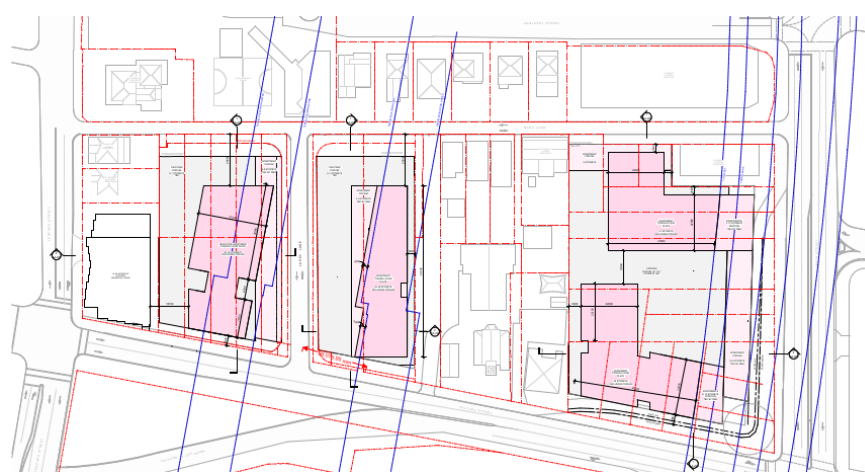


Kangaroo Point Precinct Draft Geotechnical Report - Western (CRR) Side Brisbane, QLD



Prepared for:
Phillip Usher Constructions

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Kangaroo Point Precinct
Draft Geotechnical Report - Western Side
Brisbane, QLD

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I Project Background

Phillip Usher Constructions Pty Ltd (PUC) is in the process of preparing a Development Application (DA) for a new residential development located in Kangaroo Point, QLD. The proposed development will comprise four high-rise residential towers and surrounding podium structures. The exact arrangement of the development including the max building height, number of storeys etc. will be confirmed prior to DA submission. This assessment has considered that the buildings would comprise residential apartments between 30 and 41 levels and that car parking will be provided by a combination of above ground and below ground podium levels.

Parts of the development on the western side, are located above the recently constructed Cross River Rail (CRR) mined cavern, mined running tunnels and Tunnel Boring Machine (TBM) running tunnels. The eastern side of the development is located above the Clem7 TBM tunnels which were completed in 2010. Development of the proposed sites will therefore be required to satisfy the requirements of the CRR and Clem7 infrastructure, to ensure the proposed works do not exceed prescribed limits / criteria.

PUC has engaged EDG Consulting Pty Ltd (EDG) to provide geotechnical analysis and advice relating to the proposed works as part of the DA. The outcomes of our preliminary geotechnical assessment for the western side of the development, located above the CRR cavern and tunnels, are documented herein.

The outcomes for the eastern side of the development, located above the Clem7 tunnels, was presented in a separate geotechnical report Ref. B01554-IAB.

2 Site Location and Description

The proposed residential development is shown in Figure 1 and 2 and is bounded by Mark Lane to the north and Vulture Street to the south. The two eastern towers are bounded by Main Street towards the east. The westernmost tower is bounded by Leopard Street towards the west. The central tower and the two eastern towers are separated by Saint Nicholas Cathedral. Lahey Lane separates the westernmost tower and the central tower.

The site ground surface elevation ranges between approximately RL 18m AHD and RL 12m AHD, generally falling towards the south of the site.

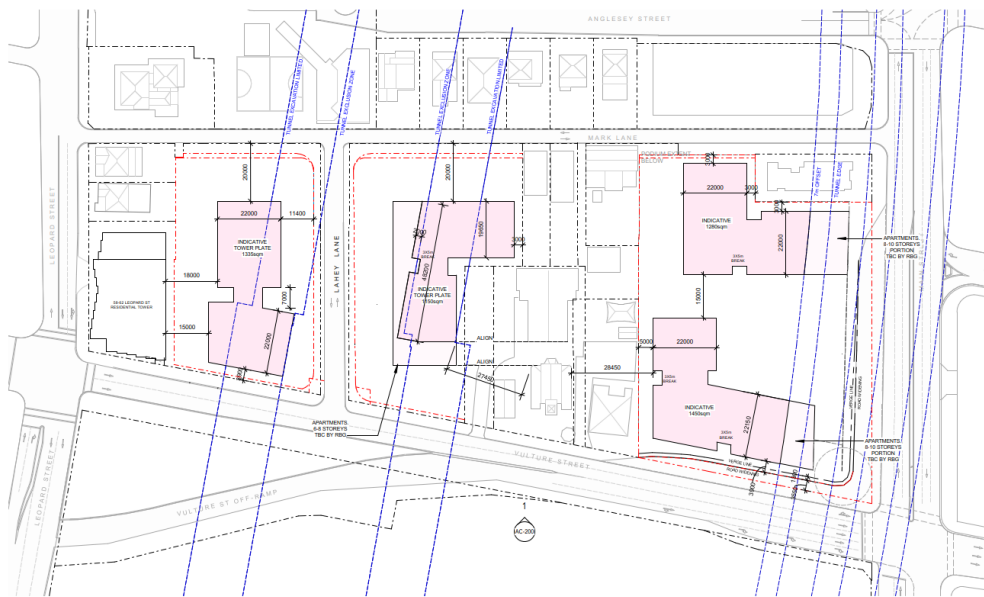


Figure 1 – Site Plan

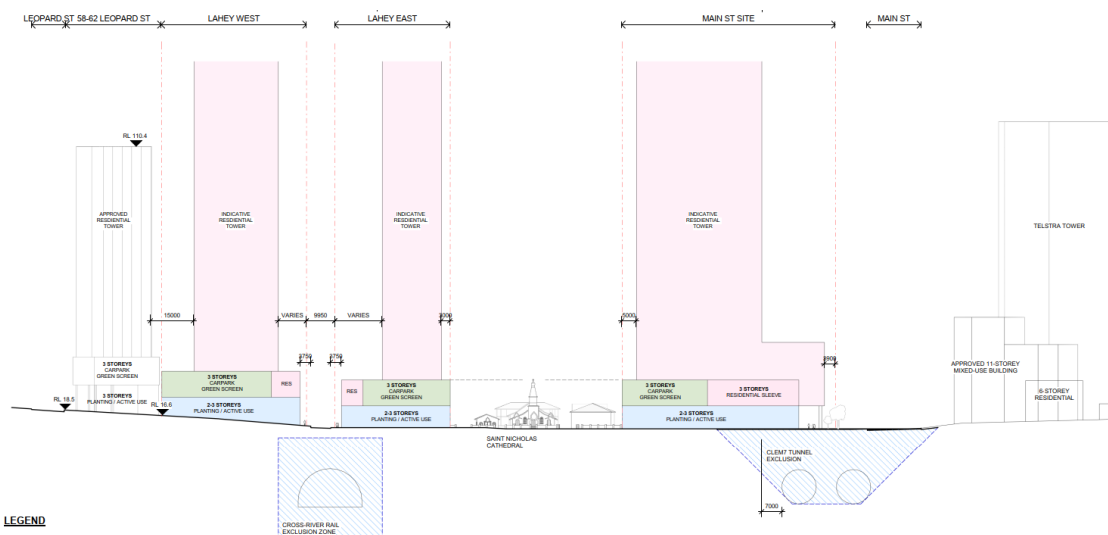


Figure 2 – Site Cross Section

3 Ground Conditions

3.1 Available Geological Information

Based on a review of the 1:100,000 series Brisbane geological map (2015), the site is expected to be underlain by Triassic age Brisbane Tuff, underlain by weathered rock of the Triassic age Aspley Formation, which in turn is underlain by Devonian age, Neranleigh Fernvale Beds. The published geological information is shown as Figure 3.

The following geotechnical investigation information has been supplied to EDG by PUC and has been considered in our interpretation of the site conditions:

- Cross River Rail Project: Tunnel, Stations and Development Package (TSD) – Geotechnical Interpretive Report (GIR) by PSM (Report Ref. CRRTSD-000-0351-RPT-PSMQ-1 | 20-030021) and associated borehole logs.

- Cross River Rail Project: Tunnel, Stations and Development – Factual Report on Geotechnical Investigation by Douglas Partners (Report Ref. 97335.00.R.001.Rev0)
- 58-64 Leopard Street, Kangaroo Point – Report on Additional Geotechnical Investigation by Douglas Partners (Report Ref. 210250.06.R.001.Rev0)
- Proposed Development at Corner of Main Street and Vulture Street, Woolloongabba – Geotechnical Investigation Report by Coffey (Report Ref. GEOTNEWS20843AB-AC)
- 364 Vulture Street, Kangaroo Point – Report on Geotechnical Investigation by Douglas Partners (Report Ref. 97013.01.R.001.Rev0)
- North South Bypass Tunnel – Geotechnical Interpretive Report – Driven Tunnels by Golder Associates (Report Ref. NSBT-0802-GT-RP-055005[05])

A summary of the available geotechnical investigations considered in this assessment is included as Appendix A.

Typically, fill due to previous development activities is present at surface level, which overlies Residual Soil derived from weathered Brisbane Tuff. Beneath the Residual Soil, rock units derived from variably weathered Brisbane Tuff are present. The Brisbane Tuff is underlain by interbedded siltstone and conglomerate of the Aspley formation, which overlies the Neranleigh Fernvale Group.

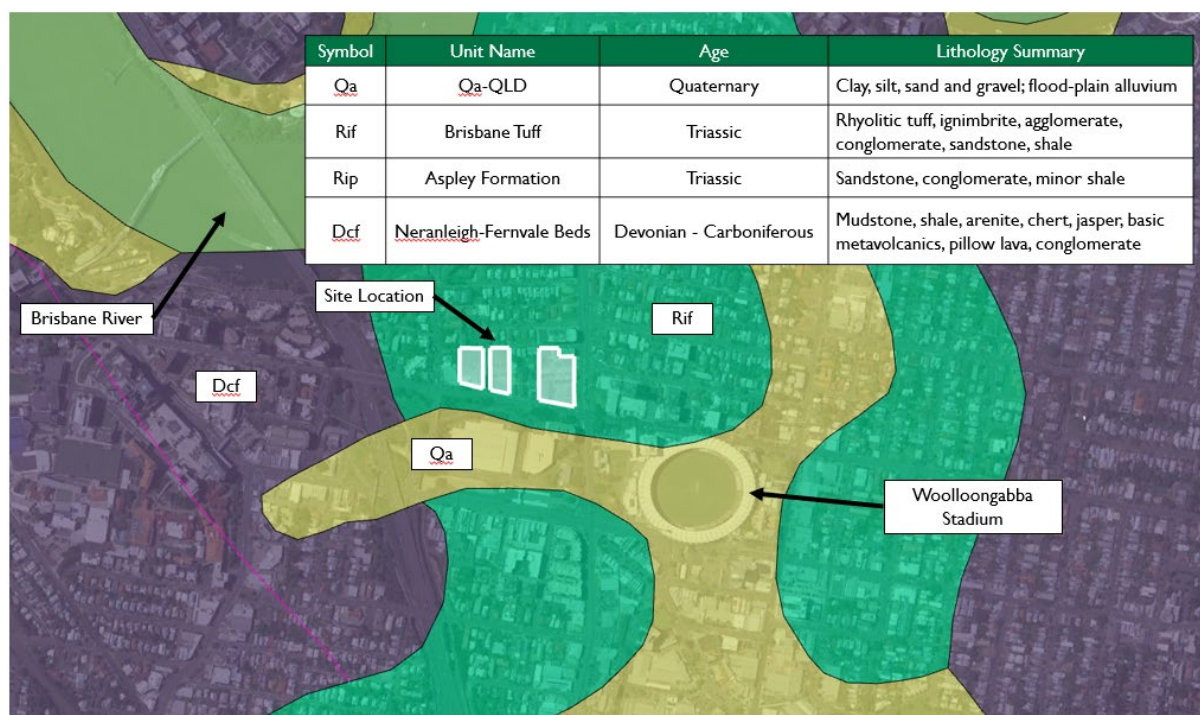


Figure 3 – Published Geological Information

3.2 Material Units

Several material units have been identified based on our interpretation of the available geological information, along with our previous experience with similar projects in the areas surrounding the site. To maintain compatibility between this assessment and the CRR project, we have adopted similar terminologies for the geological units previously identified in the CRR Geotechnical Interpretive

Report by PSM¹. The project material units are summarised below and are discussed in further detail in the following sections.

- Fill (F)
- Residual Soil (RS)
- Very Low to Medium Strength Tuff (Tuff VL-M)
- Medium to High Strength Tuff (Tuff M-H)
- High to Very High Strength Tuff (Tuff H-VH)
- Medium Strength Conglomerate dominant Aspley Formation (CD M)
- Medium Strength Siltstone dominant Aspley Formation (SD M)
- Medium to High Strength Phyllite of the Neranleigh Fernvale Group (NFG M-H)

3.3 Soil Units

3.3.1 Fill (F)

Fill associated with previous site use is present throughout the area. The fill thickness typically varies between 1m and 2m and generally comprises firm to stiff clay and/or loose to medium dense sand and gravel.

3.3.2 Residual Soil (RS)

Residual Soil derived from weathered rock from the Brisbane Tuff rock units underlies the fill material. The Residual Soil generally ranges in thickness between approximately 1m and 5m. The material is typically fine grained comprising a high plasticity clay of stiff to hard consistency, with occasional rock fragments.

3.4 Rock Units

3.4.1 Very Low to Medium Strength Tuff (Tuff VL-M)

Very low to medium strength tuff was typically encountered beneath the residual soil across the site. The material is generally very low to medium strength, is highly to moderately weathered, with defects typically spaced at an approximate frequency of 20mm to 200mm.

3.4.2 Medium to High Strength Tuff (Tuff M-H)

Medium to high strength tuff was typically encountered beneath the very low to medium strength tuff across the site. The material unit is generally medium to high strength and is moderately to slightly weathered, with defects typically spaced between 60mm to 600mm.

3.4.3 High to Very High Strength Tuff (Tuff H-VH)

High to very high strength tuff was typically encountered beneath the medium to high strength tuff across the site. The material unit is generally high to very high strength and is slightly weathered to fresh, with defects typically spaced between 200mm to 1m.

¹ Cross River Rail Project: Tunnel, Stations and Development Package (TSD) – Geotechnical Interpretive Report (GIR) by PSM (Report Ref. CRRTSD-000-0351-RPT-PSMQ-1120-030021)

3.4.4 Medium Strength Conglomerate dominant Aspley Formation (CD M)

Medium strength conglomerate dominant Aspley formation is typically encountered beneath the Tuff H-VH. The material is generally medium in strength and is slightly weathered, with defects typically spaced between 200mm and 2m.

3.4.5 Medium Strength Siltstone dominant Aspley Formation (SD M)

Medium strength siltstone dominant Aspley formation is typically encountered beneath the Tuff H-VH. The material is generally medium in strength and is slightly weathered, with defects typically spaced between 60mm and 600m.

3.4.6 Medium to High Strength Phyllite of the Neranleigh Fernvale Group (NFG M-H)

Medium to high strength phyllite of the Neranleigh Fernvale Group is typically encountered underneath the Aspley Formation. The material is generally medium in strength and is slightly weathered, with defects typically spaced between 500mm and 1m.

3.5 Stratigraphy

Our interpretation of the ground stratigraphy is presented on the interpreted geological cross sections presented in Drawings B01554-IAC_001 to B01554-IAC_005.

4 Geotechnical Lab Testing and Parameters

Geotechnical parameters have been developed adopting a 'moderately conservative' basis for the material units present at the site based on the results of field and laboratory testing, supplemented with published data, correlations and previous experience. The basis of parameter selection is presented in Table I.

Table I – Basis of Geotechnical Parameter Interpretation

Parameter	Basis of Interpretation
Unit weight of soil	Previous experience with similar soils and rock, along with engineering judgement.
At rest earth pressure coefficient (K_0)	Previous experience with similar soils and rock, along with engineering judgement.
Undrained shear strength parameters (S_u)	Interpretation of results of in-situ testing including hand shear vane and pocket penetrometer testing.
Drained shear strength parameters (c' , ϕ')	Results of available triaxial shear strength testing along with previous experience with similar soils and rock, and engineering judgement. Assessment using RocLab based on rock mass characterisation to assess equivalent Mohr-Coulomb parameters.
Unconfined Compressive Strength of Rock Units	Interpretation of available point load and UCS test data.
Soil and Rock Elastic Modulus	Previous experience with similar soils and rock, along with engineering judgement.
Rock Mass Elastic Modulus	Assessment of the rock mass GSI and use of the Generalised Hoek and Diederichs equation (2006).
Poisson's Ratio	Previous experience with similar soils and rock, along with engineering judgement.

