This advice is in response to request: Provide an interim report to the IAC to be tabled at the commencement of the Hearing on Thursday 25 July 2019 which sets out, within your area of expertise:

a. The matters required by the PPV Practice Note – Expert Evidence including all facts matters and assumptions upon which you have proceeded;

b. The key issues, including whether the key issues you identified prior to the circulation of evidence have changed, and if so, how;

c. Your expert view on the matters raised by paragraph 31 of the Terms of Reference in so far as they relate to the key issues you have identified;

d. Any areas in which you consider that there is insufficient information, having regard to the current and proposed future stages of the project (e.g., detailed design); and

e. Recommended changes to the approval documentation including the EPRs (if any).

Where referring to evidence, the EES or submissions please provide specific references.

List of Abbreviations

<table>
<thead>
<tr>
<th>Abbreviation</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>AECOM</td>
<td>Consulting company</td>
</tr>
<tr>
<td>AHD</td>
<td>Australian Height Datum</td>
</tr>
<tr>
<td>AS</td>
<td>Australian Standard</td>
</tr>
<tr>
<td>BOM</td>
<td>Australian Bureau of Meteorology</td>
</tr>
<tr>
<td>Coffey</td>
<td>Consulting company</td>
</tr>
<tr>
<td>DELWP</td>
<td>Victorian Department of Environment, Land, Water and Planning</td>
</tr>
<tr>
<td>ecoMarkets</td>
<td>GHD 2010, Port Philip CMA, Transient model development report, prepared for the Department of Sustainability and Environment, ecoMarkets project</td>
</tr>
<tr>
<td>EES</td>
<td>Environmental Effects Statement</td>
</tr>
<tr>
<td>EPA</td>
<td>Environment Protection Authority - Victoria</td>
</tr>
<tr>
<td>EPR</td>
<td>Environmental Performance Requirement</td>
</tr>
<tr>
<td>FE</td>
<td>Finite element model</td>
</tr>
<tr>
<td>FEA</td>
<td>Finite element analysis model</td>
</tr>
<tr>
<td>FH</td>
<td>Angle taken from the horizontal plane</td>
</tr>
<tr>
<td>GDE</td>
<td>Groundwater Dependent Ecosystem</td>
</tr>
<tr>
<td>GHD</td>
<td>Consulting company</td>
</tr>
<tr>
<td>GIS</td>
<td>Geographical Information System</td>
</tr>
<tr>
<td>GW</td>
<td>Groundwater</td>
</tr>
<tr>
<td>Haack</td>
<td>Water leakage class rating devised by Dr. A. Haack (1991)</td>
</tr>
<tr>
<td>HydroGeologic</td>
<td>Consulting firm</td>
</tr>
<tr>
<td>IREA</td>
<td>Independent Reviewer and Environmental Auditor</td>
</tr>
<tr>
<td>IAC</td>
<td>Inquiry and Advisory Committee (for the North East Link Project)</td>
</tr>
<tr>
<td>ISO</td>
<td>International Standards Organisation</td>
</tr>
<tr>
<td>Kh</td>
<td>Hydraulic conductivity (horizontal plane)</td>
</tr>
<tr>
<td>Km</td>
<td>Kilometres</td>
</tr>
<tr>
<td>Kv</td>
<td>Hydraulic conductivity (vertical plane)</td>
</tr>
<tr>
<td>L/sec</td>
<td>Litres per second</td>
</tr>
<tr>
<td>LIDAR</td>
<td>Light Detection and ranging - remote sensing method</td>
</tr>
<tr>
<td>Abbreviation</td>
<td>Description</td>
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<tr>
<td>--------------</td>
<td>-------------</td>
</tr>
<tr>
<td>m</td>
<td>Metres</td>
</tr>
<tr>
<td>m/day</td>
<td>Meters per day</td>
</tr>
<tr>
<td>m²</td>
<td>Squared metres</td>
</tr>
<tr>
<td>m³</td>
<td>Cubic metres</td>
</tr>
<tr>
<td>MODFLOW-USG</td>
<td>Unstructured Grid Version of MODFLOW</td>
</tr>
<tr>
<td>MMRT</td>
<td>Melbourne Metropolitan Rail Tunnel</td>
</tr>
<tr>
<td>MRI</td>
<td>Manningham Road Interchange</td>
</tr>
<tr>
<td>mya</td>
<td>Million years ago</td>
</tr>
<tr>
<td>NELP</td>
<td>North East Link Project</td>
</tr>
<tr>
<td>NZ</td>
<td>New Zealand</td>
</tr>
<tr>
<td>SCM</td>
<td>Site Conceptual Model</td>
</tr>
<tr>
<td>SEM</td>
<td>Sequentially mined excavation</td>
</tr>
<tr>
<td>SON</td>
<td>State Observation Network</td>
</tr>
<tr>
<td>SWL</td>
<td>Standing water level (applies to both groundwater and surface water)</td>
</tr>
<tr>
<td>TDS</td>
<td>Total dissolved solids</td>
</tr>
<tr>
<td>TBM</td>
<td>Tunnel boring machine</td>
</tr>
<tr>
<td>VVG</td>
<td>Visualising Victoria’s Groundwater</td>
</tr>
</tbody>
</table>
1 Response to Practice Note Information

(i) Name and address of the expert.

Craig Stephen Barker

(ii) Expert's qualifications and experience.


(iii) The expert's area of expertise to make the report.

Currently work in my own private engineering consultancy firm, which conducts geotechnical investigations, groundwater investigations and contaminated land investigations. Spent the first eight years of my consulting working life as a full-time Geotechnical Engineer, involved with rock mechanics studies, slope stability studies, detailed site investigations and finite element numerical modelling (Sydney Harbour immersed tube road tunnel and Malanjkhand Copper Mine expansion study, India).


I have a Masters of Geotechnical Engineering Studies from Sydney University.

I am a Fellow with the Institution of Engineers Australia and a Member of the Australian Geomechanics Society.

(iv) Any other significant contributors to the report and where necessary outlining their expertise.

There were no other contributors.

(v) All instructions that define the scope of the report (original and supplementary and whether in writing or oral).

IAC request to prepare this report on tunnelling and geomechanics (‘ground movement’) issues as per letter of 14 May 2019.

(vi) The identity of the person who carried out any tests or experiments upon which the expert has relied on and the qualifications of that person.

None.

(vii) The facts, matters and all assumptions upon which the report proceeds.

The North East Link Project (NELP) Environment Effects Statement (EES);
Expert witness reports;

Submissions;

Information meeting attendance with applicable NELP technical and planning staff held on 11 July 2019 at NELP Offices, Melbourne.

(viii) Reference to those documents and other materials the expert has been instructed to consider or take into account in preparing his or her report, and the literature or other material used in making the report.

NELP EES Summary Report
NELP EES Main Report Volumes – Chapters 1 to 28
NELP EES Technical Report M – Ground Movement
NELP EES Technical Report N – Groundwater
NELP EES Attachment III Risk Report
NELP EES Attachment IV Stakeholder Consultation Report
NELP EES Attachment V Planning Scheme Amendment
NELP EES Attachment VI Works Approval
NELP EES Map Book
NELP Environmental Performance Requirements (EPRs) for the project

Expert Witness Statements:

Public Submissions to the EES
Australian Standards
References (see detailed list below).
References


(ix) A summary of the opinion or opinions of the expert.

In reviewing the EES documentation as outlined under Point (viii), I have formed the following opinions:

That ground movements can be managed by strict adherence to the objectives, care in exceeding those objectives and the implementation of measures (through the EMF and EPRs) to review and audit for compliance in working to objectives.

(x) A statement identifying any provisional opinions that are not fully researched for any reason (identifying the reason why such opinions have not been or cannot be fully researched).

The report of the Ground Movement Conclave was not available at the time of writing this advice, so I have not considered its content at this time.

(xi) A statement setting out any questions falling outside the expert’s expertise, and whether the report is incomplete or inaccurate in any respect.

Not applicable.
(xii) Declaration

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 IAC.
2 Further Information

(i) Question
Has the information that you previously requested in your letter dated 18 June 2019 been provided? (Noting that some responses are not due until the commencement of the Hearing).

(ii) Response
Some verbal responses have been provided by NELP project personnel associated with the technical discipline of ground movement, through the conduct of an information seeking meeting attended at the Offices of NELP, by the following persons and myself on 11 July 2019:

• Mr. Steve Macklin – Senior Technical Director – Tunnelling and Engineering Geology, GHD;
• Mr. Toby Shurler – Junior Tunnel Engineer, GHD; and
• Ms. Irene Clarke – Planning Team, GHD.

Responses were provided by Mr. Macklin and Mr Shurler, following my questions put to them across the discipline subject. Outstanding previous information request are listed in Table 1.

