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# Assessment of stability and ground movement associated with tunnelling under a major highway

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## ABSTRACT

A major component of Sydney's South West Rail Link project is the Hume Highway Underpass, an 80m long twin track driven tunnel excavated at a depth of just 5 m below the pavement of the Hume Highway, one of Australia's busiest road corridors. The underpass was planned to control risk to the highway and its users, and was excavated without any disruption to the Hume Highway traffic. Tunnel stability and effects on the overlying highway pavement were extensively analysed during design development, and ground movements were continuously monitored during excavation.

## 1 INTRODUCTION

Sydney's South West Rail Link (SWRL) project involves the design and construction of a new 11km twin track passenger rail line from Glenfield to Leppington via Edmondson Park, and a Transport Interchange and major upgrade at Glenfield Station. The Glenfield to Leppington Rail Line (GLRL) component of the SWRL is being constructed by the John Holland Group under a D&C contract. SMEC Australia was engaged by John Holland to undertake the civil works design for the GLRL.

A critical component of the SWRL project is the crossing of the Hume Highway, the main highway link south of Sydney and one of Australia's busiest road corridors. The solution adopted for the crossing was an underpass comprising an 80 m long mined tunnel excavated at a depth of just 5 m below the pavement of the two existing 4 lane carriageways of the Hume Highway. (Refer Figure 1).

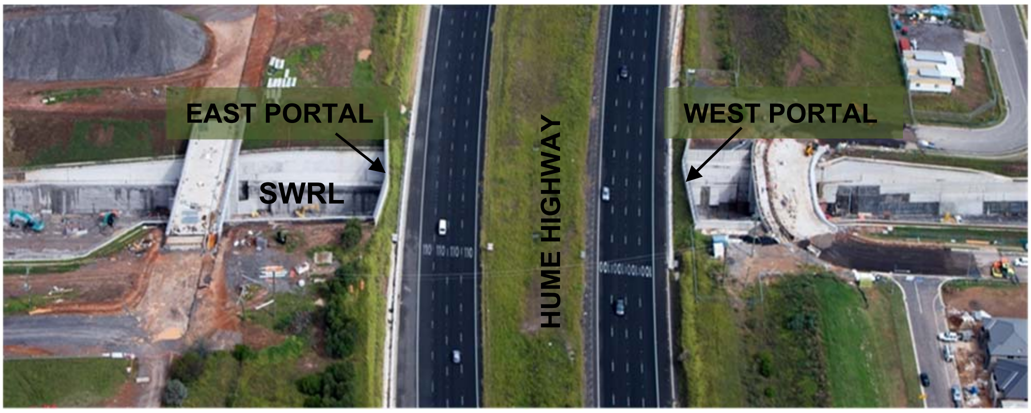



*Figure 1. Hume Highway underpass layout*

Due to the limited rock cover between the underpass crown and the Hume Highway pavements, design and construction of the underpass required specific focus on the prediction, limitation and control of ground movements in order to provide a safe project execution and mitigate risks presented to the Hume Highway users. A comprehensive monitoring system was installed during the works to measure the actual movements encountered so that the design predictions could be confirmed and measures incorporated should deformations be higher than anticipated.

The underpass was successfully constructed during 2012 and early 2013.

## 2 GEOLOGY

Subsurface conditions at the underpass location were interpreted for detail design based on 55 boreholes of varying depth. A representation of the geological long section through the underpass is included in Figure 2.

