Building around existing tunnels
Seminar

Basement excavation analysis potential impact on the North Georges River Sewer tunnel

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Sydney, 27 April 2023

AUSTRALIAN TUNNELLING SOCIETY

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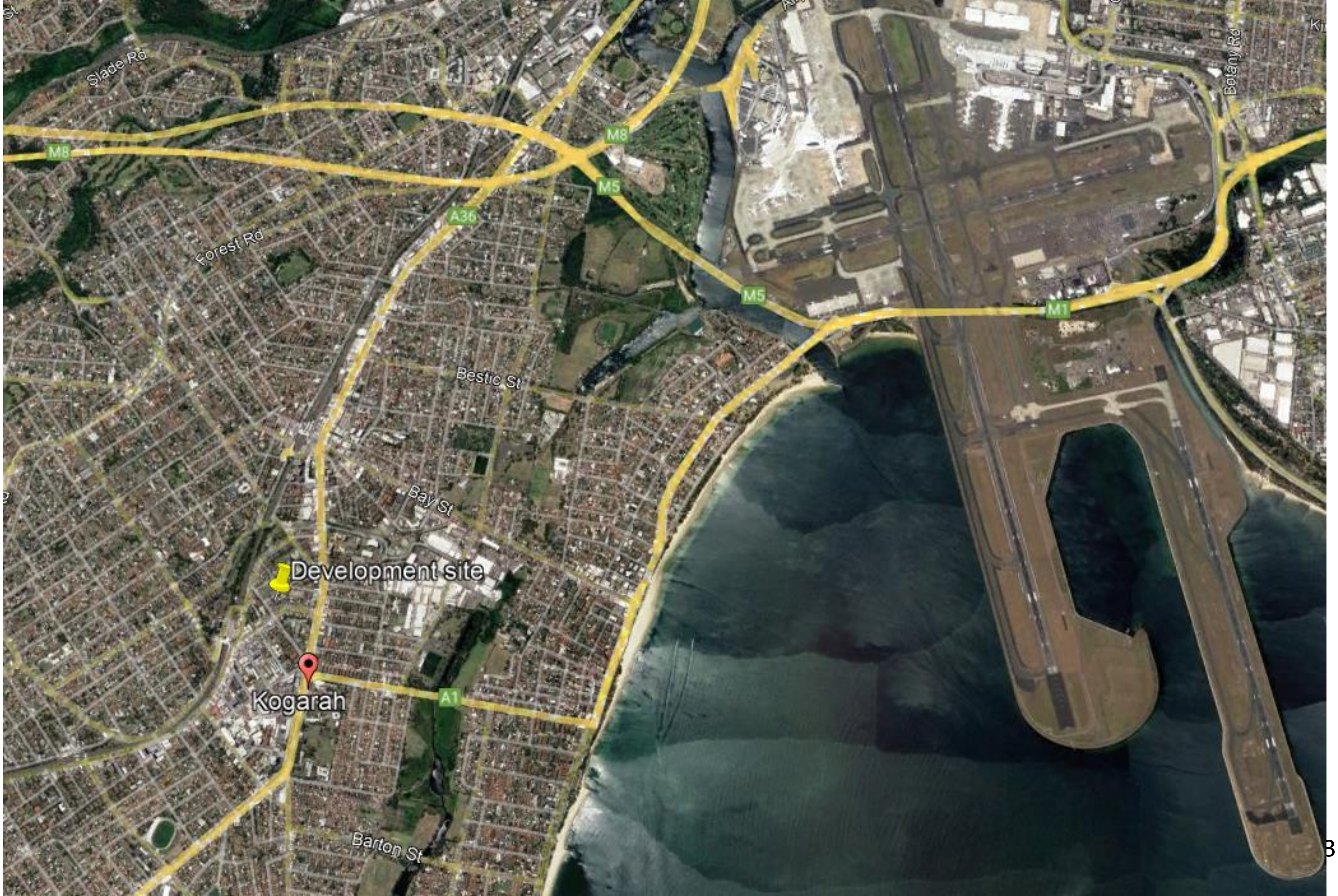
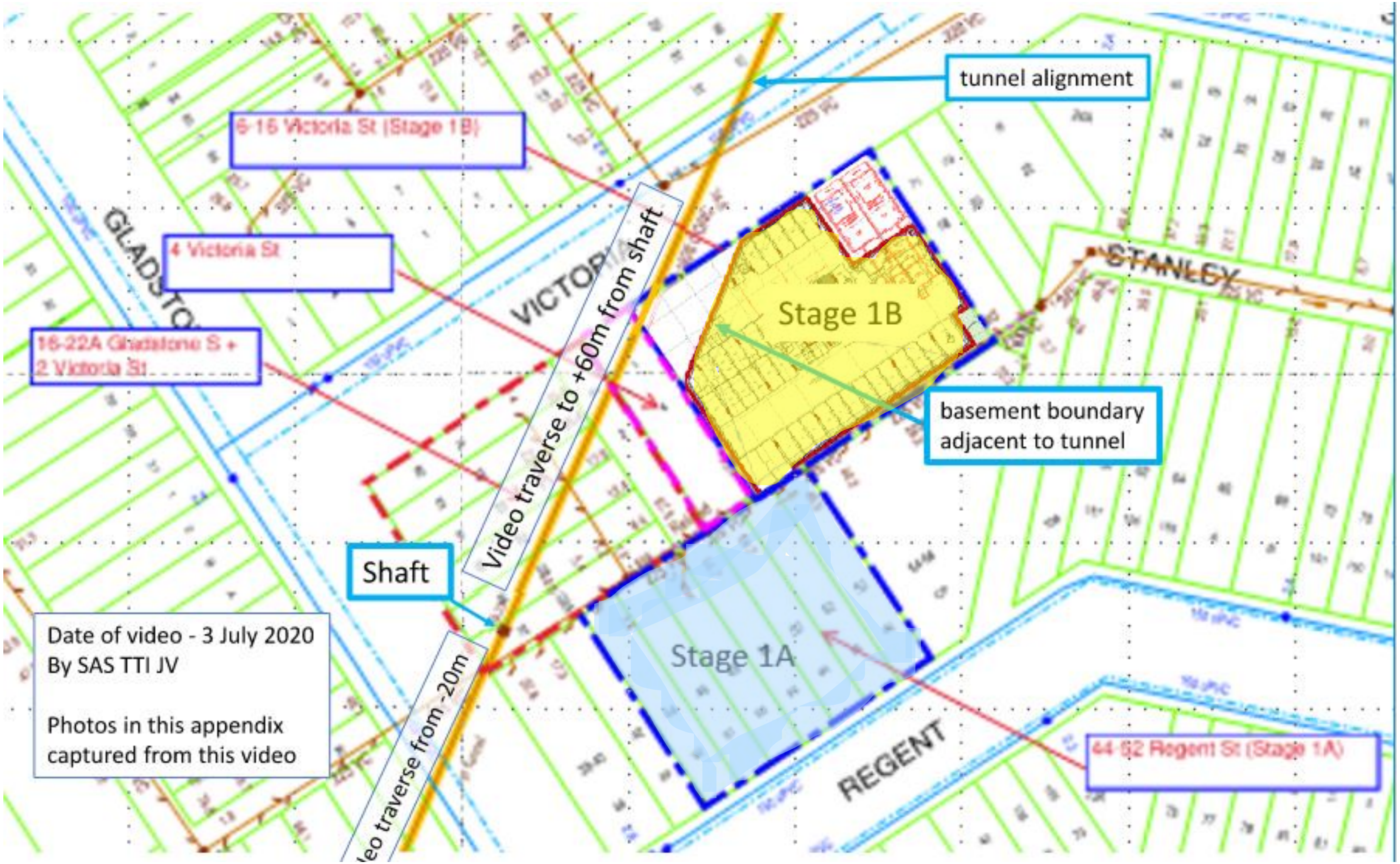
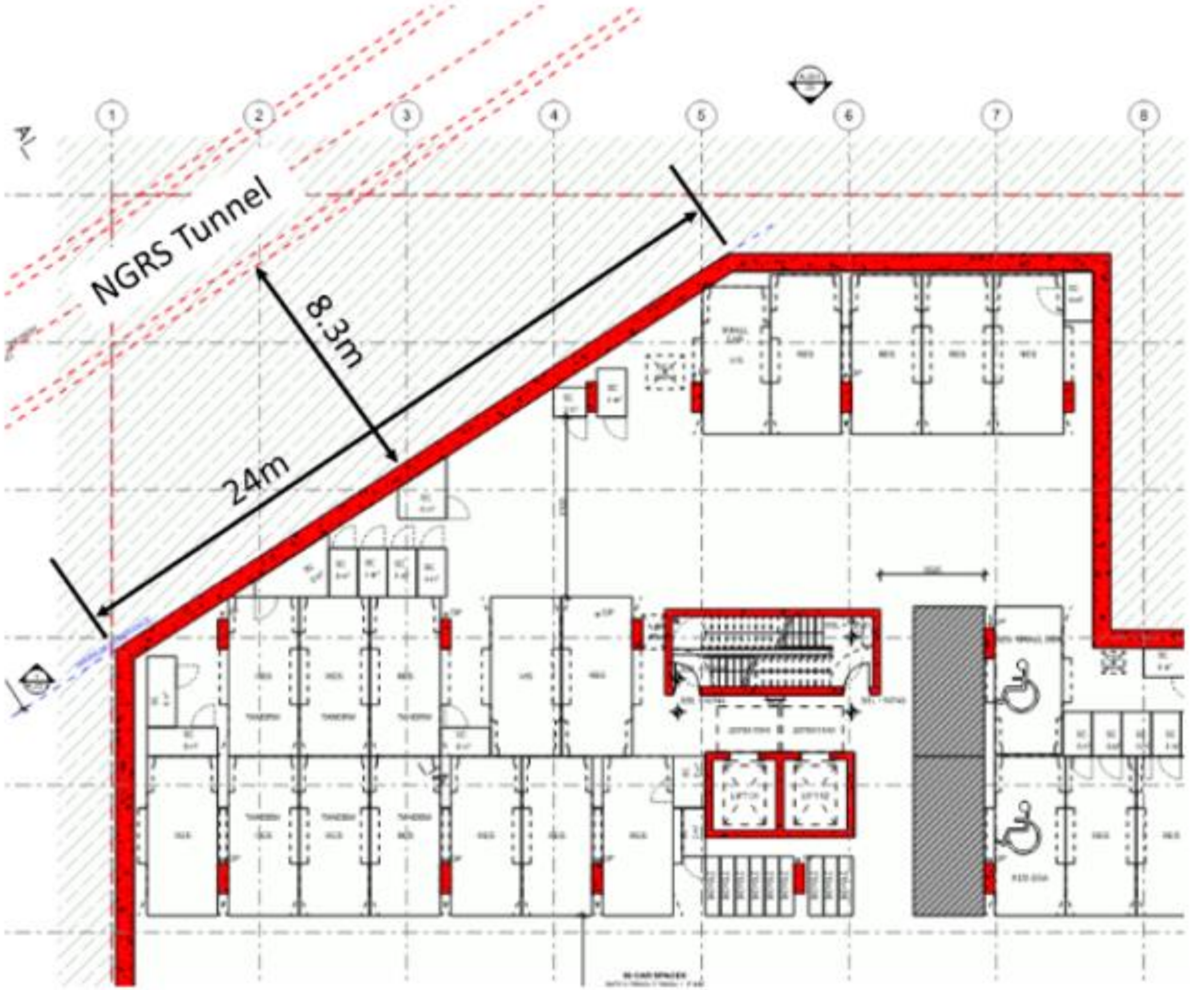
1. Introduction
2. Site Description
3. Sewer Tunnel
4. Geology
5. Sydney Water Criteria
6. Finite Element Model
7. Groundwater Pressure
8. Conclusions