Table 1 – Summary of Outstanding Information Requests

<table>
<thead>
<tr>
<th>Question</th>
<th>Page</th>
</tr>
</thead>
<tbody>
<tr>
<td>Have any ‘small utilities’ thus far been identified by NELP that warrant a particular assessment?</td>
<td>Page 16, Section 5.2</td>
</tr>
<tr>
<td>Provide the output evidence of the FEA related to the Banyule Homestead slope stability appraisal.</td>
<td>Page 54, Section 8.2.5</td>
</tr>
<tr>
<td>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).</td>
<td>Page 1, Appendix A and Page 81</td>
</tr>
<tr>
<td>Provide the forming calculations that were the basis for Table 1</td>
<td>Appendix D1 – Numerical Validation</td>
</tr>
</tbody>
</table>
(iii) Question
Please list any further requests for information that have arisen from your further review of the documents.

(iv) Response
No requests required.
3 Key Issues

(i) Question
Please list the issues that you consider to be the key issues arising from the proposed North East Link project relevant to your expertise and falling within the scope of the IAC’s Terms of Reference.

(ii) Response
The EES scoping requirement involved considering potential for ground movement, or other related geophysical condition that may pose risk to land, river and creek banks or to river/creek bed stability.

Key project-wide issues raised from the EES include:

- Potential for adverse impact from construction staging for tunnels and excavations, impacting upon existing building and infrastructure assets, via ground movement (both from construction and operational stages of the Project);
- Most of the tunnels and related structures are generally below the groundwater table;
- The potential for unanticipated variance in the geological conditions along the Project alignment, which could result in the use of inappropriate tunnelling or excavation techniques, resulting in higher than expected groundwater inflows to excavations and respective groundwater drawdowns, construction delays, increased ground movements, or possibly all of these in combination. There are zones of significantly varying and complex geology along the Project alignment.

The following discussion topics and related outcomes are discussed herein:

- Mechanisms of Ground Movement;
- Geology / Conceptual Site Model;
- Tunnelling Aspects;
- Movements Associated with Cut & Cover or Retained Trenches;
- Consolidation Settlements;
- Slope Instability Movements;
- Ground Movement Effects on Buildings & Infrastructure;
- Project Risk Profiles; and
- Comment Across EPRs.
(iii) Question

In a document dated 18 June 2019, you identified a number of issues which you considered to be the key issues arising from the proposed NELP relevant to your expertise and falling within the scope of the IAC’s Terms of Reference.

If your list above differs from the list previously provided, please provide a brief explanation for the change.

(iv) Response

There are no significant differences.
4 Mechanisms of Ground Movement

Ground movements that are associated with major engineering, dewatering of groundwater and soil/rock excavation works may result from stress changes imposed by:

- Initial elastic response and plastic creep of strata as a response to excavated trenches or created voids in the ground, until the in-situ stress balance is re-adjusted through the placement of supporting structures or countering loads (convergence);
- Surface loadings on existing sediments or underground services (such as the placement of embankment structures, or removal of bulk filling at certain areas);
- The placement of cuttings within the ground or the disturbance of the ground from void forming, which may cause the mobilisation of pre-existing failure planes along defects in the ground (such as faults, joints or bedding planes), resulting in slope instability;
- The dewatering of strata, which results in a change in the porosity of that supporting strata and increasing of effective applied stress on the material, causing consolidation settlement within compressible strata.

Ground movements involve both settlements and deflections of the ground surface, where the magnitude and direction of these movements are influenced by the proximity to the applied stress, the intensity and area over which the stress is applied, the geotechnical parameters of the strata, timing sequence of the applied stress through construction methods and the deployed quality/control of the workmanship for the works.

(i) Issues Raised by Submitters

Please include a brief summary of the key issues raised by submitters. If you refer to a particular submission please refer to the submission by number and not by the name of the submitter.

(ii) Response

No comment.

(iii) Question

Where your opinion(s) materially differ from the relevant circulated evidence statements, please briefly outline the difference and reasons for it.

(iv) Response

No significant differences.
(v) **Question**
Please discuss the magnitude, likelihood and significance of adverse and beneficial environmental effects.

(vi) **Response**
Covered elsewhere in the document.

(vii) **Question**
Please address the adequacy of the proposed environmental management framework, including the proposed environmental performance requirements and environmental management measures contained in the EES, with reference to applicable legislation and policy.

(viii) **Response**
Refer to Sections 1 and 12.

(ix) **Question**
Please address the adequacy of the impact assessment and whether the proposed environmental performance requirements are capable of being met.

(x) **Response**
See Sections 10 and 11.

(xi) **Question**
Please address the question of feasible modifications to the design of the Project within or reasonably proximate to the project boundary that could offer demonstrably overall superior outcomes.

Not relevant.
5 Geology / Conceptual Site Model

The NELP is to be constructed through a range of natural geological strata, where these materials are relatively well known within the Melbourne area. There are also certain areas through the project, where past anthropogenic land uses have resulted in poor soils/fill (quarries, former landfills and existing embankments), where behaviour prediction for ground movement can become problematic.

Basement rock (steeply dipping sandstone and siltstone) is of Middle Palaeozoic age, belonging to the Silurian and Devonian periods (354 to 441 million years ago (mya)). This is overlain at certain areas by younger rock, mostly of Tertiary and Quaternary age (2 to 65 mya). Anderson Creek Formation (early Silurian) rock outcrops in the north-east (around Warrandyte). Melbourne Formation (late Silurian (418 mya)) basement rock is found south of Warrandyte and within the Melbourne CBD. These Silurian and Devonian rocks were folded into a series of anticlines and synclines in the Middle Devonian, during a folding event called the Tabberabberan orogeny (about 380 mya). Following this event, the bedrock was intruded in places by plutonic rocks (granite family of Late Devonian age).

Following the end of the Devonian period, there was a long period of weathering and erosion across 300 million years. Then, in Early Tertiary times (from 65 mya), large block-faults produced a wide shallow depression, known as the Port Phillip Sunkland. Port Phillip Bay and its immediate land surrounds are located within the Port Phillip Sunkland, where this block is bounded to the east by the Selwyn Fault and across to the west by the Rowsley Fault. The northern extent of the block is estimated to occur near the Melbourne Warp feature (ref 13). Overlying the Werribee Formation (Tertiary) is a thick sequence of Tertiary basaltic lava, known as the Older Volcanics. These occur in the north from Bundoora to Greensborough. In many places these lavas are highly weathered. They are thought to have been erupted in Eocene and Oligocene time (about 30-55 mya) and may have been associated with the block faulting forming the Port Phillip Sunkland. Dykes - intrusions of igneous rocks filling wide fissures in rocks, thought to be associated with the Older Volcanics - are visible in places of the Eastern Freeway road cuttings, through Silurian bedrock.

The ‘Nillumbik Terrain’ (across the north and east of Melbourne) as described by Neilson (ref. 9) as an erosional landform cut into the folded Silurian rock. This feature is overlain in some places by almost flat lying, ferruginous Tertiary sands (‘Brighton Group’ – late Miocene) which cap hills in certain places (i.e., Kew, Camberwell and Heidelberg). The Yarra River, Gardiners Creek, Plenty River, Diamond Creek and their associated tributaries have deeply cut, old valleys through the Tertiary deposits, into Silurian bedrock (leaving only remnants of these Tertiary sands at isolated locations).

The Yarra River emerges from Warrandyte Gorge to a mature, broad, alluviated valley at Templestowe and Heidelberg (wide alluvial flats, subject to flooding). These valleys have originated from downstream damming lava flows (Newer Volcanics), that flowed down the ancient valleys of Merri Creek and Darebin Creek, to the old valley of the Yarra River.

Basement (Silurian) rock consists of a series of wave/folding (average wavelength of 1.2 km) where two key anticline features bound the general project area (Templestowe and Whittlesea). Major syncline features for the project area are the Bulleen and
Greensborough Synclines. The general strike of these folds across most of Melbourne is suggested at North 20° to 25° East. Rock folding is accompanied by minor faults, where small reverse strike faults and transverse faults are common. The closest known major fault is the Selwyn Fault, which has a general North 20° East trend, which passes up the eastern side of Port Phillip Bay (from Dromana to Frankston). To the west of this fault, folding of basement rock is typically on a North 20° East strike. The Brushy Creek Fault (near Wonga Park) runs sub-parallel to the Selwyn Fault and is thought to be a subsidiary movement to the Selwyn Fault.

The Project geotechnical model interprets the following:

- Inter-bedded siltstone/sandstone folded on a general north to north east trending axis (matched to the above geological observations), associated with faulting and some intruded dyke zones:
  - Project drilling has shown that beneath the Yarra Valley sediments there are a several potentially thick, high persistent faults, comprising crushed rock, sand and clay derived from parent siltstone, which are interpreted to behave as a soft soil material (considered by the Author as a reasonable assumption);
  - No dyke intrusions through the siltstones (from current Project investigations) have been encountered with vertical drilling investigation methods – their occurrence may complicate tunnelling methods; and
  - The siltstones are deeply weathered, often to 30 m depth, but weathering and general rock strength can be variable.

