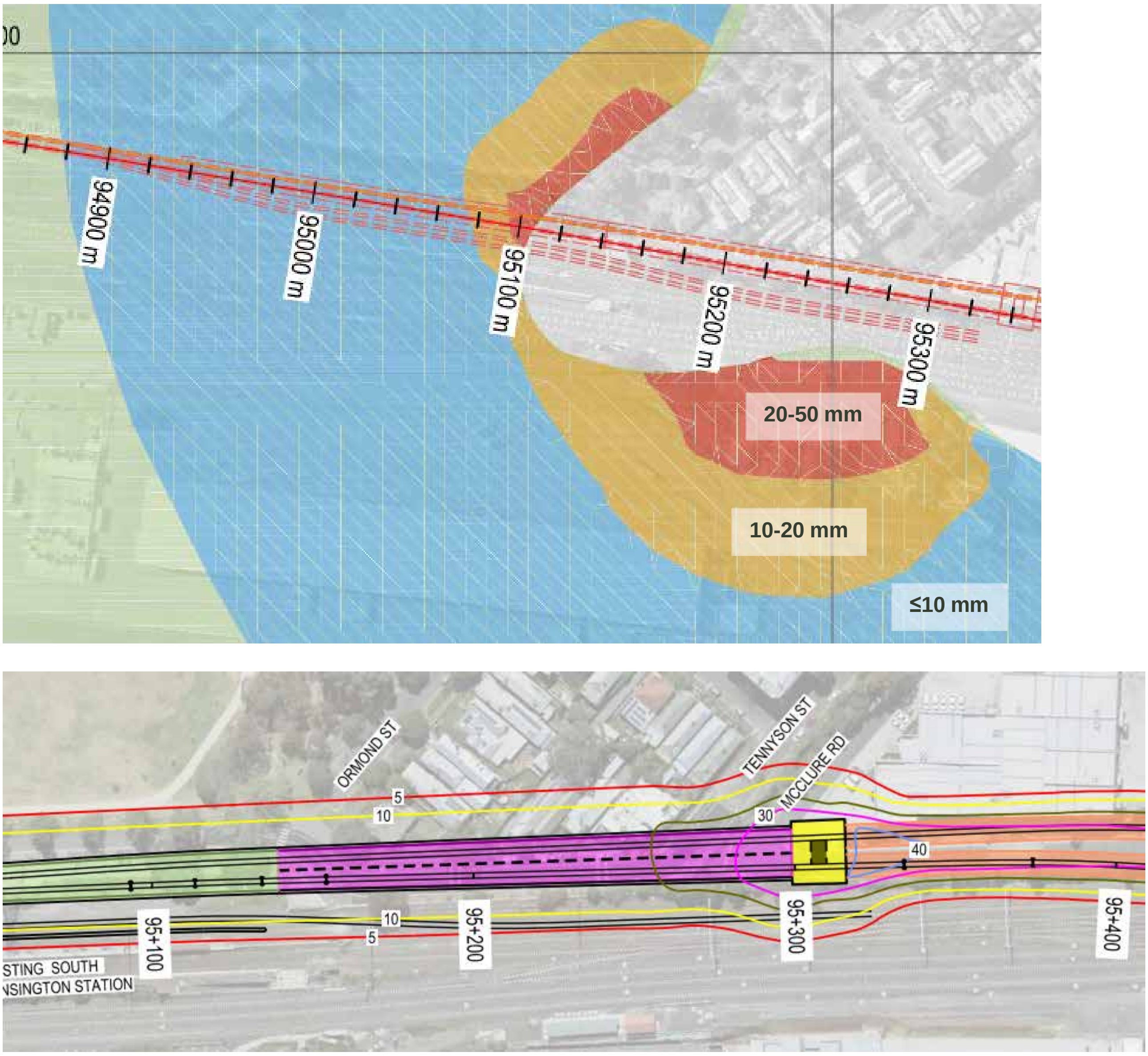

Figure 6-1 Western Portal: Comparison of consolidation and excavation induced settlement

It can be seen that the residential zone at the western portal that may experience excavation induced ground movement lies outside the zone of potential consolidation settlement due to the variation in the underlying geology.

Localised zones along the existing rail embankment may experience combined effects. Some light industrial structures south of the portal area may experience localised consolidation settlement. Existing silos that are located immediately south of the western portal are likely to be founded on piles or directly on basalt and are considered at very low risk due to Melbourne Metro works.

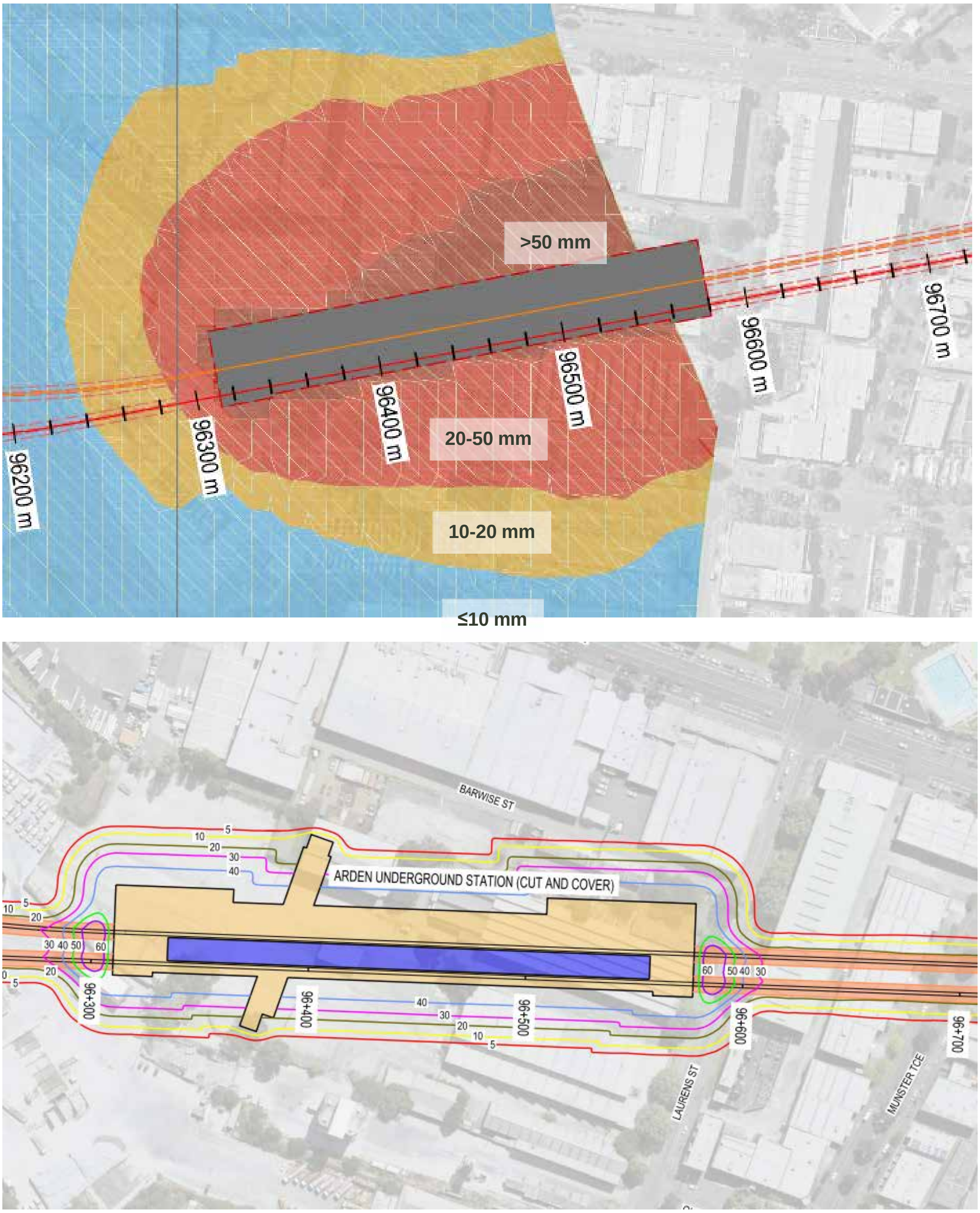

Figure 6-2 Arden station: Comparison of consolidation and excavation induced settlement

It can be seen that there is potential for unacceptable ground movement in the immediate area surrounding the proposed Arden station, potentially impacting some buildings that front onto Arden Street.

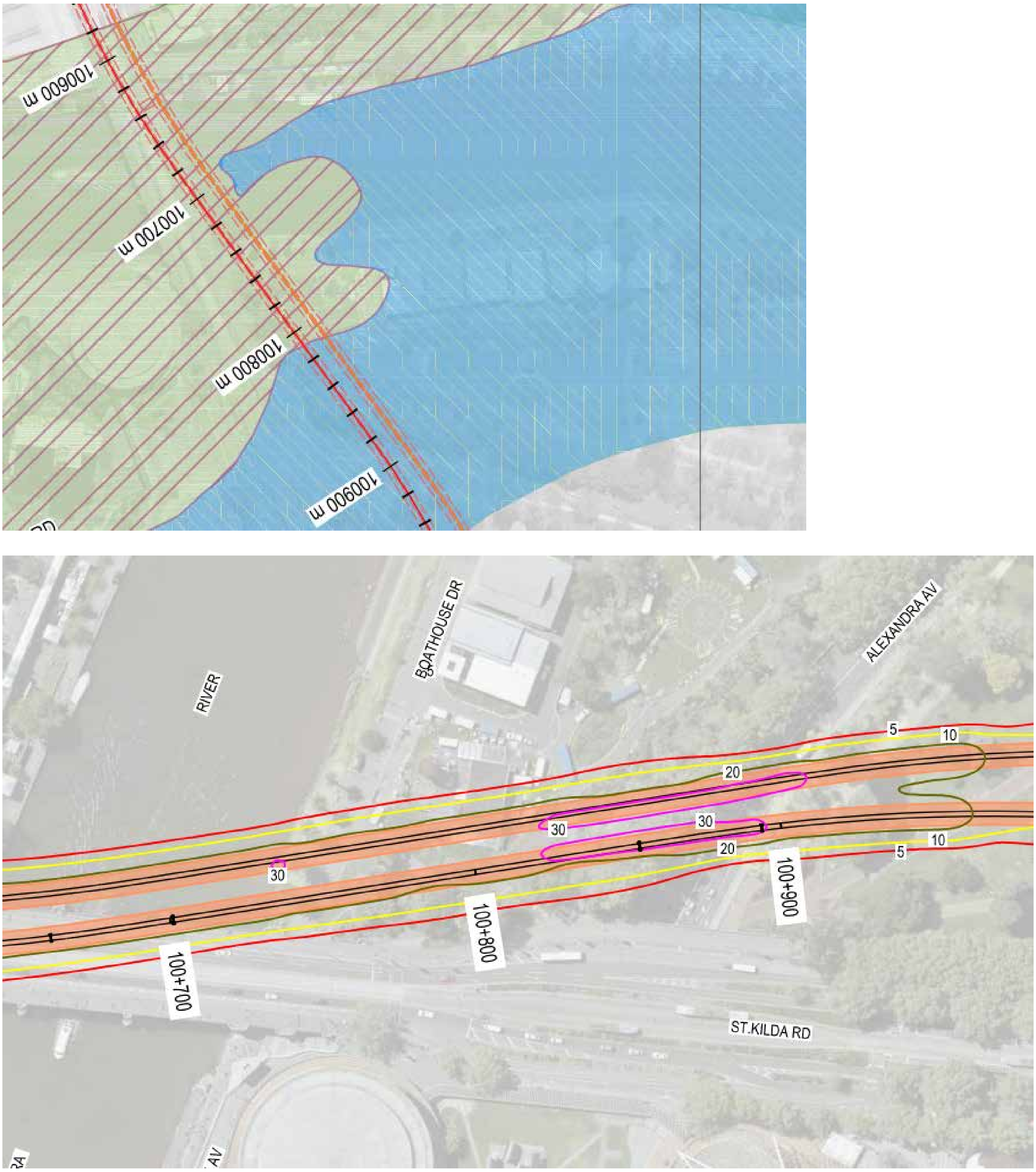

Figure 6-3 South bank of the Yarra River: Comparison of consolidation and excavation induced settlement

There is potential for combined effects of consolidation settlement plus tunnelling induced settlement in the parkland immediately south of the Yarra River crossing up to the Queen Victoria Gardens. The estimated primary consolidation settlement is equal to or less than 10 mm. This magnitude might be comparable to the ongoing background secondary compression and due to the inferred consistent thickness of soft soils across this zone, The primary consolidation settlement is not considered to increase the vulnerability of existing assets to adverse impacts due to ground movement. Excavation induced ground movement alone was considered in the impacts assessments in this location.

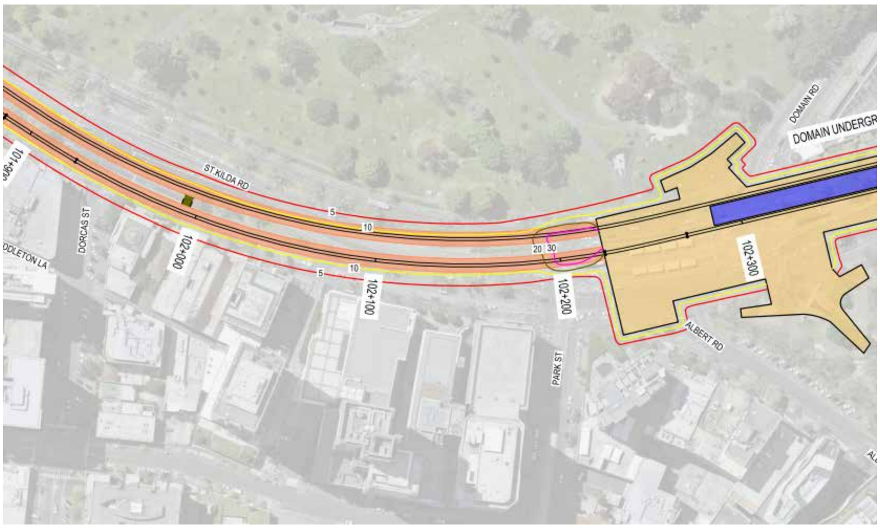

Figure 6-4 Domain station: Comparison of consolidation and excavation induced settlement

It can be seen that there is no overlap of excavation induced settlement contours and the estimated primary consolidation settlement contours in the vicinity of Domain Station. Impacts due to excavation induced settlement were considered in the assessment. Estimated primary consolidation settlement of 10 mm or less was not considered to pose a measurable risk to existing assets in this area.

6.2 Early Works

A number of early works are required prior to the commencement of the main construction works. The early works would comprise modifications, temporary works, relocations or new works associated with existing utilities and tram works.

Any other temporary works assessments are outside the scope of this ground movement assessment report. Temporary works excavations and trenching would typically be much smaller in magnitude to the assessed works and any ground movement effects would be localised. The Contractor would be required to take appropriate measures during construction to mitigate or manage any associated ground movement risks.

The early works package comprises selected structures at the proposed stations and the portals. As described in Section 4.4, excavation induced ground movements may occur wherever excavations are undertaken. Works that would intersect the water table and cause groundwater drawdown may pose a risk of inducing primary consolidation settlement.

Early works with the potential to induce ground movements and potentially impact the groundwater table would be:

- North Yarra Main Sewer
- South Yarra Main Sewer replacement and realignment at Domain station
- The Franklin Street east shaft
- The A'Beckett Street shaft
- The demolition of the car park beneath City Square.

A summary of the estimated impacts to buildings overlying the realigned South Yarra Main Sewer and impacts to North Yarra Main Sewer are found in Section 8 and Section 9.8 respectively.

Estimations of excavation induced settlement are presented in the Appendix C *Excavation Induced Settlement Contour Drawings* of this report.

The groundwater impacts associated with the above shafts and City Square car park demolition have been assessed together with the other Melbourne Metro structures in the station precincts and are reported in the relevant station precinct sections of the Technical Appendix O *Groundwater*.

Potential impacts resulting from proposed demolition works are summarised in Section 8.1.2.

6.3 Potential Zone of Influence

The Potential Zone of Influence relating to ground movement is defined by:

- The 5 mm excavation induced ground surface settlement contours and
- The primary consolidation settlement contours with estimated consolidation settlement of 10 mm or greater.

Structures and underground services located between or touching the contours defined by the above zones are considered within the Potential Zone of Influence and should be included in the register of potentially affected properties and structures, in relation to potential ground movement impacts.

The excavation induced 5 mm contour was conservatively selected to increase the extent of ground movement impact assessments around Melbourne Metro excavations. Adopting the 5mm contour increases the number of assets in the Potential Zone of Influence and ensures a review of the nature of the existing structures would be undertaken. A preconstruction condition inspection might also be undertaken to inform detailed design assessments, where required. The Potential Zone of Influence may be revised based on the detailed design stage assessments.

The geological characteristics and extents of soft soils prone to consolidation settlement, namely Coode Island Silt, are documented in Appendix A of this report. Zones with estimated potential consolidation settlement of less than 10 mm might be experiencing equivalent magnitudes of ongoing creep on a yearly basis, depending on the actual thickness of soft sediments. For this reason, along with the fact that ground movement of less 10 mm has been documented to pose negligible risk to structures, where slope of deformation profile is less than 1/500 (Rankin 1988), the 10 mm consolidation settlement contour was adopted to define the extent of the Potential Zone of Influence relating to potential ground movement. It is also assumed that potential consolidation settlement would be generally uniform for the soft soil zones potentially impacted by groundwater drawdown.

It is also noted that seasonal movements of the ground mass along the project alignment as well as seasonal movements of existing structures may be in the order of a few millimetres and differentiation of

Melbourne Metro induced ground movements where less than 10 mm from seasonal ground movements would be difficult.

The potential for damage to structures is dependent on the structure type, the current condition of the structure and the differential settlement across the structure. Generally, the potential for property damage was found to be negligible or minor. Zones of soft soils might be vulnerable to the effects of groundwater drawdown with some potential for impacts to predominantly residential buildings and some civil infrastructure.

Where necessary, mitigation measures have been identified to limit unacceptable predicted effects so that acceptable outcomes would be achieved in respect of settlement on structures, utilities or infrastructure. Results of assessments are given in tabular form along with suitable protective measures to be adopted, where appropriate.

Prior to construction, detailed condition surveys of potentially affected structures would be conducted that would identify increased vulnerability of some structures. These structures could have higher susceptibility to adverse impacts from ground movements. If such structures are identified, the detailed design stage impact assessment inputs would be selected to reflect the current condition of the particular structure, additional mitigations to minimise ground movement might be required or a requirement for protective works might be realised.

The drawings in Appendix C of this report present the preliminary estimates of the excavation induced ground settlement contours. Potential consolidation settlement due to groundwater drawdown are presented in Appendix B of this report. Figure 6-5 to Figure 6-9 show the estimated extent of the Potential Zone of Influence.

6.4 Timeline of Ground Movement Occurrence

Excavation induced ground movements typically occur in response to the actual excavation works, when the state of stress in the ground mass is altered. Excavation induced ground movements typically cease when underground excavation primary linings have been installed and open cut excavations have been completed and retention systems fully installed, at which time the altered state of stress in the ground mass has reached equilibrium around the new excavation.

Primary consolidation settlement is a secondary effect of groundwater inflow to excavations and the effects of the associated groundwater table drawdown would be measurable sometime after Melbourne Metro excavations commence. The highest groundwater drawdowns typically result from construction stage groundwater inflows to excavations. Once excavations are tanked and inflows limited to acceptable levels, the potential for consolidation settlement reduces substantially. However, the subsequent recovery of groundwater levels does not result in a recovery of consolidation settlement that has already occurred. Time for groundwater levels to recover may overlap with the project operational phase, so consolidation settlement triggered during the construction phase could continue into the project operational phase.

The excavation induced settlement results generally represent the potential ground movements that would have occurred at completion of construction stage. A consideration in detailed design assessments is the interim condition where the differential settlement above an advancing underground excavation face might present the risk of adverse impacts to existing assets, although total settlement estimates might suggest a negligible or minor impact. An example of such a situation is where the settlement trough associated with the excavation of one of the twin tunnels is more adverse, from a differential settlement perspective, than the settlement trough associated with the excavation of both tunnels.

6.5 Measures to Limit Ground Movement

Potential for impacts to existing structures and infrastructure cannot be eliminated and would be managed through the adoption of measures to limit ground movement. Measures would be taken to limit ground movement around an excavation or it’s propagation to ground surface level. Ground improvement measures (pre-injection, jet grouting, etc.) may be adopted at some locations to improve ground mass strength and resist local deformation. Additional mitigations for potential ground movement risks may also need to be incorporated in the final design and adopted construction method.

6.5.1 Control Measures Inherent in the Concept Design

Table 6-1 below summarises the measures that are inherent in the Concept Design. These measures are incorporated into the completed impacts assessments and are reflected in the initial risk ratings.

The assessment also assumes that ground movements and associated potential impacts would be minimised by adopting sound engineering practices which would include engaging contractors with the appropriate levels of skill and experience, using the proposed or equivalent construction methodologies and managing the excavation sequencing and appropriate controls on TBM operation. In addition, comprehensive ground movement and groundwater monitoring programs would be implemented from the onset of construction.

Table 6-1 Ground Movement Control Measures inherent in the Concept Design

Precinct	Construction type[1]	Controls inherent in the Concept Design Scheme[2] and proposed methodology
Precinct 1 Tunnels: ***Western portal to Arden***	• Driven twin tunnels using closed mode TBM with precast reinforced concrete segmental lining.	• Provision of ground support using precast reinforced concrete segmental lining. Segments are provided with hydrophilic gaskets for ground water control. • Ground improvement at the interface with Arden station box to limit settlement.
	• Cross passages mined by small road header or excavator with a combination of rock bolt, shotcrete and steel set primary support dependant on ground conditions.	• Ground treatment to limit groundwater inflows and consolidation settlement and ground treatment to strengthen the ground and limit ground settlement and the risk of ground collapse.
Precinct 1 Tunnels: ***Arden to Parkville***	• Driven twin tunnels using closed mode TBM with precast reinforced concrete segmental lining	• Provision of ground support using precast reinforced concrete segmental lining. Segments are provided with hydrophilic gaskets for ground water control. • Ground improvement at the interface with Arden station box to minimise excessive settlement.
	• Cross passages mined by small road header or excavator with a combination of rock bolt, shotcrete and steel set primary support dependant on ground conditions.	• Ground treatment to limit groundwater inflows and consolidation settlement.
Precinct 1 Tunnels: ***Parkville to CBD North***	• Driven twin tunnels using closed mode TBM with precast reinforced concrete segmental lining.	• Provision of ground support using precast reinforced concrete segmental lining. Segments are provided with hydrophilic gaskets for ground water control.
Precinct 1 Tunnels: ***CBD North to CBD South***	• Mined technique for twin tunnels using road header or excavators.	• Design of primary tunnel support to ensure excavation stability and limit ground movement. • Grouting of the excavation face may be required to limit groundwater inflows.
Precinct 1 Tunnels: ***CBD South to Domain***	• Driven twin tunnels using closed mode TBM with precast reinforced concrete segmental lining.	• Provision of ground support using precast reinforced concrete segmental lining. Segments are provided with hydrophilic gaskets for ground water control. • Potential ground improvement where the alignment

Precinct	Construction type[1]	Controls inherent in the Concept Design Scheme[2] and proposed methodology
		goes over the existing CityLink tunnels with shallow cover to the ground surface to limit ground settlement and the risk of ground collapse. • Provision of temporary ground water injection wells within the Moray Street Gravels to limit groundwater drawdown within overlying units (especially Coode Island Silt) to limit consolidation settlement.
	• Cross passages mined by small road header or excavator with a combination of rock bolt, shotcrete and steel set primary support dependant on ground conditions.	• Ground treatment to limit groundwater inflows and consolidation settlement (XP11, XP12) and ground treatment to strengthen the ground and limit ground settlement and the risk of ground collapse (potentially XP14).
	• Linlithgow Avenue emergency access shaft using Soldier Piles with shotcrete lagging or similar retaining walls at shallow depths, with precast segmental lining at depth.	• Ground Support designed to limit ground movement. • Provision of pre-injection grouting to limit groundwater drawdown and limit consolidation settlement.
Precinct 1 Tunnels: ***Domain to Eastern Portal***	• Driven twin tunnels using closed mode TBM with precast reinforced concrete segmental lining.	• Provision of ground support using precast reinforced concrete segmental lining. Segments are provided with hydrophilic gaskets for ground water control. • Potential ground improvement where the TBM travels below properties at shallow cover on approach to the eastern portal.
	• Fawkner Park emergency access shaft using Soldier Piles with shotcrete lagging or similar.	• Piles and retaining walls designed to limit ground movement.
Precinct 2 Western portal	• Precast reinforced concrete walls or reinforced soil wall for embankment tie-in. • Secant pile walls with precast reinforced concrete walls at shallow sections. • Secant piles at cut and cover section and western TBM retrieval point. • Embankment.	• Secant pile wall around perimeter of the dive structure, cut and cover structure and TBM retrieval shaft. • Soil mixing in Coode Island Silt to provide retaining wall toe stability or soil replacement. • Toe grouting extending 5 m beneath the wall. (see Table 8 in Appendix B of this report). • Provision of temporary injection wells within the Moray Street Gravels to limit groundwater drawdown within overlying units (especially Coode Island Silt) to limit consolidation settlement (See Technical Appendix O *Groundwater*).
Precinct 3 Arden	• Diaphragm walls.	• Diaphragm wall around the perimeter of the station box. • Toe grouting extending 10m beneath the wall. (see Table 7 in Appendix B of this report). • Provision of temporary injection wells within the Moray Street Gravels to limit groundwater drawdown within overlying units (especially Coode Island Silt) to limit consolidation settlement (See Technical

Precinct	Construction type[1]	Controls inherent in the Concept Design Scheme[2] and proposed methodology
		Appendix O *Groundwater*).
Precinct 4 Parkville station	• Soldier Piles or similar retaining walls.	• Pile and retaining walls designed to limit ground movement.
Precinct 5 CBD North station	• Mined cavern and adits constructed with primary rock support of rock bolts and shotcrete. permanent lining would be cast in place reinforced concrete. • Access shafts Soldier Piles or similar retaining walls.	• Design of primary cavern and adits support to ensure excavation stability and limit ground movement. • Grouting of the excavation face may be required to limit groundwater inflows.
Precinct 6 CBD South station	• Mined cavern and adits constructed with primary rock support of rock bolts and shotcrete. Permanent lining would be cast in place reinforced concrete. • Access shafts Soldier Piles or similar retaining walls.	• Design of primary cavern and adits support to ensure excavation stability and limit ground movement. • Provision of pre-injection grouting and temporary ground water injection wells to limit groundwater drawdown and in the local compressible soils, limit consolidation settlement (See Technical Appendix O *Groundwater*).
Precinct 7 Domain station	• Diaphragm walls.	• Diaphragm wall around the whole station. • Potential pipe roof canopy at the TBM launch and retrieval points to limit ground settlement.
Precinct 8 Eastern portal	• Secant pile at western retrieval shaft and cut and cover section. • Secant pile walls with precast reinforced concrete wall at shallow section at dive structure. • Soil nail walls, where appropriate and contiguous bored piles at cutting retention system (existing rail corridor).	• Secant piles and soil nail walls are designed to limit ground movement. • Pipe roof canopy at the TBM recieval point to limit ground settlement.
Precinct 9 Western Turnback	No underground or open cut excavations in the Concept Design that could cause ground movement or groundwater drawdown.	

1 Minimum water tightness requirements are as described in the Technical Appendix O *Groundwater*.
2 Comprehensive ground movement and groundwater monitoring network would be required through Melbourne Metro construction stage and the operational stage at zones susceptible to consolidation settlement and located within the potential zone of influence.

Table 6-2 Ground Movement Control Measures inherent in the Alternative Design Options

Precinct	Construction type[1]	Mitigations inherent in the Proposed Scheme[2] and proposed methodology
Precinct 1 Tunnels: ***CBD South to Domain***	**CityLink: Melbourne Metro crossing under** • Driven twin tunnels using closed mode TBM with precast reinforced concrete segmental lining.	• Provision of ground support using precast reinforced concrete segmental lining. Segments are provided with hydrophilic gaskets for ground water control. • Potential ground improvement where the alignment goes over the existing CityLink tunnels with shallow cover to the ground surface to limit ground settlement and the risk of ground collapse. • Provision of ground water recharge facilities within the Moray Street Gravels to limit groundwater drawdown within overlying units (especially Coode Island Silt) to limit consolidation settlement.
	Linlithgow emergency access shaft at Tom's Block • Linlithgow Avenue emergency access shaft using Soldier Piles with shotcrete lagging or similar retaining walls at shallow depths, with precast segmental lining at depth.	• Ground Support designed to limit ground movement. • Provision of pre-injection grouting to limit groundwater drawdown and limit consolidation settlement.
Precinct 1 Tunnels: ***Domain to Eastern Portal***	**Fawkner Park emergency access shaft at Tennis Courts** • Fawkner Park emergency access shaft using Soldier Piles with shotcrete lagging or similar.	• Piles and retaining walls designed to limit ground movement.
Precinct 2 Western portal	**Western Portal Option 4** • Precast reinforced concrete walls or reinforced soil wall for embankment tie-in. • Secant pile walls with precast reinforced concrete walls at shallow sections. • Secant piles at cut and cover section and western TBM retrieval point.	• Secant pile wall around perimeter of the dive structure, cut and cover structure and TBM retrieval shaft. • Soil mixing in Coode Island Silt to provide retaining wall toe stability. • Toe grouting extending 5 m beneath the wall (Table 8 of Appendix B of this report). • Temporary recharge scheme (See Technical Appendix O *Groundwater*).

1 Minimum water tightness requirements are as described in Technical Appendix O *Groundwater*.
2 Comprehensive ground movement monitoring network would be required through Melbourne Metro construction stage and the operational stage at zones susceptible to consolidation settlement and located within the zone defined by the estimated groundwater drawdown cones.

Appropriate measures to limit ground movement would be refined in the detailed design stage. Ground movement assessment inputs and control measures that would need careful consideration include:

- Additional geotechnical investigations for improved definition of the geological profile along the alignment
- Modelling of groundwater drawdown resulting from Melbourne Metro works and consideration of the likely effects of estimated groundwater drawdown as part of detailed design
- Volume loss estimations
- Design of underground excavation support and liners
- Design of open cut excavation support systems

- Design of a comprehensive ground movement and groundwater monitoring program
- Completion of more detailed assessments of movement sensitive structures within the potential zone of influence to support excavation design and the ground movement and groundwater monitoring programs.

When considering the need and type of mitigation measures, the sensitivity of the structure or building features which are of heritage value and the sensitivity of the structure to ground movement are examined.

In the event that mitigation measures are not considered to reduce the risk of asset damage to acceptable levels, protective measures could be recommended for an asset. For difficult or severe cases, these could include underpinning or structural strengthening.

To mitigate estimated impacts on utilities that were assessed to be moderate or greater to lower levels, installation of a lining might be necessary. Any requirement to line existing utility conduits would be developed on completion of further analysis work at detailed design stage. The requirement for strengthening works would also depend on the current condition of the utilities. Although the risk to utility serviceability might be assessed as relatively low, installation of lining might be considered as a control measure to minimise consequences of potentially unacceptable serviceability damage. The appropriate mitigation measures can only be fully developed in consultation with the respective asset owners.

6.5.2 Construction Stage Controls

During the Melbourne Metro construction stage, measures to manage the implementation of ground movement control and risk mitigation would be required to reduce or avoid the potential for adverse impacts of ground movement on buildings, civil infrastructure, utilities and parkland, including:

- Conducting condition surveys before construction commences of buildings (including heritage properties), structures, pavements and other significant features within the Potential Zone of Influence to establish baseline conditions. The actual settlements would be compared to predicted settlements and further mitigating measures taken where adverse departures from predictions are noted
- Identifying the potential effects (if any) of settlement as a consequence of Melbourne Metro by reviewing the condition survey results in consultation with property owners, where appropriate
- Developing and implementing a ground movement and groundwater monitoring plan to detect ground movement and changes in groundwater levels. The monitoring plan would include trigger levels to ensure appropriate action is taken when the measured responses approach maximum allowable levels
- Groundwater management strategy such as carrying out targeted pre-excavation grouting where necessary to limit construction stage groundwater inflows
- Implementing feasible and reasonable measures during construction to limit operational inflows to excavations
- Making provision for reinstatement works in the unlikely event of damage to structures resulting from project works
- Designing Melbourne Metro structures and utility connections to accommodate potential differential settlement that might occur between a zone undergoing consolidation settlement and stiffer components of proposed Melbourne Metro structures, founded in in deeper strata
- Control of volume losses, see Section 6.5.2.1.

Monitoring would be used as a management tool to check that the actual amounts and patterns of movement are similar to those predicted and not exceeding allowable limits. It would also be used where needed to identify whether reactive protective works are required. Details of typical monitoring phases are described in Section 11.4 of this report.

Ongoing monitoring would be implemented at surface settlement points along the proposed Melbourne Metro alignment, preferably commencing at least 12 months before construction commences and continuing for not less than six months after any ground movement has stabilised. This monitoring program would include a series of ‘trigger levels’ to ensure that action is taken early to review or revise work methods to avoid any exceedances of the agreed settlement criteria.

Monitoring of existing assets is not a protective measure but would trigger timely implementation of mitigation measures; if required. These measures would apply across all construction activities with potential to cause ground movement along the Melbourne Metro alignment.

6.5.2.1 Control of Volume Losses

Volume losses would be limited on Melbourne Metro to limit ground deformations and any subsequent potential impacts. This would be achieved by the contractor by maintaining tight control on TBM operations such as:

- Selection of the appropriate machine type and cutter head arrangements and mode options
- Making provision to the extent feasible, to use TBM with control systems for tunnel construction where practical, to minimise ground loss
- Management and control of spoil extraction, and coordination with TBM advance rate
- Implementing excavation monitoring systems within the Tunnel Boring Machines with scales to allow real time comparison of theoretical excavation volumes against actual extracted volumes from the face for early indication of excessive face loss. Ensure the machines have adequate power to balance face pressure and support the ground without causing overpressure or weakening of the ground mass. Ensuring adequate supply of annular grout at all times and ensuring grout is placed within a suitable timeframe in the construction cycle
- Maintenance of a comprehensive ground movement and groundwater monitoring system above and adjacent to TBM operations with real time reporting capability.

The Environmental Performance Requirements establish a regime to ensure that these types of measures are incorporated within the design, construction and operation of Melbourne Metro.

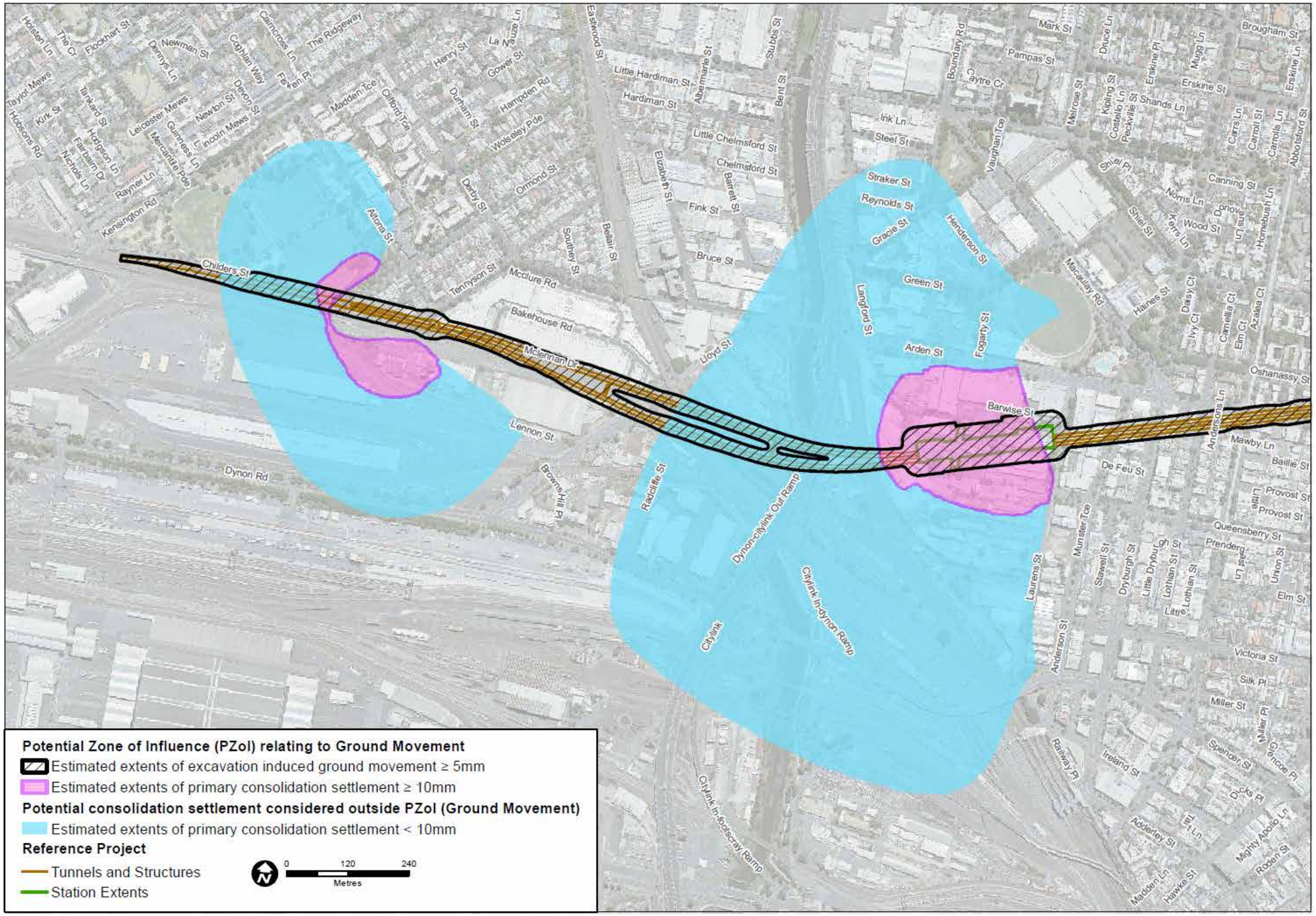

Figure 6-5 Potential Zone of Influence, relating to Ground Movement Sheet 1 of 5

Figure 6-6 Potential Zone of Influence, relating to Ground Movement Sheet 2 of 5

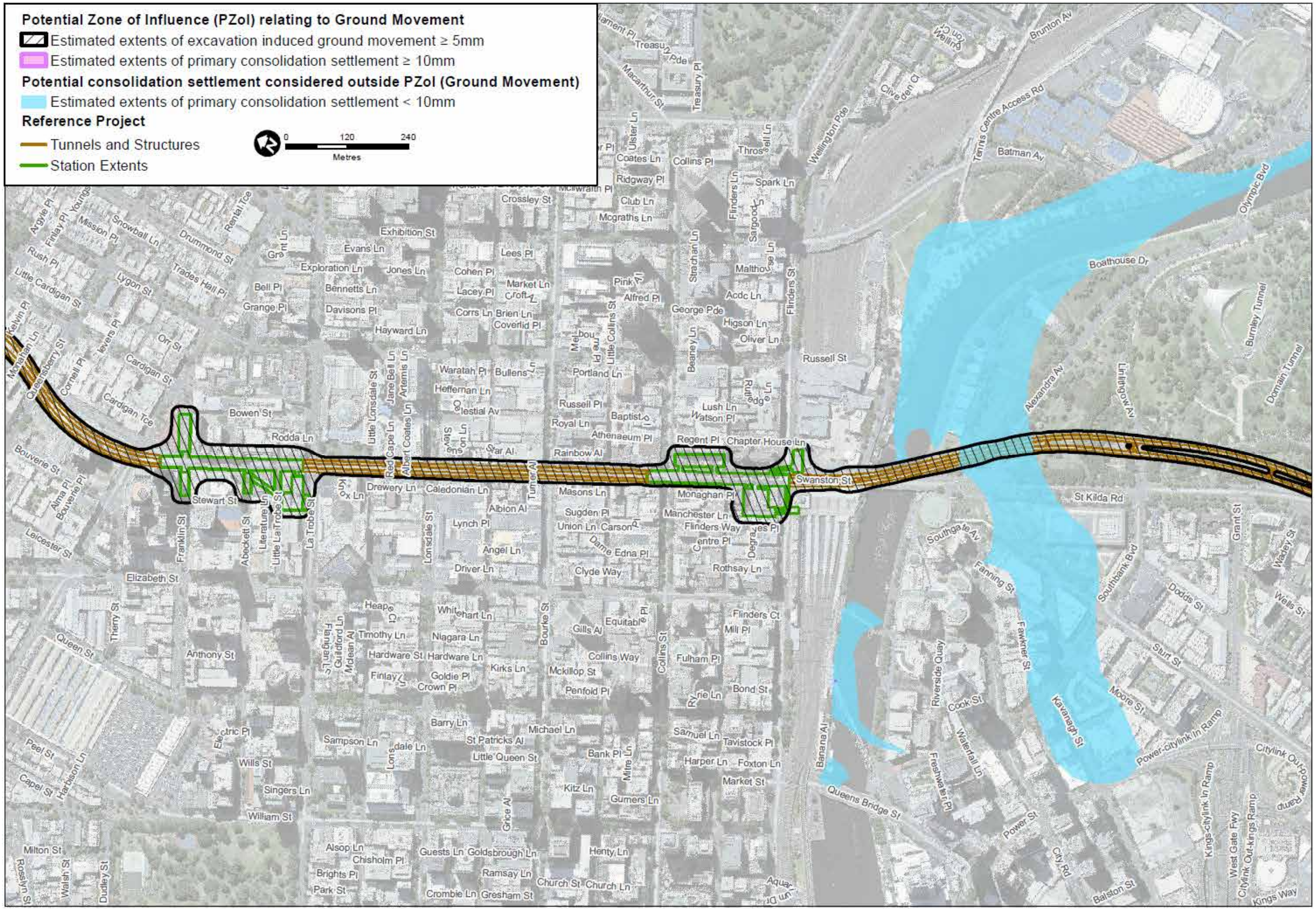

Figure 6-7 Potential Zone of Influence, relating to Ground Movement Sheet 3 of 5

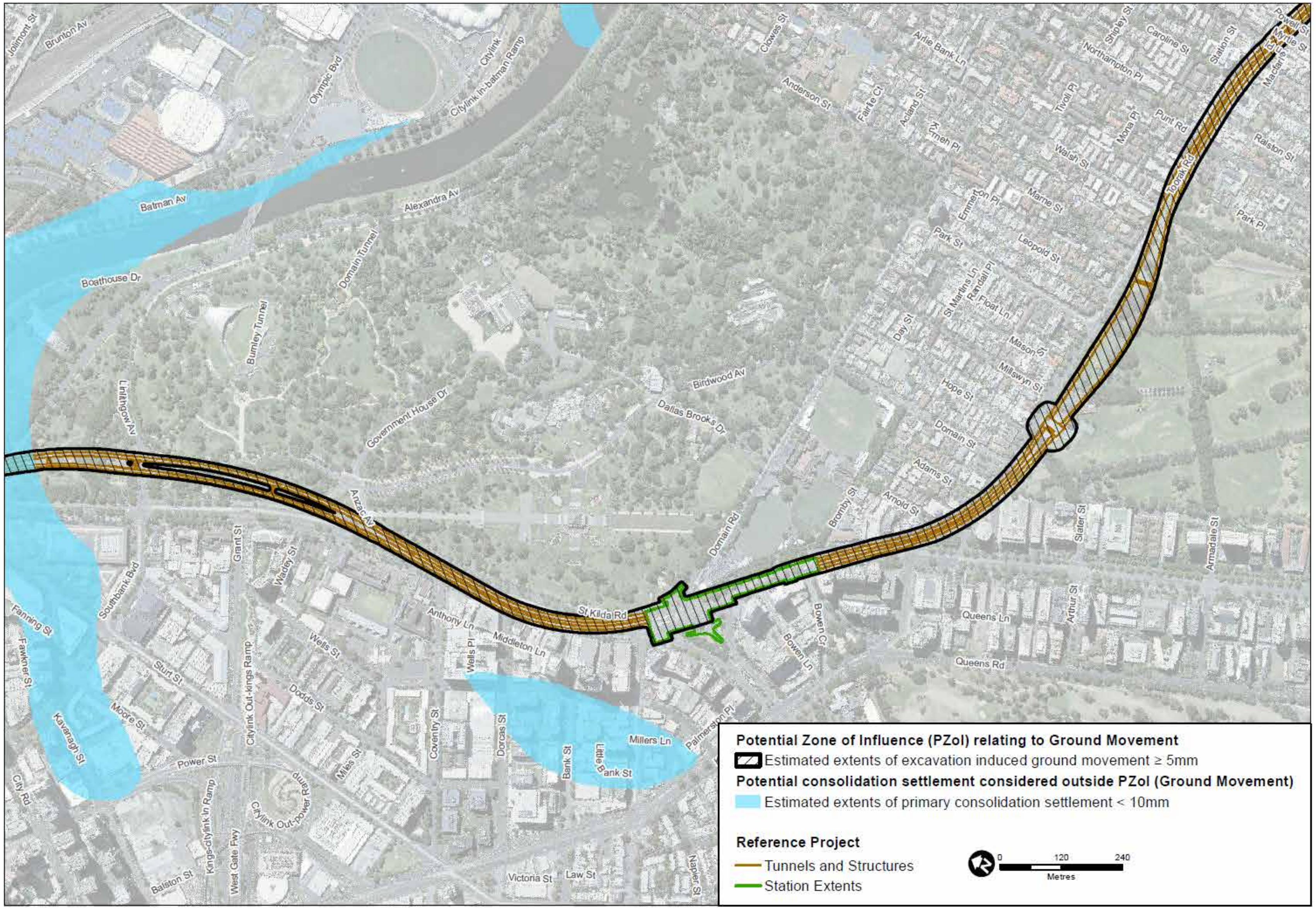

Figure 6-8 Potential Zone of Influence, relating to Ground Movement Sheet 4 of 5

Figure 6-9 Potential Zone of Influence, relating to Ground Movement Sheet 5 of 5

7 Ground Movement Risk Assessment

Table 7-1 presents the ground movement risks associated with the project, based on a precinct basis. The environmental risk assessment methodology is outlined in Section 5.4. Golder Associates participated in and contributed to the EES Ground Movement risk register workshop along with representatives from the EES technical discipline team representatives.

