Geotechnical Investigation Report

Project

Preliminary Geotechnical Report
30 Diggings Terrace, Thredbo NSW

Prepared for

Hidali P/L c/o Bellevarde Constructions Pty Ltd

Date

4 May 2022

Report No

13526-GR-1-1 Rev E

NSW GOVERNMENT
Department of Planning
and Environment

Issued under the Environmental Planning and Assessment Act 1979

Approved Application No DA 22/4825

Granted on the 1 August 2022

Signed M Brown

Sheet No 10 of 18

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0	6/9/2021	Original issue	HP	MAG
A	15/9/2021	Updated for PMI structural drawings	HP	MAG
B	8/12/2021	Updated for PMI structural drawings	HP	MAG
C	4/2/2022	Updated for PMI structural drawings	HP	MAG
D	7/3/2022	Updated for PMI structural drawings	HP	MAG
E	4/5/2022	Revised DA reference, updated drawings and risk assessment	HP / MAG	MAG

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Appendices

1 INTRODUCTION

Alliance Geotechnical Pty Ltd (Alliance) is pleased to submit this Geotechnical Interpretive Report (GIR) to Hidali P/L c/o Bellevarde Constructions Pty Ltd (the client) for the proposed development at 30 Diggings Terrace, Thredbo NSW (the Site) – ref DA22/4825. To assist with this report Alliance have been provided the following documents:

- Geotechnical Report by Coffey Geotechnics, Reference No,: GEOTLCOV23158AA-AB Rev 1 dated 14 May 2007 (Appendix A);
- Excavation Plan and Details drawings prepared by PMI Engineers, Drawing Nos. S02-A(1) dated 29/11/2021, S10(5) dated 28/2/2022, and S10a(5), S10b(6) and S10c(5), S10d(3), and S10e(3), and S10f(3) all dated 29/4/2022 (Appendix B);
- Foundation plan drawing Prepared by PMI Engineers, Drawing No. S15, dated 29/11/2021 (Appendix B);
- Geotechnical Report by Crozier Geotechnical Consultants, Project No.: 2019-121 dated August 2019 with reference to earlier boreholes by Coffey and including completed Kosciuszko Thredbo (KT) Form 1;
- Preliminary Site Retention Design Statement and drawing by Bond James Murtagh dated 8 October 2020;
- Determination of Development Application DA 10064, Applicant; Hidali Pty Ltd for site Black Bear Inn, Lot 794 DP 1119757, Diggings Terrace, Thredbo Village, Thredbo Alpine Resort, Kosciuszko National Park, dated 17 May 2021 – further resubmitted as DA22/4825;
- Popov Bass Architectural drawings “Black Bear – Apartments” last dated 16 December 2020 (Rev 7); and
- Site Survey Plan by Peter W Burns, Reference 3576, Drawing No.: CD01, Rev C dated 24 September 2007

Alliance has agreed to provide this report based on the documents above, the key being the site investigation and geotechnical report completed by Coffey in 2007 and the Crozier Geotechnical Report. Additional verification geotechnical site investigation work was undertaken post-demolition of the existing building and is separately reported in technical memo 13526-GR-2-1 dated 8/12/2021.

This Revision E of the report includes a revised Kosciuszko Alpine Resorts Geotechnical Policy Form 1 Declaration and Certification attached as Appendix C.

2 PROPOSED DEVELOPMENT

Based on the provided architectural drawings, it is understood that construction activities associated with the proposed development include:

- Demolition of the existing building “Black Bear Inn”;
- Construction of a seven-storey building, including a cellar basement level (the lowest level). Four of the levels are below the street level of Diggings Terrace;

- The existing ground surface is a moderately steep slope, so excavation depths vary significantly between little to no excavation at the northern end and up to approximately 9.0m at the southern end. There are three stepped excavation levels on the site, best illustrated in Figure 1, which are:
  - The carpark level which is RL 1,388.2m
  - The restaurant / lobby level which is approximately RL 1,382m
  - The cellar basement floor level which is approximately RL 1,379.3m.

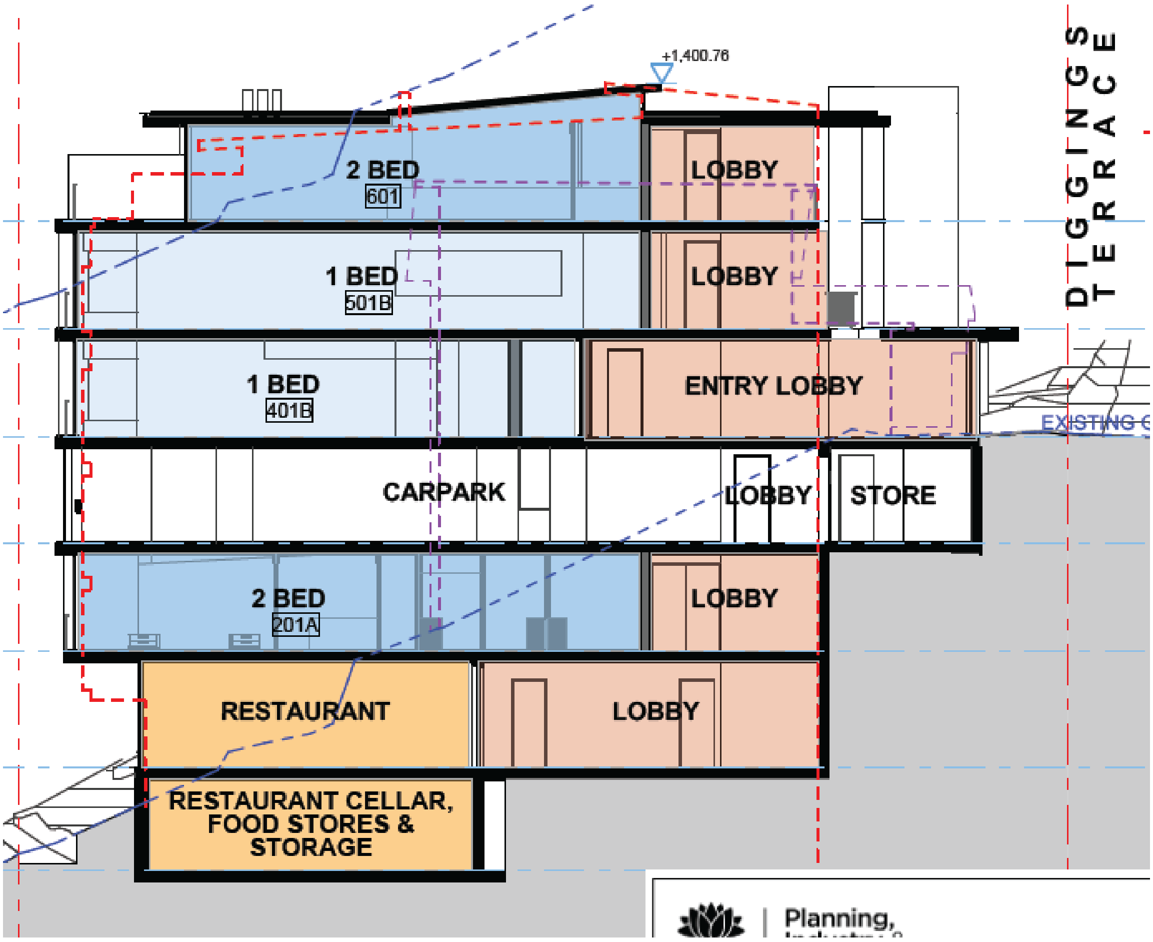

Figure 1: Section looking east (extracted from Popov Bass Architectural Drawings)

Based on the architectural drawings, the proposed building has approximate setbacks of 2.6m from the northern boundary, 3.0m from the eastern and western boundaries, and 4.0m to 6.5m from the southern boundary.

3 SITE DESCRIPTION AND REGIONAL GEOLOGY

The site is located within the Thredbo Alpine Village and Ski Resort, an area which consists predominantly of ski lodges, restaurants and other commercial buildings. The Site is irregular square-shaped block of land with an approximate total area of 675m^2. Based on aerial images and publicly available information, it is currently occupied by "Black Bear Inn", a three-storey ski lodge and restaurant. It is bound by other ski lodges to the North, East and West, and Diggings Terrace to the South as shown in Figure 1.

The NSW Seamless Geology Project (May 2021) indicates the site is underlain by Mowambah Granodiorite (*Sbum*). Granodiorite is a medium to coarse grained intrusive igneous rock, similar to granite, containing quartz and plagioclase feldspar as its primary constituents.

We note the Crackenback Fault runs parallel and very close (less than 10 m) to the northern boundary of the site. This could locally impact the integrity of the bedrock at the site.

Figure 2: Site boundary with respect to the NSW Seamless Geology Map and 20m contours (extracted from minview.geoscience.nsw.gov.au)

4 PREVIOUS SITE INVESTIGATION

Two rounds of intrusive site investigations have been completed by Coffey Geosciences in June 2000 and June 2003. The details of this fieldwork can be found in their report referenced above.

We note that both of the boreholes were drilled at the southern end of the site, on the roadside, presumably due to access constraints. No information is available for the northern end.

A site walkover and inspection were also completed by Crozier Geotechnical Consultants on 21 May 2019. The details of this can be found in their report referenced above.

We have consolidated and summarised the results of the above in Section 4.1 below

4.1 Results

Summarised descriptions of the encountered subsurface geotechnical units are provided in Table 1.

Table 1 – Summary of Subsurface Profile

Soil Profile	Depth and RL to Top of Unit	
	BH1	BH2
Fill / Colluvium – Silty SAND and SILT with gravel fragments, loose density	1.5 mbgl* ~ RL 1,390.1	1.5 mbgl ~ RL 1,391.4
Extremely Weathered Granodiorite– Silty SAND, medium dense to very dense	1.6 mbgl ~ RL 1,388.5	1.45 mbgl ~ RL 1,389.95
Highly Weathered Granodiorite, medium to high strength ‘corestones’ surrounded by extremely weathered material of very low to low strength.	4.7 mbgl ~ RL 1,385.4	3.5 mbgl ~ RL 1,387.9
Termination Depth (m)	11.4 mbgl ~RL 1,378.7	3.5 mbgl ~RL 1,387.9

* mbgl = metres below ground level

Detailed engineering logs including defects and seams are provided in Appendix A of the Coffey Geotechnics report.

4.2 Groundwater

A piezometer was installed in BH1 and a standing groundwater table was interpreted by Coffey at 9.77mbgl (RL 1,380.3m at Diggings Terrace and RL 1,285.0m at the northern boundary of the site). Based on this and experiences in nearby developments, we expect that the proposed development is likely to encounter minor inflows at the base of the excavation, particularly after rainfall events or snow melt, but is unlikely to intersect the standing groundwater table. It should be noted that groundwater conditions are subject to seasonal variations and major weather events (i.e. prolonged rainfall).

5 COMMENTS AND RECOMMENDATIONS

5.1 Excavation Conditions

Based on the subsurface conditions encountered and summarised in Table 1, bulk excavations are expected to encounter loose sands (fill /colluvium) to an average depth of 1.5m overlying extremely weathered granodiorite which can be characterised like a very weakly cemented, medium dense to very dense silty sand. Excavations through these overlying soils are expected to be readily achievable using conventional earthworks equipment such as a tracked excavator.

The majority of the basement slab and footings are expected to be founded in highly to extremely weathered granodiorite.

Assessment of material excavatability can be based on the method published by Pettifer and Fookes (1994). The degree of excavatability of rock is based on its Point Load Index (Is_{50}) and fracture spacing. Excavatability categories range from easy to hard digging, through easy to hard ripping.

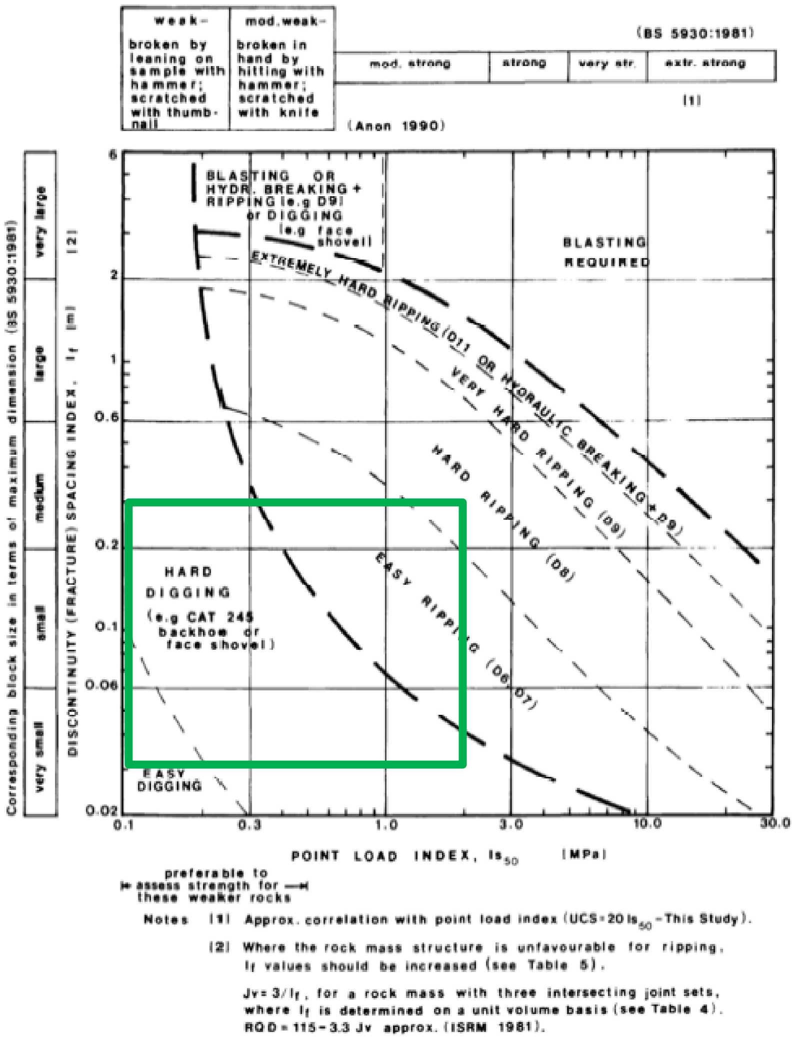

Figure 3: Excavatability nomogram (extracted from Pettifer and Fookes (1994))

Our review of the borehole logs indicates that bedrock conditions encountered were generally closely spaced with defect spacing in the order of 30mm to 300mm. It is therefore expected that the excavation conditions will vary greatly from easy to hard digging and easy to hard ripping conditions. This will be largely dependent on the size of the high strength ‘corestones’ and proportion of extremely weathered material surrounding it. Excavation conditions are likely to get more difficult with depth. This advice may be able to be refined with additional borehole investigations. Local experience indicates that some larger corestones may need to be broken up with rock breakers, rotary rock grinding or rock sawing.

Low vibration equipment will be necessary near all site boundaries where vibrations could impact on adjacent building footings and structures.

Alternatively, to limit the transmission of vibrations, it is recommended that the perimeter of the excavation be saw-cut prior to any ripping or excavation of the rock mass. Blocks of the saw-cut rock mass can then be progressively dislodged using small rock hammers and lifted out without generating large vibrations. A rotary rock grinder may also need to be used to trim rock faces instead of a large impact hammer.

Vibration monitoring may be required prior to excavation due to its proximity to residential boundaries.

Generally, the ground vibration Peak Particle Velocity (PPV) should be limited to 5mm/s at the property boundaries. The maximum 5mm/s vibration limit is not expected to be exceeded provided that rock breaker equipment and excavation methods are restricted to those listed in Table 2 below.

Table 2 – Recommendations for Rock Breaking Equipment

Distance from Adjacent Structure (m)	Maximum Peak Particle Velocity 5mm/s	
	Equipment	**Operating Limit (% of Maximum Capacity)**
1.5 to 2.5	hand-operated jack-hammer only	100

It is recommended that vibration monitoring be included as part of the geotechnical monitoring program.

A dilapidation survey on nearby structures and infrastructure is recommended to be undertaken by a structural engineer prior to the commencement of any site excavations. The report should include precise measurements of the existing defects and cracks presented with the relevant photos.

5.2 Excavation Stability and Batter Slopes

The excavation stability can be controlled by adopting a combination of a shoring systems and unsupported cuts, as described below.

5.2.1 Unsupported Batter Slopes in Soil

Unsupported temporary batter slopes are feasible provided that the excavations do not extend below the ‘zone of influence’ of any adjacent structures, road and infrastructure (i.e. a 45° line from the footing of adjacent structures or infrastructures). The feasibility of using unsupported batter slopes will depend on the footing level of the adjoining structures and infrastructure, surrounding services invert levels, and should be assessed by a structural designer.

Based on the proposed basement excavation setbacks, temporary batter slopes within the upper soil/rock layers (fill, colluvium and extremely weathered bedrock) may be feasible in parts of the site.

Temporary batters up to 2m in height within Fill, Colluvium and Extremely weathered Granodiorite can be excavated to a maximum batter slope of 1.5H:1V provided they are above the water table or within dewatered ground.

If the civil contractor prefers an equivalent benched profile, then a maximum bench height of 1.5m and width of 1.5m could be adopted. This is subject to the installation of surface water drains which direct water away from the cut slope or sub-horizontal drains in the cut face, whichever is assessed as appropriate by a geotechnical engineer.

Alternatively, these batter slopes can be made steeper with the incorporation of shotcrete and soil nails. This would have to be assessed separately (if required) based on specific boundary conditions.

The above recommendations are for batters exposed up to a maximum of three months and provided no surcharge is located along/near the cut crest.

5.2.2 Unsupported Rock Cuts

Based on the proposed basement excavation setbacks, temporary and permanent unsupported batter slopes within highly weathered granodiorite may be feasible on the southern, eastern and western boundaries of the lowest cellar basement level (see Figure 4).

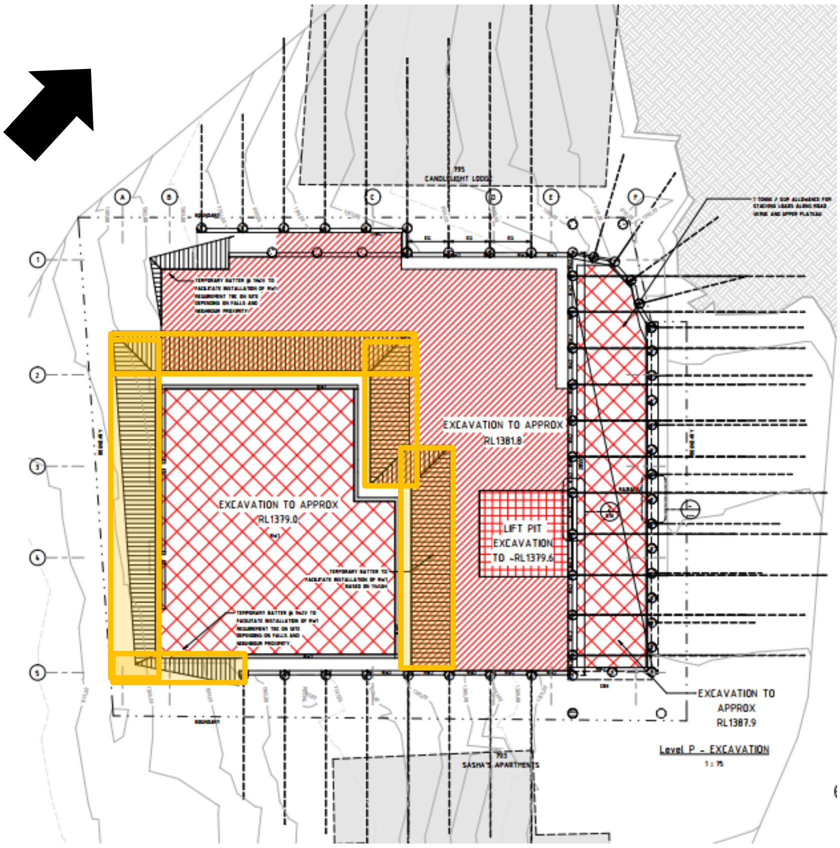

Figure 4: Excavation plan (by PMI) showing the locations where unsupported cuts may be feasible in yellow

Temporary batters within highly weathered granodiorite can be excavated to a maximum batter slope of 1H:1V, provided they are above the water table or within dewatered ground, and not exposed for longer than three months. Slopes which are between 2V:1H and vertical may be possible subject to inspection by a competent geotechnical engineer and carrying out any remedial works such as shotcreting or rock bolting.

5.2.3 Excavation Support

In the areas where temporary batter slopes are not feasible, a suitably designed shoring system is recommended. Anchored contiguous piled walls are recommended. Weep holes or drains (e.g. vertical drains) must be provided behind shotcrete to avoid build-up of hydrostatic pressure in the overburden soils and rock mass. For the southernmost retaining wall with RP2 piles (see Appendix B), the contiguous bored pile wall will need pile spacings no more than 150mm due to the presence of fill material at the edge of Diggings Terrace. Subject to approval, temporary ground anchors are recommended to control wall deflections. Retaining Wall RW2, being in less weathered granodiorite can be permitted to have wider spaced piles. To avoid later complications in removing walings, it is suggested a “one temporary anchor per pile” approach to avoid a need for walings is considered. Use of a capping beam may still be prudent. The lower basement/cellar cut is anticipated to be feasible by unsupported steeply battered rock cut. This must be verified by further deep geotechnical investigation post-demolition prior to further construction.

Any anchoring system should be designed to provide temporary support with long-term lateral support being later transformed on to the permanent structure. Anchors will need to be installed progressively as the excavation proceeds and will require the permission of the adjacent landowners for anchors to be extended into their land. Permissions may be subject to provision of ground anchor installation rights documentation beyond the site boundary. In addition, the adjacent neighbouring footing levels and underground service levels in the road reserve must be confirmed prior to finalising anchor design.

Temporary anchors in highly weathered granodiorite may be designed using an ultimate bond stress of 100kPa. Greater bond stresses may be available at depth subject to further investigation.

Periodic lift-off checks of installed anchors should be carried out during anchor installation to ensure lock off-load is maintained. It is recommended that the anchors be installed and proof-tested in accordance with the requirements of AS4678-2002 and RMS QA Specification B114. It is recommended that an experienced geotechnical engineer be engaged to check the design of the excavation support system.

The specific requirements set out above for excavation support at the upper levels and also the stability of the face should be assessed by an experienced geotechnical engineer as the excavation proceeds. Excavation depths should not exceed 1.5m until it has been inspected by an experienced geotechnical engineer before proceeding further or applying any face treatment.

Survey monitoring should be carried out during the construction of a shoring system to check and confirm that deflections and movements are within tolerable limits accepted in design. Readings should be taken at least every 3m depth excavation, before and after installation of anchors,

5.3 Retaining Structures

The temporary shoring system or permanent retaining wall should be designed in accordance with AS 4678 Earth Retaining Structures.

If it is critical to limit the horizontal deformation an earth pressure coefficient ‘at rest’ (K_0) should be adopted. Where some lateral movement is acceptable, an ‘active’ lateral earth pressure coefficient (Ka) is recommended.

A triangular earth pressure distribution should be adopted for free standing cantilevered walls only. For progressively anchored or propped walls, a rectangular pressure distribution between 6H and 8H should be adopted depending on the structure's tolerance for movement, where H is the retained height in meters.

Recommended design parameters for the design of temporary and permanent support are provided in Table 3 below.

Table 3 – Recommended Parameters for Retention Design

Geotechnical Units	Approx. Depth below Existing Ground Level (m)	c' (kPa)	$\emptyset'$ (degrees)	γ (kN/m3)	Ka	Kp	Ko	E' (MPa)	v'
Fill, Colluvium	0.0 – 1.6	0	30	18	0.33	3.00	0.50	20	0.3
Extremely weathered granodiorite	1.4 – 4.7	0	34	21	0.28	3.54	0.44	100	0.3
Highly weathered granodiorite	3.5+	50	38	24	0.24	4.2	0.38	1,000	0.2

Legend:

$\emptyset'$: Effective Friction Angle

c': Effective Cohesion

$\boldsymbol{\gamma}$: Bulk Unit Weight

Ka: Active earth pressure

Ko: Earth pressure at rest

Kp: Passive earth pressure

E': Elasticity Modulus

ϑ': Poisson's Ratio

The above values assume appropriate measures are taken to provide complete drainage behind the walls such as strip drains protected by geotextile fabrics or weep holes.

An allowable toe resistance for piles in highly weathered granodiorite is 500kPa. This value assumes excavation is not carried out within the zone of influence of the pile toe. The upper 1.0m of the pile socket should not be considered to provide any resistance to allow for some tolerance and disturbance during excavation.

5.4 Footing Recommendation

Both shallow and deep options of foundations could be adopted for the proposed sequence of works. Parameters for both footing options are provided below.

5.4.1 Shallow / Pad Footings

Pad / raft footings may be feasible to found the building structure provided the footings are founded into a natural stratum. As footing dimensions and loads are not yet available, final allowable bearing capacities have not been calculated. Once these details are available, Alliance can assist to optimise the footing size and depth to suit the loading on the founding material.

Bearing capacity is not a soil property but is dependant of footing size, depth, slope and loadings. The parameters provided in Table 4 are for preliminary sizing of shallow footings for centric vertical loads, but can be optimised to consider footing size, depth, slope (ground surface and/or footing base) and actual loadings. A footing subjected to pull out forces should be further assessed geotechnically in addition to bearing capacity for overturning and sliding.

Table 4 – Recommended Parameters for Shallow Foundations

Material	Parameters		
	Ultimate Bearing Capacity (kPa)	Allowable Bearing Capacity (kPa)	Modulus E’ (MPa)
Extremely weathered granodiorite	1,500	500	100
Highly weathered granodiorite*	4,500	1,500	1,000

Notes:

- *Ultimate values occur at large settlements (>5% of minimum footing dimensions)
- *Allowable bearing pressure to cause settlement of <1% of minimum footing dimension.
- *Clean socket of roughness category R2 or better is required

The base of all footings should be inspected by a geotechnical engineer prior to any concrete pours, to confirm the founding material and bearing capacities.

5.4.2 Deep Foundations

Where larger structures are proposed with higher loading conditions, these structures are recommended to be founded on piles that transfer the column loads to more suitable founding strata at depth. The type of pile will depend on the specific ground and groundwater conditions and relative cost. For piles founded in highly weathered granodiorite the following parameters can be adopted:

- An allowable bearing capacity of 1,500 kPa;
- A shaft adhesion of 150 kPa; and
- Young’s Modulus of 1,000 MPa.

Settlements of piles designed using the above loads would be expected to be less than 1% of the minimum footing dimension.

To adopt the shaft adhesion above, a minimum socket of 2 x pile diameters is required into the founding stratum.

If bored piles are adopted, the base of the piles must be inspected during construction to ensure that material of adequate capacity supports each pile and that the piles have been adequately cleaned. Concrete should be poured on the same day shortly after drilling. If groundwater is encountered, concrete shall be placed from the bottom up using a tremie.

Note that the construction of bored piles in the highly weathered granodiorite may require drilling through both extremely weathered material that may cave in, and high strength granodiorite corestones. Allowances such as casing and drilling methods to break high strength rock should be considered by the contractors.

5.4.3 Seismic Activity

Current Australian standards AS 5100 and AS 4678 both refer to AS1170.4 for earthquake actions. As required in AS1170.4 a site sub-soil class of B_e and a minimum acceleration coefficient (a) of 0.10 are recommended.

5.4.4 Construction Inspections

The inspections during the basement excavation should be undertaken at every 1.5m depth interval. The purpose of the inspections is to assess the stability of the unsupported slope and provide recommendations for any remedial works, if required.

Shallow footing excavations should be inspected before installation of the reinforcement cage and pouring concrete, and deep foundations should be inspected during drilling of the piles.

6 SLOPE RISK ASSESSMENT

6.1 Introduction

With the site being in a state of “Stop Work” whilst a revised DA is being considered, the previous slope risk assessment undertaken by Coffey (see appendix A) requires updating as follows.

The risk assessment for the site falls into two parts namely risk to property and risk to life from slope instability. The assessments are generally in accordance with the recommendations of the Australian Geomechanics Society publication, March 2000 and updated 2007 (AGS Guidelines) and in the DIPNR Kosciusko Alpine Resorts Geotechnical Policy. The guidelines recommend a qualitative method of assessment, based on the identification of potential hazard, the likelihood of occurrence and the consequence of failure. The assessments are combined using a risk assessment matrix to produce a qualitative risk assessment for each hazard.

6.2 Identified Hazards and Risks

The potential hazards identified by Coffey in their previous assessment in May 2007, are considered to be essentially unchanged except where the partially completed shoring walls are now in place. Namely:

- Failure of the slope under “High Noon” with debris moving downslope on to the subject site
- Failure of the retaining wall and supported fill forming Diggings Terrace (now the roadside temporary shoring wall)

- Failure of the slope on the subject site (now removed and replaced by the boundary shoring walls to “Candlelight” and “Sasha’s”)
- Failure of the cut slope behind “Mowamba” and downslope of the subject site.

Coffey concluded that the risk to property, at that time, was low to moderate in line with the village-wide risk assessment which was deemed to be acceptable. The risks to life are at better than acceptable levels. The risks to the village are considered to be unchanged and the impact of the proposed development does not change the risk rating from that of the overall village risk.

A brief slope risk assessment was also prepared by Crozier Consultants in their report of August 2019. They made no reference to the earlier Coffey report and considered only two simplified hazards cases for potential slipes within the limits of the site footprint. This gave a risk to property of very low to low and an acceptable risk to life. These are now considered superfluous now that the construction has commenced.

For the site in its interim “Make Safe” state, the hazards are considered to be as followed:

- Failure of internal batter slopes: Property – probability of failure = Rare; consequence to property = minor. Hence risk to property is assessed to be Very Low. Maintenance of surface to minimise surface water infiltration is required alongside control of surface water run-off to prevent gullying.
- Temporary shoring wall failure (ref Candlelight/ Sasha’s and Diggings Terr): Property – probability of failure = Rare; consequence to property = Major. Hence risk to property is assessed to be Low. On-going monitoring of lateral deflection is primary control measure.
- Failure of the cut slope behind “Mowamba” and downslope of the subject site; Property – probability of failure = Rare; consequence to property = Medium. Hence risk to property is assessed to be Low. Maintenance of surface to minimise surface water infiltration is required alongside control of surface water run-off to prevent gullying.
- Failure of the slope under “High Noon” with debris moving downslope on to the subject site – this is unchanged – ie Low – Medium (village wide accepted risk level). These risks are under third party control.

For risk to life, reference is made back to the commentary by Coffey in their report. The received risk to life due to the development are better than acceptable to society. The village-wide risk to life based on the historic failure (The “Thredbo Landslide”) the perception is for a higher risk. However, as the greatest risk is considered to be (based on the historic failure) from fast moving debris flows landslides, these are extremely rare and with particular regards to the subject site, there are no geomorphological features (gulley features) upslope of the subject site. All new man-made structures or slopes above the site have been constructed to the best standards (post- Thredbo Landslide) and are again considered to be rare. Hence, the risk to life is assessed to be very low and at better than acceptable levels of societal risk.

All development at the site is to be undertaken in accordance with sound engineering principals and good hillside practice. Hence, the site is considered suitable for the proposed development.

7 REFERENCE TO SECTION 4.1 OF DEPARTMENTS GEOTECHNICAL POLICY

Section 4.1 of the Policy states:

> "4.1 The geotechnical report to be submitted with a development application required under this policy is to include the following elements:
>
> (a) An assessment of the risk posed by all reasonably identifiable geotechnical hazards which have the potential to either individually or cumulatively impact upon people or property upon the site or related land to the proposed development in accordance with the guidelines set out in 'Landslide Risk Management Concepts and Guidelines' first published in the Australian Geomechanics Journal, Vol. 35 No.1, March 2000 (guidelines). Note: Appendix A provides an example of qualitative terminology for use in assessing risk to life and property.
>
> (b) Plans and sections of the site and related land from survey and field measurements with contours and key features identified, including the locations of the proposed development, buildings/structures on both the subject site and adjoining site, stormwater drainage, sub-surface drainage, water supply and sewerage pipelines, trees and other identifiable geotechnical hazards.
>
> (c) Details of all site inspections and site investigations and any other information used in preparation of the geotechnical report. A site inspection is required in all cases. Site investigation may require sub- surface investigation; appropriate investigation may involve boreholes and/or test pit excavations or other methods necessary to adequately assess the geotechnical/geological model for the site. At Thredbo, reference may be made to the suite of existing geotechnical data and regional studies held by Kosciuszko Thredbo Pty Ltd, as part of the information to be used in assessing the site. Where similar information data exists for the other Kosciuszko Ski Resorts then this information may be similarly used in assessing the site.
>
> (d) Photographs and/or drawings of the site and related land adequately illustrating all geotechnical features referred to in the geotechnical report, as well as the locations of the proposed development.
>
> (e) Presentation of a geological model of the site and related land showing the proposed development, including an analysis of sub-surface conditions, taking into account thickness of the topsoil, colluvium and residual soil layers, depth to underlying bedrock, and the location and depth of ground-water.
>
> (f) A conclusion as to whether the site is suitable for the development proposed to be carried out either conditionally or unconditionally. This must be in the form of a specific statement that the site is suitable for the development proposed to be carried out, subject to the following conditions:
>
> (i) Conditions to be provided to establish the design parameters, including but not limited to;
> • footing levels and supporting rock quality,
> • degree of earth and rock cut and fill,
> • recommendations for excavation batters,
> • parameters, bearing capacities, and recommendations for use in the design of all structural works including all footings, retaining walls, surface and sub-surface drainage,
> • recommendations for the selection of building structure systems consistent with the geotechnical assessment of risk, and
> • signing of form 2 as the mechanism to check that these parameters have been interpreted correctly and incorporated into the structural design
> (ii) Conditions applying to the detailed design to be undertaken for the construction certificate, including but not limited to;

• any structural design relating to geotechnical aspects of the proposal is to be checked and certified by a suitably qualified and experienced geotechnical engineer,
• any other design conditions the geotechnical engineer preparing the geotechnical report believes are required in the design phase in order to ensure the design will achieve the "acceptable risk management" level as defined in this policy for potential loss of both property and life, and
• signing of form 2 as the mechanism to check that these design conditions have been interpreted correctly and incorporated into the structural design.
(iii) Conditions applying to the construction phase, including but not limited to;
• constructed works which require inspection and/or signoff by a suitably qualified and experienced geotechnical engineer. The report must highlight and detail the inspection regime to provide the builder with adequate notification for all necessary inspections,
• any other construction conditions including works methodology and temporary works that the geotechnical engineer preparing the geotechnical report believes are required in the construction phase to ensure the design will achieve the "acceptable risk management" level as defined in this policy for potential loss of both property and life, and
• signing form 3 as the mechanism to verify that the above methodology and inspections have been completed in accordance with the report.
(iv) Conditions regarding ongoing management of the site/structure, including but not limited to;
• any conditions that may be required for the ongoing mitigation and maintenance of the site and the proposal, from a geotechnical viewpoint.
(g) A copy of form 1 bearing the original signature of the engineering geologist or geotechnical engineer as defined by this policy, who has either prepared or technically verified the geotechnical report."

Our response to this is summarised in the following table:

Part 4.1 part	Alliance Report section	Comments
(a)	6	Cross-reference to the earlier Coffey and Crozier reports is suggested.
(b)	3, 4	Cross-reference to the earlier Coffey and Crozier reports is suggested.
(c)	4	Cross-reference to made to Alliance technical memo 13526-GR-2-1 dated 8/12/2021. Cross-reference to the earlier Coffey and Crozier reports is also suggested.
(d)	Figures 1, 2 & 4	Cross-reference to the earlier Coffey and Crozier reports is suggested. Cross reference to 13526-GR-4-1 Rev D, dated 24 March 2022 - "Stop Work Order – Made Safe technical report.
(e)	Figure 1, Table 1 and PMI Engineers drawings S10a Rev 5 copy within Appendix B of this report	See also Drawing Figure 2 – "Section A-A'" of the Coffey report.
(f)	Final paragraph of Section 6.2	This is subject to sub-clauses i, ii, iii and iv of clause (f) of Section 4.1 of the Policy.
(g)	Appendix C of this report.	

8 LIMITATIONS

In addition to the limitations inherent in site investigations, it must be pointed out that the recommendations in this report are based on assessed subsurface conditions from limited investigations. To confirm the assessed soil and rock properties in this report, further investigation is required including coring and strength testing of rock and should be carried out post-demolition once access permits.

It is recommended that a qualified and experienced Geotechnical Engineer be engaged to provide further input and review during the design development; including site visits during construction to verify the site conditions and provide advice where conditions vary from those assumed in this report. Development of an appropriate inspection and testing plan should be carried out in consultation with the Geotechnical Engineer.

This report may have included geotechnical recommendations for design and construction of temporary works (e.g. temporary batter slopes or temporary shoring of excavations). Such temporary works are expected to perform adequately for a relatively short period only, which could range from a few days (for temporary batter slopes) up to six months (for temporary shoring). This period depends on a range of factors including but not limited to: site geology; groundwater conditions; weather conditions; design criteria; and level of care taken during construction. If there are factors which prevent temporary works from being completed and/or which require temporary works to function for periods longer than originally designed, further advice must be sought from the Geotechnical Engineer and Structural Engineer.

This report and details for the proposed development should be submitted to relevant regulatory authorities that have an interest in the property (e.g. KT, NP&WS and NSW Planning) or are responsible for services that may be within or adjacent to the site, for their review.

Alliance accepts no liability where our recommendations are not followed or are only partially followed.

9 REFERENCES

AS1726-1993 - Geotechnical Site Investigations

AS 2159-2009 - Piling - Design and Installation

AS4678 – Earth Retaining Structures

APPENDIX A – COFFEY GEOTECHNICAL REPORT, MAY 2007 & CROZIER REPORT AUGUST 2019

coffey geotechnics
SPECIALISTS MANAGING THE EARTH

BLACK BEAR INN

Alex Popov & Associates
Lot 49 Diggings Terrace, Thredbo

GEOTLCOV23158AA-AB Revision 1
14 May 2007

Coffey Geotechnics Pty Ltd ABN 93 056 929 483
8/12 Mars Road Lane Cove West NSW 2066 Australia

coffey geotechnics
SPECIALISTS MANAGING THE EARTH

14 May 2007

Alex Popov & Associates
2 Glen Street
Milsons Point, NSW 2061

Attention: Melissa Doherty

Dear John

RE: Black Bear Inn

Lot 49 Diggings Terrace,Thredbo

Please find enclosed our revised report regarding geotechnical investigations undertaken for the proposed redevelopment of Lot 49 Diggings Terrace in Thredbo Alpine Village.

Should you have any queries regarding any of the matters raised in this report, please do not hesitate to contact the undersigned on 9911 1000.

For and on behalf of Coffey Geotechnics Pty Ltd

Paran Moyes

Senior Geotechnical Engineer

Distribution: Original held by Coffey Geosciences Pty Ltd

6 copies Alex Popov & Associates

1 copy Coffey Geotechnics Pty Ltd

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1 INTRODUCTION

This report prepared by Coffey Geotechnics Pty Ltd (Coffey) on behalf of Alex Popov & Associates provides a review of previous advice for the proposed reconstruction at Lot 49 Diggings Terrace, (currently known as Black Bear Inn), Thredbo Alpine Village. The original geotechnical investigation was carried out by Coffey Geosciences Pty Ltd (Ref. S20449/2 – AD, dated 12 June 2003), on behalf of Elwyn Wyeth Management Architecture, This review, based on our previous report provides advice with regards to a revised layout of the proposed development.

Coffey Geosciences Pty Ltd (Coffey) carried out geotechnical investigation in June 2000 for a proposed two-storey extension to the southern side of the existing Black Bear Inn. This previous investigation involved the drilling of two boreholes up to 4.4m deep located at the front of the lodge adjacent to Diggings Terrace.

We understand that the purpose of this geotechnical report is to address slope stability concerns as well as provide geotechnical parameters and constraints for design and construction of the development.

2 PROPOSED DEVELOPMENT

Lot 49 currently contains the 40-year-old Black Bear Inn, which is proposed to be demolished as part of the new development. Our previous report (Ref. S20449/2 – AD, dated 12 June 2003) was based on a proposed development comprising a seven level ski lodge, of which four levels were to be excavated below the level of Diggings Terrace in a series of benches extending downslope.

Based on the supplied architectural sketches, the current lodge proposal includes construction of a six level ski lodge with a footprint area of approximately 295m^2. It is understood that the proposed building is to occupy the same position on the site, although the shape of the building has changed.

3 FIELD WORK

Field work for the June 2003, investigation, comprised the drilling of a single borehole using a trailer mounted drilling rig. The borehole (BH1) was drilled using continuous spiral flight augers to a depth of 4.7m, extending through the upper fill and soil materials, encountering V-bit refusal in the underlying weathered granodiorite bedrock. The borehole was then continued in extremely weathered granodiorite using rotary coring techniques to a depth of 11.4m. The borehole was drilled at the same location of the previous borehole (BH1) drilled by Coffey in June 2000, which terminated at 4.4m depth. Information (including SPT information) from the previous borehole log was used for the borehole drilled for the Coffey Geosciences Pty Ltd 2003 investigation. At the completion of drilling, borehole BH1 was completed with a PVC standpipe piezometer to allow for the monitoring of groundwater levels. Monitoring by Kosciusko Thredbo (KT) staff on behalf of Coffey 11 days after drilling, measured the standing groundwater at a depth of 9.77m.