Slade Rd
M8
M8
A36
M5
M5
Forest Rd
M1
Botany Rd
Bestic St
Bay St
Development site
Kogarah
A1
Barton St

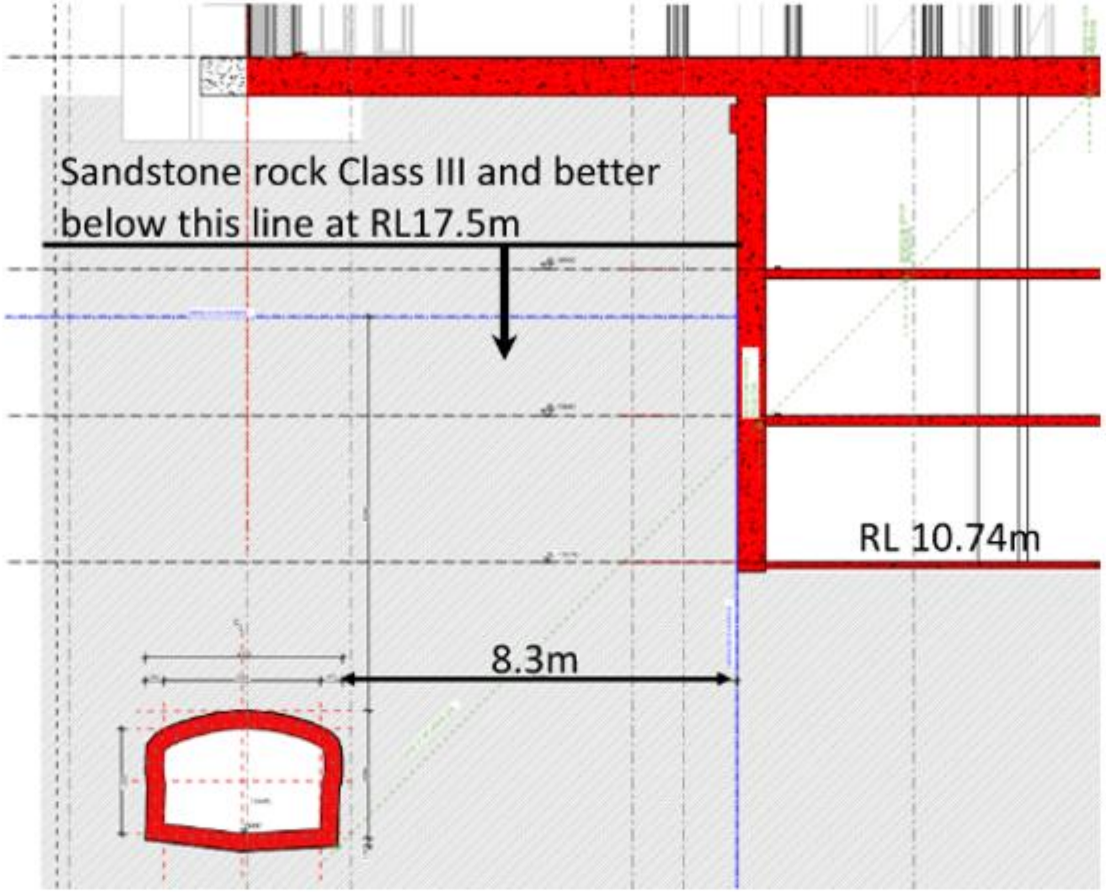
Victoria and Regent Street Sites - Koranga



Stage 1B and Stage 1A - this presentation mainly relates to Stage 1B



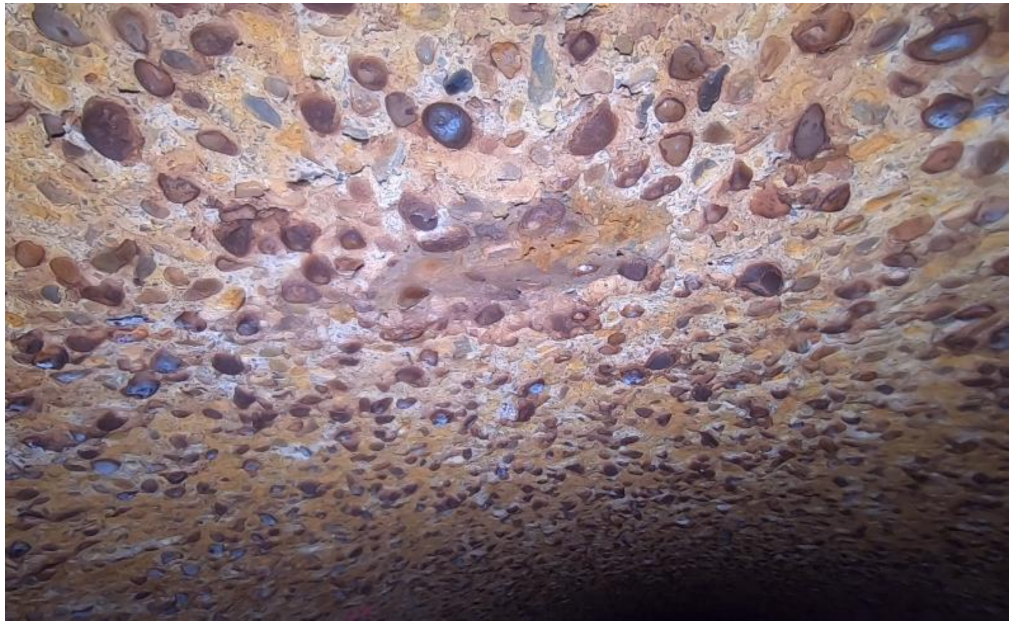
Plan



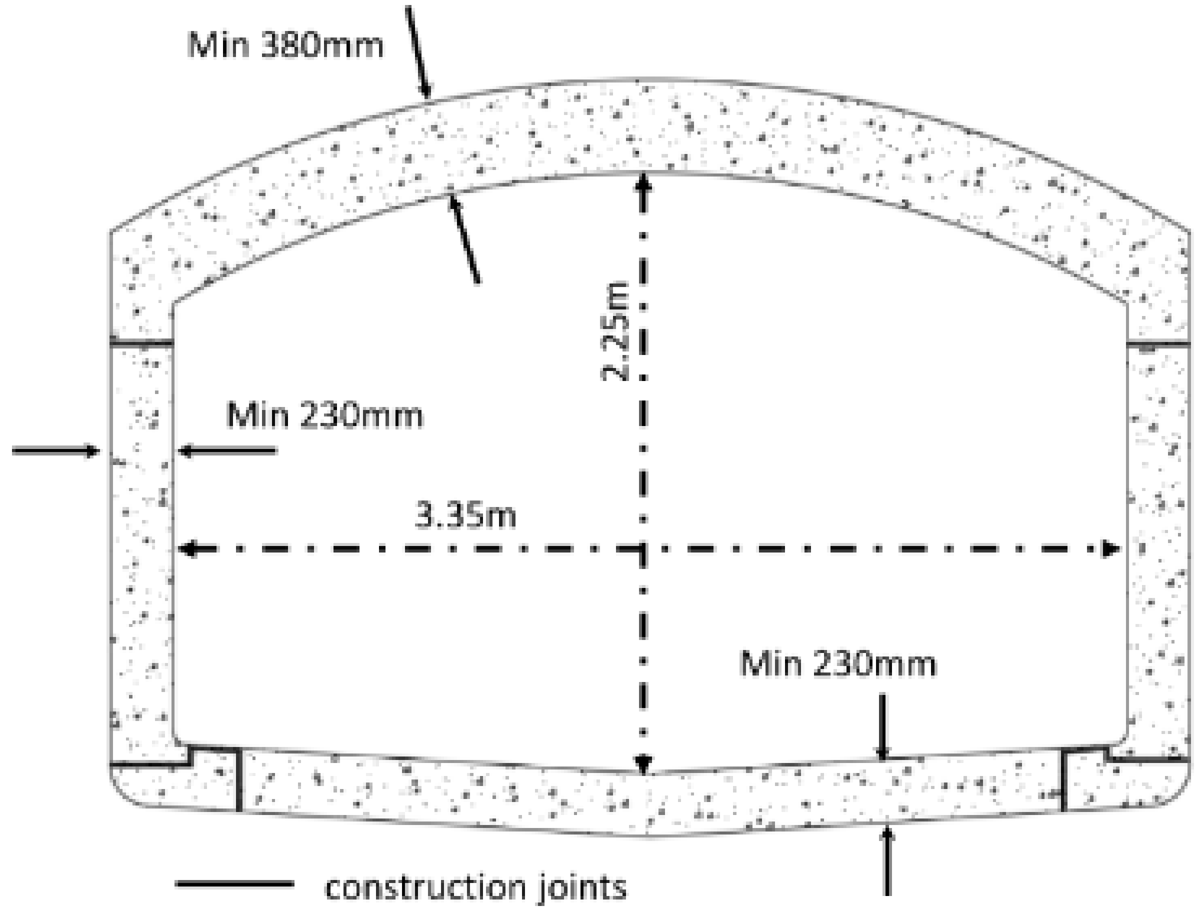
Section (12m deep basement excavation)

Stage 1B Site

Base of manhole access



This still images were taken in 2021 from
a video taken by a dilapidation consultant



Note some important issues here:

1. No steel reinforcement
2. Vertical walls and flat invert
3. Wide span of invert
4. Construction joints
5. Corners – stress concentrators
6. Which is the most critical element walls or tunnel invert

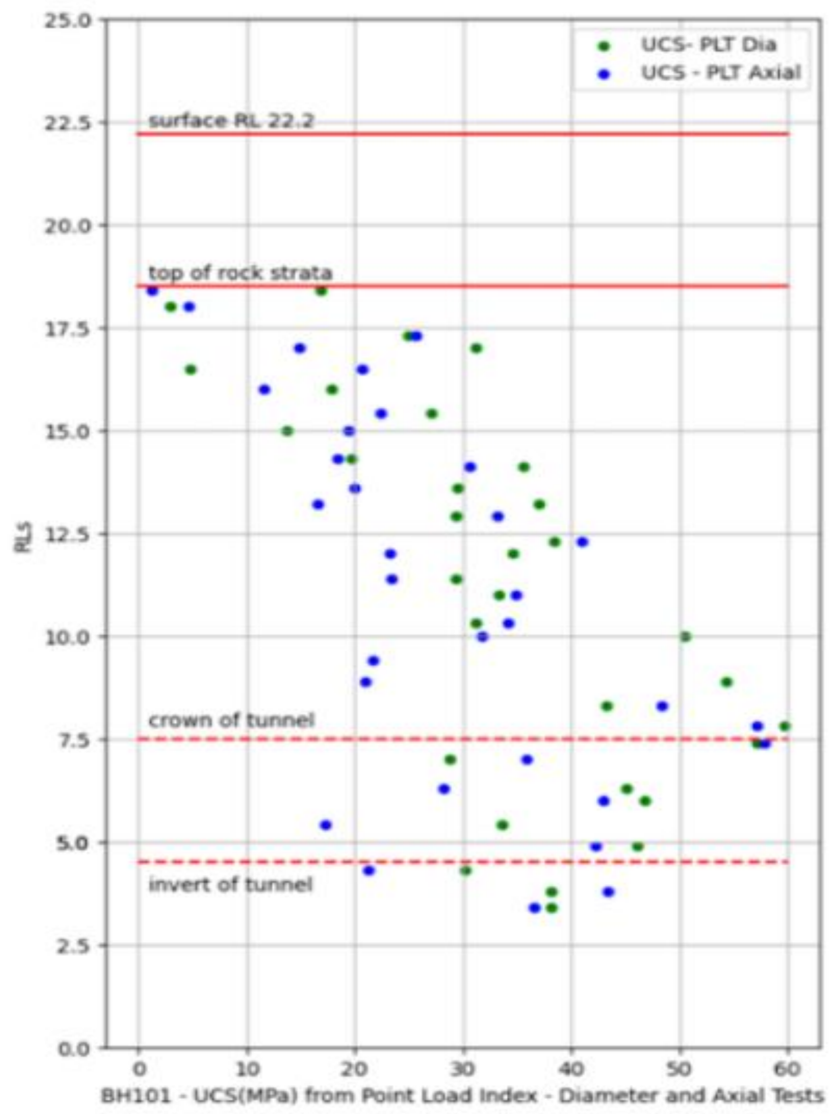
Tunnel section – unreinforced concrete – constructed in 1942