Histograms showing the distribution of available Point Load Strength Index (Is_{50}) tests and Unconfined Compressive Strength (UCS) tests for each of the rock units have been included on Figure 4 and

summarised in Table 2. We have adopted a factor of 20 to assess the UCS from Point Load Strength Index, which we consider to be appropriate based on our experience in Brisbane CBD.



Figure 4 – Distribution of UCS and Point Load Strength Index

Table 2 – UCS and Point Load Strength Index Summary

Unit	Typical UCS Range (MPa)
Tuff VL-M	0.6 to 7
Tuff M-H	6 to 24
Tuff H-VH	20 to 80
SD M	8 to 13
CD M	6 to 12
NFG M-H	2 to 25

The recommended material parameters for the nominated soil and rock material units are summarised in Table 3 and Table 4.

Table 3 – Geotechnical properties for soil units

Unit	Materials	Unit Weight	Soil strength materials							
			Undrained Shear Strength	Drained Cohesion	Drained Friction Angle	Undrained Young's Modulus	Drained Young's Modulus	Drained Poisson's Ratio	Over Consolidation Ratio	At Rest Earth Pressure Coefficient
			su (kPa)	c' (kPa)	φ' (Degrees)	E _u (MPa)	E' (MPa)	ν' (-)	OCR (-)	K ₀ (-)
Fill	Firm to stiff clay / loose to medium dense sand and gravel	18	40	0	28	15	10	0.3	N/A	0.5
Residual Soil	Mainly clay (stiff to hard)	19	100	5	28	20	15	0.3	5.0	1.0

Table 4 – Geotechnical properties for rock units

Geological Age	Unit	Sub Unit	Materials	Unit Weight	Rock strength materials												
					Drained Cohesion	Drained Friction Angle	Intact Rock Strength	Intact Rock Stiff.	Geo. strength index	Rock Mass Stiff.	Poisson's ratio	Hoek-Brown param.			Tensile Strength	Insitu Stress Ratio K_h / K_v	
												m_b	s	a		Min	Max
					(kN/m^3)	c' (kPa)	ϕ' (Degrees)	σ_{ci} (MPa)	E_i (GPa)	GSI (-)	E (GPa)	ν' (-)	(-)	(-)	(-)	σ_t (kPa)	(-)
Triassic	Brisbane Tuff	Tuff VL-M	Rhyolitic tuff, ignimbrite, agglomerate, conglomerate, sandstone, shale	24	30	35	4	3	20-35	0.2	0.3	0.535	0.0003	0.526	30	1	1
		Tuff M-H		24	260	50	20	8	45-60	3	0.2	1.403	0.0067	0.504	95	1	2
		Tuff H-VH		24	700	60	40	15	55-75	9	0.2	3.725	0.0205	0.502	220	2	3
	Aspley (conglomerate dominant)	CD M	Conglomerate, sandstone, shale, minor coal	25	200	45	8	5	50-65	2	0.2	2.799	0.0084	0.504	25	1	1
	Aspley (siltstone dominant)	SD M		24	170	45	10	6	40-55	1	0.2	1.994	0.0029	0.507	15	1	1
Devonian - Carboniferous	Neranleigh-Fernvale Beds	Phyllite M-H	Mudstone, shale, chert, jasper, basic metavolcanics, conglomerate	27	400	50	20	12	45-65	5	0.2	3.007	0.0067	0.504	45	1	2

5 Groundwater

We have undertaken a review of the Clem7 Geotechnical Interpretative Report² and the CRR Woolloongabba Permanent Lining Design Report³.

As part of the Clem7 project, groundwater levels across the tunnel corridor had been monitored at selected locations over a time frame from mid-2004 until late 2005. Based on the monitoring and interpretation undertaken during the Clem7 project, the groundwater level is expected to vary between 8m to 15m below existing ground level across the site as shown on the available Clem7 drawings⁴. The report described that groundwater levels will be largely unaffected in the TBM sections except for short periods during construction of the cross passages and substations. Any groundwater level drawdown is expected to only occur for limited periods prior to construction of the tunnel permanent lining.

In the Woolloongabba Permanent Lining Design Report, it was described that the measured groundwater head in the vicinity of Woolloongabba Station is about RL6.0, which is located approximately 5m to 6m below ground surface. The Woolloongabba Station cavern is designed to be a drained structure, and groundwater collection for the drained cavern and adits are achieved using slotted pipes running around the cavern lining and sheet drainage below the full extent of the invert slabs.

The site groundwater conditions are expected to have been affected by construction of the adjacent Woolloongabba Station Cavern. As the Woolloongabba Station has been designed as a drained structure, drainage measures have been installed to prevent build-up of groundwater pressure adjacent to the cavern lining. We have therefore interpreted that the site groundwater levels are locally drawn down within proximity of the Woolloongabba Station Cavern as illustrated in Figure 5. This local drawdown is expected to be limited to the western side of the development (i.e. the western tower and the central tower), while the eastern side that is in proximity of the Clem7 tunnels remains largely unaffected.

It will be required to confirm the local groundwater conditions and the extent of the groundwater drawdown during subsequent design stages. Definition of site groundwater levels would be a requirement of future geotechnical investigations associated with subsequent design phases.

² Golder Associates. August 2009. *North South Bypass Tunnel. Design Lot. 0802. Zone 0 Across All Zones. Geotechnical Interpretative Report – Driven Tunnels*. Report No. NSBT-0802-GT-RP-055005[05].

³ PSM. January 2022. *Permanent Works Design Report. Cross River Rail Project – Tunnel, Stations and Development Package (TSD). Permanent Lining (Tunnel – Woolloongabba)*. Report No. CRRTSD-300-0320-RPT-PSMQ-1330-160087.

⁴ Golder Associates. 2009. *North South Bypass Tunnel Geotechnical Plan and Section*. Drawing Ref. NSBT-0802-GT-DG-055001[02] and NSBT-0802-GT-DG-055002[02].

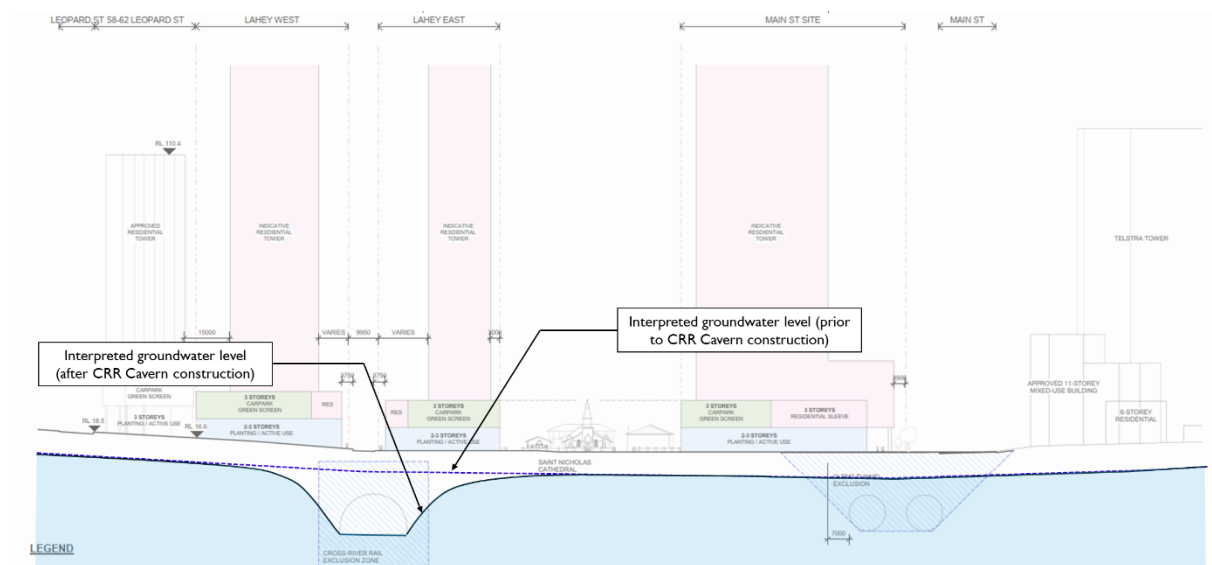


Figure 5 – Interpreted Groundwater Levels

6 Shallow Foundations

The exact foundation arrangement has yet to be confirmed, however, it is possible that shallow footings could be used to support loads from selected tower and podium columns. The elevation of the base of the footings is not confirmed at this stage, however is expected to be approximately at the nominated basement excavation levels of RL 14m, 11m and 5m AHD.

The material at the deeper foundation level of RL 5m is expected to be predominantly either very low to medium strength tuff (Tuff VL-M), medium to high strength tuff (Tuff M-H) or high to very high strength tuff (Tuff H-VH). At the shallower basement levels of RL 14m and RL 11m, it is likely that fill or residual soil will be encountered. Considering the likely magnitude of loads associated with the proposed building, the fill is unlikely to be a suitable founding unit for shallow foundations, and therefore it would likely be required to excavate across the full footprint of the footing to expose the underlying residual soil or very low to medium strength tuff.

We have carried out a preliminary assessment of allowable bearing capacity considering a shallow footing foundation founding in Residual Soil, Tuff VL-M, Tuff M-H and Tuff H-VH. For the soil unit, the assessment is based on the general bearing capacity analysis method proposed by Terzaghi⁵. For the rock units, the assessment is based on the method presented by Wyllie⁶, which represents the strength of the rock mass based on the generalised Hoek Brown failure criterion. The outcomes of our assessment are summarised in Table 5. Note that the allowable bearing capacity has been calculated using a Factor of Safety of 3.0 applied to the ultimate bearing capacity.

Preliminary estimates of settlement and the associated vertical Modulus of Subgrade Reaction have been carried out using the software FLEA (Finite Layer Elastic Analysis) based on the Finite Layer Method. This software accounts for the size and shape of the foundation and layering of the rock profile. The estimates of allowable bearing pressure and associated settlements are shown in Table 5 and have considered footing dimensions of 3m x 3m for individual column foundations.

⁵ Terzaghi, K., 1943. *Theoretical soil mechanics*.

⁶ Wyllie, D.C., 1999. *Foundations on Rock*, 401pp. Spon: New York.

Table 5 – Summary of Shallow Footing Assessment

Material Unit	Foundation Dimensions	Ultimate Bearing Capacity (MPa)	Allowable Bearing Capacity (MPa)	Estimated Vertical Settlement at Allowable Bearing Pressure (mm)	Modulus of Subgrade Reaction NOTE 1 (MPa/m)
Residual Soil	3m x 3m	0.6	0.2	20	10
Tuff VL-M	3m x 3m	1	0.35	3.2	100
Tuff M-H	3m x 3m	9	3	2	1500
Tuff H-VH	3m x 3m	30	10	2.5	4000

Notes: ¹ The structural designer should consider the nominated values of modulus of subgrade reaction together with a sensitivity range of 50% to 200%, (i.e. reduce the modulus of subgrade reaction for all foundations to 50% and increase the modulus of subgrade reaction for all foundations to 200%).

7 Piled Foundations

7.1 Geotechnical Reduction Factor

A geotechnical strength reduction factor of $\phi_g = 0.52$ is recommended at this stage in accordance with the process in AS2159:2009 Piling, based on the quality and quantity of information available.

The geotechnical reduction factor considers that no pile load testing will be carried out during construction, however, it would be possible to increase the geotechnical strength reduction factor should pile load testing be carried out during construction.

7.2 Pile Design Parameters

Design values of ultimate skin friction and ultimate end bearing have been assessed for all soil and rock units. Ultimate skin friction and end bearing have been calculated based on the method proposed by Zhang and Einstein (1998)⁷. Recommended values of ultimate skin friction and ultimate end bearing are summarised in Table 6.

Table 6 – Ultimate Pile Design Parameters

Material	F _s , Ultimate Skin Friction (kPa)	F _{bu} , Ultimate End Bearing (MPa)
RS	50	N/A
Tuff VL-M	500	8
Tuff M-H	1000	20
Tuff H-VH	1500	30

⁷ Zhang, L. and Einstein, H. (1998). "End bearing capacity of drilled shafts in rock." ASCE Jnl. Geot. Eng., Vol. 124 (7), 574-584)

8 Retention System

8.1 Concept Arrangement

At this stage, the arrangement of the proposed retention system is not confirmed, however, it would likely comprise a soldier pile retaining wall or possibly be supported by a combination of soil nails and rock bolts. If the soldier pile arrangement is adopted, it is likely that piled retention system will be supported by several rows of horizontal bracing either from ground anchors, or internal props.

8.2 Preliminary Design Advice

To allow a preliminary sizing and spacing of the pre-stressed anchors and/or props for the south and west faces, an approximation of the lateral earth pressure (unfactored) can be based on the following rectangular lateral earth pressure distribution:

$p = 5H + 0.5q$, where:

p = lateral earth pressure (kPa)

H = depth of retained ground (m)

q = surcharge behind the wall (kPa)

This earth pressure distribution relates to the failure mechanism as illustrated in Figure 6 and assumes that adequate drainage is provided behind the wall such that no hydrostatic pressure build-up occurs.

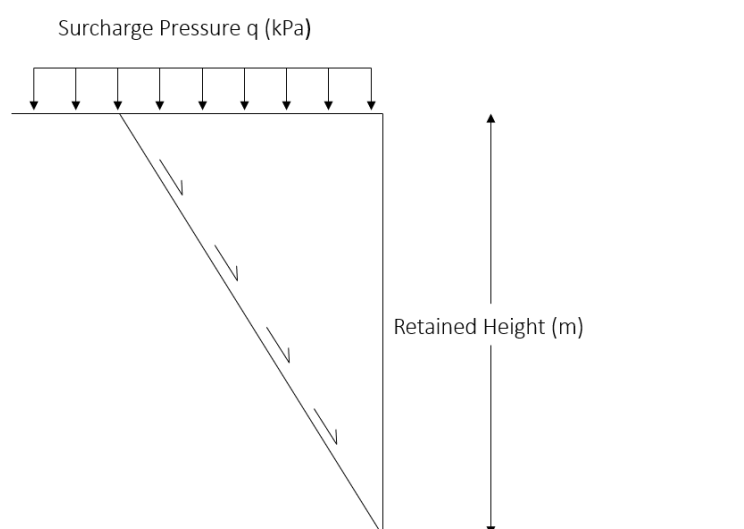


Figure 6 – Failure Mechanism

Earth pressure distributions of this type have been shown to give a reasonable estimate of required anchor pre-stress values or prop forces in propped flexible retaining wall systems. However, estimates of ground movement and structural actions require rigorous soil-structure interaction modelling using finite element methods such as RS2, FLAC or PLAXIS, which would be required during the Detailed Design stage.

Design of anchors may consider the values of ultimate bond stress presented in Table 7. Estimates of ultimate bond stress for fill and residual soil have not been provided as we do not recommend anchoring within those materials. Appropriate reduction factors, given the nature of the loading must

be applied to the ultimate bond stress in the anchor design. The designer must give careful attention to bond zone lengths, hole diameters and the spacing of the anchors to avoid interaction issues.

We recommend that at this stage (in advance of detailed soil-structure interaction modelling of the retention system) a minimum free length extending beyond a 45° plane extending up from the excavation should be adopted for the retention anchors.

Table 7 – Anchor Bond Stress

Material Unit	Ultimate Bond Stress (MPa)
Tuff VL-M	0.4
Tuff M-H	1.5
Tuff H-VH	2.5

Drainage must be provided behind the retention system to permanently relieve any water pressures, and therefore pressure from ground water is expected to be zero under normal service condition. However, to account for potential failure or partial failure of the drainage system (temporary blockage etc.), a water head on the retaining wall equivalent to 1/3 of the retained height should be considered.

8.3 Excavation Staging

Excavation of the proposed basement should be carried out in stages to help control retaining wall deflections and settlement of material behind the retention system.

