GEOTECHNICAL RISK MANAGEMENT POLICY FOR PITTWATER
FORM NO. 1 – To be submitted with Development Application

Development Application for _____
Name of Applicant

Address of site 2A William Street, Avalon

The following checklist covers the minimum requirements to be addressed in a Geotechnical Risk ***Declaration made by geotechnical engineer or engineering geologist or coastal engineer (where applicable) as part of a geotechnical report***

I, Ben White *on behalf of* **White Geotechnical Group** Pty Ltd
(Insert Name) (Trading or Company Name)

on this the 9/9/25 certify that I am a geotechnical engineer or engineering geologist or coastal engineer as defined by the Geotechnical Risk Management Policy for Pittwater - 2009 and I am authorised by the above organisation/company to issue this document and to certify that the organisation/company has a current professional indemnity policy of at least $10million.

I:
Please mark appropriate box

☒ have prepared the detailed Geotechnical Report referenced below in accordance with the Australia Geomechanics Society's Landslide Risk Management Guidelines (AGS 2007) and the Geotechnical Risk Management Policy for Pittwater - 2009

☒ am willing to technically verify that the detailed Geotechnical Report referenced below has been prepared in accordance with the Australian Geomechanics Society's Landslide Risk Management Guidelines (AGS 2007) and the Geotechnical Risk Management Policy for Pittwater - 2009

☐ have examined the site and the proposed development in detail and have carried out a risk assessment in accordance with Section 6.0 of the Geotechnical Risk Management Policy for Pittwater - 2009. I confirm that the results of the risk assessment for the proposed development are in compliance with the Geotechnical Risk Management Policy for Pittwater - 2009 and further detailed geotechnical reporting is not required for the subject site.

☐ have examined the site and the proposed development/alteration in detail and I am of the opinion that the Development Application only involves Minor Development/Alteration that does not require a Geotechnical Report or Risk Assessment and hence my Report is in accordance with the Geotechnical Risk Management Policy for Pittwater - 2009 requirements.

☐ have examined the site and the proposed development/alteration is separate from and is not affected by a Geotechnical Hazard and does not require a Geotechnical Report or Risk Assessment and hence my Report is in accordance with the Geotechnical Risk Management Policy for Pittwater - 2009 requirements.

☐ have provided the coastal process and coastal forces analysis for inclusion in the Geotechnical Report

Geotechnical Report Details:

Report Title: Geotechnical Report **2A William Street, Avalon**

Report Date: 9/9/25

Author: BEN WHITE

Author's Company/Organisation: **White Geotechnical Group Pty Ltd**

Documentation which relate to or are relied upon in report preparation:

Australian Geomechanics Society Landslide Risk Management March 2007.
White Geotechnical Group company archives.

I am aware that the above Geotechnical Report, prepared for the abovementioned site is to be submitted in support of a Development Application for this site and will be relied on by Pittwater Council as the basis for ensuring that the Geotechnical Risk Management aspects of the proposed development have been adequately addressed to achieve an "Acceptable Risk Management" level for the life of the structure, taken as at least 100 years unless otherwise stated and justified in the Report and that reasonable and practical measures have been identified to remove foreseeable risk.

Signature _____

Name Ben White

Chartered Professional Status MScGEOL AIG., RPGeo

Membership No. 10306

Company **White Geotechnical Group Pty Ltd**

GEOTECHNICAL RISK MANAGEMENT POLICY FOR PITTWATER
FORM NO. 1(a) - Checklist of Requirements for Geotechnical Risk Management Report for Development Application

Development Application for _____
Name of Applicant

Address of site 2A William Street, Avalon

The following checklist covers the minimum requirements to be addressed in a Geotechnical Risk Management Geotechnical Report. This checklist is to accompany the Geotechnical Report and its certification (Form No. 1).

Geotechnical Report Details:

Report Title: Geotechnical Report **2A William Street, Avalon**

Report Date: 9/9/25

Author: BEN WHITE

Author's Company/Organisation: **White Geotechnical Group Pty Ltd**

Please mark appropriate box

- ☒ Comprehensive site mapping conducted 13/11/24 (date)
- ☒ Mapping details presented on contoured site plan with geomorphic mapping to a minimum scale of 1:200 (as appropriate)
- ☒ Subsurface investigation required
  - ☐ No Justification _____
  - ☒ Yes Date conducted 13/11/24
- ☒ Geotechnical model developed and reported as an inferred subsurface type-section
- ☒ Geotechnical hazards identified
  - ☒ Above the site
  - ☒ On the site
  - ☒ Below the site
  - ☐ Beside the site
- ☒ Geotechnical hazards described and reported
- ☒ Risk assessment conducted in accordance with the Geotechnical Risk Management Policy for Pittwater - 2009
  - ☒ Consequence analysis
  - ☒ Frequency analysis
- ☒ Risk calculation
- ☒ Risk assessment for property conducted in accordance with the Geotechnical Risk Management Policy for Pittwater - 2009
- ☒ Risk assessment for loss of life conducted in accordance with the Geotechnical Risk Management Policy for Pittwater - 2009
- ☒ Assessed risks have been compared to "Acceptable Risk Management" criteria as defined in the Geotechnical Risk Management Policy for Pittwater - 2009
- ☒ Opinion has been provided that the design can achieve the "Acceptable Risk Management" criteria provided that the specified conditions are achieved.
- ☒ Design Life Adopted:
  - ☒ 100 years
  - ☐ Other _____ specify
- ☒ Geotechnical Conditions to be applied to all four phases as described in the Geotechnical Risk Management Policy for Pittwater - 2009 have been specified
- ☒ Additional action to remove risk where reasonable and practical have been identified and included in the report.
- ☐ Risk assessment within Bushfire Asset Protection Zone.

I am aware that Pittwater Council will rely on the Geotechnical Report, to which this checklist applies, as the basis for ensuring that the geotechnical risk management aspects of the proposal have been adequately addressed to achieve an "Acceptable Risk Management" level for the life of the structure, taken as at least 100 years unless otherwise stated, and justified in the Report and that reasonable and practical measures have been identified to remove foreseeable risk.