- Yarra Valley Alluvium may attain thickness of generally between 9 m to 15 m across river flats up to 1.6 km wide:
  - The lower beds of this alluvium are considered as lacustrine, deposited when the river was lake dammed from downstream lava flows;
  - Higher flow energy lenticular river channels of sands and gravel may also occur through these alluvium deposits;
  - The EES makes the assumption (considered to be conservative) when dealing with alluvial deposits (i.e. relatively unconsolidated) and associated behaviour under stress response – that the thickness of the soft/compressible layer is to be taken as the total estimated thickness of the alluvium layer, less its starting unsaturated thickness from the top of this alluvium layer; and
  - The alluvial aquifer and the underlying siltstone bedrock aquifer are conservatively assumed to be inter-connected (which affects aquifer depressurisation predictions from deployed models).

- Near Manningham Road, there are a series of interpreted east to west draining, ancient alluvial ‘paleo-channels’, stranded from the continued down-cutting of the Yarra River. The in-fill material here is believed to be Brighton Group marine sandy clay. To the south of Manningham Road (near Ilma Court) there are also in-filled drainage channels (Miocene alluvium) close to the proposed tunnelling ‘crown’.
(i) Question
Please include a brief summary of the key issues raised by submitters. If you refer to a particular submission please refer to the submission by number and not by the name of the submitter.

(ii) Response
No comment to add here.

(iii) Question
Where your opinion(s) materially differ from the relevant circulated evidence statements, please briefly outline the difference and reasons for it.

(iv) Response
No differences.

(v) Question
Please discuss the magnitude, likelihood and significance of adverse and beneficial environmental effects.

(vi) Response
Refer to other Sections for this discussion.

(vii) Question
Please address the adequacy of the proposed environmental management framework, including the proposed environmental performance requirements and environmental management measures contained in the EES, with reference to applicable legislation and policy.

(viii) Response
Not applicable.

(ix) Question
Please address the adequacy of the impact assessment and whether the proposed environmental performance requirements are capable of being met.

(x) Response
Not applicable.
(xi) **Question**

Please address the question of feasible modifications to the design of the Project within or reasonably proximate to the project boundary that could offer demonstrably overall superior outcomes.

Not applicable.
6 Tunnelling Aspects

General

The Project’s underground excavations (TBM-bored, or cavern-type tunnels and cross-passages) will contribute to some degree of ground deformation, as the influence from the excavation void propagates through the rock or soil formation to ground surface (even with tunnel lining support implemented). Tunnel mining will use TBMs, or in the case for Reach 5, other cavern construction and linking cross passage, tunnels are likely to be stage-mined using road headers or other mobile plant.

- With TBM tunnelling, the ground formation just ahead of the TBM face can deform inwards to the tunnelling face, where also, in some circumstances, an amount of ‘ground heave’ may occur just ahead of the TBM, if outward tunnel pressure is required to be applied at the TBM face for suitable tunnelling support (such as with soft sediments or water-charged areas). Above and behind the advancing TBM face, the ground typically deforms towards the resultant excavation from this ‘volume loss’. This volume loss effect at the ground surface, takes the form generally of a smooth ‘trough’ of surface depression. If tunnelling volume loss significantly propagates to the ground surface (say in the case of relatively shallow tunnel cover, or with poor ground support), then the settlement trough may have sides that are of steep-enough slope, to induce tension into an overlying ground surface structure (due to the resultant differential ground movement that the structure experiences); and

- With mined caverns in rock, their excavation requires careful staging, to combine rapid ground support both prior to, during and after excavation to minimise the stress relief of the rock and to limit ground movement. Tunnel support is provided through a staged combination of temporary supports, via steel cables, canopy reinforcement tubes, steel girders, rock bolts and shotcrete lining, followed by further final concrete lining of the tunnel towards the end of the excavation process. The final mode of tunnel support depends on the ground condition.

Whilst geology and ground conditions are critical to the design of underground openings, tunnel excavation shape is also a key design parameter. Increased tunnelling risk can occur where flat roofed tunnels are required, such as with those used for road tunnels where at road intersections, tunnel cover to span ratio can reduce (often to as much as 0.25). Generally, underground caverns utilise arched profiles to eliminate large zones of tensile stresses within cavern roofs (the main exception to this is where major horizontal rock bedding allows arched profiles to be avoided). Conventional arched cavern design generally aims to achieve a uniform compressive (tangential) stress around the excavation perimeter.

- High compressive stresses may instigate rock mass shear failure; and

- Zones of high tensile stress can significantly influence rock structure-controlled instability.

Use of two-dimensional rock mass models can provide suitable guidance on rock mass behaviour, leading into more detailed tunnel route and intersection planning. For complex
geometrical and geological aspects related to cavern design it is important however, to use well-suited three-dimensional numerical modelling methods for lead-in design. These models need to cater of the suitable idealisation of the rock mass (geologic structures, rock mass properties), applied in-situ stresses and applicable boundary condition effects. This requires very detailed geotechnical input data for the model to carefully plan for final roof span considerations, cavern shape and the planned tunnel construction sequence. Careful tunnelling supervision and experience is also required, together with careful monitoring across tunnelling, as even subtle changes in rock condition can cause issues.

The Segmented Excavation Mined (SEM) method is characterised by a process of sequential excavation and construction of tunnel support measures. Because of this, tunnel construction behaviour is dependent on both spatial and temporal development. Predictive numerical models need therefore, to be able to facilitate such properties, as well as offering an efficient treatment of non-linear support material behaviour and inhomogeneous ground conditions.

The design of tunnelling support involves three main types of concept allowance when designing (Pells, ref. 10):

- **Loosening Pressure Allowance**, which is that caused from the weight of loosened rock blocks or prisms bearing onto the tunnel roof and walls;
- **Swelling Pressure Allowance**, which may be caused by volumetric increases in clays or claystone, due to exposure to the atmosphere under altered stress conditions (not likely the case for this Project); or
- **True Rock Pressure Allowance**, which is the pressure on the tunnel structure when compressive stresses in the rock at zones around the tunnel perimeter are sufficiently large to cause rock failure (this can occur around the general tunnel perimeter, or may propagate, via a defect plane (such as a small fault).

### Tunnelling Volume Loss & Settlement Prediction

Volume loss is defined as the additional excavation for a tunnel, in excess of the theoretical required void volume (expressed as the percentage ratio of additional volume to the estimated tunnel volume).

- The range of volume loss from TBM tunnelling has been assumed to range from between 0.4 % to 0.8 %:
  - Lower Plenty Road to Banyule Flats (‘Reach 4’): 0.8 % volume loss;
  - Banyule Flats to Manningham Road Interchange (‘Reach 5’): 0.4 to 0.8 % volume loss; and
  - Avon Street to Rocklea Road (‘Reach 7’): 0.3 % volume loss.
- Primarily most of the tunnelling is targeted through Silurian siltstone (fractured, relatively soft rock); and
- As a comparative guide, for the Melbourne Metropolitan Rail Tunnel (MMRT) project, estimated volume losses through Melbourne Formation siltstone (at generally similar depths of tunnelling) were:
o TBM through siltstone rock: 0.5 % volume loss.

o SEM in siltstone rock: 0.5 % volume loss.

o Use of TBM with a closed face in soft/poor ground: 1 % volume loss (such as faulted areas).

The values as assumed seem comparable. The NELP EES suggests a volume loss using SEM of 0.3 % which may prove optimistic.

The EES discusses the process of settlement trough width prediction and how some degree of numerical modelling validation has been input to this process. For the prediction of settlement and strain response the Gaussian Bell-Curve Model has been deployed (which is an empirical curve fitting method). There are some issues with ground movement prediction inaccuracy and applicability of using this method when estimating total settlements, extent (width) of the settlement-effect trough and change in ground slope:

- Volume of the settlement ‘trough’ is matched to the Gaussian Bell Curve, taken as equal to the volume loss associated with tunnelling;

- Volume loss is defined as a ratio of the over-excavated material to the theoretical excavation volume (usually expressed as percentage);

- Trough width is taken as equivalent to ‘one standard deviation from the mean value’ which is the typical curve inflection point for the Gaussian Bell Curve shape; and

- Volume loss (VL) for a circular tunnel may be estimated based on the predicted radial convergence around the tunnel (δ) and tunnel starting diameter (D): $\text{VL} \% = 4 \times (\delta/D) \times 100$.

For non-homogeneous rock (such as high bedded /defected siltstone), ground movements may be driven more by specific displacements along discrete bedding, fracture lines or joint lines. If tunnel monitoring shows a discrepancy such as this type of movement, the general approach to counter this observation, is to add further tunnelling support at such areas to limit these more-specific inward ground movements.

Trough width parameter (i) is a ratio of the tunnel depth – generally (i) was taken as shown in Table 1. When these suggested values are compared to that deployed in association with the Melbourne Metropolitan Rail Tunnel (MMRT) EES, they compare favourably.