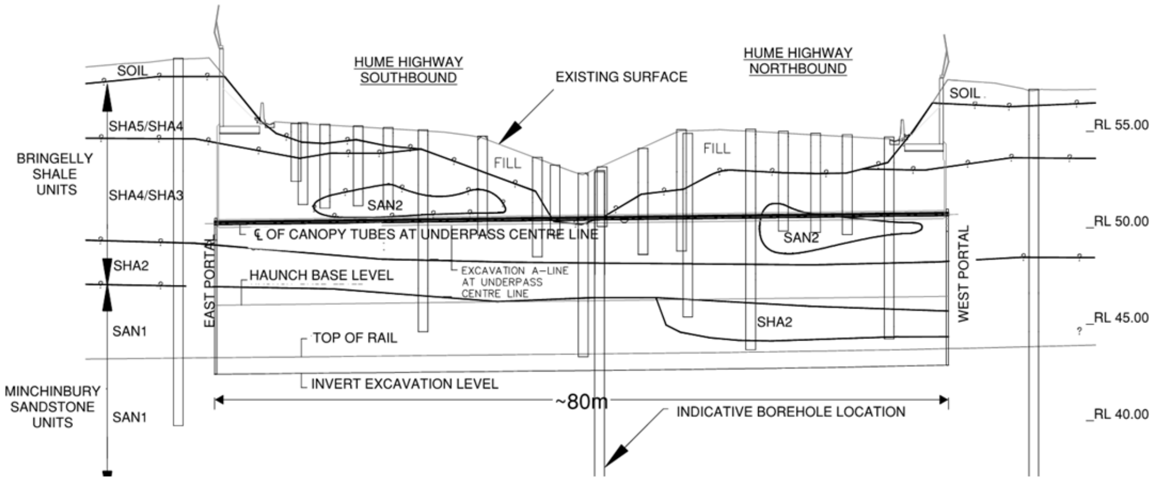



*Figure 2. Geological profile along underpass*

At the location of the underpass, the natural surface each side of the highway is around 58 AHD in elevation and this is likely to have been the existing surface level prior to construction of the highway, which is in a cutting at the underpass location. Top soil and residual clay soils of depth in the order of 1m overlay Bringelly Shale which is highly weathered at its upper horizons and gradually increases in strength as weathering reduces with depth. The Bringelly Shale generally consists of beds of siltstone and interlaminated sandstone/siltstone. Below the Bringelly Shale, the Minchinbury Sandstone unit is of a thickness in the order of 10m and extends to well below the underpass invert before transition to the underlying Ashfield Shale unit. The Minchinbury Sandstone unit generally consists of thickly bedded slightly weathered to fresh sandstone and siltstone.

The ground water level from geotechnical investigations was located at the interface between the Bringelly Shale and Minchinbury Sandstone, around the crown arch base level of the underpass.

### 2.1 Rock Classification System

Rock has been split into two types i.e. sandstone (SAN) and shale (SHA), and classified similar to the system used by Pells et.al. (1998). This split accounts for the difference in engineering properties between the more massive sandstones relative to the shale consisting of siltstone and interbedded rock types. In accordance with past experience in similar rock conditions, the rock types of the site were grouped into seven classes as shown in *Table 1*.

*Table 1 : Adopted rock classification system*

| Class | Strength | UCS (MPa) | Defect spacing (mm) | Weathering |
|---|---|---|---|---|
| **SAN 1** | High to Extremely High | 24-240 | 300-1000 | Fresh |
| **SAN 2** | High | 24-72 | 100-300 | Slightly Weathered |
| **SHA1** | High | 16–48 | 300-1000 | Fresh |
| **SHA 2** | High | 16-48 | 100-300 | Slightly Weathered |
| **SHA 3** | Moderate | 5-16 | 30-100 | Moderately Weathered |
| **SHA 4** | Low | 1–5 | 10-30 | Highly Weathered |
| **SHA 5** | Very Low | <1 | < 10 | Extremely Weathered |

### 2.2 Rock Defects

Based on the three nearby boreholes where the portable Borehole Image Processing System (BIPS) was used, bedding and one main joint set were identified through the underpass in the Minchinbury Sandstone. Their orientations and dip angles suggested that they do not form wedges, nor planar failures (adverse joint) on the underpass side walls and tunnel face. Hence, the stability assessments on the underpass side walls are undertaken on the basis of mass parameters and do not include the effect of adverse joints on the side walls.