Measures to limit ground movement that are inherent in the Concept Design are typical of contractor requirements that would be typically incorporated into construction contracts for rail projects. The measures inherent in the Melbourne Metro Concept Design are summarised in Section 6.4.

As a result of the risk assessment, project-specific performance requirements ('Environmental Performance Requirements') have been proposed to reduce risks and hence determine the 'Residual Risk Rating'. The Environmental Performance Requirements are outlined in the following sections of the impact assessment and collated in Table 7-1. All Environmental Performance Requirements are incorporated into the Environmental Management Framework for the project.

The risk assessment has been based upon the preliminary conceptual ground and groundwater models described in Appendix A that are a fundamental input to the Ground Movement Assessment to estimate the potential magnitudes of ground movement, its distribution, and potential effects on structures and civil infrastructure. The ground movement risk register was developed with the geotechnical risks associated with geological variability in mind and the potential for excavations to encounter unforeseen/unexpected conditions. The levels of confidence in the conceptual ground model are reflected in the current register. Ongoing geotechnical and hydrogeological investigations would serve to allow refinement of the model and increase levels of confidence in those areas identified with medium residual risk.

For further details refer to Technical Appendix B *Environmental Risk Assessment Report* (of the EES) which includes the full Risk Register, with existing performance requirements and Environmental Performance Requirements assigned to each risk.

Measures to limit ground movements that are inherent in the proposed Concept Design scheme, as described in Section 6.4, informed the initial consequence rating and initial risk rating. Where the initial risk rating was found to be medium or worse, potential risk mitigating measures were incorporated into the risk assessment and informed the residual risk rating.

Table 7-1 Risk Register for Ground Movement Impact Assessment

Impact Pathway		Precinct	Initial Risk			Residual Risk			Risk No.
Category	Potential Consequences		C	L	Initial Risk	C	L	Residual Risk	
Construction									
Construction stage excavations cause ground movement	Potential impacts on existing buildings and/or infrastructure.	All	Moderate	Likely	Medium	Minor	Possible	Low	GM001
	Damage to buildings on mixed or shallow foundations.	8 - Eastern portal	Moderate	Likely	Medium	Minor	Possible	Low	GM002
	Damage to North Yarra MainSewer on Lloyd Street.	1 – Tunnels *(western portal to Arden station)*	Moderate	Likely	Medium	Minor	Possible	Low	GM003
	Damage to rail lines resulting in disruption of services.	2 - Western portal 1 – Tunnels *(western portal to Arden station)*	Moderate	Likely	Medium	Moderate	Possible	Medium	GM004
	Damage to Essendon Flyover and/or Lloyd Street Bridges.	1 – Tunnels *(western portal to Arden station)*	Minor	Likely	Medium	Minor	Possible	Low	GM005
	Damage to Royal Women's Hospital, Victoria Comprehensive Cancer Centre, Grattan Street Pedestrian Bridge.	4 - Parkville station	Minor	Likely	Medium	Negligible	Likely	Low	GM006
	Damage to road pavement and tram lines at CityLink over crossing.	1 – Tunnels *(CBD South station to Domain Station)*	Moderate	Likely	Medium	Minor	Possible	Low	GM007
	Damage to tram lines resulting in disruption to services.	6 - CBD South station	Moderate	Likely	Medium	Moderate	Possible	Medium	GM008
	Damage to CityLink viaduct foundations compromising structural integrity.	1 – Tunnels *(western portal to Arden station)*	Minor	Likely	Medium	Minor	Possible	Low	GM009
	Damage to Princes Bridge resulting in disruption to bridge traffic or compromising structural integrity.	1 – Tunnels *(CBD South station to Domain station)*	Moderate	Likely	Medium	Minor	Possible	Low	GM010

Impact Pathway		Precinct	Initial Risk			Residual Risk			Risk No.
Category	Potential Consequences		C	L	Initial Risk	C	L	Residual Risk	
	Damage to exiting City Loop tunnels, resulting in disruption to operating rail lines.	5 - CBD North Station	Minor	Likely	Medium	Minor	Possible	Low	GM011
	Damage to CityLink Tunnels resulting in disruption to operating roads.	1 – Tunnels *(CBD South station to Domain station)*	Minor	Likely	Medium	Minor	Possible	Low	GM012
	Damage to Telstra Tunnels resulting in disruption to key infrastructure.	6 - CBD South station	Moderate	Likely	Medium	Minor	Likely	Medium	GM013
	Damage to utilities vulnerable to ground movements and integrity could be affected.	All	Minor	Likely	Medium	Minor	Possible	Low	GM014
Construction stage groundwater inflows to excavations result in ground movement (consolidation settlement)	Potential impacts on existing buildings, utilities and/or infrastructure.	All	Moderate	Likely	Medium	Minor	Possible	Low	GM015
Combined effects of excavation induced ground movement and consolidation settlement	Potential impacts on existing buildings, utilities and/or infrastructure.	All	Moderate	Likely	Medium	Minor	Possible	Low	GM016
Construction activities in/at waterway crossings (perpendicular or parallel to) causing destabilisation of Moonee Ponds Creek or Yarra River.	Local destabilisation of waterway banks and channel profile, leading to slips Increased erosive action on creek banks and bed.	1 – Tunnels *(western portal to Arden station)* *(CBD South station to Domain station)*	Minor	Possible	Low	Negligible	Unlikely	Very Low	GM017

Impact Pathway		Precinct	Initial Risk			Residual Risk			Risk No.
Category	Potential Consequences		C	L	Initial Risk	C	L	Residual Risk	
Groundwater drawdown during construction	Depressurisation of compressible sediments resulting in consolidation settlement with subsequent unacceptable impacts on structures, utilities and/or infrastructure.	2 – Tunnels (Western Portal) 1 – Tunnels (Western Portal to Arden Station) 3 - Arden Station; 1 - CBD North Station to CBD South Station; 6 - CBD South Station; 1 - CBD South Station to Domain Station	Moderate	Likely	Medium	Minor	Possible	Low	GM018
Unexpected ground conditions or unexpected ground movement	Moderate or worse impacts to existing structures and/or infrastructure.	All	Moderate	Possible	Medium	Moderate	Unlikely	Low	GM019
Tunnel construction encountering rock with greater rock mass strength than expected	May necessitate a change in construction methods in a zone of mixed geological conditions leading to increased ground movement or cause TBM to go off-line. Requirement to change construction method or repair/retool TBM could result in project delays.	2 - Western Portal; 1 - Western Portal to Arden Station; 3 - Arden Station; 1 - Arden Station to Parkville 1 - CBD South Station to Domain Station	Major	Possible	High	Moderate	Possible	Medium	GM020
Underground Excavations	Very high strength rock mass requires drilling and blasting as a method of excavation. This could result in delays in tanking of tunnels or underground excavations.	5 - CBD North; 1 - CBD North to CBD South; 6 - CBD South	Moderate	Possible	Medium	Minor	Unlikely	Low	GM021
Tunnel construction	Modelled levels of ground movement are underestimated as a consequence of unforeseen geology, groundwater conditions, surface conditions and unexpected building conditions or use of different equipment types.	All	Major	Possible	High	Moderate	Unlikely	Low	GM022

Impact Pathway		Precinct	Initial Risk			Residual Risk			Risk No.
Category	Potential Consequences		C	L	Initial Risk	C	L	Residual Risk	
Ground heave as a result of excessive face pressure by the TBMs in shallow cover areas	Unacceptable ground movement.	1- Tunnel 2- Western Portal	Major	Possible	High	Major	Unlikely	Medium	GM023
Construction and Operation									
Groundwater inflow to excavations much greater than that estimated due to interception of high permeability zones that are difficult to control.	Consolidation settlement magnitude and extents greater than that estimated resulting in moderate or worse impacts to existing structures and/or infrastructure.	All	Major	Possible	High	Moderate	Unlikely	Low	GM024
Operation									
Ongoing leakage into tunnels and underground structures during operation	Depressurisation of compressible sediments resulting in consolidation settlement with subsequent unacceptable impacts on structures, utilities and/or infrastructure.	All	Major	Possible	High	Moderate	Unlikely	Low	GM025

8 Building Impact Assessments

This section describes the project components, existing conditions, the key issues and findings of the impact assessment for the Concept Design and alternative design options (where they exist).

Buildings located within the project's Potential Zone of Influence were reviewed. The following building types were included in the building impact assessments:

- Buildings with/without basements in close proximity to Melbourne Metro tunnels or structures
- Tall buildings with/without deep foundations
- Approved developments (as at 15/12/2015)
- Heritage buildings.

A representative sample of buildings within the Potential Zone of Influence were selected for assessment with varying geological settings, construction types and proximity to the proposed Melbourne Metro works. The buildings were assessed based on the principles described in Section 5.

The estimated tensile strains due to settlement under the building footprints are assessed and compared with the Burland criteria and the expected building damage classification in accordance with Table 5-7. Preliminary Level 3 assessments were undertaken for some tall buildings such as St. Pauls Cathedral and Manchester Unity Building.

Table 8-1 to Table 8-4 summarise the building impact assessment results along with potential mitigation measures. The results are presented by precinct as they occur along the proposed alignment. Where potential impacts are estimated to be negligible or minor, no mitigation measures are suggested to further reduce the ground movement impact.

Basements were assessed for selected structures. Structural attributes such as basement depths are currently being confirmed through ongoing building inspections.

Impacts on receiver's sensitive to noise or vibration (i.e. near-by residences or buildings with sensitive equipment such as hospitals and performance centres) that are potentially at risk of impact from Melbourne Metro construction and/or operational noise and/or vibration are described in the Technical Appendix I *Noise and Vibration*.

The likely impact on buildings outside the estimated Potential Zone of Influence would be negligible. The modelled impacts on the vast majority of buildings within the Potential Zone of Influence are negligible or minor and within acceptable limits. There are relatively few instances in which the initial modelled impacts are moderate. In these instances specific mitigation measures have been identified that would achieve acceptable outcomes. There are no instances where potential impacts were found to be severe.

The regime established under the recommended Environmental Performance Requirements would ensure that measures equivalent to the identified potential mitigation measures would be implemented as necessary in the detailed design and construction phases of the project.

Table 8-1 Summary of Impact assessment results for selected buildings

Precinct	Storeys	Assumed Construction Type		Geology at Tunnel Axis	Volume Loss (%)	Maximum Settlement (mm)	Tensile Strain / Category	Estimated Impact*	Proposed mitigation measures
Precinct 2 Western Portal	1	Timber	Typical Residential	Older Volcanics	NA (cut and cover)	≤ 12	≤ 0.01	Negligible to minor impact	Impact management: Fine cracks are treated during normal decoration.
Precinct 1 Tunnels: Western Portal – Arden	2	Portal Frame	Typical Industrial	Older Volcanics	1	6	0.05	Negligible to minor impact	Impact management: Fine cracks are treated during normal decoration. Cracks up to 5 mm could be filled. Repointing may be required to external cracks to ensure water tightness.
	2	Reinforced Concrete	Typical Residential	Werribee Formation	1	12	0.03		
	1	Rendered Brick				29	0.01		
Precinct 3 Arden station	1	Portal Frame	Typical Industrial	Werribee Formation	1	30	0.05	Minor impact	Impact management: Cracks filled. Repointing may be required of external cracks to ensure weather-tightness.
Precinct 1 Tunnels: *Arden – Parkville*	3	Rendered Brick	Typical Residential	Melbourne Formation	1	9	<0.01	Negligible to minor impact	Impact management: Cracks filled. Repointing may be required of external cracks to ensure weather-tightness.
	3	Rendered Brick			0.5	26	0.02		
	1	Timber			0.5	11	<0.01		
	6 - 15	Reinforced Concrete	Tall Building		0.5	≤ 47	≤ 0.05		
Precinct 4 Parkville station	5	Reinforced Concrete	Basement carpark	Melbourne Formation	0.5	12	0.04	Minor impact	Impact management: Cracks filled. Repointing may be required of external cracks to ensure weather-tightness.
Precinct 1 Tunnels: *Parkville – CBD North*	5-35	Reinforced Concrete	Tall Building	Melbourne Formation	0.5	≤ 9	≤ 0.01	Minor impact	Impact management: Fine cracks are treated during normal decoration.
Precinct 5 *CBD North*	11	Steel Frame	Tall Building	Melbourne Formation	0.5	15	0.04	Minor impact	Impact management: Cracks filled. Repointing may be required of external cracks to ensure weather-tightness.
	5	Reinforced Concrete	Tall Building	Melbourne Formation	0.5	29	0.02		

Precinct	Storeys	Assumed Construction Type		Geology at Tunnel Axis	Volume Loss (%)	Maximum Settlement (mm)	Tensile Strain / Category	Estimated Impact*	Proposed mitigation measures
	2	Masonry	Typical Residential	Melbourne Formation	0.5	≤ 18	≤ 0.02	Minor-moderate impact	Impact management: Cracks require some opening up and patching by a mason. Repointing of external brickwork and possibly a small amount of brickwork to be replaced.
Precinct 1 Tunnels *CBD North – CBD South*	5	Steel Frame	Tall Building	Melbourne Formation	0.5	5	<0.01	Minor impact	Impact management: Fine cracks are treated during normal decoration.
	2	Reinforced Concrete	Typical Commercial	Melbourne Formation	0.5	6	<0.01		
Precinct 6 CBD South station	13	Reinforced Concrete	Tall Building	Melbourne Formation	0.5	10	0.01	Minor impact	Impact management: Cracks filled. Repointing may be required of external cracks to ensure weather-tightness.
	1	Masonry	Tall Building	Melbourne Formation	0.5	17	0.03		
Precinct 1 Tunnels *CBD South – Domain*	2	Sandstone	Church	Melbourne Formation	0.5	<5	<0.01	Negligible to minor impact	Impact management: Fine cracks are treated during normal decoration.
Precinct 7 Domain	5	Reinforced Concrete	Tall Building	Melbourne Formation	0.5	7	0.03	Minor impact	Impact management: Fine cracks are treated during normal decoration.
	22	Reinforced Concrete	Tall Building	Melbourne Formation	1	≤ 14	≤ 0.07		
Precinct 1 Tunnels: *Domain – Eastern Portal*	4	Reinforced Concrete	Church	Melbourne Formation	0.5	16	0.03	Negligible to minor impact	Impact management: Fine cracks are treated during normal decoration.
	9 - 15		Tall Building			9	0.01		
	1	Rendered Brick	Church	Melbourne Formation	0.5	4	0.01		
	2	Rendered Brick	Typical Commercial	Melbourne Formation	0.5	7	0.01		
	2	Rendered Brick	Typical Commercial	Melbourne Formation	10	13	<0.01		
	2	Masonry	Typical Residential	Brighton Group	1	35	0.07		
	3	Masonry	Typical	Brighton	1	7	0.11	Negligible to minor impact	Impact management: Cracks filled. Repointing may be

Precinct	Storeys	Assumed Construction Type		Geology at Tunnel Axis	Volume Loss (%)	Maximum Settlement (mm)	Tensile Strain / Category	Estimated Impact*	Proposed mitigation measures
			Residential	Group					required of external cracks to ensure weather-tightness.
	4	Brick	Typical Residential	Brighton Group	1	30	0.09		
	4	Rendered Brick	Typical Residential	Brighton Group	1	13	0.09		
Precinct 8 Eastern Portal	4	Brick	Typical Commercial	Brighton Group	NA (Cut & Cover and cutting widening works)	<5	Settlement is due to retaining wall deflections	Negligible to minor impact	Impact management: Fine cracks are treated during normal decoration.
	2	Reinforced Concrete	Typical Residential	Brighton Group		≤ 13			
	3	Rendered Brick	Typical Residential	Brighton Group		13			
	5	Brick	Typical Residential	Brighton Group		10			
	4	Reinforced Concrete (mixed foundations)	Typical Residential	Brighton Group		11		Moderate	As-built records to be obtained or survey required to inform detailed design assessments and determination of appropriate mitigations. Further assessment at detailed design stage with the final Melbourne Metro construction methods. Determination of further mitigation works including construction controls. Use of protective works if risk remains such as reinforcement of the ground mass beneath existing shallow footing at close proximity to project works.

Buildings that may experience combined ground movement effects due to excavation induced settlement and primary consolidation settlement are those at the following locations:

- Western portal and
- Arden station precinct.

Table 8-2 Building impacts due to combined effects

Precinct	Structure type	Estimated impact	Proposed mitigation measures
Precinct 2 Western portal	1 – 3 storey residential buildings	Minor - moderate	Additional injection wells may be installed if the groundwater response to Melbourne Metro works does not match the preliminary model results. Compensation grouting for slab on grade structures. Impact management: Cracks filled. Repointing may be required of external cracks to ensure weather-tightness.
Precinct 3 Arden station	Light industrial buildings		

Additional investigations are required to refine the geological and groundwater models at the above locations and in order to inform the detailed design stage assessments.

8.1 Early works

8.1.1 Buildings overlying realigned South Yarra Main Sewer

The Xdisp assessment assumptions included:

- The realigned South Yarra Main Sewer would be located in Brighton Group
- Internal diameter of 1.9 m and outer diameter of 2.1 m
- Depth = 16.4 m to obvert (15.35 m to axis level)
- Volume loss of 1.0 per cent and a trough width factor of 0.4 t.

Two buildings were assessed with a horizontal offset of 5 m to the realigned South Yarra Main Sewer:

- Building 1, 4m deep basement, RC structure, 10 m tall
- Building 2, at surface, masonry structure, 7 m tall.

Results for both buildings show negligible impact (tensile strains were less than 500 micro strains).

Sensitivity analysis was conducted on the above figures and found the same result of negligible impact. This is due to the size of and the depth of the realigned South Yarra Main Sewer. Being a single tunnel excavation also contributes to the estimated negligible impact. No additional risk mitigations are required beyond further assessment at detailed design stage which may be informed by additional site investigation information and the final construction scheme for the realigned South Yarra Main Sewer.

8.1.2 Demolition works

Potentially adverse impacts to existing structures may result from proposed Melbourne Metro demolition works, particularly where existing underground structure are removed or reconfigured. If not controlled, removal of underground structures would result in a change in stress in the ground and potential loss of lateral restraint to adjacent foundations or retention systems. This risk would be analysed at detailed design stage incorporating as-built information, existing structure loads and condition survey information. Potentially adverse impacts would be mitigated through designing a staged demolition plan with concurrent installation of new support (temporary or otherwise) to prevent loss of bearing capacity in existing foundation or retention systems due to potential ground movement.

8.2 Heritage Buildings

Heritage buildings have been identified as a separate grouping for reporting of the impact of tunnelling. Specifically, those listed on the Victorian Heritage Register have been reviewed.

The listing of a building on a heritage register does not, per se, signify that it is more vulnerable to damage resulting from ground movement, and would be assessed in the same way as adjacent buildings of a similar type and age. However, it is the case that, as a group, heritage buildings are likely to be less tolerant of ground movement than modern buildings. It is also possible that any repair works, should such be required are less likely to be acceptable if they change some of the fabric of the structure.

In terms of the assessment, the heritage buildings within the project's Potential Zone of Influence, where the predicted ground movements were greater than 5 mm, were reviewed. The structures listed in Table 8-3 have been included in the XDisp model to obtain a Level 2 assessment, except for cases where a more detailed analysis was conducted. This latter group included the Manchester Unity Building and St Paul's Cathedral, for which 2D PLAXIS modelling was conducted to refine the prediction of the ground movement. For Manchester Unity, this was conducted because the 12 storey steel framed building is outside the types of building that fit the data used by the XDisp for the analysis of building damage predictions. Similarly, St Paul's Cathedral is of a more complex structural form that requires an understanding of the ground movements. In both these cases, the building form was not included in the ground model, but it's loading and, in the case of the Manchester Unity Building, its basements, which lower the level of loading onto the ground, were considered. The PLAXIS modelling allowed a more specific ground model also to be used. The predicted ground movements were then assessed to categorise the predicted damage.

Table 8-3 Estimated Impacts to Selected Heritage Buildings

Precinct	VHR No.	Building Address	Storeys	Assumed Construction Type	Geology at Tunnel Axis	Volume Loss (%)	Maximum Settlement (mm)	Tensile Strain / Category	Estimated Impact	Proposed mitigation measures
	VHR H0918	Main Entrance Gates, University of Melbourne 156-292 Grattan Street, Parkville	19	Stone			<5	<0.01		
Precinct 4 Parkville	VHR H1003	Vice Chancellors House, University of Melbourne 156-292 Grattan Street, Parkville	2	Brickwork	Melbourne Formation	Station box in Melbourne Formation	<5	-	Negligible impact	Impact management: Fine cracks are treated during normal decoration.
	VHR H0919	Gate Keepers Cottage, University of Melbourne 156-292 Grattan Street, Parkville	1	Brickwork			<5	<0.01		
	VHR H1686	Tram Shelter and Terminus corner Swanston Street and Victoria Street	0	Reinforced Concrete	Melbourne Formation	0.5	7	0.02		
	VHR H0466	City Baths 420-438 Swanston Street, Melbourne	3	Brickwork	Melbourne Formation	0.5	33	0.04		
Precinct 5 CBD North station	HO1042	Building 49 RMIT University (Cyclone Woven Fence Factory) 65-77 Franklin Street, Melbourne	3	Brickwork	Melbourne Formation	0.5	7	<0.01	Negligible to minor impact	Impact management: Fine cracks are treated during normal decoration.
	VHR H1498	Storey Hall, Building 16 RMIT University 344 Swanston Street, Melbourne	8	Load bearing masonry	Melbourne Formation	0.5	24	0.04		
	VHR H1479	State Library of Victoria 304-328 Swanston Street, Melbourne	7	Load bearing masonry	Melbourne Formation	0.5	5	<0.01	Negligible	Impact management: Fine cracks are treated during normal

Precinct	VHR No.	Building Address	Storeys	Assumed Construction Type	Geology at Tunnel Axis	Volume Loss (%)	Maximum Settlement (mm)	Tensile Strain / Category	Estimated Impact	Proposed mitigation measures
Precinct 1 Tunnels: *CBD North - CBD South*										decoration.
	VHR H0455	Church Of Christ 329-333 Swanston Street Melbourne 3000	2	Rendered Brick	Melbourne Formation	0.5	5	<0.01		
	VHR H2250	Century Building 125-133 Swanston Street, Melbourne	17	Steel and reinforced concrete	Melbourne Formation	0.5	6	<0.01		
	VHR H0471	Capitol Arcade 109-117 Swanston Street, Melbourne	13	Reinforced concrete	Melbourne Formation	0.5	6	<0.01	Negligible to minor impact	Impact management: Fine cracks are treated during normal decoration.
Precinct 6 CBD South station	VHR H0001	Melbourne Town Hall and Administrative Buildings 90-130 Swanston Street, Melbourne	3	Load bearing masonry	Melbourne Formation	0.5	6	0.01		
	VHR H0411	Manchester Unity Building 91 -107 Swanston Street, Melbourne	19	Concrete encased steel frame and terracotta cladding	Melbourne Formation	0.5	6	<0.01		
	VHR H2119	Nicolas Building, 31-41 Swanston Street, Melbourne	13	Steel frame	Melbourne Formation	0.5	13	0.03		
	VHR H0018	St Paul's Cathedral 202 Flinders Street, Melbourne	1	Load bearing masonry	Melbourne Formation	0.5	17	0.03		
	VHR	Young and Jackson Hotel	4	Load bearing	Melbourne	0.5	19	0.04		

Precinct	VHR No.	Building Address	Storeys	Assumed Construction Type	Geology at Tunnel Axis	Volume Loss (%)	Maximum Settlement (mm)	Tensile Strain / Category	Estimated Impact	Proposed mitigation measures
	H0708	1-7 Swanston Street, Melbourne		masonry	Formation					
	VHR H1083	Flinders Street Railway Station Dome 207-361 Flinders Street, Melbourne	4	Masonry	Melbourne Formation	0.5	4	<0.01		
Precinct 1 Tunnels: *CBD South – Domain*	VHR H1374	South African Soldiers Memorial, Domain 29A Albert Road, South Melbourne	0	Stone	To be temporarily removed or relocated during construction					
Precinct 7 Domain	VHR H0019	Melbourne Grammar School 321-369 St Kilda Road, Melbourne	4	Rendered Brick	Melbourne Formation	0.5	<5	<0.01	Negligible	-
	VHR H0668	Royce Hotel 375-385 St Kilda Road, Melbourne	6	Masonry			<5	<0.01	Outside potential zone of influence	
Precinct 8 Eastern Portal	HO447	Franklyn House Flats 137 Osborne Street, South Yarra	2	Brick	Brighton Group	1	42	0.1	Minor impact	0.5% volume loss control to reduce magnitudes of ground movement. Impact management: Cracks filled. Repointing may be required of external cracks to ensure weather-tightness.
Precinct 8 Eastern Portal	VHR H0210	Former South Yarra Post Office 162 Toorak Road, South Yarra	4	Load bearing masonry	Brighton Group	N/A	Negligible	-	Outside potential zone of influence.	

8.3 Approved Developments

Table 8-4 summarises the results from assessment of selected approved developments within the Potential Zone of Influence.

Table 8-4 Summary of results from assessment of selected approved developments

Precinct	Storeys	Construction Type	Geology at Tunnel Axis	Volume Loss (%)	Maximum Settlement (mm)	Tensile Strain / Category	Estimated Impact	Proposed mitigations measures
Precinct 1 Tunnels: *Arden to Parkville*	4	Reinforced Concrete	Older Volcanics	0.5	26	0.03	Minor to Moderate	Install flat jacks under affected columns which would be used to jack up the building as settlement occurred. To be designed and installed as part of new development works. Appropriate requirements to be agreed with development stakeholders and are subject to more detailed assessment.
	4	Reinforced Concrete	Older Volcanics	0.5	26	0.03	Negligible to minor impact	Impact management: Cracks filled. Repointing may be required of external cracks to ensure weather-tightness.
	8 - 10	Reinforced Concrete	Melbourne Formation	0.5	11	≤ 0.02		
	14	Precast Concrete	Melbourne Formation	0.5	7	0.06		
Precinct 1 Tunnels: *Parkville to CBD North*	10 - 35	Reinforced Concrete	Melbourne Formation	0.5	6	≤ 0.01	Negligible to minor impact	Impact management: Fine cracks are treated during normal decoration.
Precinct 5 CBD North station	10	Reinforced Concrete	Melbourne Formation	0.5	29	0.04	Minor impact	Impact management: Cracks filled. Repointing may be required of external cracks to ensure weather-tightness.
Precinct 6 CBD South station	3	Precast into frame	Melbourne Formation	0.5	5	<0.01	Negligible to minor impact	Impact management: Fine cracks are treated during normal decoration.
	6	Reinforced Concrete	Melbourne Formation	1	3	0.06		
	6	Reinforced Concrete	Melbourne Formation	0.5	8	0.01		

Precinct	Storeys	Construction Type	Geology at Tunnel Axis	Volume Loss (%)	Maximum Settlement (mm)	Tensile Strain / Category	Estimated Impact	Proposed mitigations measures
Precinct 1 Tunnels: ***Domain to Eastern Portal***	4	Reinforced Concrete	Melbourne Formation	0.5	10	0.02	Worst case potential structural impact is likely to be moderate	The impact could be mitigated by controlling the tunnel volume loss to 0.5% which would most likely have a minor structural impact. If the tunnel volume loss is restricted to 0.5%, only minor cosmetic repairs may be required.
	5	Rendered Brick	Brighton Group	1	4	<0.01	Negligible to minor impact	Impact management: Fine cracks are treated during normal decoration.

9 Civil Infrastructure Impact Assessments

There are numerous locations along the Melbourne Metro alignment where ground movement induced by the proposed project works may affect operating transport infrastructure due to settlement under or adjacent to operating rail, road or tram lines or the structural integrity or serviceability of other major structures. Potential ground movement impacts were assessed for existing key civil infrastructure listed in Table 9-1 and Table 9-2. Potentially impacted rail lines are listed in Table 9-3 and potentially impacted tram line sites are listed in Table 9-4.

The modelling indicates that the likely impacts on the majority of assets would be within acceptable limits. Specific mitigation measures have been identified in respect of the limited number of assets where this is not the case and which would achieve acceptable. The Environmental Performance Requirements prescribe comprehensive monitoring and maintenance regimes, to ensure that if unacceptable impacts do eventuate, they would be quickly identified and remedied.

Table 9-1 Summary of Impacts on Selected Key Civil Infrastructure included in preliminary assessments

Precinct	Key Civil Infrastructure	Basis of Impact Assessment	Magnitude of Settlement (mm)	Max. Slope	Estimated Impact	Proposed mitigation measures
Precinct 1 Tunnels: ***Western Portal to Arden***	Lloyd Street Bridge (north)	Preliminary Level 2: inspection of estimated settlement contours	14	1:700	Minor impact to abutment retaining walls on shallow foundations	Very close monitoring of ground movements as TBM excavations advance towards these bridges. Limit volume loss to 0.5 per cent.
	Lloyd Street Bridge (south)		11	1:780		
	Essendon Flyover North abutment		13	<1:1000		
	Essendon Flyover South abutment		15	<1:1000		
	West Melbourne Terminal Station	Preliminary Level 3: PLAXIS Analyses	See respective sub headings that follow		Minor impact	Subject to confirmation at detailed design. Strict control on TBM operation, limit volume loss to 0.5 per cent.
	CityLink Viaduct				Moderate impact	Strict control on TBM operation, limit volume loss to 0.5 per cent.
Precinct 1 Tunnels: ***Arden to Parkville***	Grattan Street Tunnel				Negligible to minor	Inspect post construction and

Precinct	Key Civil Infrastructure	Basis of Impact Assessment	Magnitude of Settlement (mm)	Max. Slope	Estimated Impact	Proposed mitigation measures
						undertake minor repairs as required.
Precinct 5 ***CBD North station***	City Loop (MURL) Crossing				Minor	Condition survey and monitoring of the City Loop tunnels prior to and during the excavation and construction of Melbourne Metro tunnels and CBD North cavern would be required. An appropriate management plan is required that documents acceptable stakeholder criteria for potential cracking and a predetermined program for undertaking any potentially required minor repairs. After construction of the Melbourne Metro scheme is completed, the City Loop tunnels should be inspected and if required, new cracks sealed during a regular maintenance closure.
Precinct 1 Tunnels: ***CBD North to CBD South***	Telstra Tunnel Lonsdale Street	Preliminary Level 3: PLAXIS Analyses of ground deformations	Covered in Utilities Section & Section 9.3 Existing Tunnels			
	Telstra Tunnel Little Bourke Street					
	Telstra Tunnel Little Collins Street to Flinders Street					
	Telstra Tunnel Little Collins Street to Flinders Street					
Precinct 1 Tunnels: ***CBD South to Domain***	Swanston Street Bridge (between Federation Square and Flinders Station)	Preliminary Level 2: inspection of estimated settlement contours	14	<1:1000	Negligible	-

Precinct	Key Civil Infrastructure	Basis of Impact Assessment	Magnitude of Settlement (mm)	Max. Slope	Estimated Impact	Proposed mitigation measures
	Princes Bridge	Preliminary Level 3: PLAXIS Analyses	See Section 9.7 below		Moderate	Completion of current site investigation works Strict control on TBM operation Further analysis at detailed design stage 0.5% volume loss
	Alexandra Avenue Retaining Walls	Preliminary Level 2: inspection of estimated settlement contours	33	1:420	Minor impact	Strict control on TBM operation Further analysis at detailed design with the benefit of preconstruction settlement monitoring data
	St Kilda Road Over bridge		1	<1:1000	Negligible impact	Outside Potential Zone of Influence relating to excavation induced settlement. Estimated consolidation settlement <10 mm. Structure is piled
	Existing CityLink tunnels	Preliminary Level 3: PLAXIS Analyses	See Section 9.3.2 and 9.3.3 below		Negligible to Minor	Condition survey and monitoring of the CityLink tunnels prior to and during the excavation and construction of Melbourne Metro tunnels would be required. An appropriate management plan would be required that documents acceptable stakeholder criteria for potential cracking and a predetermined program for undertaking any potentially required minor repairs. After construction of the Melbourne Metro tunnels is completed, the CityLink tunnels should be inspected and if required, new cracks sealed during a regular maintenance

Precinct	Key Civil Infrastructure	Basis of Impact Assessment	Magnitude of Settlement (mm)	Max. Slope	Estimated Impact	Proposed mitigation measures
						closure

Civil Infrastructure may experience combined ground movement effects due to excavation induced settlement and primary consolidation settlement at the following locations:

- Western portal and
- Arden precinct.

Table 9-2 Civil Infrastructure impacts due to combined effects

Precinct	Infrastructure type	Estimated impact	Proposed mitigation measures
Precinct 2 Western Portal	Roads	Minor - moderate	Additional injection wells may be installed if the groundwater response to Melbourne Metro works does not match the preliminary model results.
	Rail		
	Utilities		
Precinct 3 Arden	Roads		
	Utilities - Laurens Street sewer		The sewer might be affected by the consolidation settlement, but as it is likely to be on the boundary of the settlement zone, further work would be required to define the movements. Based on a conservative view of the extent of the consolidation zone, the effects would be minor (slope 1/1000). This occurs further south than the tunnel excavation induced settlements and therefore, there are no combined effects.

Additional investigations are required to refine the geological and groundwater models at the above locations and in order to inform the detailed design stage assessments.

9.1 Rail

Settlements assessment results at existing rail infrastructure are summarised in Table 9-3.

Table 9-3 Ground Deformation results at existing rail

Precinct	Rail Line	Geology	Max Settlement (mm)	Max Slope	Estimated Impact	Proposed mitigation measures
Precinct 2 Western Portal	Sunbury Werribee Regional Rail Link	Fill Coode Island Silt Older Volcanics	40	1:280	Moderate	Impact Management: Comprehensive monitoring system Monitor and re-tamp existing lines as required. Further assessment at detailed design stage with consideration for additional investigation

Precinct	Rail Line	Geology	Max Settlement (mm)	Max Slope	Estimated Impact	Proposed mitigation measures
						data, the proposed design and construction methods and the Stakeholder Acceptability Criteria.
Precinct 1 Tunnels: *Western Portal to Arden*	Sunbury Werribee Regional Rail Link	Fill Older Volcanics Werribee Formation	22	1:970	Negligible	Impact Management: Comprehensive monitoring system Monitor and re-tamp existing lines as required.
	Sunbury Werribee Regional Rail Link Craigieburn	Fill Coode Island Silt Pleistocene Alluvium Werribee Formation	24	1:750	Negligible	
	Upfield	Fill Coode Island Silt Fishermans Bend Silt	24	1:750	Negligible	
Precinct 1 *Tunnels: CBD South to Domain*	Flinders Street Station lines	Jolimont Clay Newer Volcanics Melbourne Formation	20	<1:1000	Negligible	
Precinct 8 Eastern Portal	Sandringham	Brighton Group	10	1:870	Negligible	Lines to be rebuilt as part of Melbourne Metro works. Monitor and re-tamp while operational
	Dandenong Frankston	Brighton Group	10	1:340	Minor	

Monitoring and re-tamping:

- Track work on the ground would need to be re-tamped as the excavations advance and as consolidation settlement progresses
- There would be potential for settlement due to consolidation settlement associated with groundwater drawdown around the proposed western portal Dive Structure site to Moonee Ponds Creek channel where compressible soils exist. At present there is limited geotechnical information in this area in particular extent of Coode Island silt. The ground movement monitoring program should extend into the operational phase of the project until any groundwater drawdowns have recovered or primary consolidation settlement has conclusively stabilised.

9.2 Tram Lines and Road Pavements

Based on the settlement contour drawings and results tabulated in Table 9-4 below, the potential impacts to operating tram lines and road pavements are considered negligible except at the sites:

- Immediately above each of the proposed cavern stations and
- Adjacent to the CityLink Over Crossing.

There are many streets and roads overlying Melbourne Metro tunnels or located within the project potential zone of influence. Typically, it is accepted that flexible pavements can take more deformation than structures. The risk of impacting serviceability is considered more probable than the risk of structural damage to the roads or tram lines. Serviceability criteria can be measured in terms of poor performance due to excessive change in gradient, cross fall and/or road drainage inefficiency.

The magnitude of estimated settlement is generally up to 30 mm along the project alignment with a relatively wide trough, suggesting negligible distortion to the tram and road pavements. The pavements would require careful monitoring during construction stage as underground excavations are in progress. The critical stage in terms of potentially adverse impacts to pavements may be during construction stage maximum displacement has occurred immediately above the advancing cavern face but relative to an undisturbed zone 10 m from the fully supported zone, a slope of 30 mm in 6 m of greater (1 in 200) has developed.

As described in Section 7, a moderate impact poses a risk of potential superficial damage, which is unlikely to have significant effect to the structure or performance of the tram line or pavement.

Table 9-4 Ground Deformation Estimates at selected locations across the proposed alignment

Precinct	Approximate Geographic Location	Geology at Tunnel Axis	Volume Loss (%)	Maximum Settlement (mm)	Maximum Slope	Impact	Proposed mitigation measures
Precinct 1 Tunnels: *Arden to Parkville*	Arden Street Abbotsford Street junction	Older Volcanics	0.5	19	1:750	Negligible to minor impact	
Precinct 4 Parkville	Flemington Road Grattan Street junction	Melbourne Formation	0.5	11	<1:1000	Negligible to minor impact	
	Royal Parade Grattan Street junction VHR H2198	Melbourne Formation	0.5	11	<1:1000	Negligible to minor impact	
Precinct 1 Tunnels: *Parkville to CBD North*	Swanston Street Lincoln Street junction	Melbourne Formation	0.5	6	<1:1000	Negligible impact	-
	Swanston Street Queensberry Street junction	Melbourne Formation	0.5	6	<1:1000	Negligible impact	
Precinct 5 CBD North station	Swanston Street Victoria Street junction (north bound and west bound lines)	Melbourne Formation	0.5	6	<1:1000	Negligible to minor impact	
	Swanston Street CBD North cavern excavation	Melbourne Formation	0.5	40	1:300	Moderate	Comprehensive real time monitoring strategy
	Swanston Street La Trobe Street junction	Melbourne Formation	0.5	20	<1:1000	Negligible	
Precinct 1 Tunnels: *CBD North to CBD South*	Swanston Street City Baths to Princes Bridge, Bourke Street junction	Melbourne Formation	0.5	8	<1:1000	Negligible	-
Precinct 6 CBD South station	Swanston Street Collins Street junction	Melbourne Formation	0.5	22	1:850	Negligible	
	Swanston Street	Melbourne	0.5	40 (inferred)	1:300	Moderate	Comprehensive real

Precinct	Approximate Geographic Location	Geology at Tunnel Axis	Volume Loss (%)	Maximum Settlement (mm)	Maximum Slope	Impact	Proposed mitigation measures
	CBD South cavern excavation	Formation					time monitoring strategy
	Swanston Street Flinders Street junction	Brighton Group	1.0	26	1:990	Negligible	-
Precinct 1 Tunnels: ***CBD South to Domain***	Alexandra Avenue	Coode Island Silt	1.0	35	1:250	Moderate	Comprehensive real time monitoring strategy
	St Kilda Road and Wadey Street junction (near CityLink crossing) to Domain station site	Brighton Group	1.0	35	1:230	Moderate	Ground Improvement, Comprehensive real time monitoring strategy
	St Kilda Road and Park Street junction	Melbourne Formation	0.5	14	<1:1000	Negligible	-
Precinct 7 Domain	Domain station site to Toorak Road and St Kilda Road junction	Melbourne Formation	1.0	32	1:340	Negligible	
Precinct 1 Tunnels: ***Domain to Eastern Portal***	Toorak Road and Marne Street junction to Toorak Road and Avoca Street junction	Melbourne Formation	0.5	7	<1:1000	Negligible	

The completed preliminary assessments have found no adverse structural effects are anticipated in the existing road pavements outside the proposed construction site footprints excepting the cavern and CityLink locations noted above. Potential serviceability impacts would be managed through establishing an appropriate monitoring program by the contractor and where required, appropriate maintenance program.

Discussions with stakeholders are required to determine appropriate acceptability criteria for the respective assets.

Tighter control may be required on the adopted cavern sequencing in the vicinity of the cavern structures.

Compensation grouting may be a suitable response measure to rectify any potential unacceptable tram slab movements, though the preliminary assessment findings indicate this would not be required. Impact management measures such as pavement repairs or resurfacing during the construction program may be required to be incorporated into the construction plan, subject to stakeholder agreement.

9.3 Existing Tunnels

9.3.1 City Loop Crossing

An investigation of the interaction between Melbourne Metro cavern and station entrance shafts and the proposed Melbourne Metro mined tunnels with the City Loop assets comprising Melbourne Central Station and the four running tunnels was undertaken using numerical modelling. The Melbourne Metro mined tunnels would pass beneath the City Loop tunnels, hence the lower 2 City Loop tunnels (Burnley Loop and Northern Loop) are most likely to be affected.

As the excavation for the CBD North station cavern, entrance shafts and mined tunnels would alter the ground stress regime, it is expected that the lining of the City Loop tunnels might respond by opening existing cracks or creating new cracks. This has been confirmed by the initial results from the analyses. Cracking of this type is not expected to be of concern for the structural capacity of the lining, nor is it expected to lead to spalling (flaking or chipping of concrete), given the low stress levels.

It is anticipated that the effects of the excavation for the CBD North station and mined tunnels would be managed by monitoring and minor sealing of cracks after the works, if proven necessary.

Further modelling and structural analysis is required at detailed design stage with refined rock mass and lining modelling inputs to confirm assumptions and develop a more representative model. However, this work should be done after the additional ground information is available from the planned geotechnical investigations.

The options listed in Table 9 – 5 are potential mitigation measures to address the impacts of the station cavern and mined tunnel excavation and construction on the existing City Loop tunnel linings. Note further investigation and analysis would be required to assess feasibility of each option in terms of engineering, economics and program.

Table 9-5 Potential mitigation measures for impacts at City Loop crossing

Action	Advantage	Disadvantage	Risk / Hazard	Comment
Mitigation measures in Melbourne Metro such as staging works and installation of compensating ground anchors	Designed to limit any effects on the tunnels. Minimal interference with City Loop tunnels and operators.	Considerable complexity added to CBD North construction in an area where surface disruption is already required to be minimised.	Tunnels might still be subjected to levels of ground movement that lead to cracking (very stiff tunnels).	Unlikely to be practical, given proximity to tunnels.
Install strengthening measures within the City Loop tunnels	Designed to protect lining from changes in loading.	Minimal space in City Loop tunnels for additional structure. Works would be restricted to tunnel closures.	Difficult to install within available space (if any) and would require modifying existing services, walkways and track bed (double sleepers).	Unlikely to be practical or effective, given the need to limit movements to very small deflections and restricted space.
Extensive monitoring of CBD North station and City Loop tunnels. Repair any cracks developed after construction	Only applied if required. Could be programed to install monitoring instrumentation and undertake repairs during available tunnel maintenance occupations.	Would require detailed inspection of City Loop tunnels before and after construction of CBD North station.	Might become works to remedy existing cracks in the tunnel. Might be less acceptable to tunnel owners than active approaches.	Expected to be the most practical and effective means to address the risk of cracking.

Figure 9-1 shows the finite element model of the CBD North shaft and cavern alongside the existing City Loop assets.

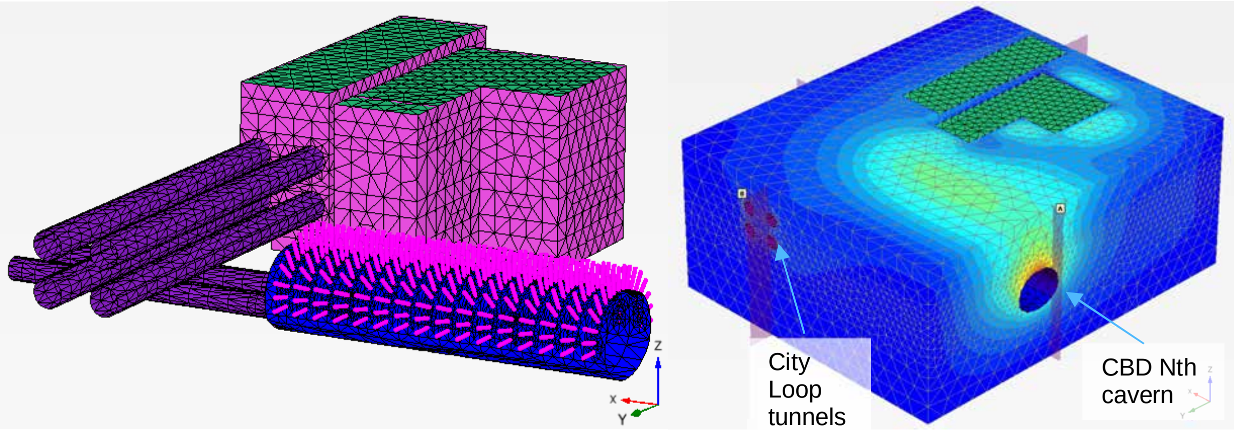

Figure 9-1 Preliminary model of City Loop tunnels and Melbourne Central Station relative to proposed works

9.3.2 CityLink Tunnels Over Crossing

A preliminary assessment of potential impacts on the existing CityLink tunnels due to the construction of the Melbourne Metro tunnels passing over the existing CityLink tunnels was carried out. Finite element models were created to model the effects of unloading on the existing CityLink Burnley and Domain tunnels due to incremental excavation of the Melbourne Metro tunnels.

The load case modelled was that where the CityLink primary lining is carrying the ground loads and the secondary lining is unstressed. This is the critical load case for impacts to the secondary lining as there is no hoop compression within the lining to increase its bending capacity.

The preliminary results suggest that the unloading due to Melbourne Metro construction would induce tension in the lining. However, the tensile stress induced is within the tensile capacity of the plain concrete lining and should be considered acceptable. Though it is possible that existing cracks in the lining, if present, might be opened further. As the linings are unreinforced, cracks do not pose any structural capacity or durability risk and are instead an aesthetic consideration.

Further assessment is required to investigate further the extent and nature of potential serviceability impacts.

It is recommended that monitoring of the existing tunnels is undertaken during construction to verify the design predictions. A monitoring scheme and appropriate management plan must be in place prior to commencement of Melbourne Metro works at this crossing site. There is the potential that post or during Melbourne Metro works, repairs such as crack infilling might be required in the existing CityLink tunnels.

Suitable mitigation measures would be developed in consultation with TransUrban.

An estimated impact response measure may be to arrange post construction inspections and if required, undertake sealing of new cracks during a regular maintenance closure.

The assessment results from modelling undertaken to date are preliminary in nature and indicative only. As with the City Loop tunnels, more detailed modelling and further structural analysis, based on the investigation results, would be required to provide a more rigorous assessment.

