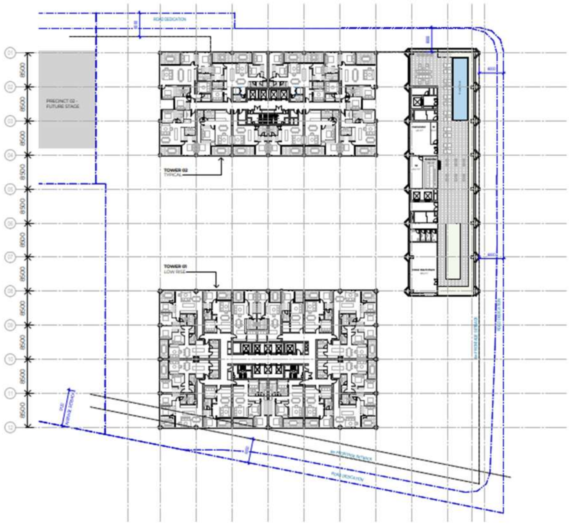
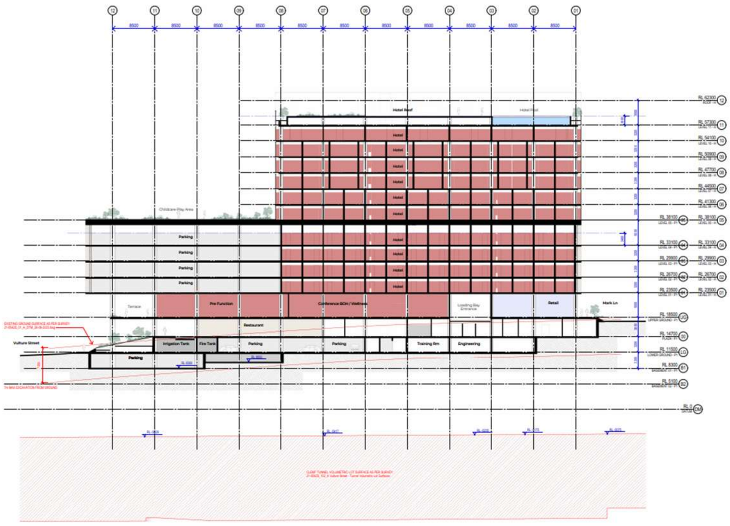
Mark Lane Development Geotechnical Report - Precinct 1 Brisbane, QLD





Prepared for:

Phillip Usher Constructions

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1 Project Background

EDG Consulting (EDG) has been engaged by Philip Usher Constructions (PUC) to prepare this geotechnical report to support a Development Application (DA) for Material Change of Use (MCU) for the proposed Multiple Dwelling, Short-term Accommodation, Food and Drink Outlet, Shop, Function Facility, Indoor Sport and Recreation, Office, Community Use, and Childcare Centre and Building Works (BW) involving Demolition of a Pre-1911 Building, buildings or structures within 10 metres of a heritage place and relocation of a Local heritage place. The DA is made over 18, 26, 26A, 32, 32A, 38, 44, 46, 48 and 52 Mark Lane, 803, 807, and 811 Main Street, and 352 Vulture Street, Kangaroo Point.

Precinct 1 of the proposed development will comprise of a hotel (Tower 3), two high-rise residential towers (Tower 1 and Tower 2), and surrounding podium structures. The eastern end of the Precinct 1 development is located above the Clem7 Tunnel Boring Machine (TBM) tunnels which were completed in 2010. Development of the proposed sites will therefore be required to satisfy the requirements of the Clem7 infrastructure, to ensure the proposed works do not exceed prescribed limits / criteria.

PUC has engaged EDG to provide geotechnical analysis and advice relating to the proposed works as part of the DA. The outcomes of our geotechnical assessment for the Precinct 1 development, located above the Clem7 tunnels, are documented herein.

2 Site Location and Description

The proposed Precinct 1 development is shown in Figure 1 and 2 and is bounded by Mark Lane to the north, Vulture Street to the south and Main Street towards the east. Saint Nicholas Cathedral is located west of the site, adjacent to Tower 1.

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

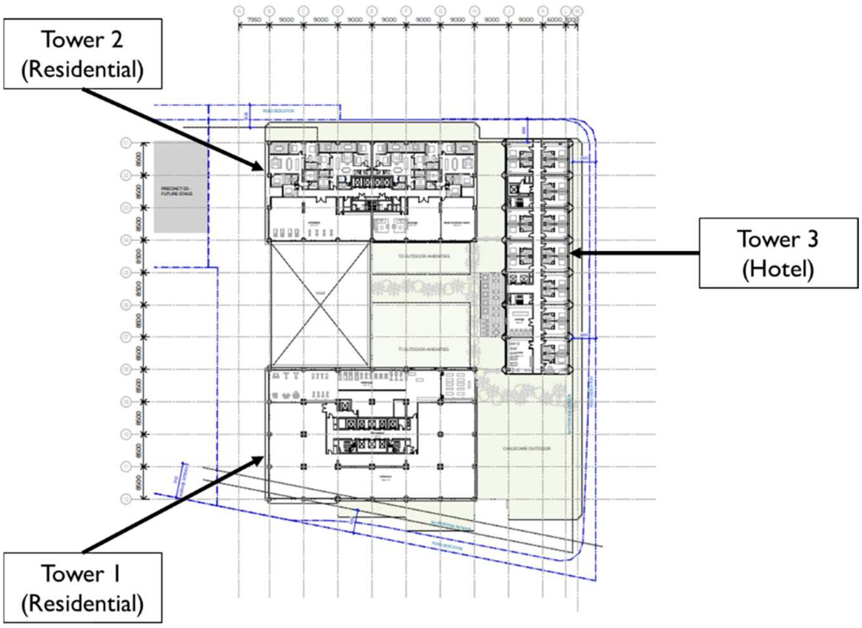

Figure 1 – Precinct 1 Site Plan

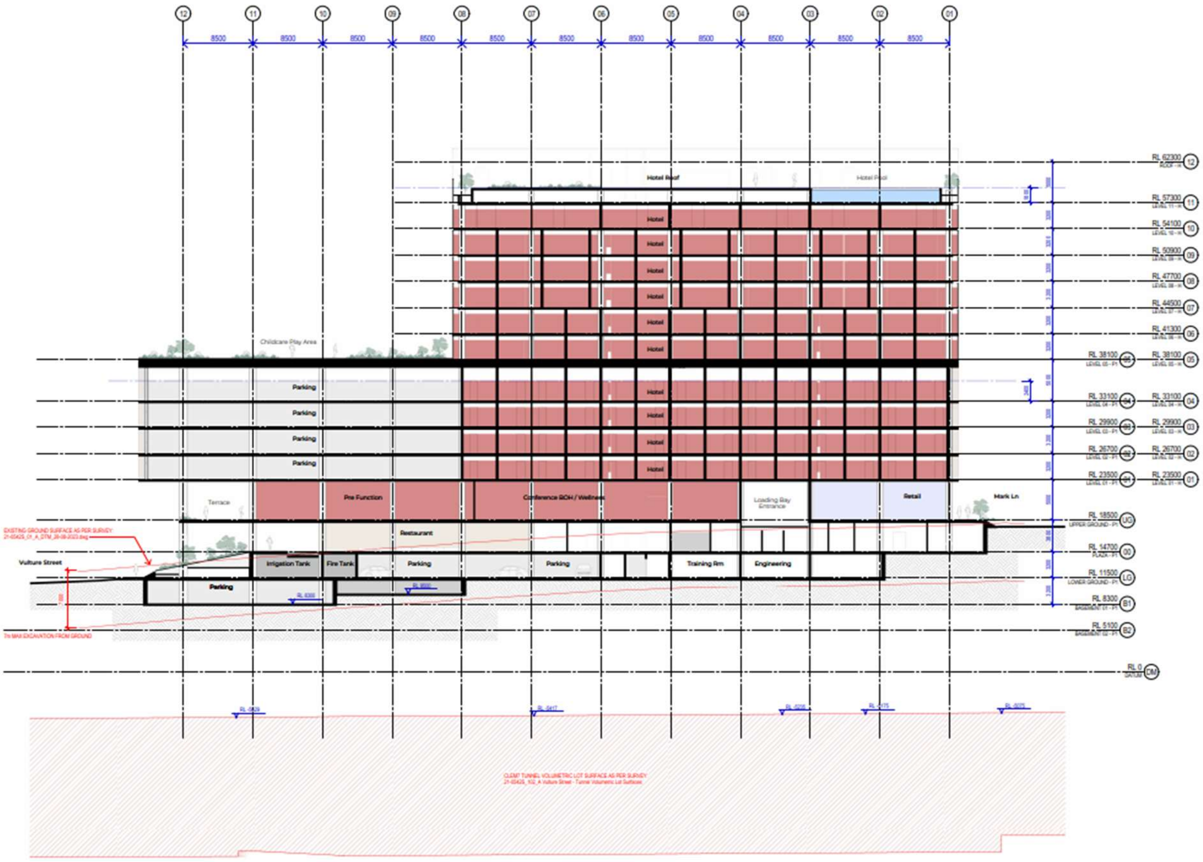

Figure 2 – Precinct 1 Elevation (looking from East towards West)

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-1120-030021) 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. Towards the eastern end of the site, an unconformity zone exists between the underlying Neranleigh Fernvale Group and the overlying Tuff.