Signature _____

Name Ben White

Chartered Professional Status MScGEOL AIG., RPGeo

Membership No. 222757

Company **White Geotechnical Group Pty Ltd**

GEOTECHNICAL INVESTIGATION:

New Garage and Pool and Lift at **2A William Street, Avalon**

1. Proposed Development

1.1 Construct a garage with gym above and lift on the downhill side of the property by excavating to a maximum depth of ~3.5m.

1.2 Install a pool on the downhill side of the property by excavating to a maximum depth of ~1.5m.

1.3 Details of the proposed development are shown on 20 drawings provided by Jane Edwards Architecture, drawings numbered DA13 Revision D. DA06 to 12, and DA14 to 19 to Revision E. DA00, Revision F. DA01 to DA05 Revision H. All dated 5/9/25.

2. Site Description

2.1 The site was inspected on the 13th November, 2024, and previously in August 2022, and March 2024.

2.2 The property is accessed by a Right of Carriageway (ROW) off the uphill side of William Street and has an E aspect. The block runs lengthways to the NW so the slope is a cross-fall. It is located on the steeply graded upper reaches of a hillslope. The natural slope rises across the property at an average angle of ~23°. The slope above the property eases in grade. The slope below the property continues at similar steep angles.

2.3 At the road frontage, a concrete ROW and driveway run to a garage on the ground floor of the house (Photo 1). Fill for the driveway on the downhill side is supported by a brick retaining wall reaching ~1.8m high which has been constructed with a tilt back into the slope (Photo 2). Stepped settlement cracking up to ~ 8mm

wide was present on the wall face (Photo 3). No deflection was observed and the wall is considered stable. A cut for the driveway and ROW and fill for landscaping below the house is supported by a stone block retaining wall reaching ~1.8m high (Photo 4). This wall will be demolished as part of the proposed works.

The two-story rendered brick house is supported on brick walls. No significant signs of movement were observed in the visible supporting walls. A cut for the house is supported by a concrete block retaining wall reaching ~3.0m high (Photo 5). Behind the house the wall is tilting at the southern end. This has recently been reinforced with 3 ground anchors taken at least 5.0m into the slope. Various cuts and fills above the house for landscaping are supported by retaining walls of dry stack stone, brick, and timber composition and reaching ~2.9m high (Photos 6 & 7). The brick wall which has been constructed with a tilt back into the slope exhibits stepped cracking. However, no deflection was observed. These walls are considered stable.

Where the steep slope above the house is not supported by retaining walls, it is well vegetated. Sandstone boulders are scattered in this location as well as across the easing slope above the subject property (Photos 8 and 9). Most of the boulders were observed to be resting/sufficiently embedded in stable positions. One boulder was observed to be perched on, and overhanging another boulder upslope from the subject property (Photo 10). It has likely remained in this position for hundreds if not thousands of years. The boulder is ~1.8m wide and ~3.0m long, and is overhanging by ~0.9m. The majority of the rock mass is on the uphill side. As such, the balance point of the upper boulder does not appear to be compromised and the boulder is considered to be in a stable position.

3. Geology

The Sydney 1:100 000 Geological Sheet indicates the site is underlain by Hawkesbury Sandstone, although at a residential scale the map is not always accurate. Ground testing and observations on site indicate that the proposed works are underlain by geology which is

consistent with the Narrabeen Group Rocks which are described as interbedded laminite, shale, and quartz to lithic quartz sandstone.

4. Subsurface Investigation

Two hand Auger Holes (AH) were put down to identify the soil materials. Eleven Dynamic Cone Penetrometer (DCP) tests were put down to determine the relative density of the overlying soil and the depth to weathered rock. The locations of the tests are shown on the site plan attached. It should be noted that a level of caution should be applied when interpreting DCP test results. The test will not pass through hard buried objects so in some instances it can be difficult to determine whether refusal has occurred on an obstruction in the profile or on the natural rock surface. This is not expected to have been an issue for this site. But due to the possibility that the actual ground conditions vary from our interpretation there should be allowances in the excavation and foundation budget to account for this. We refer to the appended "Important Information about Your Report" to further clarify. The results are as follows:

AUGER HOLE 1 (~RL67.0) **–** AH1 (Photo 11)

Depth (m)	Material Encountered
0.0 to 0.3	**TOPSOIL**, sandy soil, grey and dark brown, loose, moist, fine to medium grained.
0.3 to 0.7	**RESIDUAL CLAY**, light brown orange, firm to stiff, dry to moist.

End of test @ 0.7m in residual clay. No water table encountered.

GROUND TEST RESULTS ON THE NEXT PAGE

AUGER HOLE 2 (~RL63.7) **–** AH2 (Photo 12)

Depth (m)	Material Encountered
0.0 to 0.2	**TOPSOIL**, silty sandy, brown, fine to medium grained, Dense, dry.
0.2 to 0.6	**RESIDUAL CLAY**, orange, Very Stiff, dry, fine to medium grained.

End of test @ 0.6m auger not progressing through residual clay. No water table encountered.

DCP TEST RESULTS – Dynamic Cone Penetrometer

Equipment: 9kg hammer, 510mm drop, conical tip. Standard: AS1289.6.3.2 - 1997

Depth(m) Blows/0.3m	DCP 1 (~RL67.0)	DCP 2 (~RL68.0)	DCP 3 (~RL68.3)	DCP 4 (~RL64.0)	DCP 5 (~RL64.0)
0.0 to 0.3	2	2	3	5	28
0.3 to 0.6	10	2	3	7	32
0.6 to 0.9	12	2	4	15	#
0.9 to 1.2	15	8	7	27	
1.2 to 1.5	36	30	33	30	
1.5 to 1.8	34	45	#	#	
1.8 to 2.1	46	#			
2.1 to 2.4	24				
2.4 to 2.7	28				
2.7 to 3.0	27				
3.0 to 3.3	44				
3.3 to 3.6	#				
	End of Test @ 3.3m	End of Test @ 1.8m	End of Test @ 1.5m	Refusal on Rock @ 1.3m	Refusal on Rock @ 0.4m

#refusal/end of test. F = DCP fell after being struck showing little resistance through all or part of the interval.