The fieldwork was undertaken in the full time presence of one of our Geotechnical Engineers, who identified the previous investigation location, boxed and colour photographed the rock core on site. Engineering logs of the boreholes and colour photographs of the recovered rock core are presented in Appendix A together with Explanation Sheets that define the terms and symbols used in their preparation. Borehole locations were obtained relative to existing surface features, and are shown on Figure 1. Reduced collar levels at borehole locations were estimated from ground surface contours from a topographic plan of Thredbo Village, prepared by Peter W. Burns Surveyors.

4 SITE CONDITIONS

4.1 Surface Conditions

Thredbo Alpine Village occupies the footslopes and valley floor of the Thredbo Valley. The Thredbo River runs in west-east direction along the valley floor. The older portion of the village is situated on the north facing, southern valley slope, where overall ground slopes are of the order of 25°. Locally ste eper slopes are present where cutting and filling has been undertaken for development of the Village. Towards the base of the valley, ground slopes are of the order of about 5° to 15°. Several older gully and spur features are evident above and within the Village.

Black Bear Inn is located near the centre of the older portion of Thredbo Alpine Village, on the southern slopes of Thredbo Valley. Overall ground slopes in the vicinity of the lodge are of the order 20°. The lodge is located on the downslope side of Diggings Terrace, which is a sealed village road formed by cut and fill. Previous exposures (observed by Coffey in 1999) in the 0.8 m high road excavation on the high side of Diggings Terrace indicated a thin topsoil/colluvial layer over weathered granodiorite bedrock.

The existing Black Bear Inn lodge is four storeys high on the northern (downslope) side, and two storeys high on the (upslope) southern side, stepping downslope, with internal walls. Foundation conditions for the existing building are not known, and apart from one crack observed in a lodge foundation wall during a walkover assessment of the Village in 1997, our observations suggest that the structure is performing satisfactorily. A 2.5m high stone retaining wall supporting the road fill is located on the southern (upslope) side of the lodge.

4.2 Subsurface Conditions

The underlying bedrock within the Thredbo Valley is Mowamba Granodiorite. Based on previous investigations undertaken by Coffey Partners International Pty Ltd within Thredbo Alpine Village, the typical natural subsurface profile would comprise topsoil and colluvium to depths of 0.5m to 1.5m, overlying residual soil to extremely weathered bedrock. The bedrock is generally extremely to highly weathered weathered to depths in excess of 20m. In isolated locations in the village, moderately weathered granodiorite is exposed at the surface. Where cut and fill techniques have been employed for the construction of roads, the fill materials are typically loose, and variable in composition.

The generalised subsurface profile encountered within the current and previous boreholes is summarised in Table 1.

TABLE 1 - GENERALISED SUBSURFACE PROFILE – LOT 49

Unit	Depth to Base of Unit (m)	Description
Fill (From Diggings Terrace)	1.45 to 1.6	FILL: Silty SAND, fine to coarse grained, brown, some fine to coarse grained gravel and gravel sized granodiorite fragments, moist, loose to medium dense (?).
Topsoil / Colluvium	2.7	Silty SAND / Sandy SILT: Sand is fine to coarse grained, fines are low plasticity to non-plastic, brown to dark brown, with a trace of fine grained gravel, moist, loose.

Unit	Depth to Base of Unit (m)	Description
Extremely to Highly Weathered Granodiorite (cored rock)	>11.4	GRANODIORITE: Extremely weathered, evident in drill cuttings as a Silty SAND; fine to coarse grained, pale brown and brown, fines are non-plastic, trace of fine grained gravel, dry to moist, medium dense to very dense. Contains probable distinctly weathered corestones. Cored as extremely to highly weathered granodiorite, variable strength ranging between very low to high, coarse grained, pale brown/pink/white and black speckled, massive. Minor core loss interpreted as a zone of weaker material.

An interpreted geotechnical cross-section through the site is shown in Figure 2. The figure shows that the depth of fill and colluvial materials overlying the weathered granodiorite bedrock in the vicinity of the proposed development is about 2.7m (as identified in BH1) near the western edge of the lot, and about 2.5m further east along the face of 'Black Bear Inn' on Diggings Terrace where BH2 was drilled. Borehole BH2 had been drilled in 2000 for a previously proposed development.

Groundwater was observed in the piezometer in borehole BH1 at 9.77m. This level is similar to other piezometers constructed by Coffey along Bobuck Lane and Diggings Terrace. The level is expected to rise between 0.5m to 1m following the spring thaw and significant rainfall events. However, the installation of an improved stormwater system and some 150m long horizontal, subsoil drains within the village has generally lowered the groundwater table on average by 2m (in the area of 'Pindari' Lodge) from pre-July 1997 levels.

5 SLOPE STABILITY RISK ASSESSMENT

5.1 Risk Assessment Procedure

The risk assessment for the proposed lodge site has considered two general issues, namely the risk to property, and the risk of loss of life from slope instability. The assessment of risk to property has been carried out using a qualitative risk assessment methodology, a copy of which is included in Appendix B. The procedure is the methodology suggested in a paper published in an Australian Geomechanics Society publication, March 2000 (AGS Guidelines), and in the DIPNR (Department of Infrastructure Planning and Natural Resources) Kosciusko Alpine Resorts Geotechnical Policy. This system is a qualitative method of assessment, based on an identification of likelihood of occurrence, and consequences to the structure for the identified hazards. These assessments are then combined using a risk assessment matrix to obtain a qualitative risk assessment for the site for each hazard.

5.2 Identified Hazards

The potential hazards considered in the risk assessment for the proposed development of Lot 49 are detailed below:

- Failure of the slope under 'High Noon' with debris moving downslope to Lot 49;
- Failure of the retaining wall and supported fill in Diggings Terrace;
- Failure of the slope under 'Black Bear Inn' (Lot 49); and
- Failure of the cut slope behind 'Mowamba' and downslope of Lot 49.

The above hazards are based on the proposed developments being constructed in accordance with the discussion and recommendations provided in this report. The hazard rating for the sites may be higher if the development is not constructed in accordance with recommendations of this report. The potential failure risk of the abovementioned hazards has been reduced by the slope improvement measures installed by KT since the Thredbo Landslide. Coffey identified in 1997 that elevated groundwater beneath the Thredbo slopes can be a major risk factor. Subsequent slope improvement measures in the southern slopes of Thredbo Village included improved roof water collection systems, installation of new stormwater drains and the drilling of some 150 horizontal drains, which have been installed. These slope improvement measures have assisted the slope instability risk by generally lowering groundwater levels. In addition, sections of filled embankments within and above the Village have been reconstructed and supported by engineered retaining walls.

5.3 Risk to Property

The assessment of the risk to property in terms of the qualitative risk assessment for various hazards, and assessed likelihood and consequence of each hazard is presented in Appendix C.

The overall outcome for the risk assessment process for the proposed property on Lot 49 is assessed as **low to moderate risk** in accordance with the risk matrix provided in Appendix C. Coffey considers that, provided the development on Lot 49 is carried out in accordance with sound engineering principles and good hillside practice (refer to Appendix D) that the development should be suitable for the site and the risk classification should not increase above the assessed **low to moderate risk**.

5.4 Risk of Loss of Life

A report prepared by Coffey in 2000 for the assessment of the risk of loss of life within Thredbo Village considered the types of landslides that may result in loss of life; assessed the risk of loss of life associated with those types of landslide; and compared the result to suggested guidelines for tolerable risk.

The Thredbo Landslide assessment indicated that loss of life is generally associated with fast moving landslides derived from the natural slopes. Cut and filled slopes are a small percentage of the total slopes in the area and the risk to life needs to be assessed on a case by case basis. The Coffey assessment for Thredbo concluded that the risk of loss of life from the natural hazards is far lower than the suggested criteria in the AGS Guidelines, and lower than many risks to which people are already exposed to and appear to accept in Australia.

Of the conceivable hazards for the proposed lodge site, those with the possibility of becoming fast moving landslides include debris flows involving the natural slopes above the site; rockfalls leading to boulders rolling down the slope; and the failure of small cut or fill slopes within the site.

Presented below is a general discussion on the types of hazards that may pose a risk to residents in the proposed lodge site.

- Fast Moving Debris Flow Landslides: The likelihood of fast moving debris flows involving the natural and altered slopes above, at and below the site are judged to be extremely rare, and would likely be confined to any gully areas. No significant gully areas were observed upslope or downslope of the site.

- Fast Moving Slides from Local Cut / Fill Slopes: Provided the cut slopes proposed in the development are supported by adequately designed and constructed retaining walls, and appropriate measures to reduce instability risk during construction are implemented, we consider that the likelihood of a fast moving landslide developing from the local cuts/fills is rare. Similarly, the Alpine Way fill embankment, further upslope, is understood to have been reconstructed and supported by an engineer designed retaining wall, and is therefore assessed to have a rare likelihood of developing into a fast moving landslide that could extend downslope to Lot 49.

Therefore, on the basis of the previous risk assessment to life undertaken by Coffey for the entire Thredbo Village generally, and application of that work to Lot 49 Diggings Terrace, Coffey assess that the risk to life from fast moving landslides is below the levels typically accepted by society for risk to life.

6 RECOMMENDATIONS FOR PROPOSED DEVELOPMENT

6.1 General Discussion

It is understood that the proposed development will comprise a six storey structure, with five levels of accommodation and a lower level comprising a lobby and storage areas. Due to the nature of the investigation, the subsurface conditions downhill towards the 'Mowamba' Apartments are relatively unknown and should be evaluated by a suitably experienced geotechnical practitioner at the time of construction or by drilling of investigation boreholes. However, based on the scope of the investigation carried out, the design of foundations for the structure forming the development should be carried out in accordance with the recommendations detailed in this section.

In general terms, the proposed development is shown to comprise one large excavation for the lowermost three levels. Based on the results of the geotechnical investigation, the excavation is likely to be through fill and colluvial materials into the underlying extremely to highly weathered granodiorite. The retention of the excavation through an engineer designed retaining wall is in line with good hillside construction practices as shown in Appendix D - Figure 2.

6.2 Excavation

It is considered that such an excavation as shown in the architectural drawings supplied (as shown in Figure 2) would need to be carefully carried out, to reduce the risk of slumping within the fill and colluvial materials, and will require the construction of an engineer designed retaining wall on the upslope side of the lodge. Along the eastern and western sides of the proposed lodge, the excavation for the levels below the existing ground surface may be feasible by battering to a stable temporary batter slope or utilising temporary shoring support. A temporary batter slope of 1.5H:1V would be recommended for the fill and colluvial materials. The excavation should be carried out in two sections along the length of the proposed development, to take advantage of three dimensional stability effects. Where there is insufficient space to batter the excavation due to the proximity of Diggings Terrace and/or adjacent lodges, the use of an adequately designed shoring system would be required to support the boundary excavations. This shoring system may need to be installed during the demolition process to ensure that no unsupported soil/fill batters are exposed along the boundaries of the development. To this end, demolition may only extend to ground level prior to the installation of the shoring system.

Unsupported cuts through the fill and colluvium should be no higher than 1.5m unless supported by an engineer designed retaining wall. A summary of the recommended permanent and temporary batter slopes for each material are provided below in Table 2. Permanent exposed batters beneath the lodge may require shotcrete protection and this should be assessed during the excavation period.

TABLE 2: RECOMMENDED BATTER SLOPES

Material	Permanent Batter*	Temporary Batter
Fill and Topsoil	2H:1V	1.5H:1V
Colluvium	2H:1V	1.5H:1V
Extremely to Highly Weathered Granodiorite	1H:1V	1H:1V

* Protected (Beneath Lodge) or by shotcrete

6.3 Excavation Retention

Excavation retention will be required along the southern (upslope) side of the lodge to form the three below ground levels. Examples of alternative retaining systems include:

- Anchored retaining walls,
- Contiguous bored pile walls,
- Soldier pile retaining walls, or
- Gravity walls and concrete block.

An anchored retaining system may be required where structures that are sensitive to subsurface movement are located adjacent to the site. Should anchors be required to provide lateral restraint, they should be designed using an ultimate bond stress of 100kPa in extremely to highly weathered granodiorite. Anchored retaining structures should be constructed in panels of no more than 3m width.

Alternatively, a contiguous bored pile retaining wall or soldier pile retaining wall may be constructed. Contiguous bored pile retaining walls comprise secant piles bored into suitable foundation materials and are suitable for situations similar to that for an anchored retaining system. Soldier pile retaining walls comprise soldier piles with shotcrete or timber infill panels to support the vertical faces. Soldier pile retaining walls are suitable for situations where the consequence of subsurface movement is small. Contiguous bored pile retaining walls or soldier pile walls should not be constructed in panels exceeding 10m width.

Gravity walls and concrete block retaining walls may be designed as part of the proposed structure. If a gravity retaining wall or concrete block retaining wall is to be constructed as part of the proposed development, the temporary batter slopes given above should be excavated adjacent to the location of the wall to be constructed. If this is unachievable, temporary shoring should be provided. Construction of a gravity wall or concrete block retaining wall should be undertaken in panels of no more than 5m width. The maximum height of any unsupported temporary cut prior to the construction of an engineered retaining wall should not exceed 1.5m, with batter slopes in accordance with recommendations previously provided.

The following table provides recommended parameters for the design of temporary and permanent retaining walls.

Table 3: Parameters for Retaining Wall Design

Unit	Coefficient of Active Earth Pressure, (K_a)	Coefficient of Earth Pressure at Rest, (K_0)	Unit weight (t/m^3)
Fill/Colluvium	0.4	0.6	1.8
Extremely Weathered Granodiorite	0.25	0.3	2.2

The 'active' K_a earth pressure parameters provided above would apply if small rotational or translational movements of about 5mm to 20mm in the face of the wall are allowed. If no small movements are able to take place, such as adjacent to the neighbouring structures, the 'at rest' (K_0) earth pressure parameters would apply.

Retaining walls should be designed with either an adequate drainage system to reduce the risk of water pressure build up behind the wall, or assuming hydrostatic conditions over the full height of the wall. All retaining walls should be founded on in situ weathered granodiorite.

The design of the retaining walls may be undertaken using a triangular earth pressure distribution, where the horizontal active earth pressure, p, is calculated using the following:

$$p(z) = K_a \gamma z + K_a p_s$$

where: $p(z)$ = active earth pressure at distance z below top of wall (kPa)

K_a = active earth pressure coefficient = 0.40

γ = unit weight of soil = 20.0 kN/m^3

z = distance below top of wall (m)

p_s = uniform surcharge (kPa) – (typically 20 kPa for traffic loadings)

It is generally considered that a uniform surcharge of 20 kPa is adequate to model traffic loadings (i.e. for vehicles parked adjacent to the lodge).

BH1 encountered groundwater at a level of 9.77m. This groundwater level will fluctuate and may include an elevated perched water table within the fill/colluvium following significant rainfall. Therefore, the retaining system should incorporate a drainage system to reduce the risk of build up of water pressure behind the wall. The use of perforated Agi pipe, and free draining aggregate wrapped in geofabric would be considered appropriate.

Backfilling behind the retaining structure should involve the placement of a select backfill material, comprising extremely weathered granodiorite materials compacted to not less than 95% of Standard Maximum Dry Density. This should be readily achieved by placing the backfill material in approximately 100 mm thick layers, and compacting using hand operated compaction equipment (e.g. 'Wacker Packer"). The use of excavated fill materials may be appropriate for backfilling behind retaining walls, subject to assessment on site by a suitably qualified engineering practitioner.

6.4 Foundations

Dependent on the final site excavation levels, footings for the structure should be founded within the in situ extremely weathered granodiorite. Given the depth to suitable founding materials, appropriate foundation types would comprise pad or strip footings, or alternatively piles for highly loaded areas. Piles for retention systems are also likely to be founded within the in situ extremely weathered granodiorite.

Piles or strip and pad footings founded in the in situ weathered granodiorite may be designed for a recommended allowable bearing pressure of 500 kPa with a shaft adhesion value of 50 kPa. To adopt shaft adhesion values, piles should have a minimum socket of at least 2 pile diameters into the weathered in situ granodiorite. Piles for the shoring system and foundations may encounter groundwater inflows which can make spoil removal difficult and lead to softening of the pile base. For this reason it is recommended that piles be drilled and concreted on the same day and should excessive inflows be observed, specific pile cleaning methods (such as cleaning buckets, air-lifting and vacuum suction) may need to be employed.

Settlements of footings under these loads would be expected to be less than 1% of the minimum footing dimension. Higher allowable pressures may be adopted should it be proven during excavation that a less weathered granodiorite stratum underlies the extremely to highly weathered granodiorite within 1m to 2m of the proposed excavation depth.

A minimum socket of 300mm into the desired founding material should be provided for strip, pad or pile foundations. All soft and compressible materials should be removed from the base and walls of the foundation holes/excavations, prior to placement of concrete. A suitably experienced qualified geotechnical practitioner should assess the foundation conditions at the time of construction.

Should bored piles be adopted, it is envisaged that piles may be drilled through the fill and colluvial materials using an auger attachment fitted to a hydraulic excavator. Piles should be designed and constructed in accordance with the above recommendations. It is likely that temporary or permanent sleeves may be required to retain the upper fill and/or colluvial materials and reduce the risk of collapse into the pile holes after drilling. Allowance should also be made for the possibility of boulders within the fill materials affecting the drilling of the piles.

6.5 Stormwater Runoff

Roof and pavement runoff should be controlled and piped into the stormwater system. Methods for roof water collection could involve braced guttering or concrete lined (possibly gravel filled) dish drains beneath the drip zone.

6.6 Fill Materials

Should filling be required as part of the development, it is recommended that suitable granular materials be placed and compacted to an engineering standard of not less than 98% of maximum dry density, based on Standard compaction.

Fill materials should be placed in batter slopes of no greater than 2(H):1(V) for heights less than 2m. For fill heights greater than 2m, or if 2(H):1(V) batter slopes be impractical, fill should be retained by an engineered retaining structure.

6.7 Site Clearing

Existing trees on the site are mostly exotic species recommended for removal. Advice provided by an aborist is that the species are likely to be shallow rooted in the colluvium overly the bedrock. Removal of these trees is not considered to have a significant effect on the overall stability of the slope. The existing eucalypt is likely to more deeply rooted, potentially through the colluvium and into the underlying weathered rock. The removal of this tree may have an overall effect on the stability of the slope. However, we understand that this tree is not to be removed.

6.8 Good Hillside Practice

All development on the lot is to be undertaken in accordance with sound engineering principles and good hillside practice as set out in Appendix D – Figure 2.

Where possible, lodge construction should take into account the sloping conditions of the site by reducing the amount of earthworks by having split level or elevated structures where possible.

7 ASSESSMENT OF RISK OF PROPOSED DEVELOPMENT

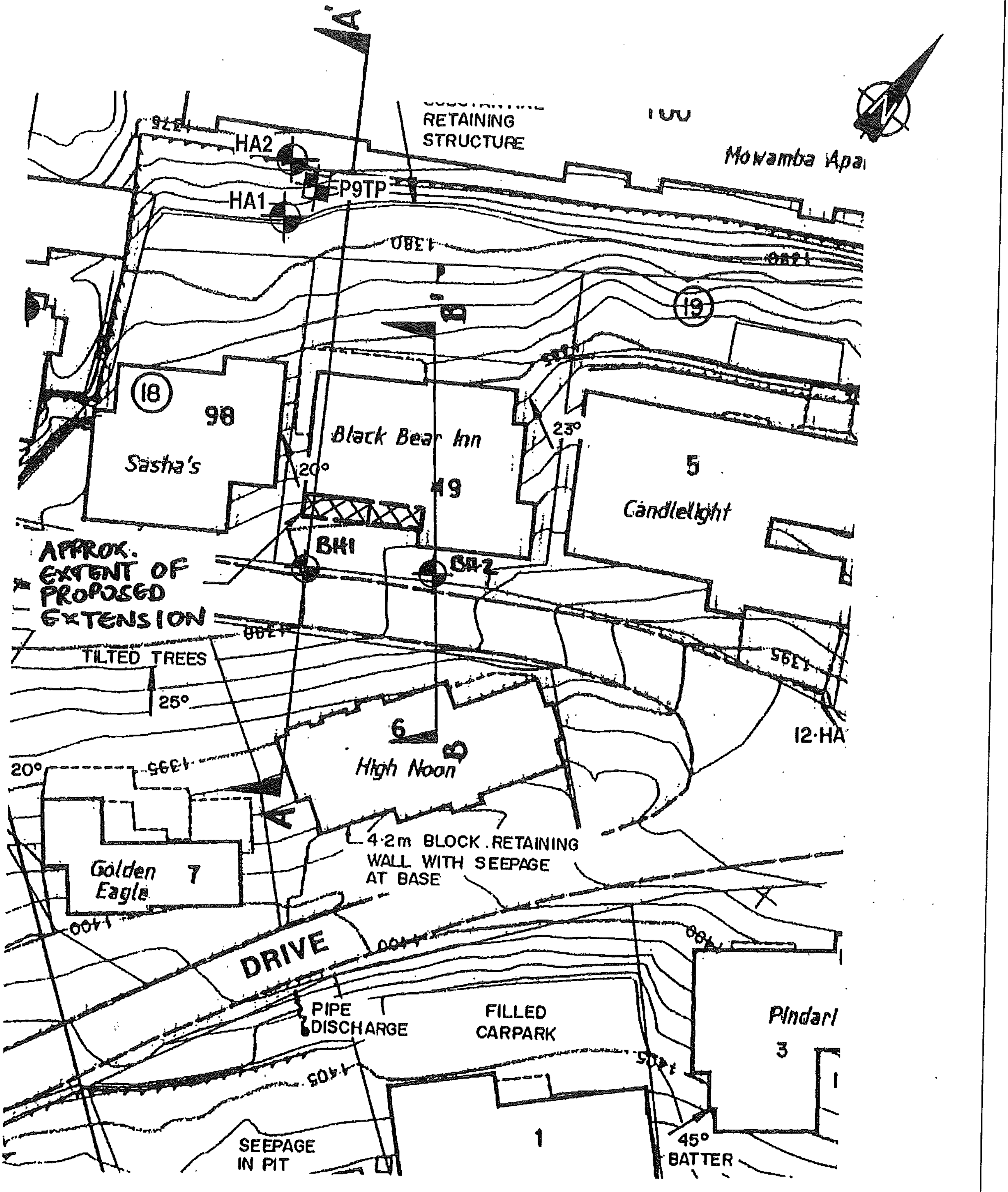
Coffey have reviewed the design advice given in our previous report with regard to the new development and have provided some additional guidance. Provided the design and construction of the proposed development is undertaken in accordance with the recommendations provided in this report, it is considered that the assessed **low to moderate** risk classification for property and the risk to life of **being better than general acceptable levels,** should not be altered by the new development. Therefore the proposed development is assessed to be suitable for the allotment. It is noted that the medium risk to property for the lot, was also applied to the lot during the overall risk assessment study for Thredbo Alpine Village undertaken by Coffey in December 1997, and revised in August 1998.

For and on behalf of Coffey Geotechnics Pty Ltd

Paran Moyes

Senior Geotechnical Engineer

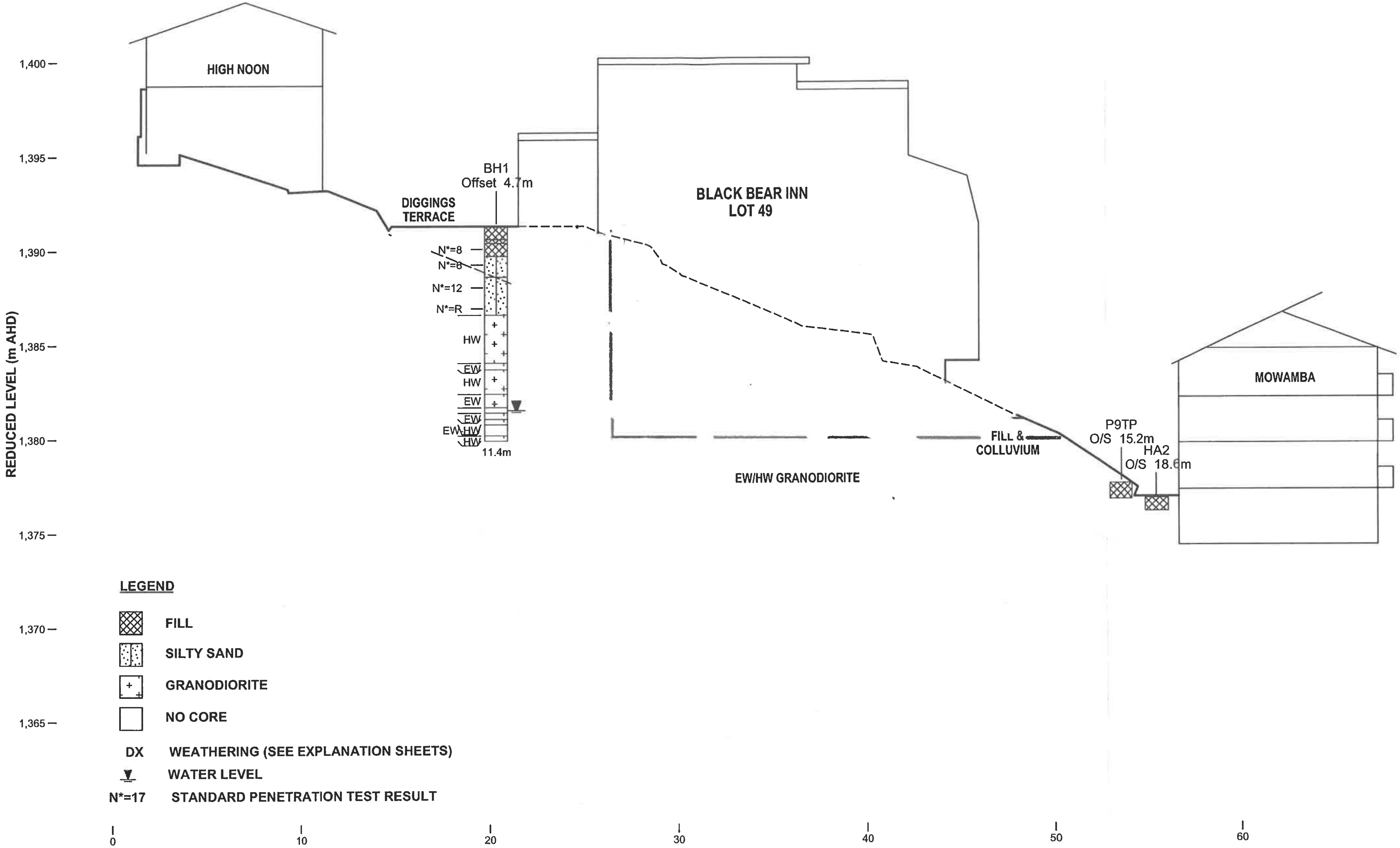
Figures



LEGEND

- GEOTECHNICAL BOREHOLE
- TEST PIT

drawn	RED/SW	coffey geotechnics SPECIALISTS MANAGING THE EARTH	client:	ALEX POPOV & ASSOCIATES
approved	PM		project:	BLACK BEAR INN LOT 49 – DIGGINGS TERRACE THREDBO ALPINE VILLAGE
date	4/5/07		title:	SITE PLAN
scale	1:500		project no: GEOTLCOV23158AA	figure no: FIGURE 1
original size	A4			



revision	description	drawn	approved	date

Horizontal Scale (metres): 2 0 10

Vertical Scale (metres): 2 0 10

drawn	PM/SW
approved	PM
date	14/5/07
scale	AS SHOWN
original size	A3

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client:	ALEX POPOV & ASSOCIATES
project:	BLACK BEAR INN LOT 49 - DIGGINGS TERRACE THREDBO ALPINE VILLAGE
title:	GEOTECHNICAL SECTION A-A'
project no:	GEOTLCOV23158AA
figure no:	

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SPECIALISTS MANAGING THE EARTH

Important information about your **Coffey** Report

As a client of Coffey you should know that site subsurface conditions cause more construction problems than any other factor. These notes have been prepared by Coffey to help you interpret and understand the limitations of your report.

Your report is based on project specific criteria

Your report has been developed on the basis of your unique project specific requirements as understood by Coffey and applies only to the site investigated. Project criteria typically include the general nature of the project; its size and configuration; the location of any structures on the site; other site improvements; the presence of underground utilities; and the additional risk imposed by scope-of-service limitations imposed by the client. Your report should not be used if there are any changes to the project without first asking Coffey to assess how factors that changed subsequent to the date of the report affect the report's recommendations. Coffey cannot accept responsibility for problems that may occur due to changed factors if they are not consulted.

Subsurface conditions can change

Subsurface conditions are created by natural processes and the activity of man. For example, water levels can vary with time, fill may be placed on a site and pollutants may migrate with time. Because a report is based on conditions which existed at the time of subsurface exploration, decisions should not be based on a report whose adequacy may have been affected by time. Consult Coffey to be advised how time may have impacted on the project.

Interpretation of factual data

Site assessment identifies actual subsurface conditions only at those points where samples are taken and when they are taken. Data derived from literature and external data source review, sampling and subsequent laboratory testing are interpreted by geologists, engineers or scientists to provide an opinion about overall site conditions, their likely impact on the proposed development and recommended actions. Actual conditions may differ from those inferred to exist, because no professional, no matter how qualified, can reveal what is hidden by earth, rock and time. The actual interface between materials may be far more gradual or abrupt than assumed based on the facts obtained. Nothing can be done to change the actual site conditions which exist, but steps can be taken to reduce the impact of unexpected conditions. For this reason, owners should retain the services of Coffey through the development stage, to identify variances, conduct additional tests if required, and recommend solutions to problems encountered on site.

Your report will only give preliminary recommendations

Your report is based on the assumption that the site conditions as revealed through selective point sampling are indicative of actual conditions throughout an area. This assumption cannot be substantiated until project implementation has commenced and therefore your report recommendations can only be regarded as preliminary. Only Coffey, who prepared the report, is fully familiar with the background information needed to assess whether or not the report's recommendations are valid and whether or not changes should be considered as the project develops. If another party undertakes the implementation of the recommendations of this report there is a risk that the report will be misinterpreted and Coffey cannot be held responsible for such misinterpretation.

Your report is prepared for specific purposes and persons

To avoid misuse of the information contained in your report it is recommended that you confer with Coffey before passing your report on to another party who may not be familiar with the background and the purpose of the report. Your report should not be applied to any project other than that originally specified at the time the report was issued.

coffey geotechnics
SPECIALISTS MANAGING THE EARTH

Important information about your **Coffey** Report

Interpretation by other design professionals

Costly problems can occur when other design professionals develop their plans based on misinterpretations of a report. To help avoid misinterpretations, retain Coffey to work with other project design professionals who are affected by the report. Have Coffey explain the report implications to design professionals affected by them and then review plans and specifications produced to see how they incorporate the report findings.

Data should not be separated from the report*

The report as a whole presents the findings of the site assessment and the report should not be copied in part or altered in any way.

Logs, figures, drawings, etc. are customarily included in our reports and are developed by scientists, engineers or geologists based on their interpretation of field logs (assembled by field personnel) and laboratory evaluation of field samples. These logs etc. should not under any circumstances be redrawn for inclusion in other documents or separated from the report in any way.

Geoenvironmental concerns are not at issue

Your report is not likely to relate any findings, conclusions, or recommendations about the potential for hazardous materials existing at the site unless specifically required to do so by the client. Specialist equipment, techniques, and personnel are used to perform a geoenvironmental assessment.
Contamination can create major health, safety and environmental risks. If you have no information about the potential for your site to be contaminated or create an environmental hazard, you are advised to contact Coffey for information relating to geoenvironmental issues.

Rely on Coffey for additional assistance

Coffey is familiar with a variety of techniques and approaches that can be used to help reduce risks for all parties to a project, from design to construction. It is common that not all approaches will be necessarily dealt with in your site assessment report due to concepts proposed at that time. As the project progresses through design towards construction, speak with Coffey to develop alternative approaches to problems that may be of genuine benefit both in time and cost.

Responsibility

Reporting relies on interpretation of factual information based on judgement and opinion and has a level of uncertainty attached to it, which is far less exact than the design disciplines. This has often resulted in claims being lodged against consultants, which are unfounded. To help prevent this problem, a number of clauses have been developed for use in contracts, reports and other documents. Responsibility clauses do not transfer appropriate liabilities from Coffey to other parties but are included to identify where Coffey's responsibilities begin and end. Their use is intended to help all parties involved to recognise their individual responsibilities. Read all documents from Coffey closely and do not hesitate to ask any questions you may have.

* For further information on this aspect reference should be made to "Guidelines for the Provision of Geotechnical Information in Construction Contracts" published by the Institution of Engineers Australia, National Headquarters, Canberra, 1987.

Appendix A

Engineering Borehole Logs

Coffey Geosciences Pty Ltd ACN 056 335 516

Engineering Log - Borehole

Borehole No. *BH1*
Sheet 1 of 3
Office Job No.: *S20449/2*
Date started: *23.6.2003*
Date completed: *23.6.2003*
Logged by: *RED*
Checked by: [signature]

Client: *ELWYN WYETH MANAGEMENT ARCHITECTURE*
Principal:
Project: *PROPOSED REDEVELOPMENT OF THE BLACK BEAR INN*
Borehole Location: *SEE FIGURE 1*

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drill model and mounting: GEMCO 210B TRAILER | Easting: 237749.2 | slope: -90° | R.L. Surface: 1390.1
hole diameter: 95 mm | Northing 958298.25 | bearing: 000° | datum: AHD

drilling information / material substance

method	penetration 1 2 3	support	water	notes samples, tests, etc	RL	depth metres	graphic log	classification symbol	material: soil type: plasticity or particle characteristics, colour, secondary and minor components.	moisture condition	consistency/density index	pocket penetrometer kPa (100 200 300 400)	structure and additional observations
ADT		N			1390	0		SM	FILL: SILTY SAND: Fine to coarse grained, brown; approximately 25% non-plastic fines; some pockets with granite fragments to 40mm in size.	M			FILL 30mm asphalt at surface.
								SM	FILL: GRAVELLY SILTY SAND: Fine to medium grained, dark brown; non-plastic fines; medium to coarse rounded gravel.				
				SPT 2,3,5 N*=8	1389	1		ML	FILL: SANDY SILT: Low plasticity, dark brown; fine to coarse grained sand; with some granite fragments to 20mm in size.				
				SPT 2,2,4 N*=6	1388	2		SM	SILTY SAND: Fine to coarse grained, brown; approximately 25% non-plastic fines; trace of fine gravel.	M	L		PROBABLE COLLUVIUM
				SPT 2,5,7 N*=12	1387	3		SM	SILTY SAND: Fine to coarse grained, pale brown and brown; approximately 20% non-plastic fines; trace of fine gravel.	D-M	MD		EXTREMELY WEATHERED GRANODIORITE
			None observed								VD		
				SPT 8,15,R N*=R	1386	4							SPT - 15 blows/70mm then refusal.
					1385	5			Borehole BH1 continued as cored hole				
					1384	6							
					1383	7							
						8							

method
AS auger screwing*
AD auger drilling*
RR roller/tricone
W washbore
CT cable tool
HA hand auger
DT diatube
B blank bit
V V bit
T TC bit
*bit shown by suffix
e.g. ADT

support
M mud N nil
C casing

penetration
1 2 3 4
no resistance ranging to refusal

water
10/1/98 water level on date shown
water inflow
water outflow

notes, samples, tests
U_{50} undisturbed sample 50mm diameter
U_{63} undisturbed sample 63mm diameter
D disturbed sample
N standard penetration test (SPT)
N* SPT - sample recovered
Nc SPT with solid cone
V vane shear (kPa)
P pressuremeter
Bs bulk sample
E environmental sample
R refusal

classification symbols and soil description
based on unified classification system

moisture
D dry
M moist
W wet
Wp plastic limit
W_L liquid limit

consistency/density index
VS very soft
S soft
F firm
St stiff
VSt very stiff
H hard
Fb friable
VL very loose
L loose
MD medium dense
D dense
VD very dense

Form GEO 5.3 Issue 3 Rev.2

BOREHOLE S20449.2.GPJ COFFEY.GDT 17.07.03

Coffey Geosciences Pty Ltd ACN 056 335 516

Engineering Log - Cored Borehole

Client:	ELWYN WYETH MANAGEMENT ARCHITECTURE
Principal:	
Project:	PROPOSED REDEVELOPMENT OF THE BLACK BEAR INN
Borehole Location:	SEE FIGURE 1

Borehole No.	BH1
Sheet	2 of 3
Office Job No.:	S20449/2
Date started:	23.6.2003
Date completed:	23.6.2003
Logged by:	RED
Checked by:	(signature)

Coffey

drill model & mounting: GEMCO 210B TRAILER		Easting:	237749.2	slope:	-90°	R.L. Surface:	1390.1
hole diameter: 95 mm	Drilling fluid:	Northing:	958298.25	bearing:	000°	datum:	AHD

drilling information (method, core-lift, water, RL, depth metres) | **material substance** (graphic log / core recovery; material: rock type; grain characteristics, colour, structure, minor components; weathering alteration; estimated strength VL L M H VH EH; $Is_{(50)}$ MPa D- diametral A- axial) | **rock mass defects** (RQD %; defect spacing mm 30 100 300 1000 3000; defect description: type, inclination, planarity, roughness, coating, thickness — particular / general)

method	water	RL	depth (m)	material	weathering	RQD %	defect description
	None observed	1390	0				
		1389	1				
		1388	2				
		1387	3				
		1386	4	Continued from non-cored borehole			
NMLC		1385	5	GRANODIORITE: Coarse grained, pale brown (pink) and white/black speckled, massive atrusive.	HW	0	~~JT, 60°, PL/IR, VR, SN, biotite.~~ PT, 5°, IR, VR, SN, biotite. PT, 15°, IR, VR, SN, biotite. CS, 70°, IR, VR, SN, biotite (70mm crushed zone). PTs (x 2), 0°, PL, VR, SN, biotite. PT, 5°-10°, IR, VR, SN, biotite.
		1384	6			12	PT, 5°, PL, VR, SN, biotite. JT (x 4), 60°, PL, VR, SN. PTs (x 2), 0°, PL, RO-VR, SN, biotite crystals. JT, 60°, PL, RO, SN, biotite. JTs (x 3), PL/CU, RO, SN.
		1383	7			46	PT, 0°-5°, PL, RO, SN PT, 0°-45°, ST, RO, SN. SZ, 5°-45°, PL, RO, SN (90mm sheared zone). PT, 0°, PL, RO, SN. PT, 5°, PL, RO, SN.
				GRANODIORITE: Medium some coarse grained, brown and white speckled, massive.	EW		PT, 0°, PL, RO, SN. JT, 45°, PL, RO, SN.
			8		HW	47	PT, 5°, PL, RO, SN. CS, 0°-30°, PL, VR, SN. PT (x 3), 0°-5°, PL, RO, SN. JT, 60°, PL, RO, SN.

method
DT diatube
AS auger screwing
AD auger drilling
RR roller/tricone
CB claw or blade bit
NMLC NMLC core
NQ, HQ, PQ wireline core

core-lift
casing used
barrel withdrawn

graphic log/core recovery
core recovered - graphic symbols indicate material
no core recovered

water
10/1/98 water level on date shown
water inflow
partial drill fluid loss
complete drill fluid loss
25 water pressure test result (lugeons) for depth interval shown

weathering
FR fresh
SW slightly weathered
MW moderately weathered
HW highly weathered
XW extremely weathered
DW distinctly weathered (covers MW and HW)

strength
VL very low
L low
M medium
H high
VH very high
EH extremely high

defect type
JT joint
PT parting
SM seam
SZ sheared zone
SS sheared surface
CS crushed seam

planarity
PL planar
CU curved
UN undulating
ST stepped
IR irregular

roughness
VR very rough
RO rough
SO smooth
SL slickensided

coating
CN clean
SN stained
VN veneer
CO coating

Form GEO 5.5 Issue 3 Rev. 3

CORED BOREHOLE S20449.2.GPJ COFFEY.GDT 17.07.03

Coffey Geosciences Pty Ltd ACN 056 335 516

Engineering Log - Cored Borehole

Borehole No.	*BH1*
Sheet	3 of 3
Office Job No.:	*S20449/2*
Date started:	*23.6.2003*
Date completed:	*23.6.2003*
Logged by:	*RED*
Checked by:	[signature]

Client: *ELWYN WYETH MANAGEMENT ARCHITECTURE*

Principal:

Project: *PROPOSED REDEVELOPMENT OF THE BLACK BEAR INN*

Borehole Location: *SEE FIGURE 1*

drill model & mounting: GEMCO 210B TRAILER | Easting: 237749.2 | slope: -90° | R.L. Surface: 1390.1

hole diameter: 95 mm | Drilling fluid: | Northing: 958298.25 | bearing: 000° | datum: AHD

drilling information | **material substance** | **rock mass defects**

method	core-lift	water	RL	depth metres	material: rock type; grain characteristics, colour, structure, minor components	weathering alteration	RQD %	defect description (type, inclination, planarity, roughness, coating, thickness)
NMLC			1382		**GRANODIORITE:** Coarse grained, pale brown (pink) and white and black speckled, massive instrusive. *(continued)*	HW	47	JTs (x 3), 60°, PL/CU, RO, SN. JTs (x 2), 60° and 5°, ST, RO, SN. JT, 60°, PL/CU, RO, SN. JT, 45°, PL, RO, SN. PT, 5°, PL, RO, SN.
			1381	9	**GRANODIORITE:** Coarse grained, orange/brown and white/black, massive, friable.	EW		JT, 50°, PL, RO, SN. JT, 70°, PL, RO, SN. PT, 0°, PL, RO, SN. JT, 90°, PL, RO, SN. PT, 0°, PL, RO, SN. JT, 45°, PL, RO, VN sand. CS, 0°, PL, RO, SN.
		(4/7/03)			**NO CORE:** (9.61-9.91m).			
			1380	10	**GRANODIORITE:** Coarse grained, orange/brown and white/black, massive, friable.	EW	0	
					GRANODIORITE: Coarse grained, pale brown (pink) with black and white speckled, massive, intrusive.	EW-HW		PT, 0°-5°, PL. JT, 60°, PL/CU, RO, SN. JT, 75°, PL, RO, SN.
			1379	11	**NO CORE:** (10.54-11.10m).		0	
					GRANODIORITE: Coarse grained, pale brown (pink), white and black speckled, massive, intrusive.	HW	0	JT, 80°, PL, RO, SN. JTs (x 2), 35°, PL, RO, SN.
			1378	12	BH1 terminated at 11.4m Piezometer installed to 11.4m. Slotted from 5.4 to 11.4m, filter sock from 5.4m to 11.4m, sand from 11.4m to 1m. Grouted from 1m to 0.5m, backfilled to surface. Metal gatic cover installed flush with surface. BH1 terminated at 11.4m			
			1377	13				
			1376	14				
			1375	15				
				16				

Additional column headings: graphic log / core recovery; estimated strength (VL L M H VH EH); $Is_{(50)}$ MPa (D- diametral, A- axial); defect spacing mm (30, 100, 300, 1000, 3000); defect description: particular / general.

method
DT diatube
AS auger screwing
AD auger drilling
RR roller/tricone
CB claw or blade bit
NMLC NMLC core
NQ, HQ, PQ wireline core

core-lift
casing used
barrel withdrawn

graphic log/core recovery
core recovered - graphic symbols indicate material
no core recovered

water
10/1/98 water level on date shown
water inflow
partial drill fluid loss
complete drill fluid loss
25 water pressure test result (lugeons) for depth interval shown

weathering
FR fresh
SW slightly weathered
MW moderately weathered
HW highly weathered
XW extremely weathered
DW distinctly weathered (covers MW and HW)

strength
VL very low
L low
M medium
H high
VH very high
EH extremely high

defect type
JT joint
PT parting
SM seam
SZ sheared zone
SS sheared surface
CS crushed seam

planarity
PL planar
CU curved
UN undulating
ST stepped
IR irregular

roughness
VR very rough
RO rough
SO smooth
SL slickensided

coating
CN clean
SN stained
VN veneer
CO coating

Coffey Geosciences Pty Ltd ACN 056 335 516

Engineering Log - Borehole

Borehole No.	2
Sheet	1 of 1
Office Job No.:	S20449/1
Date started:	24.5.2000
Date completed:	24.5.2000
Logged by:	IS
Checked by:	[signature]

Client: *HIDALI PTY LTD*

Principal:

Project: *ADDITIONS TO BLACK BEAR LODGE - THREDBO, NSW*

Borehole Location: *SEE FIGURE 1*

drill model and mounting:	Gemco 210B Trailer	Easting:	slope: -90°	R.L. Surface:	1391.4
hole diameter:	95mm mm	Northing	bearing:	datum:	

drilling information / **material substance**

method	penetration 1 2 3	support	water	notes samples, tests, etc	RL	depth metres	graphic log	classification symbol	material: soil type: plasticity or particle characteristics, colour, secondary and minor components.	moisture condition	consistency/density index	pocket penetro-meter kPa (100 200 300 400)	structure and additional observations
ADT		Nil	None Observed		1391				CONCRETE :30mm asphalt at surface.				
								SM	SILTY SAND (FILL): Fine to coarse grained, brown; non-plastic fines; with some fine to coarse subrounded gravel; some zones of dark brown clayey sand.	M			FILL
				SPT 2,4,R N*=R	1390	1		ML	Sandy SILT (FILL): Low plasticity, dark brown; fine to medium grained sand; trace of fine gravel.	M	L		POSSIBLE TOPSOIL SPT - 6 blows/120mm then refusal on granite boulder
					1389	2		SM	SILTY SAND: Fine to coarse grained, pale brown and brown; non-plastic fines; trace of fine gravel. 1.45 - 1.85m: Possible boulder 2.20 - 2.40m: Possible boulder 2.50 - 2.80m: Possible boulder	D-M	D		EW GRANODIORITE WITH DW CORESTONES
				SPT 14,R N*=R	1388	3							SPT - 10 blows/70mm then refusal on granite boulder
					1387	4			Borehole 2 terminated at 3.5m				
					1386	5							
					1385	6							
					1384	7							
						8							

BOREHOLE S20449X1.GPJ COFFEY.GDT 17.07.03

method
- AS auger screwing*
- AD auger drilling*
- RR roller/tricone
- W washbore
- CT cable tool
- HA hand auger
- DT diatube
- B blank bit
- V V bit
- T TC bit
- *bit shown by suffix
- e.g. ADT

support
- M mud
- N nil
- C casing

penetration 1 2 3 4 — no resistance ranging to refusal

water
- 10/1/98 water level on date shown
- water inflow
- water outflow

notes, samples, tests
- U_{50} undisturbed sample 50mm diameter
- U_{63} undisturbed sample 63mm diameter
- D disturbed sample
- N standard penetration test (SPT)
- N* SPT - sample recovered
- Nc SPT with solid cone
- V vane shear (kPa)
- P pressuremeter
- Bs bulk sample
- E environmental sample
- R refusal

classification symbols and soil description
based on unified classification system

moisture
- D dry
- M moist
- W wet
- Wp plastic limit
- W_L liquid limit

consistency/density index
- VS very soft
- S soft
- F firm
- St stiff
- VSt very stiff
- H hard
- Fb friable
- VL very loose
- L loose
- MD medium dense
- D dense
- VD very dense

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Soil Description Explanation Sheet (1 of 2)

DEFINITION:

In engineering terms soil includes every type of uncemented or partially cemented inorganic or organic material found in the ground. In practice, if the material can be remoulded or disintegrated by hand in its field condition or in water it is described as a soil. Other materials are described using rock description terms.