Table 1 – Comparison Across Settlement Trough Width Parameter “(i)” (NELP vs MMRT)

<table>
<thead>
<tr>
<th>Geologic Medium</th>
<th>NELP</th>
<th>MMRT (Melbourne ‘Metro’)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Clay type soils</td>
<td>0.5 * Zo</td>
<td>-</td>
</tr>
<tr>
<td>Sand type soils</td>
<td>0.3 * Zo</td>
<td>-</td>
</tr>
<tr>
<td>Soil (All types)</td>
<td>-</td>
<td>0.4 * Zo</td>
</tr>
<tr>
<td>Alluvials</td>
<td>(see sand above)</td>
<td>0.3 * Zo</td>
</tr>
<tr>
<td>Fractured siltstone rock</td>
<td>0.7 * Zo</td>
<td>0.6 * Zo</td>
</tr>
</tbody>
</table>

Table Note:
1. Zo is the depth to the tunnel crown from the ground surface.
For the case of a fractured siltstone/sandstone rock mass, particularly when movements and strains are more likely to be associated with discrete fractures or defects in the rock, the project EES proposes more-detailed, future assessment (to confirm the current risk assessment appraisal based on current ground movement prediction), through the conduct of more-detailed and complex numerical methods which may better simulate the behaviour of dominant fractures and faults/features. This future check modelling and monitoring of rock behaviour should be included with an EPR.

Discussion on ‘Confinement – Convergence Method (CCM)’ Related to Settlement Prediction

For rock, ground movement effects may be determined using the Confinement–Convergence Method (CCM), where the tunnelling excavation process is simulated back to a two-dimensional analysis predictive model, through the deployment of a rock ‘relaxation factor’. Rock movements into the excavation cease, when the tunnel lining is installed, after an appropriate degree of rock stress relaxation has occurred. The degree of rock stress relaxation which can occur is the key influencing parameter for settlement prediction (it depends largely on the delay required to install the final tunnel lining and to make meaningful stressed contact with surrounding rock (i.e., to ‘take up’ load).

The rock may also yield to some extent during this stress relaxation process (further complicating the predictive process). Yield strength potential for a jointed/detected rock mass is commonly estimated through use of descriptive rock quality ratings (such as Bieniawski’s Rock Mass Rating (RMR), or Geological Strength Index (GSI)). Jointed rock shows distinct behavioural differences when compared to intact rock. Behaviour is highly dependent on the jointed rock properties. The Hoek-Brown failure envelope criteria has been in use throughout the world since 1980 (ref. 13). It defines an empirical relationship between the applied principal stresses to the rock, or between shear and normal stresses at rock mass failure. It is often used in computer numerical modelling, due to the difficulty with direct modelling of the fracture propagation processes and failure processes for rock. The key curve defining parameters to the Hoek-Brown criteria cover the influences of rock intact compressive strength and the extent that the rock mass is broken into, prior to loading. This is done by linking updated versions of RMR to observed project geological observations. Hoek et al (ref. 14) provides the means to establish a stress-dependent relationship between the Hoek-Brown rock failure model and the Mohr-Coulomb elastic-plastic, failure model (which is routinely used with many soil mechanics studies and numerical models).

The Effect of Pre-Tunnelling Rock Stress Condition & Rock Defects/Anisotropy

Pells (ref. 10) discusses his experience in relation to tunnelling in horizontally bedded sedimentary strata across rocks in Southern Africa (Karoo sediments), the Sydney Basin (Triassic and Permian strata) and cretaceous mudstones within Central and Northern Queensland. Across all these areas, he stated it was a common feature that natural horizontal stresses were greater than overburden pressure ‘in most near surface rocks natural horizontal stresses are greater than overburden pressure’. Pells describes how elastic analysis helps to demonstrate that horizontal defects within a relatively high horizontal stress field can have a major impact on the level of stress concentration around a tunnel:
• Low shear strength materials near horizontal defects can cause very large stress concentrations where the natural horizontal stress is the major principal stress;

• Stress concentrations that develop, can lead to the development of rock shear failure and buckling in the crown and floor of a tunnel under relatively low cover (unless suitable SEM methods are deployed); and

• Shear failure and buckling within the tunnel’s invert can be just as important as a tunnel crown failure under these conditions.

One key issue from the EES, is that there is a lack of investigation and understanding of regional rock stress fields for the Project.

• The current design information assumes that horizontal and vertical stresses are in a hydrostatic type of assumed equilibrium / balance;

• There is no obvious referring to, or reporting of the measurements of stress ratios estimated in the field for these basement rocks (siltstones) from Geotechnical Investigations; and

• Varying stress ratio and rock bedding anisotropy are expected to have an influence of predicted ground movement zones and magnitude of settlement/strain.

The Author has researched the Geotechnical Investigation Summary conducted for the MMRT, where the following key points were noted by Golder Associates for the Silurian fractured siltstone in the Melbourne CBD:

• Bedding planes are typically persistent. The rock has been subject to east-west compressive regional tectonic deformation across the Devonian Period, which has folded and faulted the rock;

• The siltstone fold axis is north to south trending (approximately N 20° E), with a typical fold spacing of between 1 to 2 km (but smaller parasitic folds are also present);

• Sandstone beds within the siltstone (typically make up some 25 % of the rock mass) surround tend to be stiffer and more competent than the siltstone beds (but the sandstone tends to be more fractured);

• There are often steeply dipping normal and reverse faults that trend in a similar direction to the fold axis. Dykes typically follow the orientation of the discontinuities;

• Past in-situ stress field measurements in the basement siltstone for the earlier Melbourne Underground Rail Loop (MURL) suggested that typically, in-situ horizontal rock stress exceeded vertical in-situ stress (by a ratio of at least 1.5); and

• In-situ stress field testing conducted by Golder Associates for the MMRT suggested this stress ratio increase was even higher in certain places (up to 4 to 5 times). More typically, the major horizontal stress is up to 1.75 times the estimated current vertical stress, where the average direction of the major principal stress direction was oriented 115° from north.
The Author has also considered a simple analytical solution offered by Obert and Duval (1967, ref. 15) for radial displacement at the tunnel face resulting from a circular excavation (matched to the general project TBM tunnel dimensions), but for an unlined void opening within an infinite elastic medium. Table 2 provides a summary of the effect of altering the ratio of vertical stress to horizontal stress.

### Table 2 – Comparison of Radial Tunnel Wall Displacement to Applied Stress Ratio Variance

<table>
<thead>
<tr>
<th>Applied Stress Ratio</th>
<th>Top of Tunnel</th>
<th>Side of Tunnel</th>
<th>Base of Tunnel</th>
</tr>
</thead>
<tbody>
<tr>
<td>$\sigma_H = \sigma_V$</td>
<td>1.6</td>
<td>1.6</td>
<td>1.6</td>
</tr>
<tr>
<td>$\sigma_H = 2 \times \sigma_V$</td>
<td>4.1</td>
<td>2.8</td>
<td>0.8</td>
</tr>
<tr>
<td>$\sigma_H = 3 \times \sigma_V$</td>
<td>5.8</td>
<td>3.3</td>
<td>0</td>
</tr>
</tbody>
</table>

**Table Note:**
1. Example calculation assumes poisons ratio = 0.25, drained Youngs Elastic Modulus = 8,000 MPa.
3. Assumed tunnel diameter = 8 m.

This demonstrates the significant affect that stress ratio has on inwards settlement magnitude to the excavation void, where the general inwards movement to the void essentially doubles when the horizontal stress to vertical stress ratio increases from a hydrostatic state to a ratio of 1 to 3 (vertical stress to horizontal stress as applied).

**The Need for Pre-planning Design & Experienced Observation with Tunnelling**

Tunnelling is described by Pells as an intimate blend of engineering geology, precedent, structural analysis and the observational method across construction (after Karl Terzaghi). He makes the important point ‘design is only completed when construction is completed’. The experiences from the Lane Cove Tunnel, Sydney collapse (November 2005) should not be forgotten in relation to this. The Lane Cove tunnel was part of a Design and Construct project, which resulted in collapse of a road tunnel intersection excavated at shallow depth in poor quality Ashfield Shale rock. Tunnel cover to ground surface from crown was approximately 17 m bgs, where collapse occurred during final tunnel excavation by road header, near an L-shaped intersection of a ventilation tunnel and the exit ramp to the Pacific Highway. The collapse occurred at tunnelling works moved into poorer geotechnical condition (weathered shale with geological defects) and reduced tunnel cover. It coincided with the change of tunnelling method from full-face to partial face excavation, combined with a reduction in tunnelling supervision. The deployed intersection support of the rock at the time used a design alternative (a system of structural shotcrete and rock bolts) to the originally scheduled use of steel sets, which had previously been introduced to cope with dyke-affected shales.

- One of the outcomes of post-failure detailed investigations, was that most of the computer numerical modelling simulations (as prepared post-failure) did not predict rock failure condition, where it was offered that due to the type of observed rock failure (progressive unravelling), such models would have had to include the specific effects of dyke presence and blocky rock (through use of very specialised numerical models); and
• The project operated under a complex series of contracts, deeds and undertakings. The residing Judge commented that the contractual process lacked control, particularly around quality management. The in-place project management system at the time was suggested as a significant factor in the altered design and implementation of the tunnel roof support, where it was suggested there is often an over-reliance on such project management systems, when compared to sound, experienced engineering judgement.