### 2.3 Geotechnical Design Parameters

RocLab 1.0 was used to determine rock mass strength parameters based on the latest version of the generalised Hoek-Brown failure criterion. The GSI for each rock mass unit was quantitatively assessed and used to estimate the rock mass modulus, generalised Hoek-Brown and Mohr-Coulomb parameters. GSI values have been estimated using the procedures outlined in Cai et. al. (2004), for each rock mass unit encountered along the Hume Highway Underpass alignment. The Hoek-Brown criterion also served to derive the Mohr-Coulomb parameters cohesion ‘c’ and angle of friction ‘$\phi$’ required for PLAXIS 3D Tunnel modelling for both the lower bound and upper bound stress conditions. A summary of the design parameters are presented in table below.

*Table 2: Summary of inputs and adopted rock mass parameters*

<table>
<tr><th>Source</th><th>Symbol</th><th>SHA5 [3]</th><th>SHA4 [3]</th><th>SHA 3</th><th>SHA 2</th><th>SHA 1</th><th>SAN 2</th><th>SAN 1</th></tr>
<tr><td rowspan="7">Input Parameters</td><td>GSI</td><td rowspan="7">-</td><td rowspan="7">-</td><td>43</td><td>51</td><td>51</td><td>65</td><td>69</td></tr>
<tr><td>UCS - Intact (MPa)</td><td>11.0</td><td>18</td><td>25</td><td>28</td><td>42</td></tr>
<tr><td>$m_i$</td><td>6</td><td>6</td><td>6</td><td>17</td><td>17</td></tr>
<tr><td>MR</td><td>200</td><td>200</td><td>200</td><td>275</td><td>275</td></tr>
<tr><td>Overburden (m)</td><td>7</td><td>11</td><td>23</td><td>17</td><td>17</td></tr>
<tr><td>$\sigma_{3max}$ (MPa)(1/ 2)</td><td>0.08/ 0.3</td><td>0.13/ 0.35</td><td>0.27 /0.57</td><td>0.21/ 0.85</td><td>0.22 /0.87</td></tr>
<tr><td>D</td><td>0</td><td>0</td><td>0</td><td>0</td><td>0</td></tr>
<tr><td rowspan="7">Calculated Parameters</td><td>c (kPa)(1/ 2)</td><td>30</td><td>50</td><td>73/ 120</td><td>180/ 230</td><td>600/ 640</td><td>440/ 640</td><td>800/ 1000</td></tr>
<tr><td>$\phi$ (degree)(1/2)</td><td>28</td><td>28</td><td>51/41</td><td>52/46</td><td>51/47</td><td>63/56</td><td>62/59</td></tr>
<tr><td>$E_m$ (MPa)</td><td>80</td><td>150</td><td>430</td><td>1174</td><td>3158</td><td>4864</td><td>8244</td></tr>
<tr><td>$m_b$</td><td>-</td><td>-</td><td>0.756</td><td>1.043</td><td>1.719</td><td>4.871</td><td>5.619</td></tr>
<tr><td>s</td><td>-</td><td>-</td><td>0.0016</td><td>0.004</td><td>0.020</td><td>0.020</td><td>0.03</td></tr>
<tr><td>a</td><td>-</td><td>-</td><td>0.51</td><td>0.505</td><td>0.52</td><td>0.52</td><td>0.50</td></tr>
<tr><td>Tensile Strength (MPa)</td><td>-</td><td>-</td><td>0.025</td><td>0.075</td><td>0.30</td><td>0.12</td><td>0.24</td></tr>
<tr><td colspan="9">1: Lower bound stress derived using overburden stress.2: Upper bound stress derived using the assumed major horizontal stress. 3: No assessment made due to data limitation. Values used for both stress conditions.<br>GSI= Geological Strength Index, UCS= Unconfined Compressive Strength, $m_i$= Material’s constant,<br>$\sigma_{3max}$ = In situ confining stress condition, c= Cohesion, $\phi$ = Friction angle of rock mass, $E_m$= Young’s Modulus of rock mass, $m_b$, s and a are material constants for the rock mass for Hoek Brown failure criteria.</td></tr>
</table>

### 2.4 In situ Stress

The existence of high horizontal stress fields in the Sydney Basin sedimentary rocks has been appreciated for many years. Previously published papers provide details of the in situ stresses within the Triassic rocks based on testing and monitoring of excavations within the Hawkesbury Sandstone and Ashfield Shale units, however these papers do not consider the Minchinbury Sandstone and Bringelly Shale that are higher in the geological formation and are encountered along the underpass alignment.