(excavation method – drill and blast, no time delays)

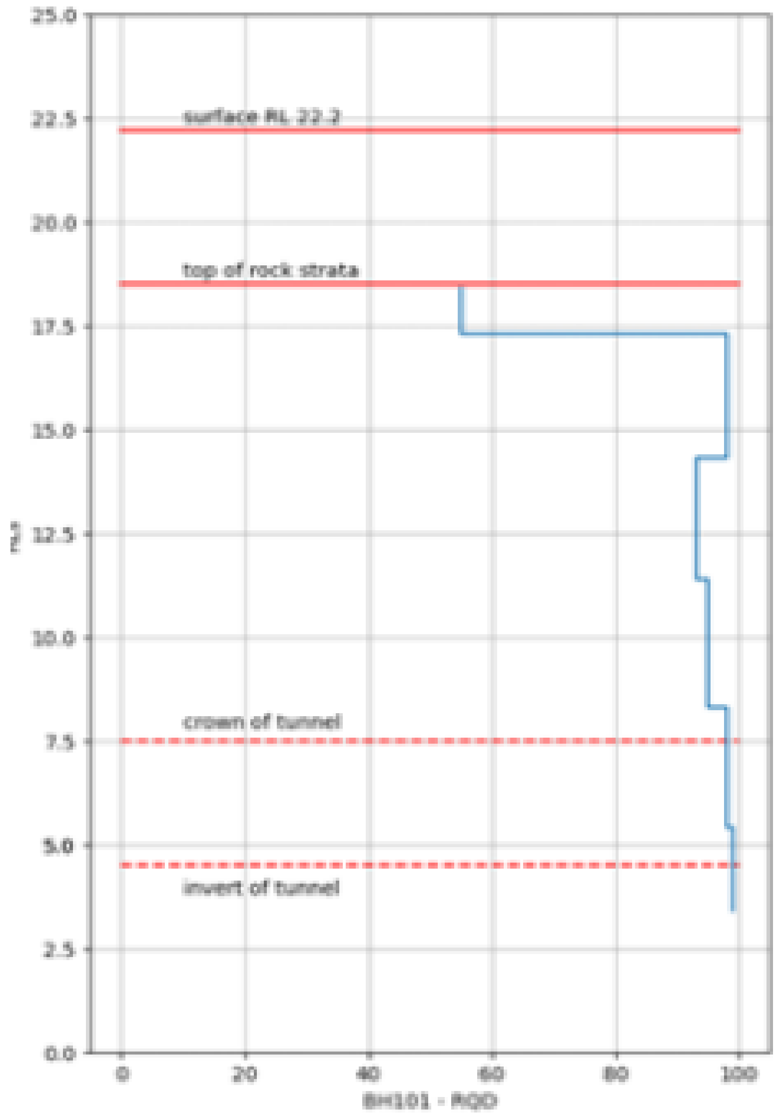
Exposed sandstone rock at a nearby site

Core Box Photo – BH101

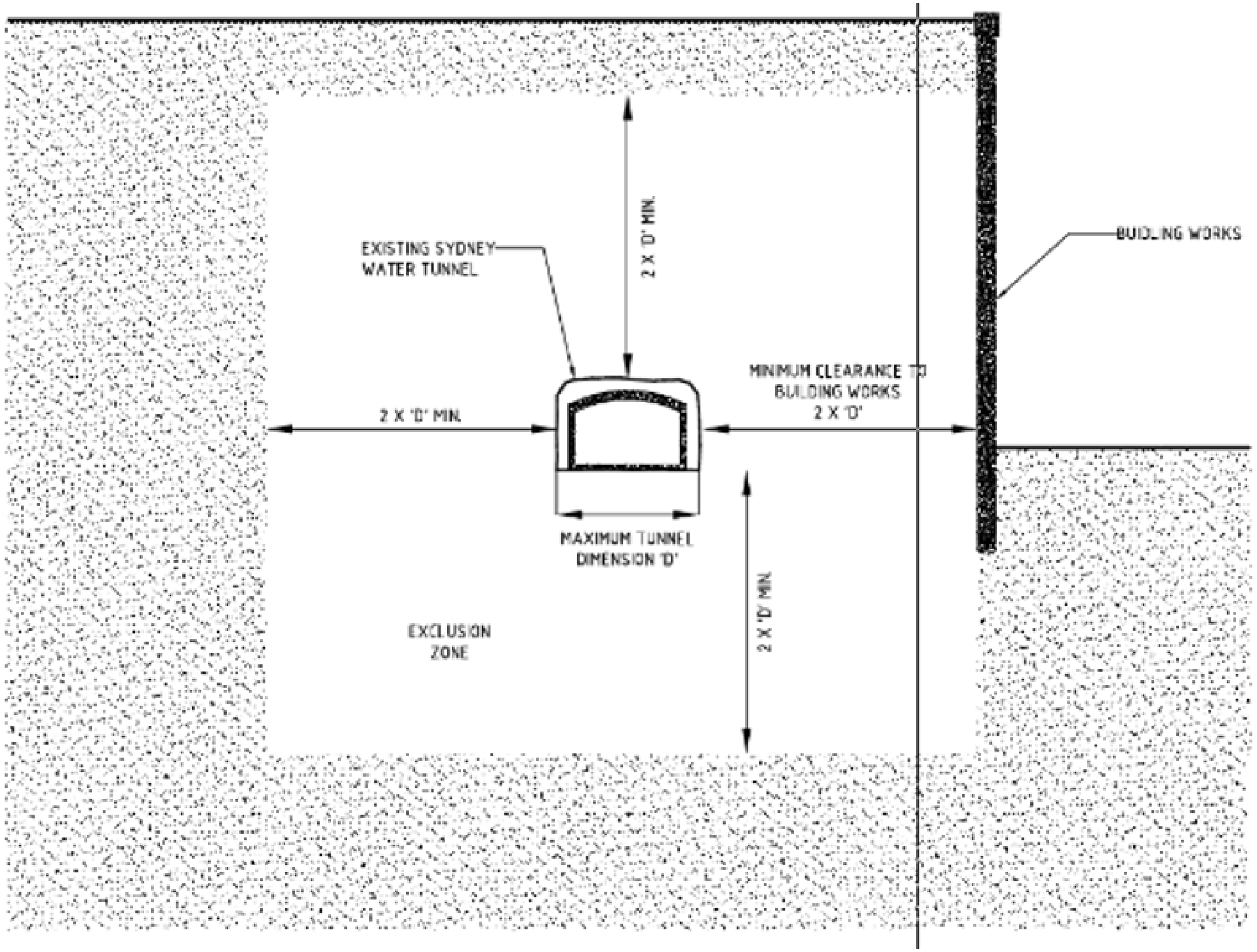
alliance	Client Name:	Landmark Group Australia Pty Ltd	Figure / Drawing Number:	-
	Project Name:	Proposed Development	Figure / Drawing Date:	14/05/2021
	Project Location:	6-16 Victoria Street, Kogarah NSW	Report Number:	10114-GR-3-3



Typical UCS data

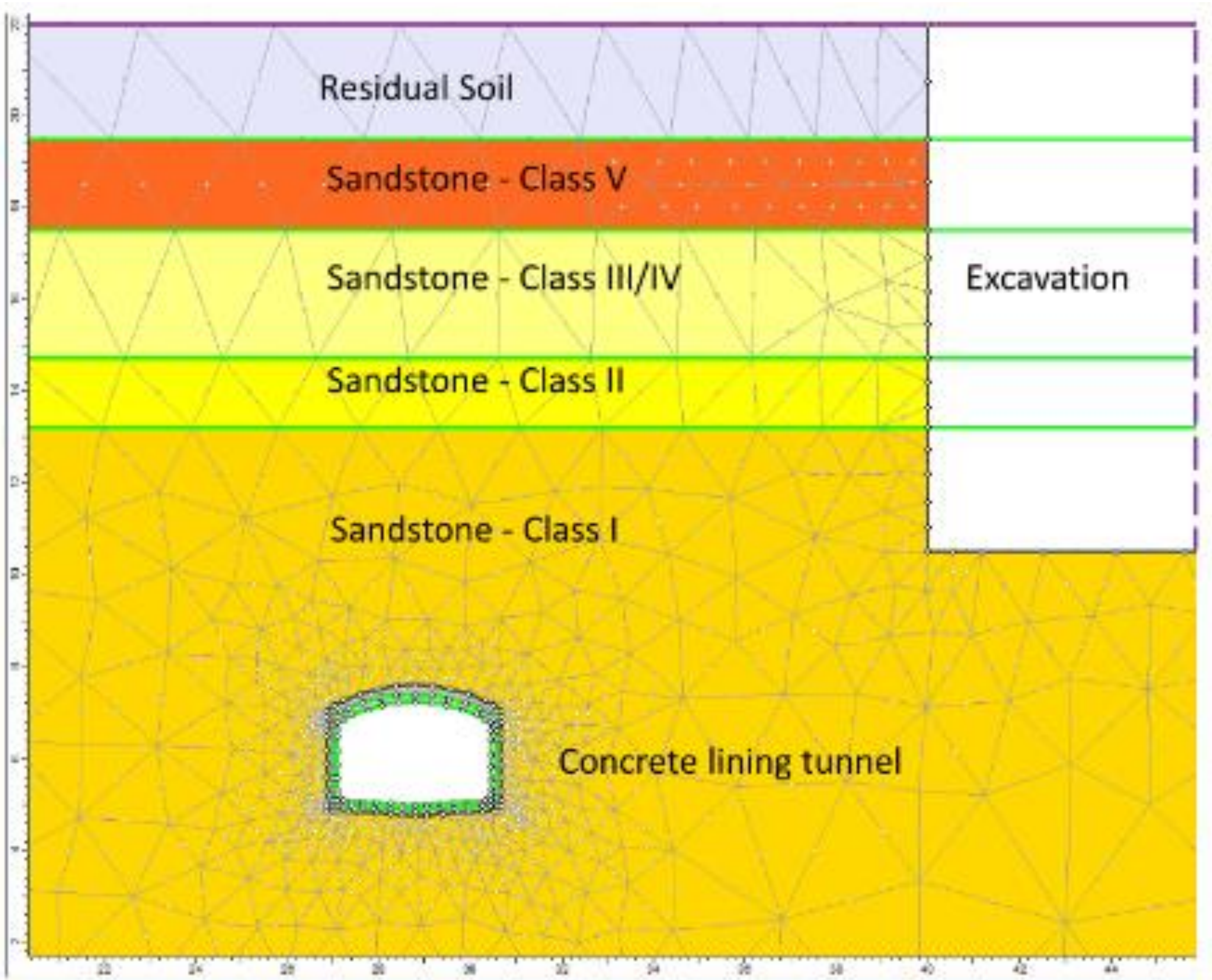


RQD with depth



Sydney Water dimensional criteria

(footnote: what is the most important issue – displacement or stresses in the tunnel lining)