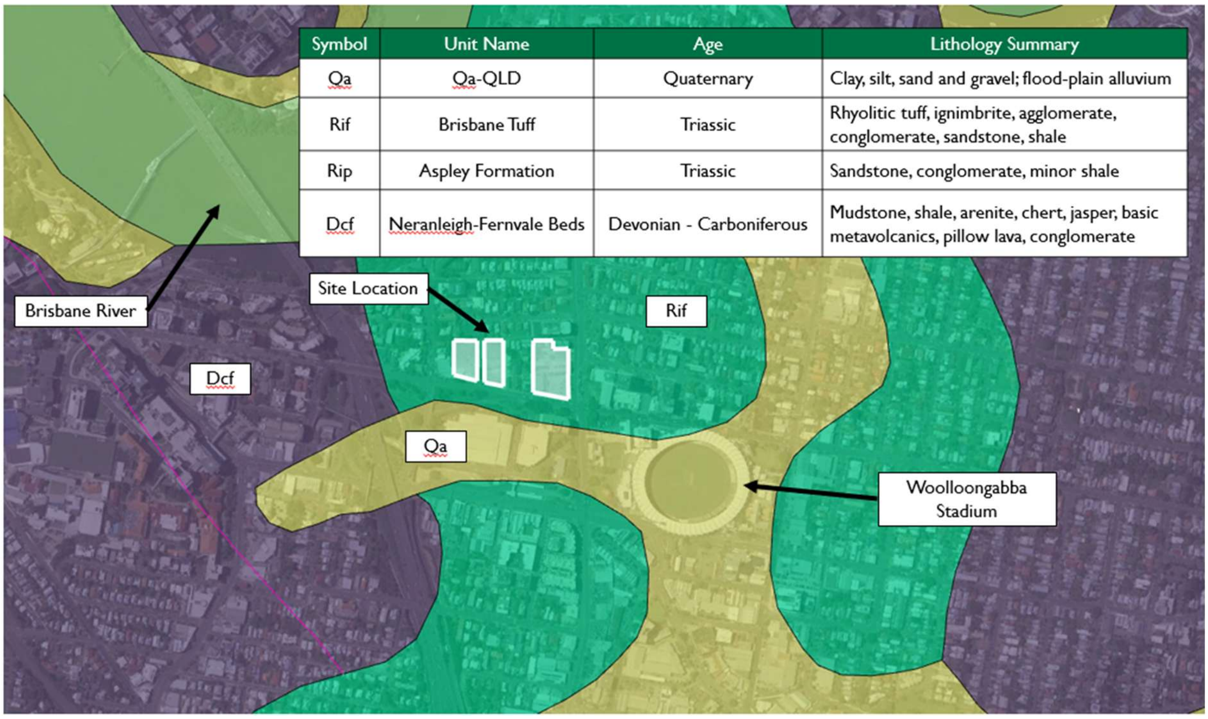

Figure 3 – Published Geological Information

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 Clem7 tunnel design, we have adopted similar terminologies for the geological units previously identified in the Clem7 Geotechnical Interpretive Report by Golder Associates[1]. The project material units are summarised below and are discussed in further detail in the following sections.

- Fill (F)
- Residual Soil (RS)

[1] North South Bypass Tunnel – Geotechnical Interpretive Report – Driven Tunnels by Golder Associates (Report Ref. NSBT-0802-GT-RP-055005[05])

- 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)
- Unconformity Zone (UC)
- 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.

3.4.4 Medium Strength Conglomerate dominant Aspley Formation (CD M)

Medium strength conglomerate dominant Aspley formation is typically encountered underneath 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 underneath 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 Unconformity (UC)

An unconformity zone exists between the underlying Neranleigh Fernvale Group and the overlying Tuff. Limited information is available associated with the unconformity rock units, with only the Clem 7 Geotechnical Interpretive report by Golder making reference to the UC unit. In addition, the two closest geotechnical investigation (NST01 and NST02) which encountered and recorded this rock unit are located approximately 300m away from the site. Based on the limited available geotechnical data, we have interpreted the Unconformity Zone unit to typically comprise medium to high strength rock units comprising of breccia, conglomerate and sandstone.

3.4.7 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 B01554-1AB_001 to B01554-1AB_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 1.

Table 1 – 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 (K0)	Previous experience with similar soils and rock, along with engineering judgement.
Undrained shear strength parameters (Su)	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 (Is50) 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
		(kN/m^3)	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
	Unconformity	UC Note 1	Breccia, conglomerate and sandstone	22	400	55	30	10	50-60	8	0.2	3.608	0.0067	0.504	60	1	2
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

Note: [1] Very limited geotechnical investigation available for UC material unit. In lieu of future investigation data, we have based material parameters for the UC unit on the parameters as documented in the Geotechnical Interpretative Report by Golder Associates (Report Ref. NSBT-0802-GT-RP-055005[05]).

5 Groundwater

We have undertaken a review of the Clem7 Geotechnical Interpretative Report[2] and the Woolloongabba Permanent Lining Design Report[3].

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 drawings[4]. 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. 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.

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 basement excavation level of RL 14.7m, 11.5m, 9.5m, 8.3m and 5.1m AHD. The material at the proposed foundation level is expected to be either medium to high strength Tuff (Tuff M-H) or high to very high strength Tuff (Tuff H-VH).

We have carried out a preliminary assessment of allowable bearing capacity considering a shallow footing foundation founding in Tuff M-H and founding in Tuff H-VH. The assessment is based on the method presented by Wyllie[5], which represents the strength of the rock mass based on the generalised

[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].
[5] Wyllie, D.C., 1999. Foundations on Rock, 401pp. Spon: New York.

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.

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[1] (MPa/m)
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 ϕ_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)[6]. Recommended values of ultimate skin friction and ultimate end bearing are summarised in Table 6.

Table 6 – Ultimate Pile Design Parameters

Material	Fs, Ultimate Skin Friction (kPa)	Fbu, Ultimate End Bearing (MPa)
RS	50	N/A
Tuff VL-M	500	8

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

Material	Fs, Ultimate Skin Friction (kPa)	Fbu, Ultimate End Bearing (MPa)
Tuff M-H	1000	20
Tuff H-VH	1500	30

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 5 and assumes that adequate drainage is provided behind the wall such that no hydrostatic pressure build-up occurs.

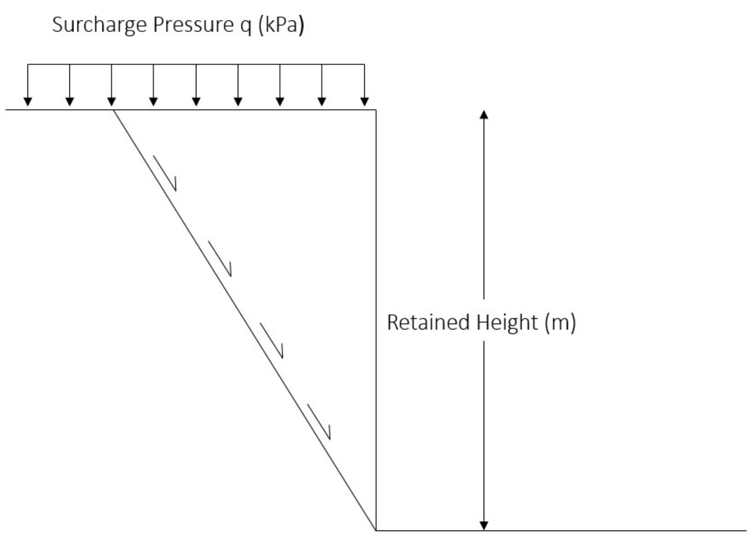

Figure 5 – Failure Mechanism

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 and medium to 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)	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 Fookes[7] 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 6.

Table 9 – Excavaeability 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, which may be hard 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 Clem7 tunnels and nearby infrastructure may be required.

[7] 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.