CLASSIFICATION SYMBOL & SOIL NAME

Soils are described in accordance with the Unified Soil Classification (UCS) as shown in the table on Sheet 2.

PARTICLE SIZE DESCRIPTIVE TERMS

NAME	SUBDIVISION	SIZE
Boulders		>200 mm
Cobbles		63 mm to 200 mm
Gravel	coarse	20 mm to 63 mm
	medium	6 mm to 20 mm
	fine	2.36 mm to 6 mm
Sand	coarse	600 µm to 2.36 mm
	medium	200 µm to 600 µm
	fine	75 µm to 200 µm

MOISTURE CONDITION

Dry Looks and feels dry. Cohesive and cemented soils are hard, friable or powdery. Uncemented granular soils run freely through hands.

Moist Soil feels cool and darkened in colour. Cohesive soils can be moulded. Granular soils tend to cohere.

Wet As for moist but with free water forming on hands when handled.

CONSISTENCY OF COHESIVE SOILS

TERM	UNDRAINED STRENGTH s_u (kPa)	FIELD GUIDE
Very Soft	<12	A finger can be pushed well into the soil with little effort.
Soft	12 - 25	A finger can be pushed into the soil to about 25mm depth.
Firm	25 - 50	The soil can be indented about 5mm with the thumb, but not penetrated.
Stiff	50 - 100	The surface of the soil can be indented with the thumb, but not penetrated.
Very Stiff	100 - 200	The surface of the soil can be marked, but not indented with thumb pressure.
Hard	>200	The surface of the soil can be marked only with the thumbnail.
Friable	-	Crumbles or powders when scraped by thumbnail.

DENSITY OF GRANULAR SOILS

TERM	DENSITY INDEX (%)
Very loose	Less than 15
Loose	15 - 35
Medium Dense	35 - 65
Dense	65 - 85
Very Dense	Greater than 85

MINOR COMPONENTS

TERM	ASSESSMENT GUIDE	PROPORTION OF MINOR COMPONENT IN:
Trace of	Presence just detectable by feel or eye, but soil properties little or no different to general properties of primary component.	Coarse grained soils: <5% Fine grained soils: <15%
With some	Presence easily detected by feel or eye, soil properties little different to general properties of primary component.	Coarse grained soils: 5 - 12% Fine grained soils: 15 - 30%

SOIL STRUCTURE

ZONING		CEMENTING	
Layers	Continuous across exposure or sample.	Weakly cemented	Easily broken up by hand in air or water.
Lenses	Discontinuous layers of lenticular shape.	Moderately cemented	Effort is required to break up the soil by hand in air or water.
Pockets	Irregular inclusions of different material.		

GEOLOGICAL ORIGIN

WEATHERED IN PLACE SOILS

Extremely weathered material — Structure and fabric of parent rock visible.

Residual soil — Structure and fabric of parent rock not visible.

TRANSPORTED SOILS

Aeolian soil — Deposited by wind.

Alluvial soil — Deposited by streams and rivers.

Colluvial soil — Deposited on slopes (transported downslope by gravity).

Fill — Man made deposit. Fill may be significantly more variable between tested locations than naturally occurring soils.

Lacustrine soil — Deposited by lakes.

Marine soil — Deposited in ocean basins, bays, beaches and estuaries.

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Soil Description Explanation Sheet (2 of 2)

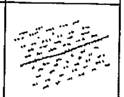
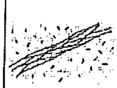
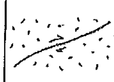
SOIL CLASSIFICATION INCLUDING IDENTIFICATION AND DESCRIPTION

FIELD IDENTIFICATION PROCEDURES (Excluding particles larger than 60 mm and basing fractions on estimated mass)				USC	PRIMARY NAME
COARSE GRAINED SOILS More than 50% of materials less than 63 mm is larger than 0.075 mm (A 0.075 mm particle is about the smallest particle visible to the naked eye)	GRAVELS More than half of coarse fraction is larger than 2.0 mm	CLEAN GRAVELS (Little or no fines)	Wide range in grain size and substantial amounts of all intermediate particle sizes.	GW	GRAVEL
			Predominantly one size or a range of sizes with more intermediate sizes missing.	GP	GRAVEL
		GRAVELS WITH FINES (Appreciable amount of fines)	Non-plastic fines (for identification procedures see ML below)	GM	SILTY GRAVEL
			Plastic fines (for identification procedures see CL below)	GC	CLAYEY GRAVEL
	SANDS More than half of coarse fraction is smaller than 2.0 mm	CLEAN SANDS (Little or no fines)	Wide range in grain sizes and substantial amounts of all intermediate sizes missing	SW	SAND
			Predominantly one size or a range of sizes with some intermediate sizes missing.	SP	SAND
		SANDS WITH FINES (Appreciable amount of fines)	Non-plastic fines (for identification procedures see ML below).	SM	SILTY SAND
			Plastic fines (for identification procedures see CL below).	SC	CLAYEY SAND

		IDENTIFICATION PROCEDURES ON FRACTIONS <0.2 mm.				
		DRY STRENGTH	DILATANCY	TOUGHNESS	USC	PRIMARY NAME
FINE GRAINED SOILS More than 50% of material less than 63 mm is smaller than 0.075 mm	SILTS & CLAYS Liquid limit less than 50	None to Low	Quick to slow	None	ML	SILT
		Medium to High	None	Medium	CL	CLAY
		Low to medium	Slow to very slow	Low	OL	ORGANIC SILT
	SILTS & CLAYS Liquid limit greater than 50	Low to medium	Slow to very slow	Low to medium	MH	SILT
		High	None	High	CH	CLAY
		Medium to High	None	Low to medium	OH	ORGANIC CLAY
HIGHLY ORGANIC SOILS		Readily identified by colour, odour, spongy feel and frequently by fibrous texture.			Pt	PEAT

• Low plasticity – Liquid Limit W_L less than 35%. • Medium plasticity – W_L between 35% and 50%.

COMMON DEFECTS IN SOIL

TERM	DEFINITION	DIAGRAM
PARTING	A surface or crack across which the soil has little or no tensile strength. Parallel or sub parallel to layering (eg bedding). May be open or closed.	
JOINT	A surface or crack across which the soil has little or no tensile strength but which is not parallel or sub parallel to layering. May be open or closed. The term 'fissure' may be used for irregular joints <0.2 m in length.	
SHEARED ZONE	Zone in clayey soil with roughly parallel near planar, curved or undulating boundaries containing closely spaced, smooth or slickensided, curved intersecting joints which divide the mass into lenticular or wedge shaped blocks.	
SHEARED SURFACE	A near planar curved or undulating, smooth, polished or slickensided surface in clayey soil. The polished or slickensided surface indicates that movement (in many cases very little) has occurred along the defect.	

TERM	DEFINITION	DIAGRAM
SOFTENED ZONE	A zone in clayey soil, usually adjacent to a defect in which the soil has a higher moisture content than elsewhere.	
TUBE	Tubular cavity. May occur singly or as one of a large number of separate or inter-connected tubes. Walls often coated with clay or strengthened by denser packing of grains. May contain organic matter	
TUBE CAST	Roughly cylindrical elongated body of soil different from the soil mass in which it occurs. In some cases the soil which makes up the tube cast is cemented.	
INFILLED SEAM	Sheet or wall like body of soil substance or mass with roughly planar to irregular near parallel boundaries which cuts through a soil mass. Formed by infilling of open joints.	

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Rock Description Explanation Sheet (1 of 2)

The descriptive terms used by Coffey are given below. They are broadly consistent with Australian Standard AS1726-1993.

DEFINITIONS: Rock substance, defect and mass are defined as follows:

Rock Substance In engineering terms roch substance is any naturally occurring aggregate of minerals and organic material which cannot be disintegrated or remoulded by hand in air or water. Other material is described using soil descriptive terms. Effectively homogenous material, may be isotropic or anisotropic.

Defect Discontinuity or break in the continuity of a substance or substances.

Mass Any body of material which is not effectively homogeneous. It can consist of two or more substances without defects, or one or more substances with one or more defects.

SUBSTANCE DESCRIPTIVE TERMS:

ROCK NAME Simple rock names are used rather than precise geological classification.

PARTICLE SIZE Grain size terms for sandstone are:

Coarse grained	Mainly 0.6mm to 2mm
Medium grained	Mainly 0.2mm to 0.6mm
Fine grained	Mainly 0.06mm (just visible) to 0.2mm

FABRIC Terms for layering of penetrative fabric (eg. bedding, cleavage etc.) are:

Massive	No layering or penetrative fabric.
Indistinct	Layering or fabric just visible. Little effect on properties.
Distinct	Layering or fabric is easily visible. Rock breaks more easily parallel to layering of fabric.

CLASSIFICATION OF WEATHERING PRODUCTS

Term	Abbreviation	Definition
Residual Soil	**RS**	Soil derived from the weathering of rock; the mass structure and substance fabric are no longer evident; there is a large change in volume but the soil has not been significantly transported.
Extremely Weathered Material	**XW**	Material is weathered to such an extent that it has soil properties, ie, it either disintegrates or can be remoulded in water. Original rock fabric still visible.
Highly Weathered Rock	**HW**	Rock strength is changed by weathering. The whole of the rock substance is discoloured, usually by iron staining or bleaching to the extent that the colour of the original rock is not recognisable. Some minerals are decomposed to clay minerals. Porosity may be increased by leaching or may be decreased due to the deposition of minerals in pores.
Moderately Weathered Rock	**MW**	The whole of the rock substance is discoloured, usually by iron staining or bleaching , to the extent that the colour of the fresh rock is no longer recognisable.
Slightly Weathered Rock	**SW**	Rock substance affected by weathering to the extent that partial staining or partial discolouration of the rock substance (usually by limonite) has taken place. The colour and texture of the fresh rock is recognisable; strength properties are essentially those of the fresh rock substance.
Fresh Rock	**FR**	Rock substance unaffected by weathering.

Notes on Weathering:

1. AS1726 suggests the term "Distinctly Weathered" (DW) to cover the range of substance weathering conditions between XW and SW. For projects where it is not practical to delineate between HW and MW or it is judged that there is no advantage in making such a distinction. DW may be used with the definition given in AS1726.
2. Where physical and chemical changes were caused by hot gasses and liquids associated with igneous rocks, the term "altered" may be substituted for "weathering" to give the abbreviations XA, HA, MA, SA and DA.

ROCK SUBSTANCE STRENGTH TERMS

Term	Abbreviation	Point Load Index, $I_{s}50$ (MPa)	Field Guide
Very Low	**VL**	Less than 0.1	Material crumbles under firm blows with sharp end of pick; can be peeled with a knife; pieces up to 30mm thick can be broken by finger pressure.
Low	**L**	0.1 to 0.3	Easily scored with a knife; indentations 1mm to 3mm show with firm bows of a pick point; has a dull sound under hammer. Pieces of core 150mm long by 50mm diameter may be broken by hand. Sharp edges of core may be friable and break during handling.
Medium	**M**	0.3 to 1.0	Readily scored with a knife; a piece of core 150mm long by 50mm diameter can be broken by hand with difficulty.
High	**H**	1 to 3	A piece of core 150mm long by 50mm can not be broken by hand but can be broken by a pick with a single firm blow; rock rings under hammer.
Very High	**VH**	3 to 10	Hand specimen breaks after more than one blow of a pick; rock rings under hammer.
Extremely High	**EH**	More than 10	Specimen requires many blows with geological pick to break; rock rings under hammer.

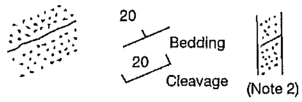
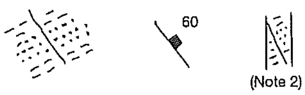
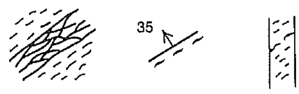
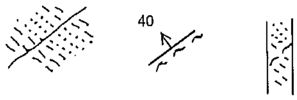
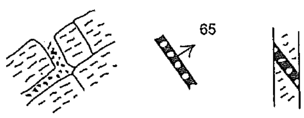
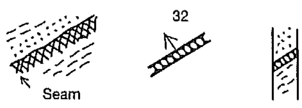
Notes on Rock Substance Strength:

1. In anisotropic rocks the field guide to strength applies to the strength perpendicular to the anisotropy. High strength anisotropic rocks may break readily parallel to the planar anisotropy.
2. The term "extremely low" is not used as a rock substance strength term. While the term is used in AS1726-1993, the field guide therein makes it clear that materials in that strength range are soils in engineering terms.
3. The unconfined compressive strength for isotropic rocks (and anisotropic rocks which fall across the planar anisotropy) is typically 10 to 25 times the point load index (Is50). The ratio may vary for different rock types. Lower strength rocks often have lower ratios than higher strength rocks.

coffey

Rock Description Explanation Sheet (2 of 2)

COMMON DEFECTS IN ROCK MASSES

Term	Definition	Diagram / Map Symbol / Graphic Log (Note 1)
Parting	A surface or crack across which the rock has little or no tensile strength. Parallel or sub parallel to layering (eg bedding) or a planar anisotropy in the rock substance (eg, cleavage). May be open or closed.	
Joint	A surface or crack across which the rock has little or no tensile strength. but which is not parallel or sub parallel to layering or planar anisotropy in the rock substance. May be open or closed.	
Sheared Zone (Note 3)	Zone of rock substance with roughly parallel near planar, curved or undulating boundaries cut by closely spaced joints, sheared surfaces or other defects. Some of the defects are usually curved and intersect to divide the mass into lenticular or wedge shaped blocks.	
Sheared Surface (Note 3)	A near planar, curved or undulating surface which is usually smooth, polished or slickensided.	
Crushed Seam (Note 3)	Seam with roughly parallel almost planar boundaries, composed of disoriented, usually angular fragments of the host rock substance which may be more weathered than the host rock. The seam has soil properties.	
Infilled Seam	Seam of soil substance usually with distinct roughly parallel boundaries formed by the migration of soil into an open cavity or joint, infilled seams less than 1mm thick may be described as veneer or coating on joint surface.	
Extremely Weathered Seam	Seam of soil substance, often with gradational boundaries. Formad by weathering of the rock substance in place.	

DEFECT SHAPE TERMS

Planar	The defect does not vary in orientation
Curved	The defect has a gradual change in orientation
Undulating	The defect has a wavy surface
Stepped	The defect has one or more well defined steps
Irregular	The defect has many sharp changes of orientation

Note: The assessment of defect shape is partly influenced by the scale of the observation.

ROUGHNESS TERMS

Slickensided	Grooved or striated surface, usually polished
Polished	Shiny smooth surface
Smooth	Smooth to touch. Few or no surface irregularities
Rough	Many small surface irregularities (amplitude generally less than 1mm). Feels like fine to coarse sand paper.
Very Rough	Many large surface irregularities (amplitude generally more than 1mm). Feels like, or coarser than very coarse sand paper.

COATING TERMS

Clean	No visible coating
Stained	No visible coating but surfaces are discoloured
Veneer	A visible coating of soil or mineral, too thin to measure; may be patchy
Coating	A visible coating up to 1mm thick. Thicker soil material is usually described using appropriate defect terms (eg, infilled seam). Thicker rock strength material is usually described as a vein.

BLOCK SHAPE TERMS

Blocky	Approximately equidimensional
Tabular	Thickness much less than length or width
Columnar	Height much greate than cross section

Notes on Defects:

1. Usually borehole logs show the true dip of defects and face sketches and sections the apparent dip.
2. Partings and joints are not usually shown on the graphic log unless considered significant.
3. Sheared zones, sheared surfaces and crushed seams are faults in geological terms.

Appendix B

Risk Assessment Procedure

APPENDIX G

LANDSLIDE RISK ASSESSMENT – EXAMPLE OF QUALITATIVE TERMINOLOGY FOR USE IN ASSESSING RISK TO PROPERTY

Qualitative Measures of Likelihood

Level	Descriptor	Description	Indicative Annual Probability
A	ALMOST CERTAIN	The event is expected to occur	$>\approx 10^{-1}$
B	LIKELY	The event will probably occur under adverse conditions	$\approx 10^{-2}$
C	POSSIBLE	The event could occur under adverse conditions	$\approx 10^{-3}$
D	UNLIKELY	The event might occur under very adverse circumstances	$\approx 10^{-4}$
E	RARE	The event is conceivable but only under exceptional circumstances.	$\approx 10^{-5}$
F	NOT CREDIBLE	The event is inconceivable or fanciful	$<10^{-6}$

Note: "≈" means that the indicative value may vary by say ±[illegible] order of magnitude, or more.

Qualitative Measures of Consequences to Property

Level	Descriptor	Description
1	CATASTROPHIC	Structure completely destroyed or large scale damage requiring major engineering works for stabilisation.
2	MAJOR	Extensive damage to most of structure, or extending beyond site boundaries requiring significant stabilisation works.
3	MEDIUM	Moderate damage to some of structure, or significant part of site requiring large stabilisation works.
4	MINOR	Limited damage to part of structure, or part of site requiring some reinstatement/stabilisation works.
5	INSIGNIFICANT	Little damage.

Note: The "Description" may be edited to suit a particular case.

Qualitative Risk Analysis Matrix – Level of Risk to Property

LIKELIHOOD	CONSEQUENCES to PROPERTY				
	1: CATASTROPHIC	2: MAJOR	3: MEDIUM	4: MINOR	5: INSIGNIFICANT
A – ALMOST CERTAIN	VH	VH	H	H	M
B – LIKELY	VH	H	H	M	L-M
C – POSSIBLE	H	H	M	L-M	VL-L
D – UNLIKELY	M-H	M	L-M	VL-L	VL
E – RARE	M-L	L-M	VL-L	VL	VL
F – NOT CREDIBLE	VL	VL	VL	VL	VL

Risk Level Implications

Risk Level		Example Implications(1)
VH	VERY HIGH RISK	Extensive detailed investigation and research, planning and implementation of treatment options essential to reduce risk to acceptable levels; may be too expensive and not practical
H	HIGH RISK	Detailed investigation, planning and implementation of treatment options required to reduce risk to acceptable levels
M	MODERATE RISK	Tolerable provided treatment plan is implemented to maintain or reduce risks. May be accepted. May require investigation and planning of treatment options.
L	LOW RISK	Usually accepted. Treatment requirements and responsibility to be defined to maintain or reduce risk.
VL	VERY LOW RISK	Acceptable. Manage by normal slope maintenance procedures.

Note:
(1) The implications for a particular situation are to be determined by all parties to the risk assessment; these are only given as a general guide.
(2) Judicious use of dual descriptors for Likelihood, Consequence and Risk to reflect the uncertainty of the estimate may be appropriate in some cases.

APPENDIX G

LANDSLIDE RISK ASSESSMENT – EXAMPLE OF QUALITATIVE TERMINOLOGY FOR USE IN ASSESSING RISK TO PROPERTY

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B	LIKELY	The event will probably occur under adverse conditions	$\approx 10^{-2}$
C	POSSIBLE	The event could occur under adverse conditions	$\approx 10^{-3}$
D	UNLIKELY	The event might occur under very adverse circumstances	$\approx 10^{-4}$
E	RARE	The event is conceivable but only under exceptional circumstances.	$\approx 10^{-5}$
F	NOT CREDIBLE	The event is inconceivable or fanciful	$<10^{-6}$

Note: "≈" means that the indicative value may vary by say ±½ order of magnitude, or more.

Qualitative Measures of Consequences to Property

Level	Descriptor	Description
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B – LIKELY	VH	H	H	M	L-M
C – POSSIBLE	H	H	M	L-M	VL-L
D – UNLIKELY	M-H	M	L-M	VL-L	VL
E – RARE	M-L	L-M	VL-L	VL	VL
F – NOT CREDIBLE	VL	VL	VL	VL	VL

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Note:
(1) The implications for a particular situation are to be determined by all parties to the risk assessment; these are only given as a general guide.
(2) Judicious use of dual descriptors for Likelihood, Consequence and Risk to reflect the uncertainty of the estimate may be appropriate in some cases.

Appendix C

Summary of Qualitative Risk Assessment

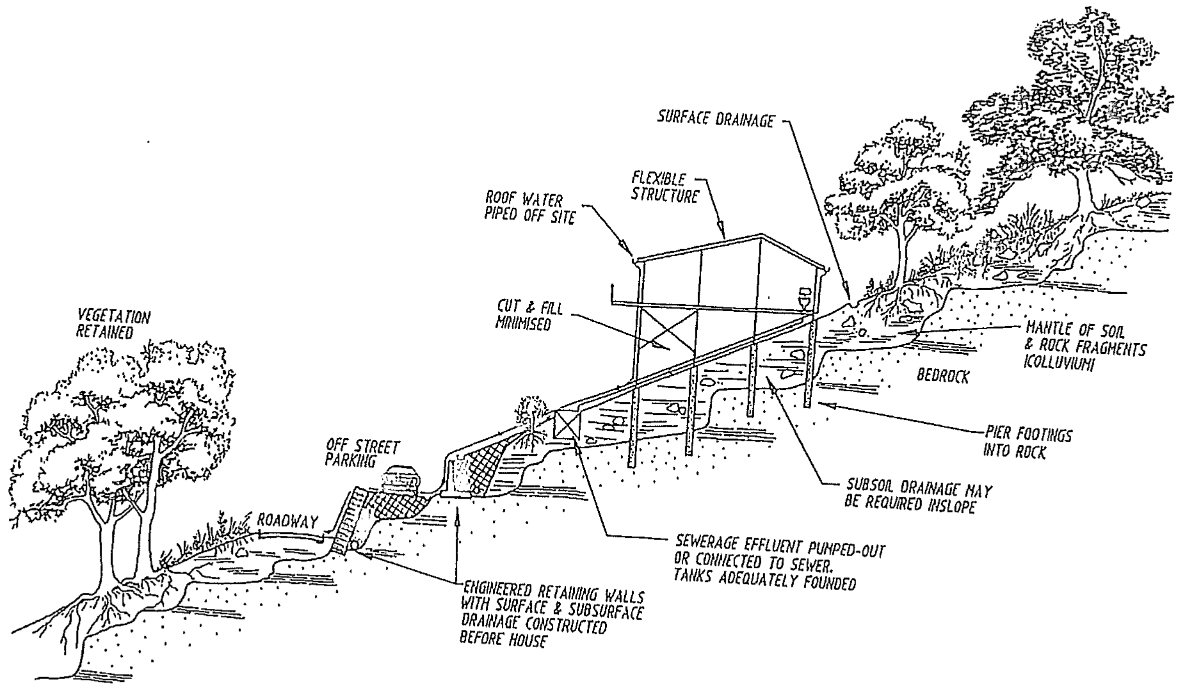
Hazard	Likelihood	Consequence	Risk	Comments
Failure of the slope under 'High Noon' Lodge	Unlikely	Medium	Low to medium	No obvious evidence of natural slope failures. Batter angle of slope under 'High Noon' Lodge is relatively flat (between 10° to 15°). There were no significant gully features observed above the site that could produce a flow.
Failure of the thin fill layer in Diggings Terrace	Unlikely	Minor	Low	Based on the relatively flat slope angle along Diggings Terrace and that there are no obvious evidence of cracking or failure in the pavement through the asphalt, it was assessed that slides would be very unlikely to develop and would be unlikely to result in a failure. Saturation of the fill soils in the pavement under Diggings Terrace could result in small scale failure, however there seems to be adequate drainage across this area.
Failure of the slope under 'Black Bear Inn'	Rare	Major	Low to Moderate	Saturation of the soils in altered slopes at the site may lead to failure. We understand the development will comprise the excavation of most of the fill and some of the colluvial materials in the slope. If the development is constructed using the recommendations of this report and in accordance with standard engineering practice a low hazard has been assessed.
Failure of the cut slope behind 'Mowamba'	Rare	Medium	Low	Based on the previous stabilisation works that have been carried out for the 'Mowamba' site and that there is no evidence of any slope instability, it is assessed that slides would be very unlikely to develop and result in a failure.

Note: The likelihood of the abovementioned hazards has been reduced since August 1997 with the installation of slope management measures including improvements in the collection of surface runoff and roof water disposal systems at each lodge, construction of over 1km of stormwater trunk drains through the village and the construction of some 150 horizontal drains to lower groundwater levels

Appendix D

Examples of Good and Bad Hillside Practice

EXAMPLES OF GOOD HILLSIDE PRACTICE



EXAMPLES OF POOR HILLSIDE PRACTICE

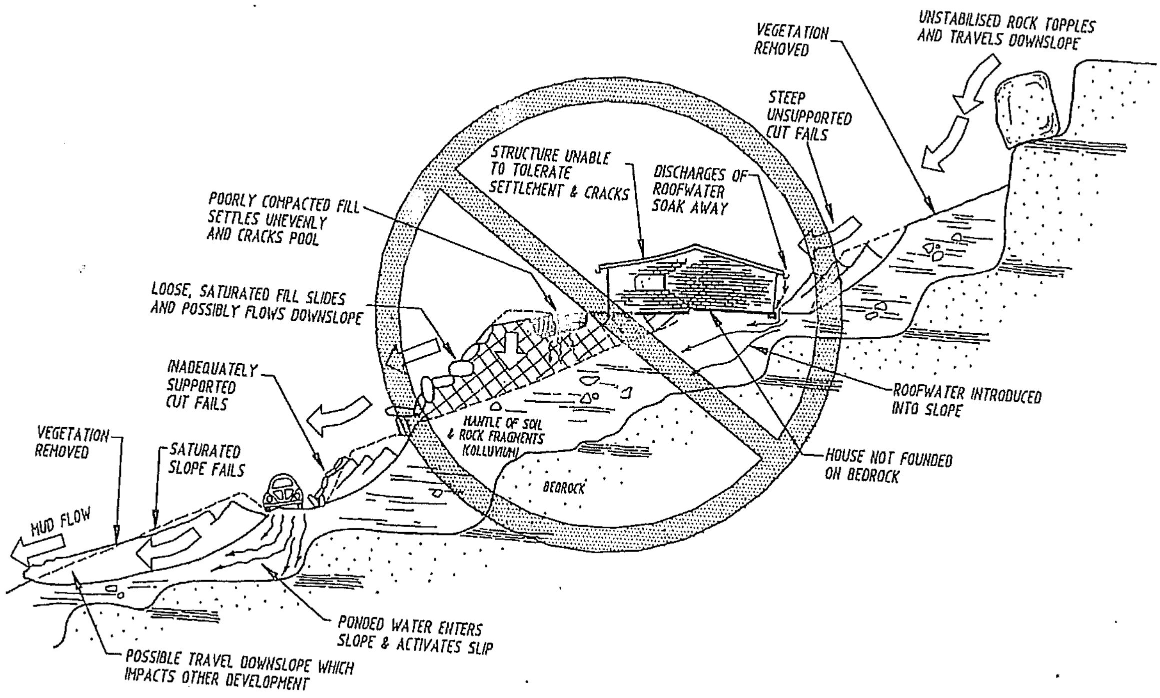


FIGURE 2: ILLUSTRATIONS OF GOOD AND POOR HILLSIDE PRACTICE

This figure is an extract form LANDSLIDE RISK MANAGEMENT CONCEPTS AND GUIDELINES as presented in *Australian Geomechanics*, Vol 35, No 1, 2000 which discusses the matter more fully.

TABLE 2

SOME GUIDELINES FOR HILLSIDE CONSTRUCTION

	GOOD ENGINEERING PRACTICE	POOR ENGINEERING PRACTICE
ADVICE		
GEOTECHNICAL ASSESSMENT	Obtain advice from a qualified, experienced geotechnical consultant at early stage of planning and before site works.	Prepare detailed plan and start site works before geotechnical advice.
PLANNING		
SITE PLANNING	Having obtained geotechnical advice, plan the development with the Risk of Instability and Implications for Development in mind.	Plan development without regard for the Risk of Instability.
DESIGN AND CONSTRUCTION		
HOUSE DESIGN	Use flexible structures which incorporate properly designed brickwork, timber or steel frames, timber or panel cladding. Consider use of split levels. Use decks for recreational areas where appropriate.	Floor plans which require extensive cutting and filling. Movement intolerant structures.
SITE CLEARING	Retain natural vegetation wherever practicable.	Indiscriminately clear the site.
ACCESS & DRIVEWAYS	Satisfy requirements below for cuts, fills, retaining walls and drainage. Council specifications for grades may need to be modified. Driveways and parking areas may need to be fully supported on piers.	Excavate and fill for site access before geotechnical advice.
EARTHWORKS	Retain natural contours wherever possible.	
CUTS	Minimise depth. Support with engineered retaining walls or batter to appropriate slope. Provide drainage measures and erosion control.	Large scale cuts and benching. Unsupported cuts. Ignore drainage requirements.
FILLS	Minimise height. Strip vegetation and topsoil and key into natural slopes prior to filling. Use and compact clean fill materials. Batter to appropriate slope or support with engineered retaining wall. Provide surface drainage and appropriate subsurface drainage.	Loose or poorly compacted fill. Block natural drainage lines. Fill over existing vegetation and topsoil. Include stumps, trees, vegetation, topsoil, boulders, building rubble etc in fill.
ROCK OUTCROPS & BOULDERS	Remove or stabilise boulders which may become unstable. Support rock faces where necessary.	Disturb or undercut detached blocks or boulders.
RETAINING WALLS	Engineer design to resist applied soil and water forces. Found on rock where practicable. Provide subsurface drainage within wall backfill and surface drainage on slope above. Construct wall as soon as possible after cut/fill operation.	Construct a structurally inadequate wall such as sandstone flagging, brick or unreinforced blockwork. Lack of subsurface drains and weepholes.
FOUNDATIONS	Support on or within rock where practicable. Use rows of piers or strip foundations oriented up and down slope. Design for lateral creep pressures. Backfill foundation excavations to exclude ingress of surface water.	Found on topsoil, loose fill, detached boulders or undercut cliffs.
SWIMMING POOLS	Engineer designed. Support on piers to rock where practicable. Provide with under-drainage and gravity drain outlet where practicable. Design for high soil pressures which may develop on uphill side whilst there may be little or no lateral support on downhill side.	
DRAINAGE		
SURFACE	Provide at tops of cut and fill slopes. Discharge to street drainage or natural water courses. Provide generous falls to prevent blockage by siltation and incorporate silt traps. Line to minimise infiltration and make flexible where possible. Special structures to dissipate energy at changes of slope and/or direction.	Discharge at top of fills and cuts. Allow water to pond on bench areas.
SUBSURFACE	Provide filter around subsurface drain. Provide drain behind retaining walls. Use flexible pipelines with access for maintenance. Prevent inflow of surface water.	
SEPTIC & SULLAGE	Usually requires pump-out or mains sewer systems; absorption trenches may be possible in some low risk areas. Storage tanks should be water-tight and adequately founded.	Discharge sullage directly onto and into slopes.
EROSION CONTROL & LANDSCAPING	Control erosion as this may lead to instability. Revegetate cleared area.	Failure to observe earthworks and drainage recommendations when landscaping.
DRAWINGS AND SITE VISITS DURING CONSTRUCTION		
DRAWINGS	Building Application drawings should be viewed by geotechnical consultant.	
SITE VISITS	Site Visits by consultant may be appropriate during construction.	
INSPECTION AND MAINTENANCE BY OWNER		
OWNER'S RESPONSIBILITY	Clean drainage systems; repair broken joints in drains and leaks in supply pipes. Where structural distress is evident seek advice. If seepage observed, determine cause or seek advice on consequences.	

This table is an extract from GEOTECHNICAL RISKS ASSOCIATED WITH HILLSIDE DEVELOPMENT as presented in Australian Geomechanics News, Number 10, 1985 which discusses the matter more fully.

Appendix E

Form 1

Department of
Infrastructure, Planning and Natural Resources

Geotechnical Policy – Kosciuszko Alpine Resorts Form 1 – Declaration and certification made by geotechnical engineer or engineering geologist in a geotechnical report.

Date received: ____/____/____

DA no: ______________

To be submitted with a development application.

You can use Form 1 to verify that the author of a geotechnical report is a geotechnical engineer or engineering geologist as defined by DIPNR Geotechnical Policy. Alternatively, where a geotechnical report has been prepared by a professional person not recognised by DIPNR Geotechnical Policy, then form 1 may be used as technical verification of the geotechnical report if signed by a geotechnical engineer or engineering geologist as defined by the DIPNR Geotechnical Policy.

Please contact the Alpine Resorts Assessments Team in Jindabyne for further information. Phone 02 6456 1733.

To complete this form, please place a cross in the boxes ☐ and fill out the white sections

1. Declaration made by geotechnical engineer or engineering geologist as part of a geotechnical report

I

Mr ☑ Ms ☐ Mrs ☐ Dr ☐ Other ______

PARAN

Family name
MOYES

OF

Company/organisation
COFFEY GEOTECHNICS

on this the 4 day of MAY 2007

certify that I am a geotechnical engineer ~~or engineering geologist~~ as defined by the "Policy" and I: (tick appropriate box)

☑ prepared the geotechnical report referenced below in accordance with the AGS 2000 and DIPNR Geotechnical Policy – Kosciuszko Alpine Resorts.

☐ am willing to technically verify that the Geotechnical Report referenced below has been prepared in accordance the AGS 2000 and the Geotechnical Policy – Kosciuszko Alpine Resorts.

2. Geotechnical Report Details

Report Title
BLACK BEAR INN

Author
PARAN MOYES

Dated

DA Site Address
LOT 49 DIGGINGS TERRACE

DA Applicant

I am aware that the Geotechnical Report I have either prepared or am technically verifying (referenced above) is to be submitted in support of a development application for the proposed development site (referenced above) and its findings will be relied upon by the Consent Authority in determining the development application.

3. Checklist of essential requirements to be contained in a geotechnical risk assessment report to be submitted with a development application

The following checklist covers the minimum requirements to be addressed in a Geotechnical Risk Management Report. This checklist is to accompany the report.

Please tick appropriate box

- ☑ Risk assessment of ~~all~~ identifiable geotechnical hazards in accordance with AGS 2000, as per 6.1 (a) of the policy
- ~~☐ Site plans with key hazards identified and other information as per 6.1 (b)~~
- ☑ Details of site investigation and inspections as per 6.1 (c)
- ☐ Photographs and/or drawings of the site as per 6.1 (d)
- ☑ Presentation of geotechnical model as per 6.1 (e)
- ☐ A specific conclusion as to whether the site is suitable for the development proposed on the above site, if applicable, subject to the following conditions:
  - ☐ Conditions to be provided to establish design parameters.
  - ☑ Conditions to be incorporated into the detailed design to be submitted for the construction certificate.
  - ☑ Conditions applying to the construction phase.
  - ☐ Conditions relating to ongoing management of the site/structure.

4. Signatures

Signature: Peter Volk	Chartered professional status: R.P.Geo
Name: PETER L. VOLK	Date: 4/5/07

5. Contact details

Alpine Resorts Assessments team
Snowy River Avenue
PO Box 36 JINDABYNE 2627
t: 02 6456 1733
f: 02 6456 1736
e: [illegible]

Crozier Geotechnical Consultants
Unit 12/ 42-46 Wattle Road
Brookvale NSW 2100
Crozier Geotechnical Consultants, a division of PJC Geo-Engineering Pty Ltd
ABN: 96 113 453 624
Phone: (02) 9939 1882
Email: info@croziergeotech.com.au

REPORT ON GEOTECHNICAL ASSESSMENT

for

PROPOSED NEW APARTMENTS

at

30 DIGGINGS TERRACE, THREDBO
'BLACK BEAR INN'

Prepared For

Hidali Pty Ltd

Project No.: 2019-121
August, 2019

Document Revision Record

Issue No	Date	Details of Revisions
0	7th August 2019	Original issue
1	15th October 2020	Response to DA and Mod Contentions

C

NSW GOVERNMENT | Planning & Environment

Geotechnical Policy

Kosciuszko Alpine Resorts

Form 1 – Declaration and certification made by geotechnical engineer or engineering geologist in a geotechnical report.