2D Finite element grid

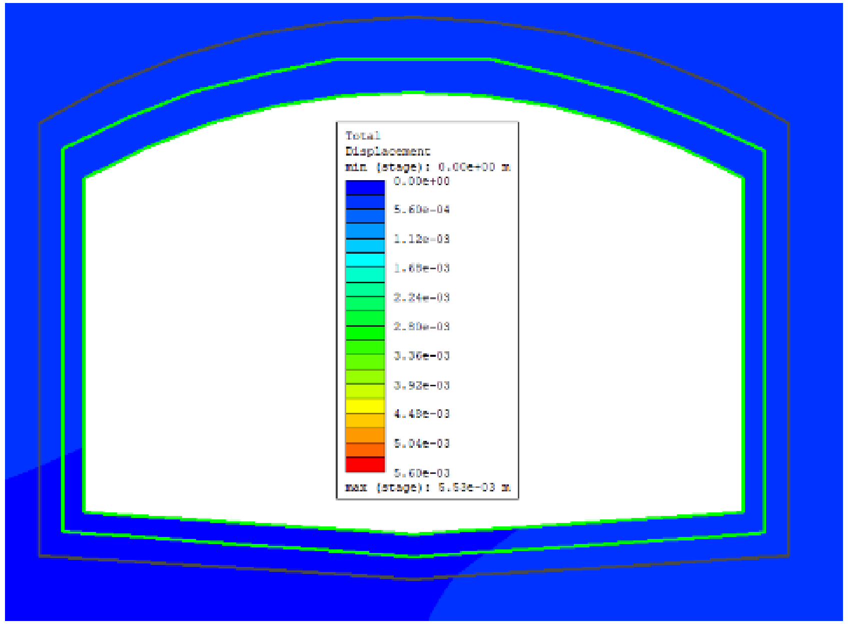
#	Strata	Weight (kN/m^3)	Modulus E(MPa)	U	Material Type
1	Residual soil – sandy stiff clay	20	50	0.3	elastic
2	Sandstone Class V	22	75	0.3	elastic
3	Sandstone Class III/IV	23	500	0.25	elastic
4	Sandstone Class II	24	900	0.25	elastic
5	Sandstone Class 1 – lower bound	24	2000	0.2	elastic
6	Sandstone Class 1 – upper bound	24	4000*	0.2	elastic
7	Blast Damaged – Sandstone Class 1	24	25% of intact	0.3	elastic
7	Tunnel Lining Concrete	24	20,000	0.2	elastic
8	Tunnel Lining Concrete* see Table 2	24	20,000	0.2	plastic

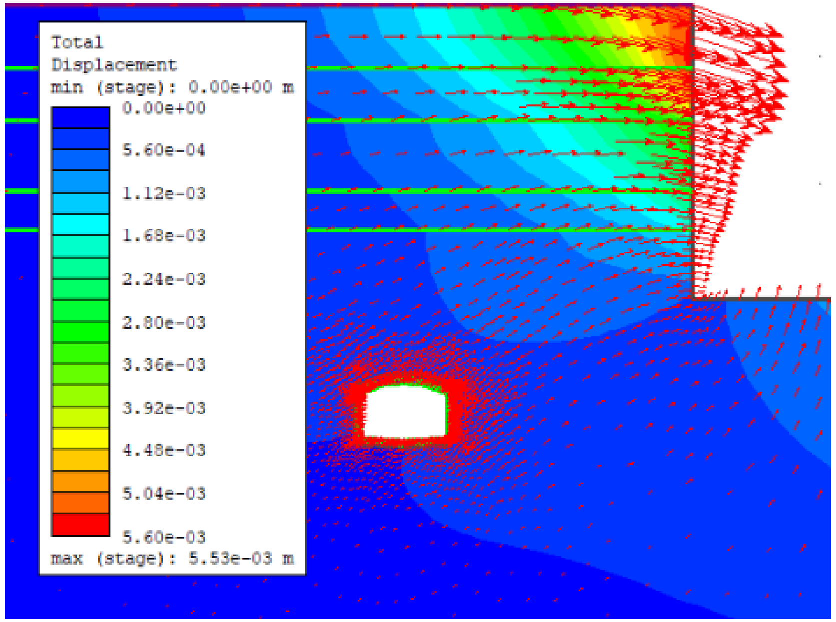
Elastic properties for the various material in the FE analyses model

Figure Numbers	Description
1A	Concrete Lining – elastic, Sandstone Class 1 E = 2000MPa, H:V stress ratio 2:1
2A	Concrete Lining – elastic, Sandstone Class 1 E = 4000MPa, H:V stress ratio 2:1
2AB	Concrete Lining – elastic, Sandstone Class 1 E = 4000MPa, H:V stress ratio 5:1
3A	As above but with construction joints added
4A	Concrete Lining – plastic, Sandstone Class 1 E = 4000MPa, H:V stress ratio 1:1 Mohr Coulomb Failure Criteria = Upper Bound peak tensile strength 0.5MPa
5A	Concrete Lining – plastic, Sandstone Class 1 E = 4000MPa, H:V stress ratio 1:1 Mohr Coulomb Failure Criteria = Upper Bound peak tensile strength 2MPa
6A	Concrete Lining – plastic, Sandstone Class 1 E = 4000MPa, H:V stress ratio 2:1 Mohr Coulomb Failure Criteria = Upper Bound peak tensile strength 2MPa
7A	Concrete Lining – plastic, Sandstone Class 1 E = 4000MPa, H:V stress ratio 2:1 Mohr Coulomb Failure Criteria = Upper Bound peak tensile strength 2MPa *Blast damaged rock around tunnel perimeter, E = 1000MPa*
8A	Foundation FE grid used for all models
9A	Zone of blast damage rock around tunnel perimeter.

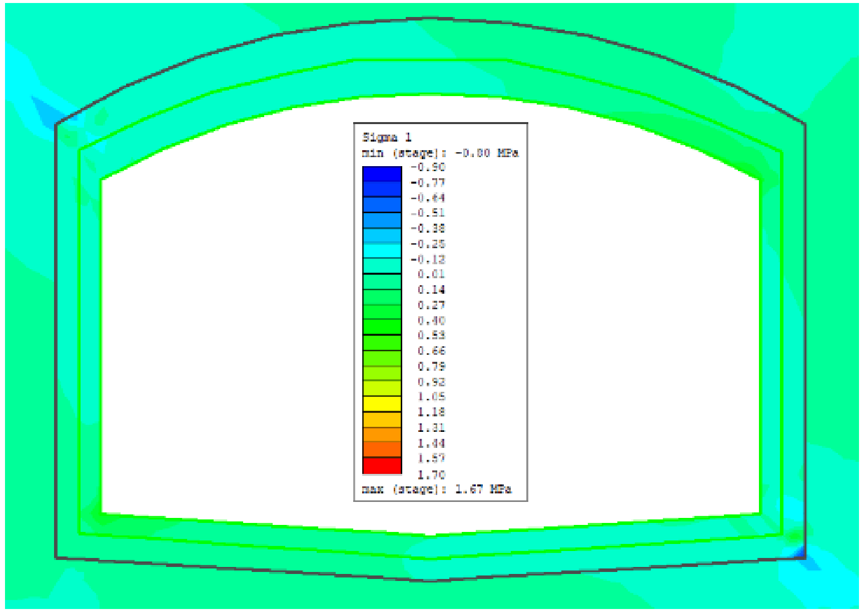
List of FE models – continuum tunnel lining/beam elements/blast disturbed zone
(additional hand calculations were also carried out relates to the tunnel invert)

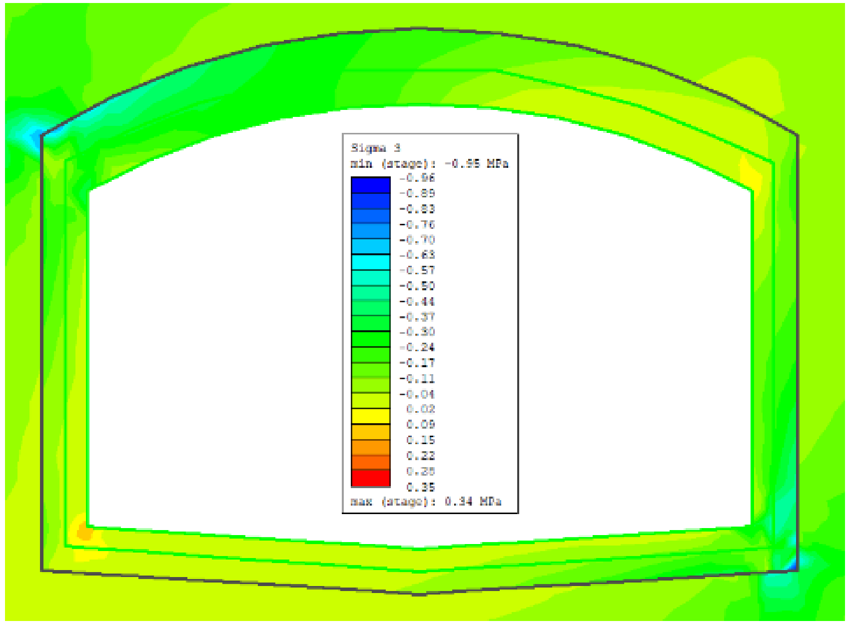
Displacement





Stresses

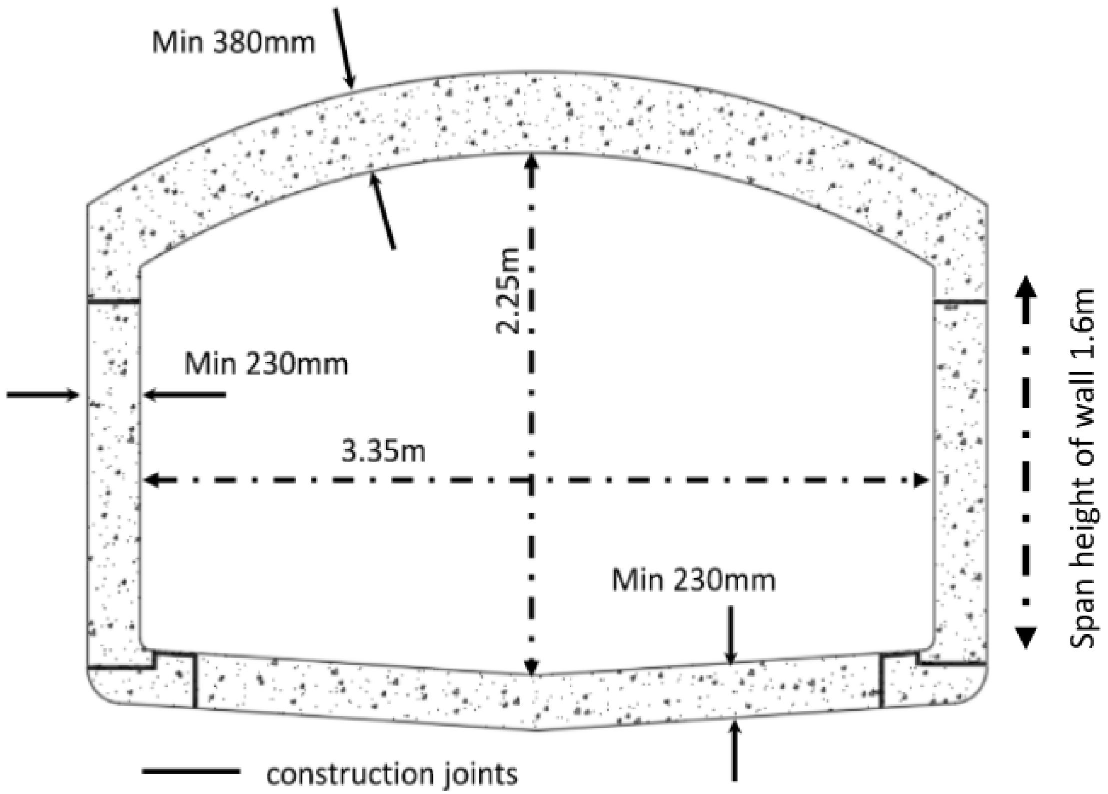




Typical FE model output

Potential Failure modes due to external water pressure(assuming that any exists)

The extremely low permeability of the surrounding rock mass and a higher permeability of the tunnel lining is likely to prevent any build up of water pressure behind the tunnel lining and this has been the case for the last 80 year life of the tunnel. Alternatively the concrete lining could be bonded to the rock. This would also mitigate water pressure acting on the lining.