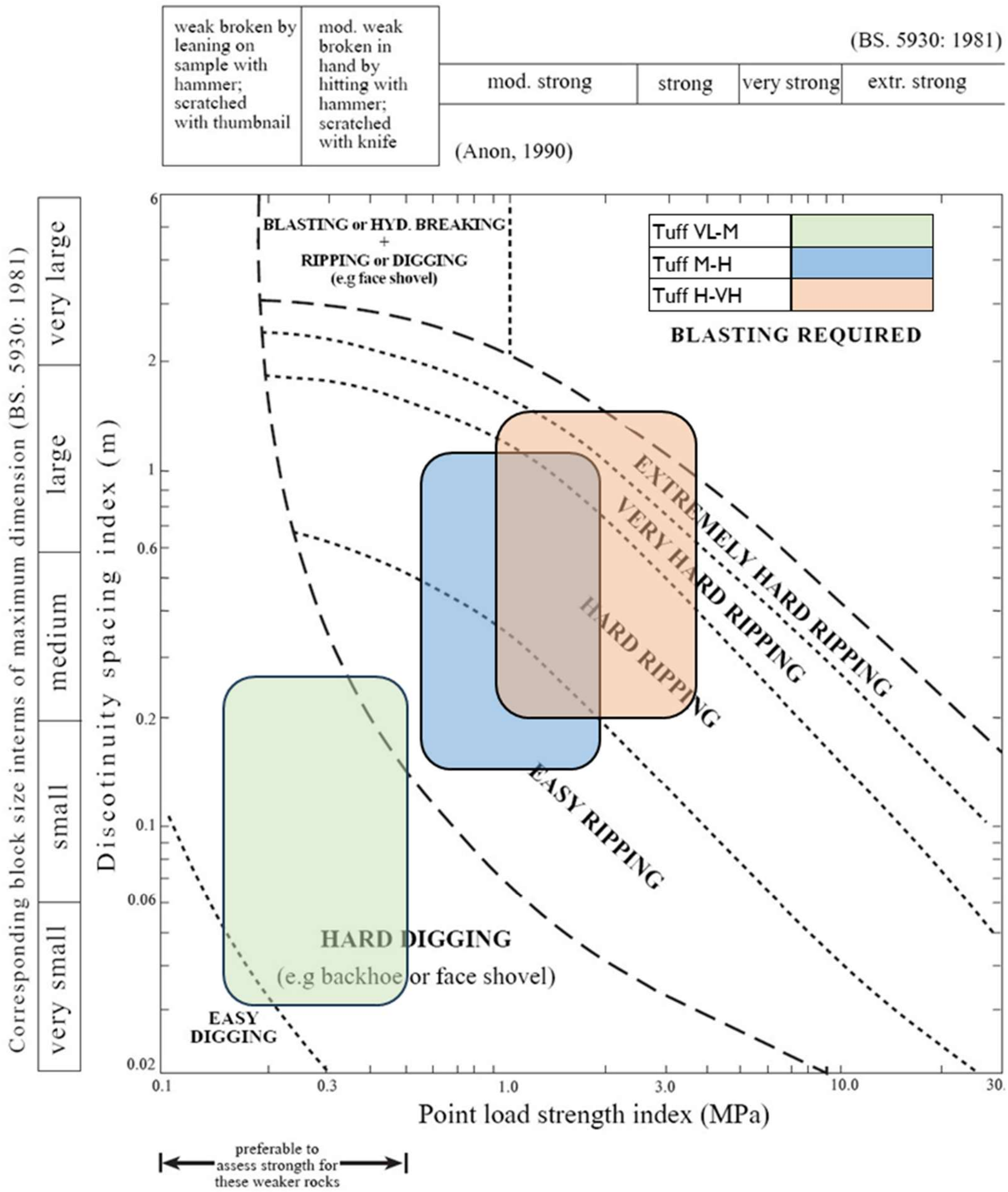

Figure 6 – Excavatability Assessment

11 Slabs on Ground

11.1 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 ≥ 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 Clem7 TLDM Requirements

The detailed design of the permanent lining of the Clem7 tunnel is outlined in the Tunnel Lining Design Manual[8] (TLDM). The nominated design constraints associated with future building loads and excavation within vicinity of the Clem7 infrastructure are summarised in Figure 7.

[8] LBBJV. February 2007. *North South Bypass Tunnel. Design Lot No. 0115 – Stage 2 – Detailed Design. Zone 0: Across All Zones. Design Manual Tunnel Lining*. Report No. NSBT-0115-TU-RP-004184[00].

Table 1: Loading and excavation design constraints

Additional Vertical and Lateral Loading	Excavations
Building vertical loading: (i) up to net 50 kPa (working load) acting on the ground at a level of 1m above the tunnel crown and in uniform and patterned (including symmetric and unsymmetrical) arrangements which give the most unfavourable loading condition on the tunnel; and (ii) allow for buildup of surface level with a minimum of one metre of fill equivalent to 20 kPa.	**Continuous excavations:** (i) up to 14m below natural surface for developable properties or developable land between Baildon St, Kangaroo Point and St Pauls Terrace, Fortitude Valley; and (ii) up to 7m below natural surface for developable properties or developable land at locations other than those listed in item (i) above; and (iii) with a minimum of 7m residual ground cover above the extrados of the tunnel crown; and (iv) with a minimum 7m pillar width between the extrados of the side wall of the tunnel and any adjacent building basement excavation.

Figure 7 – Clem7 Loading and Excavation Design Constraints

As per the TLDM, 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 $50kN/m^2$ acting at a reference level of 1m above the tunnel crown.

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 8) 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.

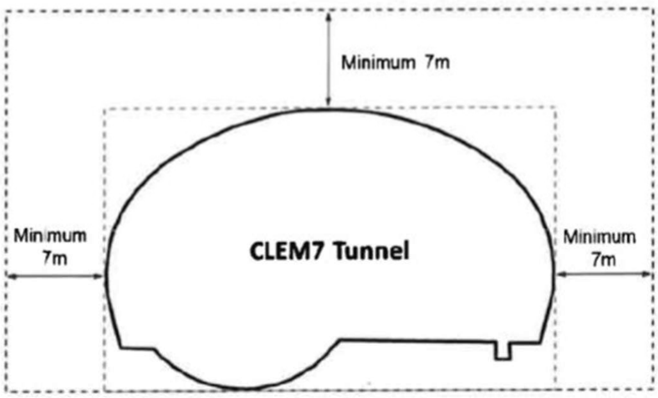

Figure 2 – CLEM7 Tunnel Extrados Excavation Exclusion Zone Planning Requirements

Figure 8 – Clem7 Tunnel Exclusion Zone

15 Preliminary Finite Element Assessment

15.1 Analysis Methodology

To provide a comparison between the TLDM 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 and the structural loads have been generated from an preliminary 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 9.

Develop a 3D FE model representing the site appropriate for analysis at DA stage

↓

Use the model to assess vertical stress acting at a reference level of 1m above the tunnel crown, under existing conditions

↓

Use the model to assess vertical stress acting at a reference level of 1m above the tunnel crown, arising from the proposed building loads

↓

Compare the analysis outcomes

↓

The increase in vertical stress arising from the proposed building loads, when compared to the existing conditions, should not exceed 50kPa.

↓

Further analysis to be carried out during detailed design

Figure 9 – 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 Clem7 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 Clem7 project (as shown in the Clem7 Geotechnical Interpretative Report[9]), 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 Clem7 project, as presented in the Clem7 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 B01554-1AF_001 to B01554-1AF_005.

Soil and rock behaviour was 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 presented in a loading plan showing both the magnitude and location of the applied loads. A working load combination comprising 1.0G (dead load) and 0.7Q (live load), together with a 10% contingency factor, was adopted for the finite element assessment, resulting in an overall load combination of 1.1(G+0.7Q).

In the finite element model, the structural loads were modelled as follows:

- Columns loads were modelled as individual point loads.
- Wall loads were modelled as line loads along the length of each wall.
- Core loads were modelled as a uniformly distributed load (UDL) over the core footprint area.

15.4 Groundwater Inputs

We have adopted a groundwater level of 8m below existing surface level (see Section 5 of this report for additional information and basis).

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 – Columns	Plate	Shallow foundation footings for the podium columns are modelled as 3m (length) x 3m (width) x 1.0m (thick) plate elements. An elastic modulus value of 32.8GPa has been assigned to all plates.

[9] 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
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.
Piles	Embedded Beam	All piles are modelled as 1.0m 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 of 50kPa has been allowed for piles within the steel sleeved zones.

The finite element model is shown indicatively in Figure 10 and Figure 11.

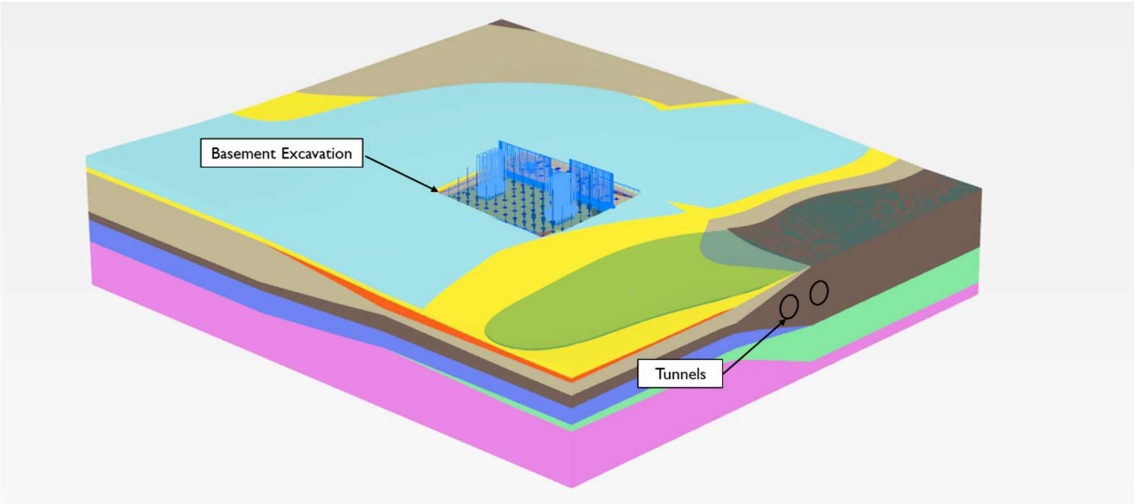

Figure 10 – Finite Element Model (Overall)

