North East Link Inquiry and Advisory Committee (IAC)

Expert Witness Statement of Stephen Macklin (Ground Movement)

1. Introduction
1.2 I have been instructed by Clayton Utz on behalf of North East Link Project (NELP) to review and respond to the public submissions (discussed in Section 7 below) and give evidence on the environmental effects of the Project relevant to my area of expertise.

2. Qualifications and experience
2.1 Annexure A contains a statement setting out my qualifications and experience, and the other matters raised by Planning Panels Victoria's Guide to Expert Evidence. A copy of my curriculum vitae is provided as Annexure B.

3. Technical Report
3.1 The role that I had in preparing the Technical Report was lead author. Other significant contributors to the Technical Report and their expertise is set out as follows:

(a) Steve Kerris – geotechnical engineer. Mr Kerris undertook calculations and assisted with field inspections, preliminary, second stage and detailed assessments, and numerical analysis for slope stability.

(b) Toby Shutler – graduate tunnelling engineer. Mr Shutler undertook calculations and assisted with field inspections, preliminary, second stage and detailed assessments.

Details of the qualifications and experience of the other contributors to this statement is included in Annexure B.

3.2 I adopt the Technical Report, in combination with this statement, as my written expert evidence for the purposes of the North East Link Inquiry and Advisory Committee's inquiry into the environmental effects of the Project.

4. Further works since preparation of the Technical Report
4.1 Since exhibition of the Technical Report in April 2019 (which in turn relied on data available relevant to the Reference Design of August 2018), ongoing borehole and other geotechnical investigations have been undertaken for the development of the project. These investigations have been incorporated into the project geological model.

4.2 Some changes to the model have arisen including greater depth of weathering north of Watsonia Station, refinement in the understanding of fault geometry beneath Borlase Reserve and, the greater depth of the alluvial valley and thickness of sediments at the Eastern Freeway interchange. Nonetheless, the model shows general consistency with the geology assumed for the EES and...
has not resulted in any changes to the conclusions presented in Technical Report – M.

5. **Response to IAC further information requests.**

Following a review of “critical issues” on Technical Report – M by the IAC (dated 18 June 2019) a number of requests for information were made. I have addressed each of these as discussed in Annexure C. The RFIs and responses by me have not resulted in any material changes to the findings of Technical Report – M.

6. **Summary (key issues, opinion, recommendations)**

6.1 In general my assessment of ground movement risks to existing structures, utilities, landscapes and parklands is that the Project presents low to very low residual risk, once the appropriate EPRs have been applied. Nonetheless, areas of particular sensitivity have been identified including Banksia Park north of the Bridge Street portal, the deep excavation north of Lower Plenty Road (Borlase Reserve) and environmentally sensitive landscapes adjacent to, and including, Banyule swamp. These areas will require application of the appropriate EPRs in order to ensure that ground movement risks are adequately managed and mitigated.

6.2 I have reviewed the public submissions to the EES and requests for information from the independent advisor to the IAC. My opinions and recommendations in response may be summarised as follows:

(a) **Ground movement risks to residential and heritage buildings, and utilities:** for all the submissions raising concern in this area, either they referred to structures lying outside the zone of influence (ZoI) or did not refer to specific structures. However as noted in the responses in Annexure C, the technical report shows that of those structures within the ZoI, none are expected to exceed the “Slight” damage risk category. Any potential ground movement risks to structures that are of particular importance or concern to the community can be managed and mitigated through the implementation of the appropriate EPRs.

(b) **Ground movement risks to vegetation, landscapes and parklands and playing fields:** the key concerns arose from perceived risks to mature trees due to root damage and integrity of the playing fields. By comparing ground strains applied to tree roots during natural seasonal wetting and drying, with strains typically considered for ground movement risk assessment, it can be shown that there is negligible risk to tree roots from ground movement.

For the playing fields, the estimated movements of less than 5 to 10 mm and 1:1000 ground slopes, would not normally give rise to concern. Whilst it is recognised that there is a potential for ground movement effects on the playing fields arising from changes in the assumed conditions (e.g. changes in geology), these risks can be managed and mitigated through the implementation of the requirements of EPRs GM1.
(c) IAC Requests for information: mainly of a technical nature, these RFIs focussed on clarifications on the Technical Report and additional explanation/modelling output with respect to: utilities less than 400 mm diameter; the Banyule Creek sewer; the outbuilding in Simpson Barracks; and, the Banyule Homestead escarpment/slope. In addition, further explanation around the analytical approach to the validation of “volume assumptions”, input parameters regarding stiffness of the ground and initial in situ stress conditions was requested. This information is provided in Annexure C of this statement.

7. Submissions

Submissions received

7.1 I have read the public submissions to the EES, Draft Planning Scheme Amendment and Works Approval Application (tunnel ventilation system) and identified those that are relevant to the Technical Report and my area of expertise. The submissions that refer to ground movement effects are: submission 62, 68, 214, 232, 272, 274, 456, 514, 539, 600, 601, 634, 666, 700, 717, 718, 728, 777, 849 and 870.

Summary of issues raised

7.2 The submissions have raised the following themes relevant to my area of expertise:

(a) Risk of damage to residential buildings; condition surveys (submissions 62, 68, 214, 232, 272, 456, 514, 663, 728, 777, 870).

(b) Risk of damage to heritage structures (submission 214).

(c) Risk of damage to utilities (submissions 272, 539, 849).

(d) Risk of damage to native vegetation (submissions 214, 274, 717).

(e) Risk of damage to the natural parkland and landscapes (submissions 214, 600, 634, 666, 700).

(f) Risk of damage to sports fields (submissions 601, 718).

Response to issues raised

7.3 Set out below are my comments and responses to the issues raised by the written submissions relevant to my area of expertise:

(a) Risk of damage to residential buildings: of the submissions listed in Section 7.2 (a) above, all but submissions 214 and 870 can be identified to lie outside the anticipated zone of influence (ZoI) of ground movements so are not expected to be at risk of damage from this “risk pathway” (see Section 7 of Technical Report-M).

The two properties referred to in submissions 456 and 777 lie on opposite sides of the same residential street adjacent to properties
earmarked for compulsory acquisition. Despite other potential construction phase effects on these properties, both lie outside the ZoI so that they will not be exposed to ground movement risks.

Submissions 214 and 870 do not specifically refer to the location of the residential property. Nonetheless, our assessments for the residential housing areas affected by tunnelling generally found that these structures were at a Slight risk of damage or less – environmental performance requirements (EPRs) GM1 to GM3 were also proposed (see Section 9 of Technical Report – M) to ensure that any ground movement risks were adequately managed during construction. Appendix B of Technical Report – M shows the locations of the residential buildings assessed.

(b) Risk of damage to heritage structures: Only submission 214 is relevant here, and refers to concerns around ground movement effects on the Banyule Homestead. Located at 60 Buckingham Drive, Heidelberg, this property is listed on the Heritage Council Victoria – Victorian Heritage Register.