DCP TEST RESULTS CONTINUED ON THE NEXT PAGE

DCP TEST RESULTS – Dynamic Cone Penetrometer						
Equipment: 9kg hammer, 510mm drop, conical tip.					Standard: AS1289.6.3.2 - 1997	
Depth(m) Blows/0.3m	**DCP 6** (~RL61.8)	**DCP 7** (~RL62.8)	**DCP 8** (~RL63.7)	**DCP 9** (~RL63.3)	**DCP 10** (~RL64.0)	**DCP 11** (~RL63.9)
0.0 to 0.3	13	19	10	7	16	10
0.3 to 0.6	21	34	24	10	20	30
0.6 to 0.9	18	26	27	20	#	34
0.9 to 1.2	34	27	26	7		#
1.2 to 1.5	#	36	30	#		
1.5 to 1.8		#	#			
	End of Test @ 1.2m	End of Test @ 1.5m	End of Test @ 1.5m	Refusal on Rock @ 1.0m	Refusal on Rock @ 0.5m	End of Test @ 0.9m

#refusal/end of test. F = DCP fell after being struck showing little resistance through all or part of the interval.

DCP Notes:

DCP1 – End of test @ 3.3m, DCP still very slowly going down, orange brown clay on dry tip.

DCP2 – End of test @ 1.8m, DCP still very slowly going down, orange brown clay and dark brown soil on damp tip.

DCP3 – End of test @ 1.5m, DCP still very slowly going down, orange brown clay and dark brown soil on moist tip.

DCP4 – Refusal on rock @ 1.3m, DCP thudding on rock surface, brown orange shale fragments on dry tip.

DCP5 – Refusal on rock @ 0.4m, DCP thudding on rock surface, grey and red orange shale fragments on dry tip.

DCP6 – End of test @ 1.2m, DCP still very slowly going down, maroon shale on dry tip.

DCP7 – End of test @ 1.5m, DCP still very slowly going down, maroon shale on dry tip.

DCP8 – End of test @ 1.5m, DCP still very slowly going down, orange impact dust on dry tip.

DCP9 – Refusal on Rock @ 1.0m, DCP thudding on rock surface, maroon and white impact dust on dry tip.

DCP10 – Refusal @ 0.5m, DCP thudding on rock surface, clean dry tip.

DCP11 – End of test @ 0.9m, DCP still very slowly going down, orange sandy clay on dry tip.

5. Geological Observations/Interpretation

The natural slope materials are colluvial at the near surface and residual at depth. In the location of the proposed works, the ground materials consist of shallow silty soils over clays. Filling has been placed to a height of ~1.8m on the downhill side of the driveway in the location of the proposed garage, and to a height ~0.6m in the location of the proposed pool. The clays merge into the weathered zone of the underlying shale at depths of between ~0.3m to ~1.0m below the current surface, being deeper due to the presence of filling and a variable weathering profile. The weathered zone is interpreted as Extremely Low to Very Low Strength Shale.

6. Groundwater

Normal ground water seepage is expected to move over the denser and less permeable clay and weathered shale layers in the sub-surface profile. Due to the slope and elevation of the block, the water table is expected to be many metres below the base of the proposed excavations.

7. Surface Water

No evidence of significant surface flows were observed on the property during the inspection. However, due to the steep grade and presence of sandstone boulders on the slope above the property, consideration should be made to improve the drainage of the property. This would likely involve a cut off drain installed across the upper reaches of the site to catch surface flows from the slope above. This will need to be designed by the stormwater engineer in consultation with the geotechnical consultant.

8. Geotechnical Hazards and Risk Analysis

No geotechnical hazards were observed beside the property. The steeply graded slope that rises across the property continuing below and decreasing in grade above is a potential hazard (**Hazard One**). Vibrations generated from the proposed excavations are a potential hazard (**Hazard Two**). The proposed excavations are a potential hazard until retaining structures are

in place (**Hazard Three**). The proposed excavation for the garage and lift undercutting the footings for the house/retaining wall are a potential hazard (**Hazard Four**).

Risk Analysis Summary

HAZARDS	Hazard One	Hazard Two
TYPE	The steep slope that rises across the property continuing below, and decreasing in grade above, failing and impacting on the proposed works.	The vibrations produced during the proposed excavations impacting on the surrounding structures.
LIKELIHOOD	'Unlikely' (10^{-4})	'Possible' (10^{-3})
CONSEQUENCES TO PROPERTY	'Medium' (15%)	'Minor' (10%)
RISK TO PROPERTY	'Low' (2×10^{-5})	'Moderate' (5×10^{-4})
RISK TO LIFE	9.1×10^{-7}/annum	5.3×10^{-7}/annum
COMMENTS	This level of risk is 'ACCEPTABLE', provided the recommendations in **Section 7 & 18** are followed.	This level of risk to property is 'UNACCEPTABLE'. To move risk to 'ACCEPTABLE' levels, the recommendations in **Section 12** are to be followed.

RISK ANALYSIS SUMMARY CONTINUED ON THE NEXT PAGE

HAZARDS	Hazard Three	Hazard Four
TYPE	The excavations collapsing onto the work site before retaining structures are in place.	The proposed excavation for the garage and lift undercutting the footings of the house (Photo 13) and brick retaining wall (Photo 2), causing damage or failure.
LIKELIHOOD	'Possible' (10^{-3})	'Possible' (10^{-3})
CONSEQUENCES TO PROPERTY	'Medium' (25%)	'Medium' (35%)
RISK TO PROPERTY	'Moderate' (2×10^{-4})	'Moderate' (2×10^{-4})
RISK TO LIFE	5.9×10^{-5}/annum	5.3×10^{-5}/annum
COMMENTS	This level of risk to life and property is 'UNACCEPTABLE'. To move risk to 'ACCEPTABLE' levels, the recommendations in **Section 13 and 15** are to be followed.	This level of risk to life and property is 'UNACCEPTABLE'. To move risk to 'ACCEPTABLE' levels, the recommendations in **Section 13** are to be followed.