For propped soldier pile walls, excavation would only proceed following completion of the construction of the retaining wall including its capping beam. Excavation would then proceed but would likely be limited to a maximum of approximately two to three metres followed by installation of the first (upper) row of excavation support and waler beam. Following installation of the upper row of support, excavation would then likely proceed to the approximate level of the subsequent rows of excavation support and waler beam. Excavation would then continue following a sequence of excavation and installation of support, until the maximum bulk excavation level was reached.

9 Temporary Batter Slope Angles

During excavation of the basement, temporary batter slopes may need to be excavated within fill, residual soil, very low to medium strength tuff, medium to high strength tuff and high to very high strength tuff material. The recommended maximum temporary batter slope angles for these materials are summarised in Table 8.

Once the contractor confirms the proposed excavation geometry, slope stability assessments should be carried out by the geotechnical temporary works engineer. Rock face mapping can be undertaken in the tuff material during construction and can be used to provide additional recommendations on slope angles. If required, these slope angles may be further steepened with the support of soil nails or rock bolts.

Table 8 – Recommended Excavation Angles

Material	Time Period	Height (m)		
		≤1.5	1.5 < H ≤ 3.5	>3.5
Fill	Less than 4 weeks	1.5H:1V	1.5H:1V	Not expected due to limited thickness of material
	More than 4 weeks	2H:1V	2H:1V	
RS	Less than 4 weeks	1H:1V	1.25H:1V	Same slope angle as per 1.5m < H ≤ 3.5m. However, all excavations greater than 3.5m in vertical height must have a minimum 1.5m wide bench.
	More than 4 weeks	1.5H:1V	2H:1V	
Tuff (VL-M)	n/a	1H:2V	1H:1.5V	
Tuff (M-H) Tuff (H-VH)	n/a	1H:3V	1H:2V	

10 Excavability

The materials present within the proposed basement excavation are expected to comprise surface fill, residual soils and variably weathered Tuff (Tuff VL-M, Tuff M-H and Tuff H-VH).

We have carried out an assessment of material excavatability based on the method proposed by Pettifer and Fookes⁸ along with the experience of EDG staff gained on similar projects. The methods consider the material rock strength and defect characteristics. The outcomes of the assessment are summarised in Table 9 and shown on Figure 7.

Table 9 – Excavatability Summary

Material Unit(s)	Estimated Excavatability Conditions
Fill Residual Soil	Soil materials within cuts are expected to be readily excavated using excavators.
Tuff VL-M Tuff M-H	Rock excavation is expected to be achievable by hard digging and/or ripping. Rock hammer may be required in areas of higher strength.
Tuff H-VH	Hard or very hard ripping may be required to excavate rock. Rock hammer is expected to be required in higher strength areas and within local excavations such as lift overrun pits and for shallow foundations.

Although not confirmed at this stage, it is likely that lift core overrun pits and local excavations would be included in the building design, that may extend approximately 2m to 3m below the lowest basement slab. Such excavations are expected to be within the high to very high strength rock (Tuff H-VH) and would likely require excavation using rock hammers. Specialist advice relating to the vibrations associated with the use of rock hammers and its impact on the CRR cavern, tunnels and nearby infrastructure may be required.

⁸ Pettifer, G.S. and Fookes, P.G., 1994. A revision of the graphical method for assessing the excavatability of rock. Quarterly Journal of Engineering Geology and Hydrogeology, 27(2), pp.145-164.

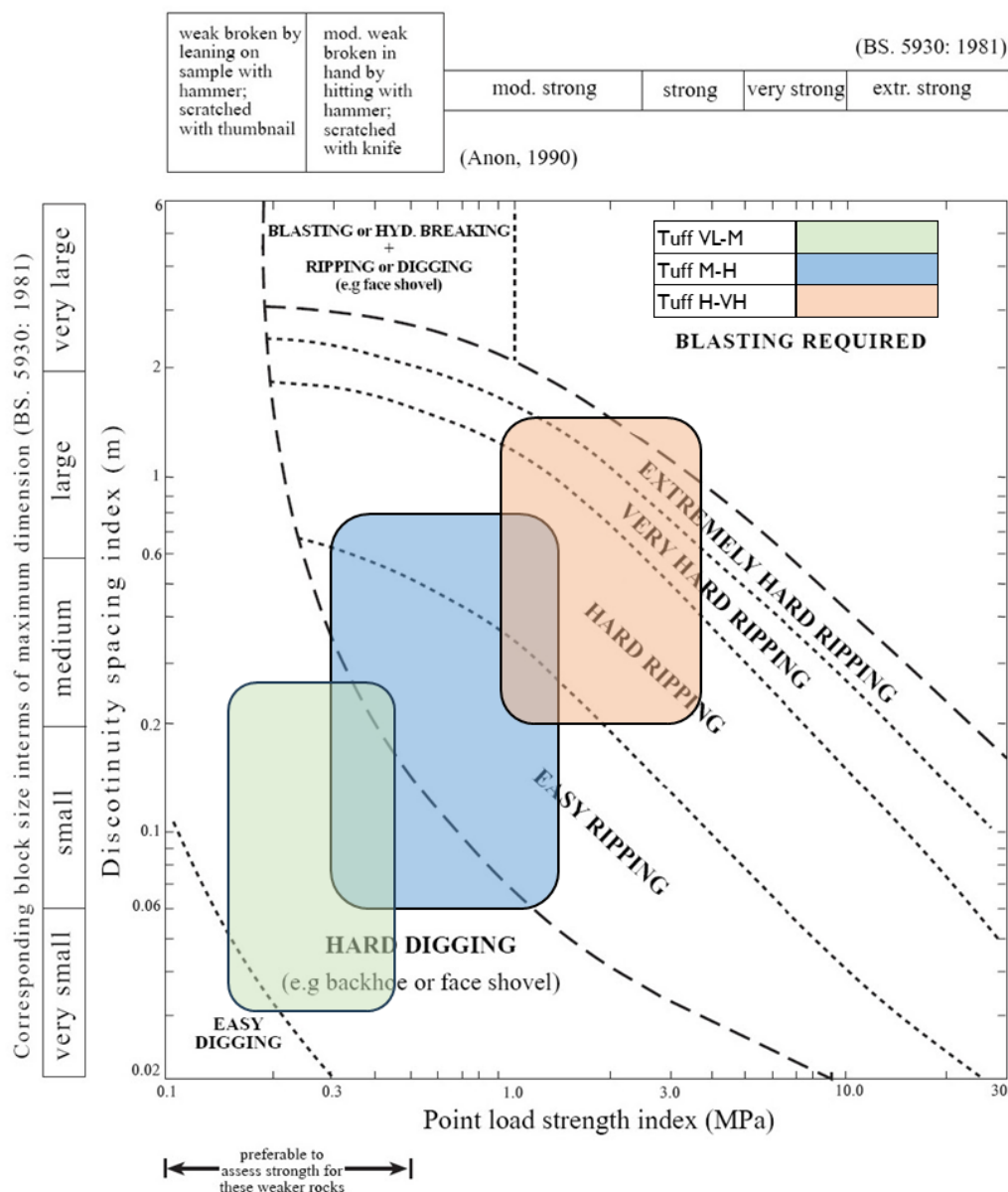


Figure 7 – Excavability Assessment

II Slabs on Ground

II.1 Slabs at Ground Level

The subgrade material present at or close to existing ground level is expected to comprise Fill (F) and/or Residual Soil (RS). Should slabs be required at existing surface level, additional in-situ testing comprising test pits, Dynamic Cone Penetrometer (DCP) and laboratory testing would be required to assess the site California Bearing Ratio (CBR) at the locations of the proposed slabs.

For the purposes of preliminary design, in advance of additional testing, we would recommend that at least 250mm of a pavement quality coarse grained fill (such as an MRTS05 compliant Type 2.3 fill) be placed and compacted beneath the proposed slab, to achieve a subgrade design CBR value of $\geq 3\%$.

11.2 Basement Slabs

The subgrade beneath the proposed basement excavations is expected to comprise variably weathered Tuff, varying from medium to high strength (Tuff M-H), to high to very high strength (Tuff H-VH). Weathered Tuff (Tuff M-H or better) is considered a suitable subgrade for the basement floor slabs, although there is the potential for extremely weathered materials to deteriorate after wetting and/or trafficking. Preparation procedures to be adopted include excavation to design level, the removal of any loose debris or water softened materials followed by a visual assessment to confirm the suitability of the exposed subgrade prior to placement of the drainage layer / blinding concrete.

12 Earthquake Subsoil Class

Based on our interpretation of the ground conditions, the site sub-soil class is classified as Class B_e – Rock in accordance with Section 4 of AS 1170.4-2007, when considering the materials below the proposed foundation level (approx. RL 8m AHD) and below.

The materials above foundation level, i.e. the materials in contact with the retention system are firm to stiff soils and are therefore consistent with Class C_e – Shallow soil site. The structural designer should therefore consider additional damping/amplification effects associated with the soils in the retention system design.

13 Acid Sulphate Soil

We have undertaken a review of acid sulphate soil using the 1:100,000 acid sulphate overlay available online via Queensland Globe. Based on our review the site is not within the acid sulphate soil overlay boundaries and therefore we interpret that acid sulphate soil is unlikely to be present within the site.

During the geotechnical investigation, soil samples will be collected specifically for acid sulphate testing to further assess the presence of acid sulphate soil across the site.

14 CRR FOSD Allowances

Requirements and limitations associated with Future Over Station Development (FOSD) for CRR infrastructure are outlined in the following sections. The CRR infrastructure within proximity of the proposed development comprise:

- Woolloongabba Cavern
- Woolloongabba Stub Tunnels (i.e. TBM launch tunnels)
- TBM Tunnels

14.1 Cavern

The cavern lining design was undertaken by PSM and the basis of design was documented in the technical report titled “Design Manual – Driven Tunnels” (report ref. CRRTSD-000-0300-MAN-PSMQ-1320-100044). The relevant diagrams from this report which relates to the cavern structure are reproduced as Figure 8 and Figure 9.

In addition to the existing rock, earth and groundwater loads, the cavern lining has been designed to accommodate additional loads resulting from future building developments. The allowance considered by the tunnel lining engineer is an additional load of 50kN/m² acting at a reference level of 1m above the cavern crown, as illustrated in Figure 8.

Allowance for excavations permits up to 7m provided at least 10m (height) of residual ground cover exists above the cavern extrados (as defined in Figure 8) and at least 10m (width) of rock pillar exists between the extrados of the side wall of the cavern and any future building basement excavation.

Allowance for additional distortion is shown in Figure 9, where a maximum additional vertical and horizontal distortion limit of 7.5mm, or when sidewall support is relaxed to zero (i.e. no sidewall support), has been considered by the lining designer.

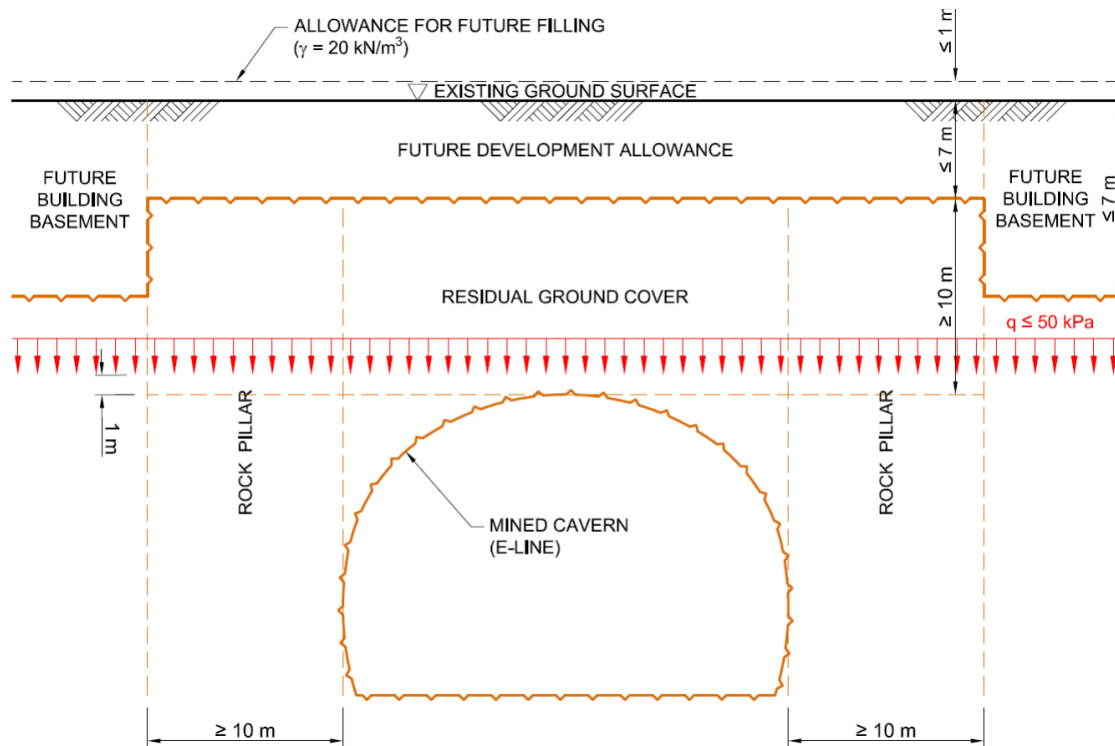


Figure 8 – FOSD Loading for Cavern – Future Development Allowance Case 1 (Figure by PSM)

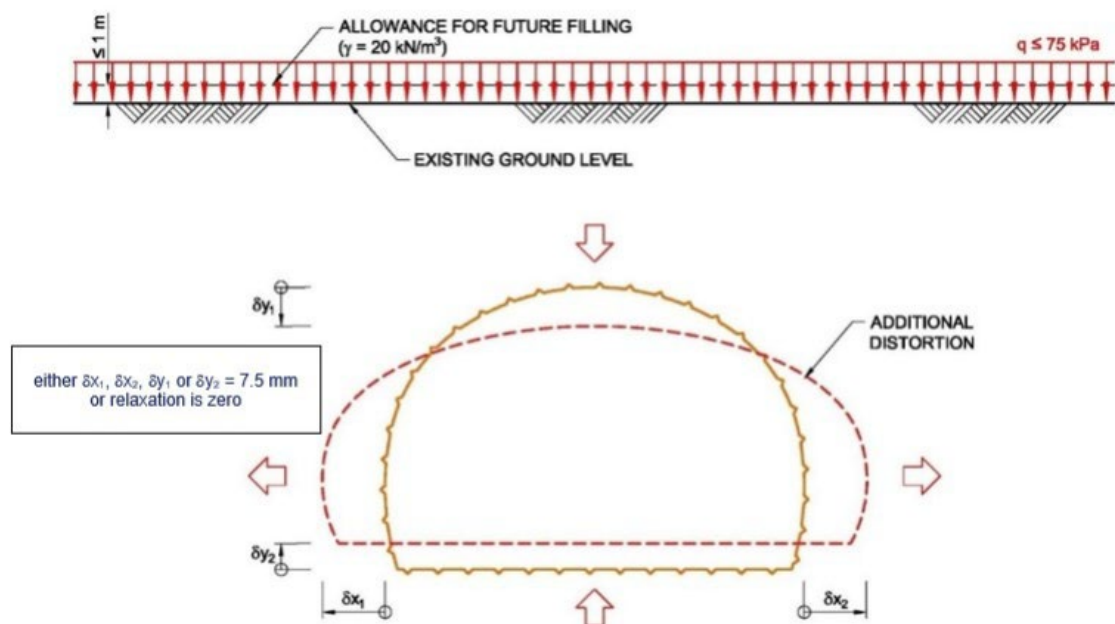


Figure 9 – FOSD Loading for Cavern – Future Development Allowance Case 2 (Figure by PSM)

14.2 Stub Tunnel

The stub tunnels were constructed for the purpose of launching the TBMs. They are approximately 14m long and connect the TBM tunnels with the cavern.

The stub tunnel lining design was also undertaken by PSM and the basis of design was documented in the technical report titled “Design Manual – Driven Tunnels” (report ref. CRRTSD-000-0300-MAN-PSMQ-1320-100044)⁹. The relevant diagrams from this report which relates to the stub tunnels are reproduced as Figure 10 and Figure 11.