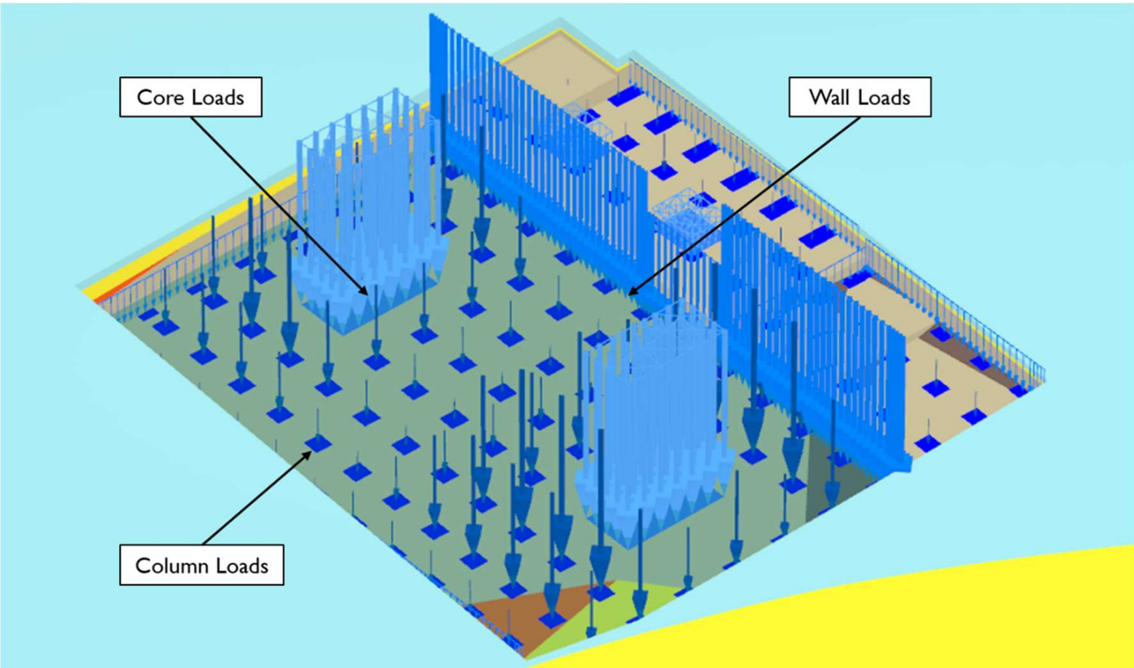

Figure 11 – Finite Element Model (Building Foundations)

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	Tunnel Excavation	The tunnels are excavated across the model. Stress relaxation of the tunnel is represented using the ΣM_{Stage} factor.
3	Basement Excavation	The basement excavation for the proposed building is excavated.
4	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.
5	Apply loading	Individual point loads, line loads and uniformly distributed loads for the columns, walls and cores, as provided by RBG, are applied.

15.7 Tunnel Construction

We have made allowance for the progressive installation of segmental lining by assessing the stress relaxation prior to support installation following the method by Hoek (2008). 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 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.

For 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 of one in our finite element assessment, which therefore allows all tunnel convergence to occur prior to installation of the TBM lining.

15.8 Sleeved Piles

All walls and columns along architectural grid "Guideline H" are founded on sleeved piled foundations. The purpose of the sleeved piles is to limit the transfer of structural loads to the rock mass within the sleeved section, and therefore transfer structural loads deeper into the rock mass and further away from the Clem 7 tunnels.

Within the sleeved section of the piles, a reduced skin friction has been adopted, with a value of 50kPa applied within these zones. The steel sleeves extend 15m below the bulk excavation level, which extends past the crown of the tunnel.

The tower columns which are founded on sleeved piled foundations in Case 2 are highlighted in Figure 12.

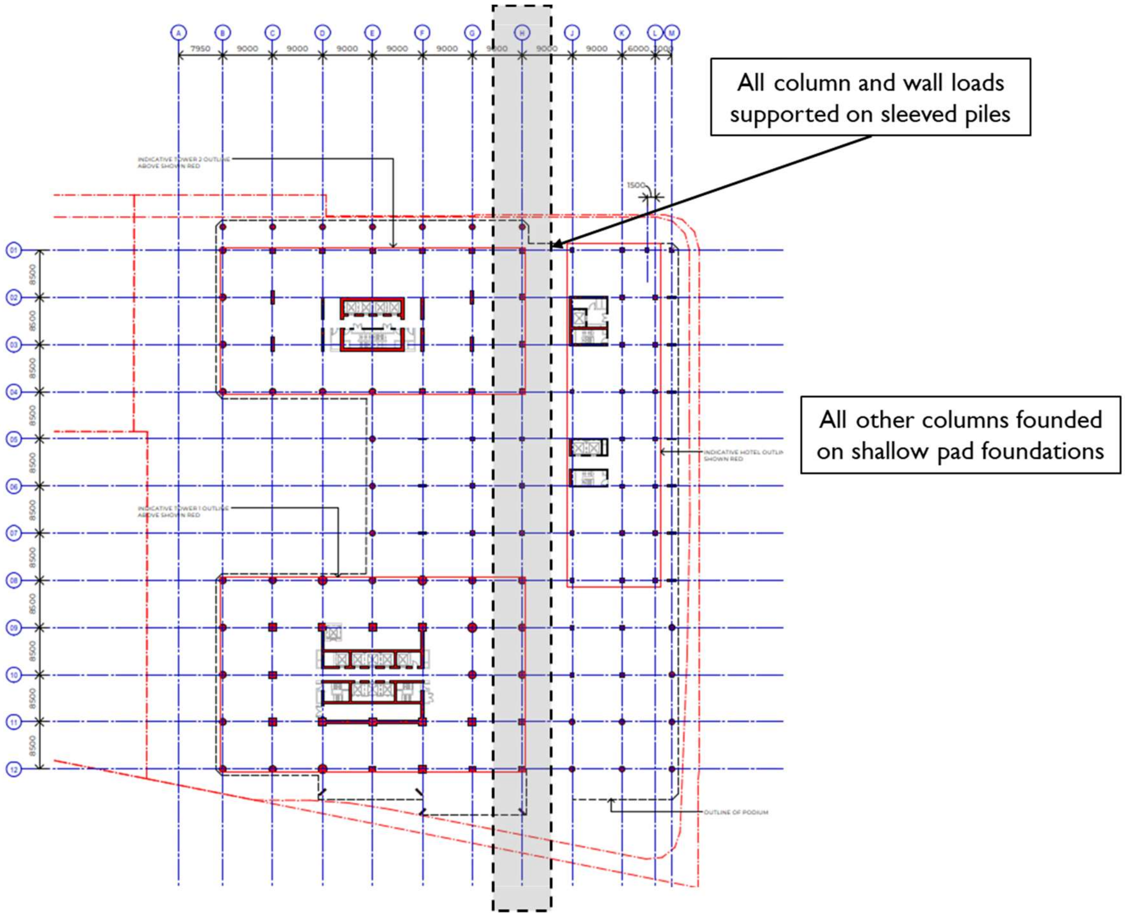

Figure 12 – Building Plan Showing Location of Sleeved Piled Foundations

15.9 Analysis Results

In accordance with the TLDM requirements, we have extracted the vertical stress at a reference level of 1m above the tunnel crown. The results of our assessment are presented graphically as contour plots on Figure B2 to B4 in Appendix B 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 B2 presents an overall view of the increase in vertical stress across the entire site. Figure B3 and B4 provide zoomed-in views along the southern side and northern side of the site, respectively, above the Clem7 tunnels.

Our assessment indicates that the increase in vertical stress 1m above the tunnel crown remains within the TLDM nominated limit of 50kPa.

The exact arrangement of the sleeved pile foundations 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.

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 12 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 12 additional proposed investigations are shown in Appendix D in blue. Existing boreholes are shown in white.

18 Risks and Limitations

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 12.

Table 12 – Risk and Limitations

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. In particular, limited information is available for the unconformity material unit (UC) between the underlying Neranleigh Fernvale Group and the overlying Tuff. Further investigation targeting this material unit will be required.
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 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 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.

For and on behalf of EDG Consulting Pty Ltd

David Cunliffe
Senior Principal
RPEQ 15674

Ground conditions and the natural environment often present the highest potential risks to project construction and operation. Helping our clients manage their geotechnical risk is fundamental to the role of EDG. We have prepared these notes to assist our clients to understand the information we provide and to help them to manage their risk. Where there is uncertainty about the site, project or geotechnical conditions, contact EDG for assistance.

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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

	Revisions/Descriptions	Drawn	Date	Approved
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Base Plan	

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Dimensions shown in metres except where shown otherwise