(See Aust. Geomech. Jnl. Mar 2007 Vol. 42 No 1, for full explanation of terms)

9. Suitability of the Proposed Development for the Site

The proposed development is suitable for the site. No geotechnical hazards will be created by the completion of the proposed development provided it is carried out in accordance with the requirements of this report and good engineering and building practice.

10. Stormwater

The fall is to William Street. Roof water from the development is to be piped to the street drainage system through any tanks that may be required by the regulating authorities.

11. Excavations

Two excavations will be required for the proposed development:

- An excavation to install the proposed pool will reach a maximum depth of ~1.5m.
- An excavation to construct the proposed garage will reach a maximum depth of ~3.5m.

The excavations are expected to be through fill, shallow soil, and clay with Extremely Low to Very Low Strength Shale expected at depths of between ~0.3m and ~1.2m in the location of the proposed works. It is envisaged that excavations through fill, soil, clay, and Extremely Low Strength Shale can be carried out with an excavator and toothed bucket. If encountered, excavations through Medium Strength Rock will require grinding or rock sawing and breaking.

12. Vibrations

Possible vibrations generated during excavations through fill, soil, clay, and Extremely Low to Very Low Strength Shale will be below the threshold limit for building damage utilising a domestic-sized excavator up to 16 tonnes.

If encountered, excavations through Medium Strength Rock or better should be carried out to minimise the potential to cause vibration damage to the subject house. Allowing ~0.5m for backwall drainage, the setbacks from the proposed excavations to the existing structures are as follows:

- The garage excavation will come ~Flush with the brick supporting walls of the subject house.
- The pool excavation will be set back ~2.3m from the brick supporting walls of the subject house.

Close controls by the contractor over rock excavation are recommended so excessive vibrations are not generated.

Excavation methods are to be used that limit peak particle velocity to 5mm/sec at the house walls. Vibration monitoring will be required to verify this is achieved. Vibration monitoring must include a light/alarm so the operator knows if vibration limits have been exceeded. The equipment is to log and record vibrations throughout the excavation works.

In Medium Strength rock or better techniques to minimise vibration transmission will be required. These include:

- Rock sawing the excavation perimeter to at least 1.0m deep prior to any rock breaking with hammers, keeping the saw cuts below the rock to be broken throughout the excavation process.
- Limiting rock hammer size.
- Rock hammering in short bursts so vibrations do not amplify.
- Rock breaking with the hammer angled away from the nearby sensitive structures.
- Creating additional saw breaks in the rock where vibration limits are exceeded, as well as reducing hammer size as necessary.
- Use of rock grinders (milling head).

Should excavation induced vibrations exceed vibration limits after the recommendations above have been implemented, excavation works are to cease immediately and our office is to be contacted.

It is worth noting that vibrations that are below thresholds for building damage may be felt by the occupants of the subject and neighbouring houses.

13. Excavation Support Requirements

Bulk Excavation for the Garage and Lift

The excavation for the proposed garage will reach a maximum depth of ~3.5m at the location of the stairs. Allowing 0.5m for back wall drainage, the setbacks are as follows:

- ~Flush with a retaining wall on the uphill side of the driveway which will be demolished as part of the works (Photo 4).
- ~Flush with a retaining wall on the downhill side of the driveway (Photo 2).
- ~0.5m from the SE common boundary.
- Close to flush with the supporting house walls at the corner of the garage excavation but angling away at 45° (Photo 13).

As such, the retaining wall on the downhill side of the driveway which is to remain, as well as the SE common boundary and the house will lie within the zone of influence of the proposed excavation. In this instance, the zone of influence is the area above a theoretical 45° line (from horizontal) from the base of the excavation towards the surrounding structures and boundaries. This line reduces to 30° through the fill and soil.

The stone block retaining wall which reaches a maximum height of ~1.8m and supports fill for landscaping on the uphill side of the driveway (Photo 4) will be demolished as part of the proposed works. The demolition should be carried out from the top down in an orderly manner with the fill, soil and clay behind the wall being systematically lowered at the same time. The soil batter slope is not to exceed 1.0 Vertical to 1.7 Horizontal (30°) as the walls are demolished.

Due to the depth of the excavation grade of the slope, and its proximity to the subject house, common boundaries, and brick retaining wall which is to remain, it is recommended ground support be installed along all sides of the excavation prior to the excavation commencing. See the site plan attached for the minimum required extent of the shoring shown in blue.

A spaced piled retaining wall is one suitable method of support. Pier spacing for the wall is typically ~2.0m but can vary between 1.6 to 2.4m depending on the design. To drill the pier holes for the wall, a mini piling rig or similar that can excavate through Medium to High Strength Rock is recommended as the ground testing did not extend to the likely required depth of the piles. If a machine of this type is not available, we recommend carrying out core

drilling before the construction commences to confirm the strength of the rock and to ensure the excavation equipment is capable of reaching the required depths. As the excavation is lowered in 1.5m lifts, infill sprayed concrete panels or similar are added between the piers to form the spaced wall. Drainage is installed behind the panels. The piers can be temporarily supported by embedment below the base of the excavation, or by a combination of embedment and temporary propping. The embedment depths are to be calculated by the structural engineer. Allowances are to be made for this wall to accommodate the surcharge loads from the house. Upon completion of the excavation, the piled walls are to be tied into the slabs of the garage to provide permanent bracing.

The geotechnical consultant is to inspect the drilling process of the entire first pile and the ground materials at the base of all pier holes/excavations for ground support purposes.

Bulk Excavation for the Pool

The excavation for the proposed pool will reach a maximum depth of ~1.5m.

Provided that the retaining wall demolition is carried out as above prior to the pool excavation, no structures or boundaries are expected to lie within the zone of influence of the pool excavation.