No project specific in situ ground stress measurements were available for the project detail design. The stress history and resulting locked-in stresses in the Minchinbury Sandstone and Bringelly Shale were therefore estimated based on engineering judgement by considering the strength and jointing of the materials in comparison to the Sydney basin units with stress data readily available. Upper bound and lower bound in situ stress conditions were adopted for detail design as follows:

*Table 3: Adopted in situ stress*

| Geological Unit | Upper Bound Horizontal Stress / Orientation | | Lower Bound |
|---|---|---|---|
| | Major / (North-South) | Minor / (East-West) | |
| **Minchinbury Sandstone** | $\sigma H = 1.0$ MPa $+ 1.5\sigma v$ | $\sigma H/\sigma h = 1.6$ | $\sigma H = \sigma h = \sigma v$ |
| **Bringelly Shale** | $\sigma H = 0.5$ MPa $+ 1.5\sigma v$ | $\sigma H/\sigma h = 1.4$ | $\sigma H = \sigma h = \sigma v$ |
| $\sigma H$= Major horizontal stress, $\sigma h$ = Minor horizontal stress, $\sigma v$ = Overburden stress | | | |

The Hume Highway Underpass alignment is oriented approximately in an east-west direction. This is nearly perpendicular to the major horizontal stress trend of Sydney Basin, therefore it was assumed that the underpass cross section is likely to incur the major horizontal stress.

### 2.5 Subsurface Conditions Encountered by the Tunnel

A series of cross sections developed based on the available borehole information indicated reasonably consistent geological profile along the underpass. The main geological units anticipated in the horizon of the underpass were rock classes varying from SHA5 to SHA2 for the Bringelly Shale in the crown, and rock class SAN1 to SAN2 from Minchinbury Sandstone in the side walls and invert of the underpass. At the western end of the underpass it is anticipated that the contact between Bringelly Shale and Minchinbury Sandstone will fall below the base level of the arch into the upper portion of the side walls.

Beneath the highway carriageways the cover was limited to approximately 5m, consisting of around 3m of rock and 2m of fill and payment layers. No rock cover was predicted in the median, with the underpass crown predicted to clash with the highway median drain and surrounding fill.

## 3 ROAD OPERATION REQUIREMENTS

The underpass design catered for multiple road operation requirements relating to the Hume Highway which were set by the operator Roads and Maritime Services (RMS). The key road operation requirements considered during the development of the excavation and support design are summarised in Table 4.

*Table 4: Key design requirements*

| Description | Requirement |
|---|---|
| **Settlement** | Maximum permitted settlement of the road pavement of 20 mm, maximum permitted pavement distortion of 5 mm over 2 m in any direction. |
| **Road Operation** | Disruptions to road operations and speed restrictions other than as an emergency response were not permitted. |
| **Settlement Impact Assessment** | A settlement Impact Assessment was required to detail the impacts of the underpass excavation on the existing pavements. |

## 4 UNDERPASS EXCAVATION AND PRIMARY SUPPORT

In recognition of the major risk being excessive settlement of the Hume Highway carriageways during the underpass construction, the underpass primary support design methodology was developed following a detailed risk assessment, including a review of the existing ground conditions and alternative design and construction methods. The goal of the designed excavation sequence and support system was to:

- Ensure stability of the underpass under all stages of excavation, construction and the design life; and
- Limit settlement of the Hume Highway carriageways to ensure safe transit for vehicles using the highway at the sign posted speeds.

To control ground settlements and minimise impacts on the Hume Highway, the underpass crown primary support included full length large diameter steel canopy tubes installed prior to the mined excavation and support of the underpass. As the underpass excavation advanced beneath the canopy tubes, these were supported by steel sets and infill shotcrete. The detailed design and analysis of the underpass excavation and support was undertaken with sophisticated 2D and 3D numerical modelling which incorporated this excavation and support staging.