DA Number: 2020/68009

To be submitted with a development application

You can use Form 1 to verify that the author of a geotechnical report is a geotechnical engineer or engineering geologist as defined by the Department of Planning & Environment (DP&E) Geotechnical Policy. Alternatively, where a geotechnical report has been prepared by a professional person not recognised by DP&E Geotechnical Policy, then Form 1 may be used as technical verification of the geotechnical report if signed by a geotechnical engineer or engineering geologist as defined by the DP&E Geotechnical Policy.

Please contact the Alpine Resorts Team in Jindabyne for further information - phone 02 6456 1733.

To complete this form, please place a cross in the appropriate boxes ☐ and complete all sections.

1. Declaration made by geotechnical engineer or engineering geologist as part of a geotechnical report

I,

Mr ☒ Ms ☐ Mrs ☐ Dr ☐ Other ☐

First Name: Troy

Family Name: Crozier

OF

Company/organisation: CROZIER GEOTECHNICAL CONSULTANTS

on this the 15 day of October 2020

certify that I am a geotechnical engineer or engineering geologist as defined by the "Policy" and I (tick appropriate box)

☒ prepared the geotechnical report referenced below in accordance with the AGS 2000 and DP&E Geotechnical Policy – Kosciuszko Alpine Resorts.

☐ am willing to technically verify that the Geotechnical Report referenced below has been prepared in accordance the AGS 2000 and DP&E Geotechnical Policy – Kosciuszko Alpine Resorts.

2. Geotechnical Report Details

Report Title: Geotechnical Assessment for Proposed New Apartments

Author: T. Crozier

Dated: 15-10-20

DA Site Address: 30 Diggings Terrace, Thredbo
Lot 794 DP1119757

DA Applicant: Hidali Pty Ltd

I am aware that the Geotechnical Report I have either prepared or am technically verifying, (referenced above) is to be submitted in support of a development application for the proposed development site (referenced above), and it's findings will be relied upon by the Consent Authority in determining the development application.

3. Checklist of essential requirements to be contained in a geotechnical risk assessment report to be submitted with a development application

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    - [x] Conditions to be provided to establish design parameters,
    - [x] Conditions to be incorporated into the detailed design to be submitted for the construction certificate,
    - [x] Conditions applying to the construction phase,
    - [x] Conditions relating to ongoing management of the site/structure.

4. Signatures

Signature	Chartered professional status
[signature]	RPGeo. Geotechnical & Eng. No. 10197
Name	**Date**
Troy Crozier	15-10-20

5. Contact details

Department of Planning & Environment
Alpine Resorts Team
Shop 5A, 19 Snowy River Avenue
PO Box 36, JINDABYNE 2627
Telephone: 02 6456 1733
Facsimile: 02 6456 1736
Email: alpineresorts@planning.nsw.gov.au

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GEOTECHNICAL ASSESSMENT FOR PROPOSED NEW APARTMENTS
'BLACK BEAR' 30 DIGGINGS TERRACE, THREDBO, NSW

1. INTRODUCTION:

This report details the results of geotechnical assessment supplied as part of a Development Application (DA) and provides response to contentions to the DA and subsequent modification application for a proposed new apartment building óBlack Bearô at 30 Diggings Terrace, Thredbo, NSW. The assessment and response was undertaken by Crozier Geotechnical Consultants (CGC) at the request of the client Hidali Pty Ltd.

It is understood that the existing óBlack Bear Innô structure will be demolished and a new seven level apartment and restaurant structure built.

The site is located within an area designated óGô within the Geotechnical Policy - Kosciuszko Alpine Resorts maps therefore a geotechnical report which meets the requirements of Section 4.0 of the Policy is required for submission with a DA.

This report includes a comparison of the new DA/modification design against the previously approved DA design, a summary description of the field work completed by others on the site, fieldwork and inspections by CGC in relation to this site and an adjacent development and provides recommendations for assessment and engineering design of the new proposal. It also includes a geotechnical assessment and landslide risk assessment and provides recommendations for construction to maintain an óAcceptableô risk level as defined by the Australian Geomechanics Society ï Guidelines for Landslide Risk Management. 2007.

A Development Consent was supplied (DA 33-7-2007, Dated: 1st August 2008) for the demolition of the existing structure and construction of a new 6 level development consisting of 18 apartments. The developer now proposed to amend the approved design therefore a new Development Application (DA 2020/68009) and a subsequent modification application (2020/68022) have been lodged. The changes/variation to the approved design are addressed within this report.

The following plans, diagrams and documents were supplied for this report;

- 2007 Consent Design by APA Architects/planners, Drawing No.: 0277-DA000 to 0277-DA022, Issue: L, Dated: 30th May 2007.
- Site Survey Plan by Peter W Burns, Reference: 3576, Drawing No.: CD01, Revision: C, Dated: 24/09/2007.Modification
- Geotechnical Report by Coffey Geotechnics, Reference No.: GEOTLCOV23158AA-AB, Revision: 1, Dated: 14th May 2007.
- Architectural Design Drawings by Popov Bass, Drawing No.: DA 000 to DA 020, Revision: 02, Dated: 19th August 2019.
- Modification CL 4.55 Design by Popov Bass, Drawing No.: 0555-DA000 to 0555-DA020, Revision: 01, Dated: 24th October 2019.
- Shoring plan and Details by Murtagh Bond Consulting Engineers, Drawing No.: SK1 and SK2, Dated: 9th September 2020.

2. DEVELOPMENT PROPOSALS:

2.1 Approved Development:

The approved development (DA 33-7-2007), which was physically commenced, involved demolition of the existing structures and construction of a six level apartment building formed within an excavation into the hill slope along its southern side. The lowest level (Level 1) was designed with Finished Floor Level (FFL) at R.L. 1381.00 that would involve a Base Excavation Level (BEL) at approximate R.L. 1380.50. The building had an east side setback of 6.795m, south boundary setback of 6.288m and west side setback of 3.070m.

Level 2 and Level 3 above had similar footprints with east side setback of 3.145m, south boundary setback of 3.583m and west side setback of Ó2.145m. Level 4 to Level 6 were above ground with an open car parking area fronting onto Diggings Terrace at Level 4.

2.2 Proposed Development:

The DA and subsequent modification application design involves demolition of the existing structure and construction of a new seven level apartment with restaurant and internal parking. The lowest level (Level 00) is designed with a FFL at R.L. 1380.60 and therefore requires an excavation of up to a maximum 8.0m depth to achieve an BEL of approximately R.L. 1380.00 at the south-east corner.

The natural ground surface fall to the south-west results in the excavation reducing to nil at the north-west corner of Level 00.

Both Level 00 and Level 1 have a similar footprint and are located approximately 2.90m from the western property boundary (No. 98 óSashasô), approximately 7.00m from the southern boundary to Diggings Terrace, >2.10m from the northern boundary and 7.00m from the eastern property boundary (No. 5 óCandlelight Lodgeô). However, Level 1 extends to the east for a gym at its northern end, which extends to 2.40m off the eastern boundary, with maximum excavation up to 3.00m depth.

Level 2 occupies a larger footprint and includes a driveway access that extends along part of the western side boundary. The excavation for this level is up to 3.50m depth at the south-east corner, reducing to nil across the entire north-western two-thirds of the development due to the hill slope. The excavation is 4.73m to 6.50m from the southern Diggings Terrace boundary, 2.60m from the eastern boundary and 4.14m from the south-east corner boundary.

Level 3 requires an excavation of up to 1.5m depth at the south-east corner only with all other levels/areas located above ground surface levels and requiring no bulk excavation.

2.3. Comparison:

The proposed design will involve a BEL of Ò1.0m depth greater than the approved design. The approved design showed an undetailed excavation support system located adjacent to developments external walls. However, due to the now proposed staged and independent excavation support system, the excavation will be of increased depth due to the need to excavate further south at Level 00, Level 1 and Level 2 to allow creation of a cavity into which the new development can be constructed.

The proposed lower level (Level 00) is located a similar distance off the east boundary (approx. 7.0m) as the approved design and a similar distance off the west boundary (approx. 3.0m). The approved design has a setback from the south boundary of 6.3m however the new design will involve a bulk excavation within proximity (<1.0m) of the south boundary.

The proposed second level (Level 1) will be located slightly closer to the east boundary and slightly further from the west boundary than the approved design (Level 2). The approved design has a setback from the south boundary of 6.3m however the new design will involve a bulk excavation within proximity (<1.0m) of the south boundary.

Similarly, the proposed third level (Level 2) will also be located slightly closer to the east boundary and slightly further from the west boundary than the approved design (Level 3). All other levels of both designs were essentially above ground surface levels and required no bulk excavation.

From a geotechnical perspective, the proposed works are very similar to approved and do not create any new or increased challenges provided the works are undertaken by a locally experienced contractor with geotechnical assessment and inspection as per the recommendations of this report.

3. SITE FEATURES:

3.1. Description:

The site is a rectangular shaped block located on the low north side of Diggings Terrace within moderately north-west dipping topography close to the base of the Thredbo Village hill slope. It contains a four level lodge and restaurant of masonry and timber construction on the front southern half with open grassed land including several low retaining walls on the northern side. The southern side of the lower level appears partly excavated into the hill slope whilst the rear northern side is raised up to 1.50m above ground at the north-west corner. The lower level appears supported on fill soils retained by a mortared rock retaining wall that appears to form part of the buildings footing system.

The site falls from an approximate high of R.L. 1392.0 in the south-east corner to a low of approximately R.L. 1379.5 in the north-west corner. The site has a stepped front south boundary of 26.295m and side west boundary of 27.88m in length, as referenced from the provided survey plan.

An aerial photograph of the site and its surrounds is provided below, as sourced from NSW Government Six Map spatial data, as Photograph 1.

Photograph: 1 ï site and surrounding properties

4. FIELD WORK:

4.1. Methods:

A field investigation was undertaken by Coffey Geotechnics in June 2000 and comprised the drilling of two boreholes up to 4.40m depth at the front southern side of the existing lodge building. Another investigation was undertaken in June 2003 and comprised extension of the previous Borehole 1 to a total of 11.40m depth along with installation of a groundwater monitoring well/piezometer and measurement of water levels. A geological model/section showing identified geological conditions, as prepared by Coffey Geotechnics, with the DA proposed excavation outline is supplied in Appendix: 2.

A walk over inspection of the site and inspection of adjacent properties was undertaken by a Principal Engineering Geologist from Crozier Geotechnical Consultants on the 21st May 2019.

Inspections were also undertaken by the Principal Engineering Geologist during excavation and construction works in 2017 to 2019 for the nearby Mittabah Lodge, located approximately 50m to the south-east at No. 716 Bobuck Lane.

4.2. Field Observations:

The existing ôBlack Bear Innô building is at least 50 years of age and is formed of masonry and timber construction that appears supported off mortared rock footing walls at shallow depth around the perimeter. This footing wall increases to approximately 1.50m in height at the north-west corner of the building. An opening within the footing wall, created for previous service line repairs on the northern side, indicates that the sub-floor area of the building is in part underlain by fill soils placed to form a level pad for construction that is retained by the rock footing walls. The existing building shows deterioration due to age and some minor cracking at the front southern side due to what is understood to be infill/repair of a concrete tank, and the western side due to footing settlement, however there are no indications of significant slope movement.

The neighbouring property to the east No. 5 ôCandelight Lodgeô contains a three level masonry and timber development on the front half of the block located within approximately 1.0m off the eastern boundary of the site. A concrete driveway provides access to the south-west corner of this property at lower floor level, past the south-east corner of the site. This driveway is retained along the boundary by an approximately 1.5m high sloped rock retaining wall, see Photograph: 2. The building structure appears of similar age to the existing ôBlack Bear Innô building and appears formed above ground surface levels. There were no indications on external walls of any foundation/footing movement adjacent to the site.

The neighbouring property to the west (No. 98 óSashasô) contains a three level masonry development located 1.5m to 2.0m from the western boundary of the site. The building structure appears of similar age to the existing óBlack Bear Innô building and appears formed above ground surface levels. There were no indications on external walls of any foundation/footing movement adjacent to the site.

Diggings Terrace is a bitumen paved road with moderate west dip and no kerb or gutter formed adjacent to the site or adjacent properties. Inspection of the road pavement did not identify any signs of excess cracking or deformation to indicate slope movement.

The neighbouring property upslope (No. 12 Banjo Driveway) is retained above the road pavement of Diggings Terrace by a low (<1.0m) rock retaining wall with moderate sloping lawn areas extending up to a two storey timber lodge building supported on its northern side above ground surface by a mortared rock footing wall. There were no indications on external walls of any foundation/footing movement adjacent to the site.

Photograph: 2 – South-east corner of site showing neighbouring (No. 5) driveway and retention.

5. COMMENTS:

5.1. Geotechnical Assessment:

The site investigations and inspections identified no signs of recent landslip instability within the site or adjacent properties with no indications of excess surface stormwater flow or groundwater seepage identified.

The borehole drilled by Coffey Geotechnics, along with the inspection results from the Mittabah excavation, indicate that granular fill soils may extend up to 1.50m depth on this site, where previous development has occurred, and overlie silty sand with trace of gravel that grades to weathered granodiorite around 2.50m depth. The granodiorite will be encountered as medium to high strength boulders/core stones of variable sizes surrounded by extremely weathered material. The concentration of hard core stones is expected to increase with depth resulting in dominantly medium to high strength rock below approximately 5.0m at all locations across the site, however it may also be highly variable.

A standing groundwater table was interpreted by Coffey at 9.77m depth based on the piezometer installed within BH 1 and other instruments they indicate were installed within the local area. This places the interpreted water table at R.L. 1380.3 within Diggings Terrace and at approximately R.L. 1285.0 at the rear north boundary of the site. During the construction of the Mittabah Apartments a moderate (estimated 10L/min) level of groundwater seepage was encountered in the base of the excavation, below approximately 7.0m depth. However, this seepage was isolated to one portion of the excavation only with all other areas above and to 8.50m depth encountering no seepage flow. The proposed excavation is therefore likely to encounter moderate levels of seepage in the lower portions however it is not expected intersect a standing groundwater table.

An engineered hydraulic system including stormwater management could be designed based on the estimated water ingress level from the Mittabah excavation in combination with measured rates encountered in that installed system to manage and capture groundwater within the site. The design for the site can then be modified based on actual groundwater seepage rates encountered during the excavation works within the site. As groundwater seepage location and depth was identified as being highly variable within the Mittabah excavation it is considered that further investigation prior to development will be of limited accuracy and use.

The proposed development involves an excavation of significant depth (up to 10.0m) however a similar excavation was recently completed in an adjacent property without inducing landslip instability or creating detrimental impact to adjacent properties/structures.

The excavation for the Mittabah development was undertaken as a staged excavation and support (reinforced shotcrete and anchors) system without incident. This system involved 1.50 to 2.0m depth cut intervals supported by an anchored shotcrete wall prior to the next phase of excavation. It dealt with the seepage inflow via installation of sub-horizontal drainage pipes in the lower portion of the excavation and a similar system could be implemented during the site works from near the excavation base.

The excavation at the site is proposed to be undertaken via the installation of a soldier pile support wall with shotcrete infill panels that will utilise an arching stress support system and bracing within the excavation via a second piled support wall and side boundary pile walls. This will involve piles being installed prior to excavation thus the excavation will be supported at all times and will not be left in an unsupported state due to weather or seasonal interruptions/delays.

The high strength of core stones within the bedrock must be considered when selecting the piling equipment as these may prove difficult and costly to drill through to achieve the required embedment/foundation depths. Similarly, the potential for significant seepage inflow/water table in the base of the soldier piles is expected to require a CFA or cased system to ensure foundation integrity is maintained in the pile bases.

The proposed changes to the original design do not significantly alter the geotechnical aspects of the proposed development or the site from those on which the original report were based. As such, the proposed works are considered suitable for the site and may be completed with negligible impact to existing nearby structures within the site or neighbouring properties provided the recommendations of this and future reports are implemented in the design and construction phases.

5.2. Slope Stability & Risk Assessment:

Based on the investigation/inspections we have identified the following credible geological/geotechnical landslip hazard which needs to be considered in relation to the proposed works. The hazard is:

A. Landslip (earth slide Ò5m^3) of soils/weathered rock from excavation for Level 2
B. Landslip (earth slide 10 - 15m^3) of soils/weathered rock from deeper excavation Level 00 to Level 2

A qualitative assessment of risk to life and property related to these hazards is presented in Table: A and B, Appendix: 3, and is based on methods outlined in Appendix: C of the Australian Geomechanics Society (AGS) Guidelines for Landslide Risk Management 2007. AGS terms and their descriptions are provided in Appendix: 4.

Hazard A was estimated to have a **Risk to Life** of up to **3.91 x 10^{-8} for persons,** while the **Risk to Property** was considered to be **'Very Low'**.

Hazard B was estimated to have a **Risk to Life** of up to **5.86 x 10^{-6} for persons,** while the **Risk to Property** was considered to be **'Low'**.

The hazards were assessed for instability during site works and were considered to be within the óTolerableô risk levels of the AGS 2007 guidelines. Provided permanent support systems, including engineered footings, are completed then the Likelihood of instability occurring over a design life of 50 years is further reduced and as such following completion of the development Risk to Life and Risk to Property values will continue to remain well within the óTolerableô criteria. Therefore, the project is considered suitable for the site provided the recommendations of this report are implemented.

5.3. Design & Construction Recommendations:

Design and the construction recommendations are tabulated below:

5.3.1. New Footings:	
Site Classification as per AS2870 ï 2011 for new footing design	Class óAô for footings into weathered bedrock at base of excavation, non-reactive granular soils
Type of Footing	Shallow strip/pad at base of excavation potential requirement for piles or deep pad footing excavations to north-west due to ground surface fall and excavation reduction
Sub-grade material and Maximum Allowable Bearing Capacity	Weathered, Bedrock: 500kPa*
Site sub-soil classification as per *Structural design actions AS1170.4 – 2007, Part 4: Earthquake actions in Australia*	B_e ï Rock Site
Remarks: *requires inspection/confirmation by geotechnical engineer/engineering geologist in each and every footing All new footings must be inspected by an experienced geotechnical professional before concrete or steel are placed to verify the bearing capacities and stability. This is mandatory to allow them to be ócertifiedô at the end of the project.	

5.3.2. Excavation:	
Depth of Excavation	Level 2 excavation up to 3.50m depth Level 00 ï 1 ï 2 excavation up to 8.0m depth.
Type of Material to be Excavated	Granular Fill to Ò1.50m depth
	Silty sand with gravel to Ò2.50m depth
	ELS bedrock with HS core stones to base of excavation

Guidelines for <u>un-surcharged</u> batter slopes for general information are tabulated below:

Material	Recommended Safe Batter Slope (H:V)	
	Short Term/Temporary	Long Term/Permanent
Fill and granular soils	1.5:1	2:1
ELS with HS*	0.5:1.0	1.5:1

Remarks:
*The ELS bedrock with HS core stones may be excavated at sub-vertical batter slopes with short term stability where by seepage is not encountered, however the stability for small scale (<2m³) failures in this situation cannot be guaranteed.

Seepage through the soils and weathered bedrock is expected, mainly in the lower portions of the excavation, and will reduce the stability of batter slopes. This may invoke the need to implement additional (temporary) support measures. Where the recommended safe batter slopes are not implemented the stability of any excavation cannot be guaranteed until the installation of permanent support measures. This should also be considered with respect to safe working conditions.

Geotechnical inspection of batters and excavation faces prior to support installation will be required at regular intervals to assess their stability and site conditions, especially for permanent batters.

Equipment for Excavation	Soils and ELS	Excavator with Bucket
	VLS bedrock	Bucket and ripper
	LS ï HS	Rock hammer

ELS ï extremely low strength, VLS ï very low strength, LS ï low strength, MS ï medium strength

Recommended Vibration Limits (Maximum Peak Particle Velocity (PPV))	5mm/s for all structures
Vibration Assessment Required	Only if large (>250kg) rock excavation equipment required within 5.0m lateral/vertical distance of any building footings
Full time vibration Monitoring Required	Unlikely
Geotechnical Inspection Requirement	Yes, as per Section 4.4
Dilapidation Surveys Requirement	Recommended on building structures or part thereof within 8m of excavation perimeter

Remarks:

Water ingress into exposed excavations can result in erosion and stability concerns in both soils and weathered bedrock. Drainage measures will need to be in place during excavation works to divert any surface flow away from the excavation crest and any batter slope, whilst any groundwater seepage must be controlled within the excavation and prevented from ponding or saturating slopes/batters.

5.3.3. Retaining Structures:

Required	New retaining structures are be required as part of the proposed development to support the excavation perimeters.
Types	Reinforced bored soldier pile support wall prior to bulk excavation or anchored shotcrete wall in stages <2.0m in height. Steel reinforced concrete/concrete block walls post excavation, where temporary batters can be maintained. All designed to Australian Standards AS4678-2002 Earth Retaining Structures.

Parameters for calculating pressures (unsurcharged) acting on retaining walls for the materials likely to be encountered:

Material	Unit Weight (kN/m3)	Long Term (Drained)	Earth Pressure Coefficients		Passive Earth Pressure/ Coefficient
			Active (Ka)	At Rest (K_0)	
Soils	18	$\phi' = 30°$	0.40	0.55	N/A
ELS bedrock with HS corestones	23	$\phi' = 38°$	0.25	0.30	200 kPa

Remarks:

In suggesting these parameters it is assumed that the retaining walls will be fully drained with suitable subsoil drains provided at the rear of the walls to release seepage. If this is not done, then the walls should be designed to support hydrostatic pressures in addition to pressures due to the soil/backfill. It is suggested that back fill for retaining walls be free-draining granular material (preferably not recycled concrete) which is only lightly compacted in order to minimize horizontal stresses. Weathered bedrock from the site is considered suitable.

Retaining structures near site boundaries or existing structures should be designed with the use of at rest (K_0) earth pressure coefficients to reduce the risk of movement in the excavation support and resulting surface movement in adjoining areas. Backfilled retaining walls within the site, away from site boundaries or existing structures, that may deflect can utilize active earth pressure coefficients (Ka).

It is considered that a triangular pressure distribution will exist for the excavation support however where negligible lateral deflection is maintained in the upper portions of a staged/anchored retention system then rectangular distribution (6H) is expected in at least the short term.

A survey monitoring program should be implemented for the excavation support wall with survey points installed by a registered surveyor prior to any bulk excavation and then re-measured at 3.0m depth intervals of excavation or at maximum 4 week intervals during any delay period to confirm that deflections remain within expected/modelled levels. Data from the surveying should be made available to the geotechnical and structural engineers for assessment upon collection.
For anchors drilled into weathered bedrock to approximately 5.0m depth below surface a grout/rock bond stress of 100kPa is considered suitable, however below 5.0m depth the concentration of MS ï HS rock is expected to increase therefore a grout/rock bond stress of 200kPa is considered suitable in this material provided inspection during anchor installation confirms this condition. However, anchors should be stress tested to the relevant standards and it is recommended that a minimum of 3 anchors be tested to failure within the full height of the excavation to allow assessment of grout/rock adhesion values.

5.3.4. Drainage and Hydrogeology		
Groundwater Table or Seepage identified in Investigation		Yes, ground water estimated at 9.77m depth below surface within Diggings Terrace
Excavation likely to intersect	Water Table	No
	Seepage	Moderate in deeper levels (10L/min), within potential isolated zones
Site Location and Topography		Moderate sloping topography, low north side of road
Impact of development on local hydrogeology		Negligible following installation of retention and hydraulic system
Onsite Stormwater Disposal		Not suitable.
Remarks: The excavation faces are expected to encounter some seepage especially at depth within isolated zones, therefore a system should be installed at the base of excavation cuts to below floor slab levels to reduce the risk of resulting dampness issues. Trenches, as well as all new building gutters, down pipes and stormwater intercept trenches should be connected to a stormwater system designed by a Hydraulic Engineer which discharges to the Councilôs stormwater system off site.		

5.4. Conditions Relating to Design and Construction Monitoring:

To allow certification as part of construction, building and post-construction activity for this project, it will be necessary for Crozier Geotechnical Consultants to:

1. Review, including 3D analysis of deflection of support system, and approval of the structural design drawings for compliance with the recommendations of this report with signing of Form 2 prior to Construction Certificate.
2. Inspection of bored excavation soldier piles during installation
3. Inspection of initial excavation works and any soil nail installation and testing results for upper row, where anchored system is proposed
4. Review of survey monitoring points for confirmation of deflection expectations and allowance for installation of additional support/stiffening systems if required
5. Inspection of benching and site/temporary batter stability where proposed across site
6. Inspect site conditions where any variability to the expected sub-surface conditions is identified during excavation
7. Inspection of lower levels of excavation (including any anchor installation and testing results)
8. Inspection of completed excavation and support systems and seepage control measures
9. Inspect all footings to confirm compliance to design assumptions with respect to allowable bearing pressure and stability prior to the placement of steel or concrete.
10. Inspection of completed works including all retention and groundwater/stormwater control systems for provision of Form 3 including maintenance and inspection program for Occupation Certificate.

The client and builder should make themselves familiar with the requirements spelled out in this report for inspections during the construction phase. Crozier Geotechnical Consultants cannot provide certification (Form 3) for the Occupation Certificate if it has not been called to site to undertake the required reviews and inspections.

A maintenance program for the life of the development will need to be determined as part of the excavation support/detailed development design prior to the Construction Certificate application and will need to applied to ensure risk levels are as per the estimations of this report. A preliminary program is provided as Table: C within Appendix: 3 of this report.

6. CONCLUSION:

The site inspection and investigations did not identify any signs of previous or impending landslip instability or significant geotechnical hazards within the site or adjacent properties.

The proposed works generally involve an excavation that will be to a similar Base Excavation Level (BEL) and will be located a similar distance to the east and west side boundaries as those approved in the original DA. However, the proposed works involve an excavation that will extend further south and therefore be up to 10.0m depth due to the installation of a support system that can be constructed prior to and during excavation to ensure stability is maintained at all times, even where delays occur and will be independent to the proposed development.

A temporary groundwater/stormwater management system should be designed based on expected levels encountered in previous local site works and this system can then be modified as required based on actual site conditions encountered during excavation to remove groundwater and ensure no detrimental impacts. Whilst subject to prevailing conditions and actual inflow rates such a system will be expected to require collection and storage with infiltration/treatment and pumping to removal at an approved discharge point.

An assessment of the risk posed by the proposed excavation indicates that the works can be undertaken within óTolerableô risk levels and that through the implementation of the recommendations of this report and a suitable excavation support system the risk levels will further reduce. Therefor e the site is considered suitable for the proposed development works.

Form 1 of the NSW Government ï Planning and Development, Geotechnical Policy, Kosciusko Alpine Resorts is attached with this report.

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7. REFERENCES:

1. Australian Geomechanics Society 2007, ñLandslide Risk Assessment and Managementò, Australian Geomechanics Journal Vol. 42, No 1, March 2007.
2. C. W. Fetter 1995, ñApplied Hydrologyò by Prentice Hall. V. Gardiner & R. Dackombe 1983, ñGeomorphological Field Manualò by George Allen & Unwin
3. Australian Standard AS 2870 ï 2011, Residential Slabs and Footings ï Construction
4. Australian Standard AS1170.4 ï 2007, Part 4: Earthquake actions in Australia

Appendix 1

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NOTES RELATING TO THIS REPORT

Introduction

These notes have been provided to amplify the geotechnical report in regard to classification methods, specialist field procedures and certain matters relating to the Discussion and Comments section. Not all, of course, are necessarily relevant to all reports.

Geotechnical reports are based on information gained from limited subsurface test boring and sampling, supplemented by knowledge of local geology and experience. For this reason, they must be regarded as interpretive rather than factual documents, limited to some extent by the scope of information on which they rely.

Description and classification Methods

The methods of description and classification of soils and rocks used in this report are based on Australian Standard 1726, Geotechnical Site Investigation Code. In general, descriptions cover the following properties - strength or density, colour, structure, soil or rock type and inclusions.

Soil types are described according to the predominating particle size, qualified by the grading of other particles present (eg. Sandy clay) on the following bases:

Soil Classification	Particle Size
Clay	less than 0.002 mm
Silt	0.002 to 0.06 mm
Sand	0.06 to 2.00 mm
Gravel	2.00 to 60.00mm

Cohesive soils are classified on the basis of strength either by laboratory testing or engineering examination. The strength terms are defined as follows:

Classification	Undrained Shear Strength kPa
Very soft	Less than 12
Soft	12 - 25
Firm	25 – 50
Stiff	50 – 100
Very stiff	100 - 200
Hard	Greater than 200

Non-cohesive soils are classified on the basis of relative density, generally from the results of standard penetration tests (SPT) or Dutch cone penetrometer tests (CPT) as below:

Relative Density	SPT “N” Value (blows/300mm)	CPT Cone Value (Qc – MPa)
Very loose	less than 5	less than 2
Loose	5 – 10	2 – 5
Medium dense	10 – 30	5 -15
Dense	30 – 50	15 – 25
Very dense	greater than 50	greater than 25

Rock types are classified by their geological names. Where relevant, further information regarding rock classification is given on the following sheet.

Sampling

Sampling is carried out during drilling to allow engineering examination (and laboratory testing where required) of the soil or rock.

Disturbed samples taken during drilling to allow information on colour, type, inclusions and, depending upon the degree of disturbance, some information on strength and structure.

Undisturbed samples are taken by pushing a thin-walled sample tube into the soil and withdrawing a sample of the soil in a relatively undisturbed state. Such samples yield information on structure and strength, and are necessary for laboratory determination of shear strength and compressibility. Undisturbed sampling is generally effective only in cohesive soils.

Drilling Methods

The following is a brief summary of drilling methods currently adopted by the company and some comments on their use and application.

Test Pits – these are excavated with a backhoe or a tracked excavator, allowing close examination of the insitu soils if it is safe to descent into the pit. The depth of penetration is limited to about 3m for a backhoe and up to 6m for an excavator. A potential disadvantage is the disturbance caused by the excavation.

Large Diameter Auger (eg. Pengo) – the hole is advanced by a rotating plate or short spiral auger, generally 300mm or larger in diameter. The cuttings are returned to the surface at intervals (generally of not more than 0.5m) and are disturbed but usually unchanged in moisture content. Identification of soil strata is generally much more reliable than with continuous spiral flight augers, and is usually supplemented by occasional undisturbed tube sampling.

Continuous Sample Drilling – the hole is advanced by pushing a 100mm diameter socket into the ground and withdrawing it at intervals to extrude the sample. This is the most reliable method of drilling soils, since moisture content is unchanged and soil structure, strength, etc. is only marginally affected.

Continuous Spiral Flight Augers – the hole is advanced using 90 – 115mm diameter continuous spiral flight augers which are withdrawn at intervals to allow sampling or insitu testing. This is a relatively economical means of drilling in clays and in sands above the water table. Samples are returned to the surface, or may be collected after withdrawal of the auger flights, but they are very disturbed and may be contaminated. Information from the drilling (as distinct from specific sampling by SPT's or undisturbed samples) is of relatively lower reliability, due to remoulding, contamination or softening of samples by ground water.

Non-core Rotary Drilling - the hole is advanced by a rotary bit, with water being pumped down the drill rods and returned up the annulus, carrying the drill cuttings. Only major changes in stratification can be determined from the cuttings, together with some information from 'feel' and rate of penetration.

Rotary Mud Drilling – similar to rotary drilling, but using drilling mud as a circulating fluid. The mud tends to mask the cuttings and reliable identification is again only possible from separate intact sampling (eg. From SPT).

Continuous Core Drilling – a continuous core sample is obtained using a diamond-tipped core barrel, usually 50mm internal diameter. Provided full core recovery is achieved (which is not always possible in very weak rocks and granular soils), this technique provides a very reliable (but relatively expensive) method of investigation.

Standard Penetration Tests

Standard penetration tests (abbreviated as SPT) are used mainly in non-cohesive soils, but occasionally also in cohesive soils as a means of determining density or strength and also of obtaining a relatively undisturbed sample. The test procedures is described in Australian Standard 1289, "Methods of Testing Soils for Engineering Purposes" – Test 6.3.1.

The test is carried out in a borehole by driving a 50mm diameter split sample tube under the impact of a 63kg hammer with a free fall of 760mm. It is normal for the tube to be driven in three successive 150mm increments and the 'N' value is taken as the number of blows for the last 300mm. In dense sands, very hard clays or weak rock, the full 450mm penetration may not be practicable and the test is discontinued.

The test results are reported in the following form.

- In the case where full penetration is obtained with successive blow counts for each 150mm of say 4, 6 and 7
  as 4, 6, 7 then N = 13
- In the case where the test is discontinued short of full penetration, say after 15 blows for the first 150mm and 30 blows for the next 40mm then as 15, 30/40mm.

The results of the test can be related empirically to the engineering properties of the soil. Occasionally, the test method is used to obtain samples in 50mm diameter thin wall sample tubes in clay. In such circumstances, the test results are shown on the borelogs in brackets.

Cone Penetrometer Testing and Interpretation

Cone penetrometer testing (sometimes referred to as Dutch Cone – abbreviated as CPT) described in this report has been carried out using an electrical friction cone penetrometer. The test is described in Australia Standard 1289, Test 6.4.1.

In tests, a 35mm diameter rod with a cone-tipped end is pushed continually into the soil, the reaction being provided by a specially designed truck or rig which is fitted with an hydraulic ram system. Measurements are made of the end bearing resistance on the cone and the friction resistance on a separte 130mm long sleeve, immediately behind the cone. Transducers in the tip of the assembly are connected buy electrical wires passing through the centre of the push rods to an amplifier and recorder unit mounted on the control truck.

As penetration occurs (at a rate of approximately 20mm per second) their information is plotted on a computer screen and at the end of the test is stored on the computer for later plotting of the results.

The information provided on the plotted results comprises: -

- Cone resistance – the actual end bearing force divided by the cross-sectional area of the cone – expressed in MPa.
- Sleeve friction – the frictional force on the sleeve divided by the surface area – expressed in kPa.
- Friction ratio - the ratio of sleeve friction to cone resistance, expressed in percent.

There are two scales available for measurement of cone resistance. The lower scale (0 – 5 MPa) is used in very soft soils where increased sensitivity is required and is shown in the graphs as a dotted line. The main scale (0 – 50 MPa) is less sensitive and is shown as a full line. The ratios of the sleeve friction to cone resistance will vary with the type of soil encountered, with higher relative friction in clays than in sands. Friction ratios 1% - 2% are commonly encountered in sands and very soft clays rising to 4% - 10% in stiff clays.

In sands, the relationship between cone resistance and SPT value is commonly in the range: -

Q_c (MPa) = (0.4 to 0.6) N blows (blows per 300mm)

In clays, the relationship between undrained shear strength and cone resistance is commonly in the range: -

Q_c = (12 to 18) C_u

Interpretation of CPT values can also be made to allow estimation of modulus or compressibility values to allow calculations of foundation settlements.

Inferred stratification as shown on the attached reports is assessed from the cone and friction traces and from experience and information from nearby boreholes, etc. This information is presented for general guidance, but must be regarded as being to some extent interpretive. The test method provides a continuous profile of engineering properties, and where precise information on soil classification is required, direct drilling and sampling may be preferable.

Dynamic Penetrometers

Dynamic penetrometer tests are carried out by driving a rod into the ground with a falling weight hammer and measuring the blows for successive 150mm increments of penetration. Normally, there is a depth limitation of 1.2m but this may be extended in certain conditions by the use of extension rods.

Two relatively similar tests are used.

- Perth sand penetrometer – a 16mm diameter flattened rod is driven with a 9kg hammer, dropping 600mm (AS1289, Test 6.3.3). The test was developed for testing the density of sands (originating in Perth) and is mainly used in granular soils and filling.
- Cone penetrometer (sometimes known as Scala Penetrometer) – a 16mm rod with a 20mm diameter cone end is driven with a 9kg hammer dropping 510mm (AS 1289, Test 6.3.2). The test was developed initially for pavement sub-grade investigations, and published correlations of the test results with California bearing ratio have been published by various Road Authorities.

Laboratory Testing

Laboratory testing is generally carried out in accordance with Australian Standard 1289 "Methods of Testing Soil for Engineering Purposes". Details of the test procedure used are given on the individual report forms.

Borehole Logs

The bore logs presented herein are an engineering and/or geological interpretation of the subsurface conditions, and their reliability will depend to some extent on frequency of sampling and the method of drilling. Ideally, continuous undisturbed sampling or core drilling will provide the most reliable assessment, but this is not always practicable, or possible to justify on economic grounds. In any case, the boreholes represent only a very small sample of the total subsurface profile.

Interpretation of the information and its application to design and construction should therefore take into account the spacing of boreholes, the frequency of sampling and the possibility of other than 'straight line' variations between the boreholes.

Details of the type and method of sampling are given in the report and the following sample codes are on the borehole logs where applicable:

D	Disturbed Sample	E	Environmental sample	DT	Diatube
B	Bulk Sample	PP	Pocket Penetrometer Test		
U50	50mm Undisturbed Tube Sample	SPT	Standard Penetration Test		
U63	63mm " " " " "	C	Core		

Ground Water

Where ground water levels are measured in boreholes there are several potential problems:

- In low permeability soils, ground water although present, may enter the hole slowly or perhaps not at all during the time it is left open.
- A localised perched water table may lead to an erroneous indication of the true water table.
- Water table levels will vary from time to time with seasons or recent weather changes. They may not be the same at the time of construction as are indicated in the report.
- The use of water or mud as a drilling fluid will mask any ground water inflow. Water has to be blown out of the hole and drilling mud must first be washed out of the hole if water observations are to be made. More reliable measurements can be made by installing standpipes which are read at intervals over several days, or perhaps weeks for low permeability soils. Piezometers, sealed in a particular stratum, may be interference from a perched water table.

Engineering Reports

Engineering reports are prepared by qualified personnel and are based on the information obtained and on current engineering standards of interpretation and analysis. Where the report has been prepared for a specific design proposal (eg. A three-storey building), the information and interpretation may not be relevant if the design proposal is changed (eg. to a twenty-storey building). If this happens, the Company will be pleased to review the report and the sufficiency of the investigation work.

Every care is taken with the report as it relates to interpretation of subsurface condition, discussion of geotechnical aspects and recommendations or suggestions for design and construction. However, the Company cannot always anticipate or assume responsibility for:

- unexpected variations in ground conditions – the potential for this will depend partly on bore spacing and sampling frequency,
- changes in policy or interpretation of policy by statutory authorities,
- the actions of contractors responding to commercial pressures,

If these occur, the Company will be pleased to assist with investigation or advice to resolve the matter.

Site Anomalies

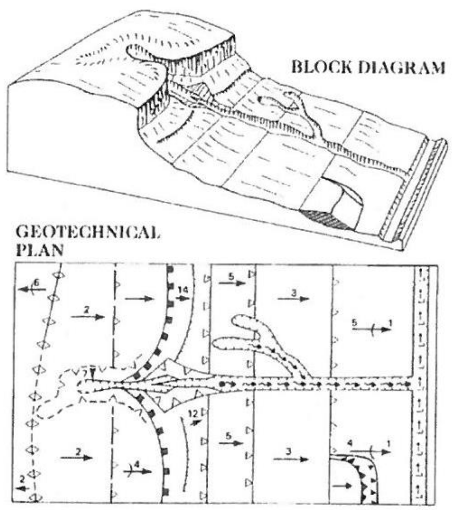
In the event that conditions encountered on site during construction appear to vary from those which were expected from the information contained in the report, the Company requests that it immediately be notified. Most problems are much more readily resolved when conditions are exposed than at some later stage, well after the event.

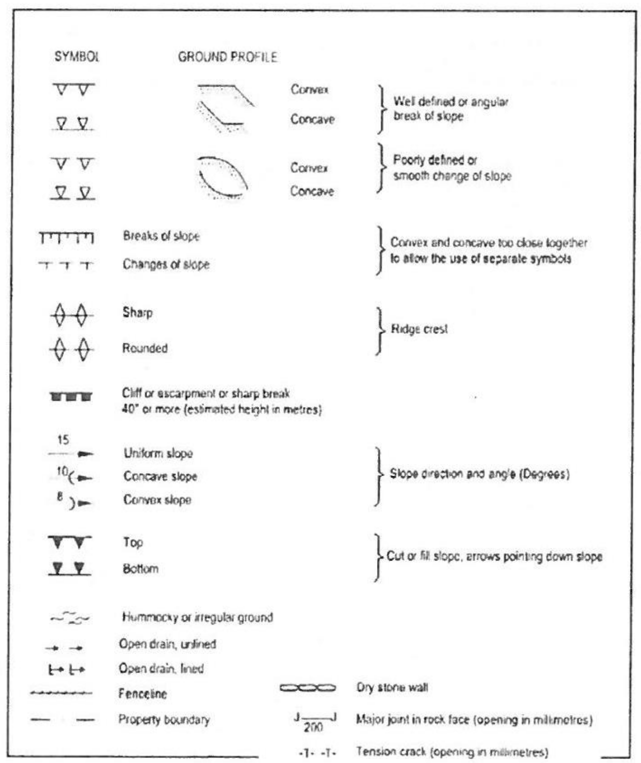
Reproduction of Information for Contractual Purposes

Attention is drawn to the document "Guidelines for the Provision of Geotechnical Information in Tender Documents", published by the Institution of Engineers Australia. Where information obtained from this investigation is provided for tendering purposes, it is recommended that all information, including the written report and discussion, be made available. In circumstances where the discussion or comments section is not relevant to the contractual situation, it may be appropriate to prepare a special ally edited document. The Company would be pleased to assist in this regard and/or to make additional report copies available for contract purposes at a nominal charge.