Tunnel Section

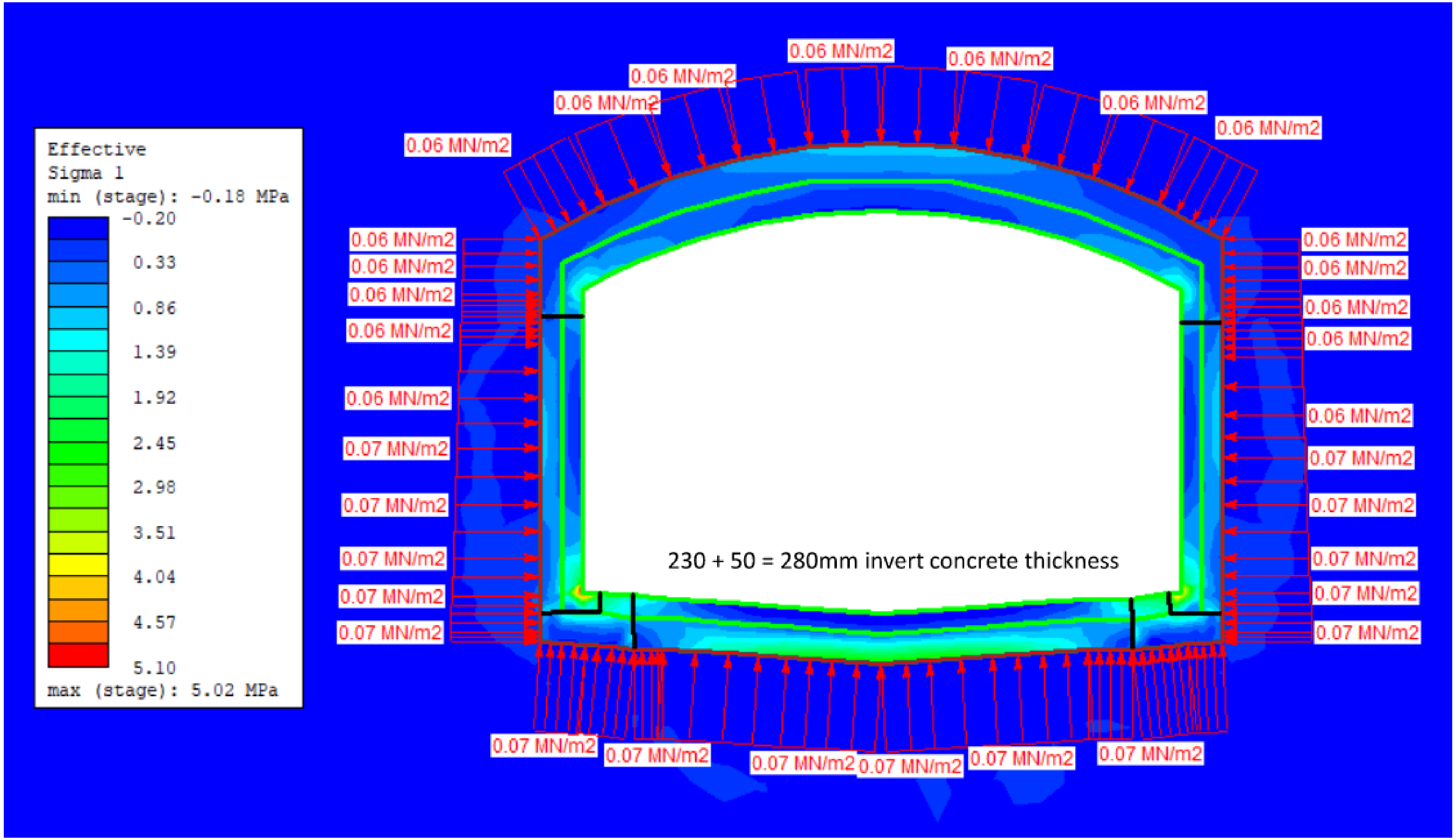
The lining thickness shown are the minimum design values which will have been exceeded during construction due to the over break caused by the drill and blast method of excavation.

The dilapidation survey(2021) core holes in the wall went to 380mm without penetrating the rock.

In theory the displacement of the tunnel invert due to water pressure up lift would require only a very small pressure head. Thickness of concrete 230mm, density = 24kN/m3, self weight = 5.52kN/m.

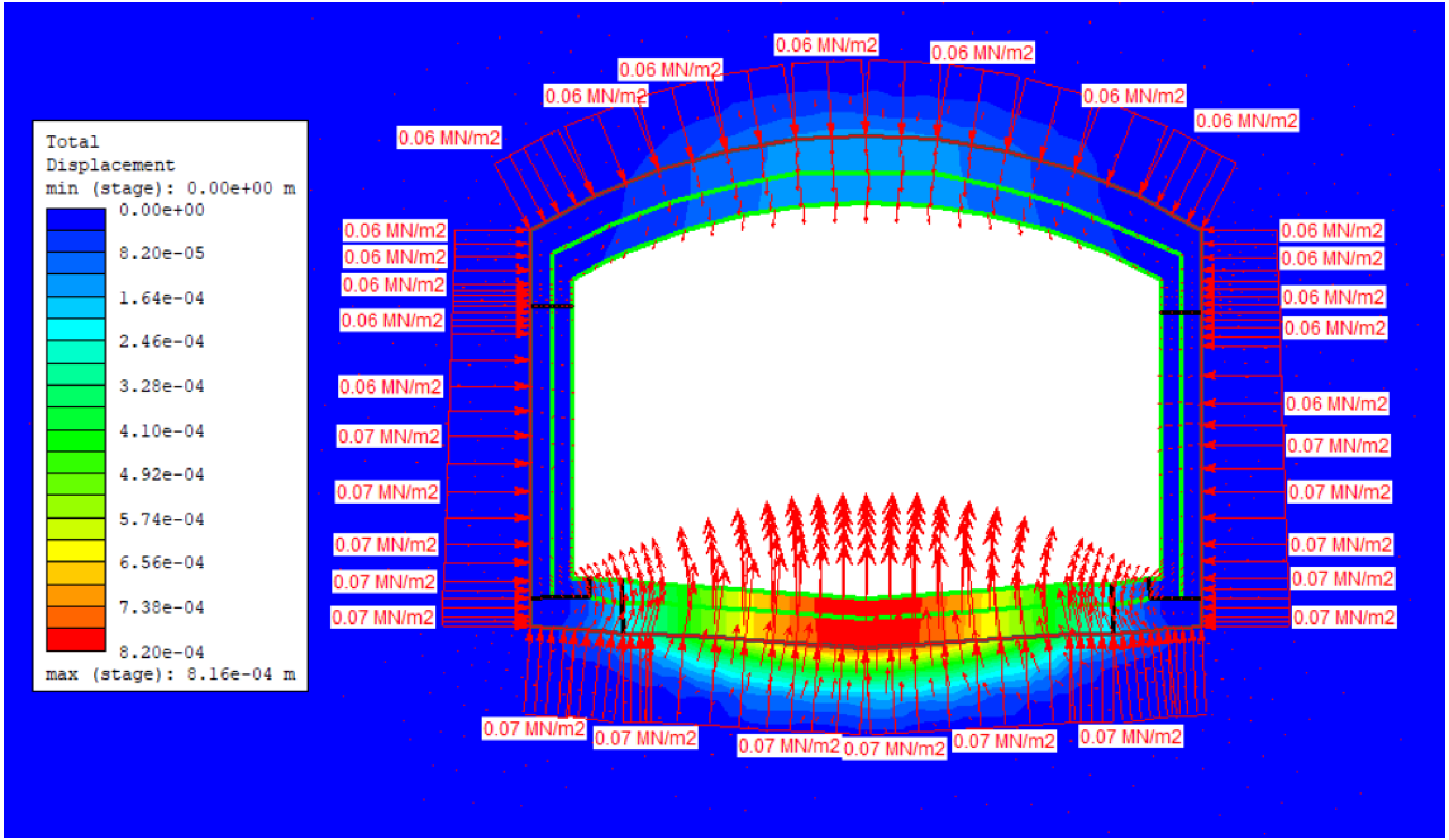
It would require only 1m head of water under the Invert slab (assuming no resistance at the vertical joints) to displacement it. Of course any minor displacement Would result in a gap for water pressure
to be relieved.

Alternatively the concrete could be bonded to the underlying rock, Sydney Water quotes Westconnex knowledge, On Westconnex the assumed bond between the shotcrete lining and the sandstone rock is 0.5MPa or 50kN (or 50m head of water). If the bond on this tunnel was only 0.2MPa this is equal to 20m head of water.