In accordance with the Driven Tunnels Design Manual, in addition to the existing rock, earth and groundwater loads, the cavern lining has been designed to accommodate additional loads resulting from future building developments. The allowance considered by the tunnel lining engineer is an additional load of 50kN/m² acting at a reference level of 1m above the tunnel crown, as illustrated in Figure 10.

Allowance for excavations permits up to 7m provided at least 7m (height) of residual ground cover exists above the tunnel extrados (as defined in Figure 10) and at least 7m (width) of rock pillar exists between the extrados of the side wall of the tunnel and any future building basement excavation.

Allowance for additional distortion is shown in Figure 11, where a maximum additional vertical and horizontal distortion limit of 7.5mm, or when sidewall support is relaxed to zero (i.e. no sidewall support), has been considered by the lining designer.

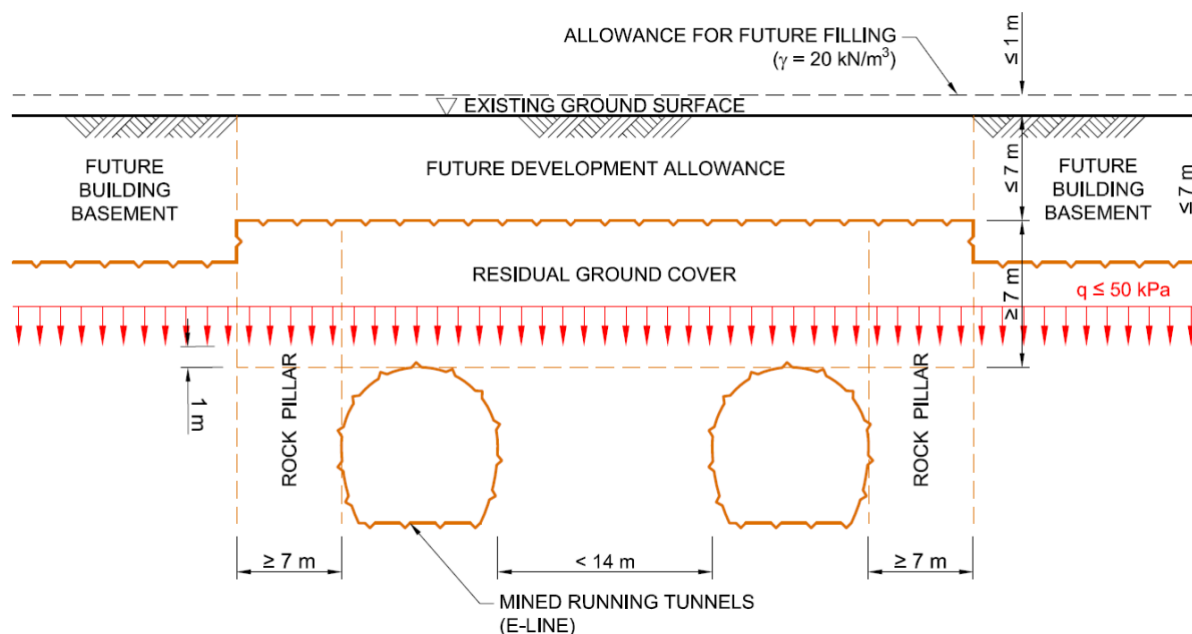


Figure 10 – FOSD Loading for Running Tunnels – Future Development Allowance Case I (Figure by PSM)

⁹ PSM, 7 July 2020. Technical Report. Cross River Rail Project – Tunnel, Stations and Development Package (TSD). Design Manual – Driven Tunnels. CRRTSD-000-0300-MAN-PSMQ-1320-100044. Rev B4.

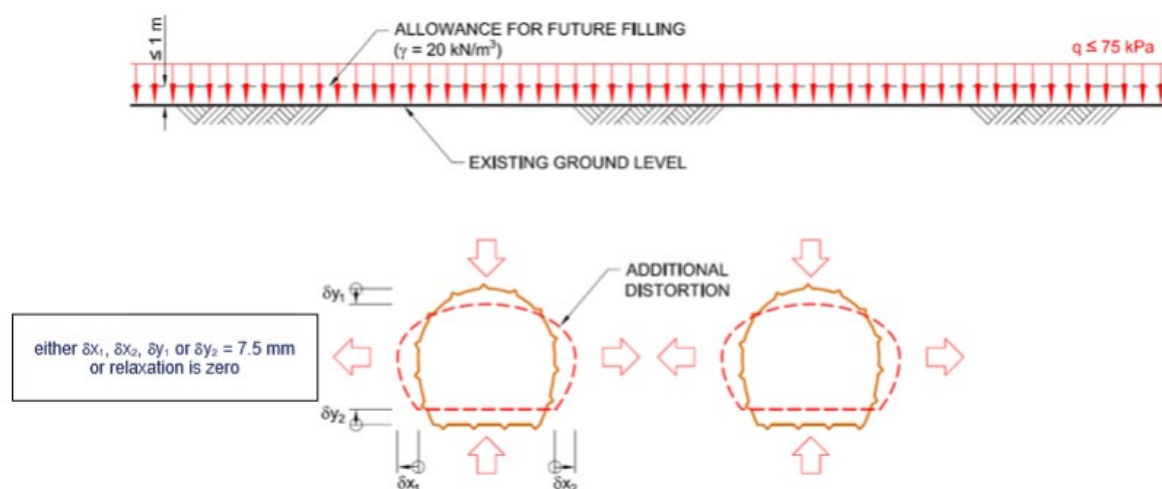


Figure 11 – FOSD Loading for Running Tunnels – Future Development Allowance Case 2 (Figure by PSM)

14.3 TBM Tunnel

The TBM segmental lining design was undertaken by Geodata Engineering and the basis of design was documented in the technical report titled “TBM Segments (Tunnel-Woolloongabba to Northern Portal)” (report ref. CRRTSD-300-0310-RPT-GEOD-1310-0001)¹⁰. The complete set of FOSD load cases considered by the lining designer are shown in Drawing Ref. CRRTSD-300-0310-DRG-GEOD-1310-010202.

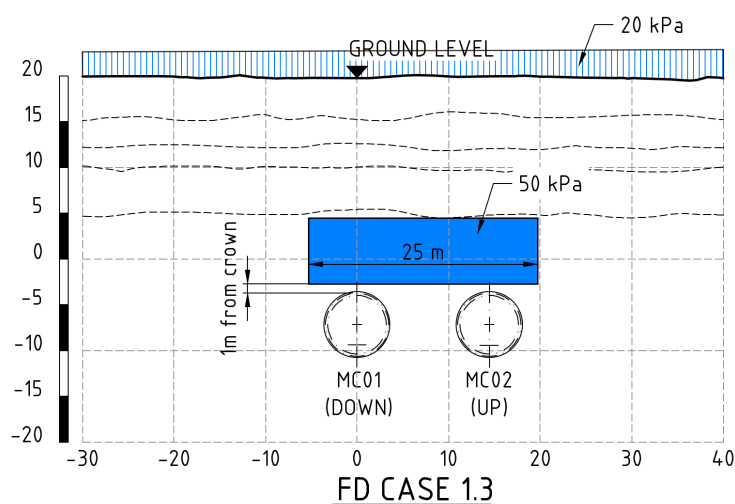


Figure 12 – FOSD Loading for TBM Tunnels – Load Case 1.3 (Figure by Geodata)

¹⁰ Geodata Engineering Pty Ltd, 3 September 2020. Permanent Works Design Report. Cross River Rail Project – Tunnel, Stations and Development Package (TSD). TBM Segments (Tunnel – Woolloongabba to Northern Portal). CRRTSD-300-0310-RPT-GEOD-1310-0001. Rev 00.

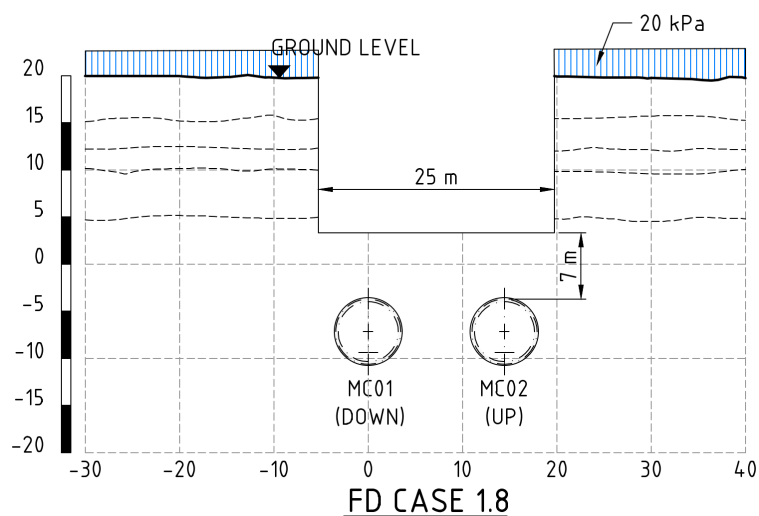


Figure 13 – FOSD Loading for TBM Tunnels – Load Case 1.8 (Figure by Geodata)

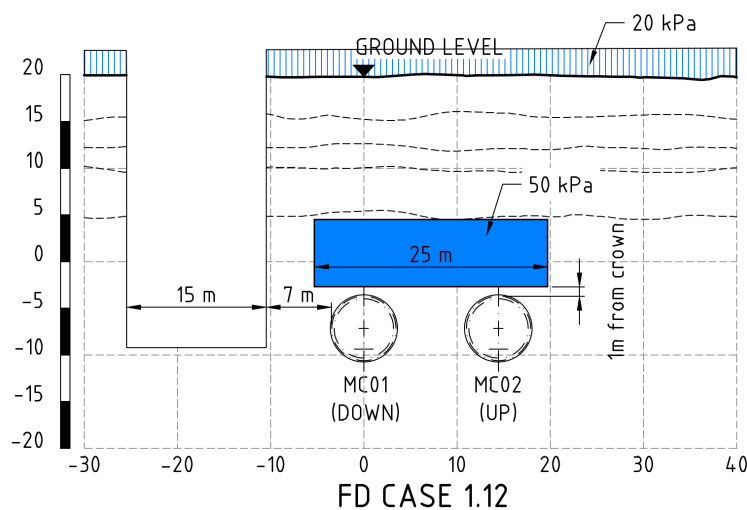


Figure 14 – FOSD Loading for TBM Tunnels – Load Case 1.12 (Figure by Geodata)

In accordance with the TBM segmental lining design report, in addition to the existing rock, earth and groundwater loads, the tunnel lining has been designed to accommodate additional loads resulting from future building developments. The allowance considered by the tunnel lining engineer is an additional load of 50kN/m² acting at a reference level of 1m above the tunnel crown, as illustrated in Figure 12.

Allowance for excavations above the tunnels up to 25m in width permitted, on the basis that at least 7m (height) of residual ground cover exists above the tunnel extrados as illustrated on Figure 13.

Excavations outside of the rock pillar have also been considered in the TBM lining design. Figure 14 illustrates that excavations of 15m (width) that extend down to approximately tunnel invert level, have been considered, provided at least 7m (width) of rock pillar exists between the extrados of the side wall of the tunnel and any future building basement excavation.

15 Preliminary Finite Element Assessment

15.1 Analysis Methodology

The CRR design requirements nominate that design of the cavern, stub tunnel and TBM tunnel lining must consider specific FOSD load cases. The relevant clauses, design reports and drawings for each of the cavern, stub tunnels and TBM tunnels were previously discussed in Section 14.

To provide a comparison between the CRR design criteria and the effects of the proposed building loads, we have developed a preliminary 3D finite element model to assess the ground stresses associated with the proposed building loads at a reference level of 1m above the tunnel crown.

The finite element model is considered as simplified as this stage as the ground model is interpreted from limited geotechnical investigation data, groundwater monitoring data and the structural loads have been generated from an early-stage structural model. In the detailed design stage, the ground conditions would be based on additional geotechnical investigations and the model would incorporate loads from a more developed structural model, which would include wind loads assessed from wind tunnel testing.

The assessment adopts the process shown in Figure 15.

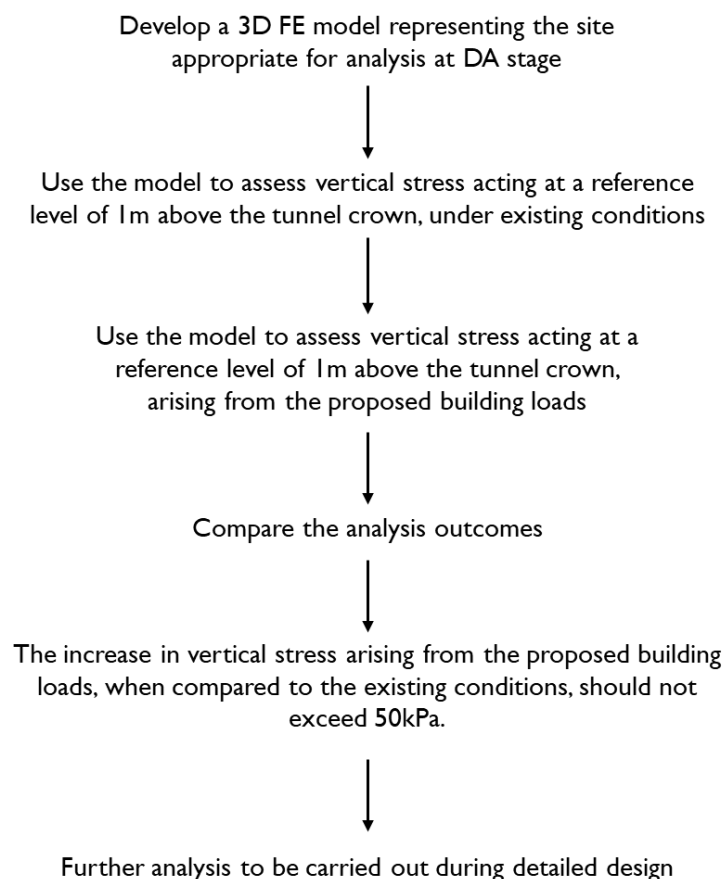


Figure 15 – Analysis Process

15.2 Model Calibration

We have carried out the following comparisons to gain confidence that the modelling and geotechnical analysis carried out as part of the CRR design and the modelling as part of this assessment are similar:

- The ground stratigraphy adopted for this assessment was compared with the ground stratigraphy adopted for the CRR project (as shown in the PSM Geotechnical Interpretative Report¹¹), and both were assessed to be similar at the tunnel location.
- The soil and rock material parameters adopted in our assessment were all similar to those adopted for the CRR project, as presented in the CRR Geotechnical Interpretative Report.

15.3 Finite Element Model Inputs

The preliminary model adopted soil and rock profiles that are consistent with the ground stratigraphy presented on the interpreted geological cross sections presented in Drawings B01554-IAC_001 to B01554-IAC_005.

Soil and rock behaviour were represented by a linear-elastic perfectly plastic continuum constitutive model for all materials. Plasticity was controlled by a stress-dependent Mohr-Coulomb failure criterion for all soil types, based on the material parameters presented in Table 3 and Table 4. Drained shear strength parameters were used for all materials in all stages.

Structural loads were defined by RBG and were applied as point loads at the pile heads or at the column location for the shallow footings. An early stage loading plan showing the magnitude and location of structural loads is provided in Appendix B.

15.4 Groundwater Inputs

For the purpose of this preliminary analysis, we have adopted a groundwater level of RL -16.5m, which aligns with the invert elevation of the drained CRR cavern. We would anticipate modelling a non-uniform groundwater level in subsequent analysis stages.

15.5 Structural Elements

Details of the structural elements used in the analysis are presented in Table 10.

Table 10 – Finite Element Model Structural Elements

Component	Plaxis Element Type	Details
Shallow Foundation Footing – Podium Columns	Plate	Shallow foundation footings for the podium columns are modelled as 2m (length) x 2m (width) x 1.5m (thick) plate elements. An elastic modulus value of 32.8GPa has been assigned to all plates.
Shallow Foundation Footing – Tower Columns	Plate	Shallow foundation footings for the podium columns are modelled as 3m (length) x 3m (width) x 1.5m (thick) plate elements. An elastic modulus value of 32.8GPa has been assigned to all plates.
Core Slab	Plate	Core slabs for the tower cores are modelled as 1.5m (thick) plate elements. The length and width for each core varies but are generally consistent with the footprint dimensions shown on the loading plan (see Appendix B). An elastic modulus value of 32.8GPa has been assigned to all plates.