Site Inspection

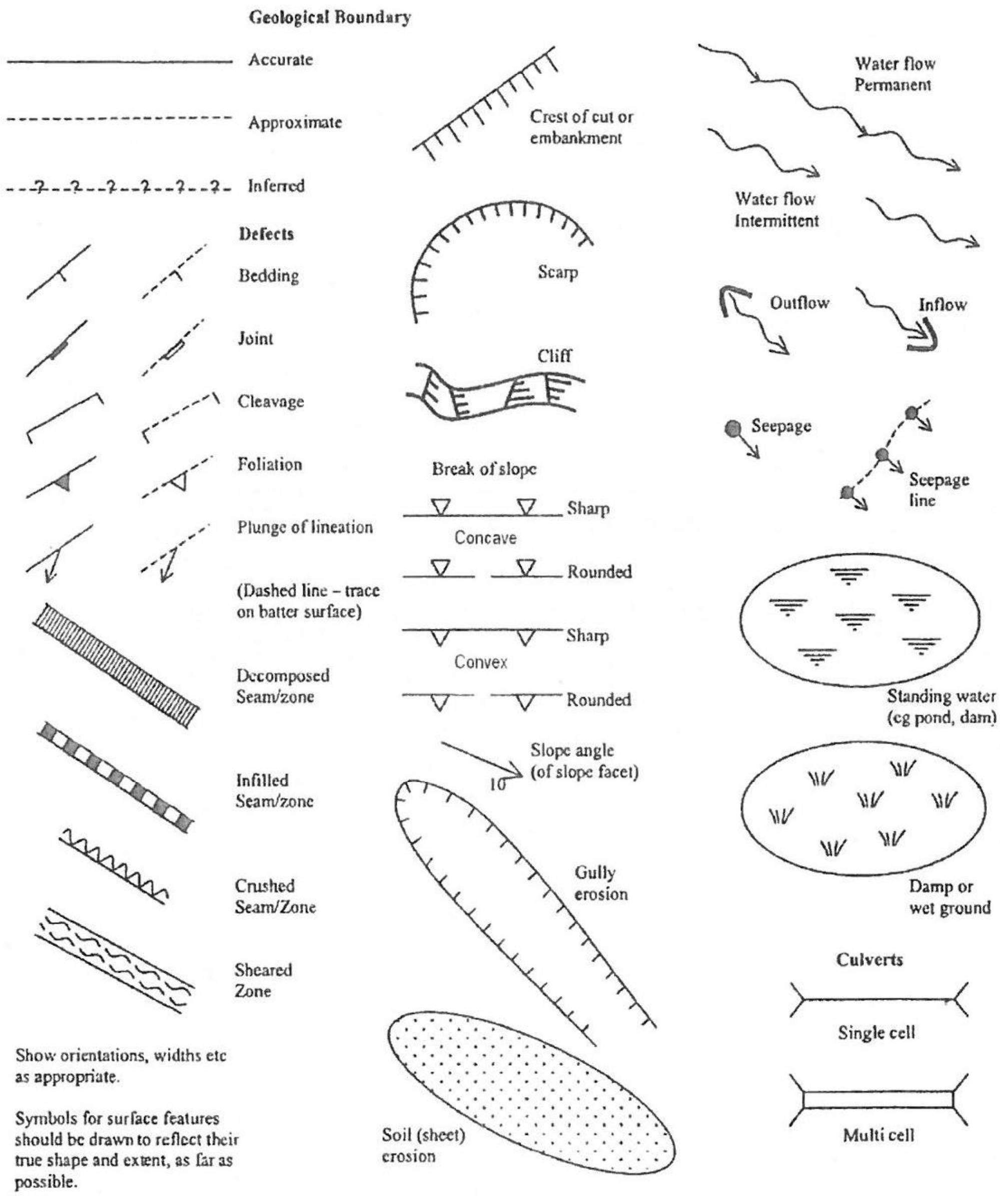
The Company will always be pleased to provide engineering inspection services for geotechnical aspects of work to which this report is related. This could range from a site visit to confirm that conditions exposed are as expected, to full time engineering presence on site.





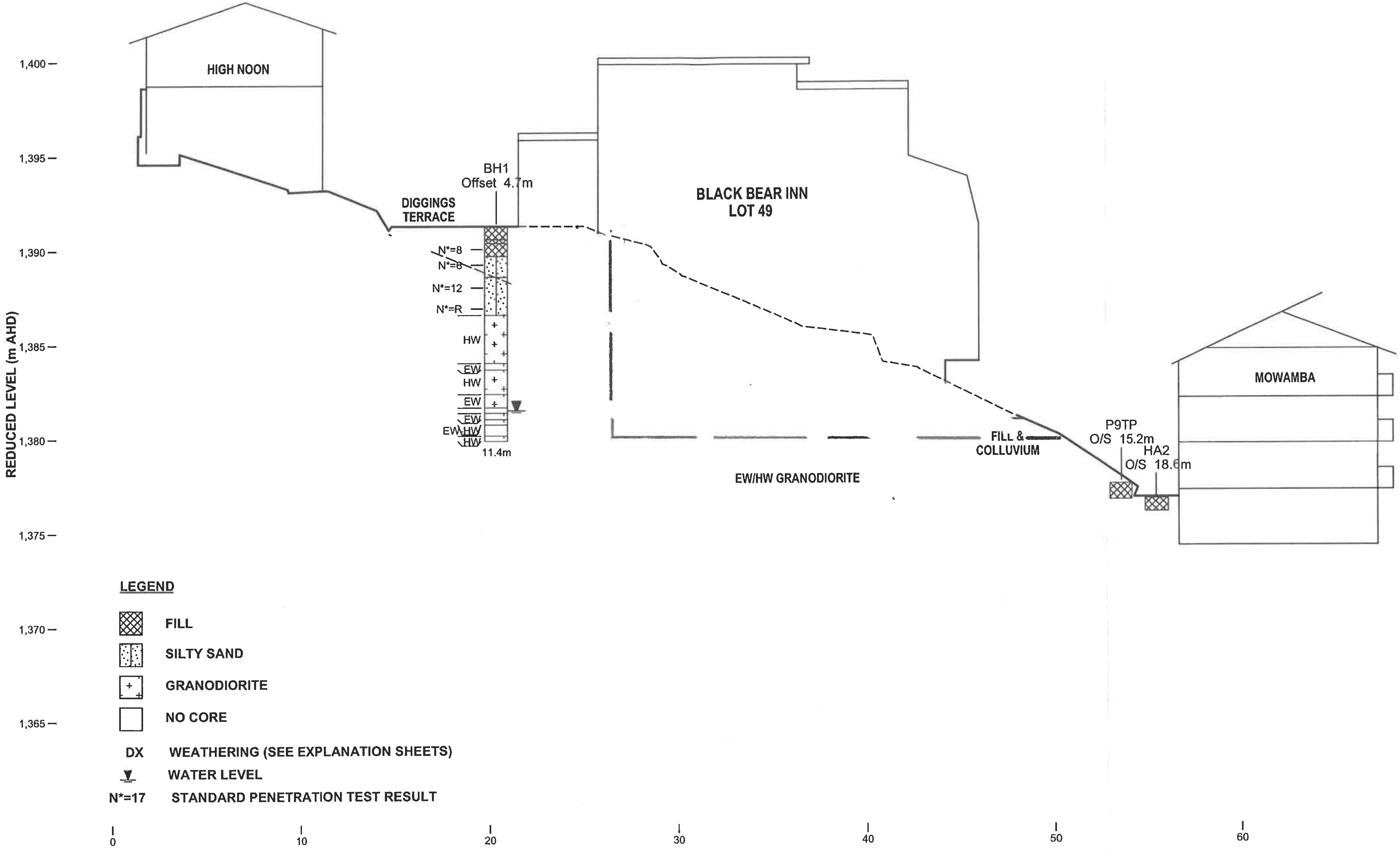
Example of Mapping Symbols
(after V Gardiner & R V Dackombe (1983).Geomorphological Field Manual. George Allen & Unwin).

APPENDIX E - GEOLOGICAL AND GEOMORPHOLOGICAL MAPPING SYMBOLS AND TERMINOLOGY



Examples of Mapping Symbols (after Guide to Slope Risk Analysis Version 3.1 November 2001, Roads and Traffic Authority of New South Wales).

Appendix 2



revision	description	drawn	approved	date

2 0 10
Horizontal Scale (metres)

2 0 10
Vertical Scale (metres)

drawn	PM/SW
approved	PM
date	14/5/07
scale	AS SHOWN
original size	A3

coffey geotechnics
SPECIALISTS MANAGING THE EARTH

client:	ALEX POPOV & ASSOCIATES
project:	BLACK BEAR INN LOT 49 - DIGGINGS TERRACE THREDBO ALPINE VILLAGE
title:	GEOTECHNICAL SECTION A-A'
project no: GEOTLCOV23158AA	figure no:

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Appendix 3

TABLE : A

L

A A			**L**					**E**		**L**
A	Landslip (earth slide <5m³) from soils from Level 2 excavation		No indications of excess creep movement, surface erosion or groundwater seepage in area at present. Soils and weathered rock expected for full height of excavation which is max. 3.50m depth, significant seepage unlikely	a) Bulk excavation located 6.50m from boundary, rare impact, may impact 20% of road at worst b) Building located 3.60m from ≤ 3.5m deep excavation, unlikely impact, impact <10% at worst c) Driveway located 4.1m from 3.50m deep excavation, unlikely impact, may impact 50% of driveway		a) Person on road, pedestrian 1hrs/day avge. b) Person in bedroom 10hr/day avge. c) Person in vehicle 0.5hrs/day avge.	a) 1 person b) 2 persons c) 2 persons	a) Unlikely to not evacuate b) Likely to not evacuate c) Likely to not evacuate	a) Person on road, not buried b) Person in building, damage only c) Person in vehicle, not buried	
		a) Diggings Terrace	0.0001	0.01	0.20	0.0417	1	0.25	0.20	**E**
		b) Candelight Lodge building	0.0001	0.25	0.10	0.4167	2	0.75	0.05	**E**
		c) Candlelight Lodge - driveway	0.0001	0.25	0.50	0.0208	2	0.75	0.10	**E**
B	Landslip (earth/debris slide 10 - 15m³) within deep Level 00 - Level 1 - Level 2 excavation		No indications of excess creep movement. Soils and weathered rock expected for full height of excavation of up to 8.0m, groundwater seepage likely in lower portions, full height of excavation not unsupported at any time	a) Bulk excavation located 6.50m from boundary, possible impact, may impact 50% of road at worst b) Building located 8.50m from 8.0m deep excavation, unlikely impact, impact part of 1 bedroom c) Driveway located 6.0m from 8.0m deep excavation, possible impact, may impact 50% of driveway d) Building located 4.2m from 6.0m deep excavation, possible impact, impact most of 1 bedroom		a) Person on road, pedestrian 1hrs/day avge. b) Person in bedroom 10hr/day avge. c) Person in vehicle 0.5hrs/day avge. d) Person in bedroom 10hr/day avge	a) 1 person b) 2 persons c) 2 persons d) 2 persons	a) Unlikely to not evacuate b) Likely to not evacuate c) Likely to not evacuate d) Likely to not evacuate	a) Person on road, buried b) Person in building, damage only c) Person in vehicle, buried d) Person in building, damage, unlikely buried	
		a) Diggings Terrace	0.0001	0.50	0.50	0.0417	1	0.25	1.00	**E**
		b) Candelight Lodge building	0.0001	0.25	0.10	0.4167	2	0.75	0.10	**E**
		c) Candlelight Lodge - driveway	0.0001	0.50	0.50	0.0208	2	0.75	1.00	**E**
		d) Sashas Building	0.0001	0.50	0.75	0.4167	2	0.75	0.25	**E**

* hazards considered for excavation, prior to completion of staged support system (i.e. staged anchor and shotcrete). Soldier pile support prior to excavation reduces Likelihood further
* staged excavation and support system expected to involve excavations of up to 3.0m depth that are unsupported for up to 7 days at any one time
* assessment is for scale of landslip stated, smaller landslips may have higher Likelihood but will not impact adjacent boundaries or neighbouring structures
* Spatial Impact - Probaility of Impact refers to slide impacting structure/area expressed as a % (i.e. 1.00 = 100% probability of slide impacting area if slide occurs).
  Impacted refers to expected % of area/structure damaged if slide impacts (i.e. small, slow earth slide will damage small portion of structure such as one bedroom (say 5%), where as large boulder roll may damage/destroy >50%)
* neighbouring buidlings considered for impact of slide to bedroom unless specified, due to high occupancy and lower potential for evacuation.
* considered for person most at risk, where multiple people occupy area then increased risk levels assessed against ALARP criteria
* for excavation induced landslip then considered for adjacent premises/buildings founded off shallow footings, unless indicated
* evacuation scale from Almost Certain to<u>not</u> evacuate (1.0), Likely (0.75), Possible (0.5), Unlikely (0.25), Rare to not evacuate (0.01). Based on likelihood of person knowing of landslide and completely evacuating area prior to landslide impact.
* vulnerability assessed using Appendix F - AGS Practice Note Guidelines for Landslide Risk Management 2007

TABLE : B

L

A A			L		C		
A	Landslip (earth slide <5m³) from soils from Level 2 excavation	a) Diggings Terrace	Rare	The event is conceivable but only under exceptional circumstances over the design life.	Minor	Limited Damage to part of structure or site requires some stabilisation or INSIGNIFICANT damage to neighbouring properties.	Very Low
		b) Candelight Lodge building	Rare	The event is conceivable but only under exceptional circumstances over the design life.	Minor	Limited Damage to part of structure or site requires some stabilisation or INSIGNIFICANT damage to neighbouring properties.	Very Low
		c) Candlelight Lodge - driveway	Rare	The event is conceivable but only under exceptional circumstances over the design life.	Minor	Limited Damage to part of structure or site requires some stabilisation or INSIGNIFICANT damage to neighbouring properties.	Very Low
B	Landslip (earth/debris slide 10 - 15m³) within deep Level 00 - Level 1 - Level 2 excavation	a) Diggings Terrace	Unlikely	The event might occur under very adverse circumstances over the design life.	Minor	Limited Damage to part of structure or site requires some stabilisation or INSIGNIFICANT damage to neighbouring properties.	Low
		b) Candelight Lodge building	Unlikely	The event might occur under very adverse circumstances over the design life.	Minor	Limited Damage to part of structure or site requires some stabilisation or INSIGNIFICANT damage to neighbouring properties.	Low
		c) Candlelight Lodge - driveway	Unlikely	The event might occur under very adverse circumstances over the design life.	Minor	Limited Damage to part of structure or site requires some stabilisation or INSIGNIFICANT damage to neighbouring properties.	Low
		d) Sashas Building	Unlikely	The event might occur under very adverse circumstances over the design life.	Medium	Moderate damage to some of structure or significant part of site, requires large stabilising works or MINOR damage to neighbouring property.	Low

* hazards considered for unsupported excavation, prior to installation of support system (i.e. staged excavation and support system). Soldier pile support prior to excavation reduces Likelihood further
* qualitative expression of likelihood incorporates both frequency analysis estimate and spatial impact probability estimate as per AGS guidelines.
* qualitative measures of consequences to property assessed per Appendix C in AGS Guidelines for Landslide Risk Management.
* Indicative cost of damage expressed as cost of site development with respect to consequence values: Catastrophic : 200%, Major: 60%, Medium: 20%, Minor: 5%, Insignificant: 0.5%.

TABLE: C

Recommended Maintenance and Inspection Program

Structure	Maintenance/ Inspection Item	Frequency
Stormwater drains.	Owner to inspect to ensure that the open drains, and pipes are free of debris & sediment build-up. Clear surface grates and litter. Owner to check and flush retaining wall drainage pipes/systems	Every year during spring thaw or following each major rainfall event. Every 7 years or where dampness/moisture issues
Retaining Walls. or remedial measures	Owner to inspect walls for deveation from as constructed condition and repair/replace.	Every two years or following major rainfall event.
Large Trees on or adjacent to site	Arborist to check condition of trees and remove as required. Where tree within steep slopes (>18°) or adjacent to structures requires geotechincal inspection prior to removal	Every five years
Slope Stability	Geotechnical Engineering Consultant to check on site stability and maintenance records	Five years after construction is completed.

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Appendix 4

APPENDIX A

DEFINITION OF TERMS

INTERNATIONAL UNION OF GEOLOGICAL SCIENCES WORKING GROUP ON LANDSLIDES, COMMITTEE ON RISK ASSESSMENT

Risk – A measure of the probability and severity of an adverse effect to health, property or the environment. Risk is often estimated by the product of probability x consequences. However, a more general interpretation of risk involves a comparison of the probability and consequences in a non-product form.

Hazard – A condition with the potential for causing an undesirable consequence (*the landslide*). The description of landslide hazard should include the location, volume (or area), classification and velocity of the potential landslides and any resultant detached material, and the likelihood of their occurrence within a given period of time.

Elements at Risk – Meaning the population, buildings and engineering works, economic activities, public services utilities, infrastructure and environmental features in the area potentially affected by landslides.

Probability – The likelihood of a specific outcome, measured by the ratio of specific outcomes to the total number of possible outcomes. Probability is expressed as a number between 0 and 1, with 0 indicating an impossible outcome, and 1 indicating that an outcome is certain.

Frequency – A measure of likelihood expressed as the number of occurrences of an event in a given time. See also Likelihood and Probability.

Likelihood – used as a qualitative description of probability or frequency.

Temporal Probability – The probability that the element at risk is in the area affected by the landsliding, at the time of the landslide.

Vulnerability – The degree of loss to a given element or set of elements within the area affected by the landslide hazard. It is expressed on a scale of 0 (no loss) to 1 (total loss). For property, the loss will be the value of the damage relative to the value of the property; for persons, it will be the probability that a particular life (the element at risk) will be lost, given the person(s) is affected by the landslide.

Consequence – The outcomes or potential outcomes arising from the occurrence of a landslide expressed qualitatively or quantitatively, in terms of loss, disadvantage or gain, damage, injury or loss of life.

Risk Analysis – The use of available information to estimate the risk to individuals or populations, property, or the environment, from hazards. Risk analyses generally contain the following steps: scope definition, hazard identification, and risk estimation.

Risk Estimation – The process used to produce a measure of the level of health, property, or environmental risks being analysed. Risk estimation contains the following steps: frequency analysis, consequence analysis, and their integration.

Risk Evaluation – The stage at which values and judgements enter the decision process, explicitly or implicitly, by including consideration of the importance of the estimated risks and the associated social, environmental, and economic consequences, in order to identify a range of alternatives for managing the risks.

Risk Assessment – The process of risk analysis and risk evaluation.

Risk Control or Risk Treatment – The process of decision making for managing risk, and the implementation, or enforcement of risk mitigation measures and the re-evaluation of its effectiveness from time to time, using the results of risk assessment as one input.

Risk Management – The complete process of risk assessment and risk control (*or risk treatment*).

Individual Risk – The risk of fatality or injury to any identifiable (named) individual who lives within the zone impacted by the landslide; or who follows a particular pattern of life that might subject him or her to the consequences of the landslide.

Societal Risk – The risk of multiple fatalities or injuries in society as a whole: one where society would have to carry the burden of a landslide causing a number of deaths, injuries, financial, environmental, and other losses.

Acceptable Risk – A risk for which, for the purposes of life or work, we are prepared to accept as it is with no regard to its management. Society does not generally consider expenditure in further reducing such risks justifiable.

Tolerable Risk – A risk that society is willing to live with so as to secure certain net benefits in the confidence that it is being properly controlled, kept under review and further reduced as and when possible.

In some situations risk may be tolerated because the individuals at risk cannot afford to reduce risk even though they recognise it is not properly controlled.

Landslide Intensity – A set of spatially distributed parameters related to the destructive power of a landslide. The parameters may be described quantitatively or qualitatively and may include maximum movement velocity, total displacement, differential displacement, depth of the moving mass, peak discharge per unit width, kinetic energy per unit area.

Note: Reference should also be made to Figure 1 which shows the inter-relationship of many of these terms and the relevant portion of Landslide Risk Management.

APPENDIX C: LANDSLIDE RISK ASSESSMENT

QUALITATIVE TERMINOLOGY FOR USE IN ASSESSING RISK TO PROPERTY

QUALITATIVE MEASURES OF LIKELIHOOD

Approximate Annual Probability		Implied Indicative Landslide Recurrence Interval		Description	Descriptor	Level
Indicative Value	**Notional Boundary**					
10^{-1}	$5x10^{-2}$	10 years	20 years	The event is expected to occur over the design life.	ALMOST CERTAIN	A
10^{-2}	$5x10^{-3}$	100 years	200 years	The event will probably occur under adverse conditions over the design life.	LIKELY	B
10^{-3}	$5x10^{-4}$	1000 years	2000 years	The event could occur under adverse conditions over the design life.	POSSIBLE	C
10^{-4}	$5x10^{-5}$	10,000 years	20,000 years	The event might occur under very adverse circumstances over the design life.	UNLIKELY	D
10^{-5}	$5x10^{-6}$	100,000 years	200,000 years	The event is conceivable but only under exceptional circumstances over the design life.	RARE	E
10^{-6}		1,000,000 years		The event is inconceivable or fanciful over the design life.	BARELY CREDIBLE	F

Note: (1) The table should be used from left to right; use Approximate Annual Probability or Description to assign Descriptor, not *vice versa*.

QUALITATIVE MEASURES OF CONSEQUENCES TO PROPERTY

Approximate Cost of Damage		Description	Descriptor	Level
Indicative Value	**Notional Boundary**			
200%	100%	Structure(s) completely destroyed and/or large scale damage requiring major engineering works for stabilisation. Could cause at least one adjacent property major consequence damage.	CATASTROPHIC	1
60%	40%	Extensive damage to most of structure, and/or extending beyond site boundaries requiring significant stabilisation works. Could cause at least one adjacent property medium consequence damage.	MAJOR	2
20%	10%	Moderate damage to some of structure, and/or significant part of site requiring large stabilisation works. Could cause at least one adjacent property minor consequence damage.	MEDIUM	3
5%	1%	Limited damage to part of structure, and/or part of site requiring some reinstatement stabilisation works.	MINOR	4
0.5%		Little damage. (Note for high probability event (Almost Certain), this category may be subdivided at a notional boundary of 0.1%. See Risk Matrix.)	INSIGNIFICANT	5

Notes: (2) The Approximate Cost of Damage is expressed as a percentage of market value, being the cost of the improved value of the unaffected property which includes the land plus the unaffected structures.

(3) The Approximate Cost is to be an estimate of the direct cost of the damage, such as the cost of reinstatement of the damaged portion of the property (land plus structures), stabilisation works required to render the site to tolerable risk level for the landslide which has occurred and professional design fees, and consequential costs such as legal fees, temporary accommodation. It does not include additional stabilisation works to address other landslides which may affect the property.

(4) The table should be used from left to right; use Approximate Cost of Damage or Description to assign Descriptor, not vice versa

APPENDIX C: – QUALITATIVE TERMINOLOGY FOR USE IN ASSESSING RISK TO PROPERTY (CONTINUED)

QUALITATIVE RISK ANALYSIS MATRIX – LEVEL OF RISK TO PROPERTY

LIKELIHOOD		CONSEQUENCES TO PROPERTY (With Indicative Approximate Cost of Damage)				
	Indicative Value of Approximate Annual Probability	**1: CATASTROPHIC 200%**	**2: MAJOR 60%**	**3: MEDIUM 20%**	**4: MINOR 5%**	**5: INSIGNIFICANT 0.5%**
A – ALMOST CERTAIN	10^{-1}	VH	VH	VH	H	M or **L** (5)
B - LIKELY	10^{-2}	VH	VH	H	M	**L**
C - POSSIBLE	10^{-3}	VH	H	M	M	**VL**
D - UNLIKELY	10^{-4}	H	M	**L**	**L**	**VL**
E - RARE	10^{-5}	M	**L**	**L**	**VL**	**VL**
F - BARELY CREDIBLE	10^{-6}	**L**	**VL**	**VL**	**VL**	**VL**

Notes: (5) For Cell A5, may be subdivided such that a consequence of less than 0.1% is Low Risk.

(6) When considering a risk assessment it must be clearly stated whether it is for existing conditions or with risk control measures which may not be implemented at the current time.

RISK LEVEL IMPLICATIONS

Risk Level		Example Implications (7)
VH	VERY HIGH RISK	Unacceptable without treatment. Extensive detailed investigation and research, planning and implementation of treatment options essential to reduce risk to Low; may be too expensive and not practical. Work likely to cost more than value of the property.
H	HIGH RISK	Unacceptable without treatment. Detailed investigation, planning and implementation of treatment options required to reduce risk to Low. Work would cost a substantial sum in relation to the value of the property.
M	MODERATE RISK	May be tolerated in certain circumstances (subject to regulator's approval) but requires investigation, planning and implementation of treatment options to reduce the risk to Low. Treatment options to reduce to Low risk should be implemented as soon as practicable.
L	LOW RISK	Usually acceptable to regulators. Where treatment has been required to reduce the risk to this level, ongoing maintenance is required.
VL	VERY LOW RISK	Acceptable. Manage by normal slope maintenance procedures.

Note: (7) The implications for a particular situation are to be determined by all parties to the risk assessment and may depend on the nature of the property at risk; these are only given as a general guide.

APPENDIX B – PMI ENGINEERS EXCAVATION & FOUNDATION DRAWINGS

REGULATED DESIGN RECORD	REV	DATE	DESCRIPTION	DP FULL NAME	REG NO
PROJECT ADDRESS: 30 DIGGINGS TERRACE, THREDBO	1	29.11.2021	ISSUED FOR CC2	THOMAS WILLIAMS	PRE0001122
PROJECT TITLE: BLACK BEAR INN					
CONSENT NUMBER:					

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ISSUE:

DRAWING TITLE: **STRUCTURAL NOTES**
JOB NUMBER: **PMI-2021-053**
DRAWING NUMBER: **S02-A** REVISION: **1**
SCALE AT B1: 1 : 10

CLIENT: HIDALI PTY LTD

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ALL SETOUT TO ARCHITECT'S DRAWINGS. DIMENSIONS TO BE VERIFIED WITH ARCHITECT AND BUILDER BEFORE COMMENCING SHOP DRAWINGS OR SITE WORK. ENGINEER ACCEPTS NO RESPONSIBILITY FOR THE USABILITY, COMPLETENESS OR SCALE OF DRAWINGS TRANSFERRED ELECTRONICALLY.

STRUCTURAL NOTES BLACK BEAR INN

GENERAL

G1. THESE DRAWINGS SHALL BE READ IN CONJUNCTION WITH SPECIFICATIONS AND OTHER CONSULTANT'S DRAWINGS.
G2. THE WEATHER PROOFING OF THE BUILDING IS THE ARCHITECT'S/BUILDER'S RESPONSIBILITY. THIS INCLUDES (BUT IS NOT LIMITED TO) THE SPECIFICATION AND FIXING DETAILS OF CLADDINGS, SHEETING, FLASHING, MEMBRANES, STEPS, SETDOWNS & RECESSES.
G3. ALL DISCREPANCIES SHALL BE REFERRED TO THE (PROJECT MANAGER) AND RESOLVED BEFORE PROCEEDING WITH THE WORK.
G4. ALL DIMENSIONS SHOWN SHALL BE VERIFIED BY THE BUILDER ON SITE. THESE STRUCTURAL DRAWINGS SHALL NOT BE SCALED FOR DIMENSIONS. THE RL's SHOWN ON THESE DRAWINGS ARE APPROXIMATE AND ARE FOR THE SOLE PURPOSE OF ASSISTING THE STRUCTURAL DOCUMENTATION. THEY ARE NOT TO BE USED FOR CONSTRUCTION PURPOSES. REFER TO ARCHITECTURAL DRAWINGS FOR CONFIRMATION OF ALL RL's, ALL LEVELS ARE IN METRES (m) AND DIMENSIONS ARE IN MILLIMETRES (mm)
G5. ALL WORKMANSHIP, TESTING, MATERIALS AND SUPERVISION ARE TO BE IN ACCORDANCE WITH THESE SPECIFICATIONS, THE WORK HEALTH AND SAFETY ACT 2011. ENFORCED BY THE WORKCOVER AUTHORITY AND CURRENT RELEVANT AUSTRALIAN STANDARDS.
G6. PROPRIETARY ITEMS SPECIFIED SHALL BE INSTALLED IN ACCORDANCE WITH THE MANUFACTURER'S WRITTEN RECOMMENDATIONS. DO NOT VARY SPECIFIED PROPRIETARY PRODUCTS WITHOUT WRITTEN APPROVAL FROM THE ENGINEER.
G7. THESE DRAWINGS AND ISSUED WRITTEN INSTRUCTIONS DURING THE COURSE OF THE CONTRACT DEPICT THE COMPLETE STRUCTURE. THEY DO NOT DESCRIBE A WORK METHOD. THE ARRANGEMENT, DESIGN AND INSTALLATION OF TEMPORARY WORKS REMAINS THE RESPONSIBILITY OF THE CONTRACTOR.
G8. THE DETERMINATION OF A SAFE WORK METHOD REMAINS THE RESPONSIBILITY OF THE CONTRACTOR. ANY ELEMENT WHICH POSES AN UNACCEPTABLE LEVEL OF SAFETY RISK TO CONSTRUCT SHALL BE REFERRED TO THE STRUCTURAL ENGINEER. TEMPORARY BRACING AND SUPPORT OF STRUCTURE IS THE RESPONSIBILITY OF THE CONTRACTOR AND SHALL BE MAINTAINED DURING ALL STAGES OF CONSTRUCTION.
G9. NOTES ON ANY DRAWING APPLY TO ALL DRAWINGS IN THE SET UNLESS NOTED OTHERWISE
G10. ALL ARCHITECTURAL FITMENTS SUCH AS GLAZING, PARTITIONS, CEILINGS ETC. SHOULD ALLOW FOR THE SHORT AND LONG TERM MOVEMENT OF STRUCTURAL ELEMENTS. FOR BEAMS AND SLABS SPANNING LESS THAN 8m AN ALLOWANCE OF AT LEAST 20mm SHOULD BE MADE (CONSULT ENGINEER WHERE SPANS EXCEED 8m).
G11. THE BUILDER SHALL PROVIDE CERTIFICATION ON ANY DESIGN AND CONSTRUCT COMPONENT BY A CHARTERED PROFESSIONAL ENGINEER (NPER).
G12. THE CONTRACTOR SHALL BE RESPONSIBLE FOR THE LOCATION OF ALL SERVICES IN THE VICINITY OF THE WORKS. ANY SERVICES SHOWN ARE PROVIDED FOR INFORMATION ONLY. THE CONTRACTOR SHALL CONFIRM THE LOCATION OF ALL SERVICES PRIOR TO COMMENCING AND SHALL BE RESPONSIBLE FOR THE REPAIR OF ANY DAMAGE CAUSED TO SERVICES, AS WELL AS ANY LOSS INCURRED AS A RESULT OF THE DAMAGE TO ANY SERVICE.
G13. THE STRUCTURAL COMPONENTS DETAILED ON THESE STRUCTURAL DRAWINGS ARE JOB SPECIFIC AND HAVE BEEN DESIGNED IN ACCORDANCE WITH THE RELEVANT AUSTRALIAN STANDARDS AND BUILDING CODE OF AUSTRALIA FOR THE FOLLOWING FIRE RATINGS, WIND LOADS, FLOOR USAGE AND EARTHQUAKE LOADS.

WIND LOADS:

- REGION	=	A
- ANNUAL PROBABILITY OF EXCEEDANCE	=	0.02
- TERRAIN CATEGORY	=	2.5
- SITE WIND SPEED	=	45 m/s

FLOOR LIVE LOADS:

- GENERAL	=	1.5 kPa
- STORES	=	5.0 kPa
- GARAGE	=	2.5 kPa
- STAIRS	=	2.0 kPa
- BALCONY	=	2.0 kPa

ROOF LIVE LOADS:

- ROOF	=	0.25 kPa

SNOW LOADS:

- ROOF	=	[4.40] kPa
- GROUND	=	[2.30] kPa
- PROBABILITY FACTOR	=	1 (SERV) 1.5 (STR)

BUSHFIRES : = DESIGN STRUCTURE TO COMPLY WITH THE REQUIREMENTS OF AS3959-2009.

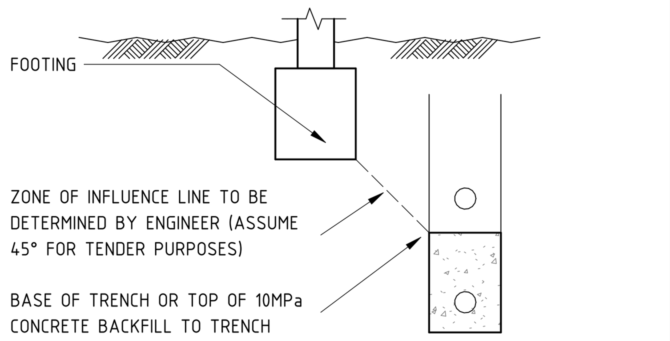
G14. THE METHOD OF CONSTRUCTION AND THE MAINTENANCE OF SAFETY DURING CONSTRUCTION IS THE RESPONSIBILITY OF THE BUILDER. IF ANY STRUCTURAL ELEMENT PRESENTS DIFFICULTY IN RESPECT TO SAFETY THE MATTER SHALL BE REFERRED TO PMI ENGINEERS FOR RESOLUTION BEFORE PROCEEDING WITH THE WORK.
G15. NO CHANGES IN ANY STRUCTURAL ELEMENT SHALL BE MADE WITHOUT WRITTEN APPROVAL FROM PMI ENGINEERS. IF THERE IS A DISCREPANCY THEN FOR TENDER PURPOSES ALLOW FOR THE MOST EXPENSIVE OPTION. PMI ENGINEERS SHALL BE CONTACTED TO CONFIRM PRIOR TO CONSTRUCTION.
G16. CONSTRUCTION USING THESE DRAWINGS SHALL NOT COMMENCE UNTIL A CONSTRUCTION CERTIFICATE HAS BEEN ISSUED AND ONLY IF THE DRAWINGS ARE DESIGNATED "ISSUED FOR CONSTRUCTION".
G17. PMI ENGINEERS ACCEPTS NO RESPONSIBILITY FOR ANY WORK NOT INSPECTED OR NOT APPROVED BY PMI ENGINEERS DURING CONSTRUCTION.

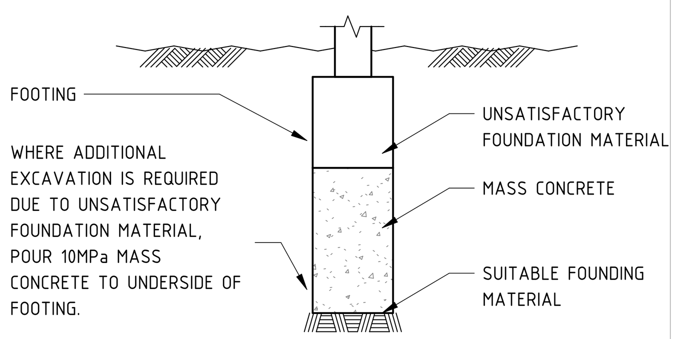
FOUNDATIONS

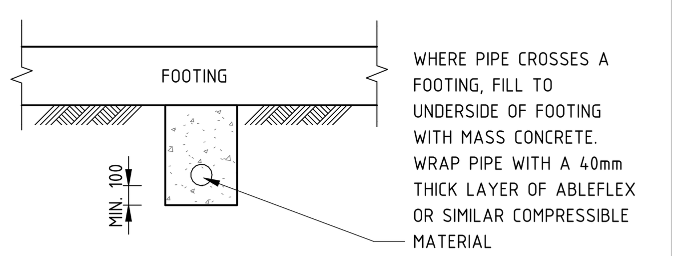
F1. ASSUMED ALLOWABLE BEARING CAPACITY:

- PAD FOOTINGS	=	[500] kPa
- STRIP FOOTINGS	=	[500] kPa
- SLABS ON GROUND	=	[500] kPa
- BORED PIERS	=	[1500]kPa END BEARING [150] kPa SKIN FRICTION

F2. A GEOTECHNICAL REPORT HAS BEEN CARRIED OUT REFER TO ALLIANCE REPORT 13526-GR-1-1 REV A DATED 15th SEPTEMBER,
F3. THE SLAB AND FOOTINGS HAVE BEEN DESIGNED IN ACCORDANCE WITH AS2870-2011 FOR CLASS [A] SITE. A SUITABLY QUALIFIED GEOTECHNICAL ENGINEER TO BE CONTACTED DURING EXCAVATION TO CONFIRM THE SITE CLASSIFICATION.
F4. THE CONTRACTOR SHALL ALLOW TO ENGAGE A QUALIFIED (NPER) GEOTECHNICAL ENGINEER TO APPROVE THE FOUNDATION MATERIAL. OBTAIN GEOTECHNICAL ENGINEERS APPROVAL AND SUBMIT CERTIFICATE IN WRITING TO PMI ENGINEERS PRIOR TO CONCRETING FOUNDATIONS.
F5. ENSURE STABILITY OF ADJACENT BUILDINGS AND PATHS IS MAINTAINED DURING ALL STAGES OF CONSTRUCTION.
F6. DO NOT ALLOW EXCAVATED MATERIAL TO BE STOCKPILED WITHIN 1500mm OF FOOTING TRENCHES OR PITS. NO EARTH OR DETRITUS IS TO FALL INTO THE FOOTING TRENCHES BEFORE OR DURING CONCRETE PLACEMENT.
F7. THE UNDERSIDE OF FOUNDATIONS SHALL CONFORM TO THE FOLLOWING REGARDLESS OF NOMINATED LEVELS:







F8. FOOTINGS SHALL BE CENTRALLY LOCATED UNDER WALLS AND COLUMNS UNLESS NOTED OTHERWISE ON THE STRUCTURAL DRAWINGS.
F9. FOOTINGS SHALL BE EXCAVATED TO THE DETAILED DEPTH AND WIDTH. FOOTINGS SHALL BE INSPECTED AND FILLED WITH CONCRETE AS SOON AS POSSIBLE TO AVOID EITHER SOFTENING OF THE FOUNDATION MATERIAL OR DRYING OUT BY EXPOSURE.
F10. THE BASE OF ALL PIER HOLES SHALL BE FREE OF WATER AND CLEANED OF LOOSE MATERIAL OR DEBRIS PRIOR TO PLACEMENT OF CONCRETE. ALLOW TO PROVIDE TEMPORARY LINERS AS DEEMED NECESSARY.

CONSTRUCTION PHASE SERVICES - WITNESS POINTS

WP1. OBTAIN PMI ENGINEERS WRITTEN INSTRUCTION AT THE FOLLOWING HOLD POINTS:
- PREPARATION OF FOUNDING MATERIAL, INCLUDING PIER BORE HOLES.
- REINFORCEMENT PRIOR TO PLACEMENT OF CONCRETE or COREFILLING OF BLOCKWORK.
- STEEL AND TIMBER FRAME INSPECTION PRIOR TO SHEETING.
WP2. PROVIDE MINIMUM 48 HOURS NOTICE FOR ANY REQUIRED INSPECTIONS.