Dr. Barry McMahon’s paper on the conduct of geotechnical design in the face of uncertainty (ref. 16) points out for major projects and related geotechnical engineering design, the resultant uncertainty associated with site investigations into a design can often pose large interruptions and financial consequences. It is important to understand the limitations of design technology as available, to plan for careful supervision and management across construction and to suitably communicate risks to the community. McMahon discusses the uncertainty influencers of bias, statistical errors, homogeneity, scale effects and spatial dependency on risk estimates, where he calls up six types of geological uncertainty:

• ‘Type 1’: That of an unknown geological condition such as a nearby hidden shear zone (‘unknown unknowns’);
• ‘Type 2’: Risk of using the incorrect geotechnical criteria (such as assuming the incorrect failure mechanism for a slope);
• ‘Type 3’: Risk of bias and/or variation in the adopted design parameters (such as rock strength input parameters) is greater than has been estimated (‘known unknowns’).

The need for bias and/or variation needs to be understood through:

  o Estimating bias;
  o Determining statistical homogeneity (through correct ‘Domain’ recognition for the geology);
  o Determining spatial dependency or spatial independency (such as occur with strike lines and dip angles for rock defects); and
  o Scale effects on variance estimates from site measurement;

• The other three uncertainty types relate to: ‘Human Error’ (errors in observations, testing, calculations or communication on the problem understanding), ‘Design Changes’ (unreported, or without back-checking across the previous models) and use of ‘Over-Conservatism’ (Inherent conservatism produced in assessing either factors of safety or probability of failure).

Dr. McMahon estimated from his experience at that time, that some 36% of major projects encounter serious geotechnical problems during their construction, where approximately two-thirds of these were due to inadequate recognition of the uncertainty from the geotechnical design analysis, where the remaining one third resulted from unknown geological conditions, use of incorrect geotechnical criteria, post investigation design alterations, or human error. Dr. McMahon suggested for major projects, there will always be a large amount of uncertainty regarding the geotechnical design of the structures at the end of the investigation phase (no matter to what level the investigations can be practically
taken to). He offered that these uncertainties therefore need to be suitably managed across the construction stage, where at that time design and implementation works should be suitably adjusted to suit the altered conditions.

(i) **Issues Raised by Submitters**

Please include a brief summary of the key issues raised by submitters. If you refer to a particular submission please refer to the submission by number and not by the name of the submitter.

(ii) **Response**

See previous.

(iii) **Question**

Where your opinion(s) materially differ from the relevant circulated evidence statements, please briefly outline the difference and reasons for it.

(iv) **Response**

Refer to the front of this Section for detailed comment.

(v) **Question**

Please discuss the magnitude, likelihood and significance of adverse and beneficial environmental effects.

(vi) **Response**

Refer to the front of this Section for detailed comment.

(vii) **Question**

Please address the adequacy of the proposed environmental management framework, including the proposed environmental performance requirements and environmental management measures contained in the EES, with reference to applicable legislation and policy.

(viii) **Response**

Please refer to the relevant Section for this comment.

(ix) **Question**

Please address the adequacy of the impact assessment and whether the proposed environmental performance requirements are capable of being met.

(x) **Response**

Refer to other Sections.
(xi) **Question**

Please address the question of feasible modifications to the design of the Project within or reasonably proximate to the project boundary that could offer demonstrably overall superior outcomes.

- Measure in-situ rock stresses for the project lead-in with geotechnical investigations;
- Look to model the effect of rock strength and stiffness anisotropy using available computer numerical models;
- Collect as much observations data for these geology’s as practicably possible (use the Victorian Government to obtain past collected data from other projects: MMRT, East Link).
7 Movements With Cut & Cover or Retained Trenches

The three main mechanisms which cause ground deformation adjacent to deep excavations (associated with either Cut and Cover or Trenching) are listed (ref. 10) as:

- Lateral ground pressure acting on the shoring system;
- Excavation induced reduction in confining stress, which leads to an inwards movement of the ground/rock mass; and
- Placement of drill-holes for supporting tensile ground anchor construction.

Lateral soil pressures and resulting movements/deflections of shoring support systems can normally be predicted with reasonable accuracy across a range of basic to highly detailed (numerical) modelling methods. Movement/deflections may be minimised through adjustment of the design and construction sequence based around model predictions:

- Rock mass relaxation resulting from a significant reduction in confining stress is considered generally independent from the deployed shoring design (where essentially it cannot be controlled by any practical supporting means);
- Attempting to predict ground deformation caused by anchor drilling normally cannot be made with any suitable accuracy (where such deformations depend on the volume of soil removed by anchor drilling); and
- Pile or diaphragm wall deformations can also impact greatly on ground movement predictions.

For retaining wall structures, given the stated typical types for the Reference Design, most of these structural supporting walls are expected to have a high relative stiffness compared to the supported ground, where variation in ground condition and the style of anchoring or propping of the excavation are typically the major influencers for ground movement.

- For stiffer ground conditions (such as is the general case for this Project) use of the trough estimation process by Clough & O’Rourke (1990) and
- For the case of Bulleen Road North Portal, where stiff clay alluvium is involved (i.e., a softer soil) – use of GABA empirical curve fitting method 2017 or CIRIA C760 report methods were considered appropriate for predictions.

(i) Issues Raised by Submitters

Please include a brief summary of the key issues raised by submitters. If you refer to a particular submission please refer to the submission by number and not by the name of the submitter.

(ii) Response

See previous.
(iii) Question
Where your opinion(s) materially differ from the relevant circulated evidence statements, please briefly outline the difference and reasons for it.

(iv) Response
See previous discussion.

(v) Question
Please discuss the magnitude, likelihood and significance of adverse and beneficial environmental effects.

(vi) Response
Covered in other sections.

(vii) Question
Please address the adequacy of the proposed environmental management framework, including the proposed environmental performance requirements and environmental management measures contained in the EES, with reference to applicable legislation and policy.

(viii) Response
Covered elsewhere.

(ix) Question
Please address the adequacy of the impact assessment and whether the proposed environmental performance requirements are capable of being met.

(x) Response
Covered elsewhere.

(xi) Question
Please address the question of feasible modifications to the design of the Project within or reasonably proximate to the project boundary that could offer demonstrably overall superior outcomes.
See previous comments.
8 Consolidation Settlements

Consolidation settlement may occur with soft or loose sediments resulting from groundwater drawdown or the placement of a new fill embankment onto the sediments (in both cases, this causes an effective stress increase to the soil, with subsequent strain settlement). These types of soils are mainly expected to be encountered across Project Reaches: 5, 6, 7, 8 and 9. Consolidation settlement can take the form of both a primary-phase settlement from the stress increment, followed by a secondary-phase ‘creep’ consolidation – sometimes called ‘secondary compression’ (normally notably less in its magnitude and rate of settlement, when compared to primary consolidation settlement). Most dewatering of the soil and rock formations associated with the Project will occur across its construction stage and not into the longer-term operational stage (where most of the underground structures will be ‘tanked’ (effectively made water-tight)).

(i) Issues Raised by Submitters

Please include a brief summary of the key issues raised by submitters. If you refer to a particular submission please refer to the submission by number and not by the name of the submitter.

(ii) Response

See previous comment.

(iii) Question

Where your opinion(s) materially differ from the relevant circulated evidence statements, please briefly outline the difference and reasons for it.

(iv) Response

No differences.

(v) Question

Please discuss the magnitude, likelihood and significance of adverse and beneficial environmental effects.

(vi) Response

Covered elsewhere.

(vii) Question

Please address the adequacy of the proposed environmental management framework, including the proposed environmental performance requirements and environmental management measures contained in the EES, with reference to applicable legislation and policy.
(viii) **Response**
Covered elsewhere.

(ix) **Question**
Please address the adequacy of the impact assessment and whether the proposed environmental performance requirements are capable of being met.

(x) **Response**
Covered elsewhere.

(xi) **Question**
Please address the question of feasible modifications to the design of the Project within or reasonably proximate to the project boundary that could offer demonstrably overall superior outcomes.

Refer to comments on dewatering prediction under ‘Groundwater’.
9 Slope Instability Movements

Certain existing slopes or project excavation cuttings (even if supported locally by retaining walls or other forms of soil reinforcing systems) can sometimes result in noticeable surrounding ground movement (particularly lateral movement towards the excavation toe). The EES conducted separate instability estimates across these ground movement mechanisms for the Reference Design at identified areas where certain soil or rock formations posed a higher risk of ground movement (i.e., Reach 5).

(i) Issues Raised by Submitters

Please include a brief summary of the key issues raised by submitters. If you refer to a particular submission please refer to the submission by number and not by the name of the submitter.

(ii) Response

See previous.

(iii) Question

Where your opinion(s) materially differ from the relevant circulated evidence statements, please briefly outline the difference and reasons for it.

(iv) Response

See previous comments offered.

(v) Question

Please discuss the magnitude, likelihood and significance of adverse and beneficial environmental effects.

(vi) Response

Refer to other sections.

(vii) Question

Please address the adequacy of the proposed environmental management framework, including the proposed environmental performance requirements and environmental management measures contained in the EES, with reference to applicable legislation and policy.

(viii) Response

Covered elsewhere.
(ix) Question
Please address the adequacy of the impact assessment and whether the proposed environmental performance requirements are capable of being met.

(x) Response
Covered elsewhere.