¹¹ Golder Associates. August 2009. North South Bypass Tunnel. Design Lot. 0802. Zone 0 Across All Zones. Geotechnical Interpretative Report – Driven Tunnels. Report No. NSBT-0802-GT-RP-055005[05].

Component	Plaxis Element Type	Details
Tower Column and Core Piles	Embedded Beam	All tower column and core piles are modelled as 1.2m diameter embedded beams. An elastic modulus value of 32.8GPa has been assigned to all beams. Skin friction values as per Section 7 of this report are adopted for the material units. A reduced skin friction value has been allowed for piles within the steel sleeved zones.
Cavern Liner	Plate	The crown and sidewalls are modelled as a 700mm thick plate element. The base slab is modelled as a 300mm thick plate element. An elastic modulus value of 32.8GPa has been assigned to both plates.
Stub Tunnel Liner	Plate	The stub tunnel liners are modelled as a 270mm thick plate element. An elastic modulus value of 32.8GPa has been assigned to both plates.
TBM Tunnel Liner	Plate	The stub tunnel liners are modelled as a 270mm thick plate element. An elastic modulus value of 32.8GPa has been assigned to both plates.

The finite element model is shown indicatively in Figure 16 and Figure 17.

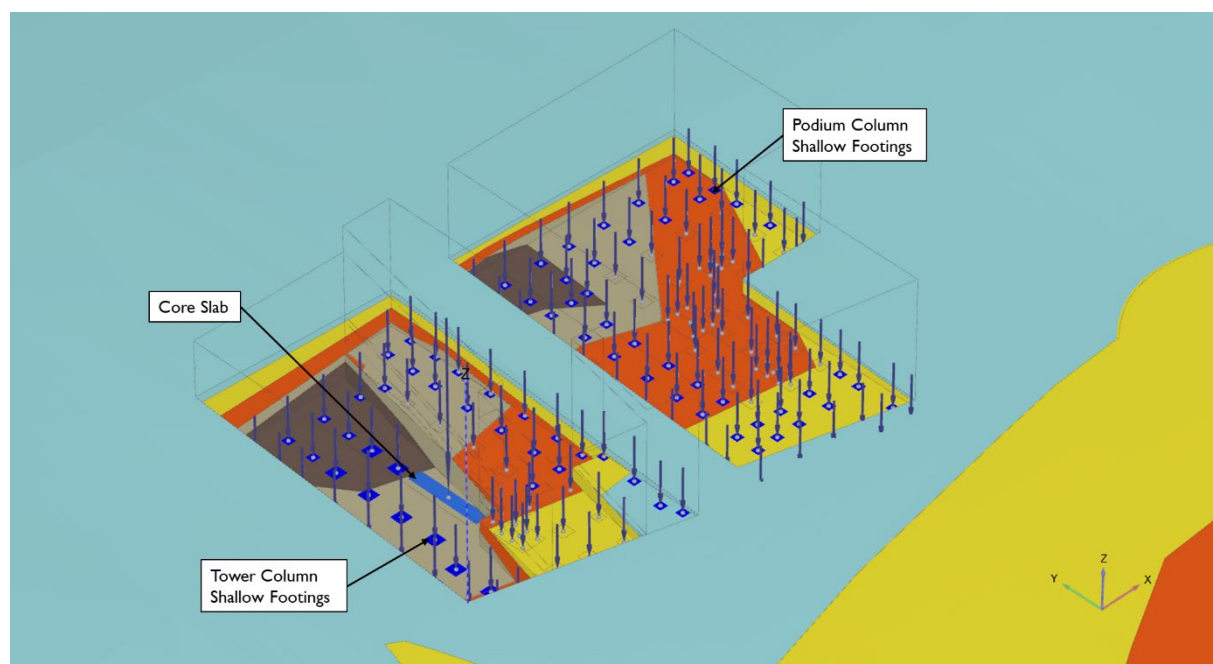


Figure 16 – Finite Element Model (Building Foundations)

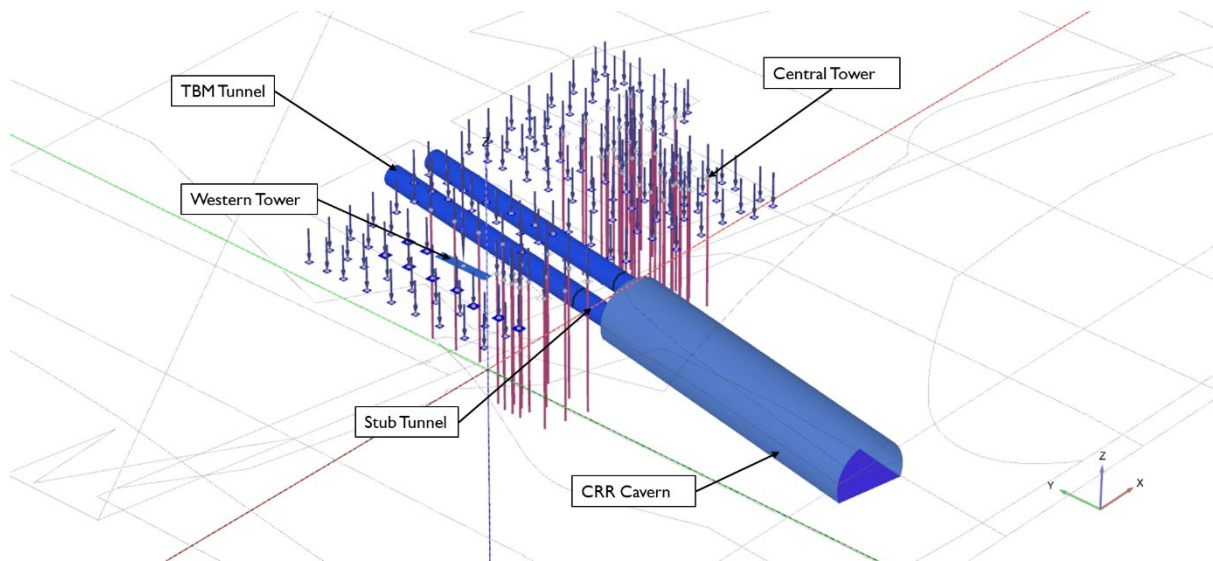


Figure 17 – Finite Element Model (Tunnels and Caverns)

15.6 Construction Stages

The analysis adopted the construction stages as presented in Table 11.

Table 11 – Construction Stages

Analysis Stage	Description	Details
1	Initial Stage	Initial stresses are defined in the model using the K_0 procedure.
2	Cavern Excavation	The cavern is excavated in the model. Stress relaxation of the tunnel is represented using the ΣM_{Stage} factor.
3	Cavern Liner Installation	The cavern liner is activated after stress relaxation from the previous stage. The cavern liner is modelled as plate elements.
4	Stub Tunnel Excavation	The stub tunnels are excavated in the model. Stress relaxation of the tunnels is represented using the ΣM_{Stage} factor.
5	Stub Tunnel Liner Installation	The stub tunnel liners are activated after stress relaxation from the previous stage. The stub tunnel liners are modelled as plate elements.
6	TBM Tunnel Excavation	The TBM tunnels are excavated in the model. Stress relaxation of the tunnels is represented using the ΣM_{Stage} factor.
7	TBM Tunnel Liner Installation	The TBM tunnel liners are activated after stress relaxation from the previous stage. The TBM tunnel liners are modelled as plate elements.
8	Basement Excavation	The basement excavation for the proposed building is excavated.
9	Shallow footings, core slabs and piles (subject to analysis case) installation	Shallow footings, core slabs and piles (subject to analysis case) are installed in a single stage. The shallow footings and core slabs are modelled as plate elements. The individual piles are modelled as embedded beam rows.
10	Apply loading	Individual point loads for the tower columns, podium columns and core slabs, as provided by RBG, are applied.

15.7 Tunnel Construction

We have made allowance for the progressive installation of primary and segmental lining by assessing the stress relaxation prior to support installation following the method by Hoek (2008¹²). This method assesses the proportion of convergence experienced at a point of interest back from the tunnel face. Using this method, we are able to estimate the relative proportion of convergence due to tunnelling that would have occurred prior to installation of the primary and segmental lining.

The proportion of total convergence at each analysis stage is considered in the Finite Element (FE) analysis via the Plaxis “ M_{Stage} ” factor input. The tunnel relaxation depends on several factors associated with the method of construction including the tunnel radius and the unsupported length prior to support installation.

For the stub tunnels, the assessment of the unsupported advance length was based on the details of the primary support design of the tunnel. Three primary support types are nominated for the tunnel, as shown on the CRR drawing ref. CRRTSD-300-0910-DRG-PSMQ-1300-230301. Our calculations indicate that the M_{Stage} factor for the three proposed primary support types varies between 0.8 and 0.9. For the purpose of this preliminary finite element assessment, we have adopted a M_{Stage} factor of 0.85 for the stub tunnels.

For the TBM tunnels, the advance length is typically large compared to the tunnel diameter. As such, significant convergence is expected to occur prior to installation of the segmental lining. Therefore, we have adopted a M_{Stage} factor equal to unity in our finite element assessment, which therefore allows all tunnel convergence to occur prior to installation of the TBM lining.

For the cavern lining, we understand that the primary support design is shown on CRR drawing ref. CRRTSD-300-0319-DRG-PSMQ-1300-150303 and 150304. At the time of writing this report, we do not have access to those drawings. However, we have previously undertaken similar assessment related to cavern structures at other CRR stations for various other development projects. Based on our previous experience and for the purpose of this preliminary finite element assessment, we have adopted a M_{Stage} factor of 0.7 for the cavern.

The M_{Stage} factor adopted in our preliminary assessment for the stub tunnels, TBM tunnels and cavern lining are summarised in Table 12.

Table 12 – Adopted M_{Stage} Factors

Structure	Adopted M_{Stage} Factors
Cavern	0.7
Stub Tunnels	0.85
TBM Tunnels	1

¹² Hoek, E., Carranza-Torres, C., Diederichs, M.S. and Corkum, B. 2008. Integration of geotechnical and structural design in tunnelling. Proceedings University of Minnesota 56th Annual Geotechnical Engineering Conference. Minneapolis, 29 February 2008, 1-53

15.8 Analysis Cases

Two different foundation arrangements have been considered in our assessment:

- **Case 1** – In the first arrangement, all tower columns and podium columns are founded on shallow footings.
- **Case 2** – In the second arrangement, for the western tower, selected tower columns and core structures closest to the CRR cavern/tunnels are founded on sleeved piled foundations, that reduce pile skin friction within the sleeved section of the pile. For the central tower, all tower columns and core structures are founded on sleeved piled foundations.

The tower columns and core which are founded on sleeved piled foundations in Case 2 are indicatively shown in Figure 18.

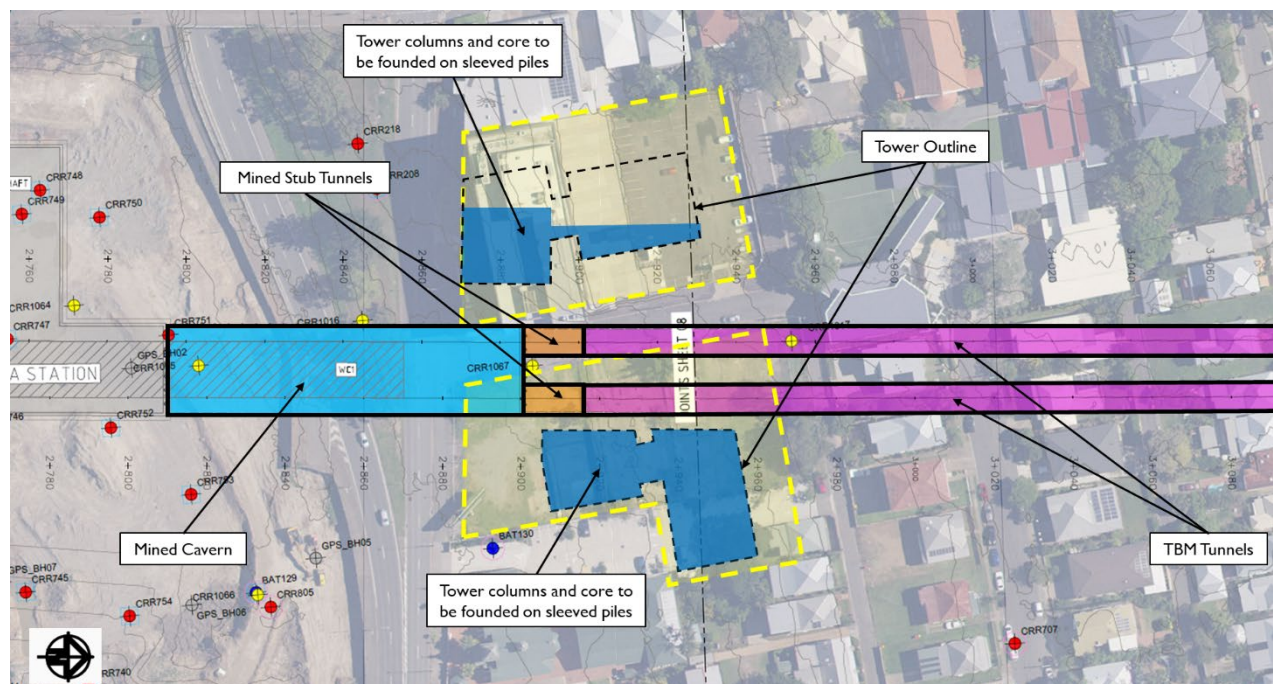


Figure 18 – Building Plan Showing Location of Sleeved Piled Foundations

15.9 Analysis Results

In accordance with the CRR design requirements, we have extracted the vertical stress at a reference level of 1m above the tunnel crown. Vertical stresses have been extracted along three cross sections (refer to Figure C1 in Appendix C), Cross Section A, B and C which intersect the cavern structure, stub tunnels and TBM tunnels, respectively. The results of our assessment are presented graphically on Figure C2 to C4 in Appendix C, and are discussed below.

Values of vertical stress are presented in terms of relative change (or delta). The delta values are the change in vertical stress following excavation of the tunnels (i.e. they reflect the additional vertical stress from the basement excavation and building loads).

Figure C2 shows that the increase in vertical stress within a 10m offset from the cavern wall exceeds the FOSD allowance limit of 50kPa when shallow footings are used to support all tower and podium columns. Similarly, Figures C3 and C4 show that the increase in vertical stress within a 7m offset from the stub/TBM tunnel walls also exceeds the FOSD allowance limit of 50kPa when shallow footings are used.

However, when sleeved piled foundations are used to support selected tower columns and cores as described previously in Section 15.9, the increase in vertical stress within a 10m offset for the cavern, and within a 7m offset for the stub/TBM tunnels, respectively, are generally within the FOSD allowance of 50kPa.

We note that for Cross Section B as shown on Figure C3, the increase in vertical stress slightly exceeds the FOSD allowance of 50kPa in the rock pillar located between the stub tunnels, 1m above the tunnel crown. However, this is limited locally within the pillar, and it is also noted that the loads considered in this assessment are preliminary, subject to further updates during the detailed design phase. In addition, this assessment has not considered an assessment of tunnel distortions, which may be considered in subsequent stages of assessment to assess the effects on the tunnel linings.

15.10 Discussion

The results indicate that the use of shallow footings to support all columns will not be suitable as the increase in vertical stress will likely exceed the FOSD allowance. To satisfy the CRR design requirements, it is likely that the use of sleeved piled foundations will be required to support selected tower columns and cores closest to the CRR tunnels. It is expected that the use of shallow footings to supported the relatively lightly loaded podium structure will be acceptable.

The exact arrangement of the sleeved pile foundation will be further developed and confirmed during the detailed design stage. It will likely comprise of a bored pile with the upper section lined with a permanent steel liner. In the lined upper section of the sleeved pile, axial transfer via skin friction will be limited due to the presence of the permanent steel liner. In the unlined lower section of the sleeved pile, full axial transfer via skin friction to the surrounding rock will be developed. It is expected that at least the length of pile that extends above the 1H:1V line of influence as illustrated in Figure 19 will be required to be lined with permanent steel liners.