TEMPORARY WORKS

TW1. THESE DRAWINGS DEPICT THE "PERMANENT" STRUCTURE, TEMPORARY WORKS REMAIN THE RESPONSIBILITY OF THE CONTRACTOR.
TW2. BUILDER MUST ENGAGE (NPER) QUALIFIED STRUCTURAL ENGINEER FOR THE DESIGN OF ALL TEMPORARY WORKS NECESSARY TO SAFELY ERECT THIS STRUCTURE. AS A MINIMUM THE FOLLOWING WORKS REQUIRE ATTENTION;
- FORMWORK / TEMPORARY PROPPING / NEEDLE BEAMS / SCAFFOLDING / UNDERPINNING
TW3. BUILDER SHALL CONTACT PMI ENGINEERS IF THEY CONSIDER ANY PART OF THIS STRUCTURE IS UNSAFE TO ERECT

STEELWORK

S1. FABRICATE AND ERECT STRUCTURAL STEELWORK IN ACCORDANCE WITH AS4100-1998.
S2. PROVIDE HOLES, CLEATS AND FIXING FOR LIGHT STEEL/TIMBER FRAMING, FINISHES, ETC. SHOWN ON ARCHITECTURAL DRAWINGS.
S3. THESE DRAWINGS HAVE BEEN PREPARED TO INDICATE THE STRUCTURAL INTENT. THE SHOP DETAILER IS TO USE THESE DRAWINGS AS A BASIS FOR DIMENSIONAL COORDINATION WITH OTHER CONSULTANT'S DRAWINGS AND IS TO PREPARE DETAILED SHOP DRAWINGS. WHERE NECESSARY, THE SHOP DETAILER IS TO MAKE ASSUMPTIONS AND SUBMIT TO PMI ENGINEERS FOR RESOLUTION. SHOP DETAILER IS TO ALLOW TO RE- WORK SHOP DRAWINGS AS NECESSARY. FABRICATOR SHALL PREPARE SHOP DRAWINGS AND SUBMIT THEM TO THE BUILDER FOR THEIR APPROVAL. BUILDER SHALL LODGE TWO HARD COPIES OF APPROVED DRAWINGS TO PMI ENGINEERS FOR REVIEW PRIOR TO FABRICATION, (ALLOW 5 WORKING DAYS FOR REVIEW).
S4. TYPICAL STEELWORK CONNECTIONS (UNLESS NOTED OTHERWISE)
- COLUMN BASE PLATES: 10 BASE PLATE, 4/M16 HILTI HIT-HY 150 MAX CHEMICAL INJECTION ANCHORS
- BEAM TO TOP OF COLUMN: CAP PLATE, 2 BOLTS TO CHANNELS, 4 BOLTS TO RHS/CHS/SHS/UB/UC
- BEAM TO SIDE OF COLUMN: FIN PLATE, 2 BOLTS
- BEAM TO SIDE OF BEAM: END OR FIN PLATE, 2 BOLTS
- COLUMNS TO TOP OF BEAM: BASE PLATE, 2 BOLTS TO CHANNELS, 4 BOLTS TO UB/UC SECTIONS
- ALL ROOF & WALL BRACING: CLEAT PLATES, 2 BOLTS
- PURLINS/WALL GIRTS: 8 CLEAT PLATES, 2 PURLIN BOLTS

UNLESS NOTED OTHERWISE, USE:
- 10mm BASE, CAP, GUSSET, FIN AND END PLATES.
- M20 8.8/S BOLTS. (4.6/S GRADE TO BE USED FOR HOLD DOWN BOLTS)
- 6mm CONTINUOUS FILLET WELDS MADE WITH E4818 MILD STEEL ELECTRODES.
- ALL WELDS SP CATEGORY

S5. NO PAINT ON MATING SURFACES WITH TF OR TB BOLTING UNLESS APPROVED BY PMI ENGINEERS.
S6. TF OR TB BOLTS TO BE INSTALLED WITH ONE HARDENED WASHER UNDER THE TURNED PART.
S7. TF AND TB BOLTING BY "PART TURN" METHOD WITH LOAD INDICATING WASHERS.
S8. ALL BOLTS, SCREWS, HOLD DOWN BOLTS, MASONRY ANCHORS SHALL BE HOT DIP GALVANISED TO AS1214-2016, AS/NZS 4534-2006, AS/NZS 4680-2006 & AS/NZS 4792-2006. NO CONNECTION SHALL HAVE LESS THAN 2 BOLTS. ALL BOLTS AND WASHERS SHALL BE GALVANISED. ALL HOLES SHALL BE 2mm LARGER THAN THE BOLT DIAMETER UNLESS NOTED OTHERWISE.
S9. MINIMUM YIELD STRESS:

- HOT ROLLED SECTIONS	=	300MPa
- SQUARE HOLLOW SECTIONS	=	350MPa
- RECTANGULAR HOLLOW SECTIONS	=	350MPa
- CIRCULAR HOLLOW SECTION	=	250MPa
- HOT ROLLED PLATE	=	250MPa

S10. COLD FORMED SECTIONS TO CONFORM WITH:
- AS/NZS 1594-2002, AS/NZS 1595-1998, AS/NZS 4600-2018 AND AS 1397-2011, AS1397, AS/NZS1594 AND AS/NZS1595.
- MINIMUM YIELD STRESSES SECTION 450MPa.
S11. SURFACE TREATMENT UNLESS NOTED OTHERWISE:

- PROTECTED FROM WEATHER	=	AS/NZS 2312-IZS2
- EXPOSED TO WEATHER	=	AS/NZS 2312-HDG600P3
- BUILT INTO THE INTERNAL SKIN OF EXTERNAL WALLS	=	AS/NZS 2312-HDG600P3

REFER TO PURLIN & GIRTS NOTES FOR SURFACE TREATMENT OF THESE ITEMS

S12. FIX CROSS BRACING TO PURLINS AT 3000 MAXIMUM CTS WITH M10 BOLTS OR M6 HOOKS.
S13. STEELWORK TO BE CONCRETE ENCASED SHALL BE FREE FROM ALL LOOSE RUST, LOOSE MILL SCALE, DIRT, OIL, GREASE, ETC. AND REINFORCED WITH SL41 FABRIC OR EQUIVALENT BLACK IRON WIRE, 3mm DIA.
S13.a ALL BURIED STEELWORK TO BE PAINTED FIRST USING 'EXPOSED TO WEATHER' TREATMENT SYSTEM FOLLOWED BY THE APPLICATION OF A TWO PART EPOXY SUCH AS 'SIKAGUARD-63N' OR APPROVED EQUIVALENT. ALTERNATIVELY, ENCASE BURIED STEELWORK IN CONCRETE WITH A MINIMUM COVER OF 75mm TO STEELWORK.
S14. BOLT SYMBOLS:
- 4.6/S = GRADE 4.6 BOLT / SNUG TIGHTENED.
- 8.8/S = GRADE 8.8 BOLT / SNUG TIGHTENED.
- 8.8/TF = GRADE 8.8 BOLT / FULLY TENSIONED FRICTION TYPE (USE LOAD INDICATOR WASHERS)
- 8.8/TB = GRADE 8.8 BOLT / FULLY TENSIONED BEARING TYPE (USE LOAD INDICATOR WASHERS)
S15. THE CONTRACTOR SHALL SUPPLY WRITTEN CERTIFICATION TO THE STRUCTURAL ENGINEER PRIOR TO THE ERECTION OF ANY STRUCTURAL STEEL STATING THAT THE BOLTS PROPOSED TO BE USED COMPLY WITH AS/NZS 1252.1-1996. HIGH STRENGTH BOLTS (8.8) ARE NOT TO BE WELDED.
S16. THE FABRICATION AND ERECTION OF THE STRUCTURAL STEEL WORK SHALL BE SUPERVISED BY A QUALIFIED PERSON EXPERIENCED IN SUCH SUPERVISION, IN ORDER TO ENSURE THAT ALL REQUIREMENTS OF THE DESIGN ARE MET.
S17. ALL MEMBERS SHALL BE SUPPLIED IN SINGLE LENGTHS. SPLICES SHALL ONLY BE PERMITTED IN LOCATIONS SHOWN ON THE STRUCTURAL DRAWINGS.

STEELWORK CONT.

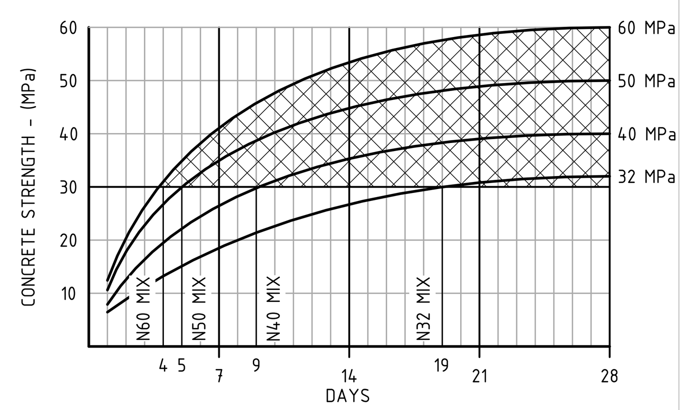
S18. ALL BUTT WELDS SHALL BE COMPLETE PENETRATION BUTT WELDS CATEGORY SP TO AS1554.1-2004 U.N.O THE EXTENT ON NON-DESTRUCTIVE WELD EXAMINATION SHALL BE AS NOTED BELOW:
- RADIOGRAPHIC OR ULTRASONIC EXAMINATION SHALL BE TO AS/NZS 1554.1-2014, AS 2177-2006 AND AS2207-2007 AS APPROPRIATE.
S19. GROUT ALL STEEL BASES BY DRY PACKING USING GROUT WHICH IS NON-SHRINK AND HAS A MINIMUM COMPRESSIVE STRENGTH AT 7 DAYS OF 40MPa
S20. PROVIDE SEAL PLATES TO THE ENDS OF ALL HOLLOW SECTIONS, WITH 'BREATHER' HOLES IF MEMBERS ARE TO BE HOT DIP GALVANISED.
S21. THESE DRAWINGS MAY NOT IDENTIFY ALL SECONDARY STEELWORK ELEMENTS THAT ARE REQUIRED FOR SUPPORT, FIXING AND FINISHING OF GLAZING, CLADDING AND LINING. THE TENDERER IS RESPONSIBLE FOR THE INCLUSION OF SUCH STEELWORK ELEMENTS TO THE EXTENT REQUIRED ON THE ARCHITECT'S DRAWINGS.
S22. IMPORTED STRUCTURAL STEEL MATERIAL
ALL STRUCTURAL STEELWORK USED ON THIS PROJECT SHALL BE COMPLIANT WITH AS4100, AND IN PARTICULAR:
- CERTIFIED MILL TEST REPORTS, OR TEST CERTIFICATES SHALL BE PROVIDED AS EVIDENCE OF COMPLIANCE WITH THE STANDARDS REFERRED TO IN AS4100. THESE CERTIFICATES SHALL BE SUBMITTED TO PMI ENGINEERS FOR APPROVAL PRIOR TO COMMENCEMENT OF FABRICATION.
- PROVIDE TEST CERTIFICATED FOR COMPLIANCE FOR ALL FASTENERS. THESE CERTIFICATES SHALL BE SUBMITTED TO PMI ENGINEERS FOR APPROVAL PRIOR TO FABRICATION.
- FOR COLD FORMED SECTIONS A "CERTIFICATE OF CONFORMITY TO AS1163-1991" SHALL BE SUBMITTED TO PMI ENGINEERS FOR APPROVAL PRIOR TO FABRICATION.
- CERTIFICATES SHALL ONLY BE ACCEPTED FROM TESTING COMPANIES ACCREDITED BY A TESTING AUTHORITY RECOGNISED IN AUSTRALIA, EG NATA or JAS-ANZ CERTIFIED.
- UNIDENTIFIED STEEL ie. ANY STEEL THAT IS NOT ACCOMPANIED WITH EVIDENCE STATING COMPLIANCE WITH THE REQUIREMENT OF AS4100 SHALL ONLY BE USED STRICTLY IN ACCORDANCE WITH CLAUSE 2.2.3 OF AS4100.

IF MATERIALS SUPPLIED AND INSTALLED ARE SUBSEQUENTLY PROVEN TO BE NON COMPLIANT WITH THE SPECIFIED AUSTRALIAN STANDARDS IT SHALL BE THE CONTRACTOR'S RESPONSIBILITY AND COST TO UNDERTAKE NATA OR EQUIVALENT CERTIFIED TESTING TO PROVE CONFORMANCE TO THE AUSTRALIAN STANDARDS AND DESIGN SPECIFICATIONS. SIMILARLY ANY RECTIFICATION WORKS THAT MAY SUBSEQUENTLY BE REQUIRED TO SATISFY AUSTRALIAN CODE REQUIREMENT SHALL BE THE RESPONSIBILITY OF THE CONTRACTOR..

FIRE PROTECTION OF STEELWORK

FP1. PROVIDE 120/120/120 FIRE PROTECTION TO ALL PERMANENT STRUCTURAL STEEL MEMBERS AND CONNECTIONS.
FP2. REINSTATE ANY FIRE PROTECTION REMOVED FROM EXISTING STRUCTURAL STEELWORK.
FP3. INSTALL FIRE PROTECTION MATERIALS IN ACCORDANCE WITH THE MANUFACTURER'S WRITTEN SPECIFICATIONS.
FP4. PROVIDE CERTIFICATION OF FIRE PROTECTION ON COMPLETION.

CONCRETE STRENGTH V AGE - TYPE A PORTLAND CEMENT



CS1. CONCRETE TO BE SAMPLED AND TESTED IN ACCORDANCE WITH AS1012.2
CS2. CHART TO BE USED AS A GUIDE ONLY AND SHOULD BE CONFIRMED WITH SUPPLIER
CS3. BUILDER TO OBTAIN WRITTEN CONFIRMATION OF CONCRETE STRENGTH FROM SUPPLIER

CONCRETE

C1. CARRY OUT ALL CONCRETE WORK IN ACCORDANCE WITH AS3600-2018 AND NATSPEC CONCRETE STANDARDS.
C2. CONCRETE PROPERTIES AND COVER TO REINFORCING :

COVER TO REINFORCEMENT

ELEMENT		CONCRETE STRENGTH f'c (MPa)	MAXIMUM 56 DAY DRY SHRINKAGE	COVER (mm)	
BORED PIERS		40	650 um	45	
SLABS ON GROUND	EXPOSED	40	650 um	TOP 30	BTM 20
	COVERED	40		TOP 20	BTM 20
STRIP FOOTING		40	650 um	45	
PAD FOOTING		40	650 um	45	
SUSPENDED SLABS	EXPOSED	40	650 um	TOP 30	BTM 30
	COVERED	40		20	
BEAMS	EXPOSED	40	650 um	TOP 30	BTM 30
	COVERED	40		20	
COLUMNS	EXPOSED	40	650 um	30	
	COVERED	40		20	
WALLS	EXPOSED	40	650 um	30	
	COVERED	40		20	

MAXIMUM AGGREGATE SIZE	= 20mm U.N.O.
SLUMP DURING PLACING	= 75mm ±10mm
EXPOSURE CLASSIFICATION	= A2 (INTERNAL CONCRETE ELEMENTS)
	= A2 (EXTERNAL CONCRETE ELEMENTS)

NO ADMIXTURES SHALL BE USED IN THE CONCRETE MIX UNLESS APPROVED BY PMI ENGINEERS IN WRITING.

C3. CONCRETE PROPERTIES FOR SLABS AND BEAMS SHALL BE VARIED FROM NORMAL CLASS AS FOLLOWS :
- MINIMUM CEMENT CONTENT 250kg/m3/
- MAXIMUM 56 DAY SHRINKAGE STRAIN = AS NOMINATED ABOVE
- PRIOR TO COMMENCEMENT CONCRETE SUPPLIER TO PROVIDE DRYING SHRINKAGE TEST RESULTS FROM PRODUCTION ASSESSMENT AS EVIDENCE THAT SPECIFIED DRYING SHRINKAGE LIMITS CAN BE ACHIEVED USING NORMAL MIX DESIGN.
C4. SUBMIT FOR APPROVAL THE FOLLOWING TO THE ENGINEER :
- CURING PROCEDURE (PVA MEMBRANES NOT PERMITTED)
- STRIPPING AND BACK PROPPING PROCEDURE
- DETAILS AND LOCATION OF CONDUITS AND PENETRATIONS
- CONSTRUCTION JOINT LOCATIONS
C5. FOR TENDER PURPOSES ASSUME MINIMUM STRIPPING TIMES AND EXTENT OF BACK PROPPING AS PER AS3610-1995 SECTION 5.0 AND AS PER GENERAL NOTES FOR FORMWORK AND PROPPING.
C6. FORMWORK FINISH CLASSIFICATION TO AS3610.1-2010 :

ELEMENT	CLASS
- INGROUND FOOTINGS	5
- RETAINING WALLS	5 EARTH FACE
- RETAINING WALLS	2 EXPOSED FACE
- COLUMNS	2
- LIFT WALLS	2
- BEAMS & SLABS	2
- STAIRS	2
- GRANO TREATED SURFACES	2

(UNLESS NOTED OTHERWISE BY ARCHITECTURAL DOCUMENTATION)

C7. SURFACE FINISHES :

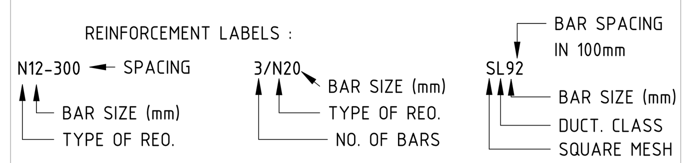
- COLUMNS & WALLS	OFF FORM
- FLOOR SLABS (U.N.O.)	MACHINE FLOAT
- SLABS TO BE TILED	WOOD FLOAT
- STAIRS STEEL TROWEL	

(UNLESS NOTED OTHERWISE BY ARCHITECTURAL DOCUMENTATION)

C8. COMPACT ALL CONCRETE, INCLUDING FOOTINGS AND SLABS USING MECHANICAL VIBRATORS.
C9. PLACE CONCRETE CONTINUOUSLY BETWEEN CONSTRUCTION JOINTS SHOWN ON PLAN. DO NOT BREAK OR INTERRUPT SUCCESSIVE POURS SUCH THAT COLD JOINTS OCCUR. ANY REVISIONS OR ADDITIONS TO CONSTRUCTION JOINTS SHOWN ON PLAN REQUIRE APPROVAL FROM PMI ENGINEERS.
C10. CONCRETE PROFILES :
- BEAM DEPTHS ARE WRITTEN FIRST AND INCLUDE THE SLAB THICKNESS.
- SIZES OF CONCRETE ELEMENTS DO NOT INCLUDE THICKNESS OF APPLIED FINISHES.
- NO HOLES, CHASES OR EMBEDMENT OF PIPES OTHER THAN SHOWN IN THE STRUCTURAL DRAWINGS SHALL BE MADE IN CONCRETE MEMBERS WITHOUT THE PRIOR WRITTEN APPROVAL OF PMI ENGINEERS.
- PROVIDE DRIP GROOVES AT ALL EXPOSED EDGES. CHAMFERS, DRIP GROOVES, REGLETS ETC TO ARCHITECT'S DETAILS.
C11. ALL PENETRATIONS TO HAVE 2/N16 TRIMMER BARS TOP AND BOTTOM TO EACH FACE. U.N.O. EXTEND TRIMMERS 600 BEYOND PENETRATION.
C12. SETDOWNS OR FALLS IN FLOOR SURFACES ARE NOT PERMITTED UNLESS SHOWN ON DRAWINGS. MAINTAIN MINIMUM SLAB THICKNESS SHOWN ON PLAN WHERE FALLS OCCUR.
C13. CONCRETE IS INCLINED TO CRACK, AND SURFACE FINISH QUALITY IS LARGELY DEPENDENT UPON FINISHING AND PLACEMENT METHODOLOGY. AS SUCH PMI ENGINEERS TAKES NO RESPONSIBILITY FOR THE QUALITY OF CONCRETE FINISH.
C14. REINFORCEMENT QUALITY AND NOTATION :
ALL REINFORCING BAR SHALL BE GRADE D500N TO AS/NZS 4671-2001 AND ALL MESH SHALL BE GRADE 500L TO AS/NZS 4671-2001. UNLESS NOTED OTHERWISE CLASS L REINFORCEMENT SHALL NOT BE USED.

REINFORCEMENT NOTATION

SYMBOL	BAR TYPE	STRENGTH GRADE (MPa)	DUCTILITY CLASS	TO COMPLY WITH AUSTRALIAN STANDARD
S	STRUCTURAL GRADE DEFORMED RIB BAR	250	NORMAL	AS/NZS 4671-2001
N	STRUCTURAL GRADE DEFORMED RIB BAR	500	NORMAL	AS/NZS 4671-2001
R	PLAIN ROUND BAR	250	NORMAL	AS/NZS 4671-2001
RL	RECTANGULAR MESH DEFORMED RIB BAR	500	LOW	AS/NZS 4671-2001
SL	SQUARE MESH DEFORMED RIB BAR	500	LOW	AS/NZS 4671-2001
L-TM	TRENCH MESH	500	LOW	AS/NZS 4671-2001



C15. REINFORCEMENT IS REPRESENTED DIAGRAMMATICALLY, AND NOT NECESSARILY IN TRUE PROJECTION. BARS SHOWN ARE INDICATIVE ONLY AND LENGTHS MAY VARY. BEAM ELEVATIONS TAKE PRECEDENCE OVER SECTIONS. SLAB PLANS TAKE PRECEDENCE OVER SECTIONS. REFER TO SECTIONS FOR EXTRA BARS THAT MAY BE REQUIRED.
C16. USE ONLY PLASTIC OR CONCRETE CHAIRS AT EXTERNAL SURFACES.
C17. SITE BENDING OF REINFORCEMENT BARS SHALL BE DONE WITHOUT HEATING USING A RE-BENDING TOOL. THE BARS SHALL BE RE-BENT AGAINST A FLAT SURFACE OR A PIN WITH A DIAMETER NOT LESS THAN THE MINIMUM PIN SIZE PRESCRIBED IN AS3600-2009.
C18. SPLICES IN REINFORCEMENT SHALL BE MADE ONLY IN POSITIONS SHOWN ON THE STRUCTURAL DRAWINGS OR IN POSITIONS OTHERWISE APPROVED IN WRITING BY PMI ENGINEERS. LAPS SHALL NOT BE LESS THAN THE DEVELOPMENT LENGTH FOR EACH BAR AND IN ACCORDANCE WITH AS3600-2018 SECTION 13.
C19. LAPS IN MESH IN ACCORDANCE WITH AS3600-2018 SECTION 13.
C20. WELDING OF REINFORCEMENT SHALL NOT BE PERMITTED UNLESS SHOWN ON THE STRUCTURAL DRAWINGS OR APPROVED BY PMI ENGINEERS.
C21. AT EXTERNALLY EXPOSED SURFACES NO METALLIC ITEMS INCLUDING FORM BOLTS, FORM SPACERS, METALLIC BAR CHAIRS AND TIE-WIRE ARE TO BE PLACED IN THE COVER ZONE.
C22. ALL REINFORCEMENT, ANCHOR BOLTS AND OTHER CONCRETE INSERTS SHALL BE WELL SECURED IN POSITION AND INSPECTED BY PMI ENGINEERS PRIOR TO PLACING CONCRETE.
C23. HOLD DOWN BOLTS SHALL BE HOT DIPPED GALVANISED.
C24. U.N.O., ALL MASONRY ANCHORS INTO CONCRETE SHALL BE RAMSET TRUBOLTS (LONGEST VERSION) OR APPROVED EQUIVALENT. BOLTS SHALL BE GALVANISED WHERE THEY ARE ADJOINING NON FERROUS OR PREPAINTED MEMBERS. PROVIDE STAINLESS STEEL BOLTS FOR ALL EXTERNAL CONDITIONS, OR WHERE EXPOSED TO THE WEATHER.
C25. ALL CONCRETE MIXES SHALL BE DESIGNED BY A RECOGNISED TESTING LAB AND SUBMITTED FOR REVIEW BY PMI ENGINEERS.
C26. ALL COMPRESSIVE STRENGTH TEST REPORTS SHALL BE SUBMITTED TO PMI ENGINEERS FOR REVIEW.
C27. TESTING SHALL BE CARRIED OUT ON ALL CONCRETE IN ACCORDANCE WITH AS1379-2007. TEST CYLINDERS ARE TO BE KEPT ON SITE.
C28. CURING OF ALL CONCRETE IS TO BE ACHIEVED BY KEEPING SURFACES CONTINUOUSLY WET FOR A PERIOD OF 7 DAYS, UNLESS SPECIFIED OTHERWISE. APPROVED SPRAY ON CURING COMPOUNDS THAT COMPLY WITH AS3799-1998 MAY BE USED WHERE FLOOR FINISHES WILL NOT BE AFFECTED. POLYTHENE SHEETING OR WET HESSIAN MAY BE USED TO RETAIN CONCRETE MOISTURE WHERE PROTECTED FROM WIND AND TRAFFIC. CURING IS TO COMMENCE IMMEDIATELY AFTER CONCRETE PLACEMENT.
C29. FOR ELAPSED TIME BETWEEN THE WETTING OF THE MIX AND THE DISCHARGE OF THE MIX, REFER TO CONCRETE - ELAPSED DELIVERY TIMES NOTE.

CONCRETE - ELAPSED DELIVERY TIMES

CE1. ELAPSED TIME BETWEEN THE WETTING OF THE MIX AND THE DISCHARGE OF THE MIX AT THE SITE MUST NOT EXCEED THE CRITERIA IN THE ELAPSED DELIVERY TIMETABLE BELOW

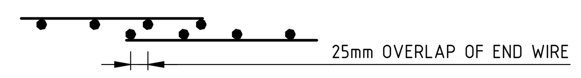
ELAPSED DELIVERLY TIME TABLE

CONC. TEMP. AT DISCHARGE (°C)	MAXIMUM ELAPSED TIME (HOURS)
≤ 24	2.00
24 to 27	1.50
27 to 30	1.00
30 to 32	0.75
32 to 35	0.50

IF THE ELAPSED TIME IS LONGER THAN THE CORRESPONDING TIME IN THE TABLE ABOVE, OR THE TEMPERATURE IS GREATER THAN 35°C, EITHER PMI ENGINEERS OR THE CONCRETE MIX DESIGN ENGINEER ARE TO BE CONTACTED TO CONFIRM WHETHER PLACEMENT IS TO PROCEED OR IF THE POUR IS TO BE STOPPED. IF THE POUR IS STOPPED, PRIOR TO ANY FURTHER CONCRETE PLACEMENT PMI ENGINEERS ARE TO BE CONTACTED TO INSPECT THE WORKS AND DETERMINE WHAT, IF ANY, RECTIFICATION WORKS ARE REQUIRED.

SLAB ON GROUND - RESIDENTIAL

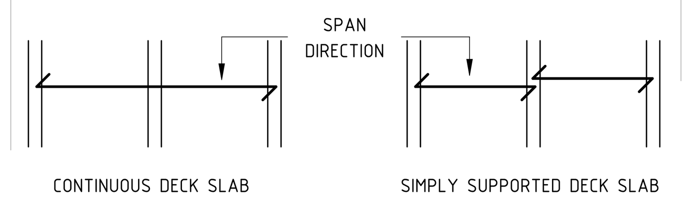
RSG1. RESIDENTIAL SLABS ON GROUND SHALL BE IN ACCORDANCE WITH AS2870-2011.
RSG2. THE SITE OF THE WORKS SHALL BE STRIPPED OF ALL GRASS, ROOTS, VEGETABLE MATTER AND COMPRESSIBLE TOPSOIL.
RSG3. THE GROUND BELOW SLABS SHALL BE PROOF ROLLED WITH AN APPROVED HEAVY COMPACTOR. ALL "SOFT SPOTS" ENCOUNTERED SHALL BE REMOVED AND REPLACED WITH COMPACTED CRUSHED ROCK OR APPROVED FILL IN ACCORDANCE WITH AS2870-2011 & AS3798-2007.
RSG4. CLEAN GRANULAR FILLING UP TO 600mm MAY BE PLACED UNDER THE SLAB IN ACCORDANCE WITH THE PROVISIONS OF AS2870-2011 PART 6.4. FILLING SHALL BE COMPACTED IN 150mm THICK LAYERS BY MECHANICAL ROLLER.
RSG5. TERMITE PROTECTION SHALL BE PROVIDED AS REQUIRED BY AS3660.1-2000 AND THE LOCAL STATUTORY AUTHORITY.
RSG6. SLABS SHALL BE LAID ON A 0.2mm POLYTHENE MEMBRANE, CONTINUOUS, LAPPED 20mm MINIMUM AND TAPED AT JOINTS, PUNCTURES AND SERVICE PIPE PENETRATIONS.
RSG7. BEAM AND STRIP FOOTING REINFORCEMENT SHALL ACHIEVE THE REQUIRED COVER AS NOTED IN CONCRETE SPECIFICATIONS
RSG8. TRENCH MESH SHALL BE LAID CONTINUOUSLY AND SHALL BE SPLICED WHERE NECESSARY WITH A LAP OF 600mm.
RSG9. TRENCH MESH SHALL BE OVERLAPPED BY THE WIDTH OF MESH AT CORNERS AND INTERSECTIONS AND THE ENDS OF TRENCH MESH SHALL TERMINATE WITH A CROSSBAR.
RSG10. MESH SHALL BE PLACED NEAR THE TOP OF THE SLAB AND SHALL ACHIEVE THE REQUIRED COVER. MESH SHALL BE LAPPED A MINIMUM OF TWO WIRES PLUS 25mm AND SHALL BE SET OUT SUCH THAT NO MORE THAN THREE THICKNESSES OF MESH OCCUR AT ANY LOCATION.



RSG11. HOT WATER HEATING PIPES MAY BE EMBEDDED IN THE SLAB IF THE THICKNESS IS INCREASED BY 25mm AND LAID ON SL52 MESH, OR IF THE SLAB THICKNESS IS INCREASED BY 25mm AND THE MESH SIZE IS INCREASED BY ONE SIZE (eg FROM SL82 MESH TO SL92 MESH).
RSG12. THE GROUND SURROUNDING THE SLAB SHALL HAVE ITS SURFACE AT LEAST 150mm LOWER THAN THE SLAB SURFACE AND BE GRADED AWAY FROM THE SLAB EDGE TO THE SITE DRAINAGE SYSTEM.
RSG13. ADDITIONAL PLUMBING REQUIREMENTS FOR CLASS M, H & E SITES.
CLASS M H or E SITES: THE BASE OF TRENCHES SHALL BE SLOPED AWAY FROM THE BUILDING. TRENCHES SHALL BE BACKFILLED WITH CLAY IN THE TOP 300mm WITHIN 1.5m OF THE BUILDING AND THE CLAY COMPACTED. WHERE PIPES PASS UNDER THE FOOTING SYSTEM THE FULL DEPTH OF THE TRENCH SHALL BE BACKFILLED WITH CLAY or CONCRETE. SUBSURFACE DRAINS TO REMOVE GROUNDWATER SHALL NOT BE USED WITHIN 1.5m OF THE BUILDING UNLESS NOTED OTHERWISE.
ADDITIONAL REQUIREMENTS FOR CLASS H & E SITES: THESE REQUIREMENTS APPLY TO ALL STORMWATER, SANITARY PLUMBING DRAINS & DISCHARGE PIPES.
- CLOSED-CELL POLYETHYLENE LAGGING SHALL BE USED AROUND PIPE PENETRATIONS THROUGH FOOTINGS. THE LAGGING SHALL BE A MINIMUM OF 20mm THICK ON CLASS H1 SITES & 40mm THICK ON CLASS H2 & CLASS E SITES. VERTICAL PENETRATIONS DO NOT REQUIRE LAGGING.
- DRAINS ATTACHED TO or EMERGING FROM UNDERNEATH THE BUILDING SHALL INCORPORATE FLEXIBLE JOINTS IMMEDIATELY OUTSIDE THE FOOTING AND COMMENCING WITHIN 1m OF THE BUILDING PERIMETER TO ACCOMMODATE A TOTAL RANGE OF DIFFERENTIAL MOVEMENT IN ANY DIRECTION EQUAL TO THE ESTIMATED CHARACTERISTIC SURFACE MOVEMENT ON THE SITE (ys). ys = ???, (IN THE ABSENCE OF THE SPECIFIC DESIGN GUIDANCE, THE FITTINGS or OTHER DEVICES TO ALLOW FOR THE MOVEMENT SHALL BE SET AT THE MID POSITION OF THEIR RANGE OF POSSIBLE MOVEMENT AT THE TIME OF INSTALLATION).
- PIPES MAY BE ENCASED IN CONCRETE or IN RECESSES IN THE SLAB WHEN PROVIDED WITH FLEXIBLE JOINTS AT THE EXTERIOR OF THE SLAB. METHODS USED SHOULD COMPLY WITH THE AS/NZS 3500 SERIES.
- COLD WATER PIPES AND HEATED or HOT WATER PIPES SHALL NOT BE INSTALLED UNDER A SLAB UNLESS THE PIPES ARE INSTALLED WITHIN A CONDUIT SO THAT IF THE PIPE LEAKS WATER IT WILL BE NOTICED ABOVE THE SLAB or OUTSIDE THE SLAB AND WILL NOT LEAK UNNOTICED UNDER THE SLAB. WATER SERVICE PIPES INSTALL UNDER CONCRETE SLABS SHOULD COMPLY WITH THE RELEVANT REQUIREMENTS OF AS/NZS 3500.1. HEATED WATER SERVICE PIPES INSTALLED UNDER CONCRETE SLABS SHOULD COMPLY WITH THE RELEVANT REQUIREMENTS OF AS/NZS 3500.4.

STEEL DECK SLABS (BONDEK or CONDECK)

BS1. STEEL DECKING TO BE INSTALLED STRICTLY IN ACCORDANCE WITH MANUFACTURER'S WRITTEN INSTRUCTIONS.
BS2. REFER TO PLAN FOR STEEL DECKING SPECIFICATION. CONTRACTOR MAY SUBMIT FOR APPROVAL EQUIVALENT DECKING PRODUCTS.
BS3. PROVIDE 40mm MINIMUM BEARING AT SUPPORTS.
BS4 AT ALL RE-ENTRANT CORNERS PROVIDE 3/N12 TRIMMERS 2000 LONG TIED TO UNDERSIDE OF MESH.
BS5. UNLESS NOTED OTHERWISE, PROVIDE TEMPORARY PROPPING OF DECK IN ACCORDANCE WITH THE MANUFACTURER'S WRITTEN INSTRUCTIONS.



REGULATED DESIGN RECORD		REV	DATE	DESCRIPTION	DP FULL NAME	REG NO
PROJECT ADDRESS:	30 DIGGINGS TERRACE, THREDBO		07.09.2021	ISSUE FOR COMMENT	THOMAS WILLIAMS	PRE0001122
PROJECT TITLE:	BLACK BEAR INN	1	15.09.2021	ISSUED FOR CC	THOMAS WILLIAMS	PRE0001122
CONSENT NUMBER:		2	07.10.2021	FOR CONSTRUCTION	THOMAS WILLIAMS	PRE0001122
		3	16.11.2021	REVISED FOR ANCHORAGES	THOMAS WILLIAMS	PRE0001122
		4	01.02.2022	REVISED FOR PARTICULARS OF REGULATED DESIGN - GROUND ANCHORS	THOMAS WILLIAMS	PRE0001122
		5	28.02.2022	CONSOLIDATED SHEETS FOR DA SUBMISSION	THOMAS WILLIAMS	PRE0001122

DRAWING TITLE
EXCAVATION PLAN ASDAD

JOB NUMBER **PMI-2021-053**

DRAWING NUMBER **S10** REVISION **5**

SCALE AT B1: As indicated

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ISSUE:

FOR CONSTRUCTION

ALL SETOUT TO ARCHITECT'S DRAWINGS.
DIMENSIONS TO BE VERIFIED WITH ARCHITECT AND BUILDER BEFORE COMMENCING SHOP DRAWINGS OR SITE WORK.
ENGINEER ACCEPTS NO RESPONSIBILITY FOR THE USABILITY, COMPLETENESS OR SCALE OF DRAWINGS TRANSFERRED ELECTRONICALLY.

NOTE:

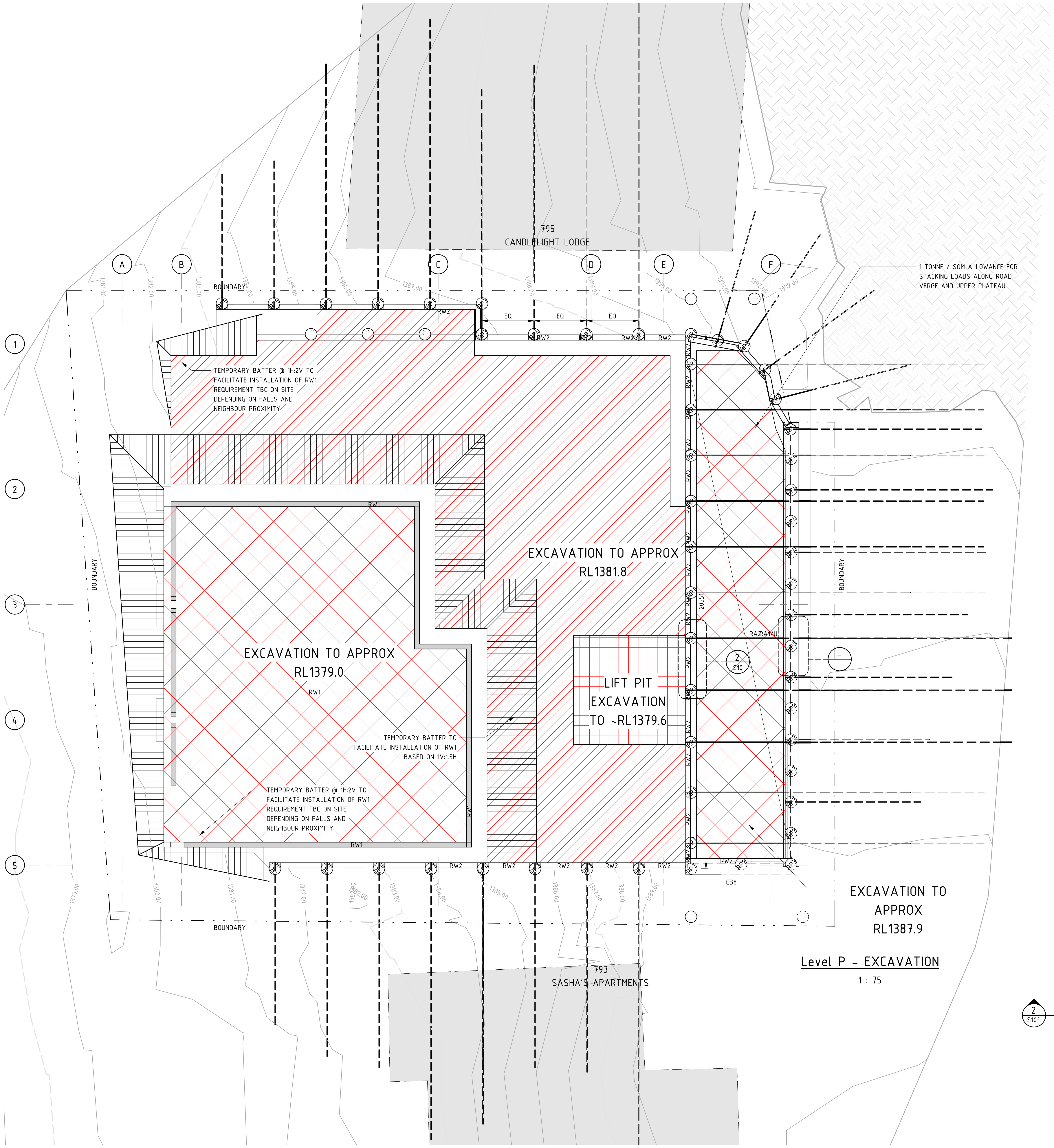
ALL ANCHORS TO BE TESTED TO TEST LOAD FOR 15 MINUTES AND ANCHOR IS TO BE CONFIRMED HOLDING 'TEST LOAD' FOR THE FULL 15 MIN DURATION
ANCHOR WORKING LOADS TEST LOADS AND LOCK-OFF LOADS ARE SOURCED FROM THE ANCHOR SCHEDULE - SEE S10d, S10e + S10f

TOLERANCES:
- ALL ANCHORS TO BE LOCATED WITHIN 250mm OF THE STATED RL
- WITHIN 5 DEG OF STATED ANGLE OFF HORIZONTAL
- ALL ANCHORS TO BE PERPENDICULAR TO EXCAVATION CUT WITHIN 5 DEG
- MINIMUM FREE LENGTH OF ANCHORS OF 3m AS NOTED ON SECTIONS