(xi) Question
Please address the question of feasible modifications to the design of the Project within or reasonably proximate to the project boundary that could offer demonstrably overall superior outcomes.
Refer to previous comments.
10 Ground Movement Effects on Building & Infrastructure

The EES correctly acknowledges that buildings and structures will respond differently to various forms of ground movement, depending on their size, type of structural design, material, footing depth/style and general flexibility. Modern timber, steel, or steel-reinforced concrete structures can be quite flexible (tolerating a significant amount of ground movement). Older masonry types of buildings, particularly if set on a relatively shallow footing system to the supporting ground, can behave as a ‘brittle’ structure (and crack significantly). Deep footings for a building may also help to isolate it somewhat from ground movements, provided the base supports of these footings are sitting outside the main ground influence zone (as defined by depth from ground surface).

The EES correctly points out that many structures and buildings, particularly those set on shallow flexible footings, can be subject to seasonal ground movements that are known to arise from the seasonal shrinking and swelling of clay-type soils. In a similar vein, some structural movement can also be triggered by natural foundation/footing movements from other causes (such as thermal effects).

The impact assessment considered ground surface level estimates of movement made for both excavation-induced settlement (underground or open-cut) and potential consolidation settlement from dewatering. Various preliminary assessment inputs were considered which included:

- Interpreting the data on project geology and hydrogeology (from intrusive investigation and consideration of excavation history along the alignment). This set-up a ‘Conceptual Site Model’ to allow the estimation of preliminary input parameters for predictive ground movement models;

- Improved estimates were established for those influencing parameters related to ground surface movements to the proposed tunnelling techniques and determined geology/hydrogeology. This included numerical model use (such as models ‘XDisp’ (see below) and two-dimensional finite element numerical modelling). This allowed the prediction of likely volume loss amounts along the alignment, associated with proposed tunnelling methods and related ground surface propagation;

- Model: XDisp predicts ground movements resulting from various excavation types (subsurface or open):
  - For TBM tunnelling, predictions of tunnel face volume loss were made for the range of potential formation materials; and
  - Program XDisp then calculated, by way of a Gaussian-mathematical trough-pattern forming, estimated the ground settlement trough for the various formation materials along the alignment.

- Via the use of predictive hydrogeological modelling, predictions of groundwater inflows and resultant groundwater drawdowns from both tunnel construction and operation were made by the EES. This allowed estimates to be made for
consolidation settlement, for the various geological zones along the Project’s alignment from sediment depressurisation.

From the above inputs, predicted combined ground surface movements were grouped, assessed and considered using program ‘XDisp’. This model uses the combined outputs of predicted ground movements to assess for associated building damage (it is regularly used for ground movement appraisal on many world tunnelling projects). XDisp outputs of excavation induced settlement at the ground surface, were then assessed in combination with similar plots of predicted consolidation settlement, to provide a total prediction of ground movement as a potential ZOI. This zone was assumed to apply where there is a 5 mm or greater project-induced ground surface vertical settlement predicted.

The EES indicates as a general ‘rule’ that previous tunnelling experience worldwide suggests that where structures and buildings subject to settlement effects smaller than the 5 mm threshold, there is negligible effect of the movement on the structure.

- **First stage risk assessment process** – if a ‘slight’ risk to damage or distress (buildings/aboveground infrastructure) or greater by:
  - Predicted 5 mm vertical total movement from the Project influence;
  - Predicted strain at 1:500 (0.2 %);
  - For underground utilities, a slightly different appraisal system was adopted based on relative stiffness of service to surrounding soil;
  - Consider is there is any ‘rigid’ pipe – joint rotation or joint pull-out risk; or
  - Considering predicted critical bending and extensional strains expected on the utility line, if the asset is thought to act more like a flexible member (i.e., then considering allowable strains based on the assumed service line material type).

- **Any identified buildings or structures of slight risk or greater, or of heritage value, or known community significance** went straight into a Stage 2 Risk Appraisal. The second stage risk assessment to buildings and structures considered:
  - Specifics of geometry and relative stiffness of the structure; and
  - Elastic modelling (beam movement adopted on the assumed Gaussian subgrade shape).

- **If determined tensile strain from the above review is determined as ‘moderate’ risk or higher, as estimated strain approaches 0.15 % to 0.3 %, then the evaluation process moves to Stage 3 - Detailed Evaluation.**

All residential properties which fall within the ZOI from the ground movement trough modelling are to be subject to a per-project works condition survey.
(i) Issues Raised by Submitters

Please include a brief summary of the key issues raised by submitters. If you refer to a particular submission please refer to the submission by number and not by the name of the submitter.

(ii) Response

See previous.

(iii) Question

Where your opinion(s) materially differ from the relevant circulated evidence statements, please briefly outline the difference and reasons for it.

(iv) Response

No differences, except that the impact of horizontal stress state should be better considered when defining a theoretical tough of settlement for the nomination of buildings/infrastructure that will be subject to an up-front condition survey. The current method in the EES underestimates the width of the settlement trough.

(v) Question

Please discuss the magnitude, likelihood and significance of adverse and beneficial environmental effects.

(vi) Response

See elsewhere.

(vii) Question

Please address the adequacy of the proposed environmental management framework, including the proposed environmental performance requirements and environmental management measures contained in the EES, with reference to applicable legislation and policy.

(viii) Response

See elsewhere.

(ix) Question

Please address the adequacy of the impact assessment and whether the proposed environmental performance requirements are capable of being met.

(x) Response

See elsewhere.
(xi) Question

Please address the question of feasible modifications to the design of the Project within or reasonably proximate to the project boundary that could offer demonstrably overall superior outcomes.

See elsewhere.
11 Project Risk Profiles

Risk Quantification Approach

The risk assessment assigned five key risk rankings related to ground movement risk (‘Negligible’, ‘Minor’, ‘Moderate’, ‘Major’ and ‘Severe’). The risk assessment process generally followed that described within AS/NZ ISO 31000:2009, where a combination of risk ‘likelihood’ rating was matched with ‘consequence’ of the risk being studied. The risk assessment suitably considered geological conditions and preliminary modelling predictions were made which some degree of conservatism for ground movements. The models provided as an output predicted distribution patterns of movements and their anticipated effects on buildings and civil infrastructure.

Risk profiles in the EES have been presented in relation to 11 nominated study ‘Reaches’ of the project:

Reach 1: M80 Ring Road to Watsonia Railway Station

Involves mainly surface works at the M80 Ring Road/Greensborough Bypass intersection with Greensborough Highway, where road carriageways start their cut into the ground surface near Watsonia Railway Station. Further lane widening earthworks are to be placed on existing road embankments for the M80 Ring Road and Greensborough Bypass, posing some extra stress on existing underground services. Minimal ground movements are anticipated. Site geology through the Reach consists of thin residual clays over highly weathered Silurian siltstone with a deeply weathered profile down to 15 m to 25 m bgs. Features of note include:

- Part of the Maroondah Viaduct passing beneath the M80 Ring Road, near Chappell Drive (2.16 m diameter, concrete outer-lined steel water supply line built in the 1970s). This was rated a ‘slight risk’ with 16 mm maximum vertical settlement predicted, where the pipeline is expected to act in a flexible manner) – Currently EPRs: GM1/GM2 recommended modelling and monitoring;
- Former backfilled quarry site (waste clay and brick in-fill up to 7 m bgs, mixed with solid inert waste (SIW)) at the M80 Ring Road/Greensborough Bypass intersection; and
- A.K. Lines Reserve/Oval at Watsonia (former landfill site that operated across 1950’s to mid-1960’s).

Reach 2: Watsonia Railway Station to Northern Tunnel Portal

This Reach involves general open cut trenching which will extend to a maximum depth of 13 m bgs. Several cross-connecting viaduct or bridge-type overhead structures are also proposed. Site geology consists of extremely to highly weathered Silurian siltstone/sandstone. Key risk features of include:

- The Hurstbridge Metropolitan Rail Line is situated within 30 m to the west of the proposed road trench (runs longitudinally to the proposed alignment in this segment). No significant ground risk is considered to apply;
The alignment passes close to the west boundary of the Simpson Army Barracks (near Yallambie Road), where an L-shaped building and associated Outbuilding may be influenced from ground movement. The L-shaped Building is considered to not be at any risk, but EPRs: GM1/GM2 and GM3) are recommended. The Outbuilding was deemed a ‘slight risk’ due to proximity of the project excavation and this building’s construction type (masonry wall. EPRs: GM1/GM2 and GM3 have been recommended for this structure; and

- Elder Street underground gas main (0.45 diameter, high pressure, welded steel circular pipe) which runs longitudinal to the road trench. This pipe is set at 10 m bgs, within a 0.6 m diameter outer conduit. The adjoining 4 m bgs project trench excavation at Greensborough Road (an 18 mm maximum vertical settlement is predicted) results in a recommendation to model and monitor (EPRs: GM1/GM2).