In the upper section of the sleeved piles where lining is present, the extent of skin friction transfer will vary depending on the specific arrangement of the permanent steel liner and the adopted construction methodology. If there is complete uncoupling and separation of the outer permanent steel liner and the inner concrete pile, there will no transfer of skin friction. The use of a compressible bentonite grout between the annulus of the outer permanent steel liner and the inner concrete pile will likely result in a small amount of skin friction transfer. Direct physical contact between the outer permanent steel liners and the inner concrete pile will result in the highest skin friction transfer.

In our preliminary assessment, we have adopted a skin friction of 25kPa in the lined upper section of the sleeved piles. The exact sleeved pile arrangement will be finalised during subsequent design stages, however based on the preliminary outcomes of this assessment, the sleeved piles will be designed to limit skin friction within the sleeved section of the piles to <25kPa.

The level of analysis carried out in this assessment is considered to be appropriate for the Development Application stage and for a proof of concept assessment. Further work will be required during detailed design including further development of the geotechnical model and updates to the finite element model to incorporate loads from a more developed structural model.

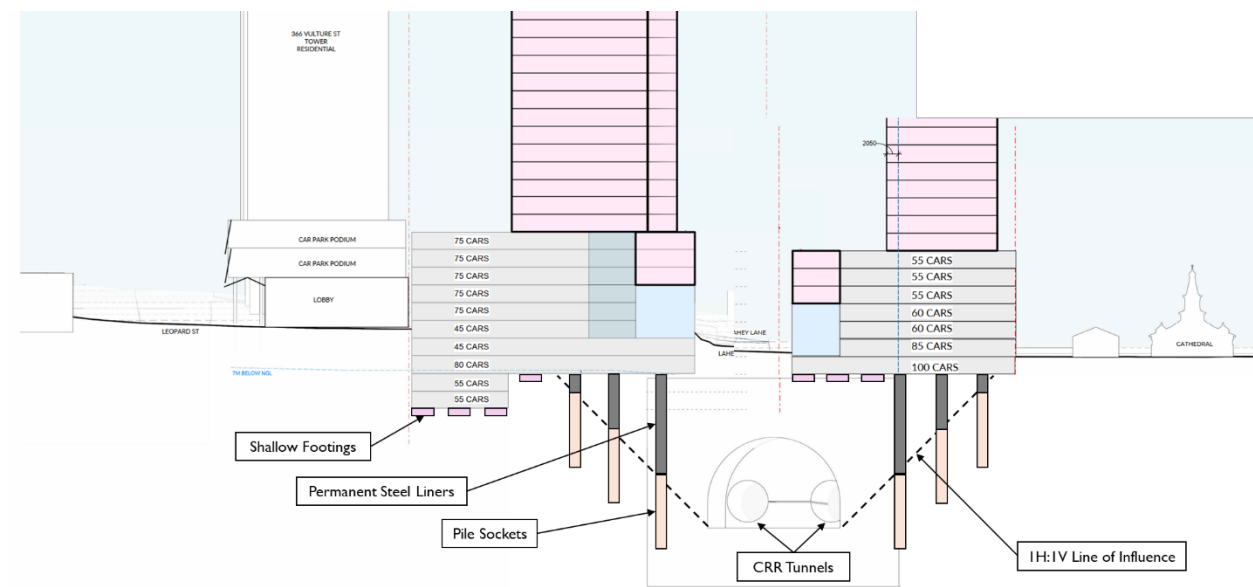


Figure 19 – Indicative Foundation Arrangement

15.11 Two-Dimensional vs Three-Dimensional Finite Element Assessment

To gain further confidence in the validity of the vertical stress obtained from the 3D finite element analysis, we have simultaneously undertaken a 2D finite element analysis in parallel using Plaxis2D. This approach allows for comparison of results between the two models. In the 2D finite element model, structural loads were simplified to on a per m run basis.

The podium loads were modelled as uniformly distributed loads, while loads from the tower columns and core were modelled as line loads. Similar to the 3D finite element analysis, the piles were modelled as embedded beam rows and the shallow footings were modelled as plates. In the 2D finite element assessment, we have adopted the same soil and rock material parameters as per the 3D finite element model, previously presented in Table 3 and Table 4.

In the 2D finite element model, the position of the cores, tower columns, podium loads and tunnels were based on the geometry along Cross Section D as shown in Figure D1 of Appendix D. Outputs from the 3D finite element model were extracted along this cross section for comparison purposes. The location of the comparison section was selected because it intersects two of the heavily loaded cores.

In accordance with the CRR design requirements, we have extracted the vertical stress at a reference level of 1m above the tunnel crown. Values of vertical stress are presented in terms of relative change (or delta). The outputs from the 2D and 3D finite element model are presented in Figure D2.

The results presented in Figure D2 suggest that the change in vertical stress between the 2D and 3D assessments are similar and follows the same general trend. It is observed that the peaks and troughs are more pronounced in the 2D analysis compared to the 3D analysis. However, this can be attributed to the plane strain nature of the 2D analysis, and its limitation in load spreading across all three dimensions. In both models, the increase in vertical stress within a 7m offset for the TBM tunnels are within the FOSD allowance of 50kPa when sleeved piled foundations are used to support selected tower columns and cores as described previously in Section 15.9.

Based on this comparison, we believe that the vertical stress obtained from the 3D finite element assessment is valid and supported by the results of the simpler 2D finite element assessment.

16 Construction Comments

The geotechnical aspects of the works are expected to encounter several challenges during construction. A commentary on construction issues / requirements identified at this stage is provided in the following sections.

16.1 Piling

At this stage the retention system walls may comprise soldier piles. Excavation associated with construction of the retention system is expected to extend through fill, residual soil and variable weathered rock units, increasing in strength from very low to high. Verification of pile socket material as well as socket sidewall and base cleanliness would be required during construction by an experienced geotechnical engineer.

16.2 Shallow Footings

Several shallow footings may be constructed to support the permanent building loads. Following excavation of the footings, geotechnical assessment must be carried out by an experienced geotechnical professional to verify that the design requirements have been achieved. Assessments would be carried out using visual and tactile methods.

16.3 Anchors

Construction of ground anchors should be carried out in accordance with an appropriate specification such as TfNSW B114.

Anchors should be proof loaded and tested during construction to ensure that design loads are being achieved. At least three sacrificial proof tests should be carried out in each unit that the anchors are bonded into (Tuff VL-M, Tuff M-H and Tuff H-VH). Every anchor should be subject to acceptance test and/or suitability test depending on the specification adopted for the works.

17 Geotechnical Investigations

The advice presented in this document has considered the available geotechnical investigation data predominantly undertaken as part of the CRR and Clem7 project. Additional geotechnical investigations will therefore be required to verify the geotechnical conditions considered in the building design and will be carried out during subsequent design stages.

The actual scope of the additional geotechnical investigation will be confirmed during the geotechnical investigation phase, however is expected to comprise approximately 18 rotary cored boreholes and in situ and laboratory testing to provide information on soil and rock strength, stiffness parameters. To assess groundwater levels, standpipe piezometers will be installed in several of the rotary cored boreholes. The approximate locations of the 18 additional proposed investigations are shown in Figure 20 in blue. Existing boreholes are shown in white.



There are several key geotechnical risks that are identified at this stage and will be investigated further in the detailed design stage. A commentary on key geotechnical risks and limitations associated with this stage of geotechnical assessment is provided in Table 14.

Issue	Discussion
Interpretation of ground conditions	To help reduce uncertainty and risk associated with the interpretation of ground conditions, additional geotechnical investigation is recommended and is expected to be carried out during subsequent design stages. It is noted that no geotechnical investigation data exists in the north west corner of the site. Confirmation of ground conditions in this area will require additional geotechnical boreholes during subsequent design stages.
Groundwater levels	It is expected that the site groundwater conditions have been affected by construction of the adjacent Woolloongabba Station Cavern. The extent of groundwater drawdown due to the construction of the drained cavern is unclear. It will be required to confirm the local groundwater conditions during subsequent design stages. Definition of site groundwater levels would be a requirement of the additional geotechnical investigation.
Surface settlements behind retention system	Note that no detailed soil-structure interaction analyses for the retention system have been carried out at this stage and therefore ground surface settlements will be assessed during subsequent design stages for assessment of the performance of any sensitive buried infrastructure.
Ground-borne vibrations affecting tunnels	The proposed construction works may cause ground-borne vibrations that have the potential to affect the cavern and tunnel linings. We would recommend that specialist advice is sought on ground-borne vibrations relating to the specific site details and proposed construction equipment be sought

Issue	Discussion
	during the detailed design stage. Construction stage work may include monitoring of vibrations on-site associated with bored piling and rock excavation activities and re-calibrating the vibration levels for the observed measurements.
Lateral loads	Lateral structural loads have not been considered in this assessment. It is intended that lateral resisting core structures will be strategically positioned outside of the rock pillar zones to minimise impacts and load transfer to the tunnel. This will be further assessed during subsequent design phases.

For and on behalf of EDG Consulting Pty Ltd


David Cunliffe
Principal

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EDG will not be liable to update or revise the document to take into account information any events or circumstances or facts occurring or becoming apparent after the date of the report.

All site conditions cannot be identified

The scope of work undertaken represents a professional assessment of the information cited to develop a basic geotechnical model of the site based on EDG's understanding of the client's risk profile. In some cases, increasing the frequency of investigations and/or sampling, or considering alternative investigation techniques may improve the interpretation, but may not identify all relevant subsurface conditions at the site.

The document presents an interpretation

Geotechnical information is an interpretation of conditions evident based on a limited number of facts established during a site investigation. Engineering logs are an interpretation of observations of samples and test results at discrete locations in the subsurface profile. A geotechnical model is an interpretation of site conditions, developed using information from discrete locations on the site and an understanding of geological processes. Interpreted conditions at and between investigation locations may be different to those inferred on the engineering logs and geotechnical model. The client must consider how variations in conditions could affect the project and seek advice to reduce risk if it is unacceptable to the client.

Conditions can change

The geotechnical information provided is based on the conditions observed at the time of the investigation. Such conditions may be time dependent and subject to external influences. Many things could influence the site conditions, including geological processes, variation in groundwater or surface water levels, other natural cycles and influence from human activities (on this site or nearby sites). Specific advice should be sought if conditions on site change from those observed at the time the report was prepared.

How to deal with different site conditions

The sub-surface conditions on the site may not be as inferred in this report. Geotechnical uncertainties can be managed throughout the project life cycle, but particularly during construction.

Knowledge of site conditions must be further developed as the ground is exposed during construction and/or operation. It is essential that the client implements the nominated design and construction requirements, including observation, interpretation and assessment of the exposed conditions during construction and operation using skilled staff familiar with the design assumptions and assumed geotechnical conditions, or engaging EDG to undertake this role on your behalf. EDG will not be held liable in any way from such misinterpretation.

Drawings

B01554-IAC_001 – Site Plan

B01554-IAC_002 – Cross Section A

B01554-IAC_003 – Cross Section B

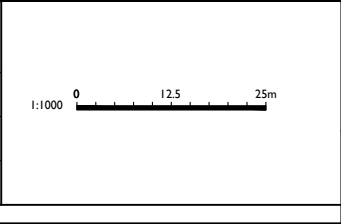
B01554-IAC_004 – Cross Section C

B01554-IAC_005 – Cross Section D



A	INITIAL ISSUE	HF	30/01/2024	DJC
Revisions/Descriptions		Drawn	Date	Approved

SURVEY DATA	
Coordinate System	
Height Datum	
Base Plan	



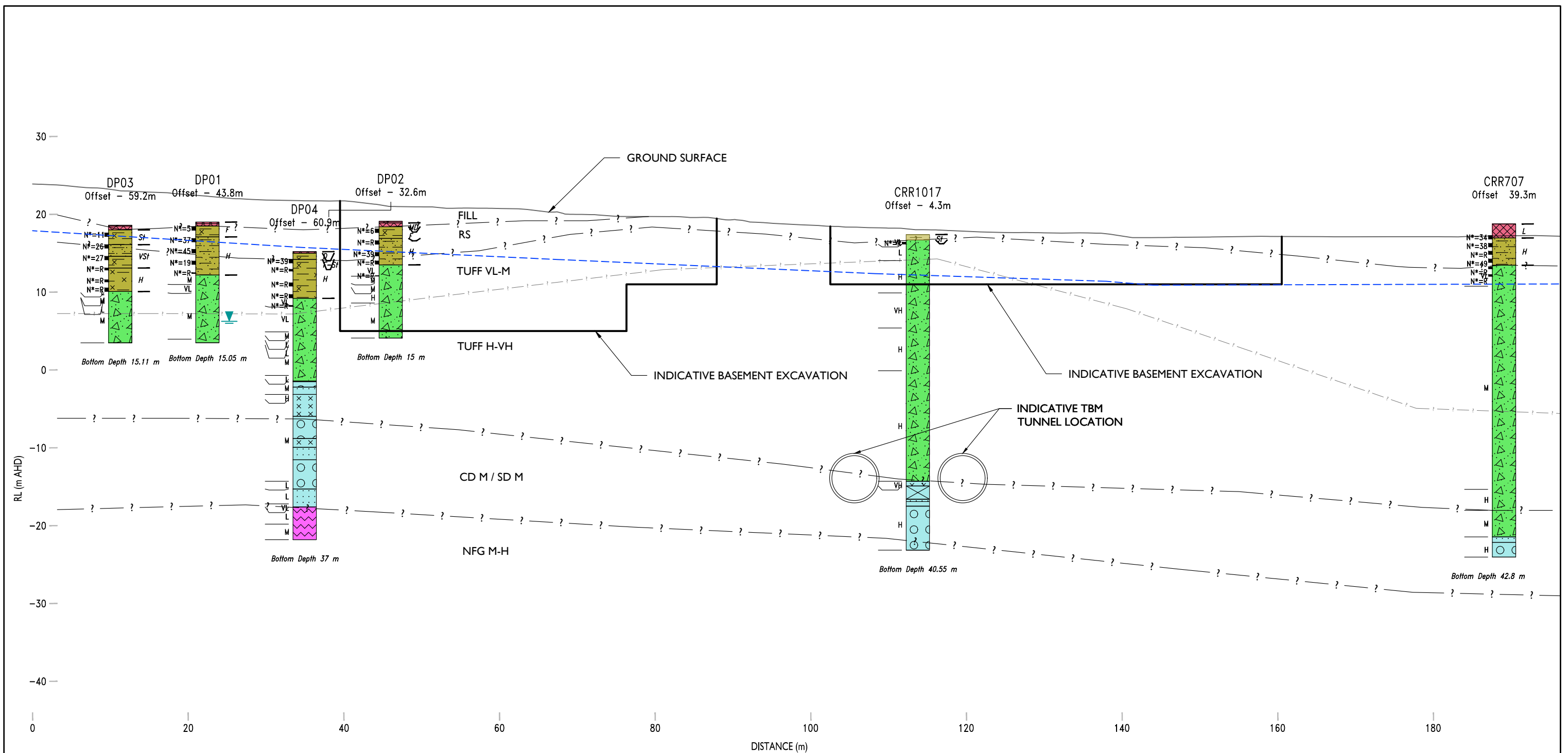
Dimensions shown in metres except where shown otherwise ENGINEERING CERTIFICATION (RPEQ)				
ENG. AREA	NAME	SIGNATURE	NO.	DATE

Drawn	HF
Checked	DJC
Designed	HF
Verified	DJC



PHILIP USHER CONSTRUCTION KANGAROO POINT PRECINCT KANGAROO POINT, BRISBANE SITE PLAN WESTERN END		
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Job No.	B01554-I	
DRG No.	B01554-IAC_001	A
Client Ref.	Client Ref.	
File Location: F:\Projects\B01017-1.....		