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KANGAROO POINT, BRISBANE
SITE PLAN
PRECINCT 1

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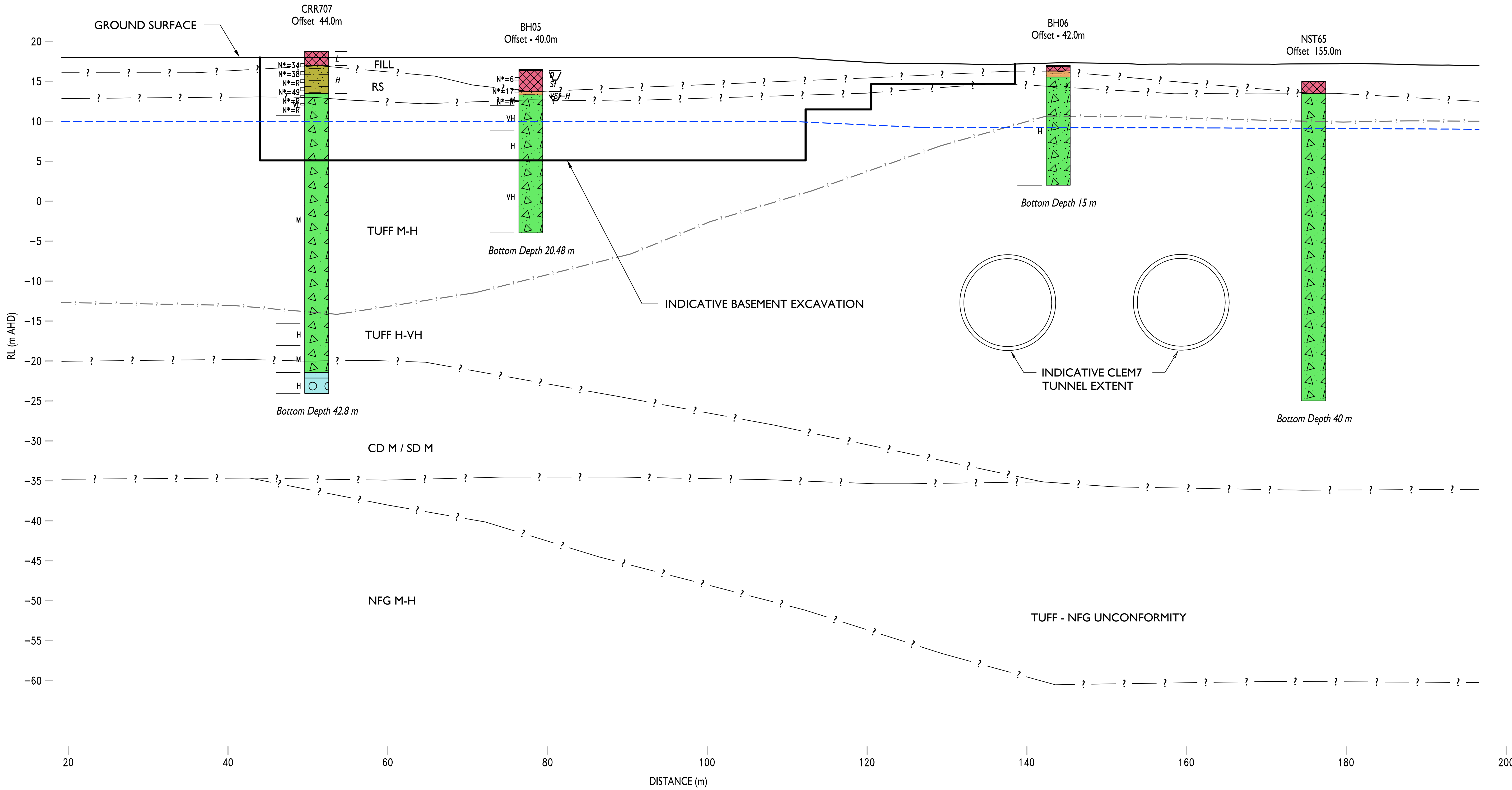

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Water Level

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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Coordinate System

Height Datum

Base Plan

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Dimensions shown in metres except where shown otherwise

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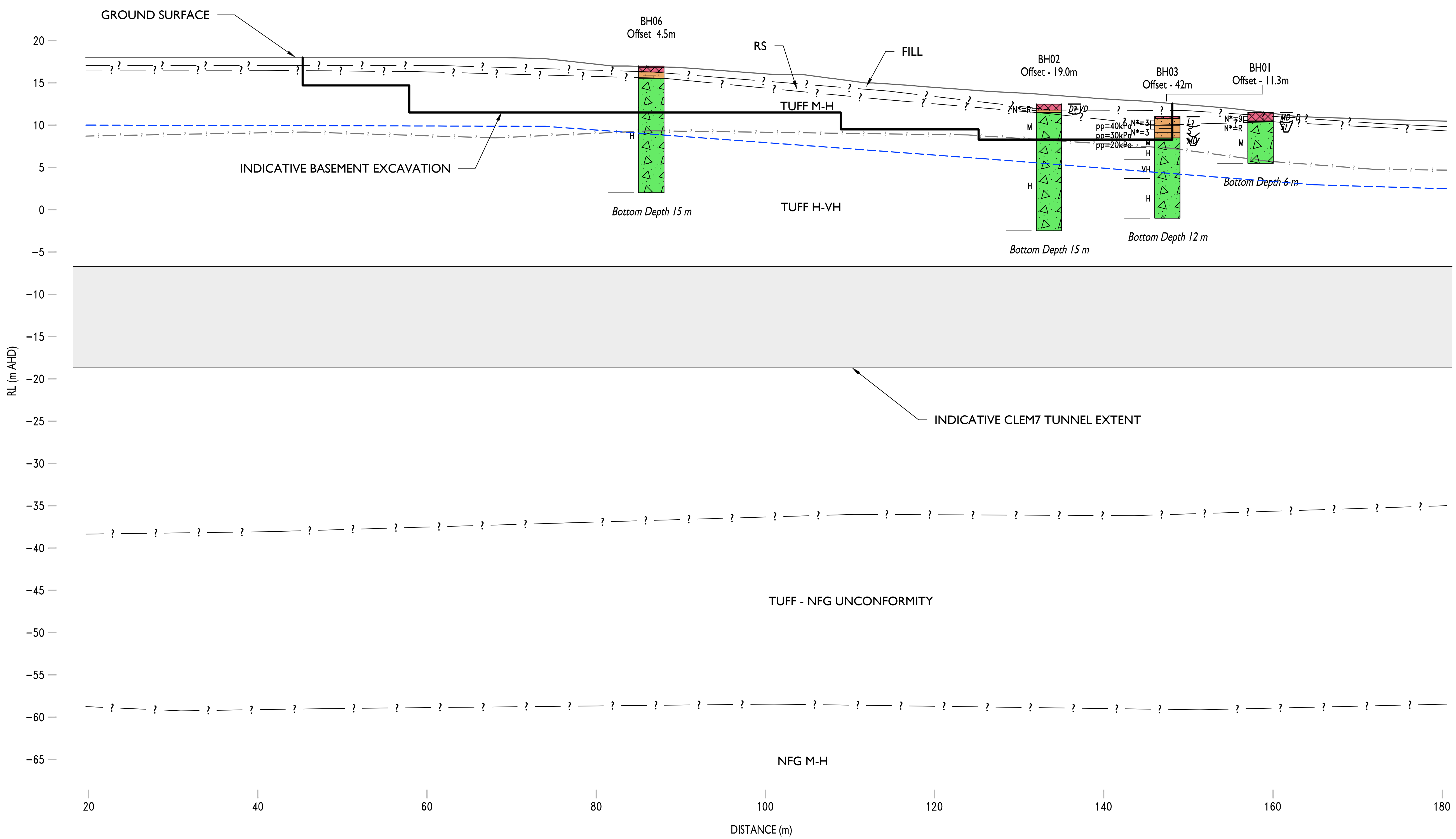

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pp=40kPa	Undrained Shear Strength from Pocket Penetrometer
VL	Rock Strength (see explanation sheets)
VS/MD	Consistency / Density

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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Base Plan

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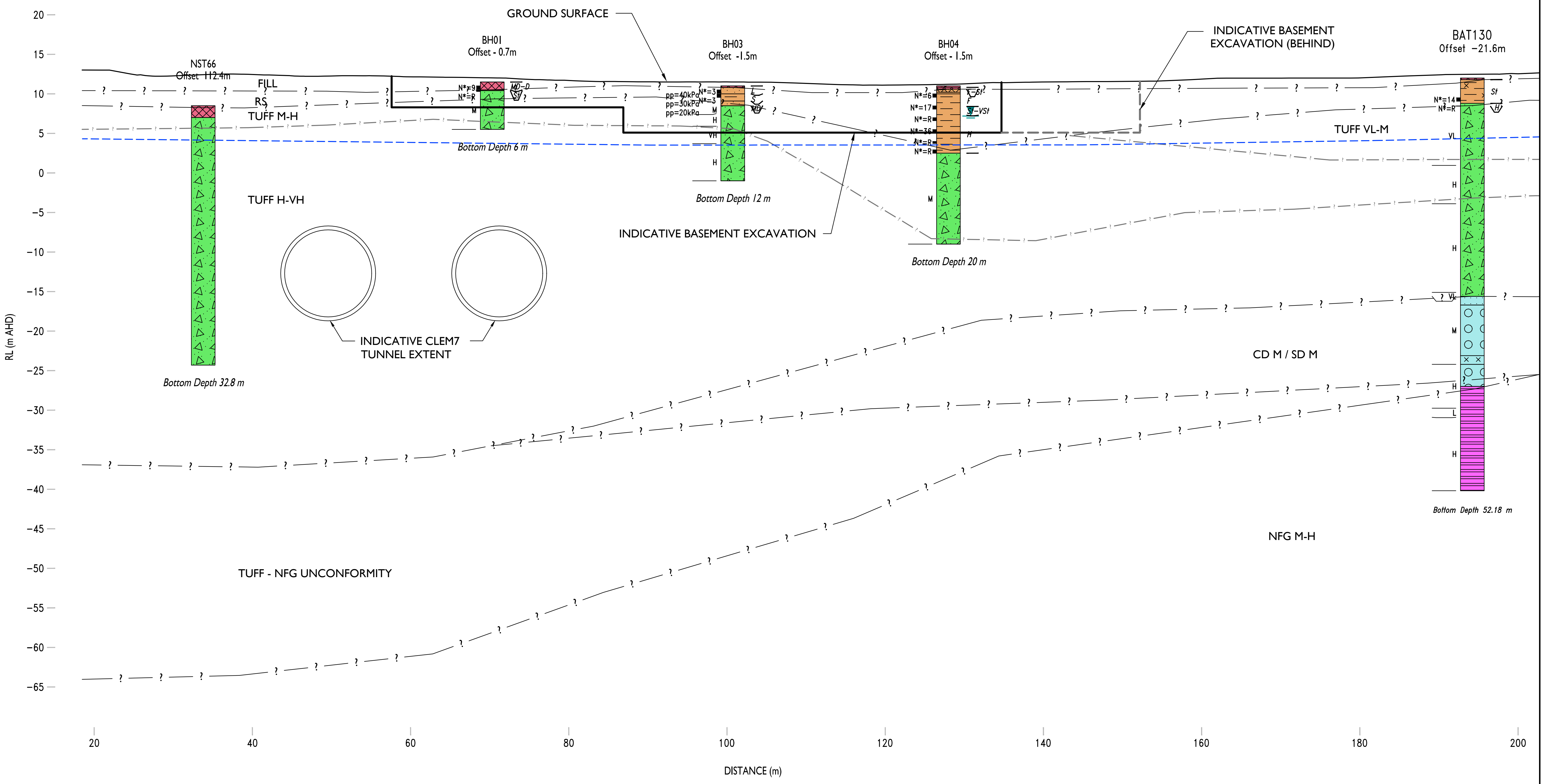