Reach 3: Northern Tunnel Portal to Lower Plenty Road

This Reach involves 1.4 km of cut and cover construction, which will extend to a maximum depth of 35 m bgs at the tunnel portal. Site geology consists of extremely to highly weathered Silurian siltstone/sandstone. It is likely that there is a steeply dipping fault zone, just north of Lower Plenty Road. Superimposed ground movements from the cut and cover and adjoining TBM tunnel interface are expected to be significant at this area (bedding plane strains and fault line influence). Key risk features include:

- Zone of influence (ZOI: > 5 mm vertical influence) expected to extend laterally from the cut by up to 45 m (most influence is near the north portal);
- Borlase Reserve, which was a former small landfill site (earthworks and SIW to 5 m deep circa 1966 to 1972);
- Low-rise residential properties within the settlement zone of influence (ZOI);
- Greensborough Road water supply main (0.6 m diameter concrete lined welded steel pipe) on west side of Lower Plenty Road (maximum predicted vertical pipe displacement of 7 mm, with no significant change in ground slope); and
- Yan Yean – Surrey Hills water supply mains (at north side of Drysdale Street - set of three concrete lined welded steel pipelines ranging in size from 0.375 m to 1.35 m diameter) and associated water pressure reducing station. Ground movement of up to 76 mm (maximum vertical movement) are predicted for this area. Requires Stage 2 Assessment (not yet done). Current EPR recommendation are: GM1/GM2 and GM4 (pipeline re-alignment).

Reach 4: Lower Plenty Road to Banyule Flats

This Reach involves placement of twin TBM cut tunnels (each at 15.7 m diameter, spaced at 16 m wall to wall) which will extend (tunnel invert) to a maximum depth of 42 m bgs. Smaller mined cross-tunnels will inter-connect the main tunnels every 120 m (for emergency evacuation of persons). Site geology consists of slightly weathered to fresh siltstone/sandstone with minor dyke ingress and fault zones. Key risk features include:
• Low-rise residential properties within the ZOI extends out some 25 m from the tunnels centreline. Many one to two story residential premises above warranted Stage 2 Assessment:
  o ‘Lower Plenty Road 2 (LPR2)’: Near the Lower Plenty Road Portal cut and cover & TBM interface (between 5 mm to 29 mm vertical structural movement matched to a ‘slight’ damage rating – recommended to deploy EPRs: GM1/GM2/GM3);
  o ‘Banyule Creek 1 (BC1)’: Low rise across TBM alignment, includes Viewbank Aged Care (all approximately 10 mm to 12 mm vertical movement, matched to a ‘negligible’ damage rating – recommended to deploy EPRs: GM1/GM2/GM3); and
  o ‘Banyule Creek 2 (BC2)’: Low rise residential and Goodstart Kindergarten (all approximately 10 mm to 12 mm vertical movement), matched to a ‘negligible’ damage rating – recommended to deploy EPRs: GM1/GM2/GM3).

• Banyule Creek sewer (0.375 m to 0.45 m diameter, vitreous clay sewer, follows Banyule Creek, constructed in 1963). This relatively shallow depth line passes above the TBM alignment at Banyule Flats (there is up to 36 mm maximum vertical movement predicted from the northbound tunnel) – warranted a Stage 2 Assessment. Stage 2 modelling prediction suggested that further assessment was not warranted. It has been recommended to deploy EPRs: GM1 and GM2.

Reach 5: Banyule Flats, Banksia Park, Yarra River

This Reach involves placement of twin TBMs, driven to the north from a temporary portal at Banksia Street. Site geology through the Reach consists of mainly slightly weathered to fresh siltstone, however some moderately weathered siltstone is also present near the tunnel crown at certain locations. Near the adjoining Manningham Road Interchange (MRI) which is to be of cut and cover construction, there is the possibility that the tunnel crown may intersect the base of ancient alluvium. Key risk features include:

• Variable conditions likely to be encountered. There are thick alluvial sediments (15 m thick) overlying the TBM tunnels, which could respond with consolidation settlement if significant dewatering occurs from tunnelling activity. There are also several suspected faults present in the area;

• Zone of > 5 mm vertical ZOI extends out some 38 m from the tunnel centreline;

• A 26o to 36o degree from horizontal (FH) northern riverbank slope at Banyule Flats (near the Banyule Homestead (the ‘North Slope’)). At this area, siltstone was estimated to have an approximate 65o FH rock bedding dip into the slope, with planar jointing (approximately orthogonal to the rock primary bedding, dipping out of the slope), forming a ‘blocky’ rock mass. There is evidence the current slope angle is controlled by this rock block arrangement, where the observed slope face angle closely matches the orthogonal jointing angle emerging from the face (between 25° to 30° FH). Tunnelling influence for this feature was assessed with finite element
analysis (FEA). This predicted minimal strain effect for this area (minimal ground movement risk); and

- The building of Banyule Homestead is estimated to be outside the tunnelling ZOI;
- Banyule Swamp is a 7 Ha shallow water body, with a controlled outfall structure on its west side that may be prone to differential ground movement effects as it is near the TBM alignment. Up to 35 mm vertical movement is predicted. EPRs: GM1/GM2/GM3/GM4 are all recommended;
- Heide Museum of Modern Art has two external large sculptures which are rated at slight risk: ‘Theoretical Matter’ (24 mm maximum vertical movement predicted) and ‘Crescent House’ (18 mm maximum vertical movement predicted). It is recommended to deploy EPRs: GM1/GM2/GM3 across both sculptures;
- The Manningham Council-owned public sculpture ‘Helmet’ situated just north of Bridge Street. This is a large, steel-framed sculpture (which may be subject to up to 93 mm max vertical movement from tunnelling) – posing a significant risk. The outcome of a Detailed Assessment was to recommend EPRs: GM1/GM2/GM3/GM4 with an associated detailed structural analysis around predicted ground movements; and
- A single storey residence just west of the ‘Journey’s End’ heritage listed residence (22 to 40 Bridge Street), may be subject to ground movement from two nearby planned TBM-retrieval excavation shafts in Banksia Park.

Reach 6: Manningham Road Interchange, Bulleen

Involves cut and cover across an existing industrial and commercial land use area, to place a large interchange ‘box’ structure to a maximum placement depth of 22 m bgs. Site geological studies suggest that the Yarra River has eroded the toe of east facing siltstone plateau (some 10 high, where the edge slope is approximately 20O FH). Siltstone is generally moderately to extremely weathered. The siltstone is overlain in certain areas by old east to west ‘fingers’ of Pleistocene paleochannels (holding Brighton Group ferruginised sands). Key risk features include:

- Yarra Flats heritage protected area, which holds thick deposits of soft alluvial soils, prone to consolidation;
- Banksia Street Pipe Bridge, which is a supporting ‘Warren Truss’ style side bridge next (north) to the Manningham Road Bridge that supports an 0.8 m diameter wrought iron water supply conduit;
- Yarra East Main Sewer which runs along Bulleen Road (1.75 m diameter reinforced concrete). This lien is proposed to be relocated for the project (Bulleen Road Diversion);
- ‘MRI Slope’ lies in an area where the Yarra River is within 35 m of the closest cut and cover. This 10 m high slope is estimated at 20o degree FH (just west of Manningham Road) an may be subject to some disturbance. Initial slope stability analysis suggested no significant risk, where EPR recommendation GM1 then applies; and
• The MRI retention structures are expected to cause groundwater mounding (potentially up to a 6 m height increase, east of the box structure). This could result in some increased ground heave associated with the ancient alluvium (reactive clayey sand/sandy clay) – EPR recommendation GM1 therefore applies.

Reach 7: Avon Street to Rocklea Road (SEM Tunnels)

Involves the mined caverns (road ramps near Golden Way) and sequentially mined excavations forming the twin tunnels across this short intermediate tunnel section. Maximum surface cover depth is of the order of 35 m bgs, where the SEM tunnels are to be of general individual dimension: 14 m wide by 12.5 m high. Site geology at the area of Ilma Court (overlying low-rise residential) shows extremely to highly weathered siltstone, but with in-filled paleochannels (Brighton Group sands) near the tunnel crown level that may result in changes to expected mining conditions. Associated alluvium nearby is also up to 8 m thick. There are also faults in the area and weathered dyke features. Key risk features are:

• These openings are expected to have more issues encountered, related to requirements for tunnelling supports and related ground movements;

• These openings will be constructed as a drained tunnel initially, followed by placements of primary and secondary reinforced concrete tunnel wall seals. As such more significant aquifer depressurisation will occur at these areas, causing higher consolidation settlement;

• Predicted ZOI up to 25 m from the tunnel centreline influencing the mainly low-rise, Bulleen Residential area (‘BR’) - St Andrews Crescent, Golden Way, Claremont Ave, Rocklea Road, Killara Mews, where up to 26 mm maximum vertical settlement has been indicated (critical locations are thought to be on the east and west side of the tunnel alignment). Damage rating expectation is ‘very slight’. It is currently recommended to enact EPRs: GM1/GM2/GM3; and

• There is a nearby former brick mining pit (Yarraleen Place), which has been infilled for a residential development. These low-rise residences are expected to be sitting of deep piled supports to the siltstone basement rock. The tunnelling passes between this quarry and nearby alluvium.