### 4.1 Canopy Tubes and Ground Treatment

A system of pre-excavation ground support was implemented in the form of large diameter 80 m long grout filled steel canopy tubes pre-installed around the crown arch profile of the underpass. Nine 406 mm diameter CHS canopy tubes with a radial centre to centre spacing of 1200 mm were specified to be installed using laser guided micro tunnelling techniques. The laser guided boring machine was required to achieve the directional tolerance of + / - 25 mm, which ensured contact between the steel sets and canopy tubes could be achieved with grout socks. Prior to boring for the canopy tubes, the highway median drain was realigned to a location clear of the underpass canopy tubes. Fill surrounding the existing median drain was removed, with the bedrock exposed prior to the ground improvement being carried with the placement of a mass concrete. The mass concrete placed was of a similar strength to the rock being bored through by the canopy tubes, to minimise any drilling deviation issues and to provide a material of consistent strength in the crown.

### 4.2 Crown Arch

The composite primary support elements for the underpass crown comprised the aforementioned canopy tubes and 200 UC profile steel sets installed at 1.5 m centres below the highway carriageways and 2.0m centres below the highway median. Grout socks placed on the steel sets ensured immediate transfer of steel set support capacity to the exposed canopy tubes above. The crown profile was completed by steel fibre reinforced shotcrete infill between the steel sets in layers of 100 mm per excavation stage up to a total 250 mm thickness.

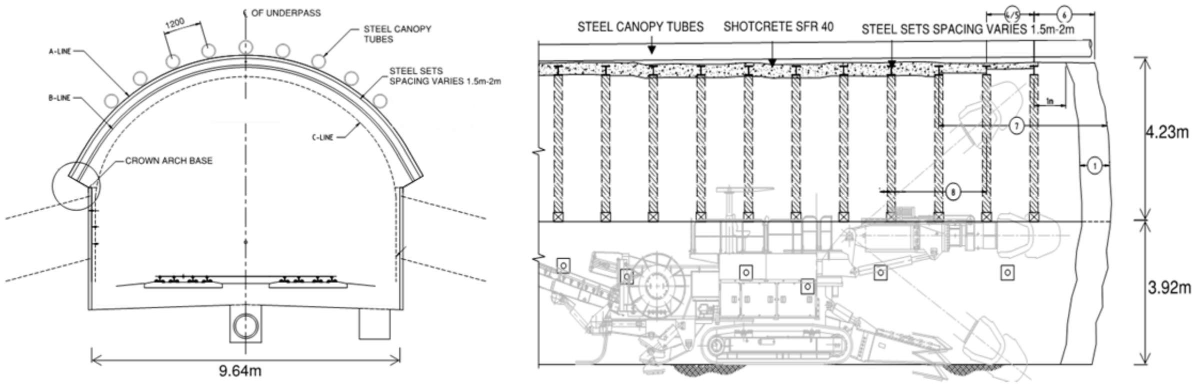


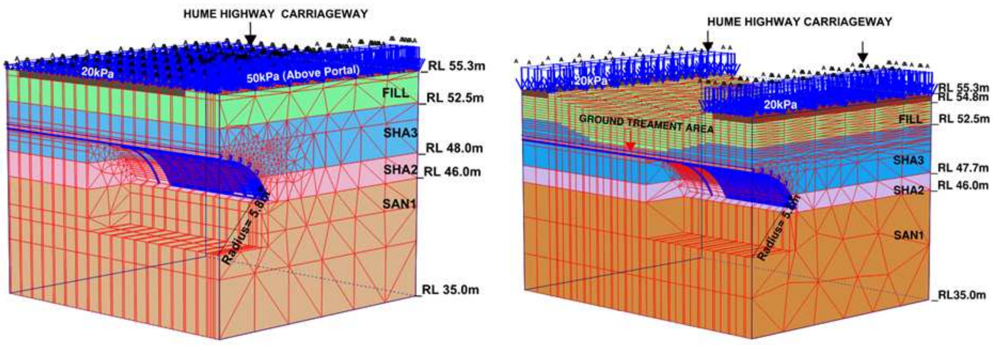

*Figure 3. Underpass primary support and excavation staging*

## 5 PLAXIS 3D TUNNEL MODELLING

A specific focus of the primary support analysis was to validate compliance with the settlement limits of the Hume Highway carriageways. Design of the primary support system was undertaken using geotechnical analysis tools as well as structural analysis software. The Geotechnical modelling was primarily undertaken using Plaxis 3D Tunnel as discussed in this section. In addition to the geotechnical modelling undertaken using Plaxis 3D Tunnel, 2D analysis was undertaken for sensitivity cases and for construction stage back analysis using Phase[2].