LEGEND

Water Level

$N^*=17$ Standard Penetration Test Results

$pp=40kPa$ Undrained Shear Strength from Pocket Penetrometer

VL Rock Strength (see explanation sheets)

VS/MD Consistency / Density

FILL

FILL

CONCRETE

SOIL

SAND

SILT

CLAY

SILTY CLAY

SANDY CLAY

ROCK

TUFF

SANDSTONE, MEDIUM, MEDIUM - COARSE GRAINED

SILTSTONE

CONGLOMERATE

ARGILLITE

PHYLLITE

CORE LOSS

INTERPRETED MAJOR UNIT BOUNDARY

INTERPRETED SUB UNIT BOUNDARY

INTERPRETED GROUNDWATER LEVEL

FILL

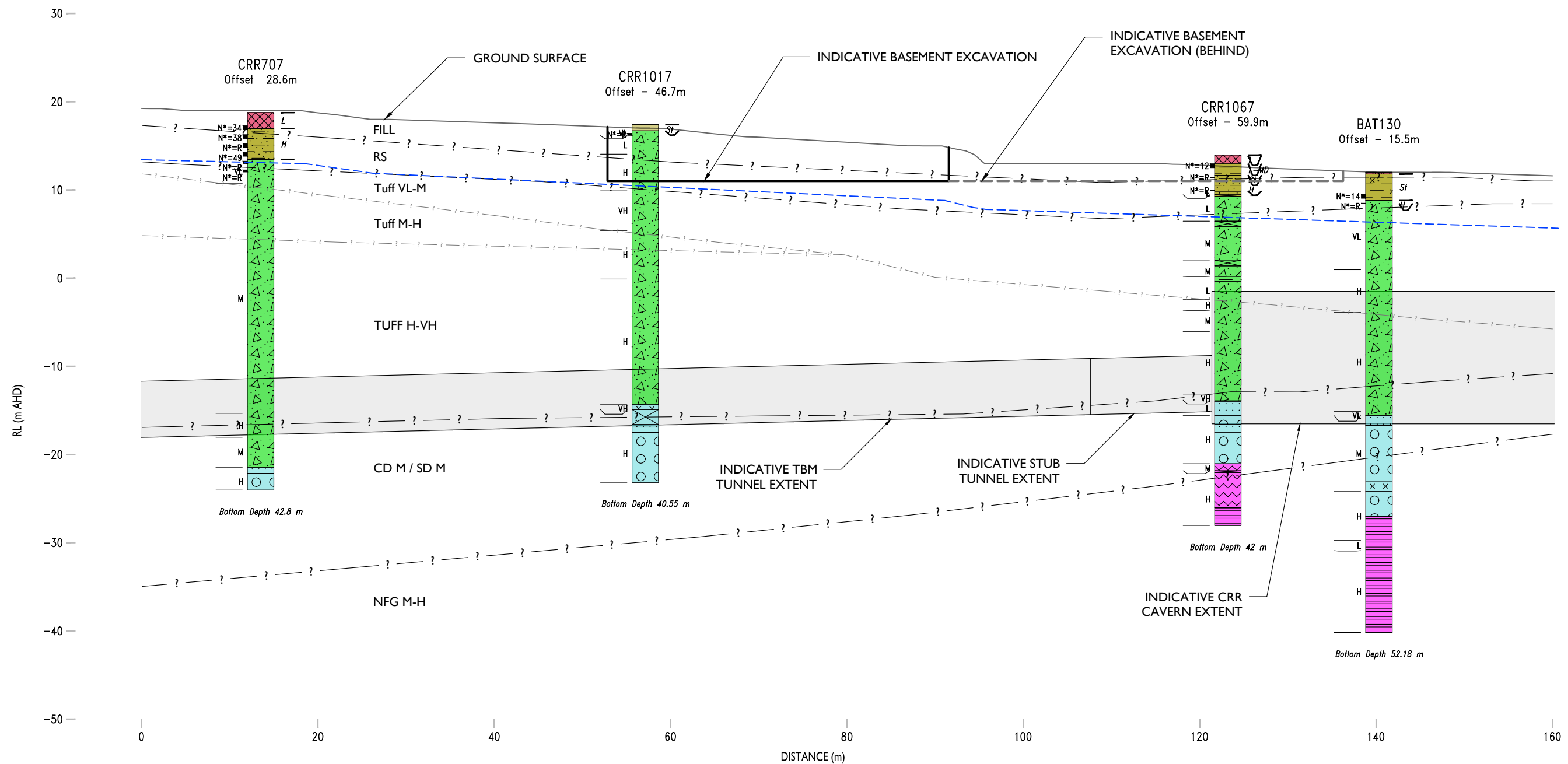
RS

TUFF

CD M/SD M

NFG M-H

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LEGEND

▽ Water Level
N*=17 Standard Penetration Test Results
pp=40kPa Undrained Shear Strength from Pocket Penetrometer
VL Rock Strength (see explanation sheets)
VS/MD Consistency / Density

FILL

□ FILL
□ CONCRETE

SOIL

□ SAND
□ SILT
□ CLAY
□ SILTY CLAY
□ SANDY CLAY

ROCK

□ TUFF
□ SANDSTONE, MEDIUM, MEDIUM - COARSE GRAINED
□ SILTSTONE
□ CONGLOMERATE
□ ARGILLITE

□ PHYLLITE
□ CORE LOSS

— ? — ? — INTERPRETED MAJOR UNIT BOUNDARY
- - - - - INTERPRETED SUB UNIT BOUNDARY
- - - - - INTERPRETED GROUNDWATER LEVEL

□ FILL
□ RS
□ TUFF
□ CD M/SD M
□ NFG M-H

SURVEY DATA

Coordinate System
Height Datum
Base Plan

1:500 0 6.25 12.5m

Dimensions shown in metres except where shown otherwise
ENGINEERING CERTIFICATION (RPEQ)
ENG. AREA NAME SIGNATURE NO. DATE

Drawn HF
Checked DJC
Designed HF
Verified DJC

EDG
consulting
engineering | design | geosciences

PHILIP USHER CONSTRUCTION
KANGAROO POINT PRECINCT
KANGAROO POINT, BRISBANE
CROSS SECTION B
WESTERN END

Job No. B01554-I
DRG No. B01554-IAC_003 A
Client Ref. Client Ref.
File Location: F:\Projects\B01017-1.....

Appendix A

Available Geotechnical Investigations

Test Location ID	Easting (GDA94)	Northing (GDA94)	Termination Depth (m)	Surface RL (m)
BAT130	503391.1	6959941.2	52.18	12
BH01	503514	6959924	6	11.5
BH02	503506	6959938	15	12.5
BH03	503483	6959924	12	11
BH04	503455	6959923	20	11
BH05	503462	6959985	20.48	16.5
BH06	503528	6959985	15	17
CRR1017	503359.88	6960024.38	40.55	17.398
CRR1067	503346.714	6959958.328	42	13.953
CRR208	503296.44	6959926.28	42.7	14.15
CRR218	503285	6959923	44.33	14.31
CRR707	503435.2	6960068	42.8	18.76
DP01	503268.6	6959984.9	15.05	19
DP02	503284.2	6959996.1	15	19.1
DP03	503257.4	6959969.5	15.11	18.6
DP04	503281.1	6959967.8	37	15.2
DP05	503268.6	6959955.6	37.25	18.1
NST65	503560	6960182	40	15
NST66	503529	6959812	32.8	8.5
Pit07	503490	6959949	0.65	13.2
Pit08	503473	6959970	2.5	16

Appendix B

Early-Stage Preliminary Load Plan

Residential Tower:

- All loads are unfactored (kN).
- Load Assumption:
Residential) LL: 1.5 KPa, SDL: 1.5KPa
Car park) LL: 2.5 KPa, SDL: 0.5KPa

Core Wall thickness: 400mm
Slab thickness: Average 200mm
Story height: 3m

Podium (5-Story):

- All loads are unfactored (kN).
- Load Assumption:
Car park) LL: 2.5 KPa, SDL: 0.5 KPa
Retail) LL: 5 kPa, SDL:1.5 KPa

Slab thickness (Carpark): Average250mm
Slab thickness (Retail): Average300mm
Retail: Double Height

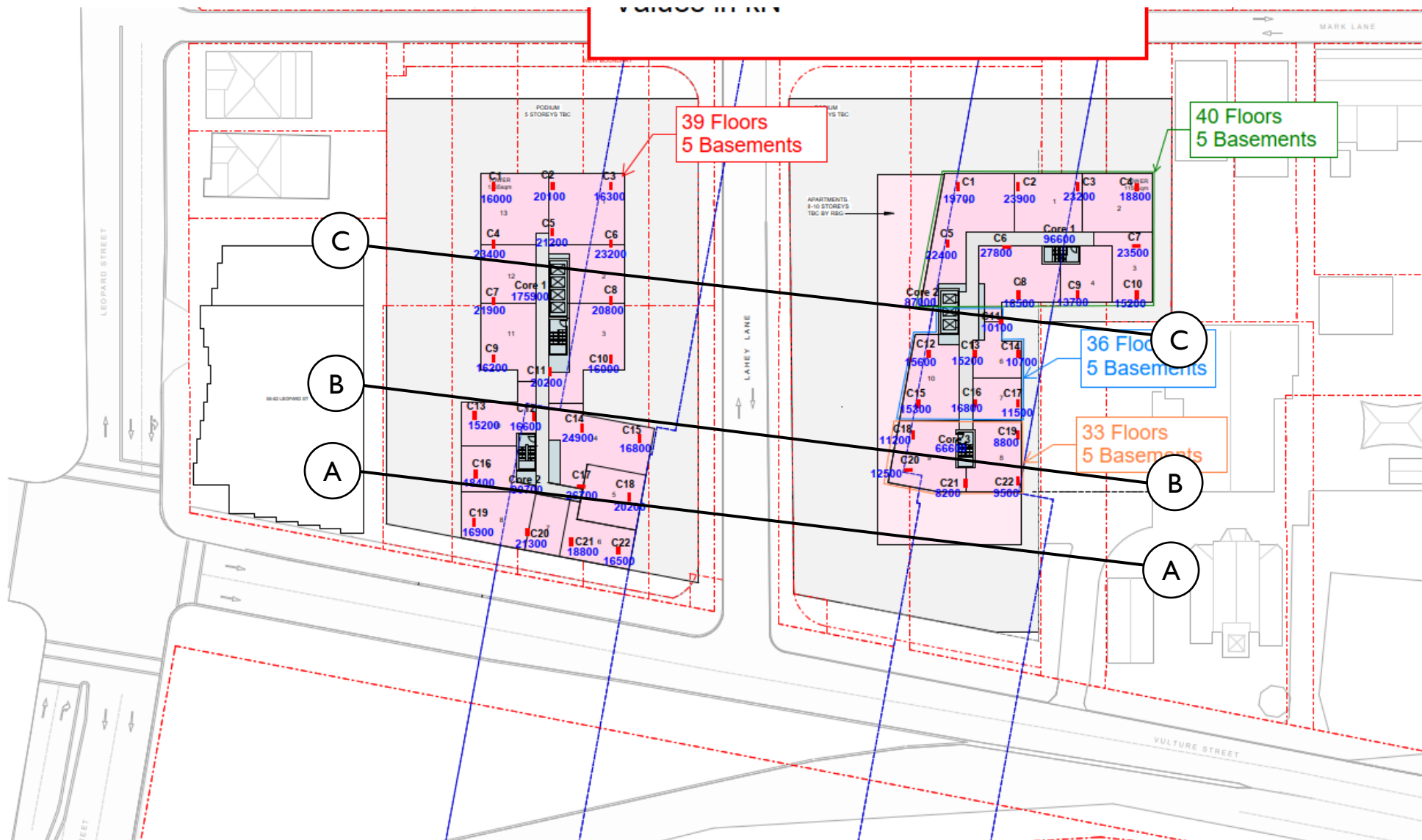
Light weight Apartment (10-Story):

- All loads are unfactored (kN).
- Load Assumption:
Residential) LL: 1.5 KPa, SDL: 1.5KPa + 1KPa (Steel Frame)
Car park) LL: 2.5 KPa, SDL: 0.5 KPa
Retail) LL: 5 kPa, SDL:1.5 KPa

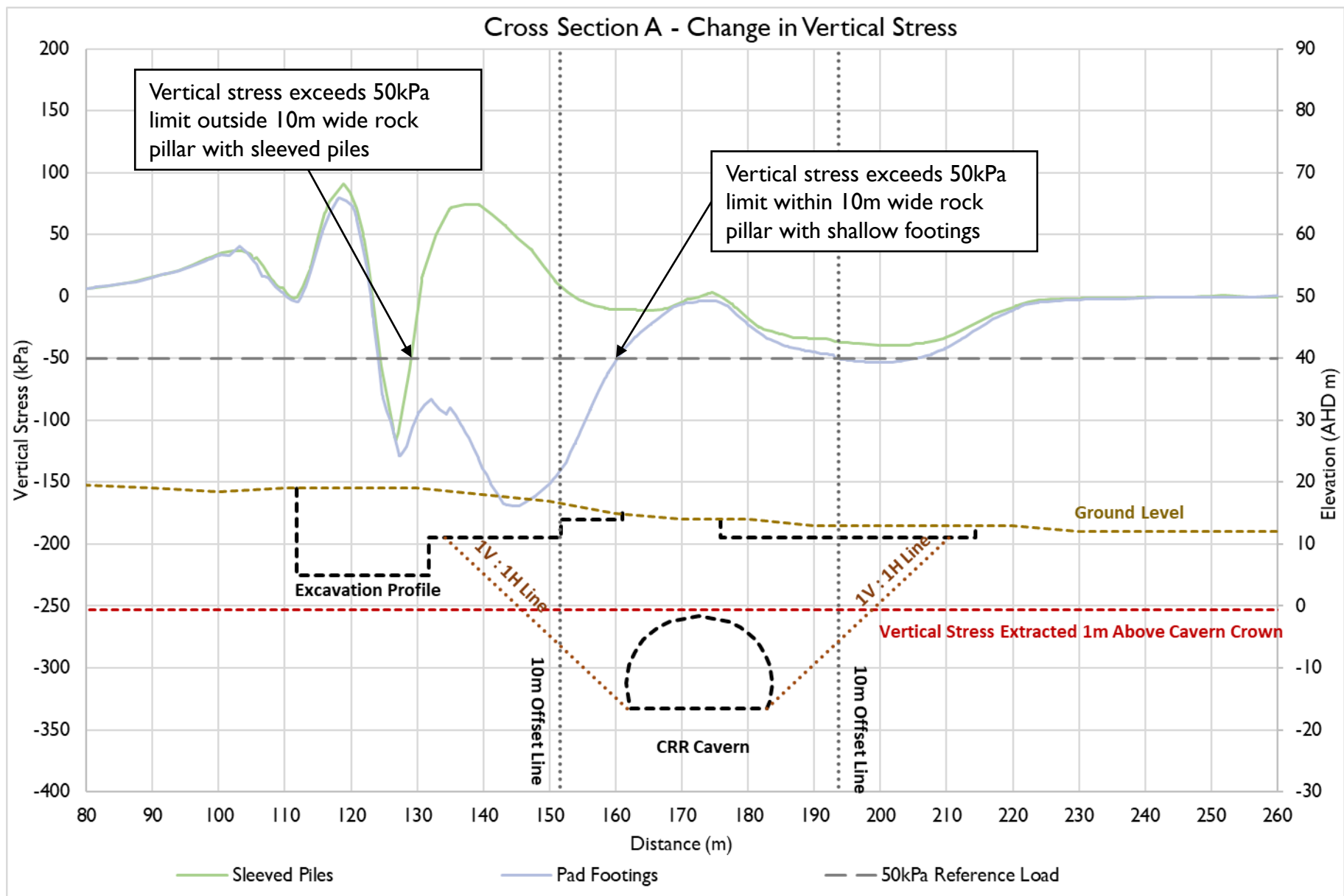
Slab thickness (Residential): 150mm
Slab thickness (Carpark): Average 250mm
Slab thickness (Retail): Average 300mm
Retail: Double Height