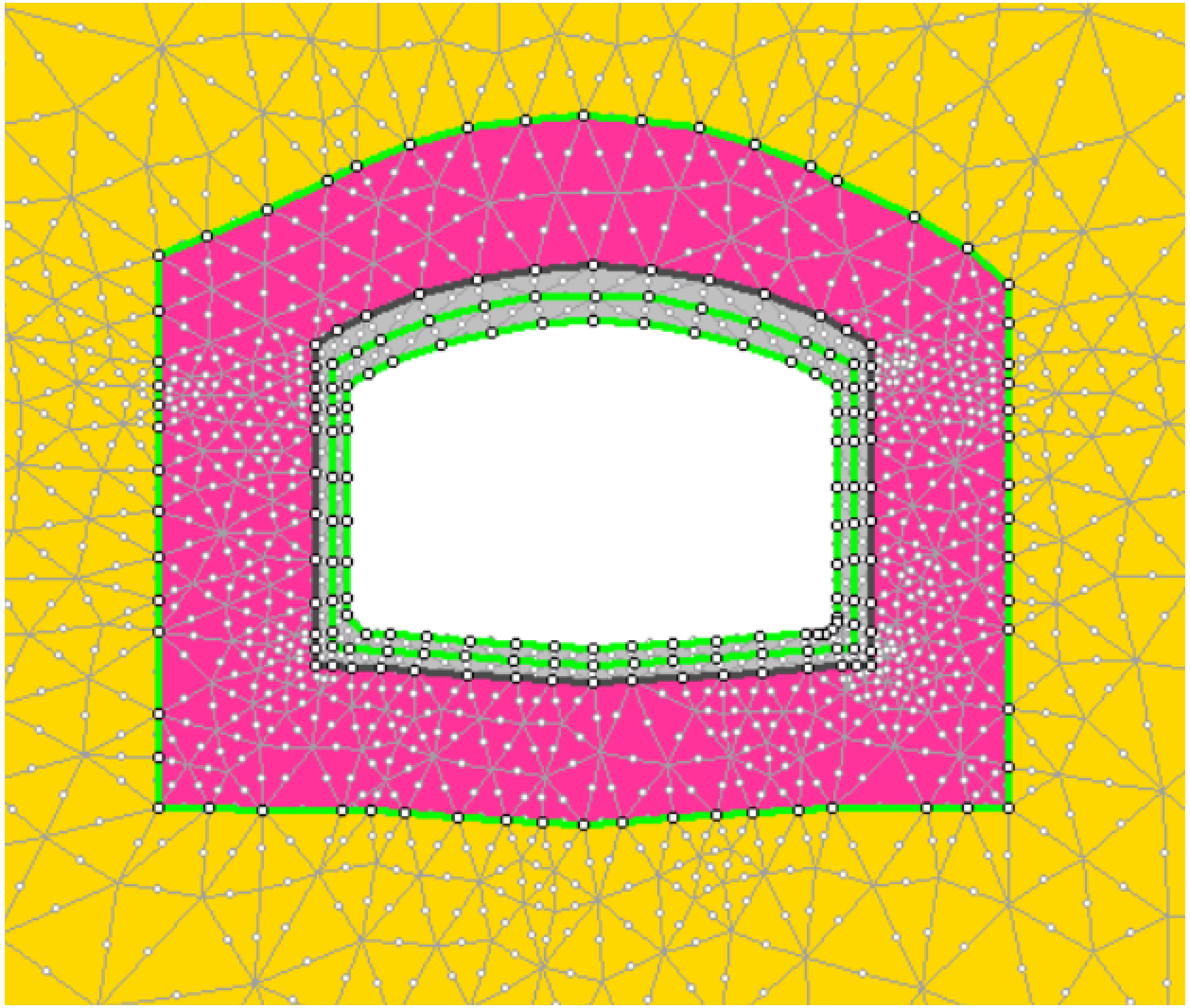
7m head of water above the tunnel invert -280mm invert

Numerous external water pressures and models were compared

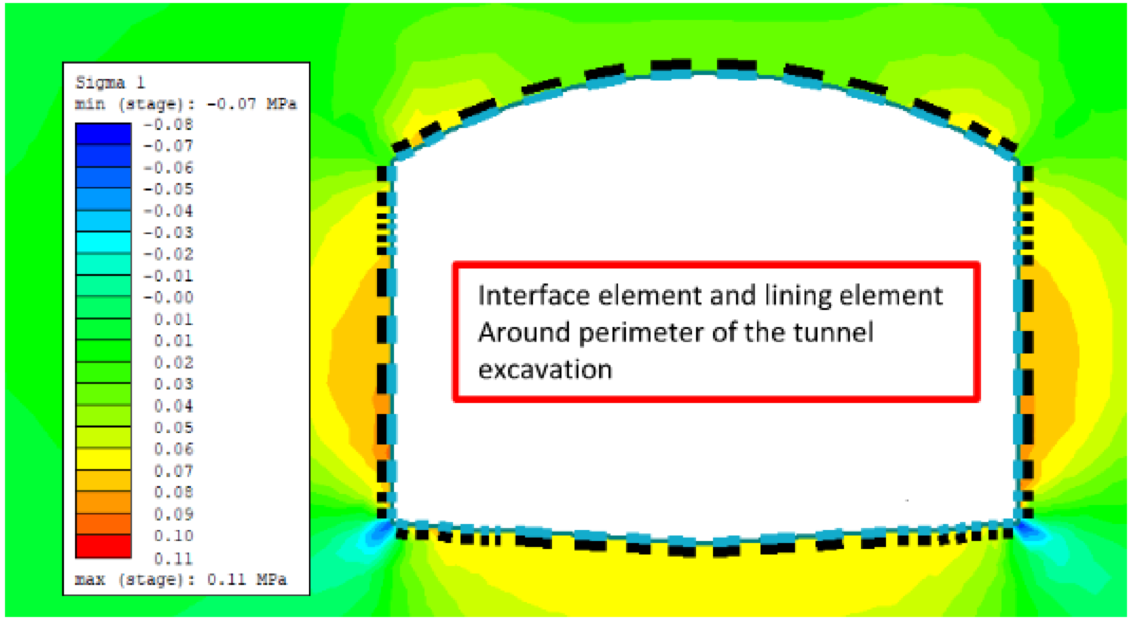


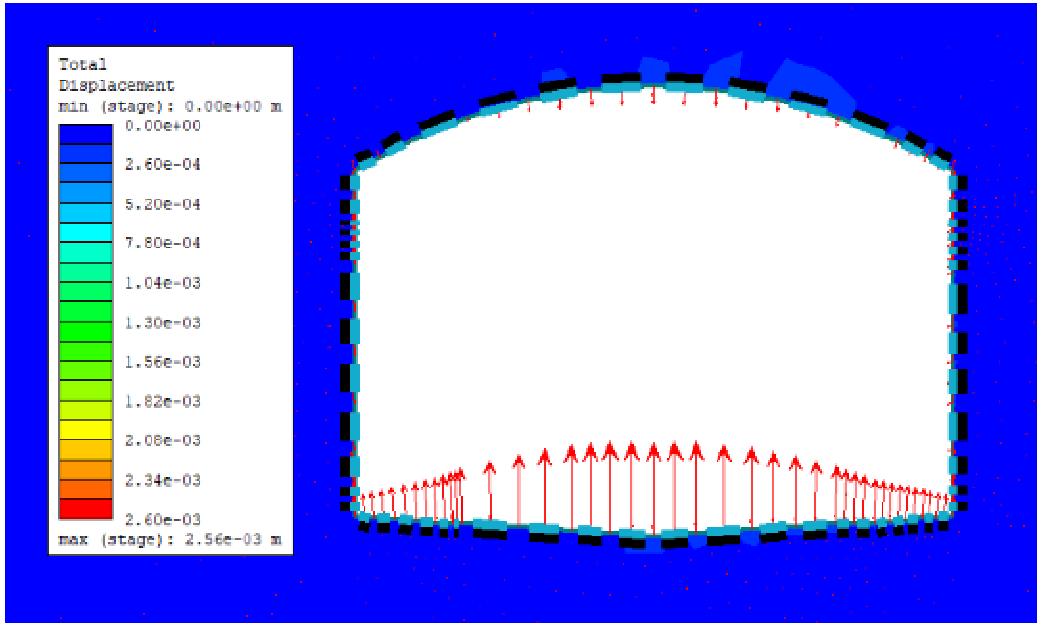
7m head of water above the tunnel invert -280mm invert