The sides of the proposed pool are expected to stand at near-vertical angles for short periods of time until the pool structure is installed, provided the cut batters are kept from becoming saturated. If the cut batters remain unsupported for more than a day before pool construction commences, they are to be supported with typical pool shoring until the pool structure is in place.

Upslope runoff is to be diverted from the cut by sandbag mounds or other diversion works. All unsupported cut batters are to be covered to prevent access of water in wet weather and loss of moisture in dry weather. The covers are to be tied down with metal pegs or other suitable fixtures so they cannot blow off in a storm.

Advice Applying to Both Excavations

The materials and labour to construct the pool structure/retaining walls are to be organised so on completion of the excavations they can be constructed as soon as possible. The excavations are to be carried out during a dry period. No excavations are to commence if heavy or prolonged rainfall is forecast.

All excavation spoil is to be removed from site following the current Environmental Protection Agency (EPA) waste classification guidelines.

14. Fill

Fill will be placed around the proposed garage and pool for landscaping. We note no structures are to be supported on fill and the fill in these locations is to be used as formwork for the overlying slabs only. No fills are to be laid until retaining walls are in place. The fills will reach a maximum height of ~1.0m. The surface is to be prepared before any fills are laid by removing any organic matter and topsoil. To prevent excessive settlement of the fill under the suspended slab, fills can to be laid in a loose thickness not exceeding 0.3m before being moderately compacted. Tracking the machine over the loose fill in 1 to 2 passes should be sufficient. Immediately behind the retaining walls (say to 1.5m), the fills are to be compacted with light weight equipment such as a hand-held plate compactor so as not to damage the retaining walls. Where light weight equipment is used, fills are to be laid in a loose thickness not exceeding 0.15m before being compacted.

15. Retaining Structures

For cantilever or singly propped retaining structures it is suggested the design be based on a triangular distribution of lateral pressures using the parameters shown in Table 1.

TABLE 1 ON THE NEXT PAGE

Table 1 – Likely Earth Pressures for Retaining Structures

Unit	Earth Pressure Coefficients			
	Unit weight (kN/m^3)	'Active' K_a	'At Rest' K_0	Passive
Fill and Topsoil	20	0.40	0.55	N/A
Residual Clays	20	0.35	0.45	Kp = 2.0 'ultimate'
Extremely Low Strength Rock	22	0.25	0.38	Kp = 2.5 'ultimate'
Very Low Strength Rock	22	0.22	0.35	400kPa 'ultimate'

For rock classes refer to Pells et al "Design Loadings for Foundations on Shale and Sandstone in the Sydney Region". Australian Geomechanics Journal 1978.

It is to be noted that the earth pressures in Table 1 assume a level surface above the structure and do not account for any surcharge loads from the slope, house, or nearby retaining walls. Noting that surcharge loads from the structures above will be acting on the wall. It also assumes retaining structures are fully drained. It should be noted that passive pressure is an ultimate value and should have an appropriate safety factor applied. No passive resistance should be assumed for the top 0.4m to account for any disturbance from the excavation. Ground materials and relevant earth pressure coefficients are to be confirmed on site by the geotechnical consultant.

All retaining walls are to have sufficient back-wall drainage and be backfilled immediately behind the structure with free-draining material (such as gravel). This material is to be wrapped in a non-woven Geotextile fabric (i.e. Bidim A34 or similar), to prevent the drainage from becoming clogged with silt and clay. If no back-wall drainage is installed in retaining wall, the likely hydrostatic pressures are to be accounted for in the structural design.

16. Site Classification

The site classification is Class M in accordance with AS2870-2011.

17. Foundations

The proposed garage and lift can be supported on a thickened edge / raft slab with piers taken to and embedded at least ~0.6m into the underlying Extremely Low to Very Low Strength Shale where necessary. This material is expected to be exposed across the uphill side of the proposed excavation. Where it is not exposed, and where weathered rock drops away with the slope, piers will be required to maintain a uniform foundation material across the structure. This ground material is expected at depths of between ~0.3m to ~0.9m below the current surface in the location of the proposed garage and lift.

The proposed pool excavation is expected to be partially seated in Extremely Low to Very Low Strength Shale. This is a suitable foundation material. To maintain a uniform foundation material across the structure, as well as to ensure no surcharge loads are added to the retaining wall, shallow piers will need to be taken to below the zone of influence of the retaining wall and embedded into the underlying Extremely Low to Very Low Strength Shale. Extremely Low to Very Low Strength Shale is expected at depths of between ~0.9m to ~1.0m below the current surface in the location of the proposed pool excavation.

A maximum allowable bearing pressure of 600kPa can be assumed for footings on Extremely Low to Very Low Strength Shale or better.

As the bearing capacity of clay and shale reduces when it is wet, we recommend the footings be dug, inspected, and poured in quick succession (ideally the same day if possible). If the footings get wet, they will have to be drained and the soft layer of wet clay or shale on the footing surface will have to be removed before concrete is poured.

If a rapid turnaround from footing excavation to the concrete pour is not possible, a sealing layer of concrete may be added to the footing surface after it has been cleaned and inspected by the geotechnical consultant.

NOTE: If the contractor is unsure of the footing material required, it is more cost-effective to get the geotechnical consultant on site at the start of the footing excavation to advise on footing depth and material. This mostly prevents unnecessary over-excavation in clay-like shaly-rock but can be valuable in all types of geology.

18. Site Maintenance/Remedial Works

Where slopes approach or exceed 20°, such as on this site, it is prudent for the owners to occasionally inspect the slope (say annually or after heavy rainfall events, whichever occurs first). Should any of the following be observed: movement or cracking in retaining walls, cracking in any structures, cracking or movement in the slope surface, tilting or movement in established trees, leaking pipes, or newly observed flowing water, or changes in the erosional process or drainage regime, then a geotechnical consultant should be engaged to assess the slope. We can carry out these inspections upon request. The risk assessment in **Section 8** is subject to this site maintenance being carried out.

19. Geotechnical Review

The structural plans are to be checked and certified by the geotechnical engineer as being in accordance with the geotechnical recommendations. On completion, a Form 2B will be issued. This form is required for the Construction Certificate to proceed.