Reach 8: Rocklea Road to Bulleen Oval

Involves placement of a cut and cover tunnel to ramp-connect to Bulleen Road. Site geology shows a deeper (21 m thick) alluvium layer and significant faulting within the underlying siltstone (typically slightly weathered to fresh rock). Bulleen Park is a former landfill site (constructed across the 1960’s). Key risk features include:

• Associated groundwater drawdowns are expected to cause more noticeable settlements for this Reach. There is up to a 80 m predicted ZOI from the alignment centreline, but the required Bulleen Road Sewer relocation works, increases the ZOI out to 110 m;

• The Bolin Bolin Billabong Is not considered at risk from ground movements;
• The Bolin Bolin Integrated Water Facility a wetland water storage lake recently constructed by Manningham Council – some 50 m from the proposed cut and cover excavation. This not considered to be at risk from ground movement;

• Veneto Club (Italian Social Club located at 191 Bulleen Road) – 100 m offset from the proposed excavation. This is a three-storey concrete framed building (constructed in the 1960’s), with a large, east-facing box-girder arched portico and nearby masonry plinth sculpture. For these features geotechnical analysis was taken to Stage 2 Assessment to ally community concerns (these features were not considered to be at risk from this conducted analysis). There is a recommendation to deploy EPRs: GM1/GM2 and GM3;

• The Bulleen Road Sewer is required to be diverted, using underground pipe-jacking and trenching methods through the Trinity Grammar playing fields. Checks should be made by the IAC, to confirm that with this ‘Early Works’ EPRs: GM1/GM2/GM3 are to be deployed;

• Marcellin College Playing Fields and associated shallow water storage lake are considered at minor risk from ground disturbance; and

• Bullen Oval has significant deposits of uncontrolled fill (with some unknown amount of asbestos mixed with the waste).

Reach 9: Eastern Freeway to Bullen Road

Involves mainly surface works, with lane widening and construction of a viaduct ramp, where the Bulleen Swim Centre and Boroondara Tennis Centre are to be acquired for construction. Site geology involves river alluvium over thin residual clay and then highly weathered siltstone. Key risk features include:

• Required relocation of the East Yarra Sewer Main (2.25 m diameter at depth of 5 m to 7 m bgs, that branches from the Bulleen Road West Sewer). This is expected to require the placement of shaft excavations for pipe jacking; and

• Sensitive receptor is the Bulleen Road West Sewer (also a 2.25 m diameter reinforced concrete pipe asset of Melbourne Water). This sewer line sits on alluvial clay and will be subject to increased loads from the new surface road ramps and nearby settlement influence from the shaft excavation to relocate the East Yarra Sewer Main (maximum ground displacement of 27 mm vertical has been predicted). A Stage 2 Assessment considering pipe joint rotation and pull-out risk found it was not at risk. It was recommended to deploy EPRs: GW1/GM2 and GM3.

Reach 10: Eastern Freeway-West

Involves no significant additional surface loads, and there are no significant nearby sensitive receptors. The area is underlain by undifferentiated river alluvium, transitioning across to basalt rock (Quaternary Newer Volcanics) when moving to the west along the Freeway. The main risk feature is the former landfill (Camberwell Municipal Landfill), constructed between 1966 to 1977, at the area of Musca Reserve and Freeway Golf (the existing Freeway is likely to have been built over a portion of this landfill area).
Reach 11: Eastern Freeway-East

Involves some significant road widening of the Freeway. Site geology transitions from younger Silurian siltstone bedrock into older Anderson Creek Formation siltstone. There is some amount of minor river alluvium associated with Koonung Creek which crosses under the Freeway. Key risk features include:

- There is a former landfill located near the intersection of Doncaster Road and the Freeway, Banyule North (suspected to have been completed around 1977 to 1978);
- The Koonung Creek Conduit is a below ground conduit placed for Freeway construction to direct Koonung Creek (which is a tributary to the Yarra River). At this conduit, a three-pin reinforced concrete ‘BEBO’ type arch (4 m high by 6.6 m wide) sits on a concrete slab to form the crossing void. NELP proposes to place a new spanning and load-supporting concrete slab across the top of this existing structural feature. Whilst the structure is not considered to be at risk, it is conservatively recommended to deploy EPRs: GM1/GM2 and GM3 on the arch; and
- The Kennet Street Water Main is a 1.2 m diameter enamel lined, steel water supply pipeline, which was installed in 1957. This Melbourne Water-owned asset will be subject to additional stresses from lane widening, where it sits atop clay alluvium up to 18 m deep, with a 3.2 m ground cover to work. Additional geotechnical assessment shows no significant risk (only to 7 mm maximum vertical settlement).

Alternative Design Options Being Considered to Reference Design

Several alternative design options are mentioned within the EES:

- MRI: Involving lowering of the alignment of the TBM tunnels and the MRI by a further 2 m:
  - Such an action is predicted to result in a 3 m increase in the predicted surrounding settlement trough width;
  - Maximum vertical settlements may increase by a further 3 mm; and
  - Structural damage ratings should not alter to the surrounding receptors.
- Northern TBM Launch Option: This is not expected to cause a large change to the excavated tunnel geometry and its related ground movement effects.
- Banksia Park TBM Retrieval Shafts:
  - There is an alternative proposal for two, separate TBM retrieval shafts to be located on the north side of Bridge Street, Banksia Park;
  - These shafts (which would be constructed before the MRI construction) are estimated to be of approximate plan size: 25 m wide x 50 m long and will be between 30 m to 34 m bgs;
  - The EES considered consolidation effects from soft soils at this area from the shaft dewatering operations (dewatering expected to occur 9 months before nearby MRI dewatering commences); and
The additional ground movement impact from this option would include one additional low-rise residence within the ZOI (on Bridge Street) – which has been assigned a ‘slight’ risk ranking sits on 10 mm predicted vertical settlement contour). Stage 2 Assessment on this residence indicated a revised ‘negligible’ risk ranking. It was recommended to deploy EPRs: GM1/GM2 and GM3.

(i) **Issues Raised by Submitters**

Please include a brief summary of the key issues raised by submitters. If you refer to a particular submission please refer to the submission by number and not by the name of the submitter.

(ii) **Response**

See previous.

(iii) **Question**

Where your opinion(s) materially differ from the relevant circulated evidence statements, please briefly outline the difference and reasons for it.

(iv) **Response**

No significant differences.

(v) **Question**

Please discuss the magnitude, likelihood and significance of adverse and beneficial environmental effects.

(vi) **Response**

Covered elsewhere.

(vii) **Question**

Please address the adequacy of the proposed environmental management framework, including the proposed environmental performance requirements and environmental management measures contained in the EES, with reference to applicable legislation and policy.

(viii) **Response**

Covered elsewhere.

(ix) **Question**

Please address the adequacy of the impact assessment and whether the proposed environmental performance requirements are capable of being met.
(x) **Response**

The impact assessment process is suitable.

(xi) **Question**

Please address the question of feasible modifications to the design of the Project within or reasonably proximate to the project boundary that could offer demonstrably overall superior outcomes.

See previous comments.
12 Comment Across EPRs

Generally the as-proposed EPRs from the EES seem suitably thought out, robust and well-constructed:

- ‘GM1’ relates to the Site Conceptual Model and understanding of the ground movement mechanism (Assessment);
- ‘GM2’ relates to measuring a ground movement ‘baseline’ and conducting careful monitoring checks across the Project life into Operation (there is a Plan for Movement Monitoring);
- ‘GM3’ relates to establishing a baseline of building and infrastructure asset condition before construction, followed post-construction surveys and consultation with the stakeholders of these assets; and
- ‘GM4’ relates to rectifying damage to buildings and infrastructure. Its noted that the Councils are currently not listed in GM4 (where some assets may be listed as a heritage item by certain Councils).

General policing of the Ground Movement EPRs is a key question – where is the presence of an independent Auditor in the process, which may occur before any independent mediation is called up as a requirement? The independent Auditor must either be suitably trained or team-supported in the identification and understanding of ground movements and soil-structure interactions.

(i) Issues Raised by Submitters

Please include a brief summary of the key issues raised by submitters. If you refer to a particular submission please refer to the submission by number and not by the name of the submitter.

(ii) Response

See previous.

(iii) Question

Where your opinion(s) materially differ from the relevant circulated evidence statements, please briefly outline the difference and reasons for it.

(iv) Response

No differences.
(v) Question
Please discuss the magnitude, likelihood and significance of adverse and beneficial environmental effects.

(vi) Response
Covered elsewhere.

(vii) Question
Please address the adequacy of the proposed environmental management framework, including the proposed environmental performance requirements and environmental management measures contained in the EES, with reference to applicable legislation and policy.

(viii) Response
See above.

(ix) Question
Please address the adequacy of the impact assessment and whether the proposed environmental performance requirements are capable of being met.

(x) Response
See above.

(xi) Question
Please address the question of feasible modifications to the design of the Project within or reasonably proximate to the project boundary that could offer demonstrably overall superior outcomes.
Not relevant to this section.
13 Approval Documents

(i) Question
Please list any recommended changes to the approval documents.

- Add Statutory Independent Environmental Auditor – with suitable geotechnical team backing.

(ii) Response
No significant changes.