PLAXIS 3D Tunnel was selected for the analysis due to its ability to analyse the staged underpass excavation and support installation, including the effect of the structurally significant large diameter canopy tubes. Due to the complexity of the 3D model with a fully staged support system, undertaking fully jointed rock models was not considered practical given their required model run time. To further simplify the computations a plane of symmetry was adopted the centreline of the underpass, for both the carriageway and median models (refer to Figure 4). Multiple models with differing geology and in situ stresses were analysed within PLAXIS 3D Tunnel, allowing assessment of:

- Ground surface movements on the existing highway and median. The outputs provided the ability to assess the predicted ground movements in both the direction of and perpendicular to the progressing excavation face. This provided sufficient information to predict the settlement trough and therefore specify the extent of ground surface monitoring for the highway;
- Global and local stability assessments for underpass rock structure during excavation; including the excavation face. This included assessment of the stress distribution and plastic zones in the underpass side wall rock, particularly in the hitch zones where the crown arch is founded; and
- Structural loading and predicted movements in the primary support system.




*(a) Under carriageway* *(b) Under median*

*Figure 4. PLAXIS 3D Tunnel modelling*

### 5.1 In situ Stress Generation

The upper and lower bound in situ stress conditions were considered in the analyses and were generated using the $K_0$ procedure. With this approach, the horizontal effective stresses in the finite element model were calculated by applying an equation representing the coefficient of lateral earth pressure at rest ($K_0$) to the vertical effective stress.

### 5.2 Structural Members

The grout filled canopy tubes, grout socks, steel sets, shotcrete and median ground treatment were all included within the models to match the construction sequence. In the analysis, the Youngs Modulus of the shotcrete was increased as the excavation advanced further away from the element to model the shotcrete strength gain with age and was also varied depending on the number of daily advances. The steel sets and shotcrete were modelled as elastic plate elements whereby the section properties of axial stiffness (EA), bending stiffness (EI), Poisson's ratio ($\nu$) and self-weight (w) were defined. A screenshot of the model with the included structural elements is provided in Figure 5.