20. Inspections

The client and builder are to familiarise themselves with the following required inspections as well as council geotechnical policy. We cannot provide certification for the Occupation Certificate or the owner if the following inspections have not been carried out during the construction process.

- The geotechnical consultant is to inspect the ground materials while the first pier for the ground support is being dug to assess the ground strength and to ensure it is in line with our expectations.

- All finished pier holes for piled wall/excavations for ground support are to be inspected and measured before concrete is placed.

- All footings are to be inspected and approved by the geotechnical consultant while the excavation equipment and contractors are still onsite and before steel reinforcing is placed or concrete is poured.

White Geotechnical Group Pty Ltd.

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Reviewed By:

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Photo 1

Photo 2

Photo 3

Photo 4

Photo 5

Photo 6

Photo 7

Photo 8

Photo 9

Photo 10

Photo 11 – downhole is top to bottom

Photo 12 – downhole is top to bottom

Photo 13

Important Information about Your Report

It should be noted that Geotechnical Reports are documents that build a picture of the subsurface conditions from the observation of surface features and testing carried out at specific points on the site. The spacing and location of the test points can be limited by the location of existing structures on the site or by budget and time constraints of the client. Additionally, the test themselves, although chosen for their suitability for the particular project, have their own limiting factors. The testing gives accurate information at the location of the test, within the confines of the test's capability. A geological interpretation or model is developed by joining these test points using all available data and drawing on previous experience of the geotechnical consultant. Even the most experienced practitioners cannot determine every possible feature or change that may lie below the earth. All of the subsurface features can only be known when they are revealed by excavation. As such, a Geotechnical report can be considered an interpretive document. It is based on factual data but also on opinion and judgement that comes with a level of uncertainty. This information is provided to help explain the nature and limitations of your report.

With this in mind, the following points are to be noted:

- If upon the commencement of the works the subsurface ground or ground water conditions prove different from those described in this report, it is advisable to contact White Geotechnical Group immediately, as problems relating to the ground works phase of construction are far easier and less costly to overcome if they are addressed early.

- If this report is used by other professionals during the design or construction process, any questions should be directed to White Geotechnical Group as only we understand the full methodology behind the report's conclusions.

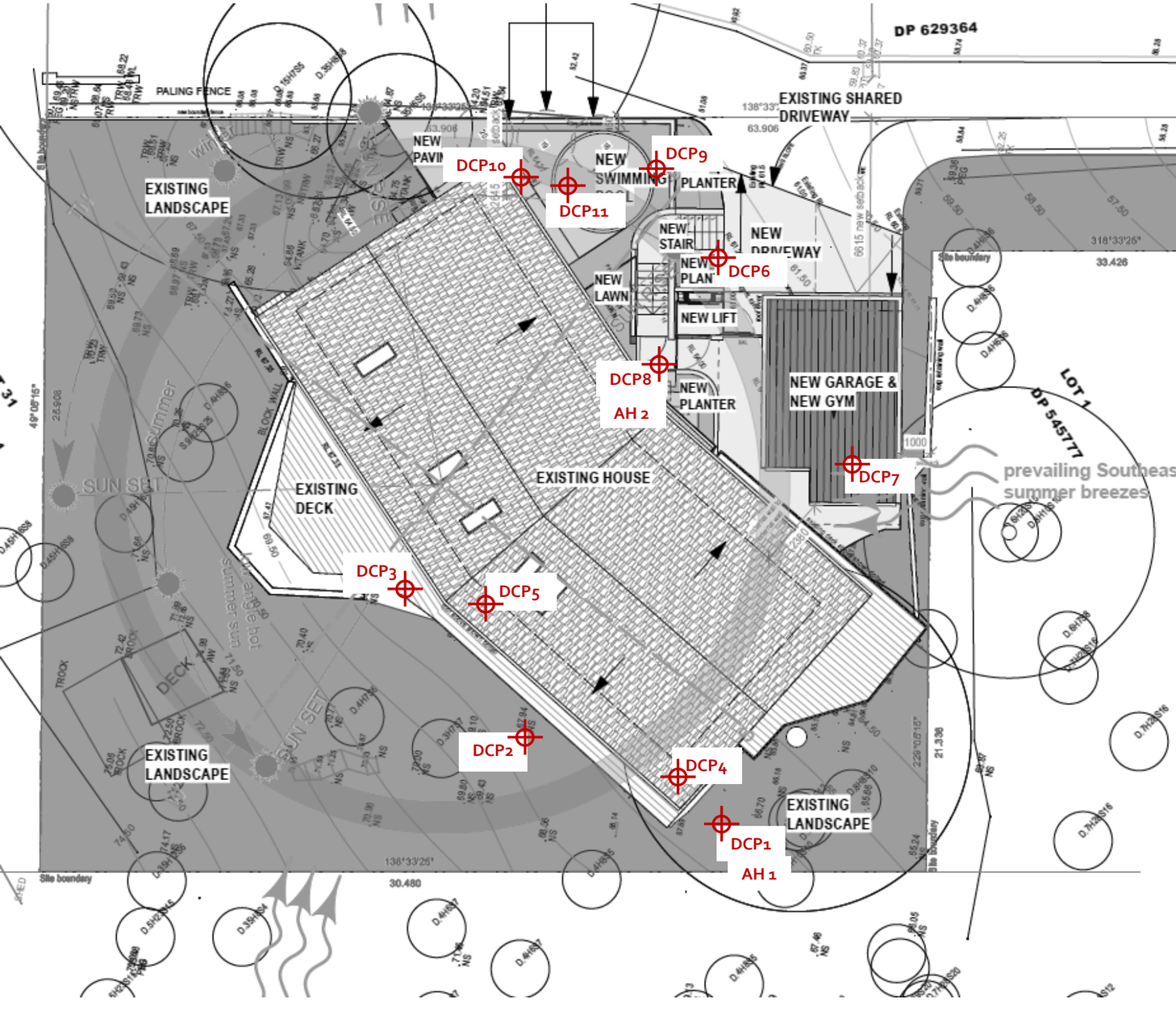
- The report addresses issues relating to your specific design and site. If the proposed project design changes, aspects of the report may no longer apply. Contact White Geotechnical if this occurs.