Surface settlement where the Melbourne Metro tunnels pass over the CityLink tunnels with low cover to the parkland surface above is expected to be in the order of approximately 40 mm, assuming face control is maintained. This magnitude of settlement within the parkland is not expected to be detrimental to its use. However, the need to temporarily monitor or support any monuments within the settlement zone during construction should be considered. Ground treatment might be required to mitigate the risk of ground movement, or in the worst case, formation of sinkholes in the parkland during construction, for Melbourne Metro alignment going over the CityLink tunnels. The potential area requiring ground treatment is shown in Figure 9-2.

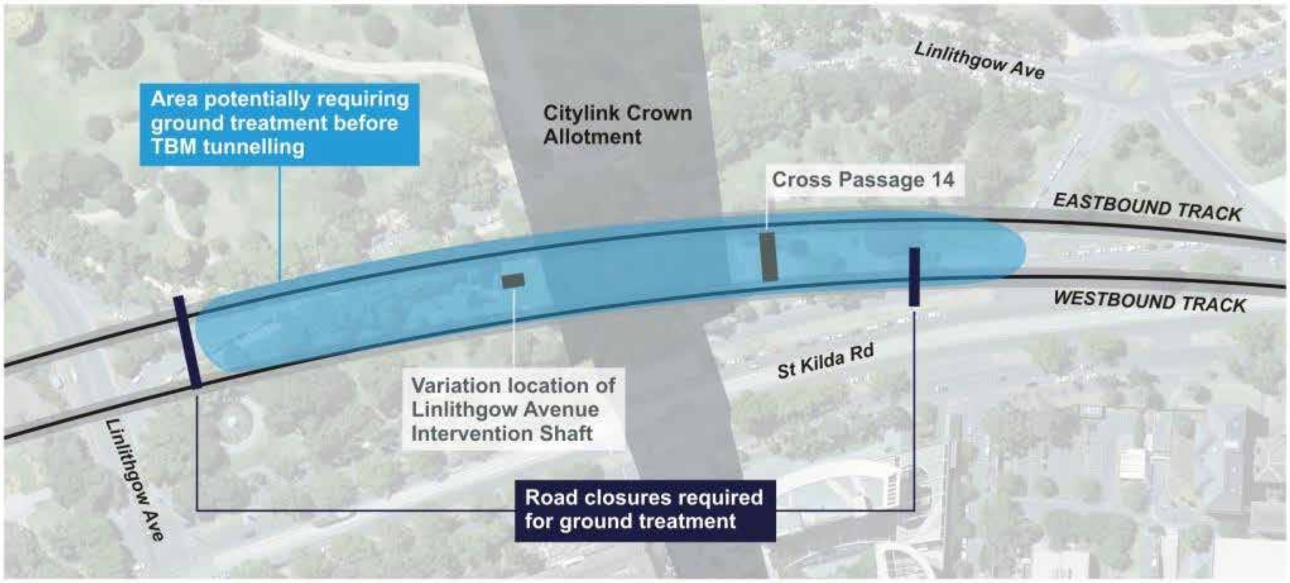

Figure 9-2 Area potentially requiring ground treatment at the CityLink crossing

9.3.3 CityLink under Crossing Option

Preliminary impact assessment results for the Under Crossing Option were found to be equivalent to those for the Over Crossing Option.

The change in stress in the existing concrete lining caused by Melbourne Metro excavation was found to be within the tensile capacity of the plain concrete lining, however it is possible that existing cracks in the lining, if present, might be opened further. As the linings are unreinforced concrete, cracks would not pose a structural capacity or durability risk, however they are an aesthetic consideration.

Further assessment is required to investigate further the extent and nature of potential serviceability impacts.

An estimated impact response measure may be to arrange post construction inspections and if required, undertake sealing of new cracks during a regular maintenance closure.

As with the over alignment, condition survey and monitoring of the CityLink tunnels prior to and during the excavation and construction of Melbourne Metro tunnels would be required. An appropriate management plan is required that documents cracking and a predetermined program for undertaking any potentially required minor repairs. After construction of the Melbourne Metro tunnels is completed, the CityLink tunnels should be inspected and if required, new cracks sealed during a regular maintenance closure.

It is noted that there would be no requirement for ground improvement to mitigate the risk of adverse ground movement, or in the worst case, formation of sinkholes in the parkland.

9.3.4 Telstra Tunnels

Preliminary assessment of ground movements and deformations indicate there would be negligible impacts to the brick arch and cast in place concrete invert to the existing tunnels except immediately above the proposed CBD South station cavern excavation where a moderate impact was indicated. Further assessment is required in the detailed design stage, including procurement of more detailed information on the as-constructed lining and a detailed inspection and assessment of the lining condition.

Following the condition survey, refinement of the modelling should be undertaken to account for the actual Telstra Tunnel and utility construction and condition. Subsequent to refinement of the modelling, a plan of protective measures (strengthening works) should be established in consultation with the stakeholder to reinforce the tunnels where potential for unacceptable lining deformation or risk to tunnel or utility operation due to Melbourne Metro works is identified.

9.3.5 Grattan Street Services Tunnel

Results of potential impact assessment to the Grattan Street services tunnel running across the alignment between the Victorian Comprehensive Cancer Centre and the Royal Women's Hospital are provided in Table 9-6.

It is estimated that the deformations, associated increases in the ground pressures at the Royal Women's Hospital and Victorian Comprehensive Cancer Centre and the subsequent increase in internal forces in the outer wall of those structures would be negligible. Similarly, the potential impacts on the tunnel are estimated to be negligible but potential for minor impacts may exist at the tunnel connection to the existing buildings which would need to be examined in detailed at detailed design stage.

Inspection and any necessary repairs to footpaths and road pavement resurfacing may be undertaken on completion of works in this precinct.

9.4 Lloyd Street Bridges and Essendon Flyover

Based on the preliminary settlement contour drawings the potential impacts are assessed to be negligible to minor for the bridges at Lloyd Street and the Essendon Flyover. Based on the available information, these bridges are primarily on spread footings with some piles supporting the approach slabs on the Lloyd Street Bridge.

For a single span bridge, settlement and differential settlement would not cause structural distress to the superstructure but might affect bridge bearing and joint performance. There are also limits to settlement to ensure a smooth ride.

Further impact assessments would be undertaken at detailed design stage by a bridge engineer who would determine the amount of settlement that the bridge can tolerate, determine the actual bearing stress and compare the corresponding settlement determined from the chart to the tolerable settlement.

As an alternative the bridge engineer may determine the amount of settlement that the bridge can tolerate, determine the maximum bearing stress from the chart for a given settlement and compare the actual bearing stress to the maximum.

A bridge engineer must evaluate whether the existing structure can handle the estimated horizontal and vertical movement in accordance with current Bridge Design Guidelines. The proposed criterion is to limit the maximum settlement to 0.25 per cent or 1 in 400 for simple span bridges.

Ground movement and associated impacts may be minimised by ensuring close control of TBM operation and limiting volume losses to 0.5 per cent and implementing a ground movement monitoring program.

9.5 West Melbourne Terminal Station

Preliminary assessment of the effects of the tunnelling on a raked pile foundation in the south east corner West Melbourne Terminal Station found potential impacts might be minor. The induced pile deformations, moments and shear forces are expected to be structurally acceptable and within the existing structural capacities. It is inferred that substantially structural capacity might remain in the pile system which would suggest the impacts due to TBM tunnelling might be negligible. Further Level 3 assessment is required incorporating the as-built information and design loads.

Ground movement and associated impacts may be minimised by ensuring close control of TBM operation and limiting volume losses to 0.5 per cent.

9.6 CityLink Viaduct

Preliminary assessment of the effects of the tunnelling on the CityLink Viaduct foundations found potential impacts might be minor to moderate. Though the induced pile lateral deformations, moments and shear forces are expected to be structurally acceptable and within the existing structural capacities. A potential increase of 10 per cent in loading within some of the viaduct pile foundations could require installation of new piles to compensate for any reduced capacity in the pile groups due to the effects of close proximity Melbourne Metro tunnelling.

Further Level 3 assessment is required at detailed design stage incorporating the as-built information and design loads.

Ground movement and associated impacts may be minimised by ensuring close control of TBM operation and limiting volume losses to 0.5 per cent and monitoring of the ground response to tunnel excavation as the TBM approaches the viaduct foundations. The ground movement response strategy would be amended, if required.

Ground improvement may be adopted to minimise movement of the ground mass surrounding the tunnel excavations.

Modification or strengthening of the existing foundations may be adopted if other mitigating measures are found through detailed design assessment, to not adequately mitigate risks of adverse impacts.

9.7 Princes Bridge

An investigation of the impacts to the existing Princes Bridge spanning the Yarra River due to the ground movement induced by TBM tunnelling has been undertaken. The Princes Bridge site was originally selected because of the basalt in this section of the river, providing a suitable founding for both the earlier single large arch, and the current structure. The river was widened during the construction of the current Princes Bridge, removing and levelling the basalt layer on which the bridge is founded. The current bridge, and the earlier profiles of the rock base and the surface profile are shown in Figure 9-3 .

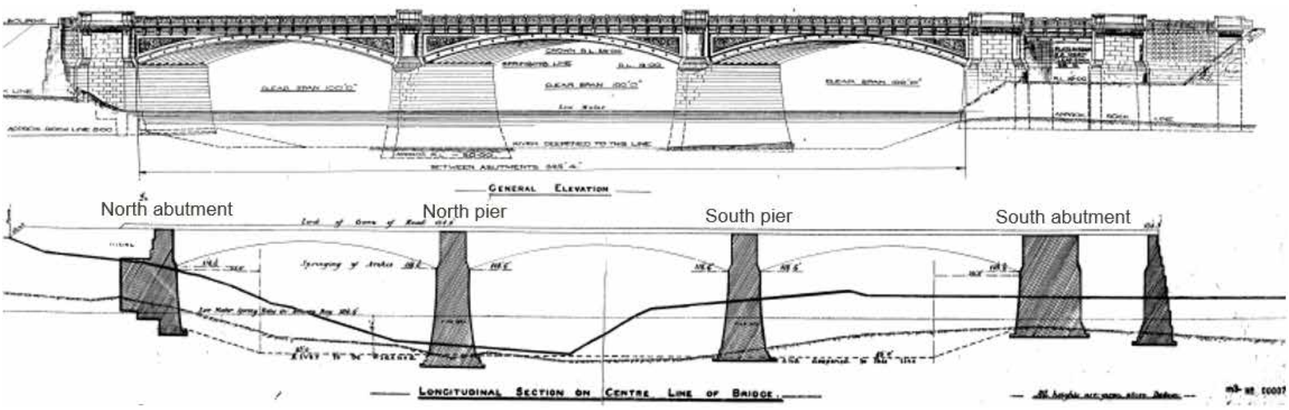

Figure 9-3 General Elevation and Longitudinal Section through Princes Bridge

PLAXIS 2D modelling was undertaken to represent TBM excavation and determine the ground movement impacts beneath the bridge abutments and piers. A number of conservative assumptions were adopted for this preliminary assessment as follows:

- The 2D models represent the bridge abutments and piers as a stiff block which continues infinitely into the page. In reality the abutments/piers are discrete structures
- Due to the limited geotechnical investigation data available at the bridge site at the time of preparation of this report, conservative parameters have been adopted for the basalt ground mass that the bridge is founded on.

The results of the modelling found up to 50 mm vertical displacement might be encountered beneath the southern and northern bridge piers (based on 1 per cent volume loss). However it should be noted that the ‘green field’ (Level 2) assessment of ground movement, predicted settlements in the order of 30 mm beneath the bridge piers.

The results found minimal ground movement beneath the existing bridge abutments.

Considering the conservative assumptions adopted in the preliminary assessment, the finding is for minor impact to the bridge.

The mitigation measures required to be adopted to prevent damage to the bridge include completion of current site investigation works, further analysis at detailed design stage and very strict control on TBM operation when tunnelling beneath the bridge. Tunnelling works would have to be planned to ensure continuous operation of TBM tunnelling in this portion of the works, without stoppages, unless critical events intervene. Ground movement and associated impacts may be minimised by ensuring close control of TBM operation and limiting volume losses to 0.5 per cent.

Ground improvement might be required to reduce deformations in the ground mass between the tunnel excavations and the bridge foundations.

Alignment optimisation may be revisited pending further assessment results with consideration for any resultant impacts on the Holocene aquifer, south of the river crossing or interactions with existing footings in the vicinity of Federation Square.

9.8 Utilities

Potential impacts on selected utilities are described in Table 9-6.

Assessment is required by service providers to identify potentially sensitive or aged utilities of high importance. In some instances CCTV or other inspection methods would be employed to assess the pre-construction condition. Along with detailed as-constructed information on the utilities, this information would be incorporated into detailed design stage assessments.

Replacement or relining of vulnerable assets might be required if shown to be required through the detailed design assessments.

Table 9-6 Estimated impacts on selected Existing Utilities

Precinct	Utility	Construction Type	Assumed depth below surface^	Geology at Tunnel Axis	Volume Loss (%)	Maximum Settlement (mm)	Maximum Slope	Estimated Impact	Proposed Mitigation Measures
	HP Gas Main - Ormond Street	Steel Welded pipe	2 m below ground	Older Volcanics	1	<5	<1:1000		
Precinct 2 Western Portal	HP Gas Main - Childers Street	Steel Welded pipe	2 m below ground	Older Volcanics	1	<5	<1:1000	Negligible	-
	HP Gas Main - McLennan Drive	Steel Welded pipe	2 m below ground	Older Volcanics	1	33	<1:1000		
	North Yarra Main Sewer	Brick	As-built level (12.774 m)	Werribee Formation	1	23	1:220	Moderate	Potential requirement to reline the sewer Condition assessment using CCTV or other methods; Further assessment at detailed design stage
Precinct 1 Tunnels: *Western Portal to Arden*	Pylon near West Melbourne Terminal Station	Transmission Tower	vertical pile group	Fisherman's Bend Silt	1	19	1:750	Minor, Induced pile deformations, moments and shear forces all expected to be structurally acceptable	Subject to confirmation at detailed design
	Electrical conduits - Moonee Ponds Creek Crossing	Conduit	As-built level (16.273 m)	Fisherman's Bend Silt	1	23	1:400	Minor	-
Precinct 3 Arden station	Lauren Street Sewer	Vitrified clay pipe	As-built level (3.995 m)	Werribee Formation	1	40	1:710	Minor	-
Precinct 1 Tunnels: *Arden to Parkville*	Flemington Road – Sewer	Vitrified clay pipe	As-built level (3.409 m)	Melbourne Formation	0.5	11	<1:1000	Negligible	-

Precinct	Utility	Construction Type	Assumed depth below surface^	Geology at Tunnel Axis	Volume Loss (%)	Maximum Settlement (mm)	Maximum Slope	Estimated Impact	Proposed Mitigation Measures
	Flemington Road - Other Typical Service	Conduit	As-built level (2.76 m)	Melbourne Formation	0.5	11	<1:1000		
	Flemington Road – Drainage	Brick Barrel	2 m below ground	Melbourne Formation	0.5	11	<1:1000		
	Grattan Street Services Tunnel - VCCC to RMH	Concrete	2 m below ground	Melbourne Formation	0.5	11	<1:1000		
Precinct 4 Parkville station	Royal Parade	Concrete	2 m below ground	Melbourne Formation	0.5	10	<1:1000	Negligible	-
Precinct 1 Tunnels: *Parkville to CBD North*	Queensberry Street - Other Typical Service	Conduit	As-built level (13.717 m)	Melbourne Formation	0.5	6	<1:1000	Negligible	-
	Queensberry Street – Drainage	Brick Barrel	2 m below ground	Melbourne Formation	0.5	6	<1:1000	Negligible	
	Queensberry Street - Sewer	OPVC fold pipe	As-built level (17.773 m)	Melbourne Formation	0.5	6	<1:1000		
	Swanston Street - Typical Service	Concrete	2 m below ground	Melbourne Formation	0.5	<5	<1:1000		
Precinct 5 CBD North station	City Baths - Oil filled electrical cables	Oil feed	2 m below ground	Melbourne Formation	0.5	19	<1:1000	Negligible	-
Precinct 1 Tunnels: *CBD North to*	Utility Tunnel: Lonsdale Street Crossing			Melbourne Formation	0.5	7	<1:1000	Moderate	Further assessment is required in the detailed design stage, including procurement of more detailed information on the as-constructed lining and a detailed

Precinct	Utility	Construction Type	Assumed depth below surface^	Geology at Tunnel Axis	Volume Loss (%)	Maximum Settlement (mm)	Maximum Slope	Estimated Impact	Proposed Mitigation Measures
CBD South station	Utility Tunnel: Little Bourke Street Crossing	CIC		Melbourne Formation	0.5	8	<1:1000		inspection and assessment of the lining condition. Following the condition survey refinement of the modelling should be undertaken to account for the actual Telstra Tunnel and utility construction and condition. Subsequent to refinement of the modelling a plan of protective measures (strengthening works) should be established in consultation with the stakeholder to reinforce the tunnels where potential for unacceptable lining deformation or risk to tunnel or utility operation due to Melbourne Metro works is identified
	Utility Tunnel: Little Collins Street Crossing			Melbourne Formation	0.5	7	<1:1000		
Precinct 6 CBD South station	Little Collins Street to Flinders Street	Brick tunnel	2 m below ground	Melbourne Formation	0.5	10	<1:1000	Minor	
	525 mm Sewer on Swanston Street	Vitrified clay pipe	As-built level (10.69 m)	Melbourne Formation	0.5	44	<1:1000		
	Melbourne Main Sewer	Brick	As-built level (11.269 m)	Melbourne Formation	0.5	41	<1:1000		
Precinct 7 Domain station	South Yarra Main Sewer	Brick	As-built level (18.376 m)	Melbourne Formation	0.5	22	-	-	Sewer to be reconstructed at Domain station. Tie-in points to the existing brick lined sewer are outside the zone of potential influence as defined by the 5mm ground movement contour.
Precinct 1 Tunnels: *Domain to Portal*	St Kilda Road - Typical Service	Concrete	2 m below ground	Newer Volcanics	1	24	<1:1000	Negligible	Allowance for pipe roof in the Concept Design scheme
	Walsh Street - Water Main	Steel Welded pipe. Open cut	2 m below ground	Melbourne Formation	0.5	6	<1:1000	Negligible	
	Toorak Road - Typical Service	Steel Welded pipe. Open cut	2 m below ground	Melbourne Formation	0.5	8	<1:1000		
Precinct 8 *Eastern Portal*	Osborne Street	Steel Welded pipe. Open cut	2 m below ground	Brighton Group	1	44	1:220	Minor	Allowance for pipe roof in the Concept Design scheme Ground improvement required at the portal plus additional conduit protection Potential requirement for fracture grouting from within

Precinct	Utility	Construction Type	Assumed depth below surface^	Geology at Tunnel Axis	Volume Loss (%)	Maximum Settlement (mm)	Maximum Slope	Estimated Impact	Proposed Mitigation Measures
									the tunnel
	Stormwater drain - Sandringham Line Rail Corridor	Steel Welded pipe. Open cut	2 m below ground	Brighton Group	1	41	1:270		Some utilities to be reinstated on completion of works

10 Parkland Impact Assessments

Potential impacts on selected waterways and parkland are described in Table 10-1.

Table 10-1 Waterways and Parklands that were considered in this assessment

Precinct	Type	Site	Estimated Max Settlement	Estimated Impacts	Proposed mitigation measures
Precinct 2 Western Portal	Parkland	JJ Holland Park	<10 mm primary consolidation settlement	Negligible	-
Precinct 1 Tunnels	Waterway	Moonee Ponds Creek	30 mm settlement (combined primary consolidation and tunnelling induced)	Negligible to minor impact	Site to be remediated on completion of works, if required
	Parkland	Moonee Ponds Creek bike path			
	Parkland	Lincoln Square Gardens	Outside potential zone of influence	Negligible to minor impact	-
	Parkland	North bank Yarra River	10 mm	Negligible to minor impact	Sites to be reinstated on completion of works if required
	Waterway	Yarra River	30 mm		
	Parkland	South bank Yarra River & Alexandra Gardens & Bike path	30 mm		
	Parkland	Queen Victoria Gardens	20 mm		
		Kings Domain	10 mm		
	Parkland	Tom's Block	50 mm	Moderate impact	Requirement for ground improvement works if impacts cannot be mitigated through close control on TBM operation Sites to be remediated on completion of works
	Parkland	Fawkner Park	40 mm (immediately around proposed shaft)	Potential impacts on parklands are anticipated to be minor immediately surrounding the shaft construction site	Sites to be reinstated on completion of Melbourne Metro works
			<10 mm elsewhere	Negligible impact	Sites to be reinstated on completion of works, if required
Precinct 8 Eastern Portal	Parkland	South Yarra Siding Reserve	20 mm	Negligible to minor impact	Sites to be reinstated on completion of works if required
	Parkland	Lovers Walk, North Batter Eastern Portal Cutting	10 – 20 mm		

Measures to manage or mitigate potential impacts on waterway crossings include monitoring of ground surface levels as the tunnelling approaches the channel/river to compare estimated movements with actual movement and re-assess associated risks if required. Tighter control on construction might be necessary. Channel geometry may be restored after completing of a section of work if any slips detected by ground movement monitoring.

11 Further Work

The impact assessments and risk assessment documented in this report are preliminary in nature yet suitable for a major project at this stage of design development and for the purposes of the EES. It provides a sound basis to evaluate the likely impacts of the proposed project along its alignment. Further work would be required as the project scheme is further developed and additional information becomes available to inform the detailed design stage assessments. The completed preliminary assessments provide estimates of the potential ground movement effects of the Melbourne Metro works and the scale and extents of potential impacts on existing assets.

Many of the measures that are required to manage performance i.e. minimise ground movement magnitudes and extents resulting from Melbourne Metro works, along with mitigation of any potential adverse impacts on existing assets, would depend on scheme details that are not yet fully developed. Monitoring and contingency measures appropriate for the final project scheme would be formulated at the detailed design stage.

The following sections describe further work that is required prior to and during the detailed design stage. Some work would continue into project construction phase, to be further refined and reassessed where required, as the project progresses.

11.1 Additional Studies

Additional geotechnical and hydrogeological investigations would be undertaken prior to construction, for the purposes of refining the conceptual geological and hydrogeological model, and ensure the reliability of the interpreted soil and rock engineering parameters adopted in the detailed design.

The following would be assessed:

- Potential effects of groundwater drawdown at detailed design, by developing refined models of the geotechnical and hydrogeological conditions
- Gather as-built data and survey information to inform potential ground movement impact assessments of potentially affected structures and utilities
- Baseline ground movement data comprising seasonal movements and secondary compression data to inform an assessment of potential impacts on existing assets
- Additional site investigation data to inform a detailed assessment of potential impacts to Princes Bridge looking at each abutment and pier independently
- The extents of the Potential Zone of Influence relating to ground movement at detailed design stage, with due consideration for the detailed construction methodologies and sequencing, detailed design schemes and alignment and updated interpreted geological and hydrogeological models.

11.2 Stakeholder Engagement

A communications and engagement strategy would be established to provide advance notice to stakeholders and the community of potential construction disturbance.

Stakeholders would be engaged in accordance with a project Stakeholder and Community Consultation Plan that would describe, for all property owners within the potential zone of influence, the community communications strategy in relation to the various levels of ground movement and associated assessments of potential impacts; protocol for structure condition survey, preparation of defects schedules, compensation or dispute resolution procedure.

Pre-construction stage property condition surveys would be undertaken on all buildings and structures within the Potential Zone of Influence, which would be confirmed at detailed design stage.

Post-construction stage property condition surveys would be undertaken, where required, in accordance with a Property Condition Survey and Stakeholder Consultation Plan.

11.3 Development of Acceptability Criteria

Acceptable levels of ground movement would be established for each piece of infrastructure identified with potential to be affected by the works, in consultation with the relevant stakeholders.

Acceptability criteria would be developed in consultation with the respective train and tram asset owners to establish the minimum performance specifications and tolerances with which construction works must conform to so that operations are not impacted adversely during the construction and operation stages.

Acceptability criteria would be established for buildings, based on the relationship of building damage to angular distortion and horizontal strain and qualitative factors including but not limited to the type of structure and its existing condition.

Acceptability criteria would be established for utility mains, in consultation with the respective utility owners.

11.4 Instrumentation and Monitoring

Frequency of measurement would be determined based on the extent of possible movement and the time over which it could occur. The nature of the works to be undertaken and traffic arrangements should also be considered so that instrumentation is not damaged or compromised.

A monitoring plan would include:

- Monitoring of ground movements due to Melbourne Metro works and develop a geotechnical instrumentation and settlement monitoring plan with appropriate alarm, alert and action levels and response strategies, to be agreed in consultation with the relevant stakeholders, where appropriate
- Conducting baseline monitoring prior to the commencement of Melbourne Metro works and dewatering to identify pre-existing movement, including rates of secondary compression where required
- Where settlement predictions exceed the allowable settlement criteria, implementation of feasible and reasonable management measures to minimise potential ground movement
- Accurate measurement of ground movements prior to, during and post construction with building audits, monitoring of ground movements and structure monitoring where required, and identify and implement any additional required ground support or required protection measures
- Monitoring the encountered soil and rock conditions and monitor this against the anticipated soil conditions, during tunnel construction
- Establishing a routine groundwater monitoring plan to continue throughout the construction period. A groundwater monitoring network would contain monitoring wells along the entire Melbourne Metro alignment
- In the event of unforeseen or unacceptable settlement, preparing new building condition survey report(s) and establish recommendations for repairing any building damage. Actual settlements would be compared to the predicted settlements and further mitigating measures taken where required in accordance with an MMRA approved ground movement monitoring and management plan.

Monitoring of ground surface settlement points at works sites and along the twin tunnel alignment should commence at least 12 months prior to start of construction and continue for a period of not less than 6 months after ground movement has stabilised with particular reference to risk areas.

Tunnel and excavation monitoring requirements would be described in contract documents.

11.4.1 Monitoring Phases

This section describes the three monitoring phases followed by brief description of typical monitoring systems for different structures and infrastructure located within the Potential Zone of Influence.

Vibration monitoring due Melbourne Metro excavations is not covered in this report, however it should be noted that it can be a significant factor causing damage to the surrounding buildings, structures and infrastructures.

An extensive monitoring program of baseline and construction stage vibration and settlement impacts in the vicinity of the proposed works would be undertaken. Pre-construction stage property condition surveys are required and provision for reinstatement, in the unlikely event of damage due to project works, would be in place for any potentially affected structures. In addition, a communications and engagement strategy would be in place to provide advance notice of potential construction disturbance.

Definition of the minimum offset to define the area for condition surveys is to be determined in consultation with MMRA. It is also noted that a contactor may choose to determine a wider offset in order to provide additional insurance against potentially inappropriate claims of damage.

11.4.1.1 Baseline Monitoring

Prior to construction, a condition survey would be conducted of structures, railway and tram tracks, pavements, significant utilities and parklands within the Potential Zone of Influence would be conducted in order to establish baseline conditions.

Review the condition surveys would be undertaken in consultation with property owners and stakeholders, as appropriate to identify potential risks relating to ground movement as a consequence of the project.

A condition survey of structures or features of heritage value within the zone of potential influence would be undertaken to assess structural vulnerability, and to provide a baseline for monitoring.

11.4.1.1.1 Geotechnical and Hydrogeological Baseline Monitoring

Reliable baseline measurements (typically of the ground surface and groundwater level trends) are required well in advance of construction as a reference against which subsequent changes can be measured. Typically, these baseline measurements are undertaken 12 months ahead of construction. Baseline deformation measurements can also be used to determine the sources of movement that may otherwise be attributed to the works. These include:

- Shrink-swell movement due to seasonal moisture changes in the soil profile
- Movement due to thermal response of structures which are sensitive to temperature changes and may move in diurnal or seasonal cycles
- Movement due to existing traffic loading (pedestrian and vehicular, light and heavy rail etc.).

Baseline monitoring of sufficient duration is required to reliably establish seasonal movement in order to capture the ambient behaviour of the structure(s) within the Potential Zone of Influence.

External influences such as existing groundwater drawdown and recharge sources and the effect of adjacent construction works would also require consideration.

11.4.1.2 Construction Phase Monitoring

Buildings, services and infrastructure along the Melbourne Metro alignment are required to be monitored during Melbourne Metro construction to ensure any adverse impact as a result of construction activities are identified early and appropriate response measures are implemented. The monitoring frequency would depend on a number of factors including the rate at which change is expected to develop and the feedback requirements for practical control of the construction process e.g. time required to allow an effective response to alarm levels.

Trigger levels would be linked to the ground movement and building damage assessments and would be implemented on a progressive basis (traffic light system). The trigger values and the related actions would be related to safety and serviceability considerations for the structures such that construction work would be stopped before the movements exceed the serviceability limits of the subject structures.

The frequency of the construction phase monitoring would typically increase with the proximity of construction works e.g. as the tunnel approaches within 50 m of a site, the monitoring frequency may increase from monthly to weekly and then to daily / hourly as excavation progresses to within 25 m and then 10 m. As the tunnelling progresses past a specific site, the frequencies of monitoring can then decrease and key instruments such as robotic total station units can be redeployed elsewhere, with subsequent measurements performed via manual survey.

Monitoring of initial tunnelling work on Melbourne Metro should be used to calibrate settlement modelling during the construction stage.

Table 11-1 summarises the monitoring response strategy and intervention limits adopted on NSRU project in NSW for shallow cover tunnel excavations beneath live railway tracks. A comparable strategy could form the basis for development of an appropriate monitoring response strategy on Melbourne Metro.

Agreed limits may be equal to an agreed specific maintenance intervention limit for some operating systems such as road, tram or rail. Trigger levels of movement may lie within a specific bank of movement/distortion.

Table 11-1 Typical Monitoring Response Strategy

Intervention Level:	Level 1 - ALERT	Level 2 - ACTION	Level 3 - ALARM	Level 4 - ALARM
Potential response banding	>25 % but <50% of Agreed Limit	>50% but ≤75% of Agreed Limit	Sudden movement >50% of Agreed Limit	Sudden movement >100% of Agreed limit
Sequence of response steps	- Notify all parties	- Notify all parties of the movement	- Works cease immediately	- Cease works and apply emergency procedures immediately
	- If movement exceeds predictions, Geotechnical Engineer to reassess	- Verify movement by reviewing and repeating the survey - Geotechnical Engineer to reassess results and recommend continuation or identify remedial measured prior to recommencing work		- Competent track person to reassess track geometry
Prior to works proceeding	- Works may proceed with caution and increased vigilance in areas of detected movement	- Approval required prior to proceeding	- Work does not recommence until methodology and monitoring procedures are reviewed and remedial measures are approved	- Work does not recommence until methodology and monitoring procedures are reviewed and approved remedial measures are complete

11.4.1.3 Close-out Monitoring

At the completion of construction, close-out monitoring would typically be performed during the construction defects period to check the ongoing performance of the project components and the response of the ground profile to the construction works.

Groundwater and consolidation settlement monitoring would continue through the project operational stage:

- For a period of not less than 6 months after ground movement has stabilised
- Until any groundwater drawdowns have recovered or
- Until primary consolidation settlement has conclusively stabilised.

11.5 Condition Surveys

Damage classifications are based on many simplifying assumptions. Hence, detailed pre-construction building condition assessments, baseline and construction monitoring are a vital part of the on-going process of limiting potentially adverse effects of Melbourne Metro construction. A condition survey would assist in identifying potential vulnerabilities of a building fabric to damage as a consequence of ground movements. Suitable mitigations for such risks would be developed at the detailed design stage.

Prior to commencing any construction activities which may affect the existing ground profile, condition surveys would be conducted on all existing buildings and structures, within the Potential Zone of Influence relating to ground movement. The conditions surveys would be performed by a qualified structural engineer or a chartered building surveyor and would typically involve a detailed inspection of the structure including video and photo records as well as measurement of any existing cracking/structural distress. A written report would then be prepared to establish the baseline conditions for the structure in advance of any construction. This would provide the owner with assurance that damage caused by tunnelling or other construction activities can be objectively identified.

The owners of the subject buildings / structures would be contacted prior to start of construction to arrange a formal condition (dilapidation) survey. A copy of the record of condition would be sent to the property owner to form a written and photographic factual record of the existing conditions of the property as well as details on the construction type(s), finishes and evidence of any existing cracks or visible defects.

If a property owner believes damage has been caused to their property resulting from Melbourne Metro works, the owner would contact the project Community Engagement Team. A second condition survey would be carried out and a comparison of pre and post construction stage condition surveys would form the basis of any claim. Details on the stakeholder and community engagement process would be outlined in a formal document that also describes the proposed assessment, monitoring and mitigation measures that would be implemented by the project. Any required repair works would be carried out in accordance with the terms and conditions of an associated asset protection agreement.

A policy document would be developed that describes the procedures to be followed by eligible property owners in order to communicate any changes that may be attributable to the project and how any changes would be rectified by Melbourne Metro where attributable to the project.

Similar to the property condition survey outlined above, a condition assessment would be required for the infrastructure including the existing tunnels (City Loop, CityLink, and Telstra) and sewers where they intercept the Potential Zone of Influence, with this to be performed prior to undertaking any construction works. The condition assessment may include but not be limited to a review of as-built and construction data and a detailed inspection of the tunnel and sewer lining. This inspection would note any cracking, leakage, spalling and evidence of deterioration in any support structure. A survey of the tunnel and sewer linings may also be performed to check for any distortion to the as-built profile. Photographic and video records would be collected and measurements taken of any existing cracking.

The extent and level of detail of infrastructure inspections would be determined in agreement with the asset owners and operators.

11.6 Construction Stage Control Measures

Ground movement and ground movement impacts assessments would be undertaken at the detailed design stage to inform the project risk register in relation to ground movement. Measures to limit ground movement and ground movement effects would be incorporated into the construction scheme as far as reasonably practical. Provision for additional ground movement risk mitigations such as ground improvement or protective works would be designed at detailed design stage.

12 Environmental Performance Requirements

This section provides a list of the Environmental Performance Requirements identified as a result of this ground movement assessment. Environmental Performance Requirements, listed in the table below, apply across the project and are linked to the draft EES evaluation objective.

Appropriate ground movement limiting measures would be developed, initially, in the detailed design process and applied prior to or during the construction stage. Issues which would need careful consideration are tunnel volume loss, design of tunnel support and liners, and stability assessment of open excavation retention systems, as well as driven tunnel and groundwater modelling of any impact by Melbourne Metro works.

Additional geotechnical investigations are required for improved definition of the subsurface profile and materials along the alignment and hence reduce the risk of encountering conditions not accounted for in the design. These measures would limit predicted damage to negligible or minor consequences, and hence, damage would be easily repairable if it occurred.

All structures and utilities within the Potential Zone of Influence with potential for adverse impacts would have a condition survey completed prior to construction. Condition surveys and other displacement monitoring would be used to monitor the effects of settlement, if any, from Melbourne Metro works. The actual settlements would be compared to predicted settlements. As described in Section 11.4.1.2, appropriate ground movement and structure response strategies would be developed in consultation with the relevant stakeholders and implemented in the construction stage.

The requirements to be met in order to meet the draft EES evaluation objective "To avoid or minimise adverse effects on land stability that might arise directly or indirectly from project works" are listed in Table 12-1 below.

An Environmental Performance Requirement that stipulated that the Melbourne Metro should be constructed in such a way so that there is no ground movement would be excessively onerous on the project scheme, impractical to achieve and uneconomical. To minimise the risks associated with ground movement, it is important to adhere to good construction practices and ensure that effective monitoring and management approaches are implemented and reviewed from the onset of construction.

Environmental management compliance requirements are described in the EES Report Chapter 23 *Environmental Management Framework*. To evaluate environmental performance outcomes, the Contractor would implement a CEMP with strategies that enable conformance to and measurement of delivery of Environmental Performance Requirements.

Table 12-1 Environmental Performance Requirements for Ground Movement and Land Stability

Objective	EPR no.	Environmental Performance Requirements	Proposed mitigation measures	Risk No
Land Stability – To avoid or minimise adverse effects on land stability that might arise directly or indirectly from project works	GM1	Develop and maintain geological and groundwater models (as per GW2) which: • Use monitored ground movement and ground water levels prior to construction to identify pre-existing movement; • Inform tunnel design and the construction techniques to be applied for the various geological and groundwater conditions; • Assess potential drawdown and identify trigger levels for implementing additional mitigation measures to minimise potential primary consolidation settlement; and • Assess potential ground movement effects from excavation and identify trigger levels for implementing additional mitigation measures to minimise potential ground movement effects	While not specifying required mitigation measures, the recommended Environmental Performance Requirements are framed to ensure appropriate mitigation and management measures would be adopted and implemented in the design and construction of Melbourne Metro. Refer also to the potential impact management measures identified in the ground movement assessment tables provided for each precinct.	GM001 – GM025
	GM2	Design and construct the permanent structures and temporary works so as to limit ground movements to within appropriate acceptability criteria (to be determined in consultation with the relevant stakeholders) for vertical, horizontal, and angular deformation, as appropriate, for project activities during the construction and operational phase	As above	GM001 – GM025
	GM3	Develop and implement a ground movement plan for construction and operational phases of the project that: • Addresses the location of structures/assets which may be susceptible to damage by ground movement resulting from Melbourne Metro works; • Identifies appropriate ground movement impact acceptability criteria for buildings, utilities, trains, trams and pavement in consultation with the various stakeholders; • Identifies mitigation measures to ensure acceptability criteria can be met; • Identifies techniques for limiting settlement of buildings and protecting buildings from damage; • Addresses additional measures to be adopted if acceptability criteria are not met such as reinstatement of any property damage; • Addresses monitoring ground movement surrounding proposed Melbourne	As above	GM001 – GM025

Objective	EPR no.	Environmental Performance Requirements	Proposed mitigation measures	Risk No
		Metro works and at the location of various structures/assets to measure consistency with the predicted model; • Consult with land and asset owners that could potentially be affected and where mitigation measures would be required		
	GM4	Conduct pre-construction condition surveys for the assets predicted to be affected by ground movement. Develop and maintain a data base of as built and pre construction condition information for each potentially affected structure, specifically including: • Identification of structures/assets which may be susceptible to damage resulting from ground movement resulting from Melbourne Metro works; • Results of condition surveys of structures, pavements, significant utilities and parklands to establish baseline conditions and potential vulnerabilities; • Records of consultation with landowners in relation to the condition surveys; • Post construction stage condition surveys conducted, where required	As above	GM001 – GM025
	GM5	Adopt construction techniques for Melbourne Metro to limit ground movement to within appropriate acceptability criteria (to be determined in consultation with the relevant stakeholders)	As above	GM001 – GM025
	GM6	For properties and assets affected by ground movement, undertake any required repair works	As above	GM001 – GM025
	Also refer to the Environmental Performance Requirements for “Groundwater” GW1 and GW3, and Heritage CH2		As above	GM001 – GM025

13 Conclusions

The objective to avoid or minimise adverse effects on land stability that might arise directly or indirectly from project works can be practically achieved through implementing engineering solutions that would minimise ground movements through:

- Adoption of suitable excavation equipment and construction methodologies
- Where required, improvement of the ground mass surrounding the excavation to minimise ground movement and/or groundwater inflows to proposed works.

The methodology adopted for assessing ground movements and their potential effects has been based upon methods which were developed over the past 40 years in Europe and the USA, and which have been employed extensively on past tunnelling projects in Australia and internationally. These are documented in the technical papers by Rankin 1988, Burland et al 2001, Mair et al 1996, and Boscardin et al 1989.

The assessment has been based upon review of the ground conditions and modelling that provides a guide on the general settlement values, their distribution, and their effects on structures and underground services.

Particularly complex interactions, such as between Melbourne Metro and the existing City Loop station and tunnels, have been the subject of more detailed analyses to assess the potential impacts of the anticipated ground movements with respect the structural response of the affected infrastructure.

Based on the available information, it is our view, that:

a) The underground works for Melbourne Metro would lead to some degree of settlement of the ground in the vicinity of the project works. Most of these effects would occur during construction but some longer term effects might extend beyond that period

b) Ground movement may occur due to a number of mechanisms related to the excavation of the tunnels or station structures and other major underground structures such as station entrances, shafts and portal structures

c) Settlement due to loading from new embankment works would only occur locally around widening works in the western portal area

d) Ground movement may also occur as a response of softer materials to the changing of groundwater levels. As these are determined by the behaviour to the groundwater level changes, their effects can be farther reaching than those associated with excavation

e) Selection of a skilled and experienced contractor, maintaining tight control on excavation procedures and ensuring quality of construction would address the root cause of most potential ground movement issues, where otherwise potentially costly surface level measures might be required to minimise or control impacts.

Whilst construction of Melbourne Metro necessitates interaction with numerous existing buildings and infrastructure (including existing tunnels), these are interactions which are successfully managed in the construction of every metro system the world over.

While the details of many buildings and other structures are not yet available, enough information is available to make an initial assessment of the likely impacts with a reasonable degree of confidence, particularly where the outcome of the assessment indicates negligible or minor impact. This is based upon a combination of experience of similar structures in response to ground movement on past projects (such as South Island Line C901 and C904 - Hong Kong) and a knowledge of what changes in the outcome would result from possible differences in the assumptions made in this report.

There are, however, zones where the ground conditions or the proximity of the excavation could potentially lead to damage, if not well managed. For such cases, the assessment includes descriptions of the mitigation measures that could be applied, and lists, in the Environmental Performance Requirements, where risk management or mitigation would need to be applied. These are, typically, standard tunnelling construction practices that have been included in the ground movement impacts assessments.

These outcomes would require the adoption of appropriate construction methods, including use of appropriate equipment, staging, and mitigation works, together with selection of adequately experienced staff and a monitoring program and achieving the Environmental Performance Requirements.

In general, the preliminary assessment predicts that impacts of the ground movements for buildings and other infrastructure would be in the range of negligible to minor, with ground movement creating, at worst and in a few instances, minor serviceability cracking or superficial damage that could be readily repaired. Proposed mitigation measures for potential impacts, where found to be moderate, would reduce potential impacts to acceptable levels.

With the mitigation measures specified, the estimated impacts associated with the described risk pathways are considered to be acceptable in ground movement terms, particularly in the context of a project of this scale.

14 Limitations

This report has been prepared by the AJM JV at the request of the Melbourne Metro Rail Authority exclusively for the purposes of informing the Environment Effects Statement. Therefore, there is some important information that should be noted regarding this report.

The limited scope of AJM JV's brief in this matter, including the time constraints imposed on the assessments, means that the report necessarily concentrates on readily apparent major items, selected by consideration of the risk of adverse effects. However, notwithstanding the limits on the analyses that have been conducted to date, the assessments are considered adequate to provide preliminary indication of the potential impacts of ground movement for the purposes of informing the EES.

Any interpretation or recommendations in the Report were based on the MMRA's specific brief, other information available to the AJM JV at the time, and the AJM JV's professional experience. Site conditions and/or information may have altered since the report was completed and the assessment may become irrelevant or inaccurate in regard to such contingencies.

The limitations on the completed ground movement assessment and impacts assessment are as follows:

- The extent of investigation required to provide a report on the assessment of the effects of ground movement would be greater than has been carried out to provide this EES report based on the preliminary assessments
- The assessments are subject to the limitations on the estimates of potential ground movement and potential groundwater drawdown described in the following reports:
  - Appendix A of this report – Golder Associates Interpreted Geological Setting EES Summary Report
  - Appendix B of this report – Golder Associates Ground Movement EES Summary Report
  - Technical Appendix O *Groundwater*
  - Golder Associates Interpreted Hydrogeological Setting EES Summary Report
  - Golder Associates Regional Groundwater Numerical Modelling EES Summary Report
- The issued report is relevant to the defined Concept Design and is not intended to address changes in project configuration or modifications which occur over time
- For many buildings and infrastructure, only very general information is available. Therefore, in these cases, the current assessment is based upon photographs of the building or infrastructure and experienced judgement of AJM JV Engineers in determining the likely structural form and foundation type. Where site inspections have been made, they have been limited in their scope to visual inspections, typically conducted externally. No detailed testing or inspection etc. was carried out
- In the absence of structure or building specific preliminary condition assessments, the impact assessments to date have assumed that the current structural condition and serviceability of buildings and structures are sound
- For some buildings and infrastructure, the AJM JV has been able to use information sources provided by other parties. While reviewed as far as reasonable, the data have not been verified by the AJM JV and the AJM JV has no control over this information. This assessment is provided on the basis that the information that has been provided is accurate, complete and adequate. Should these information sources prove to be incomplete or inaccurate, the AJM JV assessment of that particular aspects would become irrelevant or inaccurate
- The potential impacts described in this report are based on the results of the site investigations up to September 2015 which were incorporated into the current geotechnical and hydrogeological reports. Therefore, the limitations described in the referenced reports on the data, and particularly the

interpretations presented in those reports, should be understood, as they directly affect the findings of the assessments for this report

- The assessment was based on the Concept Design and the associated Alternative Design Options. This report would require updating if any design changes, additional information or design development comes to hand or occurs.

It is not possible to make a full assessment of this report without a clear understanding of the terms of engagement under which the report has been prepared, including the scope of the instructions and directions given to and the assumptions made by the engineer who has prepared the report.

The report may not address issues which would need to be addressed with a third party if that party's particular circumstances, requirements and experience with such reports were known and may make assumptions about matters of which a third party is not aware. Therefore, the AJM JV does not assume responsibility for the use of the report by any third party for purposes other than the EES and the use of the report by any third party is at the risk of that party.

Subject to the limitations referred to above, the AJM JV has exercised all due care in the preparation of the Report and believes that the information, conclusions, interpretations and recommendations of the Report are both reasonable and reliable. This Report is not a certification, warranty or guarantee. It is a report scoped in accordance with the MMRA's instructions, having due regard to the assumptions that AJM JV can be reasonably expected to make in accordance with sound engineering practice.