Technical Report - M shows that the structure lies outside of the anticipated 5 mm settlement contour, calculated at the level of the Banyule flats (alluvial soils) at the toe of the east-facing escarpment forming the Yarra Valley here. Substantially less movement would thus be anticipated at the homestead because it is founded on the stronger Silurian siltstone rock at the top of the slope.

As noted in Technical Report - M, preliminary finite element analysis of the slope between the homestead and the tunnel alignment indicated that negligible ground movements would be felt by Banyule Homestead due to tunnelling, and slope instability was not anticipated.

(c) Risk of damage to utilities: Of the three relevant submissions on this theme, none refer to specific utilities. As discussed in Section 5.7.4 of Technical Report - M, a review of published empirical information suggested that only utilities of greater than 400 mm diameter would require specific consideration, because it is the larger diameter pipes that tend to suffer the highest tensile strains and joint “pull-out” forces.

Furthermore, Section 5.6 of the report describes the approach to “second stage” assessments, required when the preliminary assessment indicates that the utility may be subject to a slight category of damage risk (or greater). Where appropriate, the assessment of the condition of the utility may be warranted, although it should be recognised that there are practical difficulties in doing this with operating utilities at EES stage.

The approach thus adopted in the Technical Report (where a second stage assessment was warranted) was to assess the estimated level of strain or joint displacement in the utility, and compare this to a conservative estimate of allowable strain/movement (published values) for utilities in good condition. Where the imposed strain/displacement...
was well below that value, no further assessment was undertaken.

Nonetheless, in recognition that the actual condition of the utility can have a bearing on the outcome of second stage and detailed assessments (see submission 539), these risks can be managed and mitigated through the implementation of the requirements of the appropriate EPRs. For example EPR-GM3 (Condition surveys) will be appropriate for a number of relevant utilities as summarised in Section 9 of Technical Report - M, including the Yarra Valley Water asset – Bulleen Road West sewer.

(d) Risk of damage to native vegetation: Submissions 214, 274 and 717 are relevant here, with a common theme being concern for damage to native eucalypt tree roots due to tunnelling induced ground movements.

Whilst the assessment of the effects of ground movements on tree root systems is not a typical application of the methods employed in the ground movement risk assessment described in Technical Report - M, the analysis of horizontal ground strain provides a convenient means to assess the potential effects on tree roots. Published data on tree roots suggest that the direct tensile strain at the elastic limit varies between 2 and 5%, with rupture strains typically occurring between 10 and 20% (Cofie, 2000).

The analytical methods adopted for the assessment of damage risk to structures and utilities rely partly on an estimate of direct tensile strains in the ground. Tree roots typically spread within the upper 3 m of the ground, so we can compare the horizontal ground strains used to assess shallow depth foundations with the elastic strain limit for tree roots as a preliminary indication of risk of damage.

For surface buildings and their foundations, the damage classification of Burland et al (1995) suggests that horizontal ground strains of over 0.15% are required to present a risk of “Category 3 (Moderate)” damage or greater. If this is compared to an elastic strain limit of approx. 2% for tree roots noted above, it may be determined that the ground movements anticipated for the Project excavations are unlikely to result in damage to existing tree root systems.

An alternative check may be made if strains in the upper soil layers due to seasonal changes in water level are considered, accepting that trees are generally tolerant to summer to winter seasonal changes in the soil water content. For instance, vertical movements of up to 30 mm are not uncommon in moderately reactive clay or silt soils (AS2870-2011). If it is assumed that these vertical movements decrease linearly to a depth of approx. 2 m below ground level, an average vertical strain of 1.5% can be estimated. Again, the limiting tensile ground strains at the Slight to Moderate damage risk category boundary of 0.15% are 1/10th of this value so it would be reasonable to infer that typical excavation induced ground strains are well below tolerable limits for tree roots.
Risk of damage to natural parkland and landscapes: Submissions 214, 600, 634 and 700 are relevant here, with a common theme being concern for the Banyule Swamp and Bolin Bolin Billabong due to ground movement effects associated with dewatering of the trench/cut-and-cover excavations. Ground movements due to tunnelling vibrations were also raised (submission 700) and two submissions (600 and 634) requested that monitoring be undertaken to ensure that damage did not occur.

Hydrogeological modelling undertaken for the EES (see Technical Report N – Groundwater) shows that the extent of any groundwater drawdown associated with the construction of the Manningham interchange or the trench north of lower plenty road has a negligible effect on groundwater levels in the Yarra Valley around the Banyule wetlands. This is because natural recharge rates from the river are such that negligible drawdown in the alluvial sediments occurs, so negligible “consolidation settlement” is anticipated.

Appendix B2 of Technical Report - M shows the calculated contours of vertical settlement associated with the tunnels in proximity to the Banyule Swamp and wetlands. It is important to note that at this location, negligible “construction phase” groundwater drawdown effects occur because a pressurised Tunnel Boring Machine (TBM) is anticipated to be used at this location in order to satisfy groundwater EPR GW3 (see section 9 of Technical Report – N). The settlement contours reflect the short term movements associated with the TBM tunnelling process and have assumed a conservative “volume loss” of 0.8% of the tunnel face area when calculating ground movement.

The calculated ground movement was assessed for potential effects on the integrity of the embankment and outfall structure on the south west side of the approx. 7 Ha pond comprising the “Banyule Swamp”. Because there are no structural elements like building foundations or utility pipelines, a qualitative assessment was undertaken. A maximum settlement of 35 mm was compared to a notional limiting settlement of 100 mm. Nonetheless, due to the sensitivity of this feature, EPRs GM1 to GM4 are proposed, including ongoing monitoring of the feature (EPR GM2) as discussed in the next section.

Appendix B3 of Technical Report - M shows the calculated contours of vertical settlement associated with the cut-and-cover tunnels in proximity to the Bolin Bolin Billabong. Again, due to high levels of water recharge, the analysis reported in Technical Report N - Groundwater indicates that a drawdown of approximately 0.1 m is estimated at the eastern tip of the billabong, and no drawdown from the Manningham interchange excavation reaches the billabong from the north. The cumulative excavation and drawdown related settlement is thus estimated to be less than 5 mm.

Regarding the potential for vibration induced ground movements (submission 700), Technical Report - D – Vibration, provides an
estimate of peak particle velocities (PPV) during construction of less than 0.5 mm/sec at the eastern tip of the Bolin Bolin Billabong. A simple assessment of the potential for vibration to cause settlement may be undertaken using the concept of a “cyclic stress ratio” (CSR). The CSR provides a simple means to assess the potential for cyclic shear stresses and build-up of water pressure during shaking to cause settlement of the soil.