- This report should not be applied to any other project other than that outlined in section 1.0.

- This report is to be read in full and should not have sections removed or included in other documents as this can result in misinterpretation of the data by others.

- It is common for the design and construction process to be adapted as it progresses (sometimes to suit the previous experience of the contractors involved). If alternative design and construction processes are required to those described in this report, contact White Geotechnical Group. We are familiar with a variety of techniques to reduce risk and can advise if your proposed methods are suitable for the site conditions.

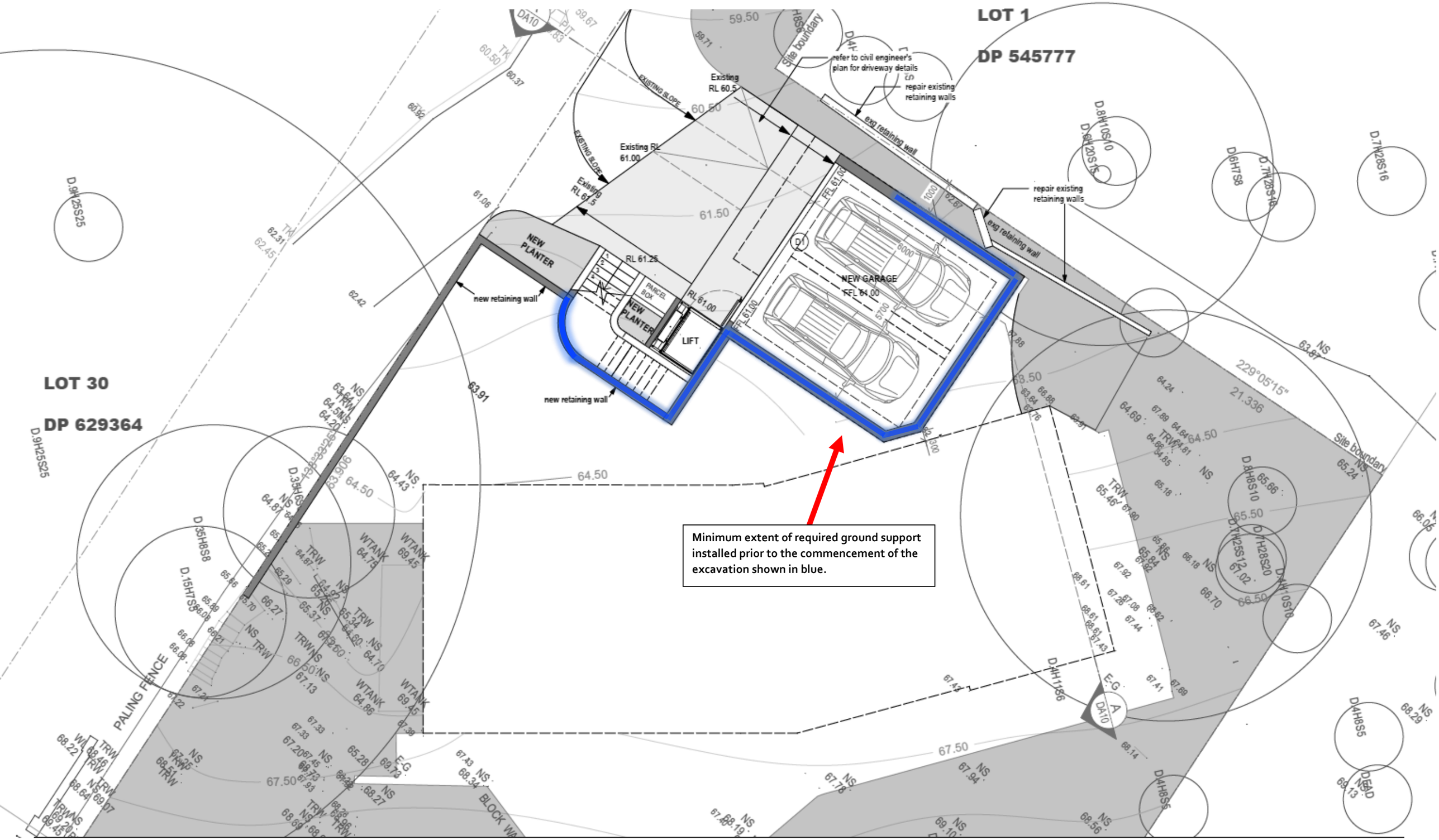
SITE PLAN – showing test locations



DA AREA CALCULATIONS
COMPLIANCE TABLE (m2)

	EXISTING	PROPOSED	CONTROL	COMPL
SITE AREA	942.5m2	942.5m2	N/A	N/
SOFT LANDSCAPED AREA	549m2 (58.2%)	540.5m2 (57.3%) 8.5m2 reduced from	min 565.5m2 (60%)	NO, 4.4% un

LOWER GROUND FLOOR PLAN – showing minimum extent of required shoring



Legend:

AD	ALUMINIUM DOOR	FC	FIBRE CEMENT
AW	ALUMINIUM WINDOW DOOR	FFL	FINISHED FLOOR LEVEL
BAL	GLASS BALUSTRADE	GD	NEW GRATED DRAIN
CC	COLORBOND CLADDING	GF	GLASS POOL FENCE
CF	COLORBOND FENCE	MRS	METAL ROOF SHEETING
CL	CEILING	MC	METAL CLADDING
CR	CEMENT RENDER	RW	NEW RETAINING WALL
EG	EAVES GUTTER	SC	STONE CLADDING
EXG	EXISTING	TL	TILE FINISH
P1, 2	PAINT FINISH COLOUR 1, 2	VC	VERTICAL CLADDING

WALL LEGEND: EXISTING WALL; WALL TO BE DEMOLISHED; NEW WALL

Revision:	Date:	For:
D	23/5/2025	For client's review
E	10/6/2025	For client's review
F	15/7/2025	For consultants
G	3/9/2025	For consultants
H	5/9/2025	DA Submission