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Appendices

Appendix A

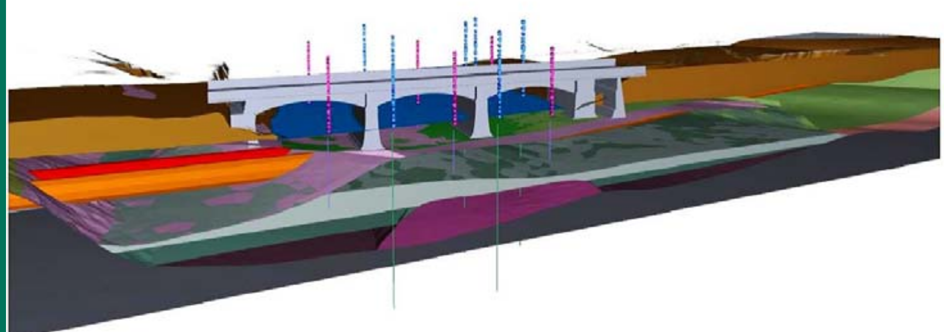
Golder Associates Interpreted Geological Setting EES Summary Report

20 April 2016

MELBOURNE METRO RAIL PROJECT

Interpreted Geological Setting EES Summary Report

Submitted to:
AJM Joint Venture
121 Exhibition Street
Melbourne, Vic, 3000



REPORT

Report Number. 1525532-218-R-Rev2

Distribution:

1 Copy - AJM Joint Venture
1 Copy - Golder Associates Pty Ltd

GLOSSARY OF ABBREVIATIONS AND TERMS

AHD	Australian Height Datum
AJM JV	Aurecon Jacobs Mott Macdonald Joint Venture
CBD	Central Business District
CAI	Cerchar abrasivity index
CPT	Cone Penetrometer Test
EPB	Earth Pressure Balance
EW, HW, MW, SW, FR	Extremely Weathered, Highly Weathered, Moderately Weathered, Slightly Weathered, Fresh
GSI	Geological Strength Index
Ma	Million years ago
MMRA	Melbourne Metro Rail Authority
Melbourne Metro	Melbourne Metro Rail Project
MURL	Melbourne Underground Rail Loop
MURLA	Melbourne Underground Rail Loop Authority
PLI	Point Load Strength Index
PTV	Public Transport Victoria
RD	Reference Design
SPT	Standard Penetration Test
TBM	Tunnel Boring Machine
UCS	Uniaxial Compressive Strength
VCCC	Victorian Comprehensive Cancer Centre

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1.0 INTRODUCTION

Aurecon Jacobs Mott Macdonald Joint Venture (AJM JV) has engaged Golder Associates Pty Ltd (Golder) to provide geotechnical, hydrogeological and environmental services for the proposed Melbourne Metro Rail Project (Melbourne Metro). The services provided by Golder are to support the development of the Environment Effects Statement (EES) for the Melbourne Metro 'Concept Design'.

The Melbourne Metro Concept Design comprises approximately 9 km of rail tunnels running from Kensington to South Yarra, including five new stations. The proposed alignment would connect into the existing rail network near South Kensington station, run beneath North Melbourne and Parkville, then continue south beneath Swanston Street, under the Yarra River, east of and beneath St Kilda Road, then east beneath Toorak Road and Fawkner Park. The Concept Design connects to the existing rail network at South Yarra.

This EES summary report provides discussion of the field investigation results and ground conditions likely to be encountered along the Melbourne Metro Concept Design alignment. The relationship of this report to the other EES specialist reports is summarised in Table 1.

Within this report, the areal extent of the Melbourne Metro Concept Design, which incorporates the station boxes, portals and tunnels, would be referred to as "the Study Area". The extent of the Study Area is presented on the geological cross section within Appendix A.

Table 1: Relationships between EES Specialist Reports and the supporting Golder EES Summary Reports

		EES Specialist Reports			
		Ground movement and Land Stability	**Future Development Loading**	**Groundwater**	**Contaminated Land and Spoil Management**
Golder EES Summary Report	**Ground Movement Assessment**				
	Interpreted Geological Setting				
	Interpreted Hydrogeological Setting				
	Regional Groundwater Numerical Modelling				
	Contaminated Land Assessment				

1.1 Background

Between 2011 and 2013, Golder was engaged by Public Transport Victoria (PTV) to provide geotechnical services to investigate potential Melbourne Metro route options. The works over this period included the undertaking of a desk study, the intent of which was to collate existing subsurface information along the proposed alignment, and completion of preliminary geotechnical investigations.

During 2015, Golder was engaged to undertake a further stage of geotechnical investigation to support the development of the Concept Design for Melbourne Metro. This report builds upon the ground models developed previously in the earlier stages of the project and present an updated conceptual ground model for the Melbourne Metro Concept Design, which considers all of the factual information which has been collected for the project up to September 2015.

1.2 Project Description

The Melbourne Metro Concept Design comprises 7.2 m external diameter twin rail tunnels approximately 9 km long, running from Kensington to South Yarra. The proposed alignment is presented in Figure 1.

Key aspects of the project include:

- Portals at South Yarra and Kensington;
- Three cut and cover station excavations at Arden, Parkville and Domain;
- Two underground cavern station excavations at CBD North and CBD South; and
- Ventilation shafts and cross passages along the twin tunnel alignment.

Based on discussion with AJM JV throughout the development of the Melbourne Metro Concept Design the following provides a high level summary of the concepts for proposed Civil Infrastructure, from west to east:

- The alignment branches north off the existing Sunbury line just east of the Kensington Road Bridge and dives in a cut towards the western portal. The twin track decline structure is to be fully retained.
- A shaft is to be constructed at the western portal for use in TBM retrieval during construction and as a permanent access and egress shaft.
- The rail tunnels from western portal to Arden station are to be constructed using closed face Tunnel Boring Machines (TBMs).
- Arden station is to be constructed as a cut and cover station box. Retention is to be provided over the full height of the station excavation.
- The twin rail tunnels from Arden station to Parkville station are to be constructed using open or closed face TBMs.
- Parkville station is to be constructed as a top down cut and cover excavation. Retention is to be provided over the full height of the station excavation.
- The twin rail tunnels from Parkville station to CBD North station are to be constructed using open or closed face TBMs.
- CBD North station is to be constructed in an underground cavern. The cavern is expected to have a span of approximately 23 m. An approximately 40 m deep access shaft would be constructed adjacent to the cavern. Underground adits and passages would be constructed between the shaft, cavern and the existing Melbourne Central Station.
- Twin tunnels would be mined between CBD North and CBD South Stations (as opposed to a TBM).
- CBD South station is to be constructed in an underground cavern. This would have similar dimensions to the cavern at CBD North station. Two access shafts would be constructed. The northernmost shaft is proposed at the existing City Square basement car park, and would require this existing basement to be deepened from about 10 m to 26 m. The southernmost shaft would be about 34 m deep.
- TBM tunnels are proposed between CBD South and Domain stations. This section of the alignment would pass beneath the Yarra River and would be bored through highly variable geological materials including very high strength rock and soft clay. The tunnels would pass beneath the existing footings of the Princes Bridge. Closed face TBMs are expected to be required through this section.
- Domain Station is to be constructed as a partial top down cut and cover excavation. Retention over the full excavation height would be required.
- Twin TBM tunnels are proposed between Domain station and the Eastern portal.

- The Eastern portal consist of a ventilation / emergency egress / TBM retrieval shaft in the vicinity of Osborne Street, realignment of the existing Dandenong and Frankston Line tracks, Twin track cut and cover tunnel sections including a section beneath the Sandringham Line tracks and Frankston Up track, Twin track tunnel decline structure between the reconfigured Dandenong Line tracks and surface tie-in to the existing Dandenong Line.
- There are a total of fourteen emergency egress cross-passages. This includes four low point drainage sumps with pumping facilities.
- There are two Fire Brigade emergency access shafts located at Linlithgow Avenue and Fawkner Park.

For Golder reporting purposes, the alignment has been divided into 23 segments, based on the type of infrastructure proposed and the anticipated ground conditions. The segments are numbered from west towards east. Their extents are shown on the longitudinal geological section in Appendix A and a brief description presented in Table 2 below.

Table 2: Summary of segments adopted for Golder reporting purposes

Segment	EES Precinct	Description	Key elements
1	2	Surface works and embankments	Embankment widening on potentially soft soils.
2	2	Western Portal approaches	Decline structure including retained excavation through soft soils and weak rock.
3	2	Western Portal and TBM shaft	Cut and cover excavation for TBM shaft and portal within weak rock.
4	1	TBM Tunnels	Twin bored tunnels through weak rock.
5	1	TBM Tunnels	Twin bored tunnels through dense clayey sand and sand with cross passage.
6	1	TBM Tunnels	Twin bored tunnels through soft to stiff cohesive soils, some gravel and sand.
7	3	Arden Station	Fully supported station box excavation through soft to stiff cohesive soils, some gravel and sand.
8	1	TBM Tunnels	Bored tunnels through mixed face conditions comprising dense sands, clayey sands and weak rock.
9	1	TBM Tunnels	Bored tunnels through weathered siltstone and sandstone
10	4	Parkville station	Fully retained station excavation through weathered and jointed siltstone and sandstone. Interaction with adjacent building and basements.
11	1	TBM Tunnels	Bored tunnels through weathered to fresh siltstone and sandstone.
12	5	CBD North station	Underground cavern excavation in weathered to fresh siltstone and sandstone. 40 m deep access shaft with full retention.
13	1	Mined Tunnels	Mined tunnels through weathered siltstone and sandstone.
14	6	CBD South station	Underground cavern excavation in weathered to fresh siltstone and sandstone. 34 m deep access shaft with full retention. Deepening of existing City Square basement excavation.
15	1	TBM Tunnels	Bored twin tunnels through weathered siltstone and sandstone.
16	1	TBM Tunnels – Yarra Crossing	Bored tunnels through variable, mixed face conditions comprising high strength basalt rock, dense sand and soft to stiff clay.
17	1	TBM Tunnels	Bored tunnels through weathered siltstone and sandstone. Shaft at Linlithgow Avenue and one cross passage.

Segment	EES Precinct	Description	Key elements
18	1	TBM Tunnels – City Link Crossing	Bored tunnels through mixed face conditions with dense sand, hard clay and weathered siltstone and sandstone. In close proximity to the existing City Link tunnels.
19	1	TBM Tunnels	Bored tunnels through weathered siltstone and sandstone.
20	7	Domain station	Retained station excavation through weathered and jointed siltstone and sandstone, dense sand and hard clay.
21	1	TBM Tunnels	Bored tunnels through weathered siltstone and sandstone. One access shaft in Fawkner Park.
22	1	TBM Tunnels	Bored tunnels through mixed face conditions comprising weathered siltstone and sandstone, dense sand and hard clay.
23	8	Eastern Portal and TBM Shaft	Fully retained shaft in dense sand and hard clay. Fully retained decline structure in dense sand and hard clay. Widening of existing rail corridor excavations in dense sand and hard clay.

1.3 Document Limitations

Your attention is drawn to the document – “Limitations”, which is included in Appendix C of this report. The statements presented in this document are intended to advise you of what your realistic expectations of this report should be. The document is not intended to reduce the level of responsibility accepted by Golder, but rather to ensure that all parties who may rely on this report are aware of the responsibilities each assumes in so doing.

2.0 SOURCES OF GEOTECHNICAL INFORMATION

The following describes the sources of geotechnical information used to compile the ground models presented in this report.

2.1 Desktop Audits (Stages 1 and 2)

Stage 1 of Melbourne Metro included a geotechnical desk study (undertaken by the Technical Advisor to PTV in 2010) as part of the development process and investigation of the potential Melbourne Metro alignment. Geotechnical information was obtained from geological maps, historical aerial photographs and existing geotechnical reports, including Golder's archives and information provided by PTV.

In Stage 2, between 2011 and 2013, Golder undertook a further desktop audit to gather additional geotechnical information to supplement the database compiled by the Technical Advisor in Stage 1. Golder has been operating in Melbourne for over 40 years, and during this period has undertaken many geotechnical investigations within the vicinity of the proposed Melbourne Metro alignment. Sites we have investigated that are located within 200 m of the alignment were identified. Key projects included: Regional Rail Link, Melbourne University, Carlton and United Brewery Site, Federation Square, Latrobe Street Telstra Tunnels and City Link's Southern and Western Links.

A number of data sources were also provided by the Department of Economic Development, Jobs, Transport and Resources including:

- Melbourne Underground Rail Loop Authority (MURLA)
- CBD Telecommunications Tunnels
- Melbourne Metropolitan Board of Works sewers
- The Federation Square development.

The longitudinal Sections in Appendix A of this report show the locations at which desktop audit information is available. Reports of boreholes and the site plans showing the borehole locations were extracted from these reports and used in the development of the geological model.

2.2 Geological Maps

Geological maps produced by the Geological Survey of Victoria have been digitised and geo-referenced into a GIS system. An initial assessment of the geology underlying the proposed alignment can be made from the geological mapsheet, giving an indication of the type of soil and rock that may be encountered along the alignment. The geological mapsheet is also useful in allowing an assessment to be made of the geological history of the materials expected to be encountered along the proposed alignment. Establishing the mode and timing of the deposition of the subsurface materials is important when interpreting the materials that may be present between the proposed borehole locations.

Appendix B presents a set of surface geological plans derived from the geological maps, other information sources consulted as part of the Stage 1 desktop audit and the Stage 2 subsurface information. We note that the boundaries shown on the geological mapsheet have been adjusted to reflect subsurface information obtained from the desk study and subsequent Stage 2 and investigations undertaken to support the Melbourne Metro Concept Design development.

2.3 Aerial Photographs

PTV has provided us with digital aerial photography from 2009. Historical aerial photographs from 1945 supplied by Melbourne University have also been uploaded, spatially referenced and included in the GIS system to provide an indication of how the land surface conditions have changed over time.

2.4 Project Specific Geotechnical Investigations

There have been three main stages of geotechnical investigation specifically for Melbourne Metro.

The Stage 1 investigation, undertaken by the Technical Advisor included completion of the following:

- 22 no. Boreholes: MM1BH001 to MM1BH022 drilled along the alignment proposed at that time (2010) and drilled to depths of between 25 m and 70 m, including three boreholes drilled within the Yarra River using sonic drilling methods.

The results of this study, including associated in situ defect orientation measurement and laboratory testing, have been considered in this report and supported ground model development.

The Stage 2 investigation undertaken by Golder included completion of the following:

- 25 no. Boreholes: GA11-BH001 to GA11-BH003, GA11-BH005, GA11-BH007 to GA11-BH009, GA11-BH011 to GA11-BH014, GA11-BH017 to GA11-BH027, GA11-BH031, and GA11-BH040 to GA11-BH041;
- 3 no. Cone Penetration Tests (CPTs): GA11-CPT002, GA11-CPT003 and GA11-CPT004;
- A marine geophysical survey at the Yarra River crossing;
- 8 no. Yarra River Crossing Boreholes: GA11-BH032 to GA11-BH039;
- 12 no. Yarra River Crossing Probe Holes: GA11-PH001 to GA11-PH012; and
- A bathymetric and side scan sonar marine geophysical survey.

The results of the Stage 2 investigations, including associated in situ and laboratory testing, have been considered in this report and supported ground model development.

The third stage of investigation, undertaken by Golder was commenced in May 2015 and is ongoing. The objective of the third stage of investigation is to provide information for development of the Melbourne Metro Concept Design and EES and to provide information for tenderers bidding to deliver the project.

The component of the investigation considered for this report comprised the following:

- 36 no. Boreholes drilled to depths of between 7.5 m and 60 m. GA15-BH001 to GA15-BH013, GA15-BH017 to GA15-BH019, GA15-BH021, GA15-BH025 to GA15-BH033, GA15-BH108 to GA15-BH112 and GA15-BH120 to GA15-BH123;
- Laboratory testing on samples recovered from boreholes;
- 7 no. in situ stress tests undertaken in boreholes close to key locations at which the proposed alignment would interact with existing infrastructures;
- Insitu testing including pressuremeter, acoustic teleview (defect orientation) and packer testing; and
- A borehole pump test.

Note that the Melbourne Metro Concept Design stage of the investigation comprised an initial 36 boreholes out of a greater number of boreholes scheduled to provide information for further stages of the project.

This EES summary report is based on the factual information outlined above which has been collected for the project. The locations of all boreholes drilled as part of the investigations described above are shown on the geological longitudinal sections in Appendix A.

3.0 SCOPE OF WORKS (CONCEPT DESIGN STAGE REPORT)

This report provides a summary of the expected ground conditions along the proposed Melbourne Metro Concept Design alignment and describes interpreted geological setting to support the development of EES.

A long section at project wide scale is presented in Appendix A. The text within this report provides a discussion on each of the geological units expected to be encountered along the proposed Melbourne Metro alignment and their expected engineering characteristics as relevant.

Hydrogeological and land contamination issues have not been addressed in this report. These issues are discussed in the Interpreted Hydrogeological Setting and the Contaminated Land Assessment EES Summary Reports.

3.1 Ground Model Development

The geology indicated on the long sections in Appendix A has been developed using the methods described in the following dot points:

- The ground surface profile, along the alignment of the westbound of the twin tunnels is plotted. Relevant boreholes within approximately 50 m of the alignment are plotted onto the long section.
- The long section has then been compiled based on the information presented in the boreholes.
- The relationship between the various geological units is based on the information indicated in boreholes and on the known geological history and relationship between materials, as described previously.
- We note that the only points at which the stratigraphy is known is at the borehole locations. All relationships between the geological materials as shown on the long section have been interpreted and as such there is inherent uncertainty in the interpretation.

3.2 Ground Model Reliability

During the course of the desk study audit and subsequent investigations for Melbourne Metro, boreholes in locations considered relevant to the proposed Melbourne Metro alignment have been compiled into a database. Relevant locations are considered to be not only those boreholes near the alignment, and also those that could be within the hydrogeological influence of the project or off the alignment, but useful in developing a three dimensional model of the ground within which the project would be constructed.

These boreholes have been drilled for a variety of purposes including for high and low rise buildings, tunnels, rail, sewers, roads, bridges and groundwater studies. Consequently, there is a variety of information presented on the borehole logs with varying degrees of quality. Some boreholes have been drilled specifically for the project (Stage 1, Stage 2 and more recent borehole investigations and information collected for the project up to September 2015) and these are considered more relevant and reliable in developing the ground model.

The usefulness of all boreholes used in developing the ground model depends not only on the reliability of the information presented on the borehole log, but also:

- the density of boreholes relative to the proposed alignment;
- the geological complexity and variability of the ground at the borehole location; and
- the sensitivity or susceptibility of the project to ground conditions.

The following describes the method that has been developed and used to assess the reliability of boreholes and in turn the reliability of the ground model. The ground model reliability is presented on the longitudinal sections, Figures 19 to 35 included in Appendix A.

BOREHOLE RELIABILTY ASSESSMENT

In order to assess the reliability of the boreholes, we have developed a simple method of assessment based on the borehole attributes set out in Table 3. For each borehole, numbers are assigned in each category which are then added to give a Borehole Reliability Score.

Further comments on the reliability of the categories listed in Table 3 are provided below:

Drilling Method – Boreholes with core drilling provide continuous samples which allows assessment of rock defects and greater detail of rock description as opposed to boreholes advanced using washbore or hammer drilling techniques.

Survey – Accuracy of survey allows the boreholes to be positioned at the correct location in ground models. Boreholes with recent ground survey are considered more accurate than those without survey. Where no survey is available, a site plan showing the borehole is geo-referenced against modern coordinate systems and the borehole locations estimated.

Sampling Frequency – The greater the sampling frequency in the borehole, the more reliable the soil or rock description is considered to be.

Age – There is greater uncertainty around the provenance of older borehole logs. Aspects such as the methods used to drill and describe the soil and rock, the training of the borehole loggers and the quality checks applied are uncertain.

Depth – Shallow boreholes are unlikely to provide useful information at the level of the proposed tunnel.

Installation – Boreholes with groundwater wells installed and associated groundwater measurements are considered more useful than boreholes without well installations.

In-situ Testing – Boreholes with insitu testing including SPT testing, packer testing or pressuremeter testing provide additional information on the geotechnical conditions.

There is a certain degree of subjectivity associated with the numbers assigned in each category. We have varied these parameters through a trial and error process in order to arrive at what we consider to be a reasonable representation of borehole reliability.

The reliability scores are then assigned to a category in accordance with Table 4. Similar to the parameters used to develop the reliability score, these can be varied. We have varied them through a trial and error process to develop the Borehole Reliability Ranking set out in Table 4. The implications of this borehole reliability ranking to future investigation and ground model development are also set out in Table 4.

Boreholes ranked 1 to 3 have generally been used to develop the ground models presented in this report, supplemented by information from boreholes with a lower ranking.

The cross sections in Appendix A include an indication of the borehole reliability spatially.

GROUND MODEL RELIABILITY

Ground model reliability is a function of the quality and quantity of ground information and the geological complexity at the location in which the boreholes have been drilled.

To assess information quality and quantity, the borehole reliability ranking has been used in conjunction with an assessment of the density of boreholes relative to the proposed alignments. Table 5 sets out the criteria used to assess borehole quality and quantity relative to a proposed alignment.

Table 3: Categories used to assess Borehole Reliability Score

Drilling Method		Survey		Sampling Frequency		Age		Depth		Installation		In situ testing	
Washbore and coring	5	Survey to MGA (AHD) inc. RL	5	< 1.5m	10	< 5 yrs	5	> 25m	10	Piezometer or well installation	3	SPT and packer, or pressuremeter testing	5
Washbore or hammer only	2	Survey to MGA (AHD), RL estimated from LiDAR OR Survey to AMG (AHD) inc. RL	4	>1.5 - 3m	6	>5 yrs - < 10 yrs	4	>10 – < 25m	6	No installation	0	SPT only	3
		Survey to AMG (AHD), RL estimated from LiDAR OR No Survey – Georeferenced from site plan	3	> 3m	2	>10 yrs – < 20 yrs	3	>5 m -< 10m	2			No insitu testing	0
		Converted from historical datum OR No survey – Located using georeferenced aerial imagery and Lidar	2			> 20 yrs	2	< 5m	1				

Table 4: Borehole Reliability Ranking

Borehole Reliability Score	Borehole Reliability Ranking	Implications
>34	1	Good, detailed information, known provenance, can be relied upon without need to undertake further investigation.
29 - 34	2	Good information, but information needs to be supplemented or verified through further investigation.
23 - 28	3	Information reliable, but shallow or lacking in detail. Supplemental investigation needed.
16 - 22	4	Provides some useful information, but insufficient detail or uncertain provenance. Not to be relied upon. New investigation needed.
<16	5	Minimal or no useful information, not to be relied upon. New investigation needed.

Table 5: Borehole Information Quality and Quantity

Very poor	No intrusive investigation or boreholes more than 100 m from the alignment, existing information limited to geological maps and publically available information.
Poor	Boreholes 50 m to 100 m from the alignment.
Fair	One or more boreholes within 50 m of the alignment, along a 100 m length. Borehole reliability ranking low, typically 4 or 5, some 3.
Good	Up to 5 boreholes within 50 m of the alignment, along a 100 m length. Boreholes have a high borehole reliability ranking, 1 or 2.
Very Good	More than 5 boreholes within 50 m of the alignment. Boreholes have a high reliability ranking, 1 or 2.

The geological complexity and project susceptibility to ground conditions has been assessed using the guidelines presented in Table 6.

Table 6: Geological Complexity and Susceptibility of Project to Ground Conditions

Very Simple	Single material type, no deformation, regular or repeatable structure, no discernible weathering. and/or Construction and structure proposed has a low susceptibility to uncertain or unexpected ground conditions. No significant consequences if unexpected ground conditions are encountered.
Simple	Single material type, no deformation, predictable structure, some chemical or mechanical weathering. and/or Proposed design and construction has some susceptibility to unexpected ground conditions, but these can likely be mitigated or managed through design or pre-planned contingency.
Intermediate	Multiple material types, single phase deformation, somewhat predictable structure, chemical and mechanical weathering. and/or Proposed design and construction is susceptible to unexpected ground conditions. There are expected to be implications if unexpected ground conditions are encountered during construction which may require design changes, remedial measures or delays during construction.
Complex	Multiple material types, single phase of deformation with unpredictable structures, multiple phases of chemical and mechanical weathering. and/or Proposed design and construction is susceptible to unexpected ground conditions with significant implications including project delays and cost overruns if unexpected ground conditions are encountered.
Very Complex	Many different lithologies, complex structure with multiple phases of deformation and metamorphism with complicated structure, multiple episodes of chemical and mechanical weathering. and/or Proposed design and construction highly susceptible to ground variation or unexpected ground conditions with major implications if unexpected ground conditions are encountered. Project delays, cost overruns, health and safety risks if unexpected ground conditions are encountered.

For TBM bored tunnels, the geological units listed below have generally been assigned to the categories set out in Table 7.

Table 7: Estimated complexity of geological units for TBM tunnels

Unit	Estimated Complexity		Unit	Estimated Complexity
Recent Silt (Qra)	Simple		Fill (Fill)	Very Complex
Coode Island Silt (Qhi)	Intermediate		Holocene Alluvium (Qha)	Intermediate
Jolimont Clay (Qpj)	Intermediate		Newer Volcanics Basalt (Qvn)	Intermediate
Pleistocene Alluvium (Qpa)	Intermediate		Fishermens Bend Silt (Qpfl)	Intermediate
Moray Street Gravels (Qpg)	Intermediate		Pleistocene Alluvial and Colluvial Sediments (Qpc)	Complex
Swan Street Basalt (Qvns)	Simple		Punt Road Sands (Qpp)	Intermediate
Brighton Group (Tpb)	Complex		Older Volcanics (Tvo)	Complex
Werribee Formation (Tew)	Intermediate		Devonian Granite (Dgr)	Intermediate
Melbourne Formation (Sud)	Complex			

Stations, including shaft excavations, cut and cover stations and caverns are assumed to be particularly susceptible to ground conditions and the complexity rating typically increased by one category.

The assessments of geological complexity and borehole quality and quantity are combined using the matrix in Table 8, to arrive at an overall ground model reliability ranking. Table 9 provides a general guide to the implications of the reliability score in terms of additional investigation that may be required.

Table 8: Ground Model Reliability Ranking

		Geological Complexity				
		Very Complex	**Complex**	**Intermediate**	**Simple**	**Very Simple**
Quality and Quantity of Data	**Very Poor**	VL	VL	L	L	M
	Poor	VL	L	L	M	H
	Fair	L	M	M	H	H
	Good	M	M	H	H	VH
	Very Good	H	H	VH	VH	VH

Table 9: Implication of Ground Model Reliability Score

Very low (VL)	Available information insufficient given the geological complexity to develop a basic conceptual model. Indicative only, should not be relied upon.
Low (L)	Available information sufficient given the geological complexity to develop a basic conceptual ground model. Insufficient to develop an observational model.
Medium (M)	Sufficient information given the geological complexity to develop an observational model. Significant uncertainty requiring further investigation or risk management.
High (H)	Sufficient information given the geological complexity to develop an observational ground model. Some uncertainty requiring further investigation or risk management.
Very High (VH)	Able to develop detailed observational ground model. Sufficient information given the geological complexity to identify ground related uncertainty.

A long section along the proposed alignment showing the ground model reliability ranking is presented in Figures 19 to 35 included in Appendix A.

4.0 REGIONAL GEOLOGY AND REGIONAL STRUCTURE

This section presents a background to the type and origin of materials expected to be encountered along the proposed alignment of Melbourne Metro.

4.1 Geological History

The subsurface materials expected to be encountered along the proposed Melbourne Metro alignment are variable and relatively complex. The distribution and engineering properties of these materials are a function of their geological history and the geological evolution of the Yarra Delta area. A brief, simplified summary of the geological evolution of the materials anticipated to be encountered is presented below to provide a context to the subsequent discussion of local geology.

The Silurian age (440 to 416 Million years ago (Ma)) *Melbourne Formation* forms the bedrock along almost all of the alignment. This material is a sedimentary rock comprised of sandstone and siltstone in beds of a few hundred millimetres to a few metres thick. Bedding planes are typically persistent. It was subject to east-west compressive regional tectonic deformation during the Devonian (416 to 359 Ma) which folded and faulted the bedded sedimentary material. Within the vicinity of the proposed alignment, north to south (approx N20˚E) trending fold axes with spacing of 1 km to 2 km are present. However, much smaller scale parasitic folds, with fold axes having a spacing of tens of metres have developed on the limbs of the larger folds. Typically the parasitic folds are open to isoclinal, parallel folds.

The sandstone beds are more competent than the siltstone beds (have a higher stiffness) and responded differently to the tectonic compression compared to the siltstone beds. Whilst the sandstone tended to fracture in a more brittle fashion when folded, the siltstone tended to bend. As a consequence, the sandstone beds in general tend to be more fractured.

Steeply dipping normal and reverse faults, trending in a similar direction to the fold axes also developed. Persistent, planar joints developed, with spacing in the order of tens of millimetres up to metres. Granite bodies intruded into the Silurian rock mass during this time. Granite intrusions are present in the South Yarra area, east of the proposed Melbourne Metro alignment. Associated with this, mineral rich fluids migrated upwards through the rock mass, generally along open discontinuities such as joints, faults and bedding planes. As the fluids rose through the rock mass, the pressure and temperature to which they were subject dropped and the minerals crystallised to form igneous dykes. The dykes typically follow the orientation of discontinuities, including faults and fold axes and may be present within the Melbourne Formation under all parts of the alignment.

Over the next 300 million years, the Melbourne Formation materials were subject to a long period of weathering and erosion. Drainage courses formed, carving topography within the siltstone and granite. Chemical weathering processes in the early Tertiary converted the rocks to clay materials (principally kaolinite) and leached out iron. Fresh water lakes and swamps covered the Melbourne area during the early to mid Tertiary (65.5 Ma to 15 Ma) leading to the deposition of sediments, including clay, sand and silt within the valleys and lower lying areas. Plant matter accumulated in some areas leading to the deposition of peat and coal. The material deposited is known as the *Werribee Formation*.

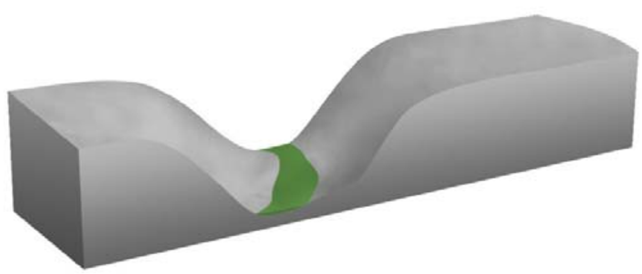
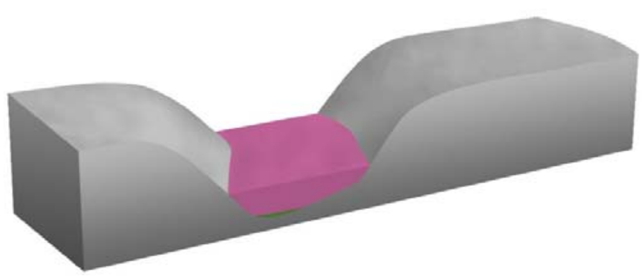
Volcanic activity followed deposition of the Werribee Formation. Ash falls covered some areas and lava flows covered areas of the Melbourne and Werribee Formations. These materials are known as the *Older Volcanics*. A further period of weathering and erosion occurred, converting much of the Older Volcanics to a clayey material containing higher strength basalt corestones, carving shallow valleys within it and removing it completely over some areas. Subsequent sea level rise caused the deposition of marine sediments (*Newport Formation*) within valleys and coastal embayments. As the sea level dropped, sandy materials were in turn deposited over the earlier materials (*Brighton Group*). Also during this time, most of Victoria experienced tropical temperatures and high humidity, leading to the development of a deep lateritic weathering profile. The Tertiary and Silurian materials exposed to the atmosphere at this time were subject to a deep lateritic weathering, less profound than the Early Tertiary weathering, but leading to deep weathering, alteration of minerals to clay (principally kaolinite-illite) and leaching of iron upwards to form limonite and goethite along discontinuities.

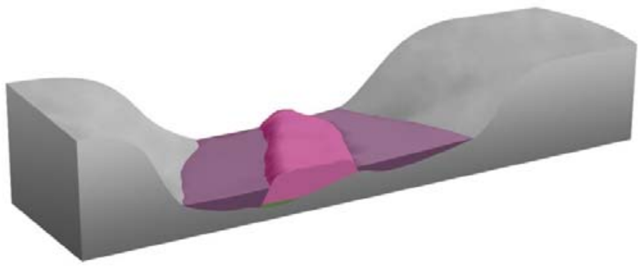
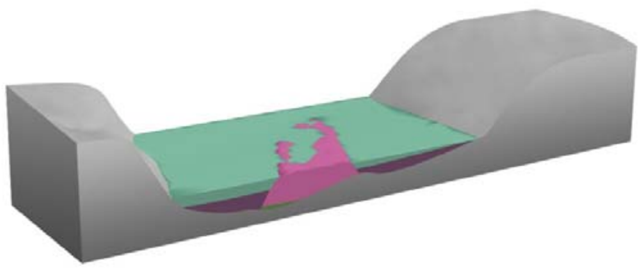
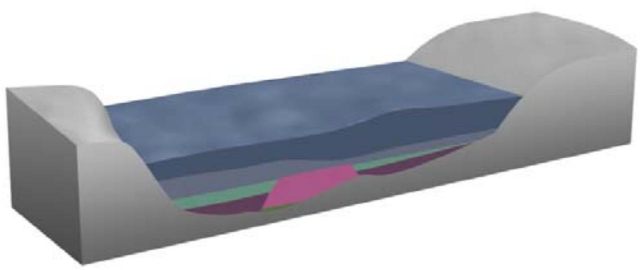
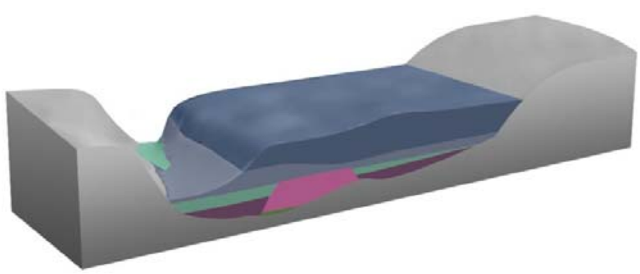
Although it has a low seismic activity, the Australian plate is subject to tectonic forces and faults within the Melbourne Area, including the Selwyn (east of Melbourne) and Rowsley (west of Melbourne) Faults which are subject to ongoing movement. Towards the end of the Tertiary (2.65 Ma), this movement produced normal faulting and some reactivation of existing discontinuities. The Melbourne Warp, a gently folded monocline (inferred to be a reactivation of a structure formed during the Devonian) formed south east of Melbourne, and is expected to underlie the southern part of Melbourne Metro alignment. The Silurian and Tertiary materials developed gentle dips (about 10˚) and lowered the topography within what is now the South Melbourne area. Northeast to southwest trending normal faults are typically associated with the Melbourne Warp.

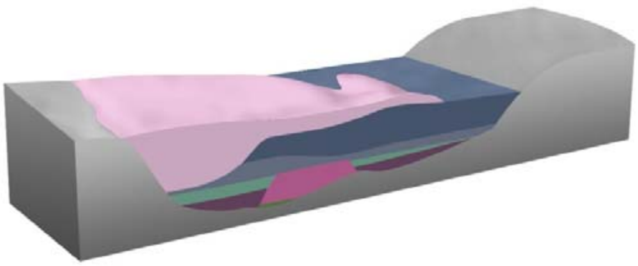
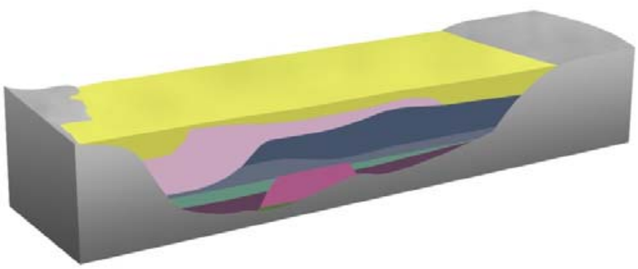
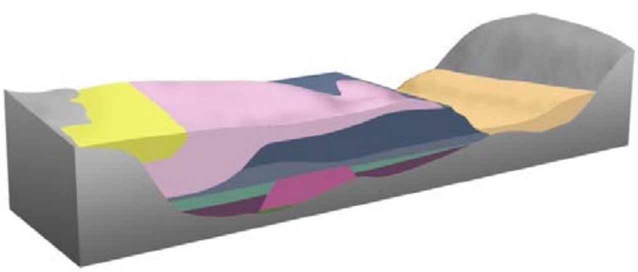
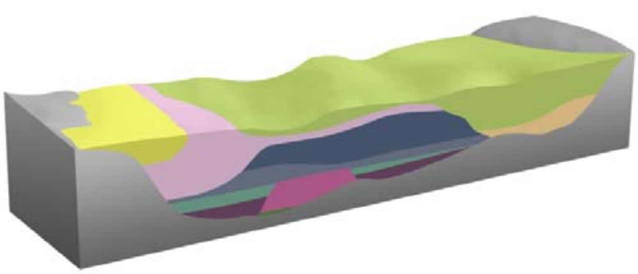
Ice ages throughout the Pleistocene (1.8 Ma to present), subsequent to the deposition of the Tertiary materials described above, induced relatively rapid, cyclical sea level rise and fall, with sea level rising and falling by as much as 120 m. At times of high sea level, marine sediments such as clays and sands were deposited in flooded lower lying areas. At times of low sea level, fast flowing rivers and streams carved steep sided valleys into the Silurian and Tertiary materials. As the valleys steepened, the sides collapsed leaving deposits of rock and soil on the valley sides and floor (colluvium). The rivers transported and redeposited this material leaving deposits of alluvium on the valley floor.

In between the rising and lowering of sea levels, volcanic eruptions occurred to the north of Melbourne and lava (basalt) flowed down creek valleys such as the Merri Creek Valley and into the Jolimont Valley (Yarra River Valley). With cycles of sea level rise and fall, valleys were carved into the pre-existing sediments and then backfilled with marine sediments. Plate 1 presents a schematic showing the evolution of the Jolimont Valley. We note that the evolution of the Moonee Ponds Creek Valley is similar to that of the Jolimont Valley.

A commentary for the evolution of the Jolimont Valley is given in Plate 1 (1a to 1k).

	Plate 1a - approx. 1.8 Ma to 1.3 Ma Period of deep down cutting into the Melbourne Formation during glacial sea level lows. Fast flowing rivers carved the Jolimont Valley (ancestral course of the Yarra River), Maribyrnong and the Moonee Ponds Creek Valleys. Colluvial and alluvial materials, boulders and gravels of siltstone, river gravels and sands were deposited on the valley walls and floors, depositing the Punt Road Sands.
	Plate 1b - approx. 1.3 Ma Volcanic eruptions to the north of Melbourne caused lava to flow down the Jolimont Valley, emplacing the Swan Street Basalt. The basalt covered and preserved the Punt Road Sands at the base of the valley.

	Plate 1c - 1.3 Ma and 0.9 Ma, the Swan Street Basalt was eroded. Aggressive periods of erosion occurred at around 1.1 Ma and 0.9 Ma during glacial minimums, removing much of the Swan Street Basalt, widening the valley and leading to the deposition of the Colluvium and Alluvium (Princes Bridge Sediments).
	Plate 1d – 0.9 Ma to 0.85 Ma, during a period of low sea level, the Jolimont Valley further widened and Moray Street Gravels were deposited.
	Plate 1e - 0.85 – 0.83 Ma, the sea level rose rapidly flooding the Jolimont Valley and depositing marine sediment, the Fishermens Bend Silt. Coarse, sandy materials were deposited initially, which were overlain by finer clayey sediments as the sea deepened.
	Plate 1f - 0.83 Ma years to about 0.81 Ma years ago sea level dropped, and the Fishermens Bend Silt was subject to a period of erosion, forming shallow channels in its surface and a main channel on the north side of the valley. The Fishermens Bend Silt was exposed to the atmosphere causing oxidation and orange staining through the upper 10 m to 15 m. The draining of this material induced consolidation, which stiffened it, and also lead to the development of fissures.

	Plate 1g - 0.81Ma, a second lava flow flooded the Jolimont Valley (Burnley Basalt). Typically, the flow is at least 7 m to 8 m thick, which it may have needed to be, in order to retain sufficient heat to flow. At points of constriction in the valley, the lava overspilled, leaving thin lobes of basalt outside of the main channel.
	Plate 1h – 0.81 Ma to 0.17 Ma - cycles of erosion and deposition occurred after the emplacement of the Burnley Basalt. Shallow channels eroded at the edge of the basalt, cutting into the softer siltstone. During one of the periods of high sea level, the Jolimont Clay was deposited. It is noted that elsewhere in the Yarra Delta, Fishermens Bend Silt is inferred to have been deposited during this period (Holdgate 2001). Cupper et. al. (2003) suggested that the Jolimont Clay was deposited as an almost continuous unit above the Fishermens Bend Silt and Burnley Basalt. Within the Jolimont Valley, the Jolimont Clay appears to be a channel infill, separated from the Burnley Basalt by a period of erosion.
	Plate 1i - Between about 0.17 Ma ago and the peak of the glacial maximum, 18,000 years ago, there was an aggressive period of erosion associated with the last glacial minimum. The Jolimont Clay was drained, causing it to consolidate and stiffen. A deep valley was carved to the south of the Jolimont Valley, extending from the edge of the Burnley Basalt, down into the siltstone. Colluvial material was deposited on the sides and base of these valleys.
	Plate 1j - Sea level rose between 18,000 years ago and the present. Initially, the Jolimont Valley filled with coarse material (Batman Avenue Gravels) and subsequently finer clayey material. This material, the Coode Island Silt, has never been significantly drained and remains normally to slightly overconsolidated.

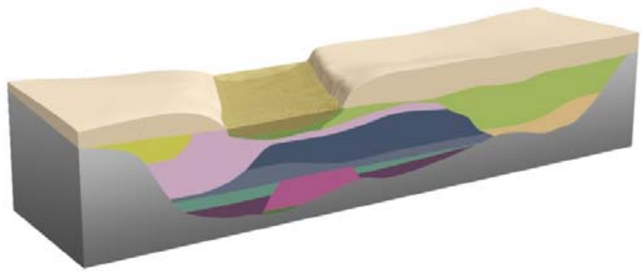
	Plate 1k - In the past 150 years, the landscape has been modified by humans. This has included realignment of the Yarra River, deepening of the river channel, filling of low lying ground and some lowering of groundwater levels.

Plate 1: Evolutionary Model of the Jolimont Valley (present day Yarra River) (Paul et. al 2014)

4.2 Regional Structure

As discussed above, the Melbourne region was subject to compressional tectonic deformation (Tabberabberan Orogeny) during the Devonian (380 Ma). The only rock type expected to be encountered along the Melbourne Metro alignment that was affected by this deformation is the Melbourne Formation.

In the Melbourne area, this event resulted in uplift and deformation, producing open folds of variable spacing (1 km to 2 km typically), separated by smaller parasitic folds.

Whiting (1967) describes the geological structure of the Melbourne Formation within the Melbourne metropolitan area as follows:

'The general structure is shown to consist of concertina type folding of an average wavelength of about three quarters of a mile. Four anticlinora and four synclinoria are recognisable. The general strike of the folds over most of the Melbourne area is N20˚ – 25˚E.'

Grainger (1992) in his summary paper on 'Geological Structure' confirms this, viz;

'…within the Melbourne metropolitan area, the average wavelength of the folds is just over a kilometre and the folds have a north-north easterly strike varying from 010 degrees to 025 degrees'.

Major fold axes indicated on the 1:63,360 geological mapsheet of Melbourne are shown on the geological plans presented in Appendix B. Plate 2 below indicates typical structural features present within the open folds of the Melbourne Formation (adapted from Fookes 2000).

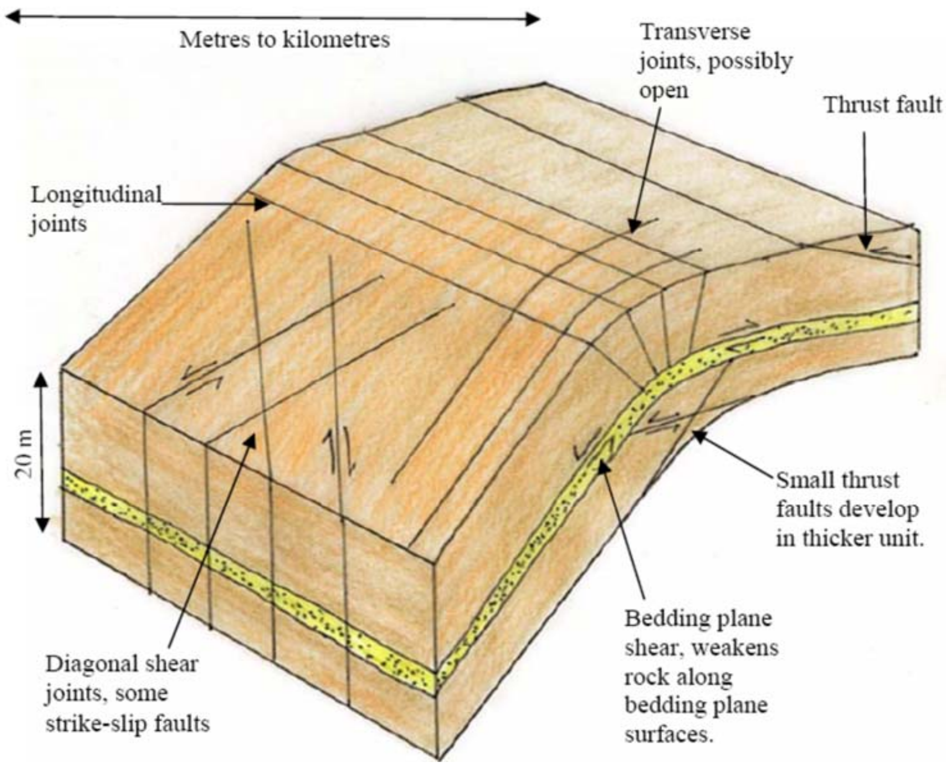


Plate 2: Features Typical of Open Folds in Sedimentary Rock (after Fookes, 2000)

Major faulting of the Melbourne Formation occurred concurrently with the folding during the Tabberabberan Orogeny. In the Melbourne area, the faults are generally steeply dipping normal and reverse faults. Of note, is the Melbourne Warp. This was initially identified as a Tertiary age feature; however, it is thought to have developed during the Devonian Tabberabberan Orogeny and reactivated during the Tertiary. The axis of the warp within the vicinity of Melbourne Metro is shown on the geological plans presented in Appendix B which have been adapted from the 1:63,360 geological mapsheet of Melbourne. The warp is described in Birch 2003, as a minor NW trending flexure passing through the western part of the CBD. It has gently downwarped Tertiary formations and the bedrock to the southwest to form a depression in which the Yarra Delta accumulated.