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VL	Rock Strength (see explanation sheets)
VS/MD	Consistency / Density

FILL: FILL, CONCRETE

SOIL: SAND, SILT, CLAY, SILTY CLAY, SANDY CLAY

ROCK: TUFF, SANDSTONE, MEDIUM, MEDIUM – COARSE GRAINED, SILTSTONE, CONGLOMERATE, ARGILLITE, PHYLLITE, CORE LOSS

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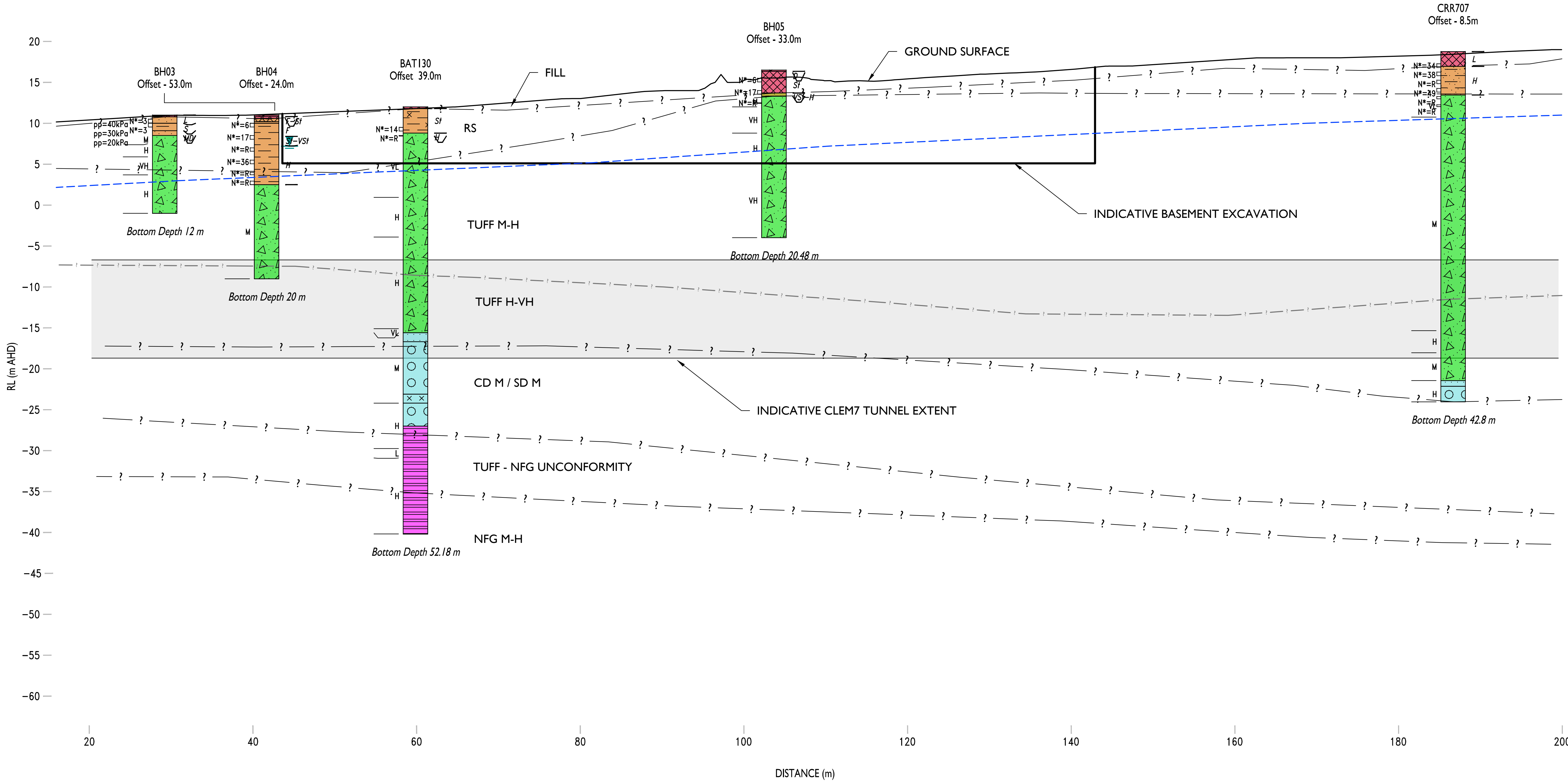

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pp=40kPa	Undrained Shear Strength from Pocket Penetrometer
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— — — — INTERPRETED GROUNDWATER LEVEL

FILL, RS, TUFF, CD M/SD M, NFG M–H

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DRG No.	B01554-1AF_005	A
Client Ref.	Client Ref.	

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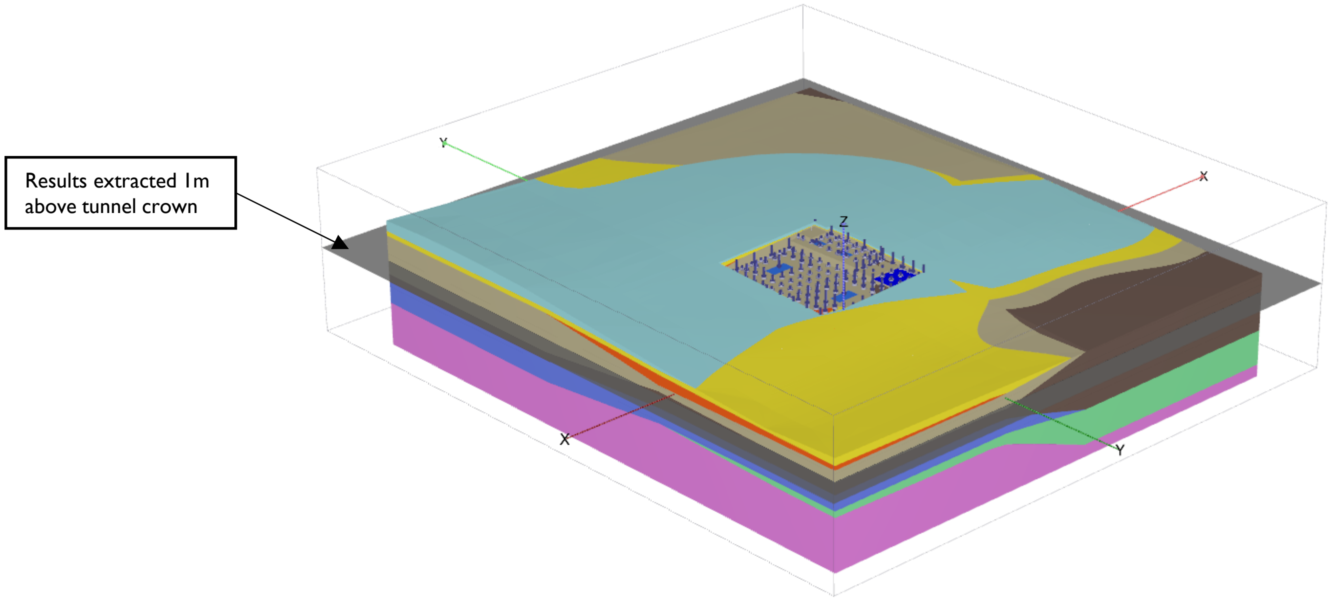
Appendix A

Available Geotechnical Investigations

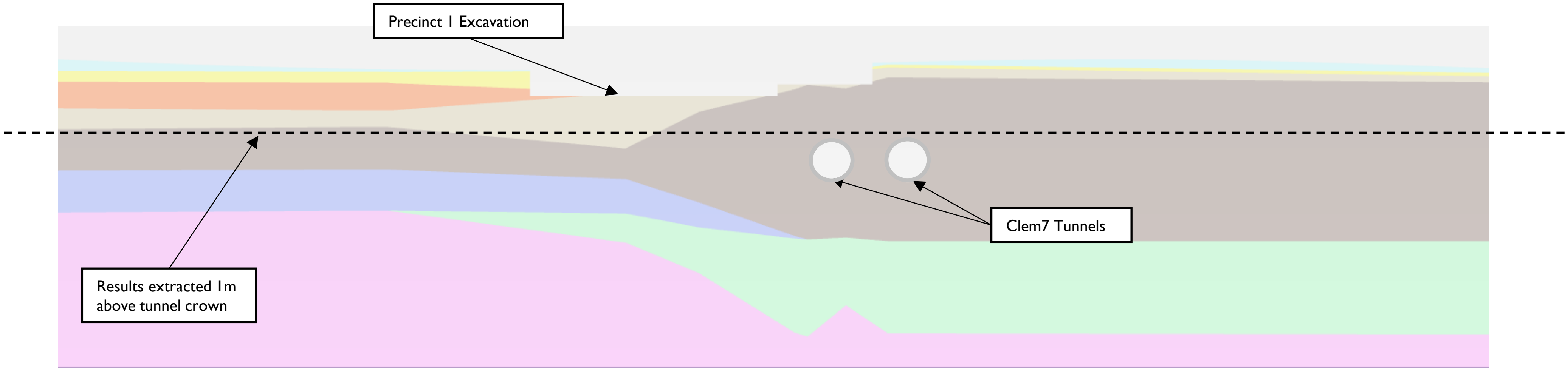
Test Location ID	Easting (GDA94)	Northing (GDA94)	Termination Depth (m)	Surface RL (m)
BAT130	503391.1	6959941.2	52.18	12
BH01	503514	6959924	6	11.5
BH02	503506	6959938	15	12.5
BH03	503483	6959924	12	11
BH04	503455	6959923	20	11
BH05	503462	6959985	20.48	16.5
BH06	503528	6959985	15	17
CRR1017	503359.88	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