Soils susceptible to this kind of movement are typically fine grained loose “cohesionless” soils. Similar ground conditions are expected beneath the billabong. A CSR of 0.0025 (i.e. the induced shear stress is estimated to be 0.25% of the vertical effective stress) would be reasonable in these conditions. Published correlations indicate that for a CSR around 0.25%, the likelihood that settlement occurs due to vibration would be negligible, suggesting that the risk of adverse settlement effects on the billabong is very low. Similarly, such low level shear stresses would be unlikely to affect the natural groundwater flow regime around the billabong.

Submission 666 also raised the concern that tunnelling vibration could cause damage to the Banyule flats escarpment. As noted above, the tunnelling induced shear stresses are such that the effects on ground stability can be considered negligible. Similarly, slope stability assessment undertaken for the EES does not indicate any cause for concern (see response to item 5.3 (b) above).

**Risk of damage to sports playing fields:** Submissions 601 and 718 are relevant here. The Trinity Grammar School submission raises two key concerns regarding ground slopes causing drainage problems for the playing fields and “blowout” risks associated with the Bulleen sewer realignment works. The Marcellin college submission similarly considers that the calculated settlements are “unacceptable” and requests active engagement in the development of any ground movement management plan.

The ground conditions beneath the playing fields vary from soft to stiff clay soils and loose to medium dense sandy soils. Whilst there is theoretically a hazard associated with “blowout” and excessive ground movements in soft/loose soils, standard engineering design practice is to determine support pressures in the pipe-jack TBM that are within an appropriate range to ensure that volume loss is minimised and the risk of blowout avoided.

The settlement contours shown in Technical Report – M, Appendix B3, show that the cut and cover tunnel and sewer diversion works will have the most impact on the Trinity College fields, with maximum settlements of 5 mm to 10 mm beneath the tennis courts and water retention pond in the north west corner of the fields. It is anticipated that both of these features will require temporary relocation/removal and reinstatement after completion of the works.
The settlement effects on the Marcellin College fields are considerably less, almost entirely due to the sewer diversion works and represent settlements of less than 5 mm above the pipe alignment and up to 10 mm around the access shaft located beneath the northern boundary access road. Maximum ground slopes less than 1:1000 are conservatively estimated.

Whilst such levels of movement would not normally be expected to affect the use of the sporting fields the submissions make clear that there are valid concerns about the possible effects of the main works and sewer diversion works on the integrity and function of the fields. These risks can be managed and mitigated through the implementation of the requirements of EPRs GM1 to GM4.

8. **Environmental Performance Requirements**

8.1 I have further reviewed the environmental performance requirements relevant to my area of expertise, being GM1, GM2, GM3 and GM4 and, in the light of the public submissions, recommend the following changes:

(a) GM1 – Geotechnical model and assessment – no amendments proposed.

(b) GM2 – Ground Movement Plan – due to extensive areas of sensitive landscapes, water bodies and vegetation within the project area, the following amendment to EPR GM2 is proposed:

After the second bullet point insert:

*A baseline monitoring report is to be compiled summarising the results of the baseline surveys undertaken.*

(c) GM3 – Conditions Surveys (property and infrastructure) – no amendments proposed.

(d) GM4 – Properties & assets impacted by ground movement & settlement – the first paragraph is reproduced below in full with the amendments underlined:

*For properties and assets (including natural landscapes and parklands) affected by ground movement caused by the project, undertake required repair works or other actions as agreed with the property or asset owner (or land manager). For places listed on the Victorian Heritage Register, consultation with Heritage Victoria must be undertaken.*
9. Declaration

9.1 I have made all the inquiries that I believe are desirable and appropriate and no matters of significance which I regard as relevant have to my knowledge been withheld from the North East Link Inquiry and Advisory Committee.

Signed

Date: 15 July 2019
Annexure A – Matters Raised by Planning Panels Victoria (PPV) Guide to Expert Evidence

a) The name and address of the expert:

Stephen Macklin
Senior Technical Director
GHD Pty Ltd
180 Lonsdale Street
Melbourne, Victoria 3000

b) The expert's qualifications, experience and area of expertise:

Bachelor of Science - Applied Geology; NSWIT, Sydney, 1986
Master of Science, Diploma Imperial College - Engineering Geology; Imperial College, London, 1992
Chartered Geologist (UK - No. 1002136)
Fellow of the Geological Society, London
Chartered Engineer (UK Engineering Council – No. 518747)
Member of the Institute of Materials, Minerals and Mining (IoM³).
Member of the Australasian Tunnelling Society (No. 4179562).
Member of the Australian Geomechanics Society

I am a chartered geologist and chartered engineer with over 30 years’ experience in the design and specification for tunnels, caverns, ground investigations, ground movement risk assessment, ground hazards assessment and feasibility studies. I have written expert reports on geotechnical and tunnelling issues for clients in the insurance, infrastructure, retail, telecommunications and government sectors. I have acted as an expert witness under cross examination at an arbitration hearing for a major power station development in the nuclear sector.

c) Details of other significant contributors to this statement:

Steve Kerris – geotechnical engineer. MEngSc, BEng, MIEAust

Mr Kerris is a geotechnical engineer with 8-years post-graduate experience in design and technical support roles for mining and tunnelling projects. He has considerable experience in hazard and risk management, rock mechanics mapping, slope stability analysis and support design, design, specification and

Mr Kerris undertook calculations and assisted with field inspections, preliminary,
second stage and detailed assessments, and numerical analysis for slope stability in preparation for this witness statement.

**Toby Shutler – graduate tunnelling engineer. MEng (Civil with Business)**

Mr Shutler is a civil engineer with 2-years post-graduate experience in design and technical support roles for tunnelling projects. He has worked on tunnelling and infrastructure projects in the transport, power and water sectors in Australia and overseas. Key experience includes ground hazard assessment and analysis of a ground movement damage risk for infrastructure projects and excavation support design for deep mined tunnels for a pumped storage hydropower scheme.

Mr Shutler undertook calculations and assisted with field inspections, preliminary, second stage and detailed assessments in preparation for this witness statement.

d) All instructions that define the scope of this statement (original and supplementary and whether in writing or verbal):

I have written this witness statement in response to the instructions provided to me in a letter from Sallyanne Everett and William Bartley of Clayton Utz, dated 28 May 2019. Further instructions were provided by Sallyanne Everett (email, dated 9 July 2019) regarding inclusion of a summary of key issues, opinions and recommendations (pertaining to direction 8b from the PPV letter dated 26 June 2019).

e) Details and qualifications of any person who carried out any tests or experiments upon which the expert relied in preparing this statement

The ground model used by me to assess appropriate ground movement parameters, was developed by the Melbourne-based GHD geotechnical team, which in turn relies on the geotechnical investigations and laboratory testing undertaken for the Reference Design. Whilst the various components of the investigation were led by senior engineering geologists and geotechnical engineers from GHD, my role as Senior Technical Director has included technical guidance, review and oversight of these activities.

f) Any questions falling outside the expert's expertise:

All submissions and RFIs pertinent to ground movement fall within my area of expertise with the exception of the response of tree roots to ground movement. However by analogy with ground strains cause by seasonal shrink swell effects and strain levels considered when assessing the risk to shallow foundations, I have been able to show that it is highly unlikely that ground movements will give rise to the magnitude of strains that could damage the root systems of trees close to the Project alignment.

g) Key assumptions made in preparing the Technical Report:

The key assumptions I have taken when undertaking the ground movement risk assessments are discussed in Sections 5.6 and 5.7, and Appendices D.1 and D.2 in Technical Report – M.
h) Any departures from the findings or opinions expressed in the Technical Report and, if so, why:

There are no departures from the findings or opinion expressed in Technical Report – M apart from recommendations for amendments to the EPRs in response to the submissions reviewed for this witness statement.

i) Whether the Technical Report is incomplete or inaccurate in any respect:

The Technical Report is considered to be complete and accurate within the limitations implied by the assumptions described in Sections 5.6 and 5.7, and Appendices D.1 and D.2 in Technical Report – M.

Nonetheless, the following corrigenda are noted:

- Technical Report – M, section 6.3.2: text should be amended as follows:

  “A section of the 450-millimetre diameter sewer runs transverse to the TBM tunnels alignment near Buckingham Drive, with a clearance ground cover of 0.5 to 1.5 metres”.

- Technical Report – M, Appendix D.1, section 2.3.1: text should be amended as follows:

  “.....the following key parameters were adopted for the RS2 model: GSI = 52 (representative of a fractured, “very blocky” rock mass); \( \sigma; = 5 \text{ MPa} \ 10 \text{ MPa} \) (representative of moderately weathered rock); Modulus ratio = 250....”

j) Details of any changed circumstances or assumptions since the Technical Report was prepared and whether these affect the opinions expressed in the Technical Report:

As described in Section 4 of this witness statement and Annexure C.
Annexure B – Curriculum Vitae

Steve Macklin

Profession: Engineering Geologist

Qualifications: BSc - Applied Geology; NSWIT, Sydney, 1986
MSc, DIC - Engineering Geology; Imperial College, London, 1992

Position in GHD: Senior Technical Director
Tunnels & Engineering Geology

Steve is currently employed by GHD Pty Ltd as a Senior Technical Director – Tunnelling and Engineering Geology, based in Melbourne. He is a geologist and engineer with over 25 years’ experience in the design and specification for tunnels, caverns, ground investigations and feasibility studies. He has been responsible for projects involving open cast and underground hard rock mining, rock and soft ground tunnels and caverns, rock and soil slopes, dams and pumped storage schemes, rock and soil foundations and ground-structure interaction analysis. He also has experience in design, specification and contract documentation for geotechnical instrumentation on large tunnelling projects.

He has written expert reports on geotechnical and tunnelling issues for major clients in the insurance, infrastructure, retail, telecommunications and State Government sectors. He has acted as an expert witness under cross examination at an arbitration hearing for a major power station development in the nuclear sector. He has lectured MSc students at the University of Melbourne (Honorary Fellow) and has presented to the Hong Kong Tunnelling Society (IoM3), British Tunnelling Society, the Australasian Tunnelling Society, the Australian Geomechanics Society, the Canadian Tunnelling Association and the Canadian Geotechnical Society.

Steve has worked on projects in Australia, New Zealand, Hong Kong, UK, Europe, USA, Mexico, Africa, the Middle East and India.

Selected projects
GHD, Melbourne, Australia (September 2011 to present)

North East Link Authority Technical Advisor

Identified in Infrastructure Australia’s “Infrastructure Priority List”, the North East Link Project, valued at approx. $16 bn, will be the biggest single transport project in Victoria’s history – connecting the M80 Ring Road at Greensborough with the Eastern Freeway at Bulleen. It will require 6 km of 15.6 m diameter TBM, mined SEM and cut-and-cover tunnel, embankments, viaducts and upgrade of the existing Eastern Freeway to over 16 lanes in part. The freeway upgrade will also include the Doncaster Busway project.

Key Experience Areas

- Technical Advice and Panel review for road and rail tunnel owner/operators
- Expert advice to insurers and ICC arbitration proceedings
- Forensic analysis of tunnel, foundation and slope failures
- Building, masonry tunnel and utility risk assessment for tunnel settlement
- Design, specification and analysis of geotechnical instrumentation
- Ground model, geo-hazard assessment and geotechnical parameters for soil and rock tunnel and cavern design
- Engineering geologist with wide ranging international experience
As Technical Adviser to the North East Link Authority (NELA), GHDs commission commenced with an initial corridor options assessment, including geotechnical investigations covering an area of 500 km²; GHD also provided advice and input to a comprehensive options assessment study. Upon selection of the preferred “western” alignment option, GHD provided initial design advice in order to cost the Business Case for the project and has subsequently developed the Reference Design. As part of this work, GHD is conducting one of the largest single geotechnical and hydro-géological investigations ever undertaken in Australia comprising nearly 300 boreholes with a comprehensive suite of geophysics, in situ and laboratory testing and complex 3D geological modelling.

With our commission commencing in March 2017, project Tender processes will start in late 2019. Steve is currently acting in a technical guidance and leadership role on tunnelling and geotechnical aspects of the project and is providing specialist advice and technical reporting to The State on: Ground Movement effects for the Environmental Effects Statement (State legislation); the Public Environment Report (Commonwealth legislation) and the Design and Development overlay (local planning requirements).

Southern Connector Improvements (SCI), Auckland

Steve provided independent review and analysis of a number of technical issues forming the basis of a claim for unforeseen conditions in a 51” Akkerman TBM pipe-jack drive, undertaken as part of the larger SCI project. The contract was let as a design & construct contract according to NZTS3916-2013 (May/June 2019). GHD were the Engineer to Contract, the principal NZTA.

Hydrostor CAES project Due Diligence, ARENA

GHD was commissioned by the Australian Renewable Energy Agency (ARENA) to undertake a technical assessment of the Funding Application for a 5 MW Advanced Compressed-Air Energy Storage (A-CAES) Project near the town of Strathalbyn in South Australia. The proposal utilises an abandoned zinc mine in Cambrian andalusite-mica schists of the Tapanappa Formation. Steve provided tunnelling and geotechnical input and advice on the Hydrostor proposals (March 2018 to May 2019).

Hobsons Bay Main Sewer Duplication, Port Melbourne

Steve provided support on tunnelling geotechnical aspects of the proposed sewer replacement (March to November 2018) for Melbourne Water. This included desk top review of the previous geotechnical investigations, consideration of alternative alignment options, review of constructability reports and assessment of potential ground movement effects. A preliminary 3D geological ground model was also constructed and additional investigations proposed, depending on the chosen alignment.