The Melbourne Warp does not outcrop at the ground surface. However, it has been recognised in subsurface construction including the Dandenong Trunk Sewer and South Eastern Trunk Sewer as a 300 m to 350 m wide fracture and shear zone with intruded igneous dykes of up to 10 m thickness. Bedding plane shears and crushed zones are common within it. We are unable to source documentary evidence indicating the northern extent of the Melbourne Warp, or assess whether the proposed alignment would intersect the warp. However based on its indicated position, it is possible that it may be encountered in the base of the proposed excavation for Domain Station, or by its proximity, reduce the local rock quality. We note that boreholes drilled within the Fawkner Park area as part of the Stage 2 geotechnical investigation did not encounter subsurface materials indicative of the Melbourne Warp, despite the geological map indicating that the Warp is very close to the location of some of the Stage 2 boreholes.

The extensional deformation which occurred during the Tertiary, led to uplift and tilting of the block to the east of the Melbourne Warp. The Tertiary surface developed a gentle dip towards the west, typically less than 10˚, forming the Port Philip basin into which the Yarra Delta sediments were later deposited. The Tertiary deformation, although relatively minor in comparison with the Devonian deformation, caused fissures to develop within the Tertiary materials including the Werribee and Brighton Group sediments.

5.0 GEOLOGICAL MATERIALS

The following provides a discussion on each of the geological units expected to be encountered along the proposed Melbourne Metro Concept Design alignment and its anticipated geological and engineering characteristics. The Units are discussed from the oldest, and typically lowest, unit upwards.

The proposed alignment is superimposed on the geological long sections in Appendix A to provide an indication of the materials through which the proposed alignment is anticipated to be excavated.

5.1 Silurian Melbourne Formation (Sud)

The Silurian age Melbourne Formation is present as the bedrock along most of the Melbourne Metro alignment. The Jolimont and Moonee Ponds Valleys cut down into this formation and are infilled with Quaternary and Tertiary rocks and soils. As such, and with reference to the long sections presented in Appendix A, the proposed Melbourne Metro tunnels are expected to be bored (at least partly) through Melbourne Formation in segments: 5, 9, 10, 11, 13, 15, 17, 18, 19, 21 and 22.

The station excavations at Parkville (Segment 10), CBD North (Segment 12), CBD South (Segment 14) and Domain (Segment 20) are expected to encounter the Melbourne Formation. The Melbourne Formation is expected to be the most commonly encountered material on Melbourne Metro. It is also the most commonly encountered rock type in buildings and infrastructure development within the Melbourne CBD, and as a result, there is probably more existing information available about this material than the other rock and soil types expected to be encountered on the alignment.

We note that at some locations, the proposed alignment is very close to an interface between the Melbourne Formation and other geological units. For example, Segment 8, the proposed alignment, whilst within Melbourne Formation materials, runs very close to the base of Werribee Formation sediments. Another example is in Segment 21 where the base of the alignment runs very close to geological boundary with the Brighton Group.

The following describes geological aspects of the Melbourne Formation including material, mass and weathering characteristics we consider relevant to Melbourne Metro.

5.1.1 Material Characteristics

The Melbourne Formation comprises interbedded Siltstone and Sandstone. Sanders (1992) estimates that about 25% of the Formation comprises Sandstone, and the remainder comprises Siltstone. The siltstones occur in thin beds typically tens to hundreds of mm thick. The sandstones are fine to medium grained and occur as thin laminations within the Siltstone or beds of up to 1.6 m thick, although commonly 200 mm to 300 mm thick. Turbidite sequences (sediments deposited via underwater debris flows) have been identified within the Melbourne Formation. The boreholes drilled specifically for Melbourne Metro encountered about less than 10% of material described as sandstone, although no clear pattern of distribution of sandstone is evident.

The mineralogy of the Melbourne Formation in its unweathered state typically comprises a mineral assemblage principally of quartz (35%), mica (20%), kaolinite (25%) and chlorite (20%) (Neilson 1970).

The Stage 1 investigation results indicate, on the basis of petrographic descriptions, that equivalent quartz content is up to 59% (note that this measure is different to the pure quartz content, and suggests a similar actual quartz content). During the process of chemical weathering, the non-quartz minerals typically alter to clay minerals (described subsequently). The chlorite tends to give the fresh rock a dark blue-grey colour.

Petrographic analyses of siltstone samples obtained from the ongoing investigation indicate a quartz content of 34% to 80%, comprised of predominantly silt sized grains. The composition of the remainder of the samples analysed was found to comprise clay minerals and limonite (more weathered samples). In laminated samples of Melbourne Formation, the proportion of quartz to clay minerals varies over several centimetres. Furthermore there can be relatively massive sandstone beds present with high quartz content. Where present, sandstone is expected to have higher quartz content. The distribution of siltstone and sandstone and quartz content is difficult to predict.

Within the vicinity of Devonian Granite intrusions, the sandstone and siltstone underwent contact metamorphism. This process alters the mineralogy of the material, generally causing an increase in strength. The southern part of the Melbourne Metro alignment approaches a Devonian Granite intrusion which underlies part of Albert Park, Fawkner Park and South Yarra. Borehole GA11-BH25 (Stage 2 investigation), which is the borehole drilled closest to the inferred location of the South Yarra granite intrusion, shows some evidence for contact metamorphism, however, the alteration appears to be comparatively mild. Boreholes GA15-BH15, near the eastern portal in Segment 23 encountered the granite pluton suggesting the zone of metamorphism around the pluton may be relatively narrow.

5.1.2 Mass Characteristics

The regional deformation that occurred within the Devonian led to the development of folds, bedding plane slips, several joint sets and minor faulting. The discontinuities expected to be encountered within the Melbourne Formation are discussed further below:

Joints and Bedding Planes

Joints within the Melbourne Formation are predominantly a result of the stresses acting during periods of folding and faulting and to a lesser extent due to igneous intrusions. It is common for three or four joint sets to be recognised, including those developed along bedding planes. However, joint orientations tend to be localised and it has not been possible from investigation undertaken specifically for the project to recognise a large scale joint set across the broader geological unit. In areas at or adjacent to folds or faults, the number of joint sets and frequency of joints typically increases. The following extract is taken from the 1966 ‘Report on Geological Investigations for the City of Melbourne Underground Railway’:

The dominant jointing is related to the folding and may be classified by relation to the bedding into the groups defined by Table 9.

Table 10: Silurian Rocks - Relation of Dominant Jointing to Folding

Classification Type	*Term used in text*	*Detailed Description*
A	*Bedding Plane Joints*	*Partings along bedding planes*
B	*180° Joints*	*Joints of parallel strike but perpendicular dip to the bedding planes*
C	*90° Joints*	*Joints with strike perpendicular to the bedding strike and with subvertical dip*

‘The overall average joint spacings of Silurian rocks in the fresh state appear to be of the order of six inches; a significant degree of “opening up” only takes effect when the rock has reached the highly weathered state.’

Experience gained from other tunnelling and building projects within the Melbourne metropolitan area suggests that the joint spacings are typically in the range of 300 mm to 750 mm, although in folded or faulted zones, such as the Melbourne Warp, spacing can reduce to as low as 10 mm to 100 mm.

In slightly weathered to fresh rock, joint planes are often clean, may have limonite staining or a pyrite coating. Occasionally joints are covered with cemented quartz, with calcite and chalcopyrite, chlorite and gypsum are also sometimes present. Joints parallel to bedding tend to have a rough surface and may be open due to slip between beds of higher and lower stiffness. Friction angles as low as 12 degrees have been measured on bedding planes within the Melbourne Formation.

Joints may be ‘refracted’ where they cross from the stiffer and less ductile sandstone beds into the more ductile siltstone beds. Typically, due to its more brittle behaviour during deformation, the sandstone has a higher concentration of joints than the siltstone.

The Stage 1 boreholes included measurement of joint orientations within the Melbourne Formation, including televiewer surveys in eight boreholes. Within these boreholes, bedding planes, joints and shears were identified. Although, the joint orientations have been measured in a very small sample of the rock mass, the results suggest that in these borehole locations:

- There is a wide range of bedding orientations which vary from hole to hole due to the folded nature of the rock mass.
- Joint orientation is highly variable, with stereographic projections of joint measurements showing appreciable scatter. However, some of the joint measurements suggest jointing that is approximately orthogonal to the bedding.

Joint orientations were also measured in currently ongoing stage boreholes using acoustic televiewer imagery. These measurements indicate that bedding is typically the predominant joint set and that the bedding orientations are relatively uniform within a single borehole. Joints are less prevalent, and although some joint sets are evident, there appears to be greater variability of joint orientations.

Major structures and bedding orientations identified through the desktop audit and geotechnical investigations are shown on the geological plans in Appendix B.

Faults

Faults, which typically present as sheared zones or crushed seams, are encountered where movement along a discontinuity has occurred. Crushed seams consist of material with soil properties (i.e. caused by crushing during faulting and usually affected by subsequent weathering), whereas sheared zones are typically zones with closely spaced (usually smooth or polished) joints. Sheared zones are usually wider than crushed seams, and some sheared zones may include crushed seams. Slip between bedding planes typically causes crushed seams to form.

In fresh to slightly weathered rock, crushed seams consist mainly of angular gravel or sand with lesser silt or clay. Some thinner seams may comprise gravelly clay or low plasticity clay. Near surface, sheared and crushed seams tend to be more weathered than the surrounding rock and may be referred to as extremely weathered seams. In this case they would typically comprise very stiff to hard silty clay.

Near surface, infilled seams may be present whereby soil has migrated into open discontinuities within the rock mass. Most infill seams are up to 10 mm thick, however seams several metres thick can occur in weathered parts of the rock mass.

Plate 3: Outcrop of Melbourne Formation (Golder archives)

5.1.3 Weathering

Chemical weathering within the Melbourne Formation (whereby minerals within the rock alter to clay) can extend tens of metres below the ground surface. The degree and type of chemical weathering present in the Melbourne Formation is dependent upon the climate and available water and oxygen to which it has been subject at various times during its history. The two main periods of chemical weathering are described below:

Devonian to Tertiary Weathering

The Melbourne Formation was subject to a long period of weathering and erosion between the end of the Devonian (360 Ma) through to the late Tertiary (about 5.5 Ma). Alteration of chlorite to kaolinite and leaching out of iron and silica produced pale grey to white kaolinite rich soils. Most of this material was subsequently eroded within the Melbourne area and it is typically now only found where it has been preserved due to capping by Tertiary materials, principally Older Volcanics, emplaced about 34 Ma.

Late Tertiary Weathering

A hot, wet climate induced lateritic type weathering whereby the chlorite and mica was altered to kaolinite, illite and hydrous micas, with iron leached upwards. The iron when exposed to oxygen higher in the weathering profile typically oxidised to form goethite and hematite on joint surfaces. This can create a cementing effect, 'healing' the joints. With depth in the rock mass, the degree of alteration typically reduces, and the rock darkens in colour. However, fractured zones such as near fold axes and faults tend to have undergone a higher degree of weathering than intact rock. This period of weathering also affected the Older Volcanics materials, discussed in Section 7.4 of this report.

Where the Melbourne Formation was covered with soils, such as the Werribee Formation or Moray Street Gravels, weathering still occurred. However, the lack of available oxygen prevented full alteration of chlorite, and typically the rock preserves the blue-grey colour that it has when in a fresh state.

Weathering Grades

A weathering classification system for the Melbourne Formation was developed by Neilson (1970) and remains in common use today. Table 10 below has been adapted from this classification and used subsequently in this report. Whilst this classification is similar to that presented for rock in Australian Standard 1726 on Site Investigation, it deviates from the Standard because with the inclusion of categories for both Highly Weathered and Moderately Weathered (grouped together as Distinctly Weathered under the standard).

Table 11: Weathering Grades within Melbourne Formation

Degree of Weathering	Material Description	Typical Colour	Moh's Hardness	Reaction to blow from hammer	Visibility of Bedding
Extremely	Silty clay or sandy clay. May contain harder rock fragments.	Yellow-brown	Max 0.5	Hammer indents	Bedding indiscernible
Highly	Very low to low strength siltstone and sandstone, with clay seams common. Clay is often from decomposition of mudstone beds; often in joints, with iron oxide also.	Yellow-brown	0.5 – 1.0	Shatters easily with light blow	Bedding somewhat discernible
Moderately	Low to moderate strength siltstone and sandstone. Thin mudstone bands weathered to clay are known but uncommon. Joints sometimes carry thin clay deposits, or often iron oxide.	Pale brown and pale grey, mottled	1.0 – 1.5	Only fractures with light blow. Shatters with fairly heavy blow.	Bedding mostly discernible

Degree of Weathering	Material Description	Typical Colour	Moh's Hardness	Reaction to blow from hammer	Visibility of Bedding
Slightly	Moderate strength mudstone. Joints sometimes contain thin clay films and often iron oxide staining.	Pale grey	1.5 – 2.5	Shatters only with very heavy blow.	Bedding clearly visible
Fresh	Moderate to high strength mudstone. Joints clean or with pyrite films or occasionally calcite.	Dark blue-grey	>2.5	Fractures, but does not shatters by very hard hammer blow.	Bedding clearly visible

5.1.4 In Situ Stresses

Gibson and Peck (1992) in their review paper on earthquake hazards and ground stress in the Melbourne Formation note that the first local measurement of ground stress occurred during preliminary investigations for the Melbourne Underground Rail Loop project in the late 1960's. These were flatjack rock stress measurements and were reported by Hurse (1966). The results suggested that in situ horizontal stresses exceeded the vertical stress, but these results are considered anomalous because the vertical stresses were greatly in excess of the overburden pressures.

Other measurements of natural insitu stresses were made during the construction of the South East Trunk Sewer at East Malvern. The results were reported by Worotnicki (1976) and others. Gibson and Peck (1992) report that the results of the flatjack tests indicated the vertical stress agreed well with the weight of overlying rock and that the maximum in situ horizontal stress was 1.5 times the vertical stress.

The direction of maximum principal horizontal stress is reported as roughly east-west. Gibson and Peck (1992) also report that earthquake data suggests compressive stresses are acting in a southeast to northwest direction.

Two in situ stress measurements reported by the Technical Advisor and using borehole hydrofracturing during the Stage 1 investigation suggest a maximum horizontal stress of 4 to 5.2 times the vertical stress, with an approximate north-south orientation for the maximum principal stress. This result is not consistent with the other measurements of in situ stress, or with the anticipated stresses in the Melbourne Formation. A lower ratio of horizontal to vertical stress is expected to be typical.

In situ stress testing was undertaken in seven boreholes during the RD stage investigation using the Sigra overcoring method. The magnitude of the major and minor stresses measured and the orientation of stresses measured is presented in Plates 4 and 5. Tests adjacent to existing structures which could have influenced results are highlighted.

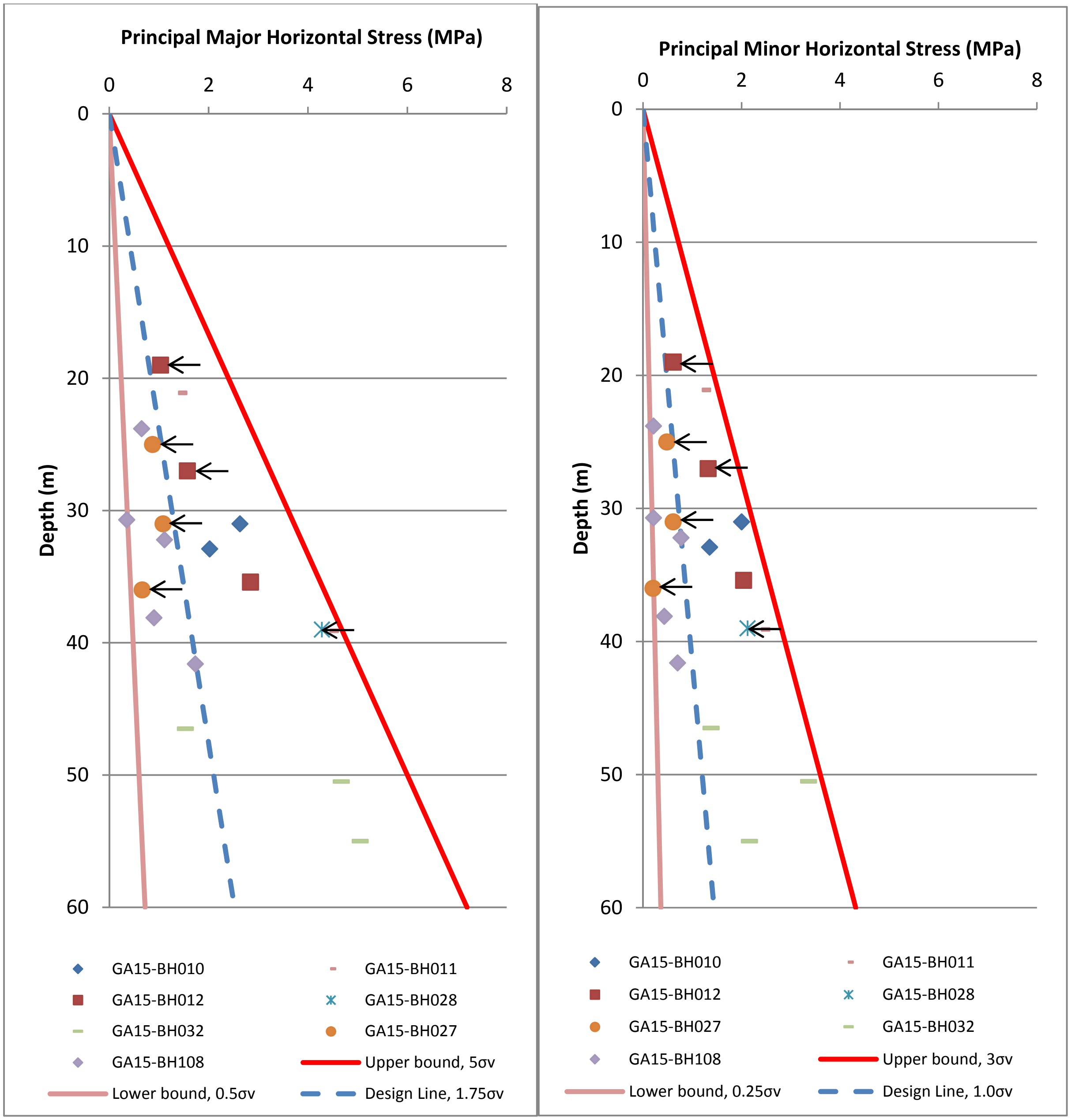


Plate 4: In Situ stress measurements undertaken during ground investigation in 2015. Tests within the zone of influence of existing underground structures indicated with arrows

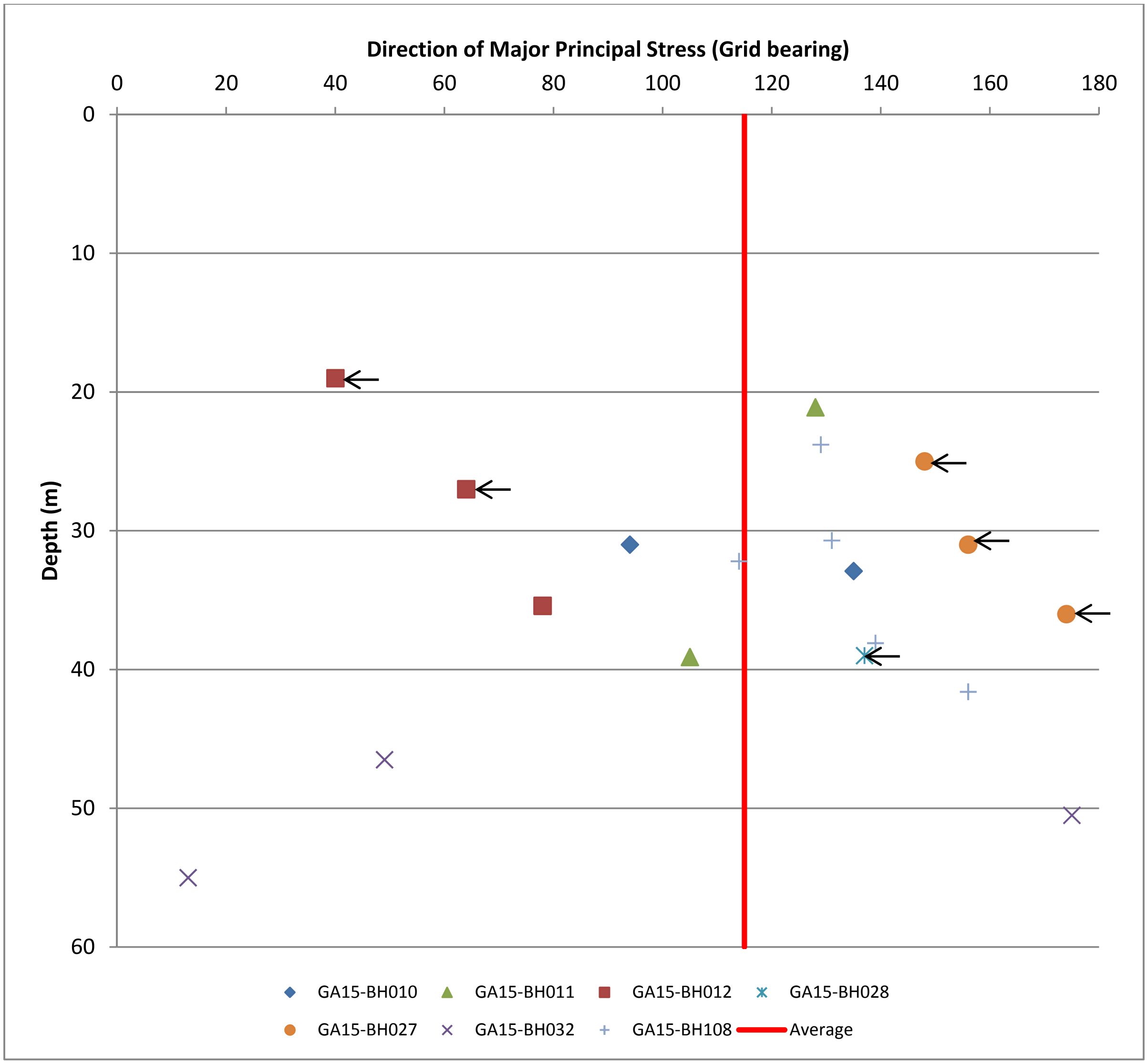


Plate 5: Orientations of in situ stress measurements made during ground investigation in 2015. Tests within the zone of influence of existing underground structures indicated with arrows.

5.1.5 Variability

Information obtained along the length of the Melbourne Metro Concept Design alignment indicates that the material, structure and degree of weathering of the Melbourne Formation vary considerably along the alignment. For example, the Melbourne Central station mapping information indicates highly folded material with faults and fractures. Within the vicinity of Parkville, the Melbourne Formation is less fractured, with massive sandstone beds. Whilst the information available for this report does not allow the spatial variability to be predicted with accuracy along the Concept Design alignment, there is sufficient understanding of this variability for the purpose of developing the EES.

5.2 Devonian Intrusions

During the Devonian (416 Ma to 359 Ma), the Melbourne Formation was subject to compressional tectonic forces. Granite intruded into the rock mass during this time, and fluid dykes, associated with the granite intrusions, intruded along fractures and weak zones within the rock mass. The proposed Melbourne Metro tunnels may encounter dykes at any location within the Melbourne Formation. At least one significant dyke was encountered in Borehole GA15-BH007, at the northern end of the CBD North Station (Segment 12). A number of dykes and sills were also mapped in the station walls for Museum Station (now Melbourne Central) and in the adjoining running tunnels during construction of the City Loop.

5.2.1 Material Characteristics

The intrusives are mostly quartz porphyries, feldspar porphyries and lamprophyres. Typically the quartz and feldspar porphyries are light coloured high strength rocks when slightly weathered, compared to the lamprophyres which are dark coloured. The dykes are igneous rocks, and when in their fresh state they are comprised of interlocking minerals, similar in appearance to granite.

5.2.2 Mass Characteristics

Previous major tunnelling works suggest that sills and dykes are mostly associated with fault and fracture zones, and fold axes. In our experience, dykes are associated with fold axes. The locations of some fold axes are presented in Appendix B, based on historical mapping by the Geological Survey of Victoria, the detailed mapping which was completed during construction of the City Loop and the Melbourne Metro site investigation results available for the EES.

The following is reproduced from the 1966 'Report on Geological Investigations for the City of Melbourne Underground Railway'.

Igneous dykes intrude the Silurian rocks in several places along the route of the Railway near Russell Street, Exhibition Street and beneath the Public Works Department Offices, Parliament Place. They are hypabyssal intermediate or acid igneous rocks; a typical specimen was classified as a pale green gray propylitised quartz feldspar mica porphyry. They are believed to have been intruded ahead of granite magmas during the Middle Devonian, when strong East-West compressions folded the Silurian rocks and formed the conjugate joint set Types D and E, (Strike 130° and 040°), associated with which the dykes are most often found. The dykes appear to reach their greatest thickness (in excess of 40 feet (12 m)) when in the 130° and 040° directions, but in places may be diverted into a roughly North-South direction, subparallel to the fold axes. The overall predominance of roughly North-South striking joints in the Silurian rocks possibly will result in many thin dykes or hydrothermally altered zones parallel to these joints ("bedding planes" or 180° joints") in the vicinity of the thicker dykes.

Whilst the orientations of the dykes as described above are generally consistent with our experience, it should be noted that their distribution and orientation can have significant variability, with intrusions along bedding planes and minor defects with variable orientation also occurring.

5.2.3 Weathering

The igneous materials comprising the dykes are more susceptible to chemical weathering than the Melbourne Formation materials into which they have been intruded. As a result, chemical weathering along dykes typically penetrates deeper into the rock mass than weathering within the siltstone materials. In some cases this can result in extremely weathered dyke materials being encountered within moderately or slightly weathered siltstone materials.

The dyke materials typically alter to white kaolin clay, containing grains of quartz, which does not alter with chemical weathering. Often the weathered dykes have orange iron oxide staining. The weathered clayey product of the dyke material is generally more uniform than that of the host sedimentary rocks, probably due to their more homogenous nature.

We are not aware of a weathering classification system that has been developed to describe these igneous rocks. Local experience, suggests where dykes are encountered, the weathering classification system as described previously for the Melbourne Formation is typically adopted.

An example of a dyke exposed within an excavation in the Melbourne Formation is presented in Plate 6.

Plate 6: Weathered dyke exposed in open excavation within Melbourne Formation (Golder archives)

5.2.4 Granite Intrusions (Dgr)

As noted above, dyke intrusions are typically associated with granite intrusions. Based on the geological model presented in Appendix A, we do not expect the Melbourne Metro tunnels to encounter granite intrusions. However, a granite intrusion is known to be present nearby, with its northern boundary running between the corner of Toorak Road and Chapel Street and Commercial Road and St Kilda Road and it was encountered in borehole GA15-BH035. Extremely weathered granite was encountered at a depth below the proposed alignment and is not expected to be encountered.

Where the granite has intruded into the Melbourne Formation, contact metamorphism occurs. This tends to alter minerals within the Melbourne Formation, typically strengthening the rock (the metamorphosed rock is known as hornfels). The easternmost borehole drilled as part of the Stage 2, GA11-BH025 (100 m from where granite has been confirmed) encountered material with some evidence for contact metamorphism. However, the material observed in the borehole was deeply weathered and the proposed alignment is not expected to encounter Melbourne Formation materials which have been subject to significant contact metamorphism.

Plate 7 presents an example of a granite outcrop near Mount Eliza. The South Yarra Granite does not outcrop, however, the Mount Eliza granite is of a similar age and composition.

Plate 7: Outcrop of Extremely Weathered Granite, Mount Eliza (Golder archives)

5.3 Tertiary Werribee Formation (Tew)

Following the tectonic compression and intrusion of dykes and granites during the Devonian, a long period of weathering and erosion occurred, leaving an undulating topographical surface. During the Tertiary (about 35 Ma), sediments (sand, silt and clay) were deposited over the Melbourne Formation via continental lakes, rivers and lagoons, emplacing the Werribee Formation. This formation is commonly encountered within the Yarra Delta.

The presence of the Werribee Formation along the proposed alignment was confirmed during the Stage 2 investigations. The Werribee Formation is expected to lie between the Tertiary Older Volcanics and the Melbourne Formation. As indicated in the figures presented in Appendix A, it is expected to be encountered by the Melbourne Metro alignment in segments 5 and 8 and may have cross passages constructed within it.

The elevation of the base of the Werribee Formation is estimated from the long sections (Appendix A) to be between RL -8 m to RL -25 m AHD. We note that in the Melbourne Underground Rail Loop at Flagstaff Station, between 0.5 m and 12 m thickness of Werribee Formation was encountered (Bennet, 1992). The elevation of the base of the Formation lowers towards the west, being measured as RL +10 m AHD at William Street and -30 m AHD at Spencer Street, over a distance of about 900 m. Similar variability in the level of the Werribee Formation may be expected in the Melbourne Metro alignment, with the level of the base of the formation typically rising towards the east up to levels of about RL -2 m AHD through Segment 8. The distribution and thickness of the Werribee Formation could vary, with there being the possibility that it has sporadic occurrence, completely removed in some areas, with Older Volcanics resting directly over the Melbourne Formation (Anderson, 1992), although boreholes drilled for the project to date do not indicate this to be the case.

It is also possible that Werribee Formation could be present in the Domain and South Yarra areas. However based on the results of investigations undertaken to date, this appears to be unlikely.

A brief discussion of the Werribee Formation is presented below.

5.3.1 Material Characteristics

The Werribee Formation is comprised of dense sand and hard clay in varying proportion, usually with a higher proportion of clay near the top of the unit and sandy or gravelly material towards the base of the unit. Ligneous material or coal is also found within the Werribee Formation and has been found within the Domain and South Yarra areas in the vicinity of Commercial Road, although not within any of the boreholes drilled to date. It can often be difficult to distinguish the Werribee Formation from weathered Older Volcanics and weathered Melbourne Formation materials which occur adjacent. Organic material occurs in the Werribee Formation but not in the adjacent materials and can be used to differentiate between these units.

The boreholes drilled to date suggest that the Werribee Formation material expected to be encountered by the Melbourne Metro tunnels in the South Kensington area is predominantly a sandy material with some clayey sand and sandy gravel.

5.3.2 Mass Characteristics

Bedding planes are typically sub-horizontal and weakly defined within the Werribee Formation (Anderson, 1992) and tend not to define significant discontinuities. Within the harder clayey (typically upper) parts of the formation, fissures are common. The fissures are generally sub-vertical to steeply inclined and can have spacings of 100 mm to greater than 500 mm. Surfaces on the fissures are smooth and may be slickensided. Deeper in the formation, sand and gravel beds, potentially with a relatively high permeability, could be present, as encountered in Borehole GA11-BH11.

5.3.3 Weathering

The Werribee Formation materials have been subject to late Tertiary weathering. Chemical weathering of the Werribee Formation within the Melbourne area is minor, typically comprising iron stained bands near the top of the formation. Physical weathering carved river channels in the top of the Werribee Formation and removed large parts of it, giving it a sporadic occurrence and uneven upper surface. The Stage 2 and currently ongoing investigations suggests some variability of the Werribee Formation in the Kensington Area, with the upper surface generally deeper towards the west.

The Werribee Formation does not outcrop in the Melbourne Area. Plate 8 presents outcrops of Werribee Formation material in Bacchus Marsh.

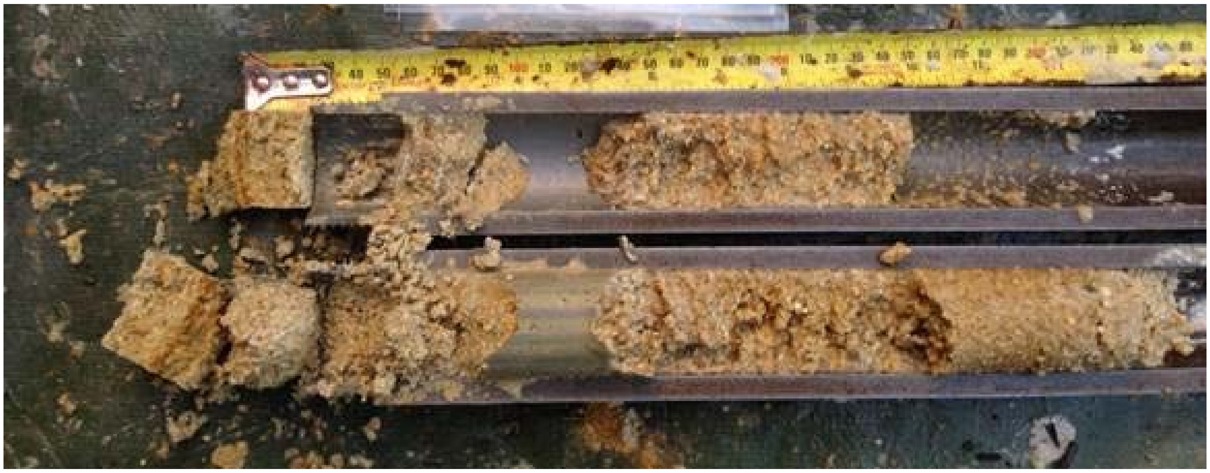


Plate 8: Werribee Formation recovered in SPT samples, Lloyd Street, Kensington (Golder records)

5.4 Tertiary Older Volcanics (Tvo)

During the mid-Tertiary (34 Ma) volcanic materials including basalt lavas and associated pyroclastic sediments were deposited in the Melbourne area, typically filling topographic lows within the Werribee and Melbourne Formations.

Older Volcanics are expected to be encountered in the western parts of the Melbourne Metro alignment, within Segments 3, 4 and 8. Older Volcanics could be encountered in the crown of the tunnel within Segment 8, where the proposed alignment runs very close to the contact between the Older Volcanics and underlying Werribee Formation.

5.4.1 Material Characteristics

In its fresh state, the Older Volcanics are typically comprised of high strength basalt. However, the basalt does not usually have a uniform composition within the Melbourne area, where it is often interbedded with pyroclastic deposits such as tuffs. The tuff material is particularly susceptible to weathering, typically weathering to a clay or clayey sand. The Older Volcanics basalts typically comprise olivine and pyroxene, with abundant volcanic glass, with the volcanic glass being highly susceptible to weathering.

The Older Volcanic basalt is typically dense, with infilled vesicles (gas bubbles). These features are termed amygdules.

5.4.2 Mass Characteristics

The mass and weathering characteristics of the Older Volcanics usually dominate its engineering behaviour. Within the Melbourne area, our experience of the Older Volcanics is that the flows typically have very close, randomly oriented jointing, with joint spacing of about 20 mm relatively common. In some locations, usually where the Older Volcanics are thicker, joints can be more widely spaced, up to 2 m, but this is rare, and usually joint spacing is less than 200 mm.

Where slightly weathered or fresh, the surfaces of joints within the Older Volcanics are rough, and the joints can be slightly open. However, this is rare, and generally chemical weathering has occurred along and outwards from joints producing a clay coating or clay infill.

Plate 9 shows an outcrop of highly weathered Older Volcanics basalt in the Kensington area. Plate 10 shows extremely weathered Older Volcanics, completely altered to white kaolin rich clay in the Parkville area.

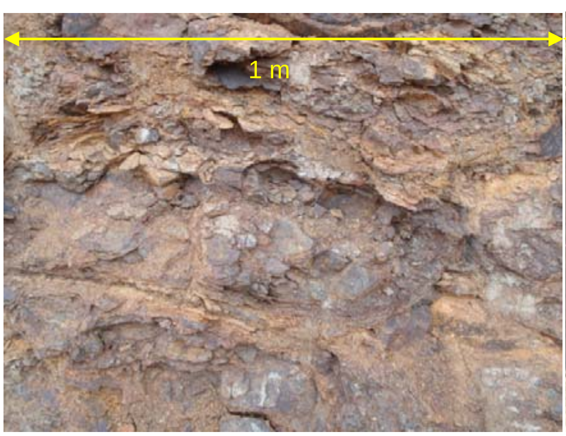


Plate 9: Outcrop of Older Volcanics, Kensington

Plate 10: Extremely Weathered Older Volcanics overlain by Brighton Group, Parkville

(Note Brighton Group material in this plate is not typical of the Brighton Group material expected to be encountered on Melbourne Metro).

5.4.3 Weathering

Late Tertiary weathering within the Older Volcanics caused extensive alteration of this material. In the basalt expected to be encountered along the proposed Melbourne Metro alignment, the Older Volcanics are expected to be weathered to some degree through their full thickness. A band within the Older Volcanics that appears to have been completely weathered to clay (residual) was consistently encountered in Stage 2 boreholes drilled within the Kensington area. This band was encountered between elevation RL 0 m and -10 m AHD through segments 3, 4 and 5 as indicated on the geological model presented in Appendix A. Residual Older Volcanics are expected to be encountered in Segments 3 and 4, including at the western portal. This prominent band could represent the weathering of a tuff or pyroclastic material.

Chemical weathering within the Older Volcanics basalt typically produces a rock mass comprised of rounded core stones of slightly or moderately weathered rock within a matrix of clayey (extremely weathered) materials.

The clays which form through the weathering of the Older Volcanics are typically high plasticity montmorillonite and kaolinite clays which are susceptible to shrink and swell in response to changes in moisture content. A number of weathering classification systems for the Older Volcanics have been proposed for different purposes. For excavation purposes, the proportion of rock, or corestones, to clay is an important consideration. Table 11 presents a classification system for the Older Volcanics, modified from that presented by Seddon and Pump in *Engineering Geology of Melbourne, 1992*.

Table 12: Weathering Grades for Older Volcanics (adapted from Seddon and Pump, 1992)

Weathering Grade	Typical Characteristics	Approximate Proportion of Rock to Soil
Residual Soil	Soil formed by weathering insitu, with the original texture of rock no longer evident.	0% Rock
Extremely Weathered	The rock material can be remoulded to a firm soil, but the original mass structure is mainly preserved. There may be occasional small corestones of less weathered material.	0% to 10% Rock
Highly Weathered	More than half of the rock material is moderately to extremely weathered. Discontinuities may be open, with weathering penetrating deeply inwards. The original fabric of the rock may be altered near discontinuities. Fresh or slightly weathered rock material may be present either as a discontinuous framework or as corestones.	10% to 50% Rock
Moderately Weathered	More than half of the rock material is fresh or slightly weathered and is present either as a continuous framework, or as corestones. Discontinuities may be open, with alteration penetrating inwards so that the balance of the rock material ranges from moderately to extremely weathered.	50% to 75% Rock
Slightly Weathered	Some to all of the rock material is discoloured by slight weathering, but the intact rock is not noticeable weaker than the fresh material. Discontinuities may be open, and will have a discoloured surface.	75% to 100% Rock
Fresh	Parent rock is fresh. Slight discoloration along joints (which may be open) may be present	100 % Rock

5.4.4 Relationship to Adjacent Materials

The emplacement of the Older Volcanics can preserve early Tertiary weathering profiles that are not otherwise preserved elsewhere. Typically, where the Older Volcanics directly overlie Melbourne Formation materials, a zone of white, kaolin rich clayey material is encountered. This material is inferred to represent an older weathering profile that has formed over the Melbourne Formation. Lavas and pyroclastics of the Older Volcanics have subsequently been emplaced over what could be a relatively loose surface (possibly underwater), and as a result, the contact between the two is not sharp (i.e. it is a transitional contact). Where Older Volcanics has been emplaced over the Werribee Formation, the Werribee Formation has been altered. It appears to have a zone cemented with iron oxide.

After emplacement, during the late Tertiary and early Quaternary, physical weathering carved valleys in and over large areas completely removing the Older Volcanics in some places. The sides of the valleys within the basalt material were relatively steep and boulders and other debris are likely to have accumulated on and at the toe of the slopes. This material is termed colluvium and is further discussed in subsequent sections. Where the proposed alignment passes from younger Quaternary sediments into the Older Volcanics, such as in Segment 2, colluvial material may be encountered.

5.5 Tertiary Brighton Group (Tpb)

During the late Tertiary, about 5.5 Ma, the sea flooded parts of the Melbourne area and deposited, shallow marine and fluvial (river) sediments over the older materials as described previously. These were typically sands and clays. The proposed Melbourne Metro alignment is expected to encounter Brighton Group materials in Segments 18, 21 and 23. The proposed alignment within those segments at some points passes close to the boundary between the Brighton Group and underlying Melbourne Formation. There is potential for mixed face tunnelling conditions with Brighton Group materials present in the tunnel crown.

5.5.1 Material Characteristics

The Brighton Group Sediments are comprised primarily of sands and clays in varying proportion. The sand is typically subrounded, fine to coarse quartz and the clay is of high plasticity. The Brighton Group has been overconsolidated, and as a result, the clays are typically hard and the sands are dense to very dense. Typically, the materials higher within the Brighton Group have a greater proportion of clayey material and consequently a lower permeability. With depth, the sand content and the associated permeability typically increases.

Sandy layers, usually towards the base of the unit may be cemented, forming low strength sandstone. No obvious trend in grain size with depth was observed in boreholes drilled through Brighton Group materials as part of the previously undertaken investigations. Although generally more sandy material than clayey material was encountered, in some boreholes, more clay rich material was encountered towards the base of the unit and in others more sand rich material was encountered.

5.5.2 Mass Characteristics

Bedding is evident within the sandier parts of the Brighton Group, but less pronounced within the clayey parts. However, stress relief induced by erosion of the Brighton Group has led to the formation of subvertical fissures. The fissures typically have smooth, slickensided surfaces.

5.5.3 Weathering

The Brighton Group materials were emplaced prior to the late Tertiary and were subject to the lateritic weathering that occurred at this time. Groundwater permeated the Brighton Group, causing the leaching of iron and precipitation of iron oxide minerals, primarily limonite. The iron oxides can cement the sandy grains together, forming weak sandstone.

Iron concretions (ironstone or ferricrete) were deposited in some of the more permeable zones within the Brighton Group. These ironstone concretions typically occur in relatively flat bands. They can be of very high strength and unjointed, which can present difficulties when excavating. We note that Borehole GA11-BH25 encountered a 200 mm thick very high strength cemented zone at a depth of 4.5 m and gravel sized ferricrete was encountered in Boreholes GA11-BH023 and GA11-BH020. Cemented zones and ferricretes are expected to be encountered in the Melbourne Metro tunnels where they are bored through the Brighton Group.

No weathering classification system has been established for the Brighton Group. It is typically described as a soil, not a weathered rock. In rare occurrences, very high strength silcrete is encountered within the Brighton Group, typically in the upper parts of the deposit.

Plate 11 presents an excavation exposing Brighton Group materials.

Plate 11: Brighton Group materials exposed in a basement excavation

5.5.4 Relationship to Other Units

The Brighton Group is expected to overlie Melbourne Formation materials along the Melbourne Metro Concept Design alignment. In a similar way to the Older Volcanics, the Brighton Group can preserve the products of weathering that are not encountered elsewhere. There is a significant age break between the Older Volcanics and the Brighton Group of almost 30 million years. The Brighton Group does not necessarily preserve the Early Tertiary, kaolin rich materials as the Older Volcanics do.

In Phase 2B boreholes which penetrated through the Brighton Group into the Melbourne Formation, a zone of residual soil, typically 1 m to 2 m thick was encountered at the interface between the two units. However, at some locations elsewhere in Melbourne, gravels, boulders and timber has been encountered at the interface between the two units. At one location in South Yarra (Commercial Road) voids were encountered between the Brighton Group and underlying formations.

Plate 12 shows a contact between Brighton Group materials and Silurian siltstone exposed in a rail cutting in Parkville.

Plate 12: Brighton Group overlying Silurian Siltstone, Parkville

5.6 Punt Road Sands (Qpp)

A period of weathering and erosion occurred towards the end of the Tertiary which carved valleys into and in places removed all of the materials described above. At the base of the Jolimont Valley, colluvial and alluvial material typically comprising a mixture of cobbles, gravels, sand and silt accumulated in the early Quaternary. The Punt Road Sands are now only preserved where covered with Lower Newer Volcanics. The proposed Melbourne Metro Concept Design alignment is not expected to encounter this material. However, it is included here as it does occur near the alignment within Segment 16 and could be encountered in the event the alignment is modified or if ground improvement works are required to extend below the vertical alignment of the tunnels in this area.

5.6.1 Material Characteristics

The Punt Road Sands are comprised of material with a highly variable composition. It contains cobble to boulder sized clasts of siltstone or sandstone derived from the Melbourne Formation, typically towards the base of the unit. The pore spaces between the clasts are typically filled with finer materials, clay, silt and sand. The finer materials tend to be present in a greater proportion higher up within this unit.

5.6.2 Mass Characteristics

The coarse size of some of the clasts within this material provides an open, permeable structure. However, this material is a soil, and typically does not contain discontinuities.

5.6.3 Weathering

This material has undergone slight chemical weathering. The weathering has occurred in a reduced environment which has resulted in a softening of the siltstone cobbles and boulders.

5.6.4 Relationship to Other Units

This material is typically only preserved where it is capped by Lower Newer Volcanics basalt. Elsewhere it has been removed by subsequent episodes of erosion.

5.7 Newer Volcanics, Swan Street Basalt (Qvns)

Basalt lava flowed down the ancestral Jolimont Valley, preserving the colluvial and alluvial material in the base of the valley and infilling it. The proposed alignment is not expected to encounter this material, currently passing over it in Segment 16.

5.7.1 Material Characteristics

The basalt typically comprises olivine, feldspars and pyroxene of high strength with a dark blue-grey colour. Testing on Swan Street Basalt encountered in the Stage 2 investigation indicate that the basalt is of high to extremely high strength.

5.7.2 Mass Characteristics

The Swan Street basalt, within the Jolimont Valley, is relatively massive compared with the newer volcanics emplaced on the open plains West of Melbourne (Werribee Plains Volcanics). Subvertical, typically clean joints are present, with relatively wide spacing, sometimes several metres, but locally 100 mm or less. Vesicles are present, but in a lower concentration than is usually encountered within the Newer Volcanics. Where encountered in the Stage 2 investigation, joint spacing was typically over 1 m.

5.7.3 Weathering

The Swan Street Basalt within the Jolimont Valley is typically slightly weathered to fresh. No residual clay is typically present above this material, likely as a result of subsequent erosion and because it was buried by other units before significant chemical weathering could occur.

5.7.4 Relationship with Other Units

The surface of the Lower Newer Volcanics within the Jolimont Valley forms a sharp, smooth contact with the overlying Moray Street Gravels. These materials preserve the underlying Tertiary Colluvium, which has typically been removed in other areas.

5.8 Early Pleistocene Alluvial and Colluvial Sediments (Qpc)

With further erosion, valleys formed along the edge of the early Quaternary and Tertiary rock. The Moonee Ponds Creek valley was also carved deeper. Fluvial sediments including colluvial and alluvial material, similar in origin and composition to the Punt Road Sands as described previously were deposited on the sides and base of these valleys. Quaternary Fluvial Sediments may be encountered in Segments 6 and 7, near the base and sides of the Moonee Ponds Creek Valley. It may also be present beneath the proposed alignment in Segment 16, near the base of the Jolimont Valley.