Finite Element Assessment Outputs



Isometric View



Elevation View

by:	DL	*client:*	Phillip Usher Constructions
date:	30/4/2026	*project:*	Mark Lane Precinct
approved:	DJC	*location:*	Brisbane, QLD
scale:	As per axis	*title:*	*Results Extracted 1m above Tunnel Crown*
		job no:	B01554-1

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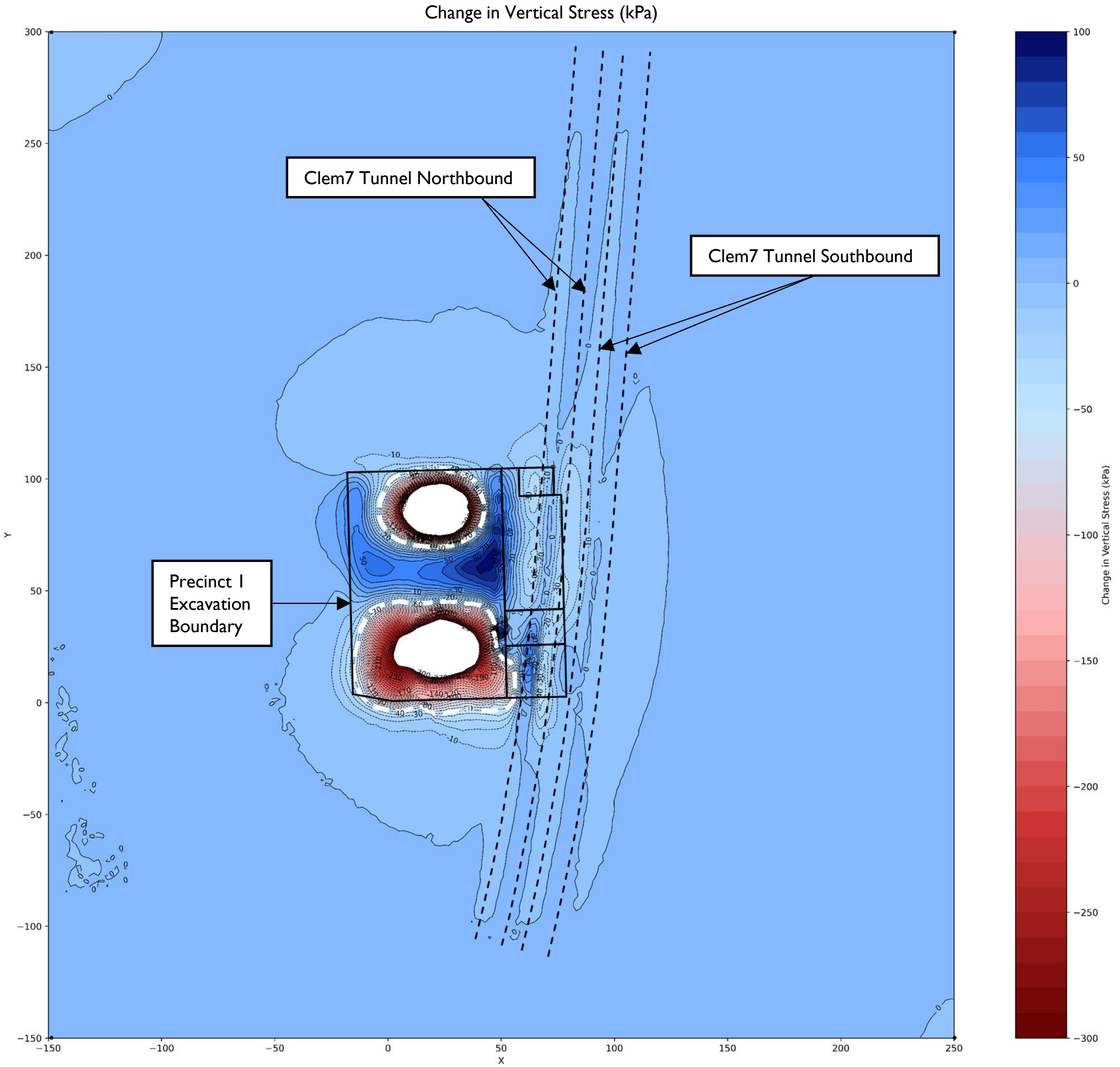
figure: B1



Notes:

- Values shown are the change in vertical stress following excavation of the tunnels (i.e. they reflect the change in vertical stress following basement excavation and application of building loads).
- Negative values represent an increase in vertical compressive stress (i.e. downwards).
- Positive values represent a decrease in vertical compressive stress (i.e. upwards).
- TLDM nominated limit of 50kPa stress increase highlighted in white dash line

by:	DL	*client:*	Phillip Usher Constructions
date:	30/4/2026	*project:*	Mark Lane Precinct
approved:	DJC	*location:*	Brisbane, QLD
scale:	As per axis	*title:*	*Plan - Overall*
		job no:	B01554-1

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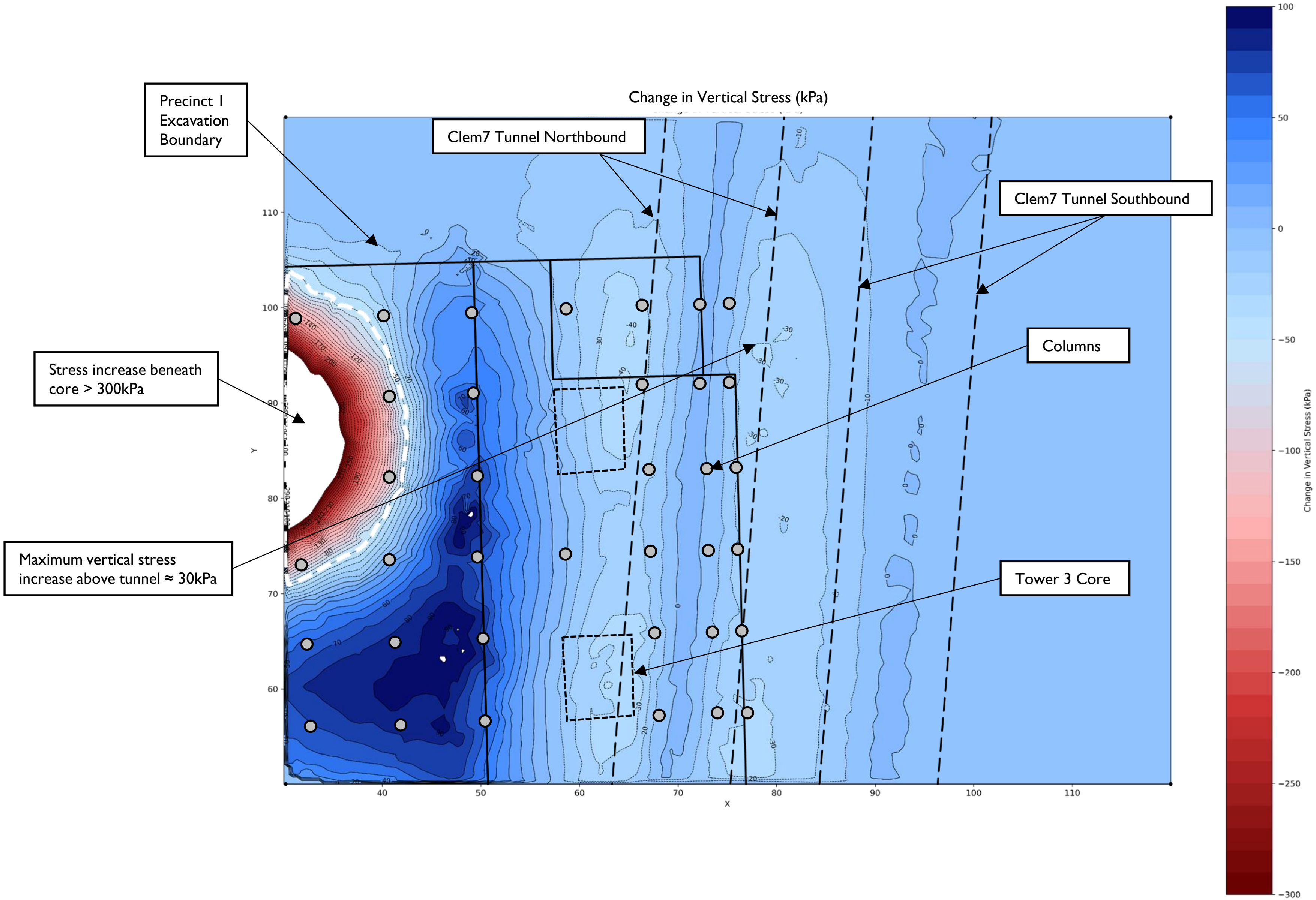
figure: B2



Notes:

- Values shown are the change in vertical stress following excavation of the tunnels (i.e. they reflect the change in vertical stress following basement excavation and application of building loads).
- Negative values represent an increase in vertical compressive stress (i.e. downwards).
- Positive values represent a decrease in vertical compressive stress (i.e. upwards).
- TLDM nominated limit of 50kPa stress increase highlighted in white dash line

by:	DL	client:	Phillip Usher Constructions
date:	30/4/2026	project:	Mark Lane Precinct
approved:	DJC	location:	Brisbane, QLD
scale:	As per axis	title:	*Plan - North*
		job no:	B01554-1