Western Distributor Road tunnels, Yarraville, Melbourne

In March 2015 the Victorian Government received a market-led proposal for the Western Distributor from Transurban (TU), the CityLink concessionaire, to construct a new toll road including twin 15.6 m diameter tunnels under Yarraville, connecting the West Gate Freeway with the Port of Melbourne, CityLink and the CBD (estimated value of $5.5 billion). Construction of the Western Distributor started in 2018 and is expected to be open to traffic in 2022. In order to ensure that the interests of the state of Victoria were served by the Transurban proposal, Steve provided technical advice and support to The Department of Economic Development, Jobs, Transport and Resources.
of Victoria (DEDJTR) on tunnelling technical matters, including review of the Environmental Effects Statements (EES), Draft Development Overlay (DDO) and Technical Review of the Tender returns.

**Westconnex Stage 2 Tender Design, Sydney NSW**

Steve was the design team lead for the tender design of shafts caverns and tunnels for the $5bn Westconnex M5 Main Works and Southern Connector tunnel works (December 2014 to March 2015). Client was the Construction Joint Venture comprising McDow-Ghella-Ferrovial and the designers were a joint venture of GHD and Mott Macdonald. The design comprised 14 m diameter twin bore TBM tunnels, three mined caverns up to 29 m span, twin bore 18 m span road-header tunnels and two ventilation shaft-adit connections. The ground conditions predominantly comprised the Triassic Hawkesbury Sandstone with part of the 12 m span ramp tunnels and portals within residual soil/Ashfield shale.

**East-West Link - East, Melbourne, Australia**

The East – West Link road tunnels (3.3 km long twin bore 15 m diameter) were proposed to provide a connection between the Eastern Freeway and the CityLink toll road, passing beneath the Upfield Line Railway cutting, Royal Park, the historic Melbourne Cemetery and Alexander Parade, rising to an eastern portal at Hoddle Street. The overall project was valued at $15 billion. Steve provided technical support (June 2013 – March 2014) to the Linking Melbourne Authority (LMA) on tunnelling induced settlement, groundwater inflows, rock mass classifications for tunnelling, potential hydrogeological impacts of tunnelling and advice on geotechnical investigation methods in the deformed and weathered Silurian “mudstone”. In addition, Steve provided technical tender assessment services in the lead up to contract award (May 2014).

**North West Rail Link (NWRL) Metro, Sydney: Tunnels and Stations Civil Works (TSC) Contract tender design.**

Technical Lead for the tender design (October 2012 to February 2013) of the Stations and shafts excavation support in Hawkesbury Sandstone, Mittagong Formation laminites, Ashfield Shale (laminites), superficial residual soil, colluvial soil and man-made soils. The tenderer comprised a Baulderstone-Bouygues joint venture. The project comprises a 15.5 km long twin bore 7 m diameter metro (undrained), with five surface excavation station boxes and two intermediate shafts. The surface excavations are drained and the support was required to be designed for a 10-year design life.

Detailed hydro-geological analysis of drawdown/inflow and assessment of landslide hazards were two of the particular technical challenges addressed. The underground section (TSC contract) extends from Epping to Bella Vista Station. The overground section (separate contract) extends from Bella Vista to the Tallawong stabling yards. The total project value (including fit-out and rolling stock etc...) is estimated to be around AUD$8 bn.

**Cable Tunnel, Sydney, Australia**

Steve co-authored an expert report in response to a summons filed in the Supreme Court of NSW (June to September 2012). GHD received instructions from to Lee & Lyons Lawyers acting on behalf of one of the co-defendants and their insurers. The matters involved alleged defects in the design of a partly unlined 3.5 km long cable tunnel in Ashfield Shale, Mittagong Formation and Hawkesbury Sandstone, constructed using both road-header and rock tunnel boring machine. The matters centred on permissible groundwater inflows, grouting design, drainage, rock support and the
accumulation of iron “sludge” (ferric oxy-hydroxide precipitate) deposits in the tunnel. A site inspection and detailed analysis of design drawings, specifications and construction records was required in composing the Expert Report.

**Legacy Way road tunnels, Brisbane, Queensland (2011 to 2012)**

GHD are the tunnel design consultants for a joint venture between a local Australian constructor (BMD) and international tunnelling contractors (Acciona and Ghella). Valued at AUD$1.5 bn, the road tunnels will connect the Western Freeway at Toowong to the Inner City Bypass (ICB) at Kelvin Grove.

Steve acted as an internal technical reviewer (November 2011 to February 2012) for the rock support design of the 4.3 km twin TBM 12.4 m diameter road tunnels through the Palaeozoic “Brisbane metamorphics”. Detailed review of the geotechnical characterisation of the rock mass and development of a rationalised design approach for the cross passages connecting the two tunnels.

**Christchurch Western Interceptor Sewer, New Zealand**

Provision of geotechnical and tunnelling support on the construction of a 1.6 m OD slurry TBM driven pipe-jacked sewer tunnel in the Recent alluvial soils. The tunnel construction was complicated by the presence of significant amounts of partly decomposed timber, variable (fine) grain size and variable (low) shear strength conditions. In addition, adverse effects including liquefaction induced movements were encountered as a result of the Lyttleton earthquake (September 2010) and the Christchurch earthquakes (February and December 2011). Recommendations for risk mitigation measures for future tunnelling works beneath key infrastructure were made.

**Perth City Rail Link Project Alliance, Public Transport Authority, Western Australia**

Internal “Challenge team” reviewer (September 2011 to June 2012) for the geotechnical design of the lowered rail tracks, bus station and new underground connections to the historic Wellington Road Railway Station. The work comprised: lowering of the existing Fremantle rail lines into cut and cover tunnels; construction of a new pedestrian underpass tunnel to link the Perth Underground and Perth Central Station; construction of a new Platform 10 and rail line; enabling works for future above-ground development, including the installation of deep foundations; and, construction of a new bus ramp for a future underground Bus Station. The ground conditions comprise a complex sequence of alluvial, marine and wind-blown sands, silts and over-consolidated clays over the Triassic Kings Park Formation. Potential hazards included ground movements potentially affecting the historic “Horseshoe Bridge” and Perth Station building and a close-proximity over-crossing of the new cut and cover Fremantle Line tunnels over the existing New MetroRail (NMR) bored tunnels.

The estimated design and construction cost of the project was AUD$609 M. The project was delivered in two phases: the new Rail Link completed 2014 and the future Bus Link in 2016.
Arup, London, UK (1999 to September 2011)

Howard Humphreys/Kellogg-Brown & Root, Surrey, UK (1996 to 1999)


MSc Studies - Imperial College of Science, Technology and Medicine, University of London, UK (1991 to 1992)

Gravelotte Emerald Mine, South Africa (1990)

Western Mining Corporation, Kambalda WA, Australia (1986 to 1989)

Affiliations
- Chartered Geologist (UK - No. 1002136)
- Fellow of the Geological Society, London
- Chartered Engineer (UK Engineering Council – No. 518747)
- Member of the Institute of Materials, Minerals and Mining (IoM³).
- Member of the Australasian Tunnelling Society (No. 4179562).