5.8.1 Material Characteristics

The Early Pleistocene Alluvial and Colluvial Sediments have a similar composition to the Punt Road Sands as described previously. The key difference is that the Early Pleistocene material was deposited after the emplacement of the Lower Newer Volcanics basalt, and as a result typically contains boulders, cobbles and gravels of basalt material, whereas in the Punt Road Sands, the coarser clasts are typically siltstone. These basalt boulders can be of high strength. The remainder of the sediments are comprised of sands, gravels and some clays.

5.8.2 Mass Characteristics

This material is effectively a granular soil and does not contain joints or other discontinuities. Due to the presence of coarse clasts (boulder and cobble size) the sediments can have an open, porous structure.

5.8.3 Weathering

Unlike the Punt Road sands, the more recent Quaternary material does not exhibit signs of weathering.

5.8.4 Relationship with Other Units

The Early Pleistocene Colluvial and Alluvial Sediments may be encountered on the base and sides of the Jolimont and Moonee Ponds Creek Valleys. Contacts between the geological units within these valleys represent depositional time breaks during which the fluvial sediments could have been deposited on the walls and base of the valleys which existed at that time. Fluvial sediments present on geological contacts could lead to local variation in hydraulic conductivity and engineering characteristics.

The Moray Street Gravels which overlie this material represent a change from a fluvial erosional environment to a depositional one. This transitional change can make identification of the boundary between the Quaternary Fluvial Sediments and Moray Street Gravels difficult.

5.9 Moray Street Gravels (Qpg)

During the period of low sea level in the mid Quaternary (about 1 Ma), high energy rivers flowed down the Jolimont and Moonee Ponds Creek Valleys. These deposited fluvial, river sediments into the Yarra Delta. The Moray Street Gravels are not expected to be encountered in the Melbourne Metro tunnels. However, the Moray Street Gravels within the Jolimont Valley in Segment 16 could be impacted by structures or if ground improvement is required to extend below the vertical alignment of the tunnels in this area. Piling for approach structures (if required) could encounter this material in Segment 1.

5.9.1 Material Characteristics

Despite the name, the Moray Street Gravels are predominantly comprised of medium to coarse grained quartz sand materials with some gravels and some finer silt and clay materials. Organic material in the form of timber is also present in trace quantities. The sand is typically dense. Where beds of clay and silt material occur, they are typically very stiff to hard.

Plate 13 presents a sample of the Moray Street Gravels.

Plate 13: Sample of Moray Street Gravels in SPT sampler

5.9.2 Mass Characteristics

The Moray Street Gravels have a variable composition, brought about by variable environments prevalent during its emplacement. Typically coarser, gravelly materials are encountered towards the base, with a general fining of materials upwards. The distribution of clayey beds and lenses, some of which may be up to 4 m thick is difficult to predict.

The typically coarse grain size and relatively wide distribution of the Moray Street Gravel makes it probably the most significant confined aquifer within the Melbourne area. The aquifer extends several kilometres upstream and downstream of the proposed Melbourne Metro crossing of the Jolimont Valley and fills a large extent of the Yarra Delta, underlying the South Melbourne, Port Melbourne and Docklands areas.

5.9.3 Weathering

With the exception of some minor iron staining, the Moray Street Gravels do not exhibit significant signs of chemical weathering. They have been subject to some physical weathering which has cut broad, shallow channels in the surface. However, variations in surface weathering are relatively minor and the upper surface of the Moray Street Gravels is at a relatively uniform level with an elevation typically around RL -20 m AHD.

5.9.4 Relationship with Other Units

The contact between the Moray Street Gravels and the overlying Fishermens Bend Silt is relatively abrupt. The sandy, basal material of the Fishermens Bend Silt directly overlies the Moray Street Gravels.

5.10 Fishermens Bend Silt (Qpfl and Qpfu)

Fishermens Bend Silt is present within the Jolimont, Maribyrnong and Moonee Ponds Creek Valleys. The proposed alignment is expected to encounter this material in the Moonee Ponds Creek Valley in Segment 6. The Arden Street Metro station (Segment 7) is expected to be excavated partly within this material. Within the Jolimont Valley (Segment 16) the tunnels are expected to be bored through Fishermens Bend Silt beneath the Yarra River and Princes Street Bridge.

5.10.1 Material Characteristics

The Fishermens Bend Silt is comprised of clay, silt and sand sized particles. Typically, there is a higher proportion of sand towards the base of the unit, with clayey material encountered towards the top. The dominant material type within the Fishermens Bend Silt is firm to very stiff silty clay with variable plasticity. Material encountered in the Stage 2 boreholes varied between clay and sand, with sandy material typically encountered below clayey material. A distinction has been made between material with a dominant proportion of sand or clay. These have been designated the Fishermens Bend Silt Upper (Qpfu) and Fishermens Bend Silt Lower (Qpfl).

5.10.2 Mass Characteristics

The Fishermens Bend Silt contains sandier material towards the base and the edges of the unit. However whilst the material in these areas contains a higher proportion of sand, it typically still has a high fines content and occurs as a silty sand and clayey sand. The Fishermens Bend Silt has been subject to erosion, draining and overconsolidation. Remnants of these processes in the form of fissures are present, particularly towards the top of the unit.

5.10.3 Weathering

The Fishermens Bend Silt has been exposed to the atmosphere and has undergone subaerial chemical weathering. The effects of this include an orange iron oxide staining and mottled appearance to the material. However, the weathering has not significantly altered the engineering properties of the material. In the Phase 2C boreholes at the Yarra River crossing, a distinct boundary between orange weathered material and grey less weathered material was observed. This boundary coincides with the boundary between the Fishermens Bend Upper and Lower units as described above.

Physical weathering has also occurred within this material which has formed channels within its surface and lead to partial erosion. Consequently, there can be some variability within the level of the upper surface of this material.

5.10.4 Relationship to Other Units

The edges of the Fishermens Bend Silt are relatively abrupt, with the surfaces forming the upper and lower boundaries being erosional surfaces. Plate 14 presents the base of an excavation which extended into the Fishermens Bend Silt.

Plate 14: Base of Excavation within Fishermens Bend Silt, Southbank

5.11 Pleistocene Alluvium (Qpa)

As noted above, the Fishermens Bend Silt was subject to a period of erosion, with streams forming on top of the material and carving channels into its surface. Within the Maribyrnong and Moonee Ponds Creek Valleys, there are firm to stiff clay and silt materials present at the top of the Fishermens Bend Silt. The ages of these materials are uncertain, however, they appear to post-date the Fishermens Bend Silt and pre date the Coode Island Silt (discussed subsequently). This material is expected to be encountered in the Arden Station excavation (Segment 7). Although the proposed tunnels are not expected to encounter these materials, they may be present beneath embankments or structures on the approach to the western portal.

5.11.1 Material Characteristics

These materials typically comprise clay, silt and sand. The proportion of each of these materials is variable, with firm to stiff silty or sandy clay the dominant material.

5.11.2 Mass Characteristics

This material is inferred to be relatively massive. However, although no fissures have been observed, its geological history suggests that subvertical fissures may be present.

5.11.3 Weathering

This material does not exhibit significant signs of chemical weathering. However, it has an inferred undulating upper surface, suggesting that it may have been subject to physical weathering and erosion.

5.11.4 Relationship to Other Units

This material underlies the Coode Island Silt and has a similar composition and colour. However, it is distinguished on the basis of its higher strength and stiffness. This property implies that this material has been drained, leading to its consolidation. Identification of this material is typically undertaken on the basis of a measured contrast in strength compared with adjacent units.

5.12 Newer Volcanics (Q_{vn}) (Burnley Basalt Flow)

The proposed Melbourne Metro alignment is expected to encounter the Newer Volcanics Basalt in Segment 16 at the Yarra River Crossing. This material is derived from lava that flowed down and filled the ancestral Jolimont Valley. The flow was mostly contained within a channel, but overtopped the channel at some locations, emplacing lobes of lava adjacent to the channel.

The Melbourne Metro alignment is expected to require excavation of this material at the Yarra River Crossing. The anticipated spatial distribution of this material within the vicinity of the Jolimont Valley crossing is indicated on the geological plans presented in Appendix B. This distribution is based on the marine geophysical survey undertaken as part of the Stage 2 investigation boreholes and probe holes. However, it is noted that although this material is shown on the geological plan, this represents a subsurface distribution, as the basalt does not outcrop in the vicinity of the proposed Jolimont Valley crossing.

5.12.1 Material Characteristics

The basalt is typically of high to very high strength with a dark blue-grey colour. It is mostly massive, but contains some areas with concentrated vesicles (small 'bubbles' that have cooled within the lava). Historically, it has been quarried in the Burnley area for use as a building stone.

5.12.2 Mass Characteristics

Joints are prevalent within the Newer Volcanics Basalt; however, their distribution is somewhat variable. Sub-vertical, planar joints with rough, clean surfaces are typical. However, clay infill towards the top of some of the joints is common.

The sporadic nature of the jointing leads to some massive parts of the rock mass with joint spacing of the order of several metres. There are other parts of the rock mass with more closely spaced joints of the order of 100 mm or less. The distribution of jointing is difficult to predict.

Where present, the high plasticity residual clay that forms over the basalt typically contains oblique, slickensided fissures.

An exposure of the basalt within the Burnley Quarry is shown in Plate 15.

Plate 15: Newer Volcanics Basalt exposed in a quarry, Burnley

5.12.3 Weathering

The Newer Volcanics Basalt has been subject to some sub-aerial chemical weathering which has led to the alteration of rock minerals to clay. Typically the pyroxene and olivine in the basalt alters to high plasticity montmorillonite clay. However, as evidenced by the Federation Square piling records and the Stage 2 boreholes within the vicinity of the proposed Yarra Crossing, the depth of weathering typically does not exceed about 2 m below the top of the unit. Within the river bed itself, there is little to no residual soil. Features within the weathered zone include joints with a high plasticity clay infill, and basalt cobbles and boulders within a clay matrix. Typically, extremely to moderately weathered materials occur in only a thin band over the relatively fresh rock and sometimes may not be evident at all.

A description of weathering grades within the Newer Volcanics Basalt is presented in Table 12. This is based on our experience with this material. We note that these weathering grades differ from those described in Table 11 for Older Volcanics. Whilst derived from a similar source rock, the different climate which induced the weathering and the timeframe over which the weathering occurred has led to different weathering products and characteristics.

Table 13: Weathering Grades within the Newer Volcanics Basalt

Weathering Grade	Typical Characteristics	Approximate Proportion of Rock to Soil
Residual Soil	Soil formed by weathering in situ, with original texture of rock no longer evident. Typically high plasticity brown or grey montmorillonite clay.	0% Rock
Floater Zone	High strength corestones (floaters) are present within a high plasticity clay matrix. The floaters may be up to several metres in diameter. The original rock structure is not apparent.	0% to 80% Rock
Highly Weathered	The original rock structure is preserved. Joints and discontinuities are prevalent and are infilled with high plasticity clay. The intact rock may be discoloured, typically to a brown colour.	80% to 95% Rock
Moderately Weathered	Most of the rock mass is comprised of intact, slightly weathered or fresh rock. Occasional clay filled joints and seams. Joints may be open. Some discolouration to a brown colour and staining on joints.	95% to 100% Rock
Slightly Weathered	Some discoloration along joint surfaces, some open joints, but otherwise similar to fresh rock.	100% Rock
Fresh	Parent rock is fresh. No discoloration or alteration of rock minerals to clay evident.	100 % Rock

5.12.4 Relationship with Other Units

The contacts between the Newer Volcanics Basalt and other units are likely to be variable. The lava flow moved down the ancestral Jolimont Valley and covered material within the base of the valley. This included organic material such as leaf matter and timber, as well as gravels, and other loose material in the base of the river valley. We note that at a site about 500 m downstream from the proposed Melbourne Metro Yarra crossing, layers of organic material (mainly leaves) were encountered immediately below the Newer Volcanics Basalt. There is typically a transitional zone between the basalt and the underlying material, the composition of which can again, be highly variable. This transitional zone can contain voids and have a relatively high permeability.

By infilling the ancestral Yarra River, the basalt flow diverted the water flow in the river. New channels were carved down either side of the river. Steep slopes were formed in this material and boulders of basalt were dislodged from the slopes and deposited at the toe of the new river channels. As a consequence, the contacts between the basalt and younger overlying materials are typically steep and there may be loose materials such as cobbles and boulders of basalt on the surface. The contact zone between the basalt and overlying materials can have variable composition, strength and permeability.

5.12.5 Historical excavation of the Newer Volcanics Basalt

As part of the construction of Princes Bridge, the Newer Volcanics Basalt was excavated both upstream and downstream of the bridge. We have obtained archival information from the Victorian State Library related to the construction of the bridge, and river deepening which involved excavation into the Newer Volcanics Basalt at the location of the proposed river crossing. Relevant information obtained from these documents is presented below:

- The Yarra River was prone to flooding in the vicinity of Princes Bridge, and the land to the south of the River in what is now Alexandra Gardens was frequently inundated. Deepening, straightening and widening of the river was undertaken in the 1880's to alleviate this flooding when the original bridge was replaced with the existing bridge. Excavations were advanced approximately 0.5 km upstream and downstream of the bridge as part of the flood relief works (Allison, 2007).
- The basalt was excavated to a nominal level of between RL -4.6 m and RL -5.5 m AHD between 1881 and 1886 (converted from Hodgekinson' s 1853 low water datum). (Department of Public Works Archives, 1881).
- Following excavation of the rock, coffer dams were constructed to facilitate construction of the piers of the current bridge (Allison, 2007).
- Dynamite was used in the excavation of the rock (Allison, 2007). Reports regarding how much rock was removed vary; however, the volume of rock removed appears to have been significant.

Information obtained from the Stage 2 investigations, including geophysics survey used in conjunction with the boreholes and probe holes indicates a channel through the basalt, close to the centre of the Yarra River. This channel is inferred to have been excavated in the early 1880's. It is about 20 m wide and relatively linear.

Some uncertainty remains around the level to which the basalt was excavated. There may also be some uncertainty associated with conversion from historical datums. Based on historical information, we currently estimate this level to be between RL -4.6 m and RL -5.5 m AHD. The Stage 2 boreholes and probe holes indicate the top of basalt level to be between RL -3.2 m AHD towards the river bank and RL -6.3 AHD closer to the channel near the mid-point of the river. Further information on the depth and thickness of the flow in the vicinity of the proposed TBM tunnel alignments will become available once the Stage 3 Yarra River Crossing investigation is completed.

The interpreted distribution of the Newer Volcanics Basalt at the Yarra River Crossing is indicated in Appendix B for the purposes of the EES.

5.13 Jolimont Clay (Qpj)

Flooding of the Jolimont Valley again occurred following the emplacement of the basalt, depositing marine clay similar in appearance and engineering characteristics to the Fishermens Bend Silt. This material may be a continuation in the deposition of the Fishermens Bend Silt. Prior to deposition of the Jolimont Clay a new channel was eroded on the northern side of the Newer Volcanics beneath what is now Federation Square. This channel was subsequently backfilled with the clay.

The Melbourne Metro tunnels are expected to pass beneath, but not encounter the Jolimont Clay in Segment 16.

5.13.1 Material Characteristics

As the name suggests, the Jolimont Clay is predominantly comprised of clay sized material, with minor silt, sand and gravel. The Jolimont clay is slightly overconsolidated and typically occurs as a firm to very stiff clay.

5.13.2 Mass Characteristics

Like the Fishermens Bend Silt, the Jolimont Clay has undergone erosion and associated stress relief. Vertical fissures within the clay are a consequence of this stress relief. Layers of gravelly material have been reported within the Jolimont Clay.

5.13.3 Weathering

The Jolimont clay has been subject to sub-aerial weathering similar to the Fishermens Bend Silt. The weathering has led to an iron oxide staining. Some physical weathering and erosion has also occurred within this material.

5.13.4 Relationship with Other Units

The Jolimont Clay has infilled a shallow valley to the north of the Newer Volcanics basalt in the vicinity of the Melbourne Metro Yarra River crossing. Prior to the deposition of the clay, it is possible that loose material (colluvium) was deposited on the walls and floor of the valley. As a consequence, there may be gravelly or more porous material present at the base and sides of the valley.

5.14 Holocene Alluvium (Qha)

Erosion during the last glacial period (about 18,000 years ago) lead to a deep river valley being carved into the Melbourne Formation on the southern side of the Jolimont Valley. The northern side of the valley appears to have been partly defined by the Newer Volcanics Basalt.

Although the Holocene Alluvium is not expected to be encountered along the Melbourne Metro alignment, it forms a significant aquifer that could be affected by the proposed tunnels. Previous investigations, most notably those undertaken for the City Link Tunnels, encountered the Holocene Alluvium within this river valley both upstream and downstream of the proposed Melbourne Metro tunnels and they were also encountered as part of the Stage 2 investigation.

Holocene alluvium and colluvium is present within the CBD. Excavation for the Telstra Cable tunnel along Lonsdale Street encountered material described as colluvium near the intersection of Russell Street. There is no indication based on the ground information associated with the Telstra Tunnel that the alluvium would be encountered underlying Swanston Street or encountered within the Melbourne Metro tunnels. However, this would need to be confirmed with future intrusive investigation within Swanston Street.

There is also expected to be minor channels within Segment 1 infilled with Holocene Alluvium, although the proposed alignment would not encounter them.

5.14.1 Material Characteristics

The Holocene Alluvium typically comprises fine to medium grained sands with some gravels and cobbles. Their composition can be variable in that a varying proportion of silt or clay can be present within the sand. Borehole GA11-BH18 encountered about 1 m of very dense gravelly sand overlying about 1 m of cobbles and boulders in a sandy gravel matrix.

5.14.2 Mass Characteristics

The Holocene Alluvium is inferred to be present for a significant length along the base of the Jolimont Valley, forming an aquifer at the base of the overlying Coode Island Silt.

5.14.3 Weathering

The Holocene Alluvium is not expected to have been affected by weathering.

5.14.4 Relationship with Other Units

The alluvium has been emplaced on a valley floor. There may be gravels or other deleterious material present at the contact between the sands and the underlying Melbourne Formation. The contact between the Holocene Alluvium and overlying Coode Island Silt is likely to be gradational. That is, there is unlikely to be a sharp contact, but rather an increase in the proportion of sand towards the base of the Coode Island Silt, until it grades into the sandy Holocene Alluvium.

5.15 Coode Island Silt (Qhi)

The Coode Island Silt is a widespread unit, infilling the Maribyrnong, Moonee Ponds and Jolimont (and Yarra) Valleys. The proposed Melbourne Metro tunnels are expected to encounter this material within the Jolimont Valley in Segment 16. The approach to the Western Portal (Segment 1) is to be constructed over and within this material. Within the Moonee Ponds Creek Valley, the crown of the proposed Melbourne Metro tunnel alignment may encounter Coode Island Silt in Segment 6 on the approach to Arden Station. This unit is also expected to be encountered within the Arden Station Box excavation.

5.15.1 Material Characteristics

Contrary to the name, the Coode Island Silt is predominantly a clayey material. It is typically soft, becoming firm towards the base with a strength profile typical of a normally to slightly over consolidated material. Parts of the Coode Island Silt contain shells and other organic materials such as timber. Plate 16 shows Coode Island Silt exposed in an excavation in Southbank.

Plate 16: Coode Island Silt Exposed in Excavation, Southbank (Golder archives)

The composition, stress history, degree of consolidation and strength of the Coode Island Silt is known to vary within the embayment in which it was deposited. The edges of the embayment were formerly estuaries or beaches and likely subject to tidal influences. Coarser materials, silts and sands can be expected near the edge of the embayment. These materials may have a higher degree of consolidation due to tidal influences. Coode Island Silt at the edges of the embayment, for example Segments 1, 2, 6, 7 and 16 may be subject to these effects. Coode Island Silt towards the centre of the embayment was likely deposited in deeper water and likely to have a lower degree of consolidation and strength, for example near the Moonee Ponds Creek in Segment 6.

The Coode Island Silt may have been consolidated in some areas due to anthropogenic influences. Fill of varying thickness has been placed over most of the Coode Island Silt inducing consolidation that might vary in different areas. However, it is noted that depending on the age of the fill, the Coode Island Silt underlying the fill may be normally consolidated. Depressurisation due to tunnels, sewers and basement excavations within the vicinity of the Coode Island Silt could also induce consolidation and locally strengthen the Coode Island Silt.

Regional settlement within the Coode Island Silt is reported to be up to 10 mm per year (Ervin 1992), depending on the thickness of the deposit and the location within the embayment. This ongoing settlement is thought to be influenced by recent filling, extraction and drainage of groundwater and ongoing secondary compression.

5.15.2 Mass Characteristics

Sand is typically present towards the base and edges of the deposit, and occurs as lenses or beds within it. The Coode Island Silt does not usually contain discontinuities such as fissures. However, if drained or allowed to dry out, fissures do form in this material. The stage 2 Borehole GA11-BH18 indicates the Coode Island Silt at the Yarra River crossing comprises predominantly silty clay with clayey sand and sand present towards the base of the unit. Organic material including wood and timber is also present within the unit, in higher concentration towards its base.

5.15.3 Weathering

The Coode Island silt does not display significant evidence of weathering. When exposed to oxygen, sulphides within this material can oxidise. It is a potential acid sulphate soil.

5.15.4 Relationship with Other Units

This material has infilled a large area of the Yarra Delta including the Maribyrnong, Moonee Ponds Creek and Jolimont Valleys. It was deposited in a typically low energy environment. The contacts between the Coode Island Silt and underlying materials may have materials such as gravels, boulders and organic material on them. This was observed in Borehole GA11-BH18 drilled at the Yarra River Crossing.

5.16 Fill (Fill)

The Coode Island Silt is typically capped with fill materials, mostly placed during the 1800’s. The surface of the Coode Island Silt was effectively a swamp and widespread placement of fill was undertaken to make the surface trafficable and allow development. Fill of varying thickness and composition is therefore expected to be encountered along most of the proposed Melbourne Metro alignment.

Existing railway embankments have been constructed using variable fill materials. The fill within Segments 1 and 2 shown on the long sections in Appendix A is associated with the existing railway embankment in this area. The area immediately to the south of the Yarra River (Alexandra Gardens) which was formerly a lagoon has been infilled to a depth of up to 5 m and is one of the thicker areas of fill along the proposed Melbourne Metro alignment.

5.16.1 Material Characteristics

The fill is expected to have a highly variable composition over a scale of metres. Local experience suggests the fill within the Melbourne area can contain soft dredge spoil, building refuse (concrete, brick, steel) and other waste. The degree of compaction within the fill is also variable. Plate 17 presents an excavation within variable fill material in Southbank.

Plate 17: Examples of Fill material encountered in excavations in Southbank (Golder archives)

5.16.2 Relationship with Other Units

The fill is typically thickest where it has been placed on the softer Coode Island Silt material. Where the fill has been placed over the harder or denser materials of the Melbourne Formation, Brighton Group or Older Volcanics, it tends to be thinner. However, this is a very general rule, and variability of fill thickness should be assumed.

5.17 Recent Silt (Qra)

The Yarra River has historically been dredged. Over time the dredged channel has filled with sediment. Historical information regarding the dredging suggests that at the location of the Yarra River crossing, the river has been dredged to a level of about RL -5.5 m AHD. Boreholes indicate that the rock has been removed to levels as low as RL -6.3 m AHD. The dredged channel has subsequently been partly infilled with recent sediment to the current river bed elevation. We note that sediment within the river is deposited and removed by erosion and by dredging over relatively short timeframes, and as such, its thickness is likely to vary. Deposition of silt in the river is an ongoing process.

5.17.1 Material Characteristics

The sediment is typically fine grained clay or silt, which may contain some deleterious material. In the Stage 2 investigation drill rods sank into this material under their own weight, suggesting that it is very soft. Some boreholes (e.g. GA11-BH35) indicate there are basalt gravels and cobbles at the base of the recent silt. This may be debris associated with the dredging. The Stage 2 sidescan sonar indicates there is an area east of the proposed alignment with cobbles and boulders embedded at the surface of the recent silt. In addition, some deleterious material was encountered in the Stage 2 boreholes including timber and rope.

5.17.2 Relationship with Other Units

The base of this material follows the contours of the dredged channel within the Yarra River. There may be some deleterious material such as gravel and other refuse on the base of the river along this contact. The upper surface of this material is likely to change in response to erosion, deposition and dredging of the base of the river.

6.0 GROUND ENGINERING PARAMETERS

Laboratory and in situ testing undertaken specifically for the project has been analysed and used to develop set of preliminary engineering parameters suitable for use to estimate ground response associated with the project and to support the development of Melbourne Metro Concept Design and EES.

This section broadly discusses the ground engineering parameters measured through laboratory and in situ testing undertaken at project wide scale and presents a characterisation for some of the geological units described in this report.

It should be noted that the measured engineering parameters in most units expected to be encountered are variable.

Where possible, the information presented in this section is based on laboratory test information obtained specifically for Melbourne Metro. Where no project specific data is available, published data or data obtained during the desktop audit is presented. Where project specific data is available it has been presented in lieu of data obtained from other sources. The information presented here is intended to provide an indication as to the range of parameters that can be expected within each formation and is intended to inform sensitivity analysis. A discussion on the limitations of the data is also provided.

Each of the geological units, as described in the previous section, is discussed separately in terms of its measured engineering properties. Some units, for example the Melbourne Formation are further characterised sub units based on their geotechnical characteristics, strength and stiffness.

6.1 Melbourne Formation (Sud)

The following section summarises the measured engineering properties of the Melbourne Formation.

6.1.1 General

The engineering properties of the Melbourne Formation vary with the degree of weathering, whether the material has been weathered in an oxidising or reducing environment and the proportion of sandstone. However, generally, the more weathered materials are weaker and have a higher compressibility. Since mineralogical content does not vary significantly, the void ratio or saturated water content (in the siltstone component) has been found to provide a useful quantitative indicator of the engineering properties of the rock. The saturated water content, w, varies from around 10% for HW to EW siltstone to less than 1% (void ratio $\approx$ 0.027) for fresh (Fr) siltstone.

Moisture content is used here to facilitate comparison of measured properties. Table 14 provides a correlation between weathering grade and saturated moisture content for the Melbourne Formation. This correlation has been used to allow design parameters to be presented for the various weathering grades of the Melbourne Formation.

It has been assumed that the measured moisture contents obtained from Stages 1, 2 and the most recent investigation are for materials in a saturated or close to saturated state. We understand that some of the testing undertaken during the Stage 1 investigation may not have been undertaken on saturated samples.

The moisture contents have been measured on both reduced and oxidised rocks and at this stage no attempt has been made to distinguish the results based on the type of chemical weathering the samples have experienced. However, we consider it reasonable to assume that based on the unit's known distribution in Melbourne, most of the data has been obtained from oxidised samples. In general, siltstone with higher water contents (w > 12%) can be considered to be oxidised.

The Melbourne Formation contains both siltstone and sandstone, although investigation undertaken to date indicates that most of the Melbourne Formation (greater than 90%) comprises siltstone. It is not possible at this stage to predict the locations at which siltstone and sandstone could be encountered. For RD purposes, we have not attempted to distinguish between the two material types, but rather provide an indication of the overall mass properties of the Melbourne Formation.

Table 14: Quantitative Weathering Index for the Melbourne Formation

Weathering Grade*	Saturated Moisture Content (w) %
Fresh	0 - 2
Slightly Weathered	2 - 5
Moderately Weathered	5 - 9
Highly Weathered	9 - 13
Extremely Weathered (residual soil)	> 13

*Weathering scheme based on a widely used correlation with saturated moisture content

Plate 18 presents the results of moisture content testing undertaken on samples of Melbourne Formation. Whilst the aggregated results indicate that the moisture content and therefore degree of weathering generally decreases with depth, there is appreciable scatter evident in the data. Correlation of engineering properties with depth in the Melbourne Formation is unlikely to be appropriate. Correlation with moisture content and degree of weathering is more suitable in the Melbourne Formation.

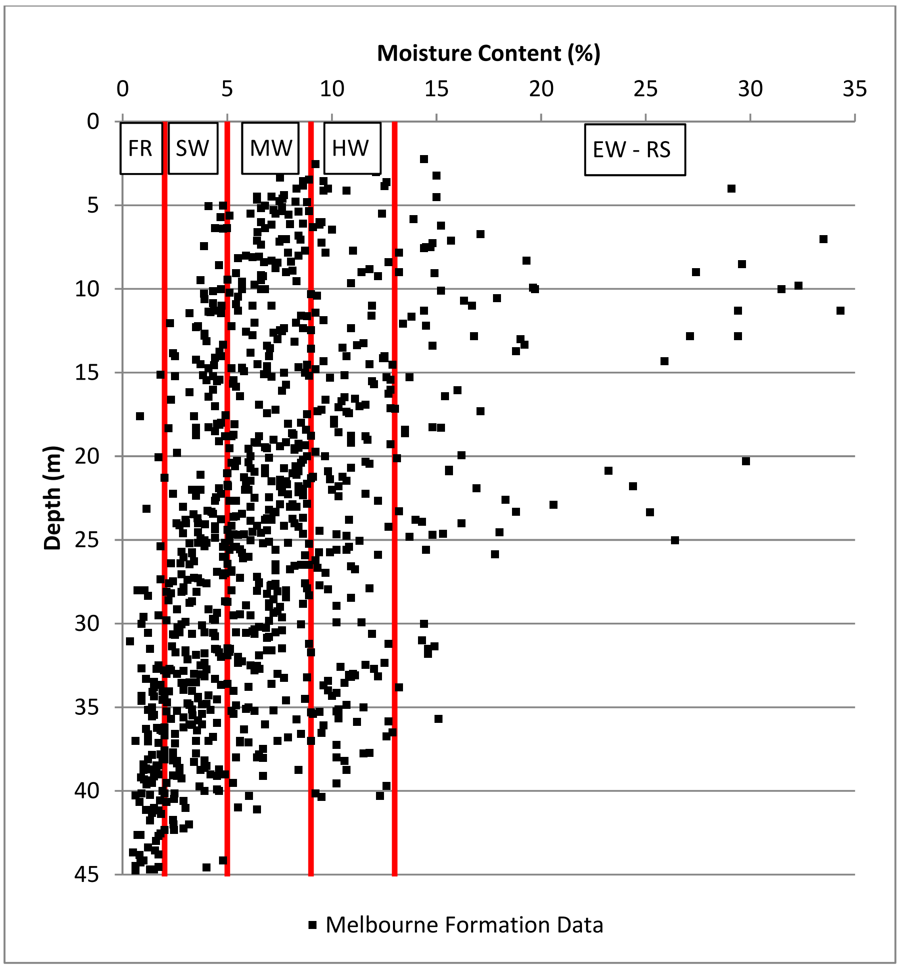


Plate 18: Summary of moisture content testing undertaken in Melbourne Formation

6.1.2 Classification Properties

Particle Size

Williams (1977) presents a summary of classification testing undertaken on eight samples of highly weathered Melbourne Formation materials in South Melbourne. Four of these samples are described as sandstone and four as siltstone. These tests indicate the following proportions of grain size:

Clay, 3% to 20%, Average 11.5%

Silt, 14% to 78%, Average 58%

Fine Sand, 7% to 37%, Average 18%

Medium (or greater) Sand, 0% to 46%, Average 12%

These tests suggest that whilst the sandstone and siltstone materials have different grain sizes, when considered en masse, the dominant particle size in the Melbourne Formation is silt. The formation is therefore commonly referred to as a siltstone on this basis.

Particle sizes estimated from petrographic testing undertaken on Melbourne Formation during the RD stage are presented in Plate 19. Note that these proportions are estimated visually (and very approximately) using thin sections. The data from Williams 1977, which is based on broken down material, is shown for comparison.

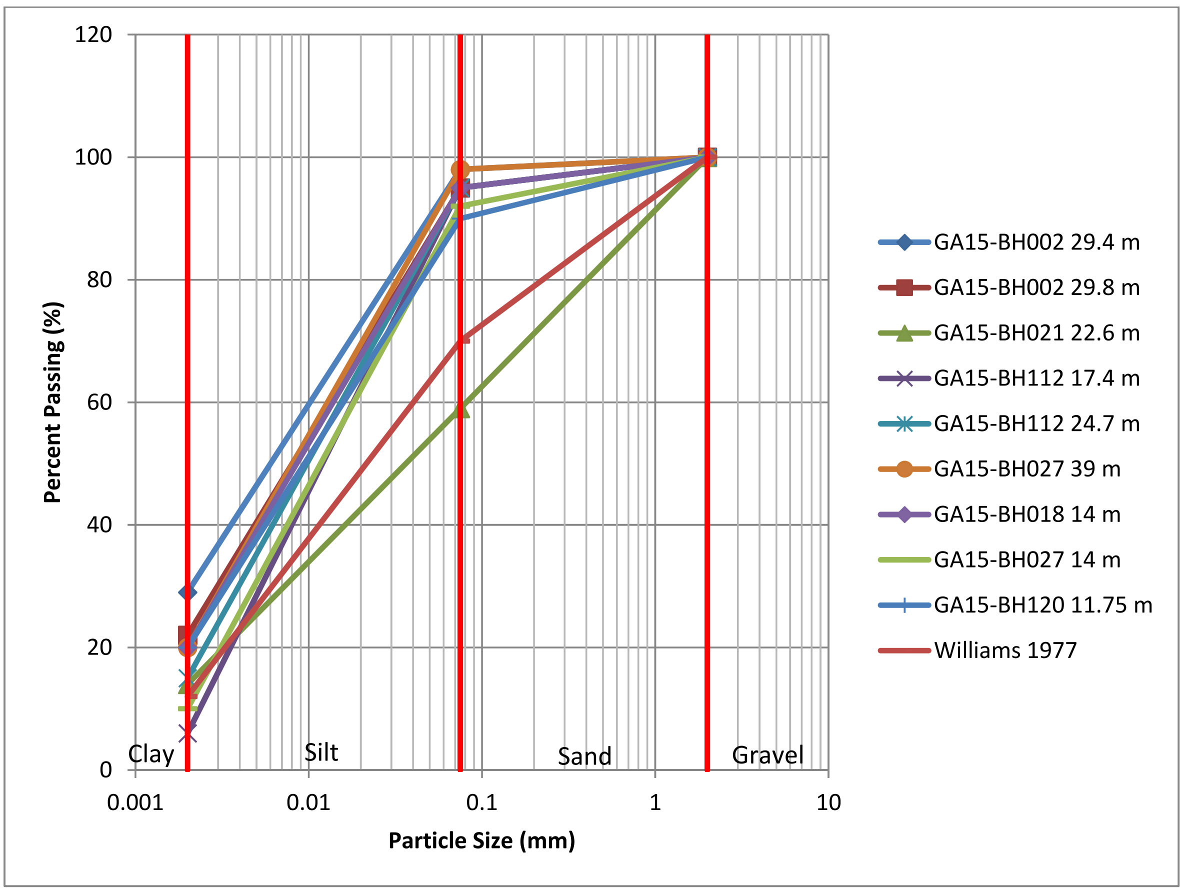


Plate 19: Estimated particle size distributions in Melbourne Formation

In summary, the Melbourne Formation is comprised predominantly of silt sized particles (0.002 mm to 0.075 mm). Sandstone typically has a high proportion of fine to medium grained sand (less than 0.2 mm grain size). The proportion of clay is typically less than 30%, with a greater proportion of clay present within more weathered material. However, the available data indicates that the proportion of clay/silt/sand varies significantly and can vary over a scale of a few centimetres.

Atterberg Limits

A collation of Atterberg Limit tests undertaken on Melbourne Formation materials is presented by Williams (1977). These tests have been undertaken on samples of highly or less weathered material obtained in the Melbourne CBD. The samples have been disintegrated and remoulded to allow the test to be undertaken. The results suggest a liquid limit of between 28% and 47% and a plasticity index of 9% to 27%. These results suggest that when disintegrated, the Melbourne Formation material is typically a low to medium plasticity clay (CL to CI). For specimens of extremely weathered or residual soil, Atterberg Limit tests usually classify the residual Melbourne Formation as a medium to high plasticity clay (CI to CH).

6.1.3 Soil Strength Properties

In its residual weathered state, the Melbourne Formation tends to behave as a soil and therefore it is appropriate to report soil properties for this material. Plate 20 presents a p-q plot for undrained triaxial tests with pore pressure measurement undertaken on samples of residual Melbourne Formation.

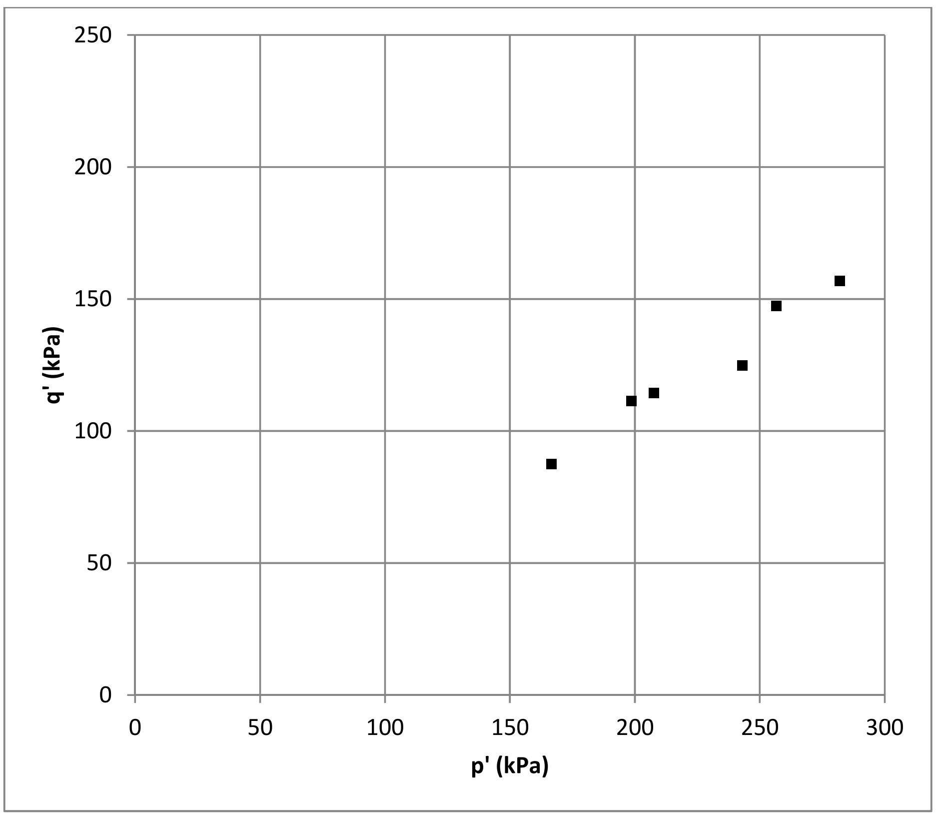


Plate 20: p-q plot for triaxial tests undertaken on residual Melbourne Formation materials

Although there is limited data available at this stage, the triaxial test results indicate a friction angle within the residual siltstone of approximately 30° and cohesion of less than 10 kPa.

6.1.4 Intact Rock Properties

Plates 21 to 25 present select results of testing undertaken on intact samples of Melbourne Formation. The intact properties of the Melbourne Formation have been measured mainly using Uniaxial Compressive Strength (UCS) testing and Point Load Strength Index (PLI) testing. Brazilian Tensile Strength testing has also been undertaken. Bulk density and stiffness measurements have also been taken on some samples tested for UCS.

Most of the Melbourne Formation siltstone is laminated with relatively close spaced bedding planes. This has a significant influence on the results of tests undertaken on intact samples of Melbourne Formation. It is typical for failure to occur along pre-existing planes of weakness or laminations within the rock mass. Consequently, measured UCS, modulus from UCS and point load indices are typically lower than they would otherwise be within massive Melbourne Formation material. There is also appreciable variability evident within the strength testing results.