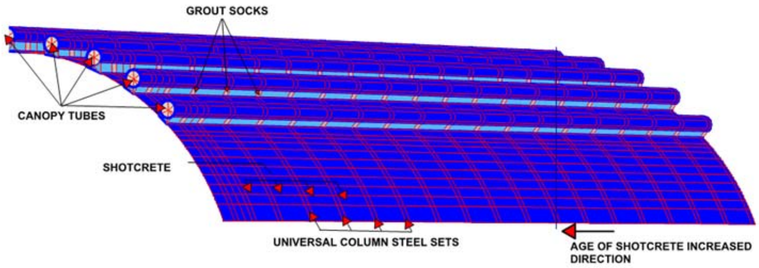



*Figure 5. Structural members in PLAXIS 3D Tunnel modelling*

### 5.3 Construction Sequence

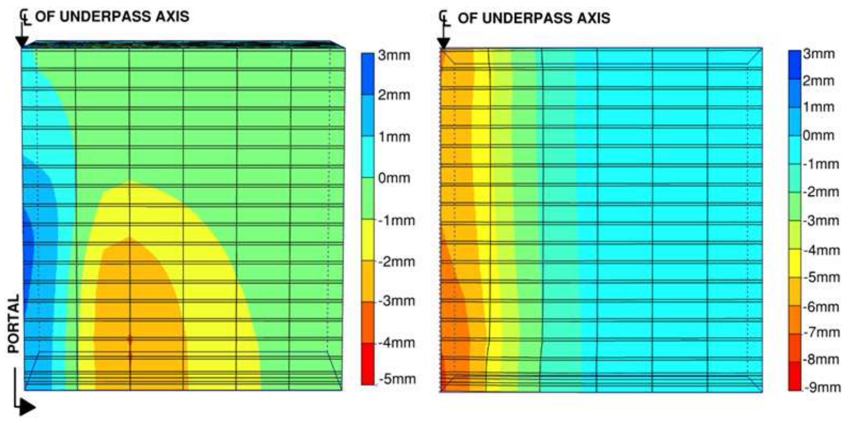
The construction sequence was idealised in the analysis with the excavation stage and the activation of the primary support from the previous stage executed in the same model step. The underpass excavation and support installation was carried out in the following sequence:1) Initial stress generation using $K_0$ procedure; 2) Apply all live loads including traffic loads; 3) Install canopy tubes and ground improvement; 4) Excavate the specified advance length (1.5m or 2m); 5) Install steel set 1 m behind the face and again excavate the specified length; 6) Install shotcrete in 100mm layers until full design thickness is achieved; and repeat the above step 4 to 6 until model is fully excavated.

### 5.4 Results of Analysis

The results obtained from the lower bound stress analysis indicated a maximum settlement and the maximum pavement distortion above the centreline of the underpass of approximately 10 mm and approximately 2mm in 2m in any direction respectively.

The upper bound stress condition results suggested that the ground will heave at the centreline of the underpass due to the side walls being subjected to high initial horizontal stresses prior to the primary lining being in place. The results indicated that the anticipated maximum vertical heave and maximum pavement distortion would be approximately 4 mm and <1mm in 2m respectively.

These values were well within the design pavement movement limits of 20mm settlement and 5mm in 2m in pavement distortion in any direction. An example of the output diagrams displaying the pavement settlement contours is provided in Figure 6.




*(a) Carriageway (upper bound)* *(b) Carriageway (lower bound)*

*Figure 6. Ground settlement contour on existing pavement (plan view)*

The face stability during the staged excavation was assessed in the Plaxis 3D Tunnel model by determining the factor of safety against collapse using the phi-c strength reduction method. This method reduced the strength parameters of rock and soil to compute the factor of safety at the threshold of failure. The study also examined the global and localised instability of the staged excavation and the installed primary support system, and provided a factor of safety of approximately 2.0 when using the documented geotechnical design parameters.

The loading on the primary support structural elements was also assessed for both stress cases, and a variety of sensitivity cases. The actions and member deformations taken from the modelling were below the respective structural limits, and this was further validated with separate structural modelling.

## 6 TUNNELLING PERFORMANCE

A comprehensive ground movement monitoring system was installed during the works to measure the actual movements encountered. This enabled the design predictions to be confirmed, or measures implemented should deformations be higher than anticipated. Automated and manual monitoring systems were adopted, with the most useful system being the real time continuous optical survey monitoring of pavement carriageways, which provided instantaneous feedback regarding the pavement deformations after the excavation advanced.

The most significant observation made during excavation was the pavement heave in the order of 20mm experienced during the tunnelling process. This outcome (magnitude) was largely unexpected, and was coupled with a similar magnitude of underpass side wall convergence (20mm) and underpass crown heave (15mm). A back analysis was undertaken using the Phase$^2$ models developed during the design stages in order to confirm the stability of the tunnel with this higher than anticipated heave, and also confirm the capacity of the underpass support.

## 7 CONCLUSIONS

The underpass was planned, designed and executed to control risk to the highway. A specific focus of the design was to ensure compliance with the highway carriageway settlement limits of 20 mm. Tunnel stability and effects on the overlying highway pavement were meticulously analysed during design and monitored during construction, with complex 3D geotechnical analysis carried out to analyse the impact of the excavation sequencing and support designed on the pavement deformations.

The underpass was safely and successfully constructed beneath the Hume Highway without disruption to the highway users, and without any observed damage to the highway pavement. Monitoring during construction indicated that the underpass excavation induced heave on the highway pavement. Back analysis and site observations indicated that support system was adequate to ensure safety for the highway uses and that excavation could continue uninterrupted.

## 8 ACKNOWLEDGEMENTS

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