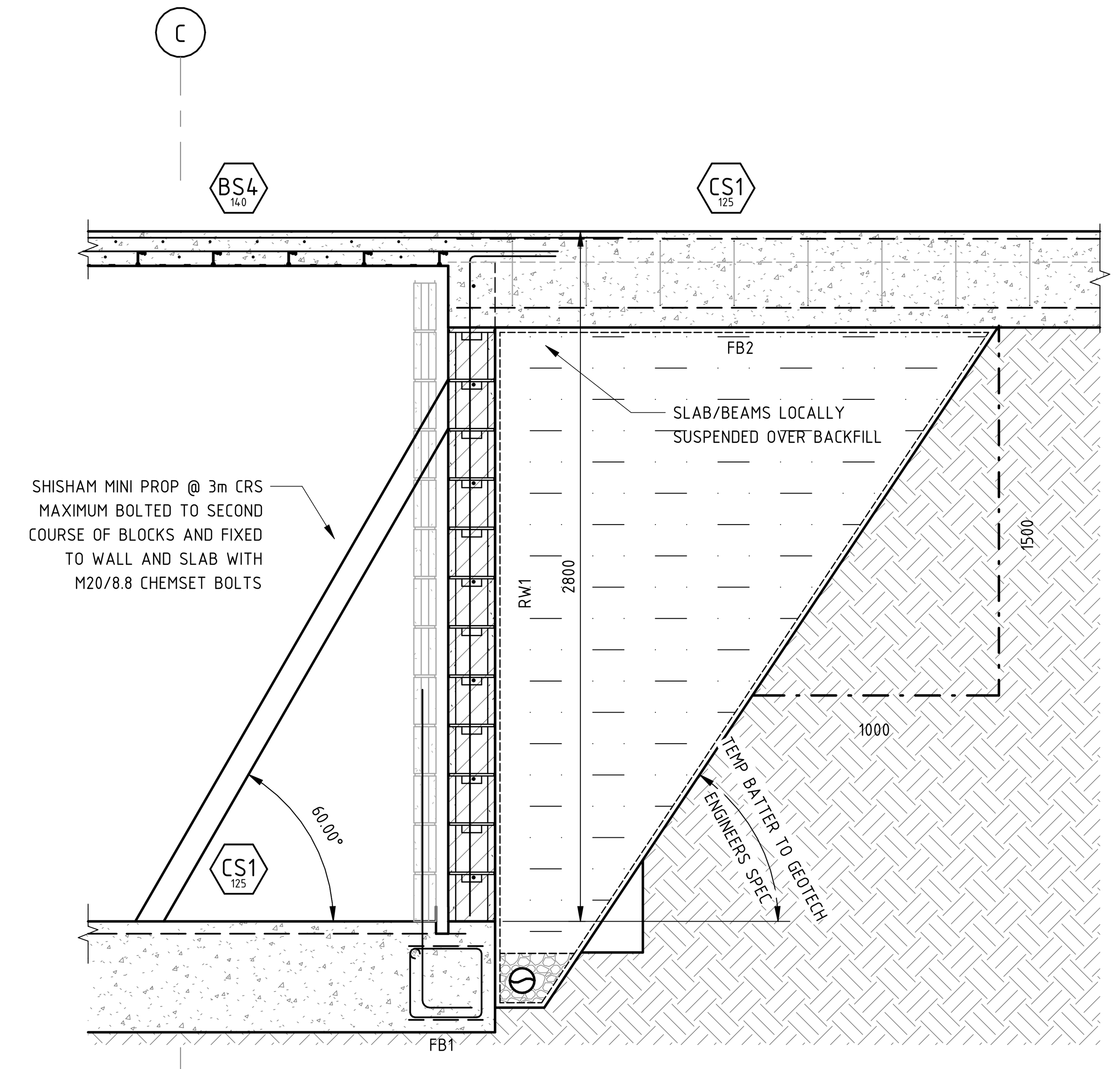
#SCHEDULE - P - RETAINING

Type Mark	Description
ANCHORS	
RA1	26.5mm DYWIDAG Y1050H PRESTRESSING STEEL BAR - OR OTHER APPROVED - SEE ACCOMPANYING SHEET FOR LOADS
RA2	32mm DYWIDAG Y1050H PRESTRESSING STEEL BAR - OR OTHER APPROVED - SEE ACCOMPANYING SHEET FOR LOADS
RA3	36mm DYWIDAG Y1050H PRESTRESSING STEEL BAR - OR OTHER APPROVED - SEE ACCOMPANYING SHEET FOR LOADS
FOUNDATIONS	
CB8	600Wx400D CAPPING BEAM TO ROAD - 3N20s TOP & BTM with N12 STIRRUPS @ 300 CRS
RETAINING SYSTEM	
RP1	450 DIA PIER REINFORCED WITH 6/N28s @ N12 SPIRAL @ 250 PITCH
RP2	450 DIA PIER REINFORCED WITH 4/N16s @ N10 SPIRAL @ 300 PITCH
RP3	450 DIA PIER REINFORCED WITH 4/N20s @ N12 SPIRAL @ 300 PITCH
RP4	450 DIA PIER REINFORCED WITH 4/N24s @ N10 SPIRAL @ 300 PITCH
RP5	450 DIA PIER REINFORCED WITH 4/N16s @ N12 SPIRAL @ 300 PITCH
RP6	450 DIA PIER REINFORCED WITH 6/N20s @ N12 SPIRAL @ 300 PITCH
RP7	450 DIA PIER REINFORCED WITH 6/N24s @ N12 SPIRAL @ 300 PITCH
RW1	190 COREFILLED BLOCKWORK WALLS - N16s @ 400 CRS VERTICAL - N12s @ 400 CRS HORIZONTAL - TEMP RESTRAINT REQUIRED AT TOP PRIOR TO SLAB OVER BEING POURED
RW2	200mm 32MPa SHOTCRETE WALLS - SEE S10 FOR DETAILS

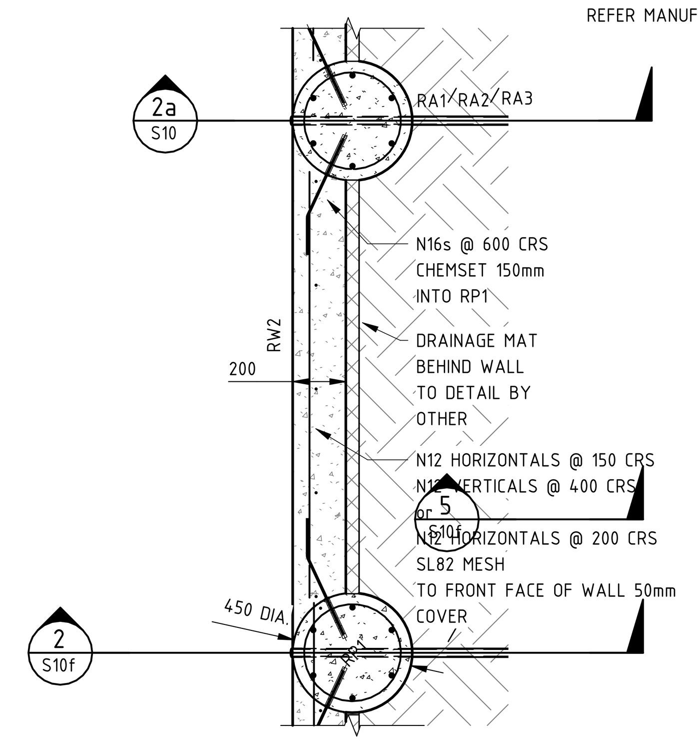
NOTE:
- RETAINING PILES DESIGNED BASED ON RECTANGULAR PRESSURE DISTRIBUTION
  - 8H + 5kPa SURCHARGE, 10kPa SURCHARGE FROM ROAD
  - ADDITIONAL 64kN/m LATERAL LOAD AT TOP OF N1/N2 PILES TO ACCOUNT FROM PRESSURE FROM ROAD RETENTION PILES
- GROUND SUPPORT MEASURES ARE INDICATIVE ONLY PRIOR TO CONFIRMATION OF GROUND CONDITIONS ON OPENING UP OF SITE
- ALLOWABLE TEMPORARY/PERMANENT BATTER ANGLES TO BE VERIFIED ONSITE WITH GROUND INVESTIGATIONS AND AS EXCAVATION PROCEEDS



Level P - EXCAVATION
1 : 75

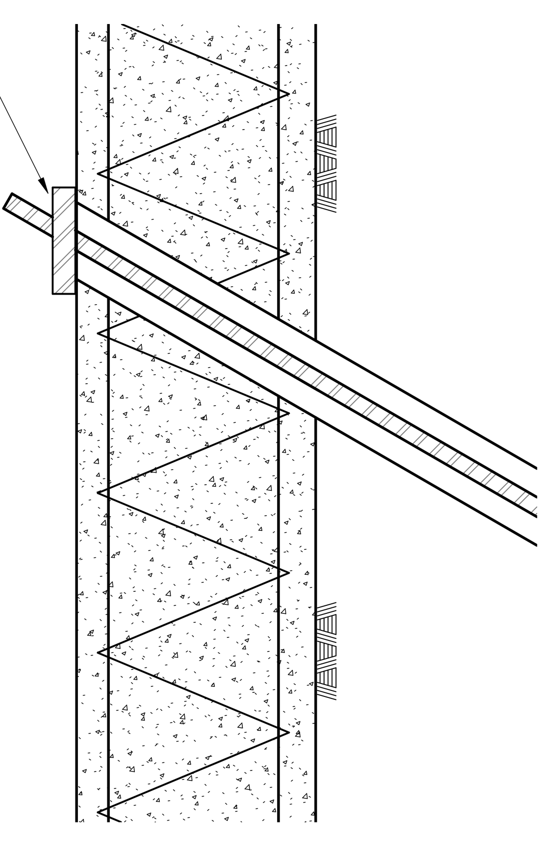


DETAIL 1 / S10a
1 : 20



DETAIL 2 / S10
1 : 20

200x40 STEEL BEARING PLATE GROUT PACKED AGAINST PILE WITH DOMED NUT FOR ANGLE COMPENSATAION BY MANUFACTURER REFER MANUFACTURER'S SPEC



DETAIL 2a / S10
1 : 10

REGULATED DESIGN RECORD	REV	DATE	DESCRIPTION	DP FULL NAME	REG NO
PROJECT ADDRESS: 30 DIGGINGS TERRACE, THREDBO		07.09.2021	ISSUE FOR COMMENT	THOMAS WILLIAMS	PRE0001122
PROJECT TITLE: BLACK BEAR INN	1	15.09.2021	ISSUED FOR CC	THOMAS WILLIAMS	PRE0001122
CONSENT NUMBER:	2	07.10.2021	FOR CONSTRUCTION	THOMAS WILLIAMS	PRE0001122
	3	16.11.2021	REVISED FOR ANCHORAGES	THOMAS WILLIAMS	PRE0001122
	4	01.02.2022	REVISED FOR PARTICULARS OF REGULATED DESIGN – GROUND ANCHORS	THOMAS WILLIAMS	PRE0001122
DRAWING TITLE: **EXCAVATION DETAILS - 1**	5	29.04.2022	DIFFERENTIATION BETWEEN BUILT AND UNBUILT WORKS	THOMAS WILLIAMS	PRE0001122
JOB NUMBER: **PMI-2021-053**					
DRAWING NUMBER: **S10a** REVISION: **5**					
SCALE AT B1: 1 : 50					

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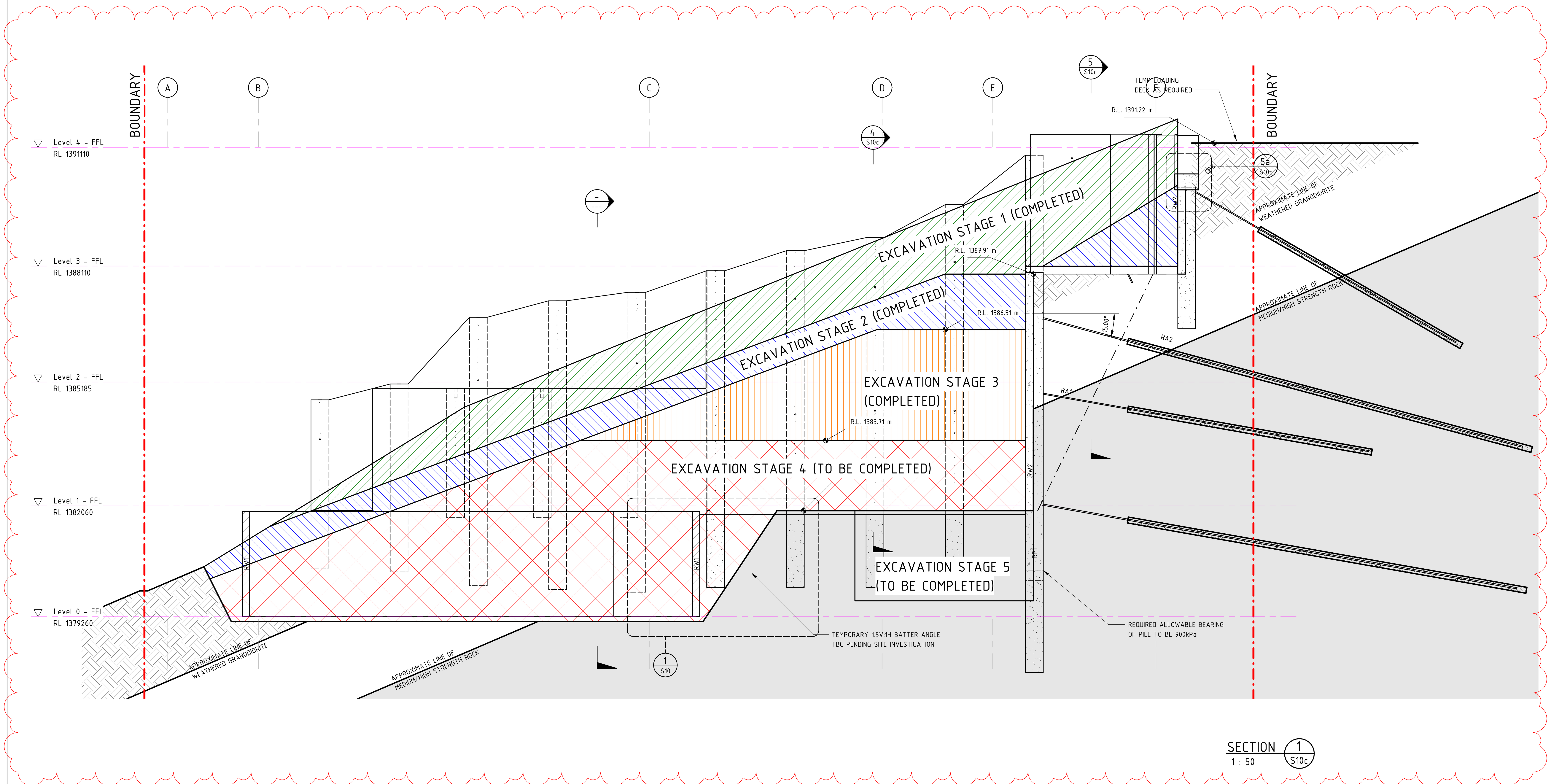
PROPOSED METHODOLOGY

1. INSTALL PILES TO LEVEL 4 @ 1.2m AND AROUND EXCAVATION PERIMETER @ ~2m CRS AND INSTALL CAPPING BEAMS AS REQUIRED
2. EXCAVATE STAGE 1 AS INDICATED TO SHOTCRETING PILES AS REQUIRED AND TAKING READINGS OF PILES TO CHECK DEFLECTIONS
3. INSTALLING ANCHORS TO SOUTHERN PILES AND FIRST ROW OF EAST AND WESTERN PILES
4. INSTALL LOWER PILES ALONG GRID E WITH ADDITIONAL EXCAVATION AS REQUIRED
5. TEST SELECTED ROCK ANCHORS TO NOMINATED LOAD TO CONFIRM CAPACITY
6. EXCAVATE STAGE 2 AS INDICATED SLOPING TO THE NORTH AS NECESSARY TO ENABLE ACCESS TO ANCHORAGES TAKING READINGS OF PILES TO CHECK DEFLECTIONS
7. SHOTCRETE BETWEEN PILES
8. POUR 200mm CS6 CAPPING SLAB TO CONNECT RP1 AND RP2 PILES AT RL1387.90
9. INSTALL TOP STAGE OF ROCK ANCHORS TO PILES ON GRID E AND OTHER PERIMETER PILES AS AVAILABLE
10. TEST SELECTED ROCK ANCHORS TO NOMINATED LOAD TO CONFIRM CAPACITY
11. EXCAVATE STAGE 3 TAKING READINGS OF PILES TO CHECK DEFLECTIONS
12. INSTALL NEXT ROW OF ANCHORS ALONG GRID E AND 2nd ROW OF ANCHORS TO EAST AND WEST WINGS
13. SHOTCRETE BETWEEN PILES
14. TEST SELECTED ROCK ANCHORS TO 1.3x WORKING LOAD TO CONFIRM CAPACITY
15. EXCAVATE STAGE 4, SHOTCRETING WALLS AS NECESSARY
16. INSTALL FINAL ROW OF ANCHORS AROUND LIFT PIT AND TEST SELECTED ROCK ANCHORS TO NOMINATED LOAD TO CONFIRM CAPACITY
17. EXCAVATE STAGE 5 LIFT PIT
18. PROGRESSIVELY CONSTRUCT STRUCTURE TAKING READINGS OF WALLS AT KEY STAGES TO MONITOR DEFLECTIONS
19. ONCE LEVEL 3 SLAB HAS REACHED DESIGN STRENGTH (40 MPa), DE-STRESS ROCK ANCHORS

WITNESS, HOLD AND MONITORING POINTS

- GEOTECHNICAL INVESTIGATION ONSITE POST DEMOLITION OF EXISTING STRUCTURE TO CONFIRM ASSUMPTIONS
- GEOTECHNICAL INVESTIGATION ONSITE EVERY 1.5m DEPTH OF EXCAVATION TO CONFIRM GROUND CONDITIONS
- STRUCTURAL INSPECTION REQUIRED:
  - PRIOR TO POURING CONCRETE PILES/PIERS TO CONFIRM BEARING CAPACITY AND REINFORCING
  - PRIOR TO SHOTCRETING WALLS
  - PRIOR TO STRESSING OF ROCK ANCHORS
  - PRIOR TO EXCAVATION RESUMING AFTER TEMPORARY BRACING STEEL INSTALLED
- VIBRATION MONITORING TO BE CARRIED OUT ON BOUNDARIES IN ACCORDANCE WITH GEOTECHNICAL RECOMMENDATIONS DURING EXCAVATION
- SURVEY POINTS TO BE ESTABLISHED AND LOCATIONS SUBMITTED FOR APPROVAL TO ALL RETAINING WALLS. SURVEY TO BE SUBMITTED TO GEOTECH AND STRUCTURAL ENGINEER TO MONITOR MOVEMENTS. SURVEY TO BE CARRIED OUT AT FOLLOWING STAGES:
  - COMPLETION OF TOP RP2 PILE INSTALLATION
  - COMPLETION OF EXCAVATION STAGE 1
  - PRIOR TO ROCK ANCHOR STRESSING
  - COMPLETION OF ROCK ANCHOR STRESSING AND TEMPORARY PROP INSTALLATION
  - ONCE EXCAVATION ACHIEVES ~RL1381.94
  - ONCE EXCAVATION IS COMPLETED



NOTE:

- EXCAVATION TO NOT EXCEED 1.5m IN ONE GO.
- EACH 1.5m EXCAVATION TO BE INSPECTED BY A COMPETENT GETOECHNICAL ENGIEER AND SIGNED OFF PRIOR TO PROGRESSING EXCAVATION TO FURTHER DEPTH

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PROJECT TITLE:	BLACK BEAR INN	1	15.09.2021	ISSUED FOR CC	THOMAS WILLIAMS	PRE0001122
CONSENT NUMBER:		2	07.10.2021	FOR CONSTRUCTION	THOMAS WILLIAMS	PRE0001122
		3	16.11.2021	REVISED FOR ANCHORAGES	THOMAS WILLIAMS	PRE0001122
		4	23.11.2021	RL CHANGES	THOMAS WILLIAMS	PRE0001122
DRAWING TITLE: EXCAVATION DETAILS - 2	JOB NUMBER: PMI-2021-053	5	01.02.2022	REVISED FOR PARTICULARS OF REGULATED DESIGN - GROUND ANCHORS	THOMAS WILLIAMS	PRE0001122
SCALE AT B1: 1 : 50	DRAWING NUMBER: S10b REVISION: 6	6	29.04.2022	DIFFERENTIATION BETWEEN BUILT AND UNBUILT WORKS	THOMAS WILLIAMS	PRE0001122

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#SCHEDULE - P - RETAINING

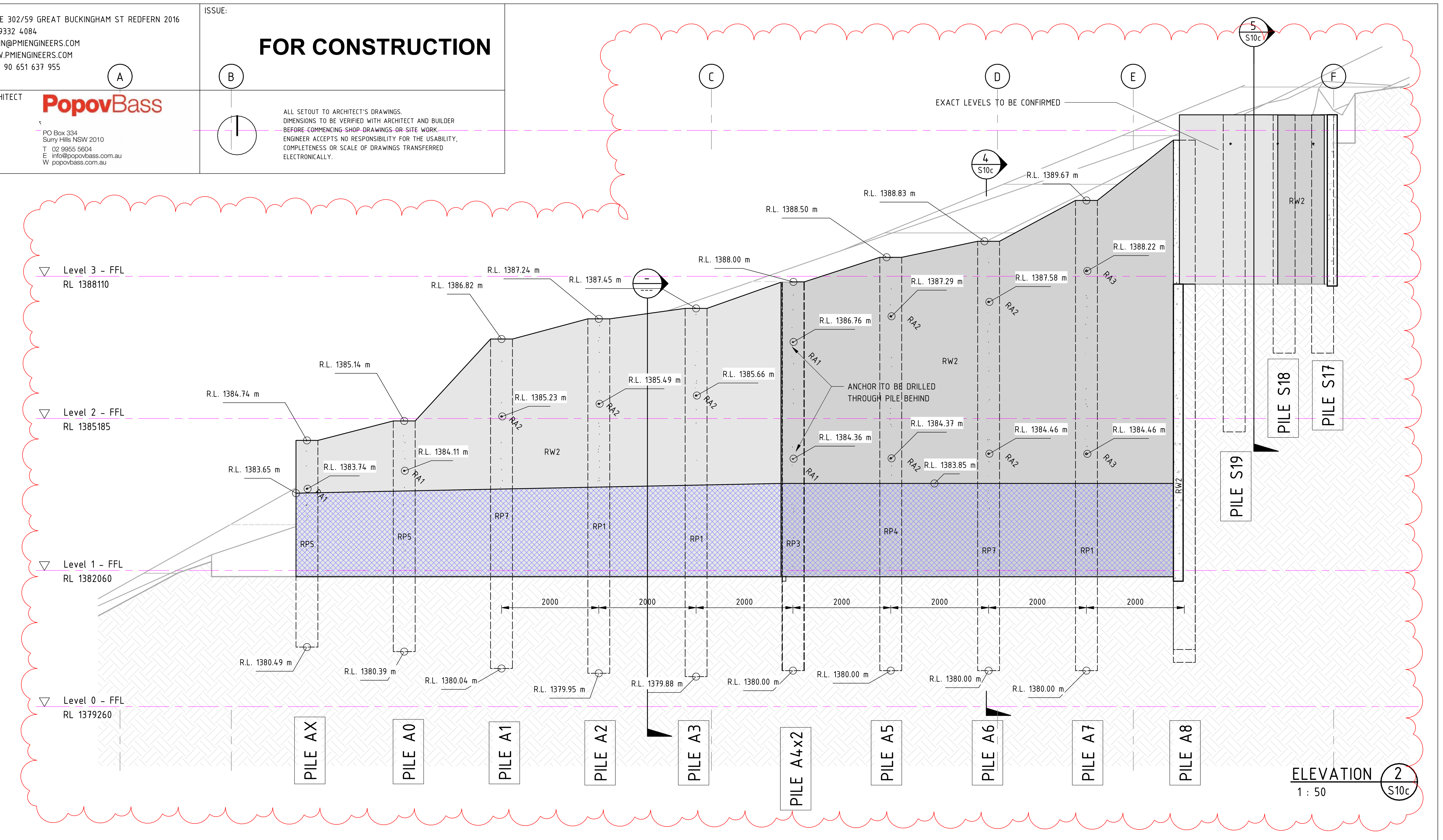
Type Mark	Description
ANCHORS	
RA1	26.5mm DYWIDAG Y1050H PRESTRESSING STEEL BAR - OR OTHER APPROVED - SEE ACCOMPANYING SHEET FOR LOADS
RA2	32mm DYWIDAG Y1050H PRESTRESSING STEEL BAR - OR OTHER APPROVED - SEE ACCOMPANYING SHEET FOR LOADS
RA3	36mm DYWIDAG Y1050H PRESTRESSING STEEL BAR - OR OTHER APPROVED - SEE ACCOMPANYING SHEET FOR LOADS
FOUNDATIONS	
CB8	600Wx400D CAPPING BEAM TO ROAD - 3N20s TOP & BTM with N12 STIRRUPS @ 300 CRS
RETAINING SYSTEM	
RP1	450 DIA PIER REINFORCED WITH 6/N28s @ N12 SPIRAL @ 250 PITCH
RP2	450 DIA PIER REINFORCED WITH 4/N16s @ N10 SPIRAL @ 300 PITCH
RP3	450 DIA PIER REINFORCED WITH 4/N20s @ N12 SPIRAL @ 300 PITCH
RP4	450 DIA PIER REINFORCED WITH 4/N24s @ N10 SPIRAL @ 300 PITCH
RP5	450 DIA PIER REINFORCED WITH 4/N16s @ N12 SPIRAL @ 300 PITCH
RP6	450 DIA PIER REINFORCED WITH 6/N20s @ N12 SPIRAL @ 300 PITCH
RP7	450 DIA PIER REINFORCED WITH 6/N24s @ N12 SPIRAL @ 300 PITCH
RW1	190 COREFILLED BLOCKWORK WALLS - N16s @ 400 CRS VERTICAL - N12s @ 400 CRS HORIZONTAL - TEMP RESTRAINT REQUIRED AT TOP PRIOR TO SLAB OVER BEING POURED
RW2	200mm 32MPa SHOTCRETE WALLS - SEE S10 FOR DETAILS

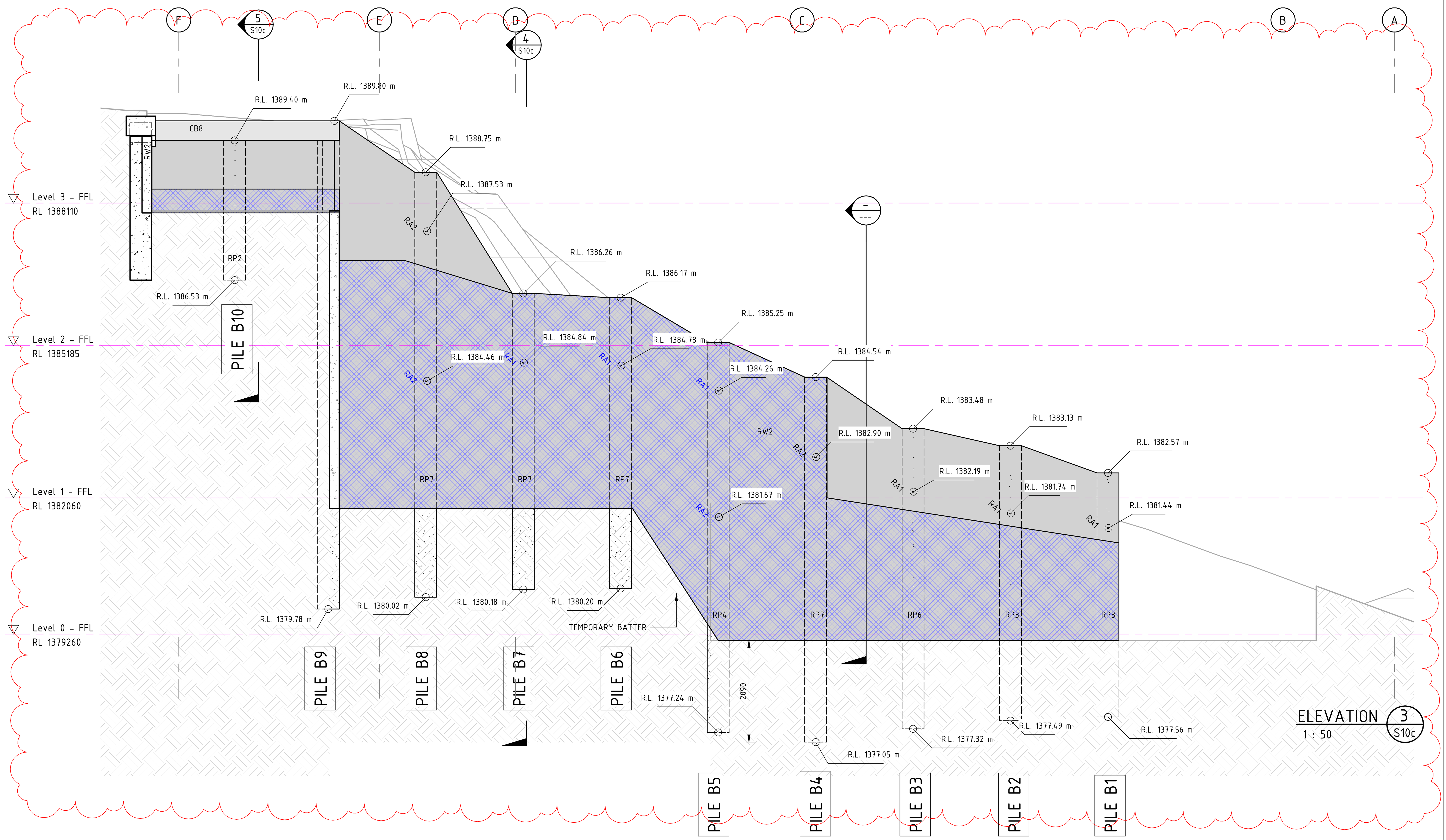
SHOTCRETE WALL SECTIONS PREVIOUSLY POURED

SHOTCRETE WALL SECTIONS TO BE POURED

RA# ROCK ANCHOR PREVIOUSLY INSTALLED

RA# ROCK ANCHOR TO BE INSTALLED





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CONSENT NUMBER:		2	07.10.2021	FOR CONSTRUCTION	THOMAS WILLIAMS	PRE0001122
		3	16.11.2021	REVISED FOR ANCHORAGES	THOMAS WILLIAMS	PRE0001122
		4	01.02.2022	REVISED FOR PARTICULARS OF REGULATED DESIGN - GROUND ANCHORS	THOMAS WILLIAMS	PRE0001122
DRAWING TITLE **EXCAVATION DETAILS - 3**	JOB NUMBER **PMI-2021-053**	5	29.04.2022	DIFFERENTIATION BETWEEN BUILT AND UNBUILT WORKS	THOMAS WILLIAMS	PRE0001122
SCALE AT B1: As indicated	DRAWING NUMBER **S10c** REVISION **5**					

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ISSUE:

FOR CONSTRUCTION

ALL SETOUT TO ARCHITECT'S DRAWINGS.
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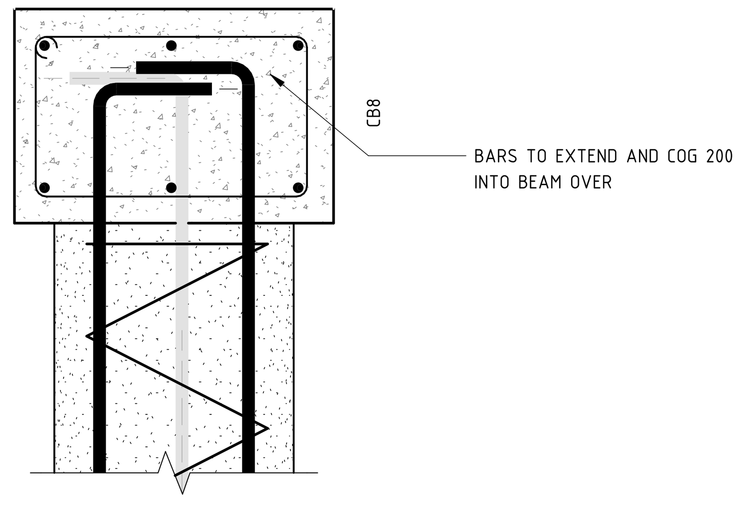
SHOTCRETE WALL SECTIONS PREVIOUSLY POURED

SHOTCRETE WALL SECTIONS TO BE POURED

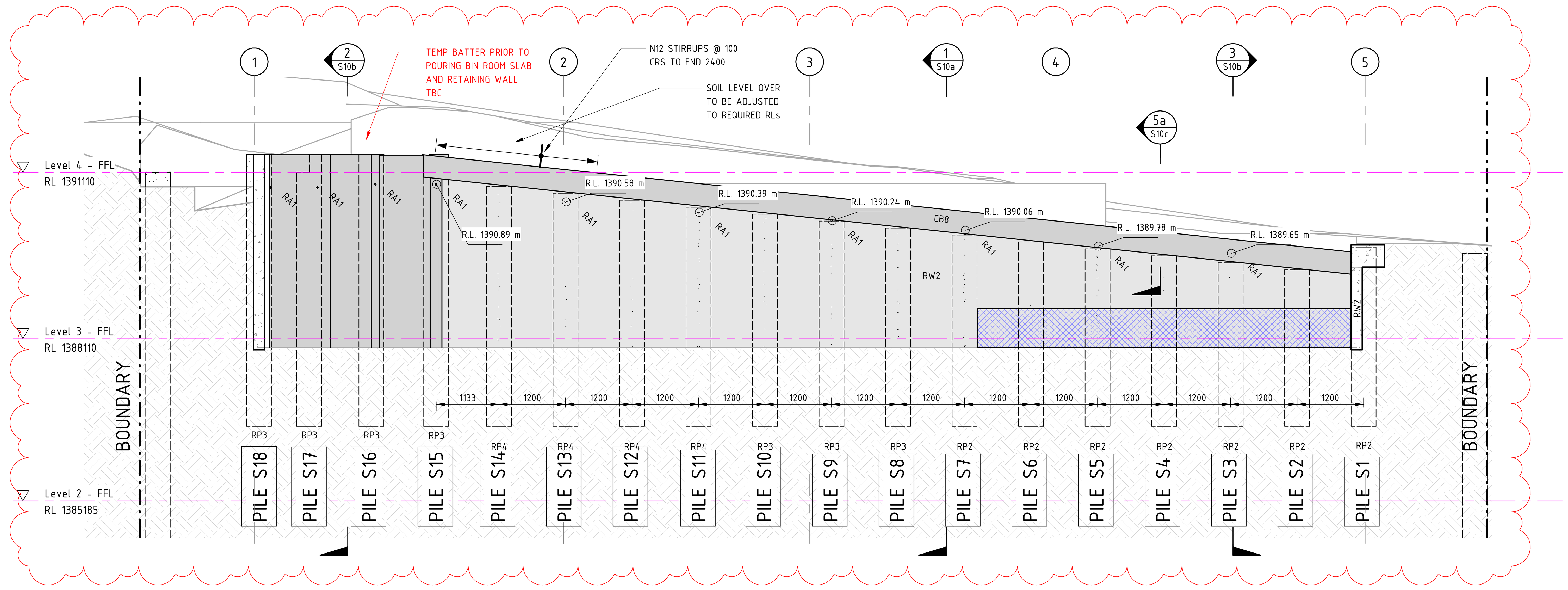
RA# ROCK ANCHOR PREVIOUSLY INSTALLED

RA# ROCK ANCHOR TO BE INSTALLED

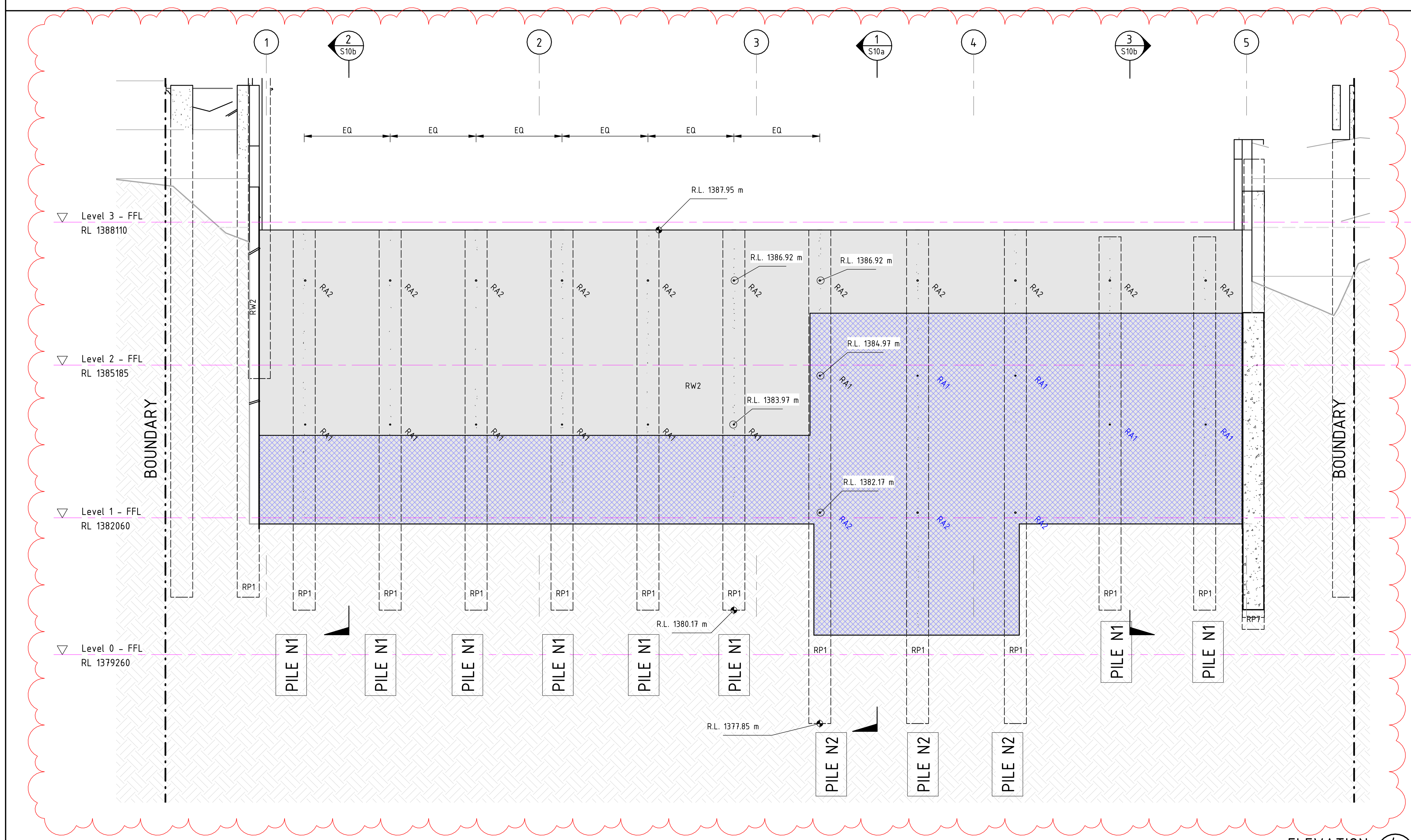


TYPICAL CAPPING BEAM CONNECTION

DETAIL 5a S10a
1 : 10



ELEVATION 5 S10a
1 : 50



ELEVATION 4 S10a
1 : 50

REGULATED DESIGN RECORD

PROJECT ADDRESS: 30 DIGGINGS TERRACE, THREDBO
PROJECT TITLE: BLACK BEAR INN
CONSENT NUMBER:

REV	DATE	DESCRIPTION	DP FULL NAME	REG NO
1	01.02.2022	REVISED FOR PARTICULARS OF REGULATED DESIGN – GROUND ANCHORS	THOMAS WILLIAMS	PRE0001122
2	28.02.2022	CONSOLIDATED SHEETS FOR DA SUBMISSION	THOMAS WILLIAMS	PRE0001122
3	29.04.2022	DIFFERENTIATION BETWEEN BUILT AND UNBUILT WORKS	THOMAS WILLIAMS	PRE0001122

DRAWING TITLE
PILING PLAN

SCALE AT B1: 1 : 50

JOB NUMBER
PMI-2021-053

DRAWING NUMBER **S10d** REVISION **3**

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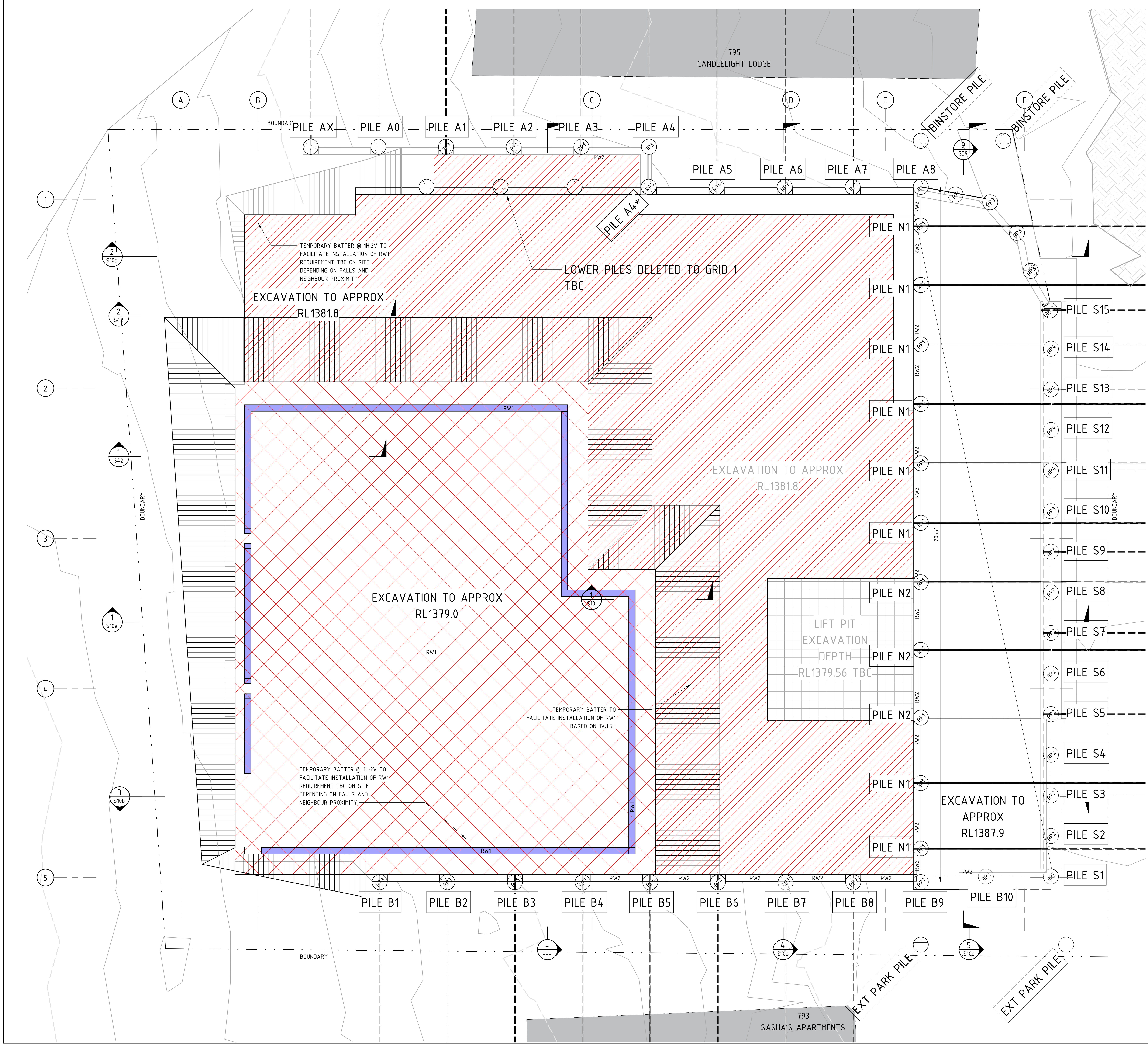