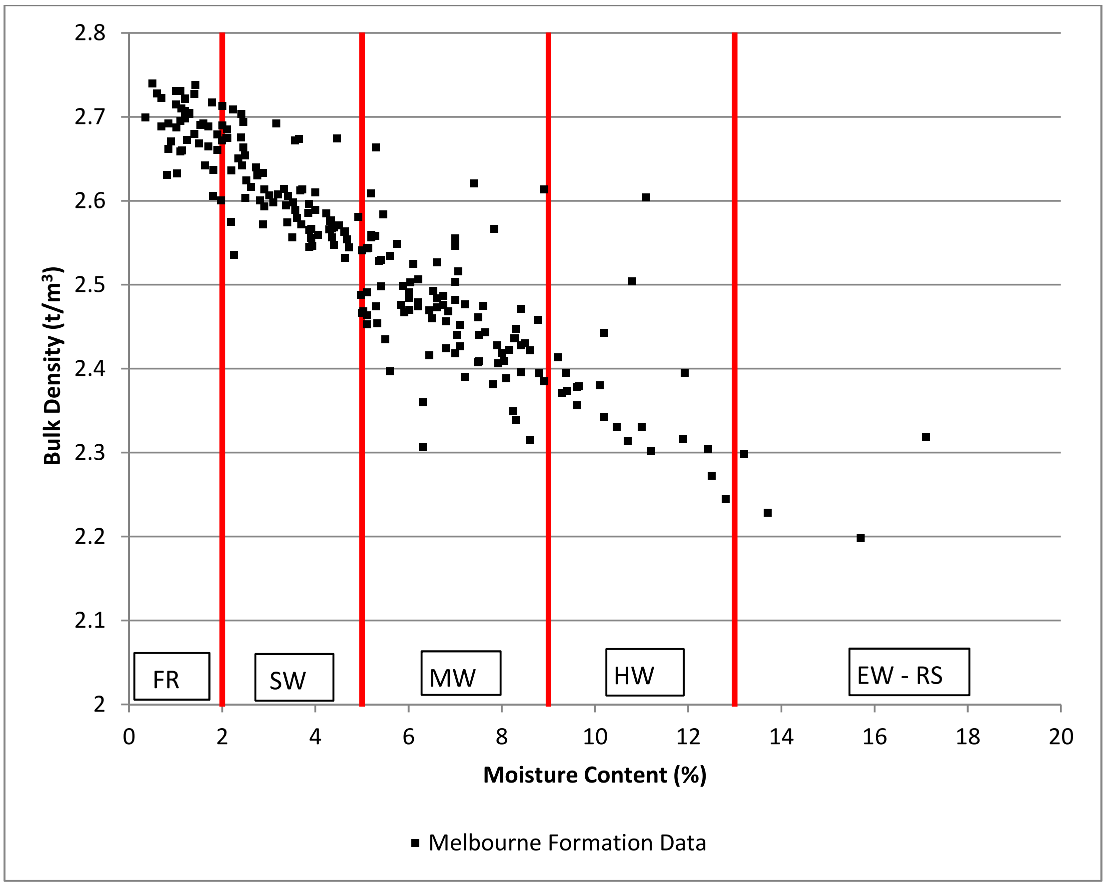


Plate 21: Bulk Density Measured on UCS samples versus moisture content for Melbourne Formation

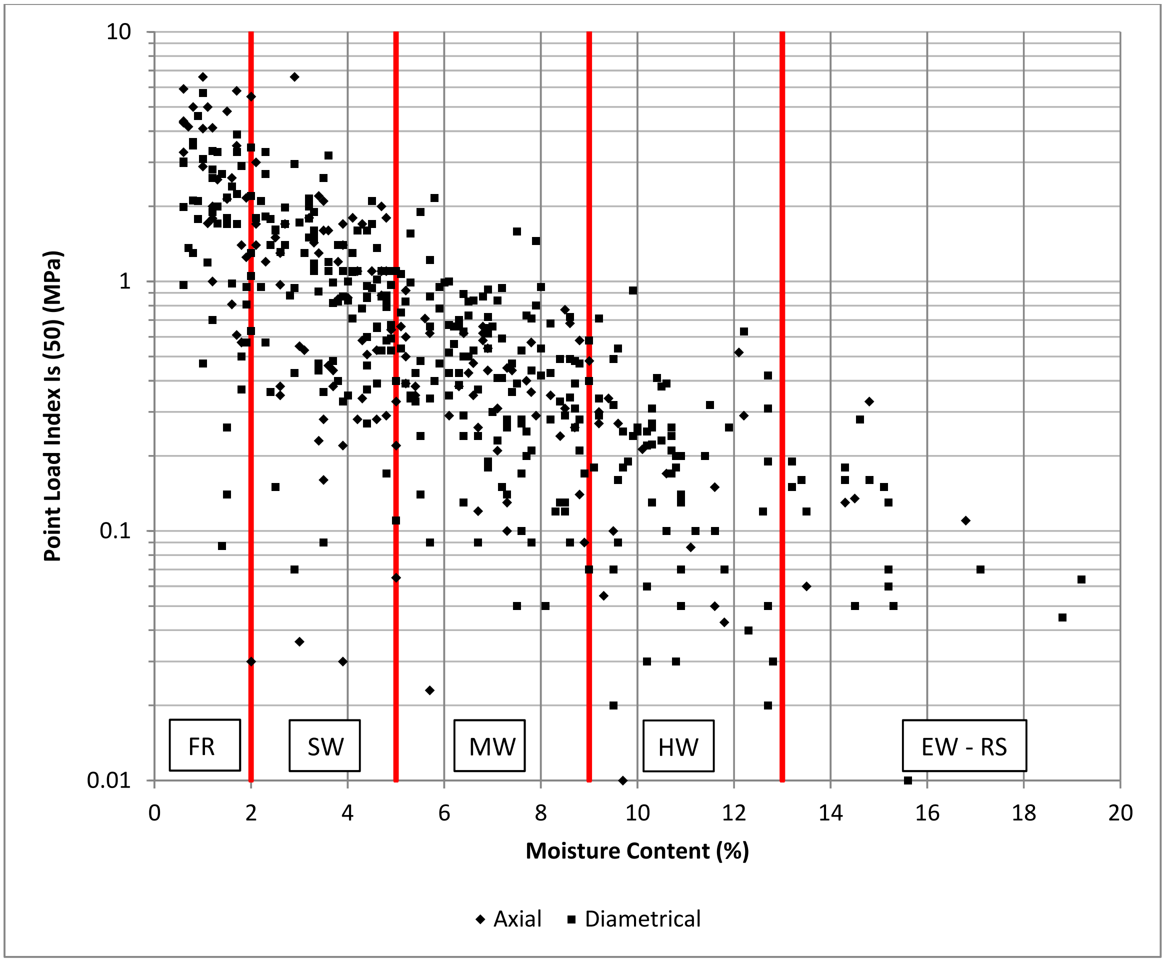


Plate 22: Point Load Index versus moisture content for Melbourne Formation

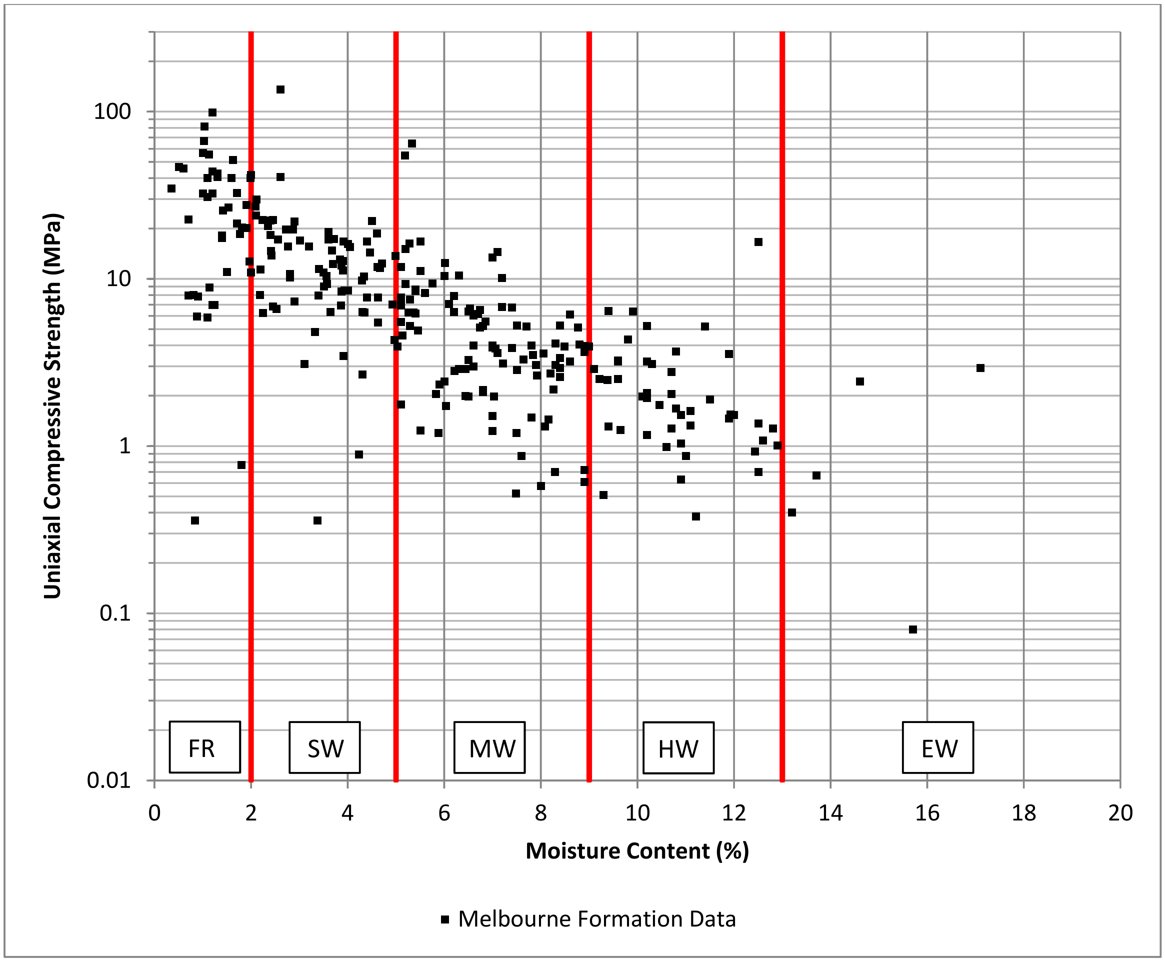


Plate 23: Uniaxial Compressive Strength versus moisture content for Melbourne Formation

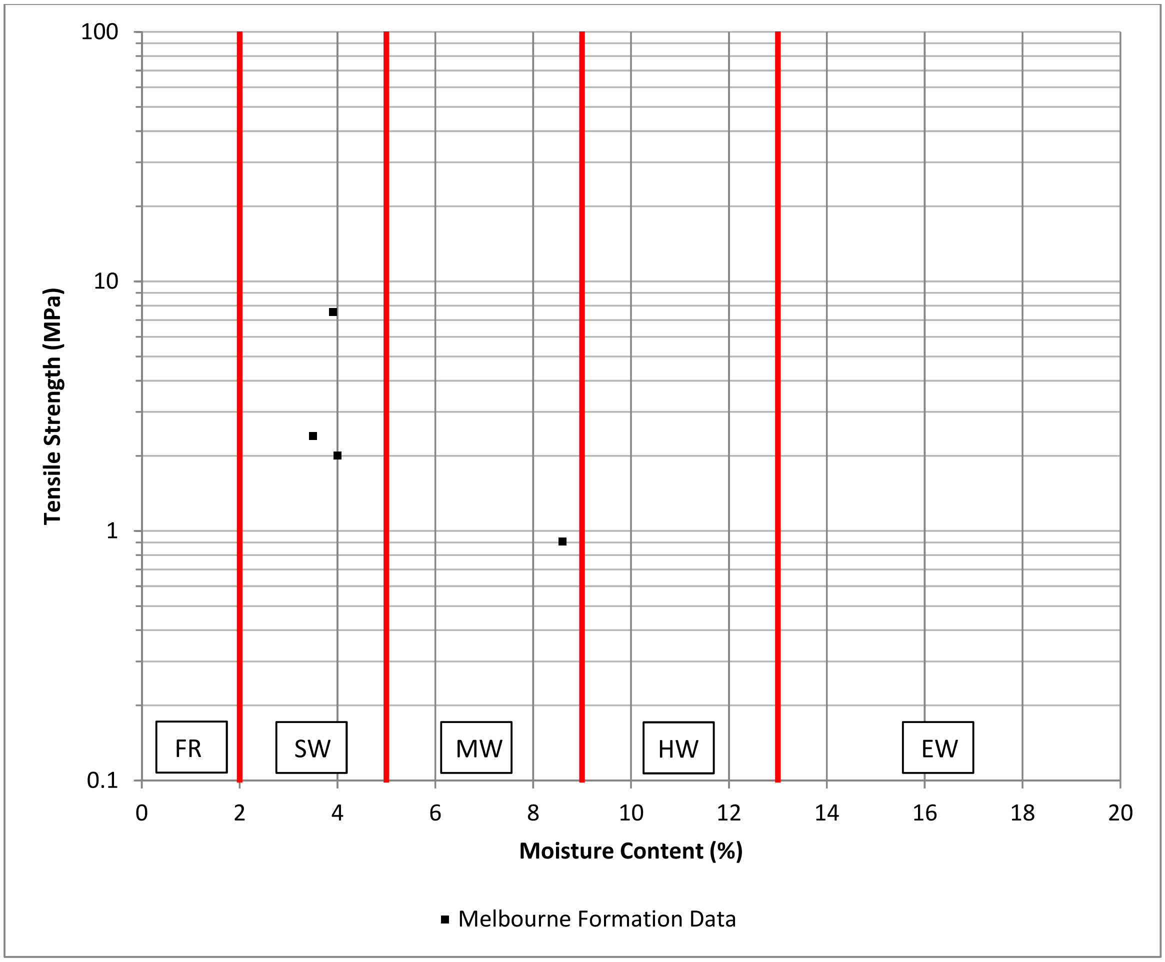


Plate 24: Brazilian Tensile Strength versus moisture content for Melbourne Formation

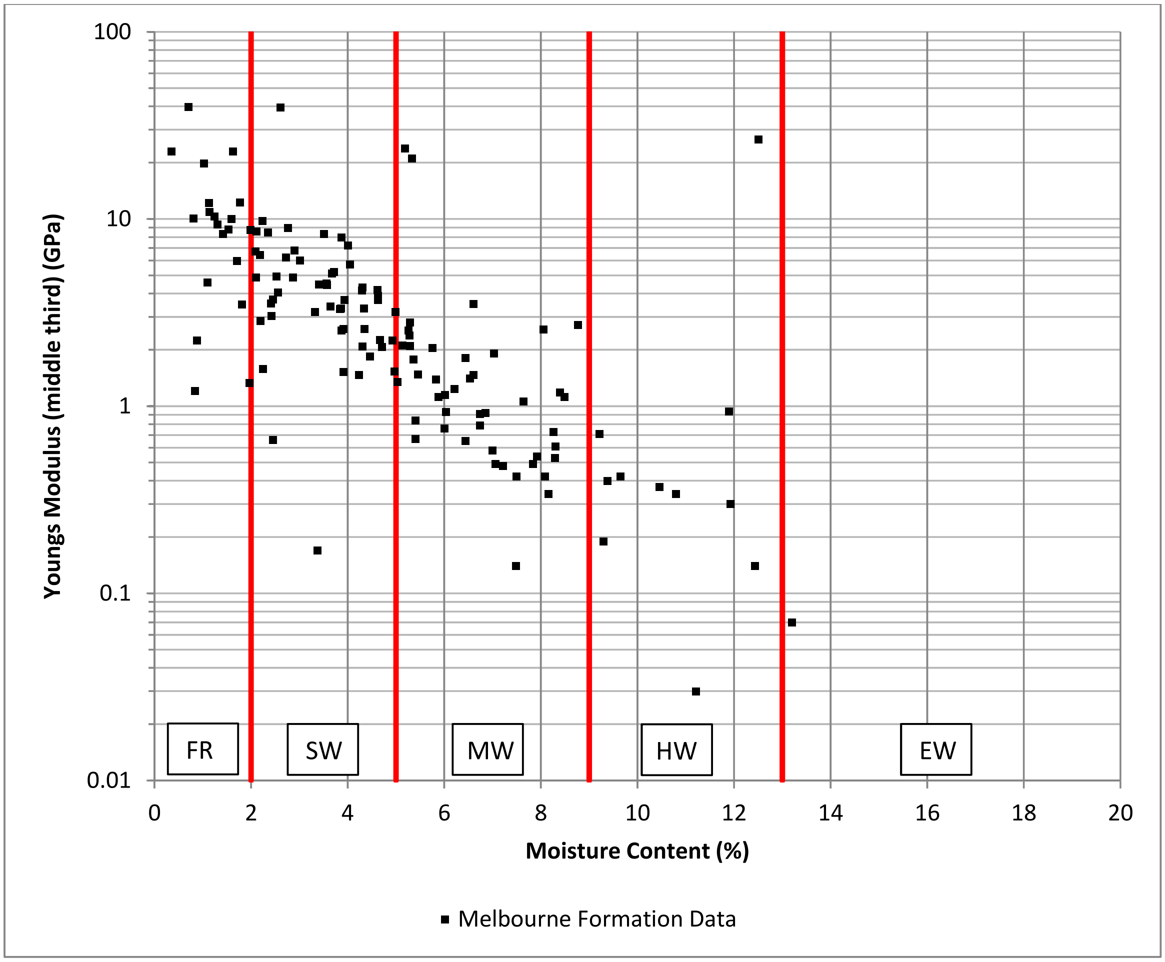


Plate 25: Middle Third Modulus versus moisture content for Melbourne Formation

Plates 21 to 25 indicate a general trend of increasing strength and stiffness with decreasing weathering grade. However there is appreciable scatter within the results of strength and stiffness testing. This scatter is inferred to be principally associated with pre-existing defects within the samples tested.

6.1.5 Rock Mass Properties

The deformation properties of the rock mass have been measured using in situ pressuremeter testing. Plates 26 and 27 present the results of pressuremeter testing obtained to date within the Melbourne Formation.

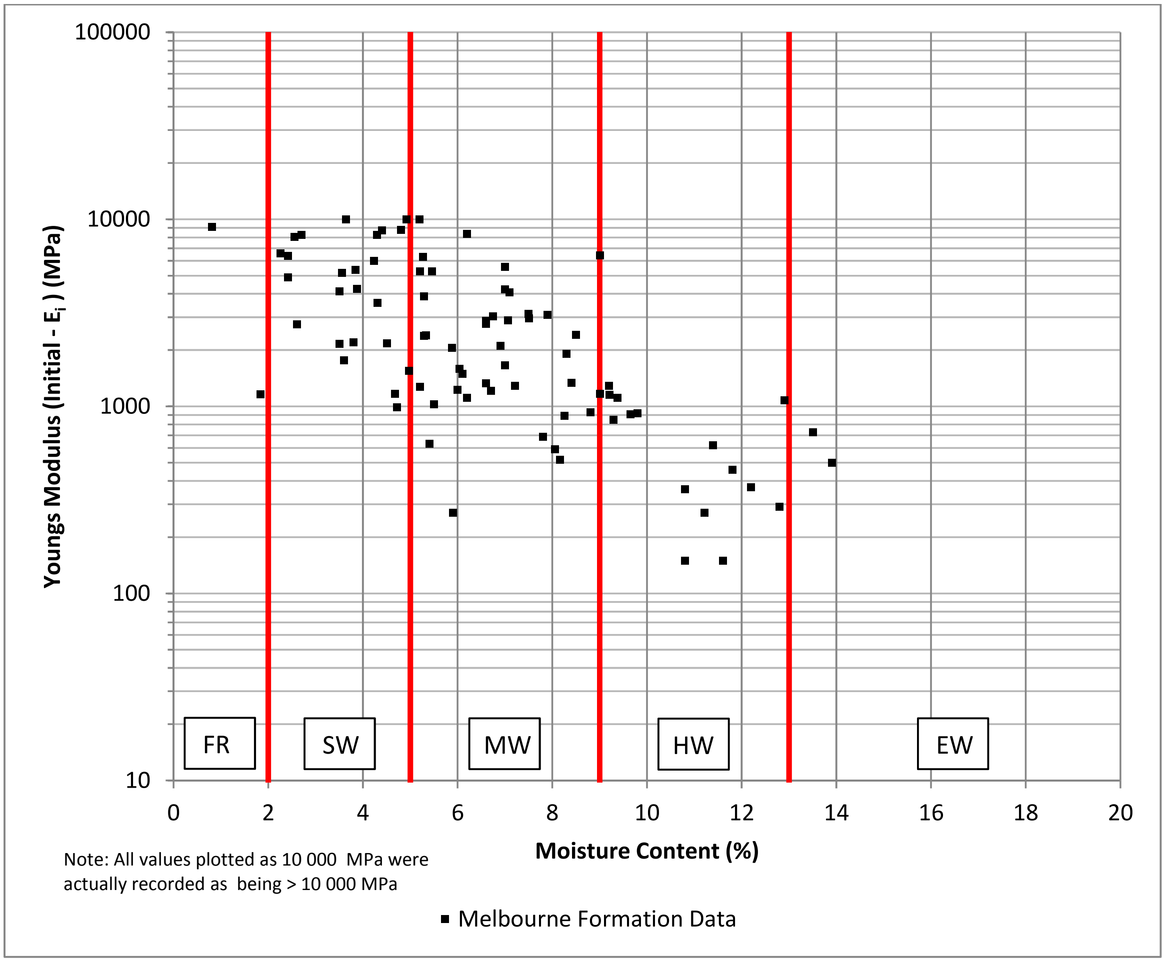


Plate 26: Initial Pressuremeter modulus versus moisture content in Melbourne Formation

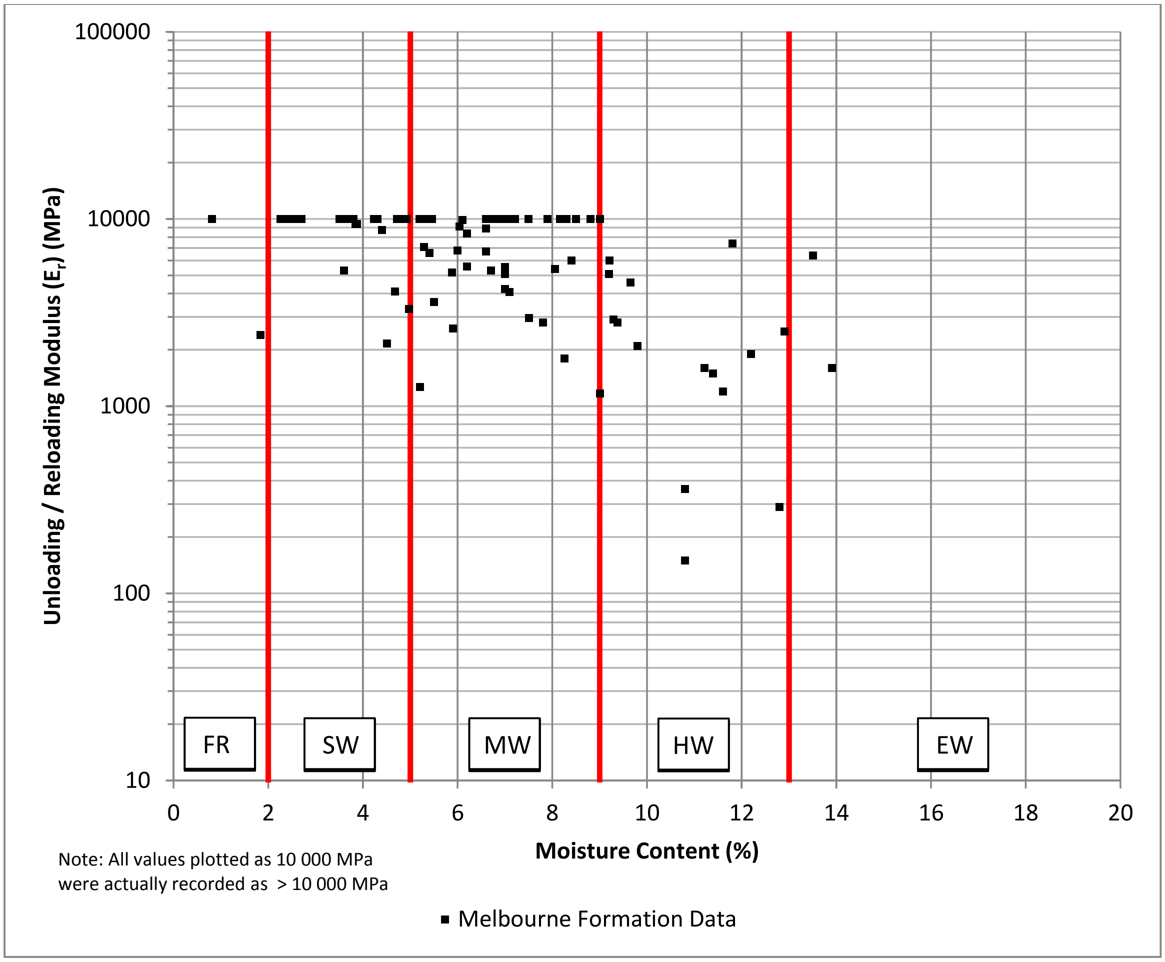


Plate 27: Unload-Reload pressuremeter modulus versus moisture content in Melbourne Formation

The variability within the modulus measured using the pressuremeter is significant and is inferred to be principally associated with the defects in the rock mass. It is noted that the variability within the mass properties measured is greater than the variability within the material properties and suggests that within Melbourne Formation it is the rock mass properties rather than material properties that have the greater influence on the behaviour of the rock mass.

The pressuremeter test loads a relatively small zone around the circumference of the borehole. An individual test is influenced by the number of discontinuities within the zone around the tested depth in the borehole that is loaded. For practical purposes, the pressuremeter tests are selectively undertaken within zones of rock with fewer discontinuities. Consequently, the modulus measured using the pressuremeter is likely to be higher than the rock mass modulus and this needs to be taken into consideration, along with the expected strain levels and whether the rock mass will be loaded or unloaded, when selecting values for design. The upper reported modulus of 10,000 MPa is considered to be the maximum modulus that should be assumed for the rock mass, irrespective of whether a test result measures a higher modulus at a select location.

6.2 In situ stress measurement

In situ stress measurement testing was undertaken in Boreholes GA15-BH010, GA15-BH011, GA15-BH012, GA15-BH027, GA15-BH028, GA15-BH032 and GA15-BH108. In situ test reports for each borehole were produced by Sigra and the in situ test results are summarised in Table 15.

Table 15: Summary of in situ stress testing

Borehole	Depth (m)	Major Principal Stress σ_{h1} (MPa)	Minor Principal Stress σ_{h2} (MPa)	Average Horizontal Stress $\sigma_{h,avg.}$ (MPa)	Estimated* Vertical Stress $\sigma_{v,est.}$ (MPa)	$\sigma_{h,avg.}$ / $\sigma_{v,est.}$	Direction of Major Principal Stress (σ_1) ° relative to grid north
GA15-BH010	31.0	2.63	2.00	2.32	0.74	3.11	95
GA15-BH010	32.9	2.02	1.35	1.69	0.79	2.13	137
GA15-BH011	21.1	1.40	1.21	1.31	0.51	2.58	128
GA15-BH011	39.1	4.45	2.41	3.43	0.94	3.66	105
GA15-BH012	19.0	1.03	0.61	0.82	0.46	1.80	40
GA15-BH012	27.0	1.57	1.32	1.45	0.65	2.23	64
GA15-BH012	35.4	2.84	2.04	2.44	0.85	2.87	78
GA15-BH027	25.0	0.87	0.48	0.68	0.60	1.13	145
GA15-BH027	31.0	1.08	0.61	0.85	0.74	1.14	156
GA15-BH027	36.0	0.66	0.20	0.43	0.86	0.50	174
GA15-BH028	39.0	4.28	2.12	3.20	0.94	3.42	137
GA15-BH032**	46.5	2.00	1.80	1.90	1.12	1.70	49
GA15-BH032**	46.5	1.53	1.38	1.46	1.12	1.30	49
GA15-BH032	50.5	4.67	3.36	4.02	1.21	3.31	175
GA15-BH032	55.0	5.05	2.16	3.61	1.32	2.73	13
GA15-BH108	23.8	0.65	0.21	0.43	0.57	0.75	129
GA15-BH108	30.7	0.35	0.21	0.28	0.74	0.38	131
GA15-BH108	32.2	1.11	0.77	0.94	0.77	1.22	114
GA15-BH108	38.1	0.90	0.43	0.67	0.91	0.73	139
GA15-BH108**	41.6	1.73	0.70	1.22	1.00	1.22	156
GA15-BH108**	41.6	2.00	0.81	1.41	1.00	1.41	156

*Assumes rock unit weight of 24 kN/m^3

6.2.1 Rock Mass Classification

Based on a review of the information obtained through the investigations undertaken specifically for Melbourne Metro, a rock mass classification system has been developed for the Melbourne Formation. Four rock mass categories (sub units) have been selected, MF1 through MF4, with the classification divisions based on the Geological Strength Index, GSI. Table 16 sets out the rock mass classification for the Melbourne Formation.

Table 16: Rock Mass Classification - Melbourne Formation

Rock Mass Unit	Description	Rock Mass Behaviour	GSI
MF1	Siltstone with interbedded sandstone, slightly weathered to fresh. Joints relatively closely spaced. Blocky rock mass with approximate joint spacing of about 200 mm. Some faults and shears. Dyke intrusions relatively unweathered.	These rock mass units are expected to behave as a 'blocky' rock mass. Rock failure mechanisms are controlled by the strength, spacing and orientation of discontinuities.	60 - 75
MF2	Siltstone and sandstone, generally moderately weathered. Blocky rock mass containing decomposed seams, shears and faults. Dykes where present are weathered to clay.		45 - 60
MF3	Siltstone and sandstone, generally highly weathered. Siltstone beds may be extremely weathered whilst sandstone is less weathered. Closely spaced discontinuities. Contains decomposed seams. Dykes where present are weathered to clay.	These rock mass units are expected to behave as a deformable rock mass with failure mechanisms controlled by the low rock mass strength i.e. failure through the low strength rock or along very weak discontinuities.	30 - 45
MF4	Generally extremely weathered siltstone and sandstone, with zones of hard clay. Discontinuities may present as fissures. Dykes where present are completely weathered to clay.		20 - 30

The defect orientations within the Melbourne Formation are variable and it is not possible to define regional scale joint sets or typical bedding orientation. Defect orientations should only be considered and interpreted in the vicinity of the borehole where they were measured.

Whilst a rock mass characterisation has been provided here, we note that the engineering behaviour of the Melbourne Formation can be dominated by persistent bedding planes. For example, the design case of bedding planes dipping into excavations should be considered. Friction angles of bedding planes within the Melbourne Formation as low as 13 degrees have been measured. Where highly to moderately weathered, the joint and bedding plane surfaces typically have iron oxide staining on their surfaces which tends to increase roughness. For the purposes of analysis, a friction angle of 25 degrees is typically assumed on the Melbourne Formation bedding plane surfaces.

6.2.2 Hardness and Durability

Laboratory test results for hardness and durability have been compiled for the Melbourne Formation and are presented in Table 17.

Table 17: Summary of Hardness and Durability Testing – Melbourne Formation

Weathering Grade	Highly Weathered	Moderately Weathered			Slightly Weathered			Fresh		
Measured		Lower Bound	Upper Bound	Average	Lower Bound	Upper Bound	Average	Lower Bound	Upper Bound	Average
Cerchar Abrasiveness Index	0.4(1)	0.56	2.25	1.1(14)	0.62	3.65	1.4(6)	1.02	2.07	1.50(3)
Brinell Hardness	-	23	380	89(12)	55	301	142(4)	71	373	187(17)
Rockwell A Hardness	-	27	71	40(12)	37	67	48(4)	40	71	55(17)
Rockwell B Hardness	-	29	125	55(12)	47	115	73(4)	54	124	88(17)
Rockwell C Hardness	-	2	41	8(12)	5	32	14(4)	6	40	18(17)
Goodrich Drillability	1150(1)	87	840	464(5)	-	-	292(1)	238	677	457(2)
Goodrich Wear Number	3.3(1)	2.3	7.9	4.8(5)	-	-	4.8(1)	2.5	5.1	3.8(2)
Goodrich Drillability/ Wear #	395(1)	15	362	144	-	-	62(1)	48	281	164(2)
Rock Toughness Index	-	-	-	-	-	-	-	-	-	3.54(1)

Number in parenthesis indicate number of test results available

Plate 28 presents the results of CERCHAR testing versus moisture content in Melbourne Formation.

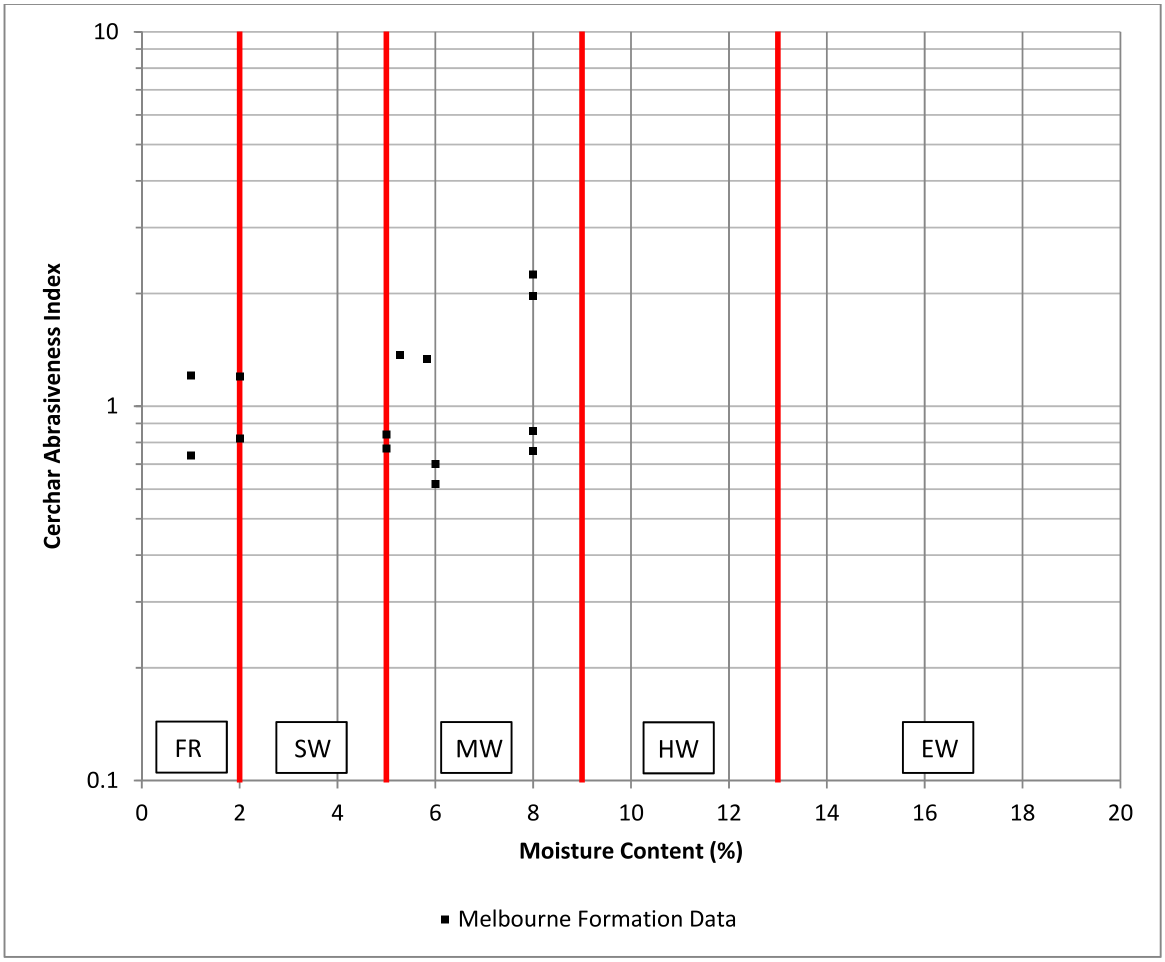


Plate 28: CERCHAR testing versus moisture content (where moisture content available)

The results indicate that the Melbourne Formation materials are slightly abrasive to abrasive. The abrasivity does not appear to vary significantly with weathering grade. The abrasivity is likely a function of the quartz content within the sample, and given quartz is largely resistant to weathering there is no significant change in weathering grade across samples. The hardness appears to be greater for lower degrees of weathering.

6.3 Tertiary Werribee Formation (Tew)

Engineering properties measured in the Werribee Formation are presented below.

6.3.1 Classification Properties

The Werribee Formation material encountered during the investigation in the Kensington area is described predominantly as sand, with minor clay beds. By contrast the material encountered during investigation in the North Melbourne area was described as clay. The results of particle size distribution and plasticity testing undertaken on Werribee Formation materials are presented in Plates 29 and 30.

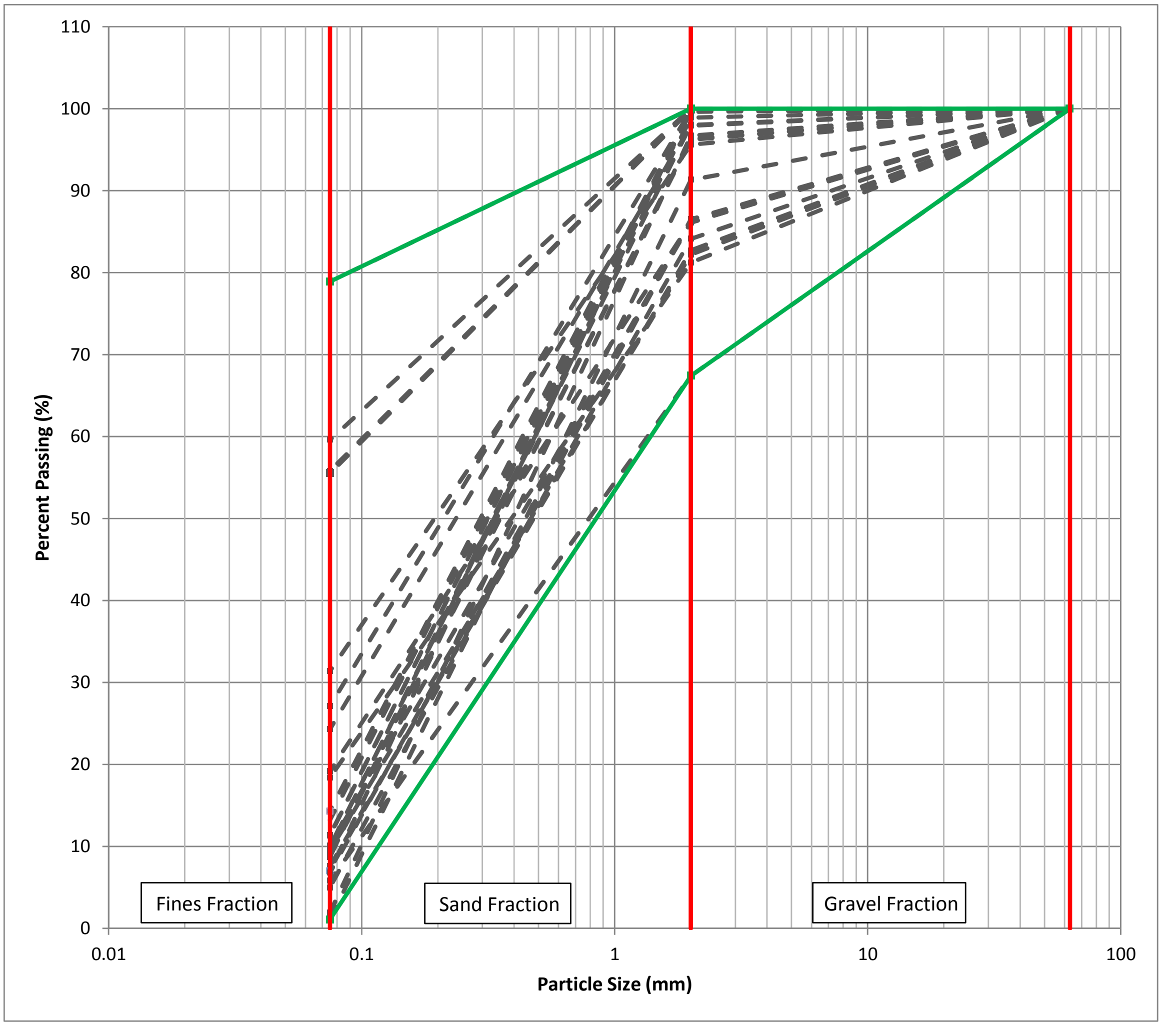


Plate 29: Approximate particle size distribution within the Werribee Formation

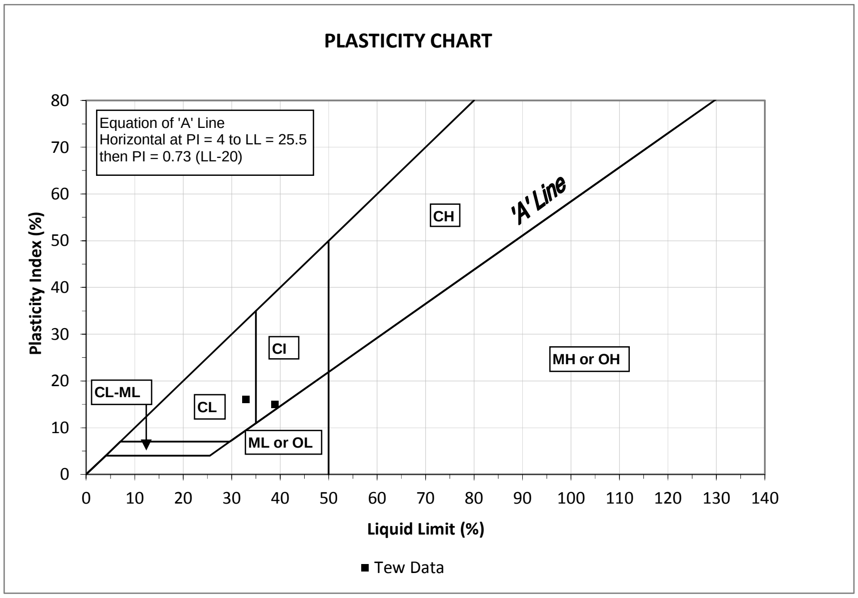


Plate 30: Atterberg Limit testing undertaken within the Werribee Formation

The results indicate that the Werribee Formation is predominantly sand, but can include beds with a higher proportion of sand and silt.

6.3.2 Consolidation Properties

The summary of consolidation characteristics for the Werribee Formation, presented in Bennet and Chandler (1992) is based on a limited amount of data and this limitation is acknowledged in the text. The tests referenced confirmed that the consolidation properties are highly variable with initial void ratios ranging from 0.5 to 1.7. An extract of consolidation parameters is presented in Table 18.

Table 18: Summary of Consolidation Parameters within the Werribee Formation

	Initial Void Ratio e_o	Coefficient of Consolidation C_v (m^2/yr)	Coefficient of Compressibility m_v (m^2/kN)	Coefficient of Secondary Consolidation C_α
Range[1]	*0.5 to 1.7*	*20 to 80*	*1 to 7.4 x 10^{-5}*	*0.0005*

1. Source Bennet and Chandler (1992)

6.3.3 Soil Strength

No triaxial test results are available as yet on samples of Werribee Formation. SPT test results within this material indicate N values of 30 to greater than 50, suggesting the Werribee Formation is predominantly comprised of dense to very dense sand. Albeit they are very approximate, correlations such as Peck et. al. 1974 indicates a friction angle of between 35 and 40 degrees.

6.4 Tertiary Older Volcanics (Tvo)

There are relatively few laboratory test results available in the Older Volcanics. The results of available tests are set out below.

6.4.1 Classification Properties

Moisture Content

Moisture content testing undertaken on residual Older Volcanics were found to be relatively high, with measured natural moisture contents of between 21% and 53%. The higher values were measured within the kaolin rich clay encountered between levels of about RL -5 m and -10 m AHD in the North Melbourne area.

Unit Weight

Limited data relating to the density of the Tertiary Older Volcanics has been gathered; however the limited available data suggest a unit weight of highly to moderately weathered material of about 24 kN/m^3.

Atterberg Limits

Results of classification testing on the residual soils of the Older Volcanics undertaken during the Stage 2 investigation in the Kensington/North Melbourne area are presented on Plate 31.

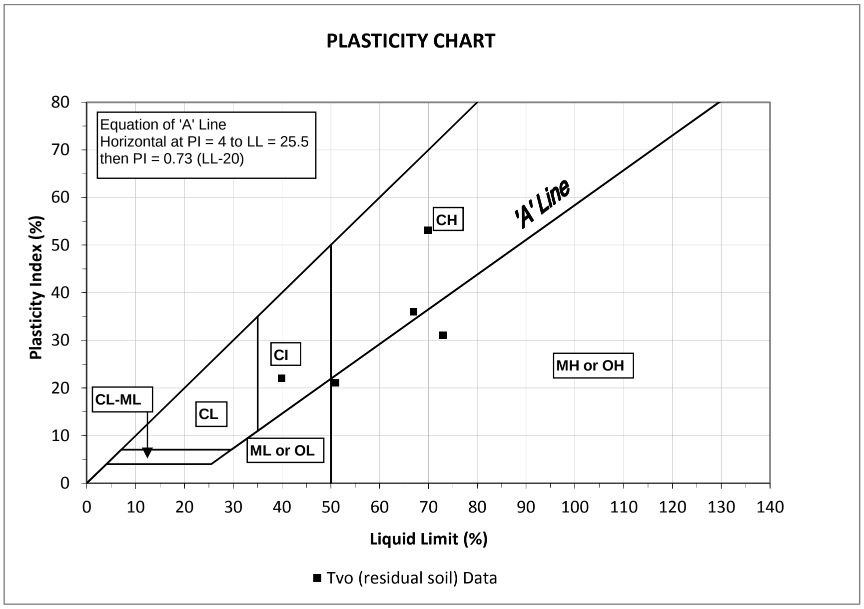


Plate 31: Results of Atterberg Limit Testing within Older Volcanics

The results indicate the residual Older Volcanics are a medium to high plasticity clay. The Older Volcanics appear to be derived from basalt and tuff. The tuff appears to be more susceptible to weathering, at some locations completely weathered to kaolinite.

6.4.2 Engineering Properties of Intact Rock and Soil

In the extremely weathered state, the Older Volcanic materials form a residual soil, and as such, their engineering properties are more appropriately applicable to soils. The results of 5 in-situ pressuremeter tests in this material indicate that the undrained shear strength (S_u) ranges from 240 kPa to 480 kPa with an average value of 360 kPa.

One triaxial test with pore pressure measurement was undertaken on a sample of residual Older Volcanics as part of the Stage 2 investigation. The results of this test suggest a drained effective/cohesive c′ of 14 kPa and drained friction angle, Φ′ of 25˚. These results are typical for a clay material.

The properties of intact rock have been measured mainly using Uniaxial Compressive Strength (UCS) testing and PLI testing. Both axial and diametrical PLI results have been considered. Stiffness measurement has been undertaken on some samples tested for UCS.

Plate 32 presents a histogram showing the available results of strength testing on intact samples of Older Volcanics.

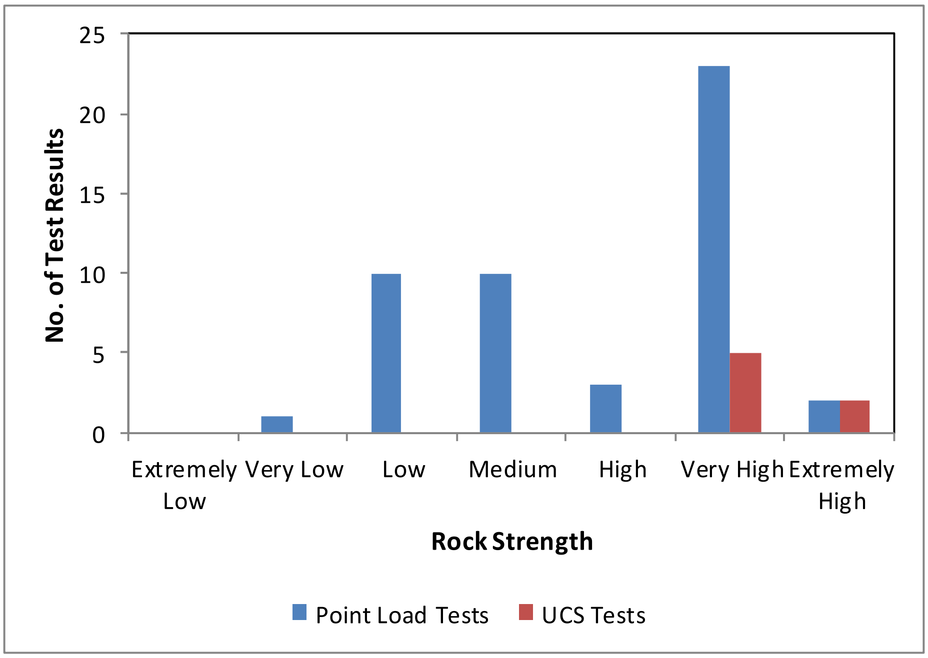


Plate 32: Summary of testing of intact Older Volcanics

Plate 32 indicates that whilst most intact material is very high strength, there is an appreciable spread of results. Intact samples of Older Volcanics of sufficient volume to test are rare. Consequently tests are biased towards higher quality rock which often represents core stones within the more weathered rock mass. Consequently, insitu stress testing within the Older Volcanics should be treated with some degree of caution as it may be overestimating the material strength of the rock.

6.4.3 Rock Mass Classification

Table 19 presents a proposed rock mass for the Older Volcanics. As is the case with the classification developed for the Melbourne Formation, four categories (sub units) have been developed based on the estimated geological strength index.

Table 19: Preliminary Rock Mass Classification – Older Volcanics

Rock Mass Unit	Description	Rock Mass Behaviour	GSI
OV1	Basalt, high to very high strength, slightly weathered, widely spaced joints, joints tight with rough joint surfaces.	These rock mass units are expected to behave as a 'blocky' rock mass. Rock failure mechanisms are controlled by the strength, spacing and orientation of discontinuities.	60 - 75
OV2	Basalt, medium to high strength, slightly weathered, moderately to widely spaced joints, joints generally tight with rough joint surfaces.		45 - 60
OV3	Basalt, low to medium strength, moderately weathered, closely to moderately spaced joints, joint surfaces altered and clay filled.	These rock mass units are expected to behave as a deformable rock mass with failure mechanisms controlled by the low rock mass strength i.e. failure through the low strength rock or along very weak discontinuities.	35 - 45
OV4	Basalt, extremely low to low strength, extremely to highly weathered, very closely to closely spaced joints, joint surfaces altered and clay filled.		25 - 35

6.4.4 Hardness and Durability

One suite of testing for hardness and durability was undertaken on a sample of highly to moderately weathered Older Volcanics obtained during the most recent investigation. Select parameters from the suite of tests are summarised in Table 20. We note that only one suite of tests to assess hardness and abrasivity has been undertaken in the Older Volcanics and that additional tests would be required to establish typical parameters for this material.

Table 20: Select hardness and abrasivity tests in Older Volcanics Basalt

Weathering Grade	Highly to Moderately Weathered
Cerchar Abrasiveness Index	1.21
Brinell Hardness	8
Rockwell A Hardness	19
Rockwell B Hardness	17
Rockwell C Hardness	1
Rock Toughness Index	-

The results of this particular test indicate the material is moderately abrasive and of low hardness.

6.5 Tertiary Brighton Group (Tpb)

6.5.1 Classification Properties

We have a number of laboratory soil classification test results in the immediate vicinity of the alignment, including those obtained in the Stage 2 and most recent stage investigations. Plates 33 to 35 present classification test data for the Brighton Group materials.