Presentations

**Introduction to Tunnelling**, 9/11 October 2017 and 12/17 October 2018. Series of four 1-hour lectures to geotechnical engineering MSc Students at the University of Melbourne. Topics covered included: Introduction to tunnelling; geo-hazards and risk for tunnelling; design considerations; and, third party impacts of tunnelling. Worked examples and exam questions were also provided.

**Predicting Change in Latrobe Valley Mining - Modelling Regional Scale Responses**, 3 May, 2016. Presentation to Government, Regulators, Mine operators and Researchers on new geotechnical approaches being developed to model large scale deformations around the operating coal mines of the Latrobe Valley. Traralgon.


The Legacy Way Road Tunnels Project, 27 March, 2013, IoM³ meeting, Hong Kong.

A 21st Century Perspective on the tombs of the Valley of the Kings, Egypt. After Dinner Speaker: AGS Annual Dinner, University of Melbourne, 12 December 2012 -


Assessment of listed buildings affected by excavation-induced ground movements: King’s Cross Station Redevelopment, 24 September 2004. BRINTEX Conference presentation, Docklands, London UK.


Publications


Tunnel Lining Design Guide BTS/ICE – Chapter 8: Instrumentation and Monitoring, 2004


Macklin SR (1999). ‘The prediction of volume loss due to tunnelling in overconsolidated clay based on heading geometry and stability number’, Ground Engineering, April


Annexure C – Responses to IAC request for information

This annexure comprises responses to critical issues and requests for information (RFI) on Technical Report – M Ground Movement. These were set out in the Inquiry and Advisory Committees (IAC) document “Preliminary Matters and Further Information Request”, published 20 June 2019 - Appendix C of that document. In addition, a meeting with the IAC representative on 11 June 2019 provided further clarity on the information requested.

Nine specific RFIs are raised in the IAC document as follows:

a) Provide some more definition around the concept of ‘good workmanship’ in the context of this tunnel build.

b) Have any ‘small utilities’ thus far been identified by NELP that warrant a particular assessment?

c) Provide the output evidence of the FEA related to the Banyule Homestead slope stability appraisal.

d) Confirm the value for ‘$\text{S}_{\text{ymax}}$’ for risk screening as part of the Stage 1 Risk Assessment. Is it 5 mm as indicated on Page 22, or 10 mm as indicated on Figure 5-3, Page 26.

e) Provide some further detail on how the structural appraisal of the Banyule Creek Sewer was assessed.

f) In regard to the Helmet sculpture (Banksia Park) owned by Manningham Council, the early part of the EES Chapter suggests this sculpture will be relocated, yet the Detailed Assessment at the end of the Chapter provides EPR recommendations GM1/GM2/GM3/GM4 – please confirm the agreed plans for this.

g) The ‘Out Building’ at the Simpson Barracks appear to have a mis-match in its assigned risk ranking, when the conclusions of the Chapter are read (indicated as beyond a slight risk).

h) Provide the actual Geotechnical Investigation Reports for the Project.

i) Provide the forming calculations that were the basis for Table 1 (Appendix D.1).

In addition, the following “critical issue” raised in the document required a response:

j) The EES information seems lacking around the investigation and understanding of regional rock stress fields for the project.

I set out my responses to these issues in the same order as follows.
Good workmanship

It is stated in Technical Report – M, that “good workmanship” can be defined as a standard of work that can reasonably be expected of a competent contractor. Whilst this definition does not provide specific measures against which it is possible to assess good workmanship, it does imply that it is the standard of work to be expected by a competent contractor. That is, a quality of work that would meet the specified performance requirements and satisfy the expectations of the ‘owner’, ‘client’ or project proponent.

Similar terms are used in Australian Standard AS 4902-2000 (General conditions of contract for design and construct) which states (under clause 29.1 – Quality of material and work):

“Unless otherwise provided, the Contractor shall use suitable new materials and proper and tradesmanlike workmanship”.

As an example of how this concept would apply to tunnelling, workmanship can affect volume loss \( (V_L) \), Burland (1995):

“The magnitude of \( V_L \) is critically dependent on the type of ground, the ground water conditions, tunnelling method, the length of time in providing positive support and the quality of supervision and control” (my underlining).

Similarly, Attewell (1995) quotes:

“...Mitchell has also suggested that for ...... poor construction control (for example, excavating ahead of the tunnelling shield, and poor jacking techniques at the shield) the ground loss estimate should be increased by a factor of about 3” (my underlining).

In terms of the Project, an example of good workmanship might include maintaining appropriate alignment control and positive face support pressures in the tunnel boring machine (TBM) drives, such that minimum volume losses (hence surface settlement) occurs.

Similarly, good workmanship might include installation of the segmental lining such that all faces of the segments are well matched, the waterproofing gaskets fully compressed and the annular gap outside the lining fully grouted. This would ensure that the required performance specification for water infiltration is achieved and the potential for long term effective stress settlement associated with groundwater drawdown minimised.

Small utilities

Due to the scale of the Project, and the unknown condition of the utilities, the approach was taken to determine an appropriate “cut-off” in terms of utility diameter, such that only utilities of a certain size would be considered. Conservative assumptions were then assumed for the remaining utilities when assessing the potential risk of damage.

In terms of the “cut-off” diameter to define “small utilities” for the Technical Report, guidance was sought from published studies, in particular O’Rourke & Trautmann (1982), who presented a number of case studies of cast-iron pipelines showing that relatively rigid, brittle pipelines of up to 500 mm diameter could potentially tolerate ground slopes of up to 1:140 –
which would equate to a “moderate risk” of “possible” damage to relatively rigid pipelines after Rankin (1988).

The actual risk of damage to a continuous (i.e. no joints) pipeline for a given radius of curvature will depend on the limiting strain of the pipe material. The assumption of continuity represents a more critical case than for rotation and pull-out of any joints in the pipeline.

Bracegirdle et al (1996) propose that an allowable increase in pipe strain of 100 µε (microstrain) should be considered for traditional “brittle” grey cast iron pipes (such as considered by O’Rourke & Trautmann) and 500 µε for more modern ductile iron pipes. Bracegirdle also quotes previous work that suggested that higher strain limits of 200 µε may be applied to grey iron pipes greater than 300 mm in diameter.

The plot below (considering bending strains only, transverse to the tunnel alignment) shows the relationship between radius of curvature, pipe diameter and limiting strain. As can be seen, for a worst case curvature scenario (tunnel depth 18 m and 0.8% volume loss), only for the very conservative 100 µε curve is there a risk (indicated by the area above the 100 µε curve and left of the 400 mm line) that damage to small diameter utilities could occur.