NOTE:
ALL RETENTION PILES HAVE BEEN COMPLETED
(RP1/RP2/RP3/RP4/RP5/RP6/RP7)
NO INTERNAL RETENTION WALLS (RW1s) HAVE BEEN CONSTRUCTED

ANCHOR SCHEDULE

IDENTIFIER	TYPE MARK	DIAMETER	LENGTH (mm)	ANCHOR RL	ANGLE	WORKING LOAD (kN)	TEST LOAD (kN)	LOCK OFF LOAD (kN)	MIN EXTENSION – TEST LOAD (mm)	MAX EXTENSION – TEST LOAD (mm)	INSTALLED
A0	RA1	26.5mm	6600	1384.12	30°	130	270	130	7.16	11.46	YES
A1	RA2	32mm	10900	1385.24	30°	290	580	290	10.55	24.45	YES
A2	RA2	32mm	12200	1385.50	30°	340	680	340	12.37	31.35	YES
A3	RA2	32mm	12900	1385.67	30°	360	730	360	13.28	35.20	YES
A4-1	RA1	26.5mm	9800	1386.77	30°	300	500	300	13.27	28.30	YES
A4-2	RA1	26.5mm	10100	1384.37	17.5°	320	520	320	13.80	30.12	YES
A5-1	RA2	32mm	10500	1387.30	30°	330	550	330	10.01	22.52	YES
A5-2	RA2	32mm	11100	1384.38	17.5°	360	590	360	10.74	25.23	YES
A6-1	RA2	32mm	11200	1387.60	30°	360	600	360	10.92	25.84	YES
A6-2	RA2	32mm	11900	1384.48	17.5°	390	650	390	11.83	29.37	YES
A7-1	RA3	36mm	13900	1388.24	30°	480	800	480	11.50	32.40	YES
A7-2	RA3	36mm	13900	1384.48	17.5°	480	800	480	11.50	32.40	YES
AX	RA1	26.5mm	6000	1383.75	30°	110	220	110	5.84	8.76	YES
B1	RA1	26.5mm	7200	1381.45	30°	150	310	150	8.23	13.98	YES
B2	RA1	26.5mm	8600	1381.75	30°	210	410	210	10.88	21.03	YES
B3	RA1	26.5mm	9100	1382.20	30°	220	450	220	11.94	24.08	YES
B4	RA2	32mm	12300	1382.91	30°	340	680	340	12.37	31.55	YES
B5-1	RA1	26.5mm	9000	1384.27	30°	270	440	270	11.67	23.35	NO
B5-2	RA2	32mm	10400	1381.68	15°	330	540	330	9.83	21.94	NO
B6	RA1	26.5mm	9100	1384.79	30°	220	450	220	11.94	24.08	NO
B7	RA1	26.5mm	9300	1384.85	30°	230	470	230	12.47	25.56	NO
B8-1	RA2	32mm	11100	1387.55	30°	360	590	360	10.74	25.23	YES
B8-2	RA2	32mm	11600	1384.48	15°	380	630	380	11.46	27.89	NO
N1-1	RA2	32mm	11900	1386.93	15°	390	650	390	11.83	29.37	YES
N1-2	RA1	26.5mm	8800	1383.98	10°	260	420	260	11.14	21.92	PARTIAL
N2-1	RA2	32mm	13000	1386.93	15°	440	730	440	13.28	35.42	YES
N2-2	RA1	26.5mm	8200	1384.98	10°	230	380	230	10.08	18.82	PARTIAL
N2-3	RA2	32mm	12600	1382.18	10°	420	700	420	12.74	33.12	NO
S3	RA1	26.5mm	6000	1389.66	30°	140	220	140	5.84	8.76	YES
S5	RA1	26.5mm	6400	1389.79	30°	150	250	150	6.63	10.39	YES
S7	RA1	26.5mm	7400	1390.07	30°	200	320	200	8.49	14.72	YES
S9	RA1	26.5mm	8100	1390.25	30°	230	370	230	9.82	18.16	YES
S11	RA1	26.5mm	8900	1390.40	30°	260	430	260	11.41	22.63	YES
S13	RA1	26.5mm	9200	1390.59	30°	270	450	270	11.94	24.28	YES
S15	RA1	26.5mm	8800	1390.91	30°	260	420	260	11.14	21.92	YES

REGULATED DESIGN RECORD

PROJECT ADDRESS: 30 DIGGINGS TERRACE, THREDBO
PROJECT TITLE: BLACK BEAR INN
CONSENT NUMBER:

REV	DATE	DESCRIPTION	DP FULL NAME	REG NO
1	01.02.2022	REVISED FOR PARTICULARS OF REGULATED DESIGN – GROUND ANCHORS	THOMAS WILLIAMS	PRE0001122
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ISSUE:

FOR CONSTRUCTION

DRAWING TITLE
ANCHOR SECTIONS

JOB NUMBER
PMI-2021-053

DRAWING NUMBER **S10e** REVISION **3**

SCALE AT B1: 1 : 100

CLIENT:
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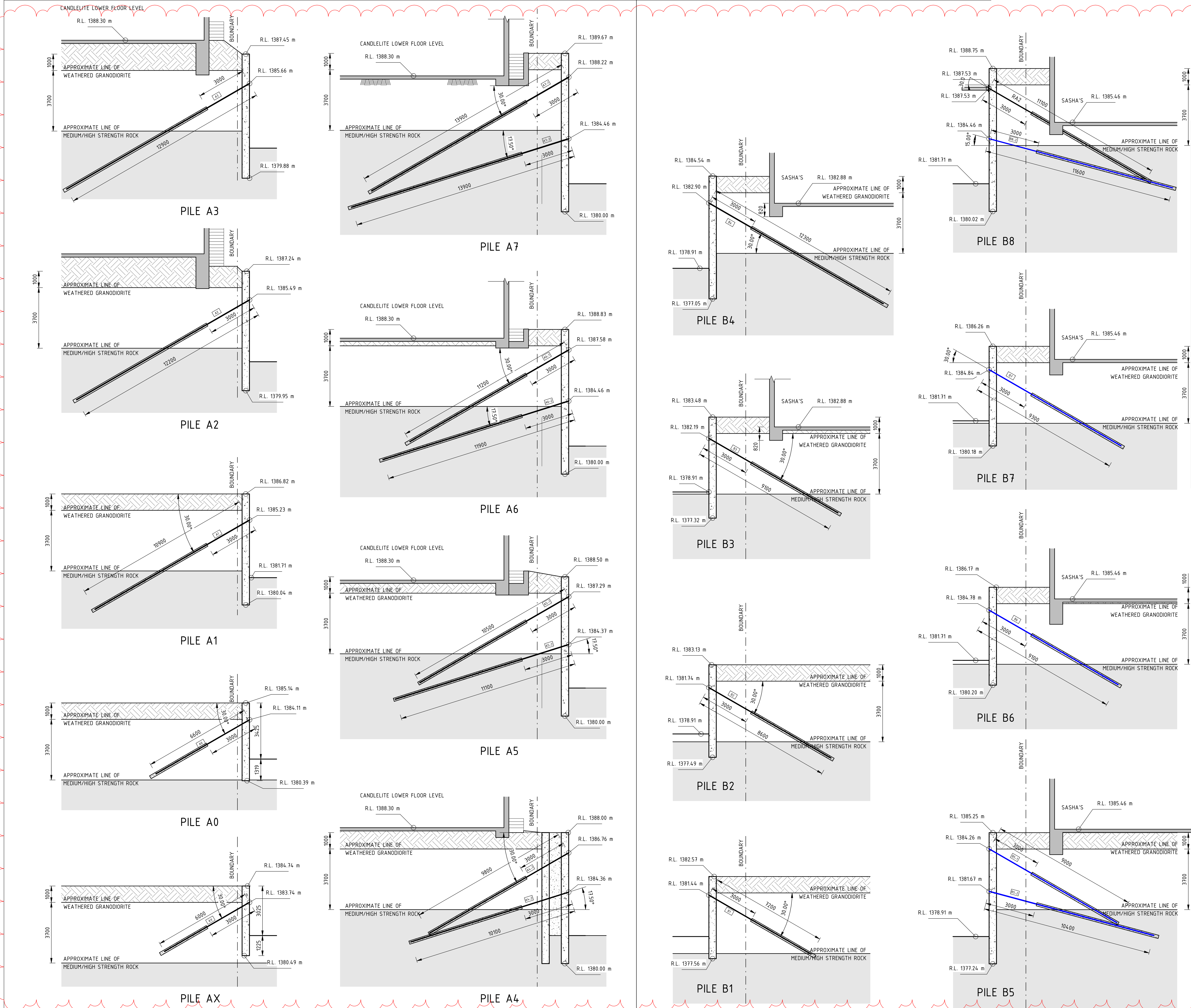

NOTE:

- ALL ANCHORS TO BE TESTED TO TEST LOAD FOR 15 MINUTES AND ANCHOR IS TO BE CONFIRMED HOLDING 'TEST LOAD' FOR THE FULL 15 MIN DURATION
- ANCHOR WORKING LOADS TEST LOADS AND LOCK-OFF LOADS ARE TO BE IN ACCORDANCE WITH SCHEDULE BELOW.
- ANCHORS TO BE DYWIDAG Y1050H PRESTRESSING STEEL BAR OR SIMILAR APPROVED
- ALL ANCHORS HOLES TO BE 125mm DIA MINIMUM
- ANCHOR BARS ARE TO BE BLACK STEEL WITH NO CORROSION PROTECTION / SHEATHING REQUIRED DUE TO TEMPORARY NATURE
- NO FIRE TREATMENT IS REQUIRED FOR TEMPORARY ANCHORS

TOLERANCES:
- ALL ANCHORS TO BE LOCATED WITHIN 250mm OF THE STATED RL
- WITHIN 5 DEG OF STATED ANGLE OFF HORIZONTAL
- ALL ANCHORS TO BE PERPENDICULAR TO EXCAVATION CUT WITHIN 5 DEG
- MINIMUM FREE LENGTH OF ANCHORS OF 3m AS NOTED ON SECTIONS

DESIGN LOADS:
- ALL ANCHORS DESIGNED FOR 8H + SURCHARGE LOADING FROM LIVE LOAD
- LIVE LOAD ASSUMED AS 5kPA FOR EAST AND WEST SIDE OF SITE
- LIVE LOAD ASSUMED AS 10kPA FOR SOUTHERN SIDE OF SITE

ANCHOR WORKING LOADS:
- WORKING LOAD SPECIFIED AS LOAD RESULTING FROM LIVE LOAD + 6H DEAD LOAD
- TEST LOAD DEFINED AS LIVE LOAD + 8H DEAD LOAD WITH APPROPRIATE SAFETY FACTORS APPLIED
- ANCHOR LENGTH DESIGN BASED ON 150kPa ULTIMATE BOND STRESS
- MIN EXTENSION BASED ON EXTENSION OVER 3m FREE LENGTH ONLY
- MAX EXTENSION BASED ON EXTENSION OVER 3m FREE LENGTH + 1/2 BONDED LENGTH

ANCHOR SCHEDULE

IDENTIFIER	TYPE MARK	DIAMETER	LENGTH (mm)	ANCHOR RL	ANGLE	WORKING LOAD (kN)	TEST LOAD (kN)	LOCK OFF LOAD (kN)	MIN EXTENSION – TEST LOAD (mm)	MAX EXTENSION – TEST LOAD (mm)	INSTALLED
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A2	RA2	32mm	12200	1385.50	30°	340	680	340	12.37	31.35	YES
A3	RA2	32mm	12900	1385.67	30°	360	730	360	13.28	35.20	YES
A4-1	RA1	26.5mm	9800	1386.77	30°	300	500	300	13.27	28.30	YES
A4-2	RA1	26.5mm	10100	1384.37	17.5°	320	520	320	13.80	30.12	YES
A5-1	RA2	32mm	10500	1387.30	30°	330	550	330	10.01	22.52	YES
A5-2	RA2	32mm	11100	1384.38	17.5°	360	590	360	10.74	25.23	YES
A6-1	RA2	32mm	11200	1387.60	30°	360	600	360	10.92	25.84	YES
A6-2	RA2	32mm	11900	1384.48	17.5°	390	650	390	11.83	29.37	YES
A7-1	RA3	36mm	13900	1388.24	30°	480	800	480	11.50	32.40	YES
A7-2	RA3	36mm	13900	1384.48	17.5°	480	800	480	11.50	32.40	YES
AX	RA1	26.5mm	6000	1383.75	30°	110	220	110	5.84	8.76	YES
B1	RA1	26.5mm	7200	1381.45	30°	150	310	150	8.23	13.98	YES
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B3	RA1	26.5mm	9100	1382.20	30°	220	450	220	11.94	24.08	YES
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B5-1	RA1	26.5mm	9000	1384.27	30°	270	440	270	11.67	23.35	NO
B5-2	RA2	32mm	10400	1381.68	15°	330	540	330	9.83	21.94	NO
B6	RA1	26.5mm	9100	1384.79	30°	220	450	220	11.94	24.08	NO
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B8-1	RA2	32mm	11100	1387.55	30°	360	590	360	10.74	25.23	YES
B8-2	RA2	32mm	11600	1384.48	15°	380	630	380	11.46	27.89	NO
N1-1	RA2	32mm	11900	1386.93	15°	390	650	390	11.83	29.37	YES
N1-2	RA1	26.5mm	8800	1383.98	10°	260	420	260	11.14	21.92	PARTIAL
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N2-2	RA1	26.5mm	8200	1384.98	10°	230	380	230	10.08	18.82	PARTIAL
N2-3	RA2	32mm	12600	1382.18	10°	420	700	420	12.74	33.12	NO
S3	RA1	26.5mm	6000	1389.66	30°	140	220	140	5.84	8.76	YES
S5	RA1	26.5mm	6400	1389.79	30°	150	250	150	6.63	10.39	YES
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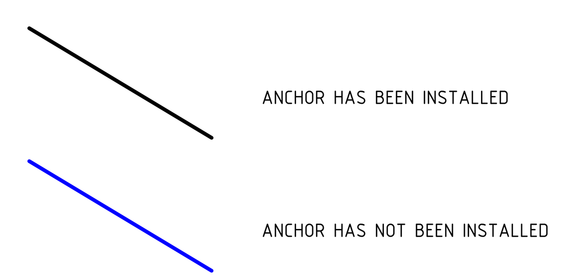
S9	RA1	26.5mm	8100	1390.25	30°	230	370	230	9.82	18.16	YES
S11	RA1	26.5mm	8900	1390.40	30°	260	430	260	11.41	22.63	YES
S13	RA1	26.5mm	9200	1390.59	30°	270	450	270	11.94	24.28	YES
S15	RA1	26.5mm	8800	1390.91	30°	260	420	260	11.14	21.92	YES



REGULATED DESIGN RECORD		REV	DATE	DESCRIPTION	DP FULL NAME	REG NO
PROJECT ADDRESS:	30 DIGGINGS TERRACE, THREDBO	1	01.02.2022	REVISED FOR PARTICULARS OF REGULATED DESIGN – GROUND ANCHORS	THOMAS WILLIAMS	PRE0001122
PROJECT TITLE:	BLACK BEAR INN	2	28.02.2022	CONSOLIDATED SHEETS FOR DA SUBMISSION	THOMAS WILLIAMS	PRE0001122
CONSENT NUMBER:		3	29.04.2022	DIFFERENTIATION BETWEEN BUILT AND UNBUILT WORKS	THOMAS WILLIAMS	PRE0001122

DRAWING TITLE
ANCHOR SECTIONS

JOB NUMBER
PMI-2021-053

DRAWING NUMBER **S10f** REVISION **3**

SCALE AT B1: 1 : 100

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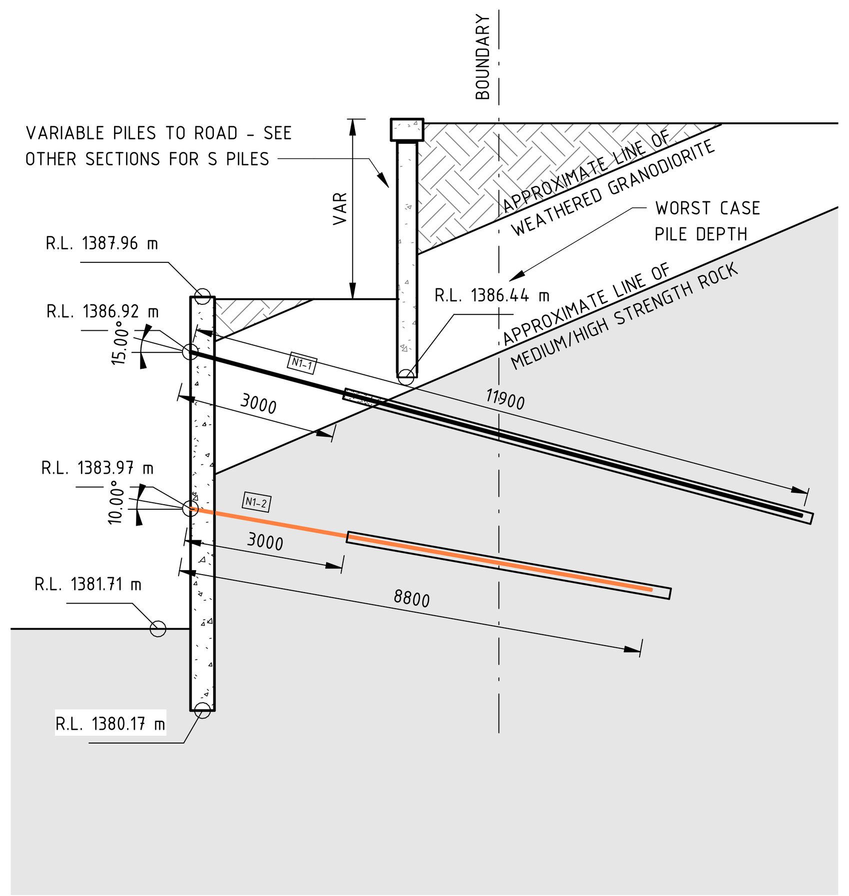
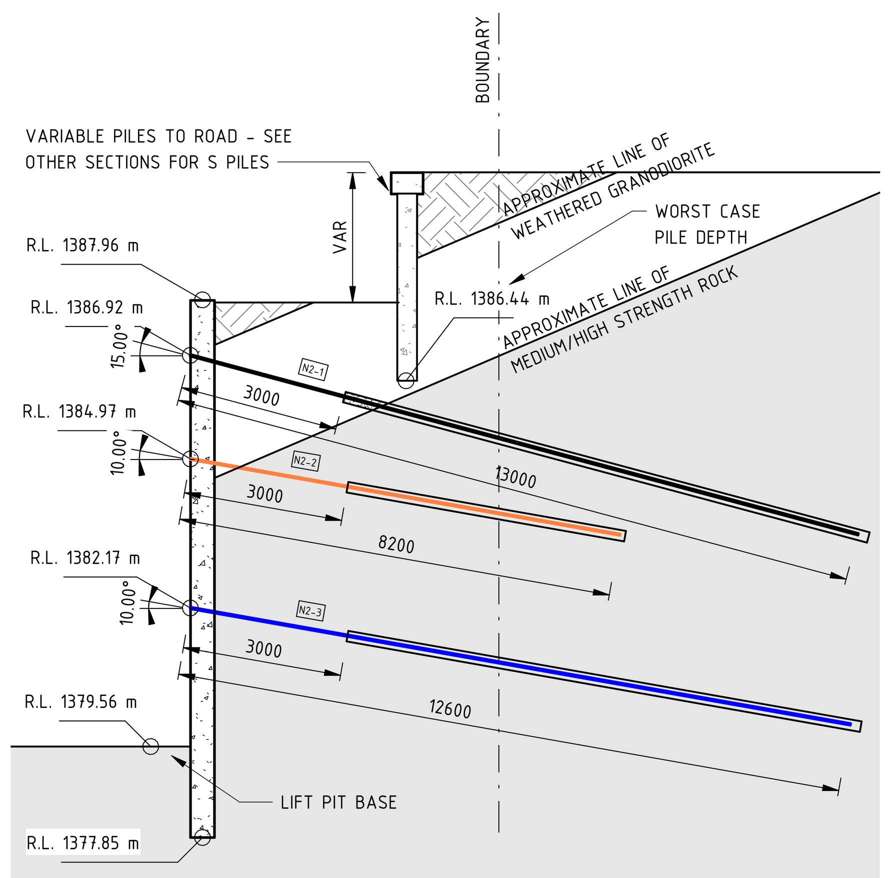


PILES N1



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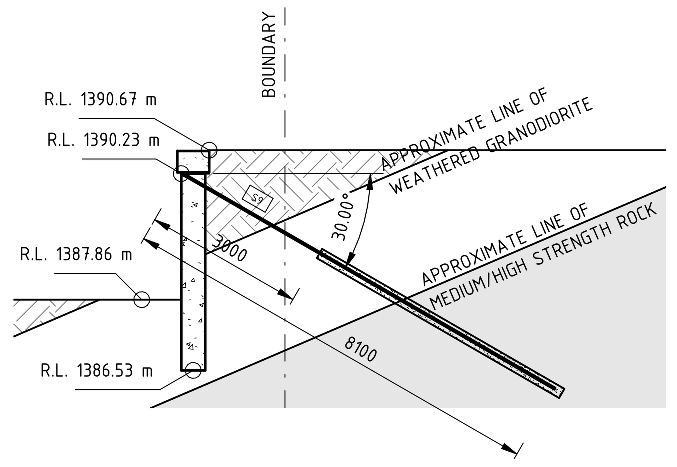
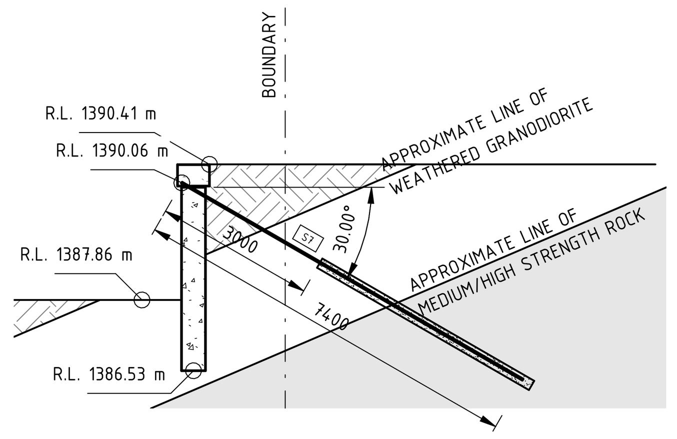


PILE S9



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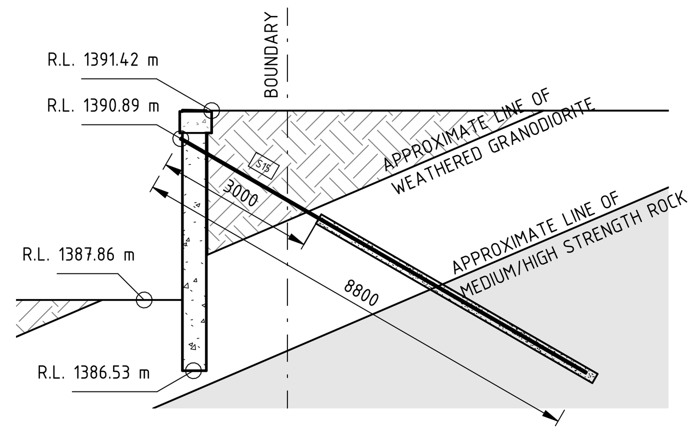

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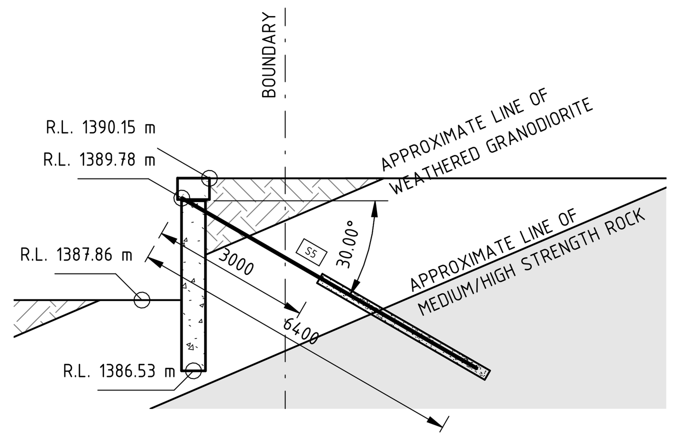

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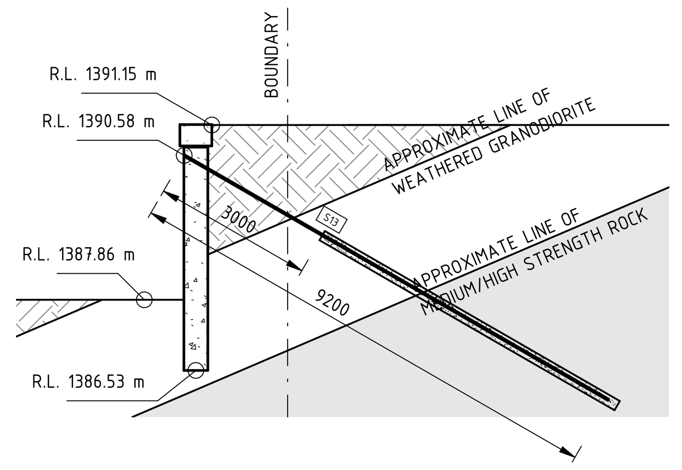

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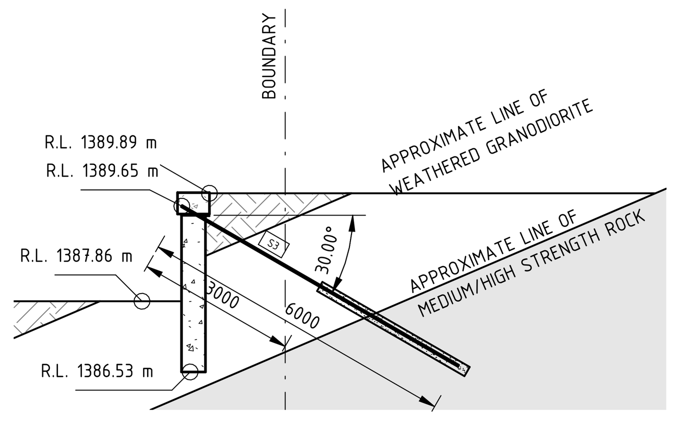
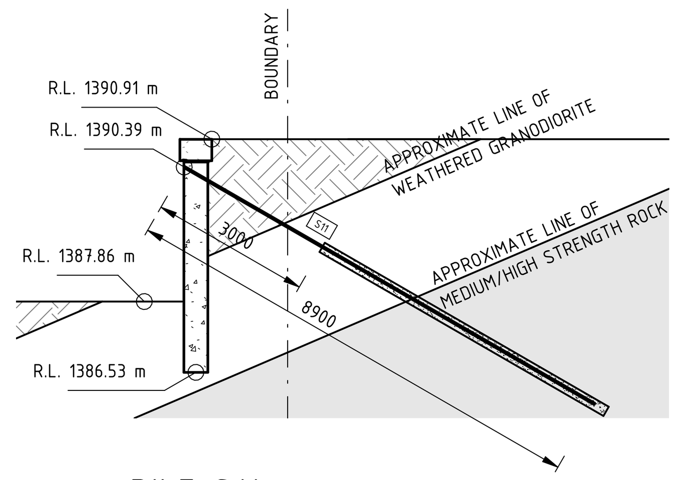


PILE S3



PILE S11

NOTE:

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- ALL ANCHORS HOLES TO BE 125mm DIA MINIMUM
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TOLERANCES:

- ALL ANCHORS TO BE LOCATED WITHIN 250mm OF THE STATED RL
- WITHIN 5 DEG OF STATED ANGLE OFF HORIZONTAL
- ALL ANCHORS TO BE PERPENDICULAR TO EXCAVATION CUT WITHIN 5 DEG
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DESIGN LOADS:

- ALL ANCHORS DESIGNED FOR 8H + SURCHARGE LOADING FROM LIVE LOAD
- LIVE LOAD ASSUMED AS 5kPA FOR EAST AND WEST SIDE OF SITE
- LIVE LOAD ASSUMED AS 10kPA FOR SOUTHERN SIDE OF SITE

ANCHOR WORKING LOADS:

- WORKING LOAD SPECIFIED AS LOAD RESULTING FROM LIVE LOAD + 6H DEAD LOAD
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ANCHOR SCHEDULE

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A3	RA2	32mm	12900	1385.67	30°	360	730	360	13.28	35.20	YES
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A5-1	RA2	32mm	10500	1387.30	30°	330	550	330	10.01	22.52	YES
A5-2	RA2	32mm	11100	1384.38	17.5°	360	590	360	10.74	25.23	YES
A6-1	RA2	32mm	11200	1387.60	30°	360	600	360	10.92	25.84	YES
A6-2	RA2	32mm	11900	1384.48	17.5°	390	650	390	11.83	29.37	YES
A7-1	RA3	36mm	13900	1388.24	30°	480	800	480	11.50	32.40	YES
A7-2	RA3	36mm	13900	1384.48	17.5°	480	800	480	11.50	32.40	YES
AX	RA1	26.5mm	6000	1383.75	30°	110	220	110	5.84	8.76	YES
B1	RA1	26.5mm	7200	1381.45	30°	150	310	150	8.23	13.98	YES
B2	RA1	26.5mm	8600	1381.75	30°	210	410	210	10.88	21.03	YES
B3	RA1	26.5mm	9100	1382.20	30°	220	450	220	11.94	24.08	YES
B4	RA2	32mm	12300	1382.91	30°	340	680	340	12.37	31.55	YES
B5-1	RA1	26.5mm	9000	1384.27	30°	270	440	270	11.67	23.35	NO
B5-2	RA2	32mm	10400	1381.68	15°	330	540	330	9.83	21.94	NO
B6	RA1	26.5mm	9100	1384.79	30°	220	450	220	11.94	24.08	NO
B7	RA1	26.5mm	9300	1384.85	30°	230	470	230	12.47	25.56	NO
B8-1	RA2	32mm	11100	1387.55	30°	360	590	360	10.74	25.23	YES
B8-2	RA2	32mm	11600	1384.48	15°	380	630	380	11.46	27.89	NO
N1-1	RA2	32mm	11900	1386.93	15°	390	650	390	11.83	29.37	YES
N1-2	RA1	26.5mm	8800	1383.98	10°	260	420	260	11.14	21.92	PARTIAL
N2-1	RA2	32mm	13000	1386.93	15°	440	730	440	13.28	35.42	YES
N2-2	RA1	26.5mm	8200	1384.98	10°	230	380	230	10.08	18.82	PARTIAL
N2-3	RA2	32mm	12600	1382.18	10°	420	700	420	12.74	33.12	NO
S3	RA1	26.5mm	6000	1389.66	30°	140	220	140	5.84	8.76	YES
S5	RA1	26.5mm	6400	1389.79	30°	150	250	150	6.63	10.39	YES
S7	RA1	26.5mm	7400	1390.07	30°	200	320	200	8.49	14.72	YES
S9	RA1	26.5mm	8100	1390.25	30°	230	370	230	9.82	18.16	YES
S11	RA1	26.5mm	8900	1390.40	30°	260	430	260	11.41	22.63	YES
S13	RA1	26.5mm	9200	1390.59	30°	270	450	270	11.94	24.28	YES
S15	RA1	26.5mm	8800	1390.91	30°	260	420	260	11.14	21.92	YES

- ANCHOR HAS BEEN INSTALLED
- PARTIAL ANCHORS HAVE BEEN INSTALLED FROM NAMING GROUP - REFER TO S10c ELEVATIONS
- ANCHOR HAS NOT BEEN INSTALLED

APPENDIX C – FORM 1 DECLARATION & CERTIFICATION

Geotechnical Policy
Kosciuszko Alpine Resorts

Form 1 – Declaration and certification made by geotechnical engineer or engineering geologist in a geotechnical report.

DA Number: DA22/4825

To be submitted with a development application

You can use Form 1 to verify that the author of a geotechnical report is a geotechnical engineer or engineering geologist as defined by the Department of Planning & Environment (DP&E) Geotechnical Policy. Alternatively, where a geotechnical report has been prepared by a professional person not recognised by DP&E Geotechnical Policy, then Form 1 may be used as technical verification of the geotechnical report if signed by a geotechnical engineer or engineering geologist as defined by the DP&E Geotechnical Policy.

Please contact the Alpine Resorts Team in Jindabyne for further information - phone 02 6456 1733.

To complete this form, please place a cross in the appropriate boxes ☐ and complete all sections.

1. Declaration made by geotechnical engineer or engineering geologist as part of a geotechnical report

I,

Mr ☒ Ms ☐ Mrs ☐ Dr ☐ Other ☐

First Name: Mark
Family Name: Green

OF

Company/organisation: Alliance Geotechnical

on this the 3rd day of May 2022

certify that I am a geotechnical engineer or engineering geologist as defined by the "Policy" and I (tick appropriate box)

☒ prepared the geotechnical report referenced below in accordance with the AGS 2000 and DP&E Geotechnical Policy – Kosciuszko Alpine Resorts.

☒ am willing to technically verify that the Geotechnical Report referenced below has been prepared in accordance the AGS 2000 and DP&E Geotechnical Policy – Kosciuszko Alpine Resorts.

2. Geotechnical Report Details

Report Title: Geotechnical Investigation Report Ref: 13526-GR-1-1 Rev E

Author: Harshan Panchalingam / Mark Green

Dated: 3/5/2022

DA Site Address: 30 Diggings Terrace, Threadbo NSW

DA Applicant: Bellevarde Constructions Pty Ltd / Hidali Pty Ltd

I am aware that the Geotechnical Report I have either prepared or am technically verifying, (referenced above) is to be submitted in support of a development application for the proposed development site (referenced above), and it's findings will be relied upon by the Consent Authority in determining the development application.

3. Checklist of essential requirements to be contained in a geotechnical risk assessment report to be submitted with a development application

The following checklist covers the minimum requirements to be addressed in a Geotechnical Risk Management Report. This checklist is to accompany the report.

Please tick appropriate box

☒ Risk assessment of all identifiable geotechnical hazards in accordance with AGS 2000, as per 6.1 (a) of the policy.

☒ Site plans with key hazards identified and other information as per 6.1 (b)

☒ Details of site investigation and inspections as per 6.1 (c)

☒ Photographs and/or drawings of the site as per 6.1 (d)

☒ Presentation of geotechnical model as per 6.1 (e)

☒ A specific conclusion as to whether the site is suitable for the development proposed on the above site, if applicable, subject to the following conditions;

- ☒ Conditions to be provided to establish design parameters,
- ☒ Conditions to be incorporated into the detailed design to be submitted for the construction certificate,
- ☒ Conditions applying to the construction phase,
- ☒ Conditions relating to ongoing management of the site/structure.

4. Signatures

Signature: [signature]

Chartered professional status: CPEng MIEAus NER RPEQ CGeol FGS JP

Name: Mark Green

Date: 3/5/2022

5. Contact details

Department of Planning & Environment
Alpine Resorts Team
Shop 5A, 19 Snowy River Avenue
PO Box 36, JINDABYNE 2627
Telephone: 02 6456 1733
Facsimile: 02 6456 1736
Email: alpineresorts@planning.nsw.gov.au

APPENDIX D – GEOTECHNICAL RESPONSE STATEMENT TO DPE REQUEST FOR ADDITIONAL INFORMATION, DATED 4 MAY 2022

alliance

Phone: 1800 288 188
Email: office@allgeo.com.au
Website: www.allgeo.com.au

HIDALI PTY LTD
11 Fitzroy St, Forrest
ACT 2603
Attention: Mr John Fielding

Project: Black Bear Inn
Site Location: 30 Diggings Terrace, Thredbo NSW
Reference: 13526-GR-6-1
Report Date: 4 May 2022

Re: Geotechnical Response to Point 1D of the Project SEE & Points 5B & 5C of the Public Enquiry Document: -Temporary Ground Anchors-

1 Introduction

Alliance Geotechnical Pty Ltd (Alliance) was engaged by Hadali Pty Ltd (the client) to provide a brief geotechnical statement in response to the Request for Information (RFI) from NSW Department of Planning and Environment in relation to development application (DA) no. 22/4825.

2 Supplied Documents

To assist in background to the project, Alliance was supplied with the following documents:

- Letter from NSW Department of Planning and Environment, ref EF22/4825 from Daniel James. “Request for additional information” re DA No: 22/4825 (PAN-204581)
- Latest Structural drawings from PMI Engineers, ref PMI-2021-053,
  - S02 – A rev 1 dated 29/11/21
  - S10 rev 5 – dated 28/2/22
  - S10a rev 5 – dated 29/4/22
  - S10b rev 6 – dated 29/4/22
  - S10c rev 5 – dated 29/4/22
  - S10d rev 3 – dated 29/4/22
  - S10e rev 3 – dated 29/4/22
  - S10f rev 3 – dated 29/4/22

3 Temporary Ground Anchors

To assist in an understanding of the potential impacts of the temporary ground anchors (aka. temporary rock anchors) to accompany the Statement of Environmental Effects (SEE) (ref Point 1D of the SEE and Points 5B & 5C of the Public Enquiry response) we would like to address this in two parts considering the temporary condition and permanent condition cases.

3.1 Temporary Case

Temporary ground anchors are proposed as part of this referenced DA application. The anchors are formed of steel bars encased in cast insitu cementitious grout within cored angled boreholes. The method of

installation only produces low levels of vibration and hence imparts very low engineering impact on adjacent structures or road infrastructure (this is managed by vibration monitoring with geophones should the adjacent structures be considered to be vulnerable). Ground anchors have a low environmental impact.
The risks of installation may include

- the striking of buried services (controlled and managed by reference to Dial Before You Dig searches and scanning of the ground by a registered services locator and direct observation by potholing if required).
- Collapse of bores – for this site the ground conditions consist of competent decomposed granite derived soils and weathered granite bedrock that is sufficiently cohesive to stand open with risk of collapse.
- Once the grout has set, the anchor is nominally stressed to take up the load, hence reducing the risk of lateral deflection of the shoring wall as further excavation proceeds. Internal propping conversely requires the shoring wall to move for it to take up load, so ground anchors are considered to be a better solution with a lower level of impact on adjacent structures and roads.
- As these are temporary anchors, the risk of creep movement (longitudinal extension of the anchor or grout interface) is of very low impact.

3.2 Permanent Case

Once the shoring system is complete, the internal substructure and the superstructure can then be constructed and completed. On completion, the temporary ground anchors are de-stress by loosening off of the head bolts and removing the face plates. The remaining inert bars remain in the ground. These cause no long-term impact. If they corrode (which away from the face is unlikely due to the lack of oxygen) there is no risk of voids as the corrosion products are of higher volume than the original steel.

For the interim case, where temporary anchors are left for a longer period due to delays in the construction, there is a slightly increased risk of creep movement. We have put in place ground deflection monitoring (line and level of survey stations with precise levelling) to check from any movement. We consider this to be of very low risk but have addressed it all the same.

The permanent structures of the building provided long term support to the ground again with a very low impact on the adjacent structures and roads.

4 Requirement for Temporary Ground Anchors and Conclusion

Temporary ground anchors are widely used in the construction industry and are designed and built by competent contractors. Their use is considered to be best practice and ensures the stability of the ground during the temporary excavation of basements and the like.

- Internal propping is not preferred due to the increase in risk of shoring wall movement for the internal propping to take up loads. This additional movement may result in an increased risk of foundation settlement in the surrounding properties,
- Internal propping presents an increased operational and safety risks to workers, the shoring wall itself, and surrounding properties, due to a reduce working space within the site footprint caused by large internal propping members, and
- Temporary ground anchors distribute the loading of the shoring wall to (more) various locations. Counter wise, internal propping predominantly relies on single span beams and fixing points. Temporary ground anchors reduce the operational risk of a catastrophic machine strike and shoring wall failure.

- Removal of the internal temporary propping is significantly more difficult once the basement is complete. This is not the case with ground anchors.

It is considered that the necessity of the Temporary Ground Anchor requirement is in response to prevailing site conditions, risk reduction in design and site operations, and best outcomes for site safety.

We also note that (for the record, for the works completed to date);

- Preconstruction condition survey reports have been completed on all surrounding properties and the public domain,
- Dial Before You Dig applications / records were sought,
- Thredbo Service Mapping were sought,
- Onsite Services Assets Locating was completed,
- Vibration monitoring was installed during the process of installing the anchors (and excavation), and
- Ground deflection monitoring is installed

These records can be provided upon request from the builder.

Regards

Mark Green
BSc(Hons) CPEng MIEAus NER RPEQ
APEC IntPE(Aus) CGeol FGS JP
NSW Reg PE/DP (geo)
Principal Geotechnical Engineer