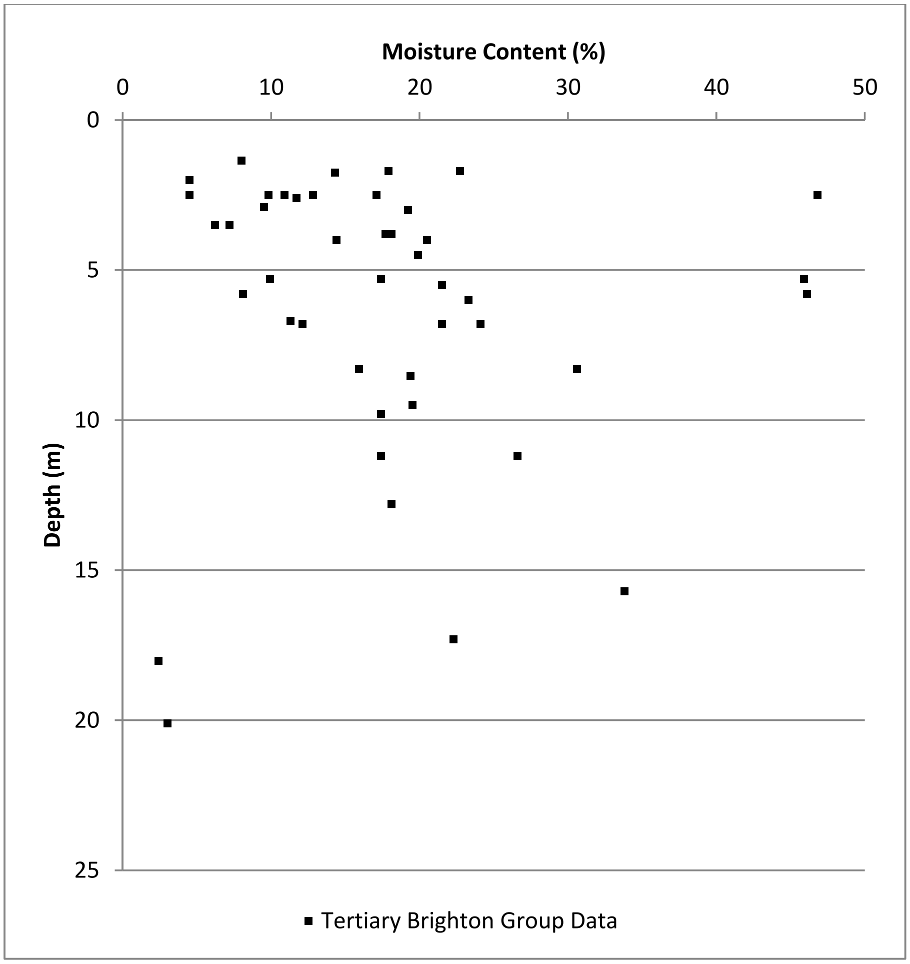


Plate 33: Moisture content versus depth in Brighton Group materials

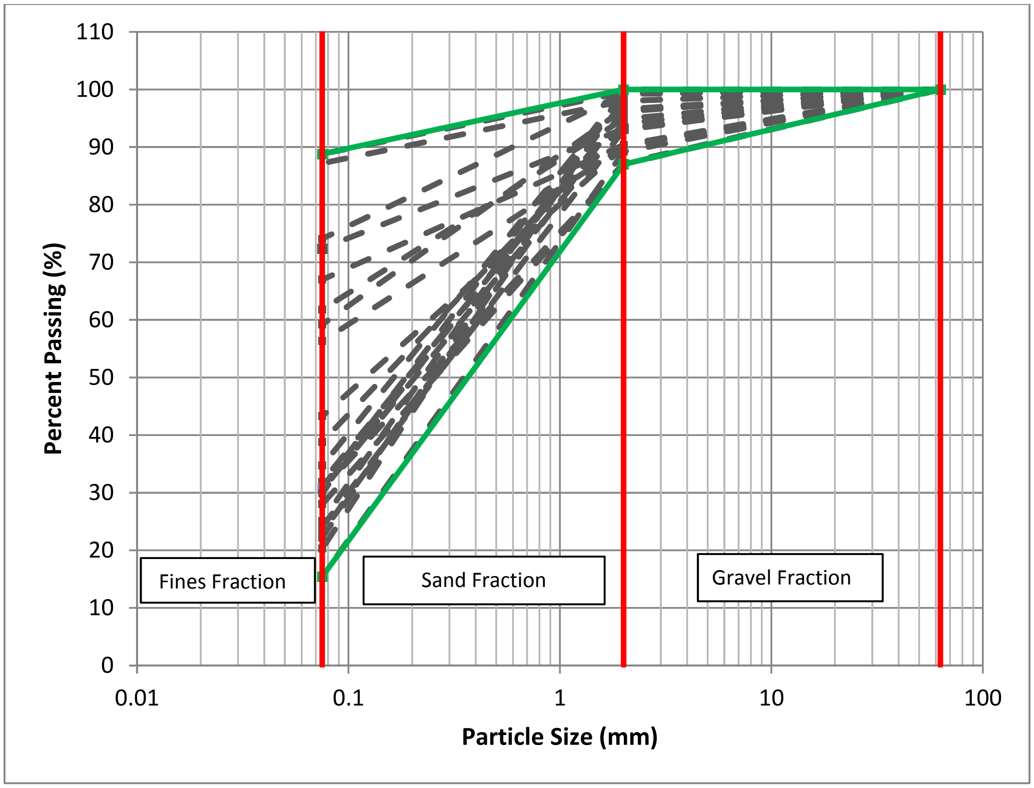


Plate 34: Particle size distributions measured in Brighton Group materials

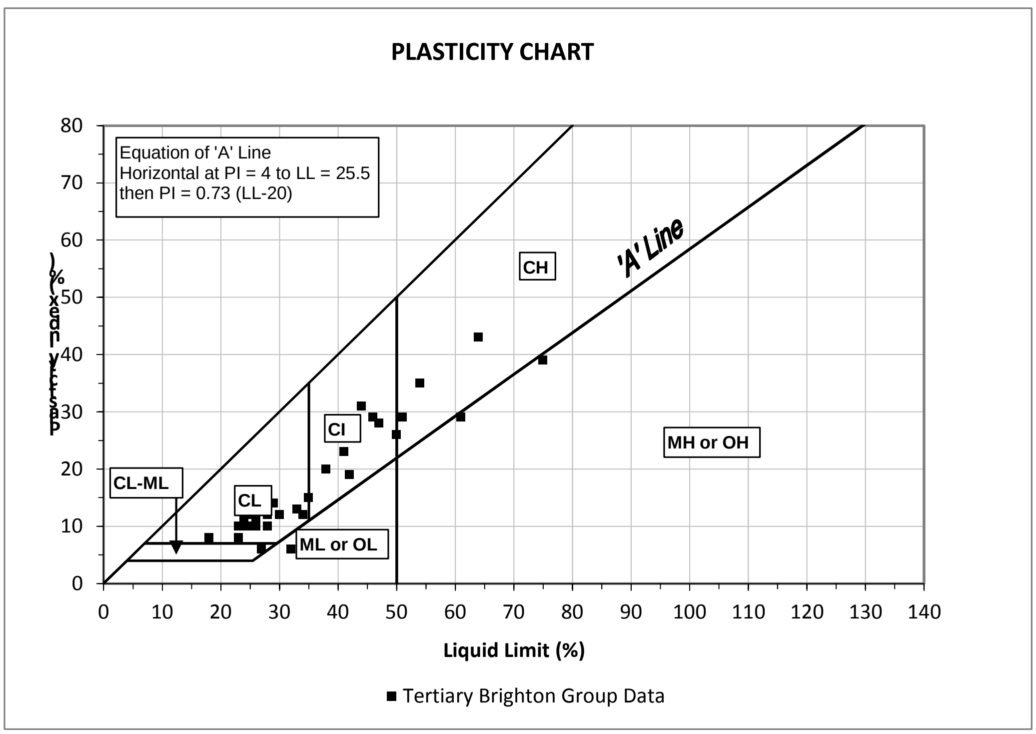


Plate 35: Plasticity of Brighton Group Materials

The results indicate that the Brighton Group is a highly heterogeneous material with composition varying between predominantly clay and predominantly sand. The resolution of ground information available at this stage of the project is insufficient to identify the spatial distribution of Brighton Group materials of different composition. With reference to Plate 33, there is no apparent trend with moisture content, or material type with depth. Caution should be used in adopting generic parameters for design in this material.

6.5.2 Soil Strength

The results of undrained triaxial tests with pore pressure measurement are presented on a p-q plot in Plate 36. Due to practicalities in sampling and testing intact samples of Brighton Group, the triaxial tests were undertaken on samples with sufficient clay to prevent the sample from unravelling when unconfined.

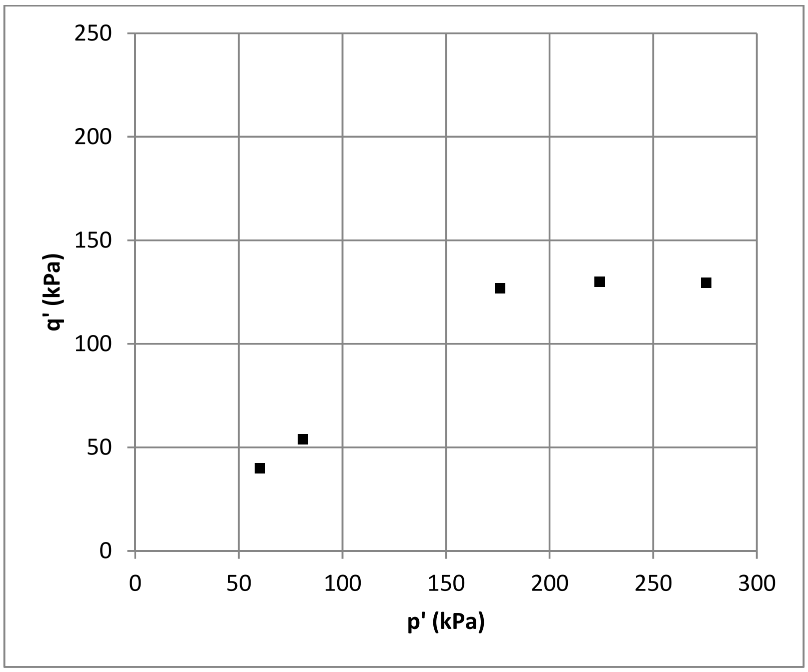


Plate 36: p - q plot for undrained triaxial testing with pore pressure measurement undertaken in Brighton Group

The p-q plot indicates a friction angle for clayey Brighton Group of 25° to 30° and a cohesion of up about 10 kPa to 20 kPa.

6.6 Punt Road Sands (Qpp)

One test was undertaken on materials recovered from the sediments at the base of the Jolimont Valley. A particle size distribution test on this material indicates that it is predominantly clay and silt, with 66.7% fines and 33.3% sand. However, the nature of deposition of this material indicates that compositional variability should be expected.

6.7 Swan Street Basalt (Qvns)

Table 21 presents the results of strength testing on this material obtained during the Stage 2 (2 tests) and testing results obtained during the desk study.

Table 21: Properties for Quaternary Lower Newer Volcanics

Measured	UCS (MPa)	Secant Modulus E_{rm} (MPa)	Point Load Strength Index (MPa)
Lower Bound	*5(16)*	*2000(5)*	*0.08(5)*
Upper Bound	*163(16)*	*46000(5)*	*12.8(5)*
Average	*95(16)*	*25800(5)*	*5.0(5)*

Numbers in Parenthesis refer to the number of test results

The Swan Street basalt is a very high strength rock. Furthermore, it contains few discontinuities.

6.8 Early Pleistocene Colluvial and Alluvial Sediment (Qpc)

There is little information available from within this unit. However, based on just two available test results and with reference to Plate 37, it is apparent that the composition of this material varies significantly between material containing predominantly clay and material containing predominantly sand. This is not unusual for an alluvial material. At this stage, the resolution of information is not sufficient to be able to delineate between material containing predominantly sand and material containing predominantly clay.

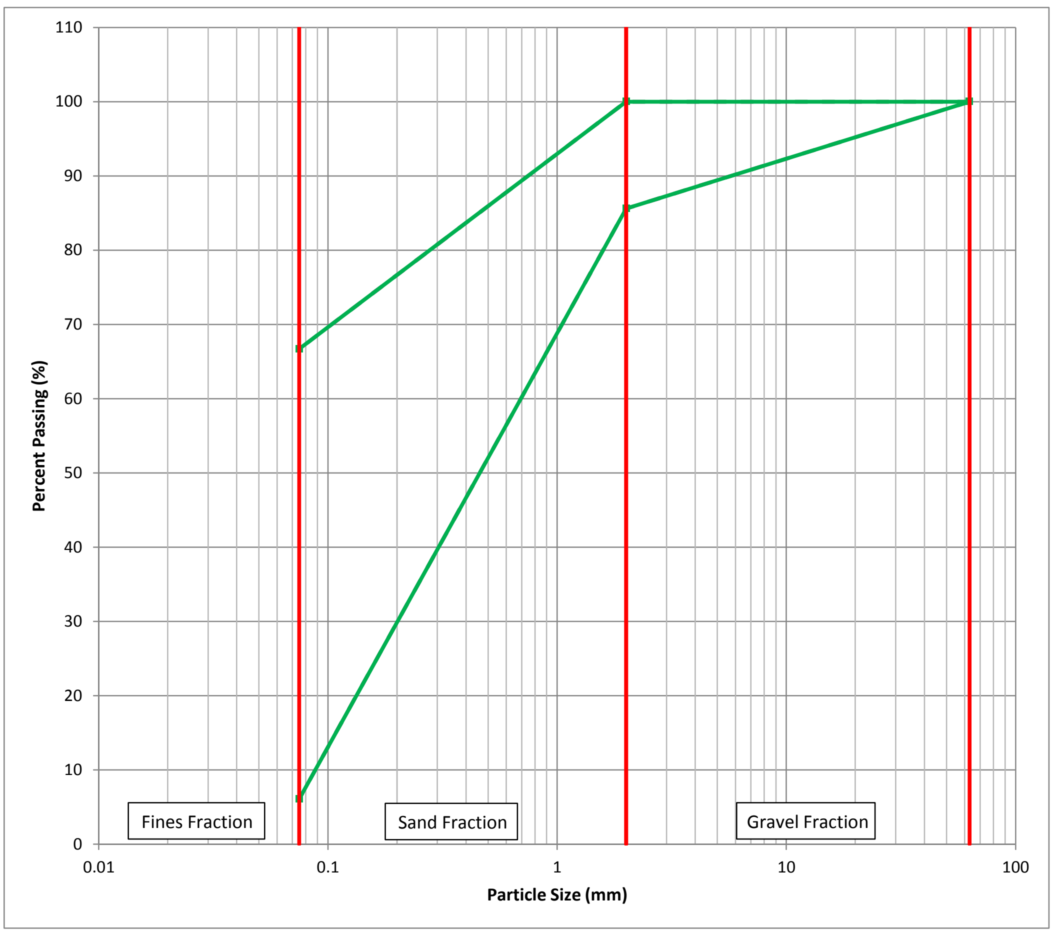


Plate 37: Particle size distribution testing (two tests) within Early Pleistocene Colluvial and Alluvial Sediment

There is no strength testing available within this material. However, SPT testing indicates N values of 28 to greater than 50. The variable composition of this material probably renders correlations with SPT unreliable. However, notwithstanding that, using Peck, 1974, a friction angle of about 35° can be considered to be reasonable.

6.9 Quaternary Moray Street Gravels (Qpg)

The Moray Street Gravels have a relatively variable composition, including clay, silt, sand and gravel. Plate 38 presents the results of classification testing undertaken on samples of Moray Street Gravels. These indicate a predominantly granular material with varying proportion of fines.

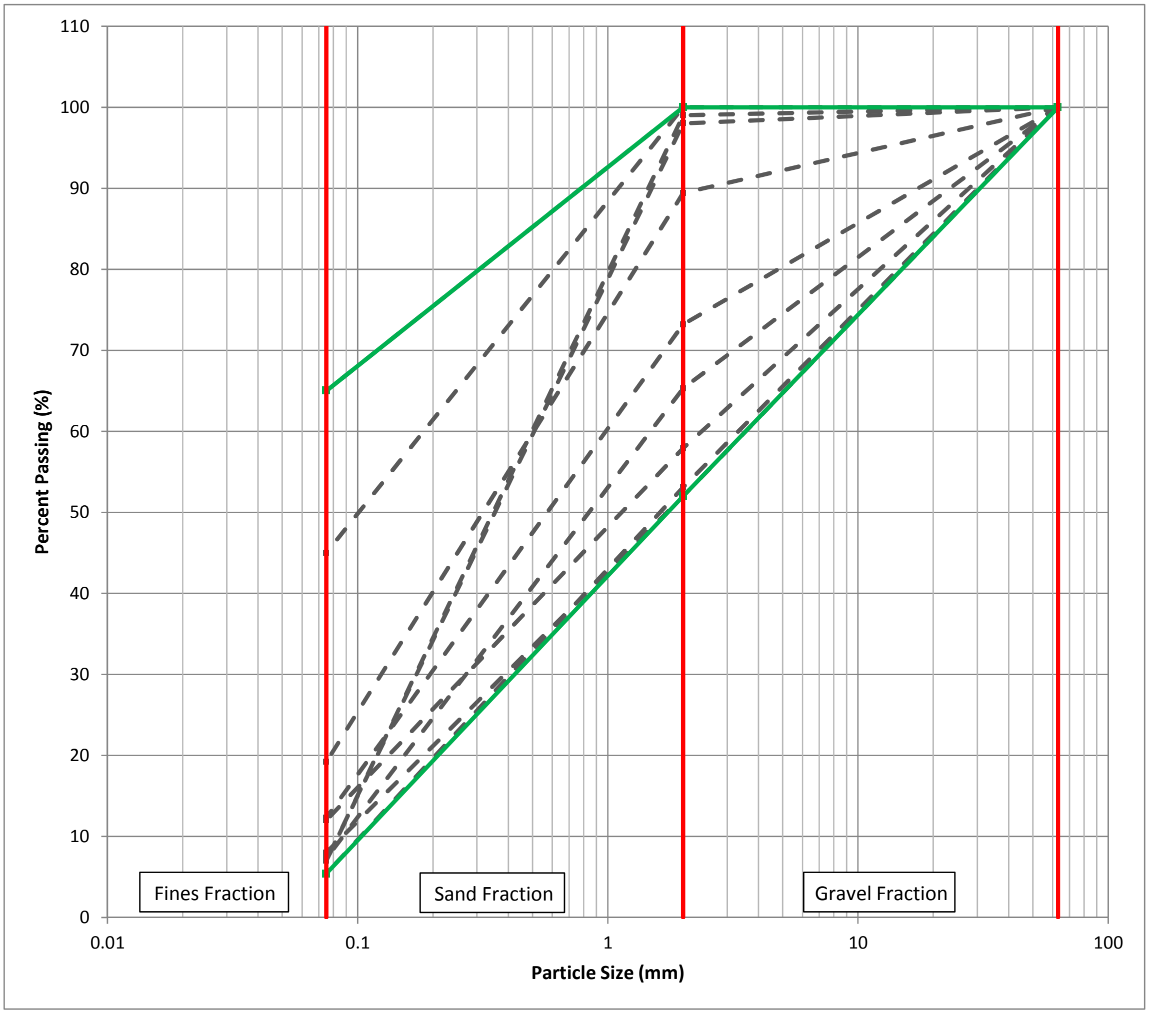


Plate 38: Particle size distribution within Moray Street Gravels

No strength testing has been undertaken within the Moray Street Gravels. However, correlation with SPT (Peck et al 1974) suggests an effective friction angle of about 35°.

6.10 Quaternary Fishermens Bend Silt (Qpf)

A reasonably large data set, including information from the Stage 2 investigation has been acquired relating to the Fishermens Bend Silt, although we note that little information regarding the consolidation characteristics is available. It is anticipated that this material would be susceptible to settlement or heave as a response to loading and unloading respectively.

Further investigation would be required to ascertain appropriate settlement characteristics where the alignment passes close to, or through the Fishermens Bend Silt. In the absence of local data, extracts (printed in italics) from Ervin (1992) are reproduced below:

There does appear to be variability in compressibility at different sites, although not necessarily following the trend indicated by the plasticity of the clays, as might be expected (Terzaghi and Peck, 1967). Data on the compression ratio is presented in Table 22.

Table 22: Fishermens Bend Silt – Compression ratio

Site	*No. of Results*	*Compression Ratio, $Cc / (1 + e_0)$*	
		Range	*Average*
South Melbourne	*13*	*0.063 to 0.197*	*0.140*
Ingles Street	*7*	*0.177 to 0.300*	*0.227*
Appleton Dock	*2*	*0.270 to 0.319*	*0.295*
Port Melbourne	*2*	*0.145 to 0.188*	*0.167*
Webb Dock	*5*	*0.075 to 0.164*	*0.127*

A summary of the available classification data within the Fishermens Bend Silt is presented in Plates 39 to 41.

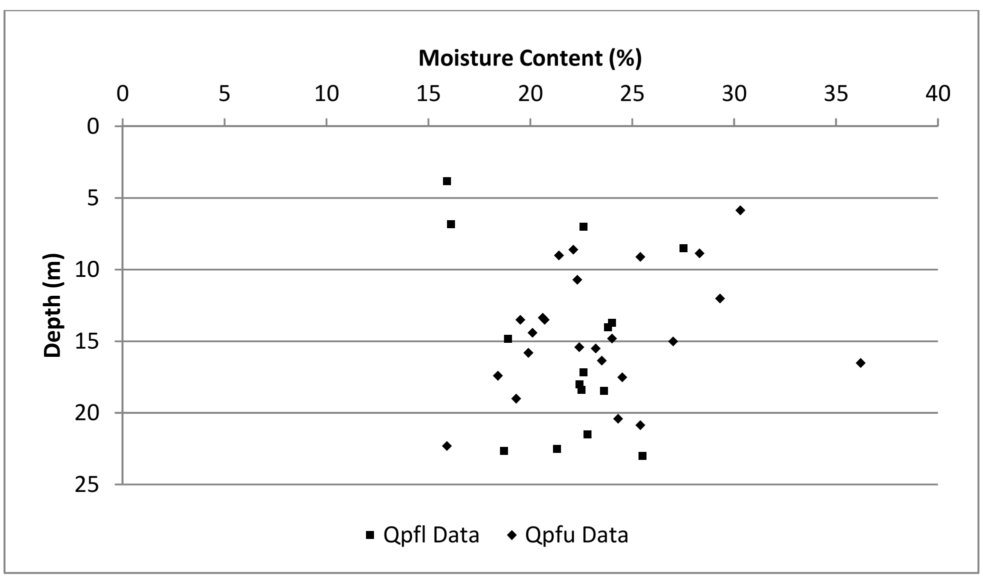


Plate 39: Moisture content versus depth in Fishermens Bend Silt. Qpfl refers to lower, typically sandier material and Qpfu refers to the upper, typically more clayey material

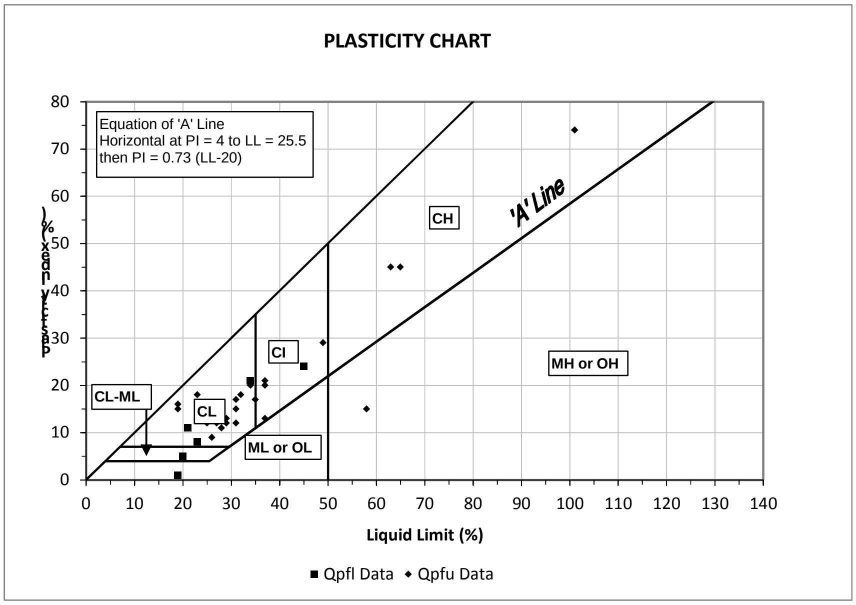


Plate 40: Plasticity of Fishermens Bend Silt

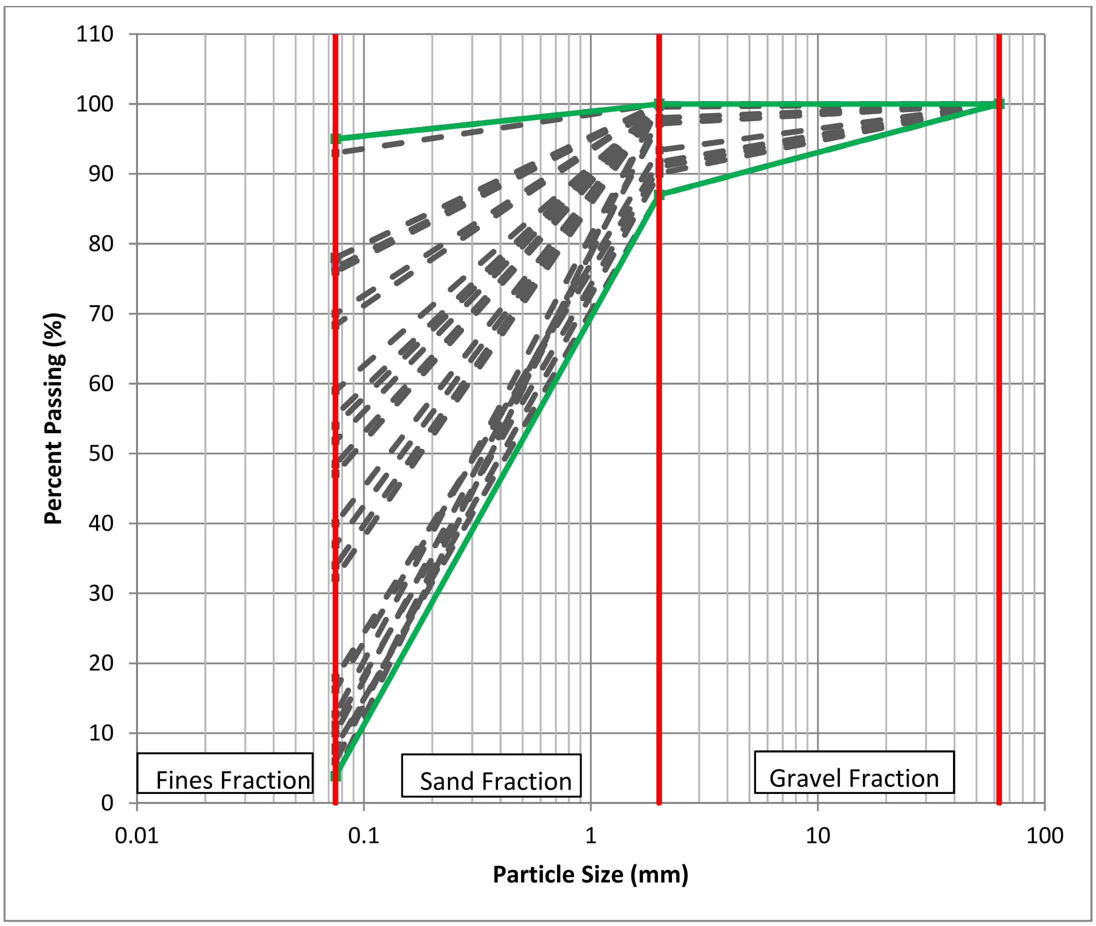


Plate 41: Particle Size Distribution in Fishermens Bend Silt

The classification tests indicate that the proportion of sand to clay varies significantly within this material. Plate 39 presents moisture contents for the Fishermens Bend Silt Upper (predominantly clay) and Lower (predominantly sand). There is no clear trend of composition with depth. Where required, classification of the Fishermens Bend Silt at a segment scale would require site specific testing to estimate the proportion of clay to sand and to estimate parameters for design.

Table 23 presents the results of triaxial strength tests performed on the Fishermens Bend Silt from data obtained within the vicinity of the alignment. These tests are not from borehole drilled specifically for Melbourne Metro.

Table 23: Undrained and Drained Strength Results within the Fishermens Bend Silt

Measured	S_U (kPa)	c' (kPa)	ϕ' (Degrees)
Lower Bound	*35*(74)	*15*(3)	*23*(3)
Upper Bound	*285*(74)	*40*(3)	*33*(3)
Average	*110*(74)	*27*(3)	*28*(3)

Number in parenthesis refer to the number of test results available

6.11 Pleistocene Alluvium (Qpa)

No laboratory test results are available within the Quaternary Alluvium. However, we note that the age and composition of this material is similar to the Fishermens Bend Silt and Jolimont Clay.

6.12 Quaternary Newer Volcanics Basalt (Qvn)

The Newer Volcanics basalt within the Jolimont Valley has a younger age and is less weathered than other Newer Volcanics Basalt within the Melbourne area. A summary of the results of laboratory strength tests performed on this material is presented in Plate 42. In addition to the data presented in Plate 42, tensile strengths of 6.5 MPa to 7.1 MPa have been measured on 3 samples.

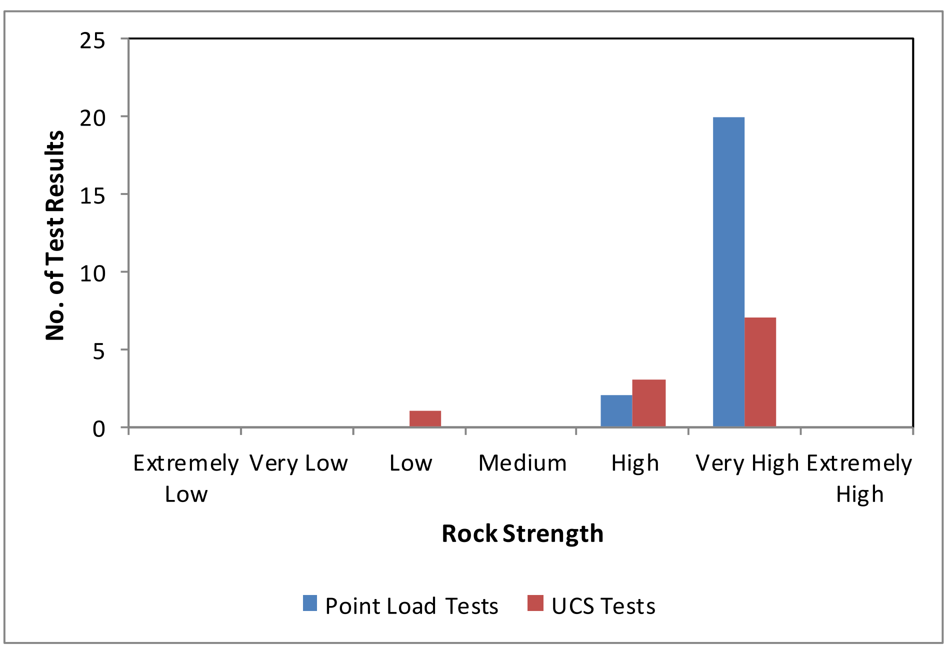


Plate 42: Histogram showing the results of strength testing within the Newer Volcanics Basalt

Two samples obtained during the Stage 2 investigation from boreholes GA11-BH033 and BH039 underwent a TBM suite of rock tests. Select parameters for hardness and durability are summarised in Table 24 below:

Table 24: Summary of Hardness & Durability Tests for Quaternary Newer Volcanics Basalt

Measured	Lower Bound	Upper Bound
Sklerograf Hardness	*40(2)*	*48(2)*
Shore Hardness	*36(2)*	*47(2)*
Brinell Hardness	*234(2)*	*305(2)*
Rockwell A Hardness	*61(2)*	*67(2)*
Rockwell B Hardness	*104(2)*	*116(2)*
Rockwell C Hardness	*24(2)*	*33(2)*
Cerchar Abrasivity	*4.5(2)*	*5.2(2)*
Schimazek Wear Index*	*2.3(2)***	*3.3(2)***

** Based on a correlation with Cerchar Abrasivity Index by the department of civil and environmental engineering, University of Melbourne.

Number in parenthesis refer to the number of test results available

These tests indicate the basalt is likely to be highly abrasive and relatively hard.

6.13 Quaternary Jolimont Clay (Qpj)

No laboratory test data relating to the Jolimont Clay close to the alignment was found. However, we note that the Jolimont Clay has a similar composition to the Fishermens Bend Silt and it is expected to exhibit similar engineering behaviour. The alignment is not expected to encounter this material.

6.14 Holocene Alluvium (Qha)

No laboratory test information is available for the Holocene alluvium and the alignment is not expected to encounter this material.

6.15 Coode Island Silt (Qhi)

A summary of the results of classification testing within the Coode Island Silt is presented in Plates 43 to 45.

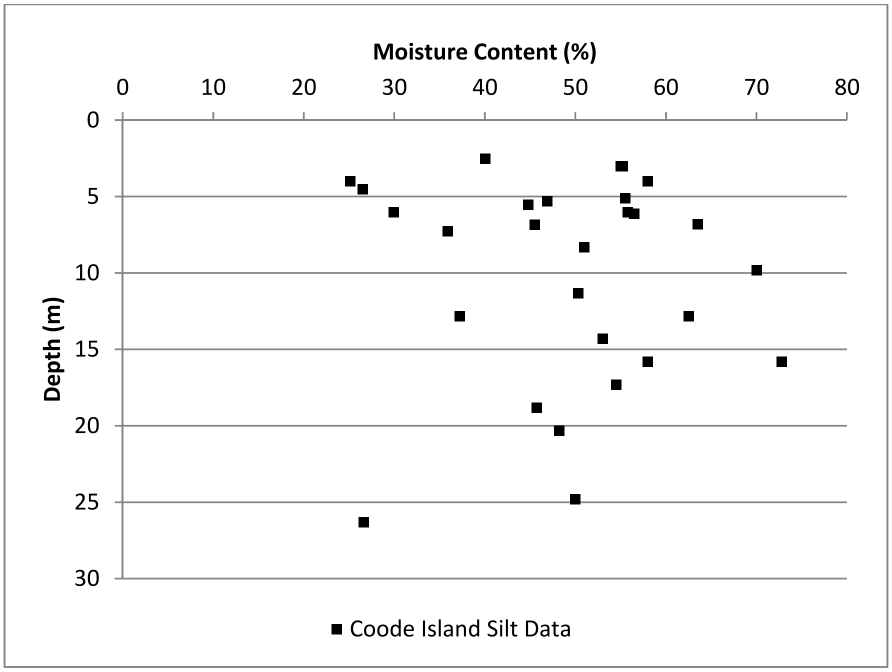


Plate 43: Moisture content versus depth in Coode Island Silt

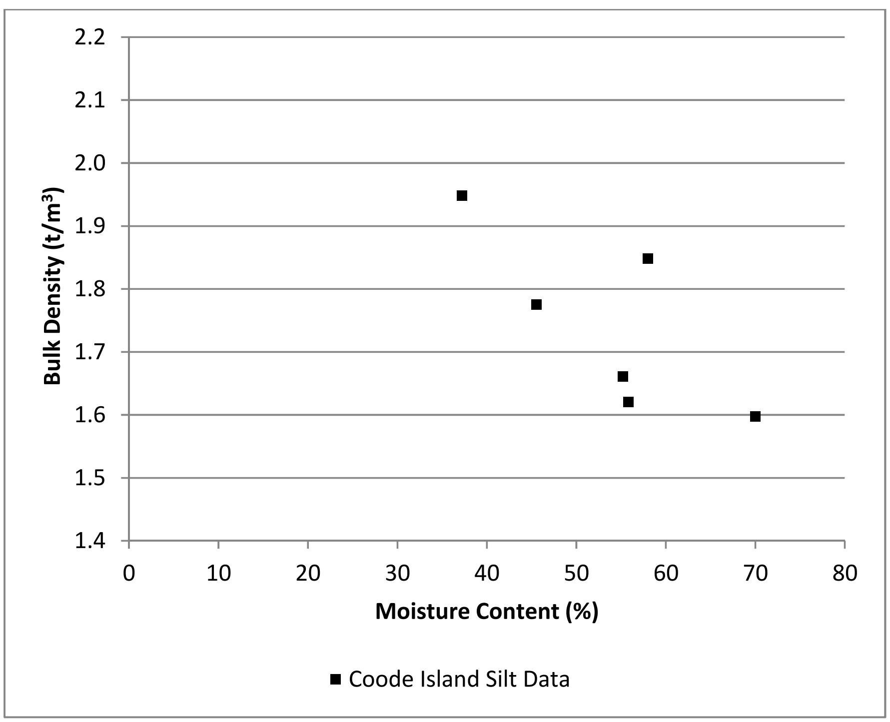


Plate 44: Bulk density versus moisture content in Coode Island Silt

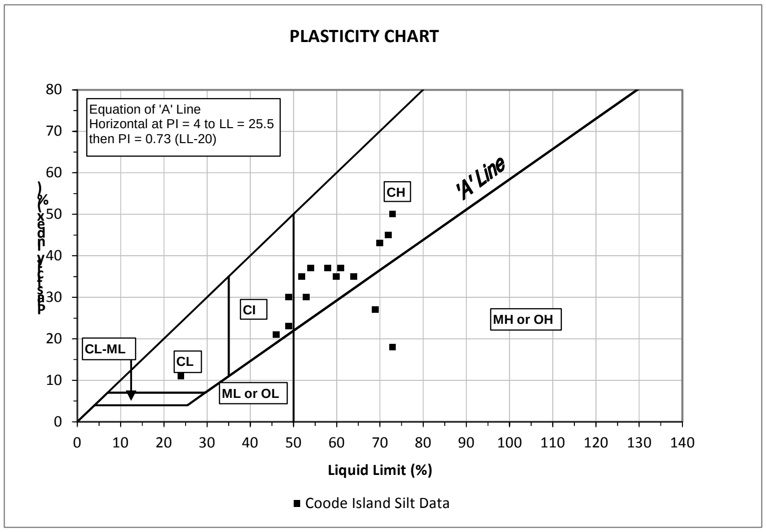


Plate 45: Plasticity of Coode Island Silt

The classification results indicate that the Coode Island Silt is predominantly high plasticity clay. The appreciable scatter within the moisture content measurements may reflect variability within the composition of the material. The geological history of this material indicates that it is slightly overconsolidated to normally overconsolidated and as such as roughly linear decrease in moisture content with depth would be expected in an homogenous clay.

Although no triaxial testing has been undertaken on the Coode Island Silt specifically for this project, data obtained from the desk study has been collated and is presented in Table 25.

Table 25: Results of Strength and Consolidation testing within the Coode Island Silt

Measured	c' (kPa)	ϕ' (Degrees)
Lower Bound	0(31)	16(31)
Upper Bound	20(31)	37(31)
Average	7(31)	29(31)

Numbers in parenthesis refer to the total number of tests results available

Published information suggests that the strength of the Coode Island Silt increases with depth below the ground surface, a characteristic typical of normally or slightly overconsolidated materials. An approximate correlation of $s_u = 10 + 1.5d$ (kPa), where d is the depth below ground surface in metres is often used to estimate undrained shear strength in the Coode Island Silt with depth.

The strength of the Coode Island Silt may be higher in some areas as a consequence of deposition close to the edge of the embayment or due to anthropogenic effects. These effects are possible:

- in Segment 6, due to the effects of the North Yarra Sewer Main; and
- within the Jolimont Valley (Segment 16) due to the effects of the Arts Centre basement and Burnley Tunnel. It is noted that a CPT within the Jolimont Valley (GA11-CPT004) indicates the Coode Island silt has a pre-consolidation pressure of about 17 kPa. Based on natural stress history, which excludes anthropogenic effects the preconsolidation pressure of the Coode Island Silt is estimated to be approximately 9 kPa (Paul et. al. 2014). The anthropogenic effects are estimated to have increased the shear strength within the Coode Island silt by 4 to 5 kPa.

Oedometer tests on Coode Island Silt collated during the desk study and presented in our desk study report indicate the typical compression characteristics set out in Table 26.

Table 26: Summary of consolidation characteristics of Coode Island Silt

Measured	Initial Void Ratio e_o	Coefficient of Consolidation C_v (Vertical) (m^2/year)	Compression Index C_c	Re-compression Ratio C_r	Coefficient of Secondary Consolidation C_α
Lower Bound	1.51	0.28(7)	0.5(54)	0.004*(65)	0.5(1)
Upper Bound	2.07	68(7)	2.6(54)	0.067*(65)	2(1)
Average	1.83	17(7)	0.9(54)	0.021*(65)	-

Numbers in parenthesis refer to number of test results available.

The engineering properties of the Coode Island Silt are expected to be variable at different locations within the embayment. Site specific investigation would be required to assess the properties of this material at locations of interest.

Dissipation testing undertaken using CPT during the Phase 2A investigation indicates a variable horizontal coefficient of consolidation, C_h, ranging between 0.56 and 1580 m^2/ year. This relatively extreme range is inferred to be a result of sand laminations within the Coode Island Silt. Field experience in the Coode Island Silt suggests that laboratory derived consolidation parameters very rarely provide a reliable prediction of field performance (Srithar 2010), with rates of field consolidation typically greater (in some cases more than 10 times greater) than those predicted by laboratory data. This is thought to be due to the deltaic depositional environment of the Coode Island Silt and the high frequency of sand lenses, in particular near the edge of the embayment near river valleys. We note that where Coode Island Silt is expected to be encountered by Melbourne Metro, it is near the edge of the embayment. Laboratory derived consolidation parameters should be assumed to be unreliable. A better estimate of consolidation rates may be made using moisture contents and published relationships such as that presented in Srithar 2010.

6.16 Fill

The fill material is highly variable and as a consequence, laboratory test results need to be taken from locations where it is expected to be encountered. Generally the fill is uncontrolled and in the absence of testing in this material we recommended relatively conservative parameters are assumed.

6.17 Recent Silt

No geotechnical testing has been undertaken on the recent sediment.

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Report Signature Page

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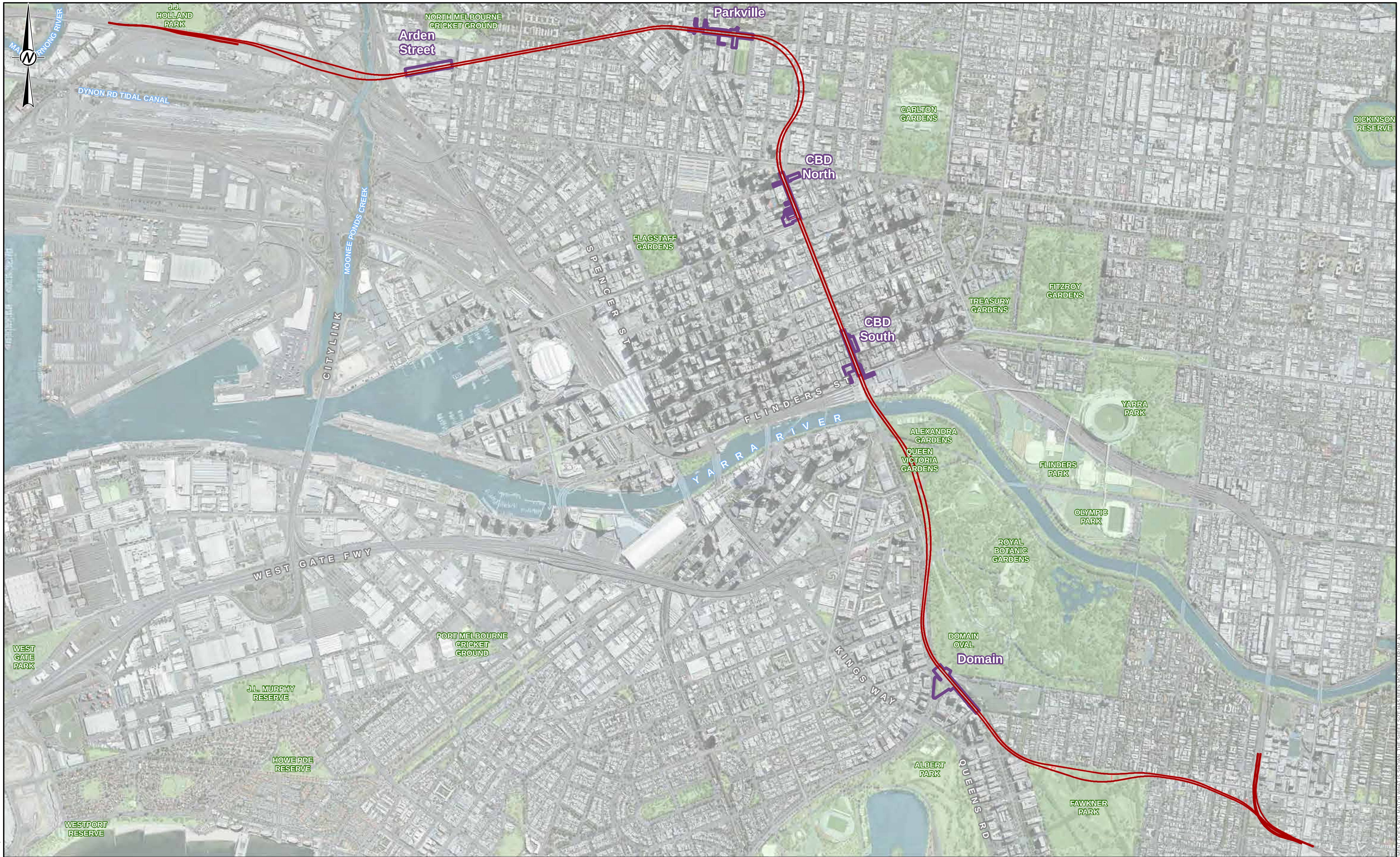

LEGEND

- Water
- Proposed Railway Track
- Proposed Rail Infrastructure
- Proposed Station Extent

MAP INFORMATION

The information and data contained in this map is for Melbourne Metro Rail Project, and is for information purposes only. It is to be used for reference only and may not be suitable for any other purpose including design. The information may not be accurate, current or otherwise reliable.

NOTES

1. Rail alignment sourced from AJM JV, revision P2.3 dated 28-10-2015.
2 Aerial imagery sourced from Public Transport Victoria, image resolution 10cm and date of capture February 2015.
3. Road and hydro information sourced from Victorian Government Data Directory 2015.

CLIENT
AJM JOINT VENTURE

PROJECT
MELBOURNE METRO RAIL PROJECT

0 200 400 600 800 1,000
METRES

REFERENCE SCALE: 1:20,000 (at A3)

PROJECTION: GDA 1994 MGA Zone 55

TITLE
RAIL ALIGNMENT

CONSULTANT
Golder Associates

YYYY-MM-DD	2016-03-24
PREPARED	JPH
DESIGN	-
REVIEW	DRP
APPROVED	DLG / SVLB

PROJECT No.	CONTROL	Rev.	DRAWING
1525532	061-R	0	1

IF THIS MEASUREMENT DOES NOT MATCH WHAT IS SHOWN, THE SHEET SIZE HAS BEEN MODIFIED FROM A3

25mm

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