Displacement

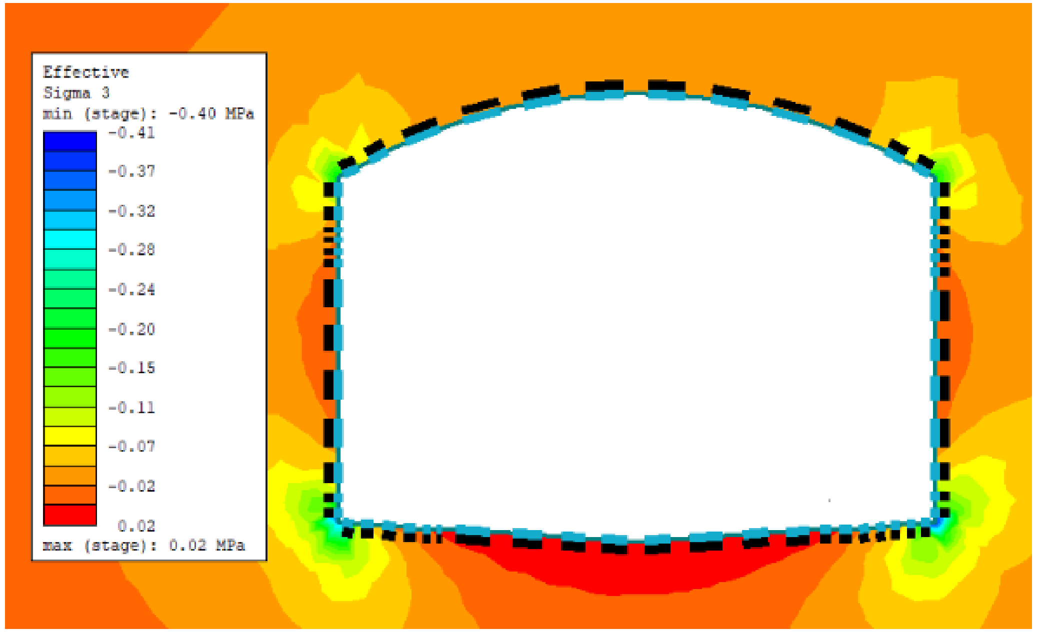


Drill and blast damage rock zone

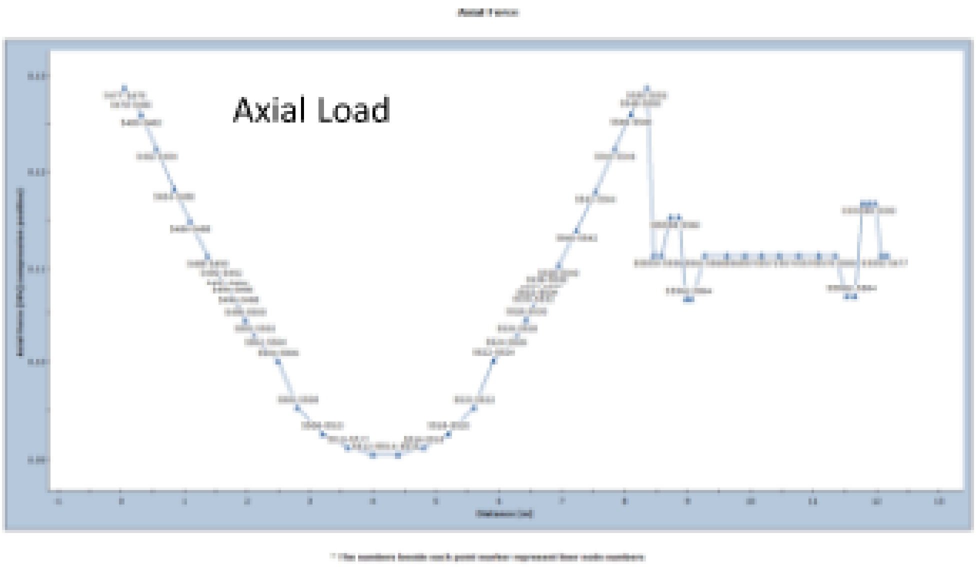


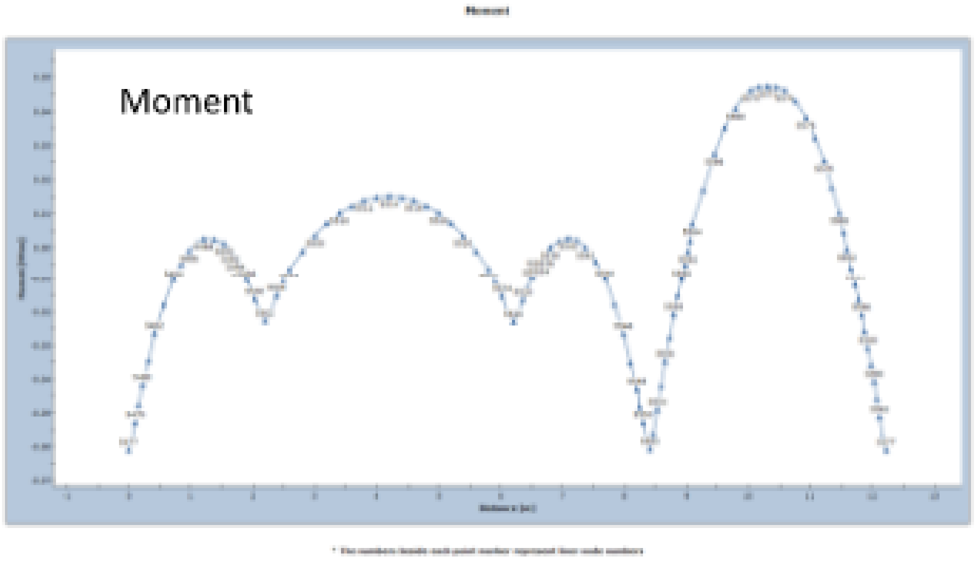


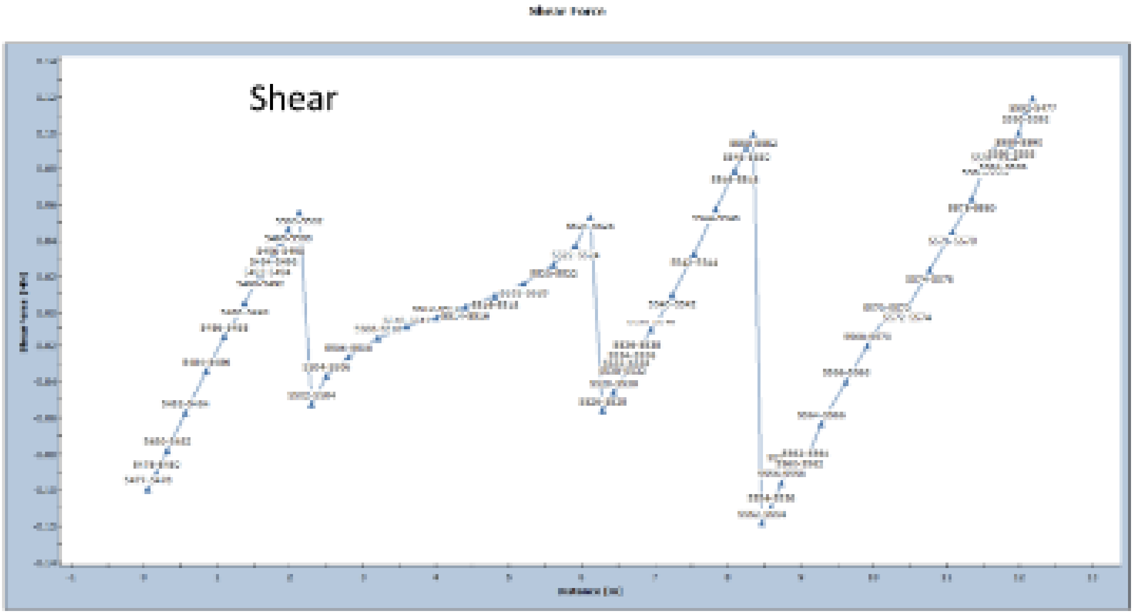
displacements



Using interface and structural beam elements







Peak Values – Tunnel Invert

Axial Load (node numbers(5564-5566) and (5578-5580))

0.113MN or 113kN Axial Stress = 113*1000/1000*230 = 0.49MPa

Moment (node number 5572 mid-span)

0.05MNm or 50kNm or Tensile Fibre Stress = M/Z – Axial Stress
B = 1000 d = 230 = 5.67 – 0.49 MPa
Z = 8.817E+6 mm3 = 5.18 MPa

Shear (node numbers (5552 -5554) and (5592-5477))

0.12MN or 120kN Shear Stress = 120*1000/1000*230) = 0.52MPa

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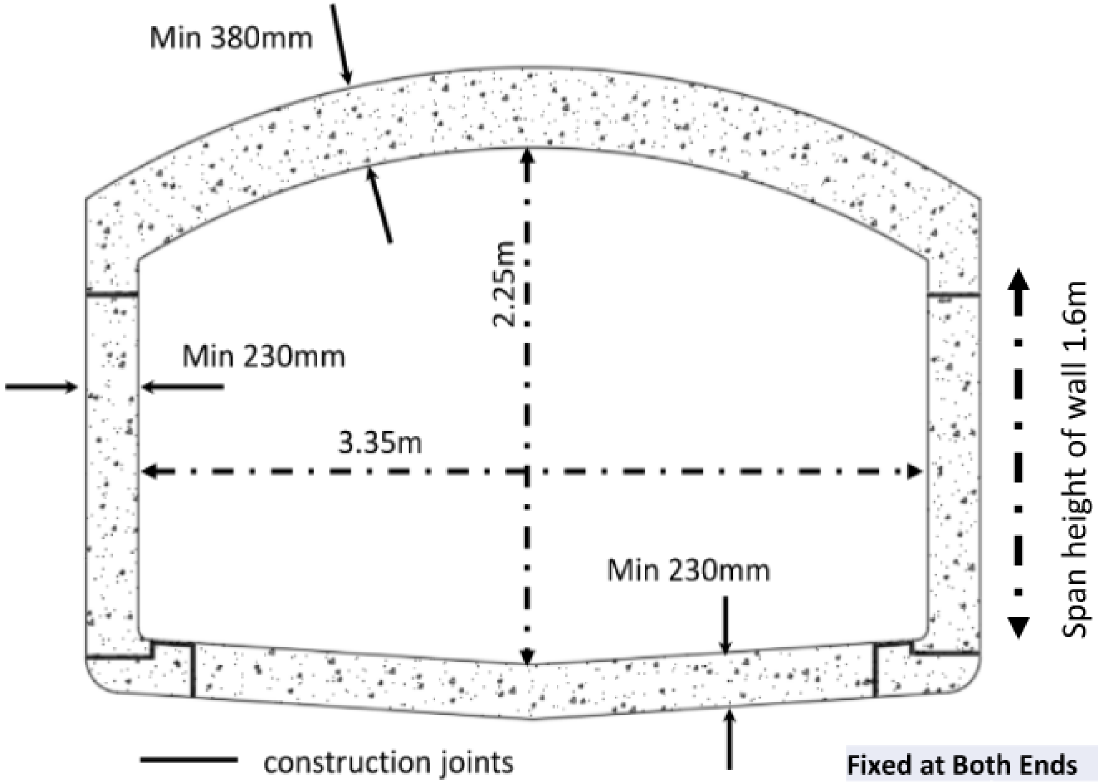
Simplistic Analysis of the Tunnel Invert Slab



Tunnel Section

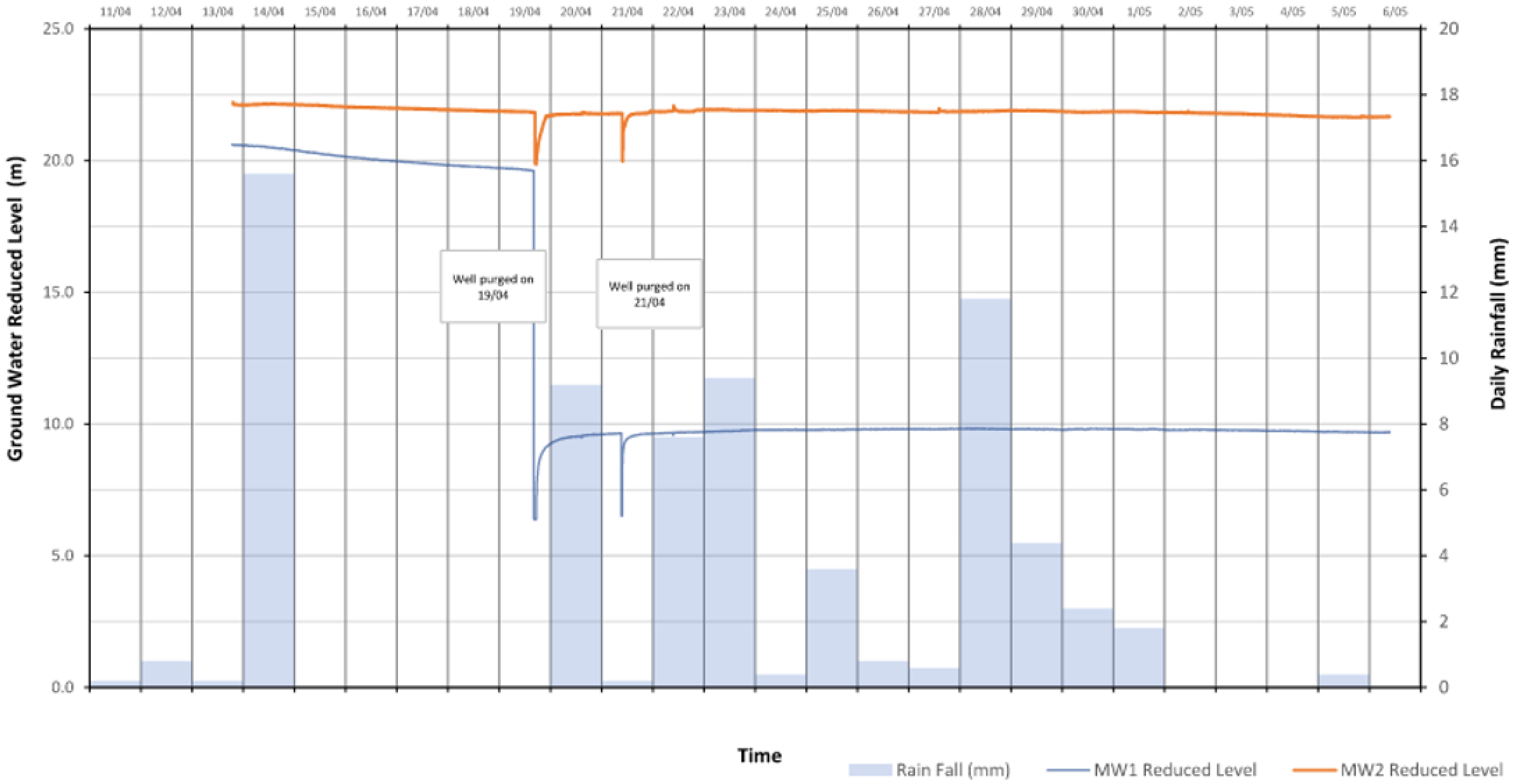
Span = 3350mm
Thickness = 230mm
L = 3350mm
b =1000mm
d =230mm
I = 1.014+09 mm4
Z= 8.817E+06 mm3
Self weigh 5kN/m
Tunnel flow empty

At 2m head of water bending stress exceeds Sydney Water allowable.