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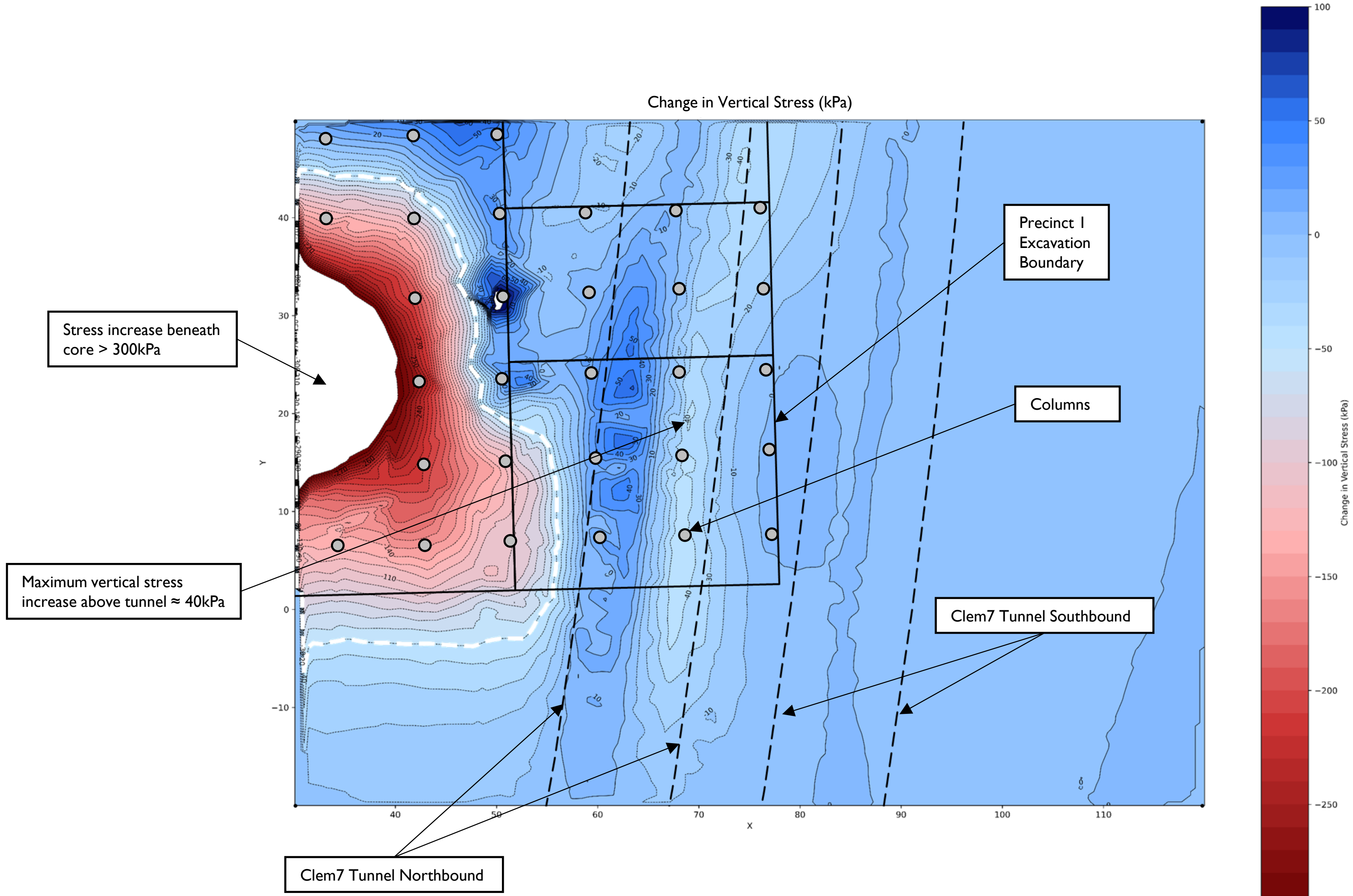
figure: B3



Notes:

- Values shown are the change in vertical stress following excavation of the tunnels (i.e. they reflect the change in vertical stress following basement excavation and application of building loads).
- Negative values represent an increase in vertical compressive stress (i.e. downwards).
- Positive values represent a decrease in vertical compressive stress (i.e. upwards).
- TLDM nominated limit of 50kPa stress increase highlighted in white dash line

by:	DL	*client:*	Phillip Usher Constructions
date:	30/4/2026	*project:*	Mark Lane Precinct
approved:	DJC	*location:*	Brisbane, QLD
scale:	As per axis	*title:*	*Plan - South*
		job no:	B01554-1

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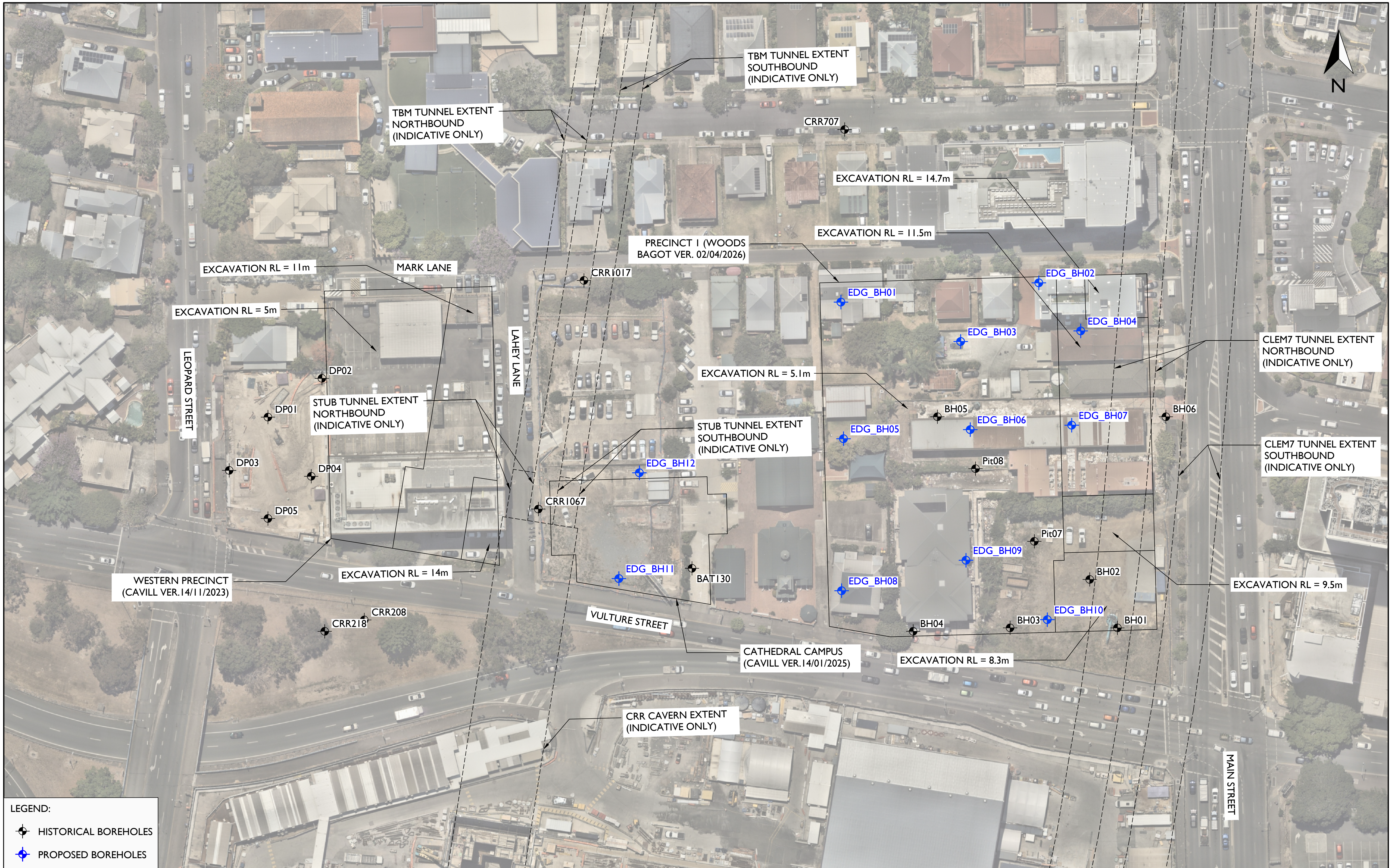
figure: B4

Appendix C

Additional Proposed Ground Investigations



A	INITIAL ISSUE	DL	07/04/2026	DJC
	Revisions/Descriptions	Drawn	Date	Approved

Dwg_Filename.dwg

SURVEY DATA

Coordinate System

Height Datum

Base Plan

1:1000 0 12.5 25m

Dimensions shown in metres except where shown otherwise

ENGINEERING CERTIFICATION (RPEQ)

ENG. AREA	NAME	SIGNATURE	NO.	DATE

Drawn	DL
Checked	DJC
Designed	DL
Verified	DJC

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PHILIP USHER CONSTRUCTION
KANGAROO POINT PRECINCT
KANGAROO POINT, BRISBANE
PROPOSED INVESTIGATIONS PLAN

Job No.	B01554-1	
DRG No.	B01554-1XX_XXX	A
Client Ref.	Client Ref.	

File Location:
F:/Projects/B01017-1......