Based on this analysis, no specific small utilities (less than 400 mm diameter) have been identified or assessed for risk of ground movements. However as is normal for infrastructure projects, the constructor will be obliged to undertake site-specific searches and investigations for all utilities, and, where required, either divert or check the potential for damage due to his construction activities.
Banyule homestead slope stability (FEA)

Geological face mapping on the rock outcropping at the base of the slope suggested that the slope angle is controlled by two joint sets, one dipping into the slope (65°/282° GN) and one dipping out of the slope face (26°/152° GN). For typical joint shear strength and persistence properties, a high factor of safety against failure of the slope was ascertained.

Differential strains acting on the slope due to TBM tunneling ground movements were assessed using a simple 2D finite element model. This provided a conservative assessment of shear strains imposed on the slope due to the tunnel. The excerpt from the model output below shows an estimated 2E-04 (0.02%) shear strain acting on the slope. When shear strain levels of this magnitude are applied to the low angle joint surfaces, the implied displacements are an order of magnitude below those which would be required to mobilise the peak shear strength of the joints.

For this reason the ground movement effects of the TBM tunnels is not expected to have an adverse effect on the stability of the escarpment slope.

Preliminary assessment - settlement criteria

The 5 mm values refers to the proposed limit of consideration of the effects of ground movement, or “zone of influence (ZoI)”, as shown on the settlement contour plans in Appendix B of Technical Report – M. This value represents half the lower limiting settlement value of 10 mm (and an associated ground slope of 1:500) that Rankin (1988) proposes as the limit for “Slight” risk – “possible superficial damage”.

However the 10 mm values quoted in Figure 5-3 represent the criterion that, if the structure assessed exceeds 10 mm settlement and/or 1:500 ground slope, it should then be subject to a Second stage assessment.
Banyule Creek Sewer

As noted in Section 6.3.2 of the Technical Report, limited data obtained from Yarra Valley Water indicates that the sewer was constructed in 1963 and is a vitreous clay bell-and-spigot type pipeline. At the assessment location the internal diameter is 450 mm. The wall thickness is assumed to be 30 mm and pipe segment lengths 600 mm. On account of the segmented construction of this pipeline, the critical behavioural modes assessed were joint rotation and pull-out.

The ground cover to the pipeline varies between 0.5 to 1.5 m, the depth of the tunnel axes are approximately 30 m and the tunnels pass under the pipeline at an angle of approx. 35-degrees.

The preliminary (first stage) assessment indicated a maximum settlement of 36 mm and ground slope of 1:1000, suggesting that the pipeline may be at a “slight” risk of damage (after Rankin, 1988). A Second stage assessment was thus considered necessary. The approach to the analysis was to assume a conservative allowable joint rotation of 0.5-degrees and pull-out of 7.5 mm based on Bracegirdle et al (1996) to account for the unknown condition of the pipeline.

The joint rotation check was made based on the tightest radius of curvature fitted to the settlement curve. This occurs in the sagging zone as shown in Appendix C6 to the Technical Report. In order to achieve a limiting joint rotation of 0.5-degrees, this radius would need to be approx. 70 m. However the settlement curve indicates a much larger radius of 18 km so excess rotation is not anticipated.

Joint pull-out occurs due to direct extension (maximum horizontal ground strains multiplied by 2x pipe lengths) plus a rotation component (determined from 2x diameter x Sin(rotation angle)). The maximum pull-out occurs associated with the hogging zone directly above the northbound tunnel (Appendix C6) where the ground strains are in tension (unlike compressional strains in the sagging zone). The estimated 0.25 mm pull-out lies well below the limiting value of 7.5 mm chosen for this assessment.

Nonetheless, because this is a key piece of infrastructure, EPRs GM1 and GM2 were proposed.

Helmet sculpture, Banksia Park

As noted for a number of utilities and other structures relocation is one of a number of mitigation measures available to the constructor in the event that an unacceptable ground movement risk is present.

For the Helmet structure, the Detailed Assessment indicates that an unacceptable risk of damage is present where the TBMs pass beneath it close to the Banksia Street temporary portals. Furthermore, the Reference Design alternative comprising two reception shafts constructed just north of Bridge Street means that moving the Helmet sculpture would be unavoidable.

The EPRs quoted in Section 8.4.1 reflect the “base case” Reference Design assessed without reception shafts. The EPRs are also broadly applicable so do not specifically discuss relocation, although EPR GM4 (see Section 9 in Technical Report – M) does state that (underlining added here):
“For properties and assets affected by ground movement caused by the project, undertake required repair works or other actions as agreed with the property or asset owner”.

The EPR is structured this way because it is not known for certain how the successful constructor will approach the management of risk to this sculpture. For instance it may choose to undertake ground treatment or underpinning works and avoid relocation; or, an alternative tunnel alignment may be chosen to avoid the sculpture altogether.

As such, no discussions have been held with Manningham Council regarding possible future mitigation works due to the uncertainty at this stage. As required in the EPR GM4, it will be the responsibility of the constructor to consult with Manningham Council to agree an appropriate mitigation strategy, specific to the preferred alignment and construction method.

Simpson Barracks outbuilding

As discussed in Section 8.3.3 the Second Stage assessment for this structure has identified that the critical component is a 750 mm high reinforced concrete wall. However the foundation depth, thickness and relative stiffness of the wall are unknown, so a simple assessment based on horizontal ground strain and deflection ratio (Burland et al, 1995) indicates that the damage risk category may be as high as “moderate”.

However this assessment was further refined by taking into account the axial stiffness of the wall after Franzius et al (2006) which suggested that the risk rating could be reduced to the “Slight” category, subject to confirmation of the assumed foundation details.

The risk categories referred to are summarised in 5-3 in Technical Report – M after Burland et al (1995). An equivalent table is available in Australian Standard AS 2870-2011 (Residential slabs and footings), Appendix C - Table C1, although this provides only the descriptive component of the classification (without the limiting tensile strain values or ‘degree of severity’ descriptors). As such the Burland table has been preferred for all building assessments.

Because access to this site and “as-built’ details are not readily available, EPRs GM1, GM2 and GM 3 were proposed for this structure.

Geotechnical investigation report

I have been instructed that the advisor to the IAC no longer requires this information to be provided.

Basis for tunnelling volume loss assumptions (Appendix D.1)

Initial estimates of volume loss were made with reference to published case histories, precedent practice (East-West Link and Westgate Tunnel projects). However due to differences in the geological conditions, specific analysis of a range of anticipated ground conditions were undertaken. The flow chart below summarises the different cases considered.

As described in Appendix D.1, a 2D finite element software package was used in conjunction with the “confinement-convergence-method” (CCM) as described in Hoek et al (2008). The model of the ground assumed a simplified homogeneous “equivalent continuum” with rock mass strength and stiffness properties appropriate to the Geological Strength Index value assumed (25 for faulted ground and 52 for fractured rock) and a horizontal to vertical stress ratio of 0.5 (see discussed in response to item (j) below).
These models provided estimates of radial convergence that could occur prior to completion of the permanent lining. The volume loss was then estimated based on the average radial convergence divided by the cross sectional area of the tunnel and rounded up to the values provided in Table 1 of Appendix D.1.