Compare 7m head result here with FE analysis results Figure 3

Fixed at Both Ends						
Head Height (m)	W (kN)	Moment mid-span (WL/12), (kNm)	Average Bending Stress (MPa)	Axial Force (kN)	Axial Stress (MPa)	Tensile Stress
1	16.75	4.68	0.53	10.00	0.04	0.49
2	50.25	14.03	1.59	20.00	0.09	1.50
3	83.75	23.38	2.65	30.00	0.13	2.52
4	117.25	32.73	3.71	40.00	0.17	3.54
5	150.75	42.08	4.77	50.00	0.22	4.56
6	184.25	51.44	5.83	60.00	0.26	5.57
7	217.75	60.79	6.89	70.00	0.30	6.59

Table of bending stresses with water depth – mid span

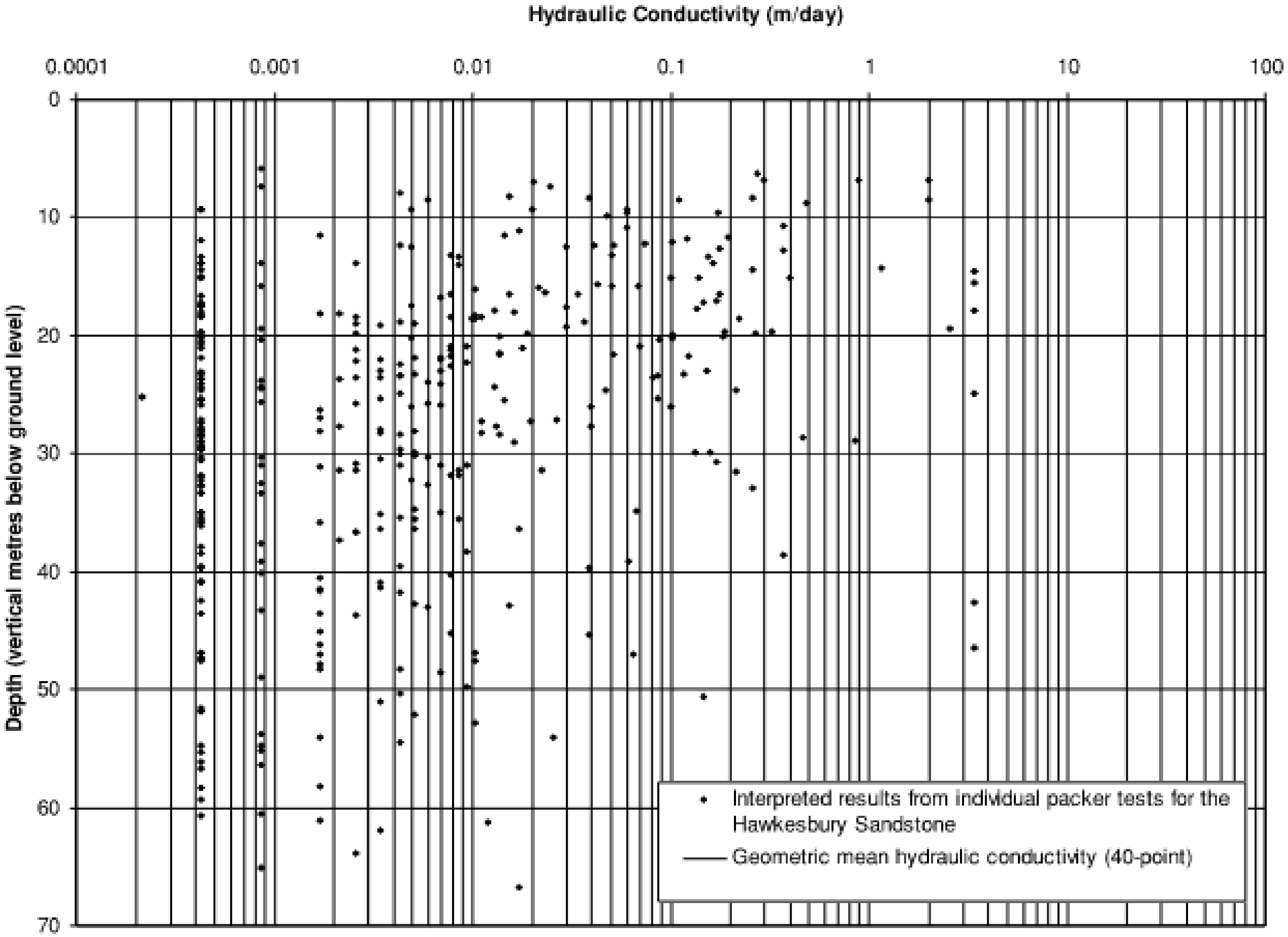


Installation and readings from piezometers

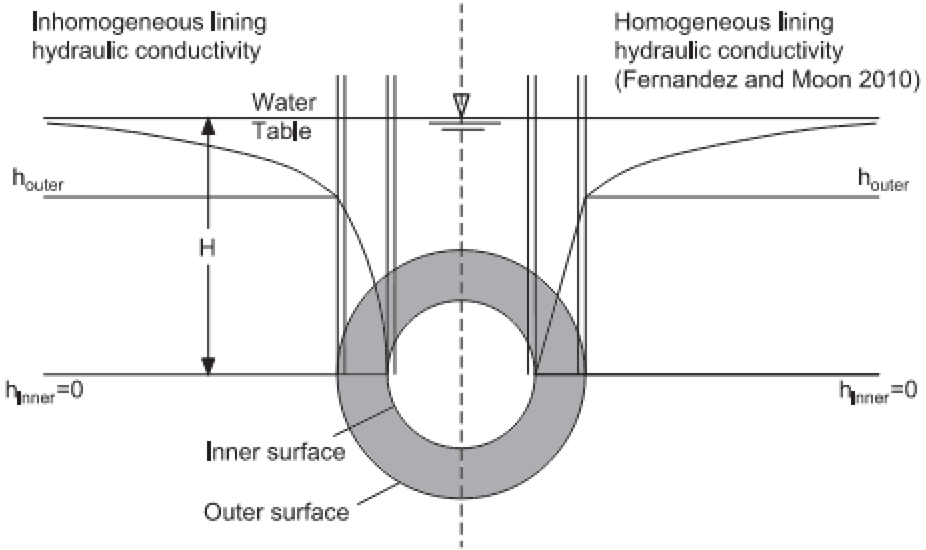
(confirmed predicted perched water table)

The site is on a high point in the area (section A-A)

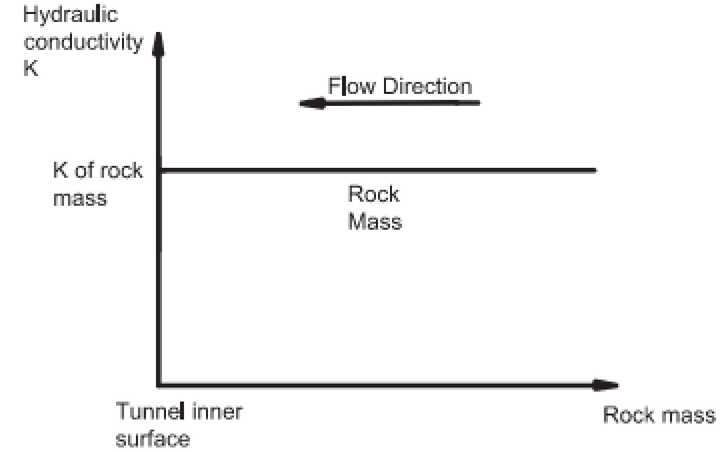
The site is on a high point in the area (section B-B)



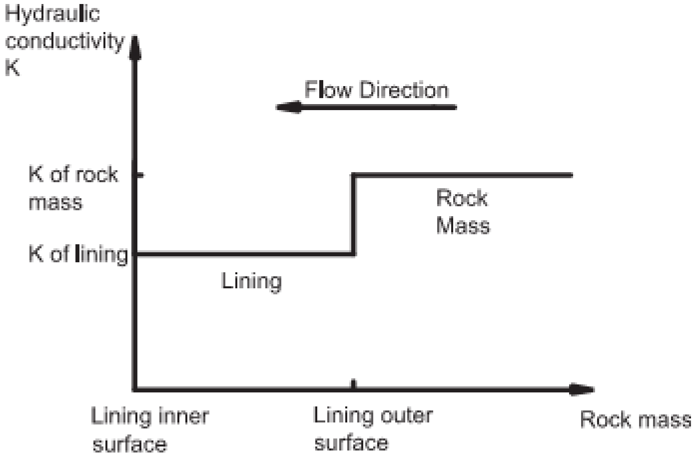
Hawkesbury Sandstone - hydraulic conductivity with depth



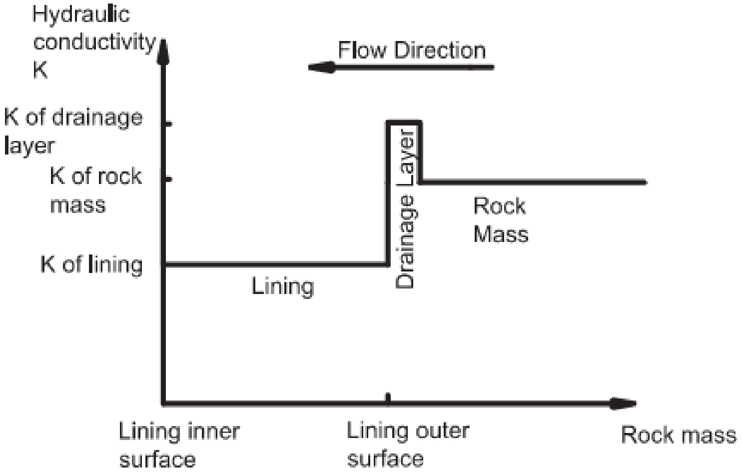
Unlined



Lined
water
sealing



Lined
Drained



The relationship between the hydraulic conductivities of the rock mass and tunnel lining

(knowing the water flow rate in the tunnel provides a partial solution)

The identified risks to the NGRS tunnel are extremely low for the following reasons.

1. The rock surrounding the tunnel has defects that are widely spaced, and the intact rock is of high strength being Class I/II sandstone.

2. The location of the tunnel crown below the basement excavation level of 3m and its lateral offset of 8m from the basement excavation boundary.

3. Potential ground vibration impacts can be managed by using appropriate construction methods and monitoring and using appropriate excavation plant.

4. The FE analysis predict minor ground movements at the level of the tunnel. Stress concentration points may develop in the upper and lower corner of the tunnel lining. Concrete cracking would also be partially mitigated by movement at the construction joints. Cracks widths will be minimal if even visible/measurable in this tunnel environment.

5. External water pressure. The basement excavation because it is drained will maintain a the existing a water table level over the crown of the tunnel whatever the rainfall or ground conditions. Both the piezometer results given in Appendix D and the ground water load calculations in Appendix B when considered together confirm that the tunnel lining has not be impacted by external ground water pressure.

6. Potential structural element instability (walls and invert) due to new cracks will not develop and there will be no change in the ground water regime. Clearly if there with stability issues with this 80-year-old tunnel they would have developed previously. The basement excavation will not change the tunnel environment to any significant degree.

7. The pile loads directly above the tunnel will dissipate into the intact rock mass well above and rock surrounding the tunnel and not impose loads onto the tunnel lining itself.

8. The dilapidation survey concrete coring of the tunnel wall confirms in our opinion that the drill and blast excavation method used to excavate the tunnel with the inherent overbreak in the rock would result in a lining concrete thickness greater than the design thicknesses shown on the Sydney Water sketches provided.