Appendix C

Preliminary Finite Element Assessment Outputs

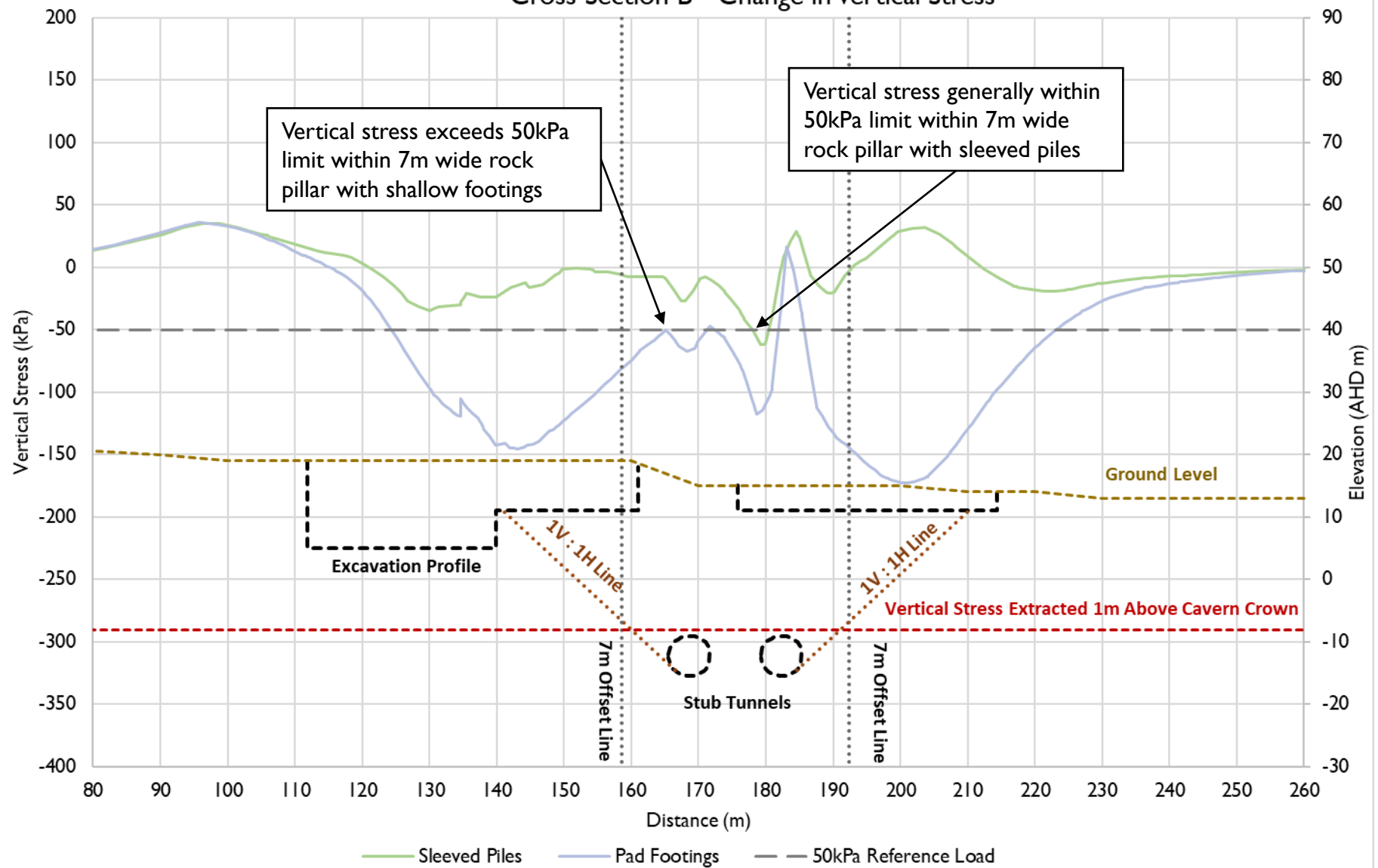


by:	DL	client:	Phillip Usher Constructions	 engineering design geosciences TYPESA Group
date:	7/3/2024	project:	Kangaroo Point Precinct	
approved:	DJC	location:	Brisbane, QLD	
scale:	As per axis	title:	Plan	
		job no:	B01554-I	
				figure: C1



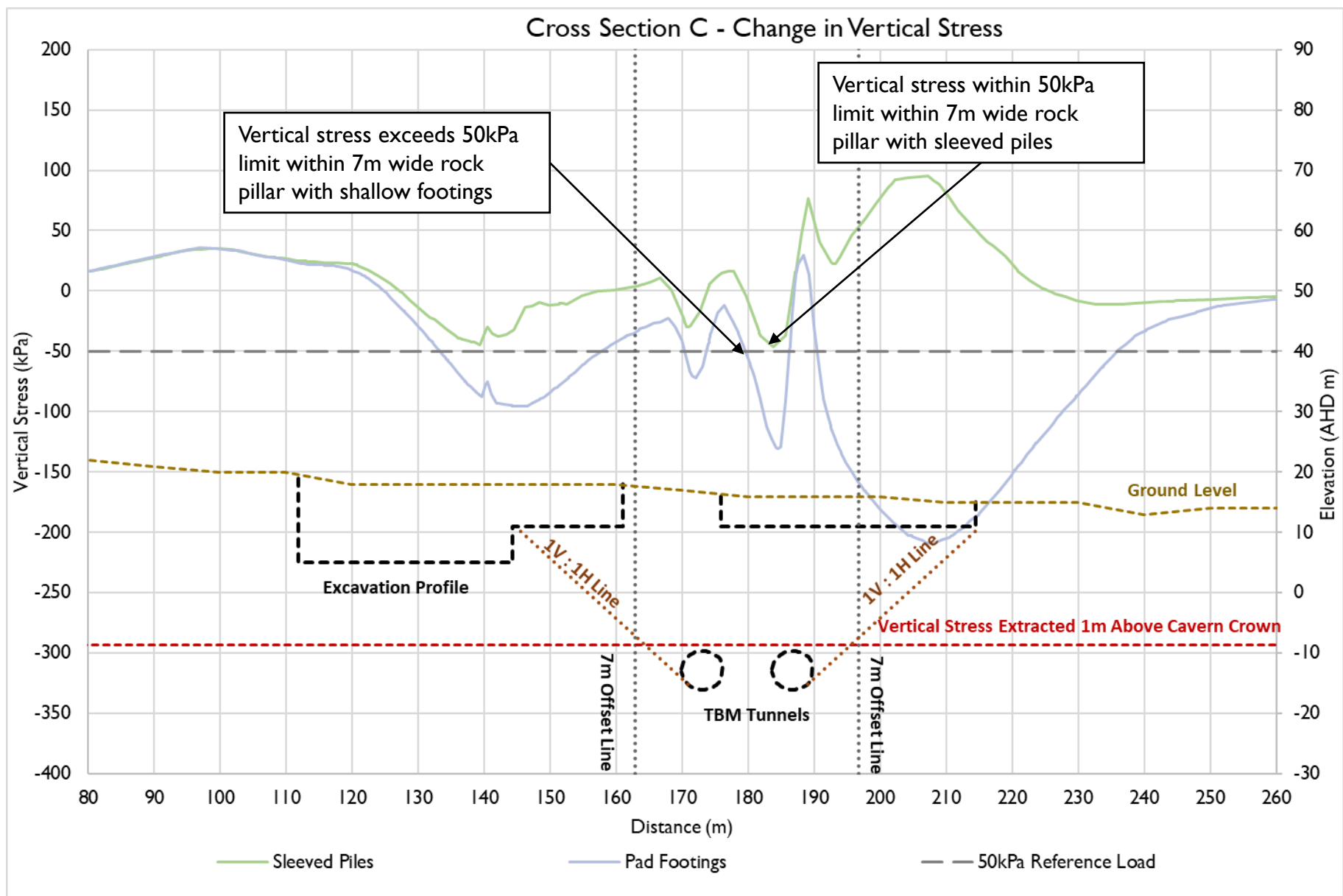
by:	DL	client:	Phillip Usher Constructions	 engineering design geosciences TYPSA Group
date:	7/3/2024	project:	Kangaroo Point Precinct	
approved:	DJC	location:	Brisbane, QLD	
scale:	As per axis	title:	Cross Section A – Change in Vertical Stress	
		job no:	B01554-I	figure: C2

Cross Section B - Change in Vertical Stress



by:	DL
date:	7/3/2024
approved:	DJC
scale:	As per axis

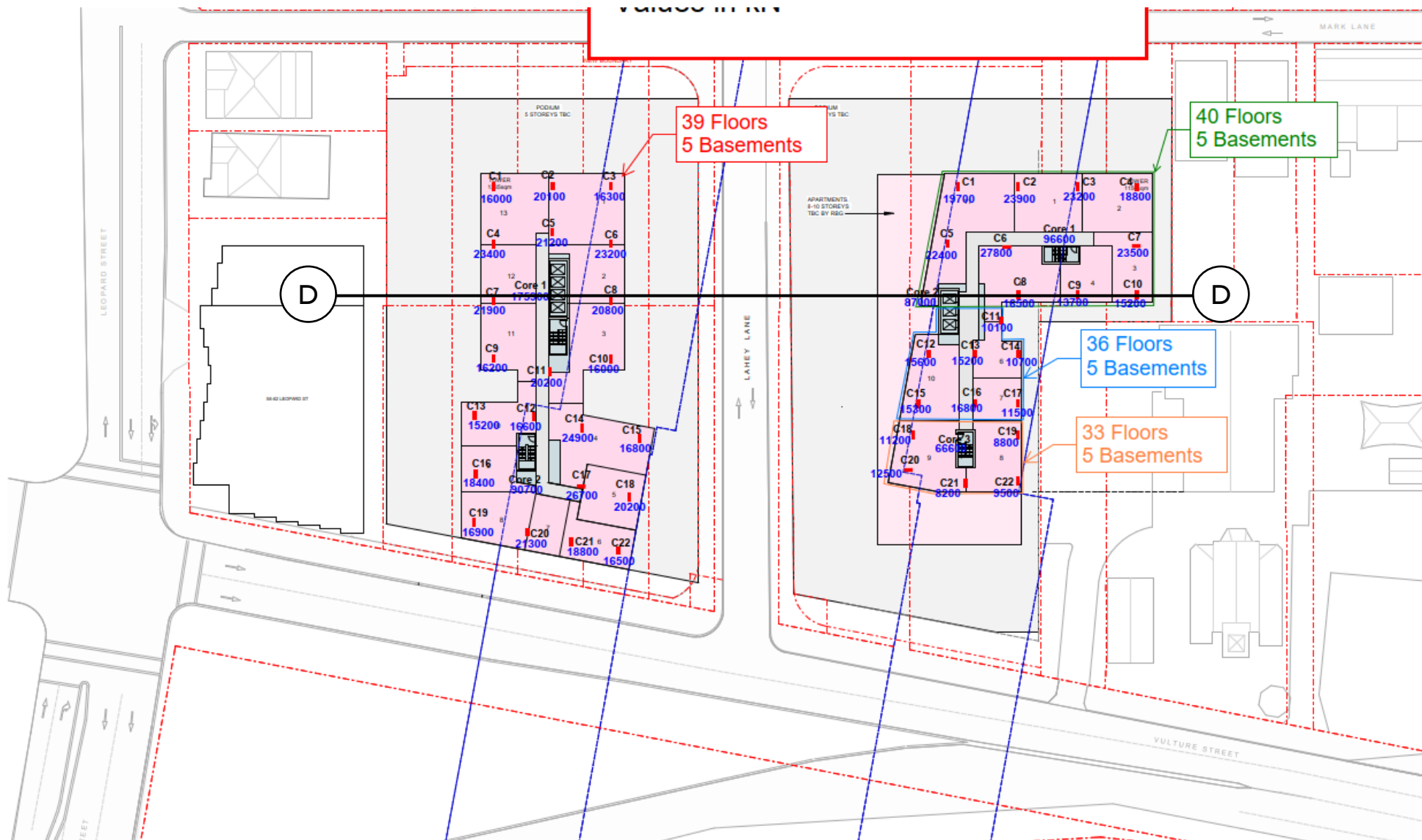
client:	Phillip Usher Constructions
project:	Kangaroo Point Precinct
location:	Brisbane, QLD
title:	Cross Section B – Change in Vertical Stress
job no:	B01554-I



by:	DL	client:	Phillip Usher Constructions	 engineering design geosciences TYPSA Group
date:	7/3/2024	project:	Kangaroo Point Precinct	
approved:	DJC	location:	Brisbane, QLD	
scale:	As per axis	title:	Cross Section C – Change in Vertical Stress	
		job no:	B01554-I	figure: C4

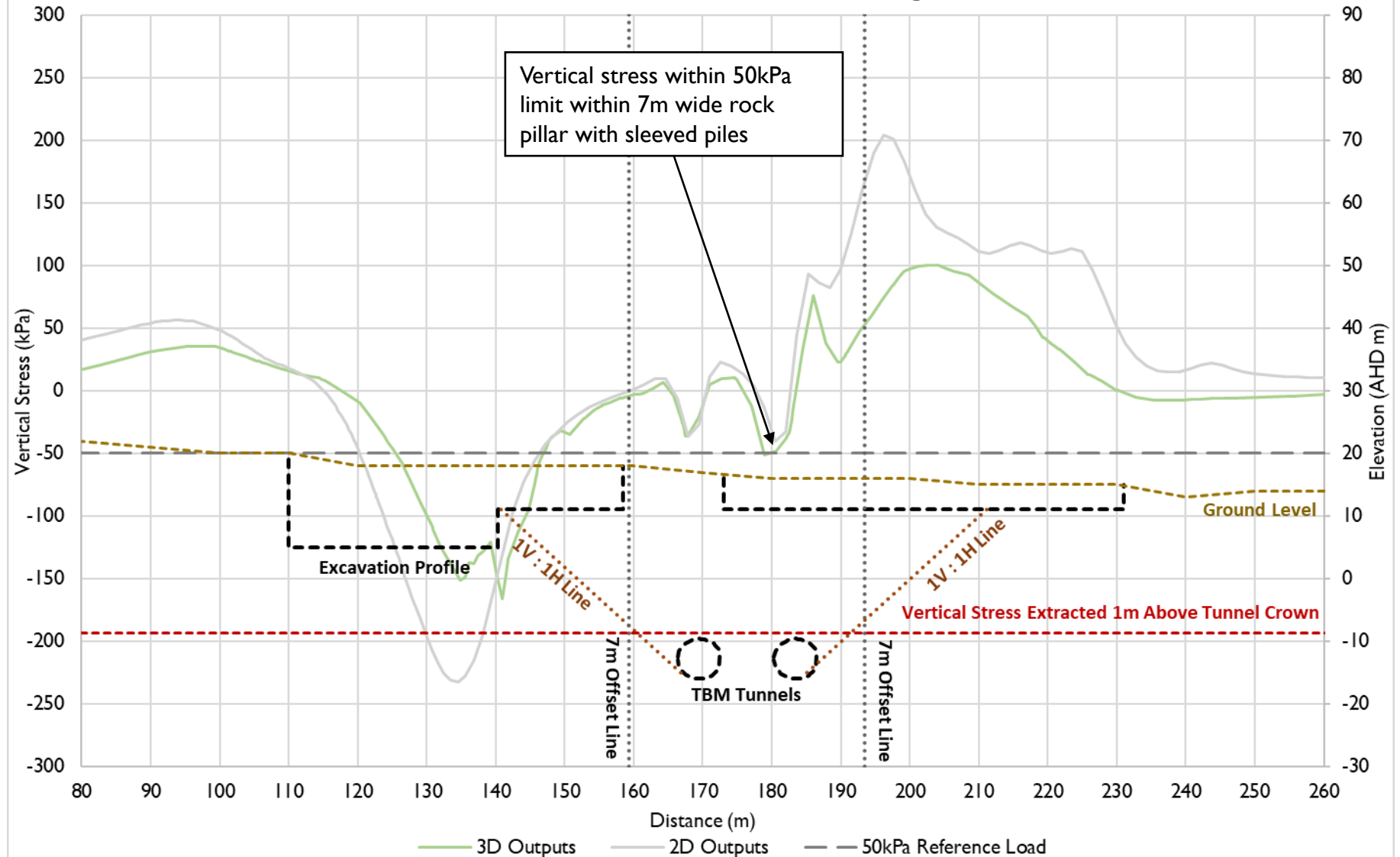
Appendix D

Comparison of 2D vs 3D Finite Element Outputs



by:	DL	client:	Phillip Usher Constructions	 engineering design geosciences
date:	7/3/2024	project:	Kangaroo Point Precinct	
approved:	DJC	location:	Brisbane, QLD	
scale:	As per axis	title:	Plan	
		job no:	B01554-I	
				figure: D1

Finite Element Assessment - Cross Section D - Change in Vertical Stress



by:	DL
date:	7/3/2024
approved:	DJC
scale:	As per axis

client:	Phillip Usher Constructions
project:	Kangaroo Point Precinct
location:	Brisbane, QLD
title:	Cross Section D – 2D FE Results
job no:	B01554-I