In addition, the trough width parameter was determined from a plot of the natural log of normalised settlement versus normalised transverse distance (after Mair et al., 1993). As is common with linear elastic models, the near-field curves (i.e. within 1 to 1.5*trough width parameter) provided a good linear fit; however they tend to be less reliable in the far-field. As such the trough width parameter was determined from the near-field data and was considered to provide a realistic value of 0.5*depth to tunnel axis.

With regards to the input geotechnical parameters for the numerical models, they were based on the Hoek-Brown criterion (Hoek and Brown, 1980) and the Geological Strength Index (GSI – Hoek, Kaiser & Bawden, 1995). The GSI enables an assessment of the strength and stiffness of a fractured rock mass (based on fracture spacing and the condition of the fracture surfaces) and the Hoek-Brown criterion enables the strength and stiffness properties to be scaled according to the volume of rock being considered for the problem.

The generalised Hoek-Brown strength criterion requires four basic rock properties to be assigned:

\[ \sigma'_{1} = \sigma'_{3} + \sigma'_{ci} \left( m_{b} \frac{\sigma'_{3}}{\sigma_{ci}} + s \right)^{a} \]

- The rock “material” uniaxial compression strength (\(\sigma_{ci}\))
- The material constant “m_b”, determined empirically or experimentally for the particular rock type, reducing with increasing intensity of fracturing (rock volume)
- The scaling constant “s” taken as 1 for intact (lab scale) rock, reducing with increasing intensity of fracturing (rock volume)
- The power constant “a” defining the curvature of the non-linear yield criterion in principal stress space.

Having ascertained an appropriately scaled non-linear yield criterion for the rock mass, equivalent linear strength properties in shear and normal stress space (i.e. cohesion “c_{mass}” and friction angle “\(\phi_{mass}\)”) can readily be derived at an appropriate value of confining stress (\(\sigma'_{3}\)). Hoek et al (2002) provide equations to estimate an appropriate range of confining stress below a maximum value (\(\sigma'_{3,\text{max}}\)) for both tunnel and slope stability problems. The equivalent Mohr-Coulomb rock mass strength parameters employed in the validation calculation presented in Appendix D.1 (cohesion = 0.1 MPa and friction angle = 43°) were then determined using the input parameters given in Appendix D.1 and the Hoek et al (2002) equations.

The GSI rating is also used to estimate an appropriate modulus of the rock mass volume (Hoek & Diederichs, 2006). This approach requires an estimate or laboratory measurements of the ratio between Youngs Modulus and \(\sigma_{ci}\) of the intact rock (termed the modulus ratio). This value is again reduced according to the intensity of fracturing/volume of rock mass being considered.
Another key input parameter is the initial state of in situ stress. Typically assumptions of high horizontal to vertical stress result in shallower-wider settlement troughs which result in lower (less conservative) ground slopes and strains. This is discussed further for the particular circumstances of the Project in the response to item (j) below.

**Rock stress**

The determination of rock stresses for design is usually most successful on projects at depth in relatively homogeneous rock masses (such as granitic or high-grade metamorphic terrains), or those that are relatively thickly bedded and sub-horizontally layered. For example rock stress measurements in the Bowen Basin coal measures and the Sydney Basin Permo-Triassic rocks typically yield good results using over-coring techniques (Macklin et al, 2014). This is because they comprise relatively undisturbed tectonic terrains and are relatively thickly bedded.

Hydro-fracture techniques have yielded reliable results in these terrains although significant uncertainty in the interpretation can arise when the assumption of vertical fractures (formed parallel to the maximum horizontal stress) does not hold. For this reason over-coring is a more common test method in current practice.

Other methods are available such as flat-jack testing and analysis of stress-induced over-break in cored boreholes. However flat-jack testing requires an underground excavation to be available and concerns typically arise from the effects of disturbance of the rock mass due to the excavation itself. Borehole “break-out” analysis requires a well-defined zone of over-break which, given known (or assumed) strength parameters for the rock, can be used to infer the state of stress around the borehole that caused the break-out to occur. A review of the down-hole geophysical data obtained for the Project have not revealed any convincing cases of borehole breakout.

For the rocks expected to be encountered on the Project Reference Design alignment, the conditions were assessed as being unsuitable for in situ stress testing because they are at relatively shallow depth, are general thinly bedded, have been affected by deep weathering and stress-relief processes, are complexly folded, faulted and affected by igneous intrusions.

It is inferred that these characteristics of the rock mass beneath Melbourne have resulted in relatively few published results on in situ stress testing because they are at larger volumes than the test itself (“scale-effect”) would mean that such values are not generally applicable.

The recent geotechnical investigations undertaken for the Melbourne Metro (18 No. tests at depths of between 20 m and 55 m below ground level, of which, 6 No. were undertaken in proximity to existing underground structures) are presented in Appendix A of the Ground Movement and Land Stability Impact Assessment report (20 April 2016). The results illustrate this variability with a very wide range in maximum horizontal stress orientations obtained, from 010° grid north (NNE) to 175° grid north (SSE).

The results presented for the Metro project also suggest that those up to 35 m below ground level have been affected by weathering and stress relief, which is common in the Melbourne Formation. The stress tests below this depth appear to show a marked increase in magnitude, inferred to reflect the lack of weathering and stress relief effects.
The upper tests show no clear trend with depth. Whilst no specific details are provided on the rock conditions at the test locations, it is inferred that the results presented may be biased towards thicker, more competent beds within the Melbourne Formation rocks, and so may not be representative of the more general rock mass conditions. As is well known for the Bowen and Sydney Basin data, the stiffness of the rock in which the test was obtained is a major factor in determining the value of stress measured (Nemcik et al, 2005). No normalisation of the data for this effect appears to have been undertaken for the results presented.

Representative values of the in situ stress ratios up to 35 m depth may be estimated from these plots as: major horizontal stress ratio between 1.4 to 0.8 and the minor horizontal stress ratio from 0.8 to 0.4. The ratio between the major and minor horizontal stress ratios is approximately 2. This suggests that an average horizontal stress ratio of between 1 and 0.6 would be representative between 20 m and 35 m depth. Given the potential for bias towards higher stiffness/thicker beds, it is considered that a horizontal to vertical stress ratio of 0.5 is a more appropriate value.

In addition, simple sensitivity studies for a higher ratio of one, shows that a shallower-wider settlement trough would be estimated, which yields less conservative settlements, ground strains and slopes. For EES purposes, the lower horizontal stress ratios were considered more appropriate, being a more realistic estimates of average stress in the rock mass and also because the settlement curves derived provided a better match to the empirical approach adopted for the risk assessments.