Project: Alterations & Additions

Client: George & Lisa Tremopoulos

Address: 2A William Street, Avalon Beach NSW 2107

Drawing Title: PROPOSED LOWER GROUND FLOOR PLAN

Stage: DEVELOPMENT APPLICATION

Scale: 1:100 | Drawn: TN | Chkd: JE

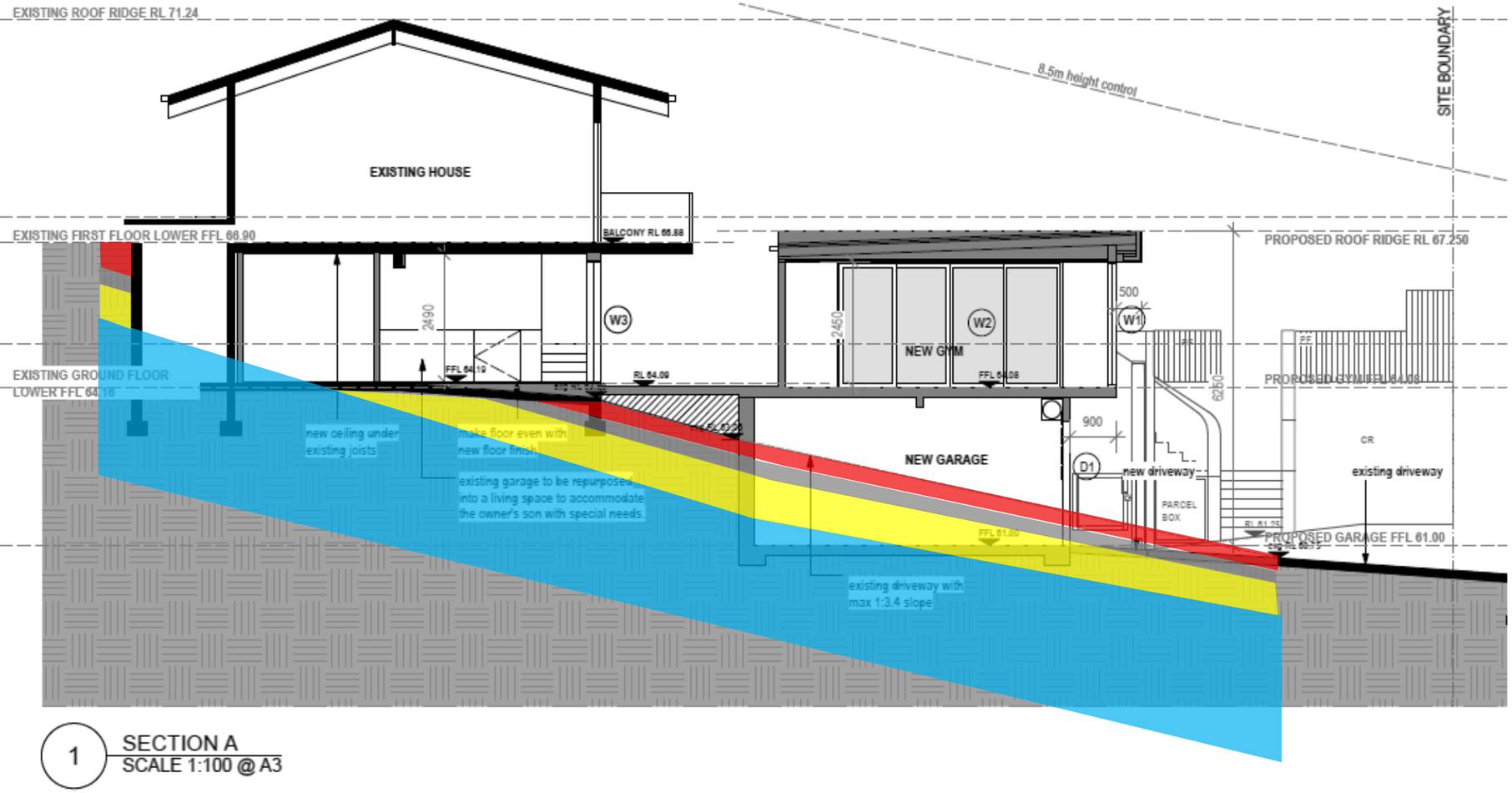
North: TN

Drawing No: DA03 | Rev: H

JANE EDWARDS ARCHITECTURE

ABN: 79626347350
Nominated Architect: Jane Edwards NSW ARB 7050

TYPE SECTION – Diagrammatical Interpretation of expected Ground Materials



1 SECTION A
SCALE 1:100 @ A3

2 SECTION B
SCALE 1:100 @ A3

Legend:

AD	ALUMINIUM DOOR	FC	FIBRE CEMENT
AW	ALUMINIUM WINDOW/DOOR	FFL	FINISHED FLOOR LEVEL
BAL	GLASS BALUSTRADE	GD	NEW GRATED DRAIN
CC	COLORBOND CLADDING	GF	GLASS POOL FENCE
CF	COLORBOND FENCE	MRS	METAL ROOF SHEETING
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CR	CEMENT RENDER	RW	NEW RETAINING WALL
EG	EAVES GUTTER	SC	STONE CLADDING
EXG	EXISTING	TL	TILE FINISH
P1, 2	PAINT FINISH COLOUR 1, 2	VC	VERTICAL CLADDING

WALL LEGEND: EXISTING WALL; WALL TO BE DEMOLISHED; NEW WALL

Revision:	Date:	For:
A	23/5/2025	For client's review
B	10/6/2025	For client's review
C	15/7/2025	For consultants
D	3/9/2025	For consultants
E	5/9/2025	DA Submission

Project: Alterations & Additions

Client: George & Lisa Tremopoulos

Address: 2A William Street, Avalon Beach NSW 2107

Drawing Title: SECTIONS

Stage: DEVELOPMENT APPLICATION

Scale: 1:100 | Drawn: TN | Chkd: JE

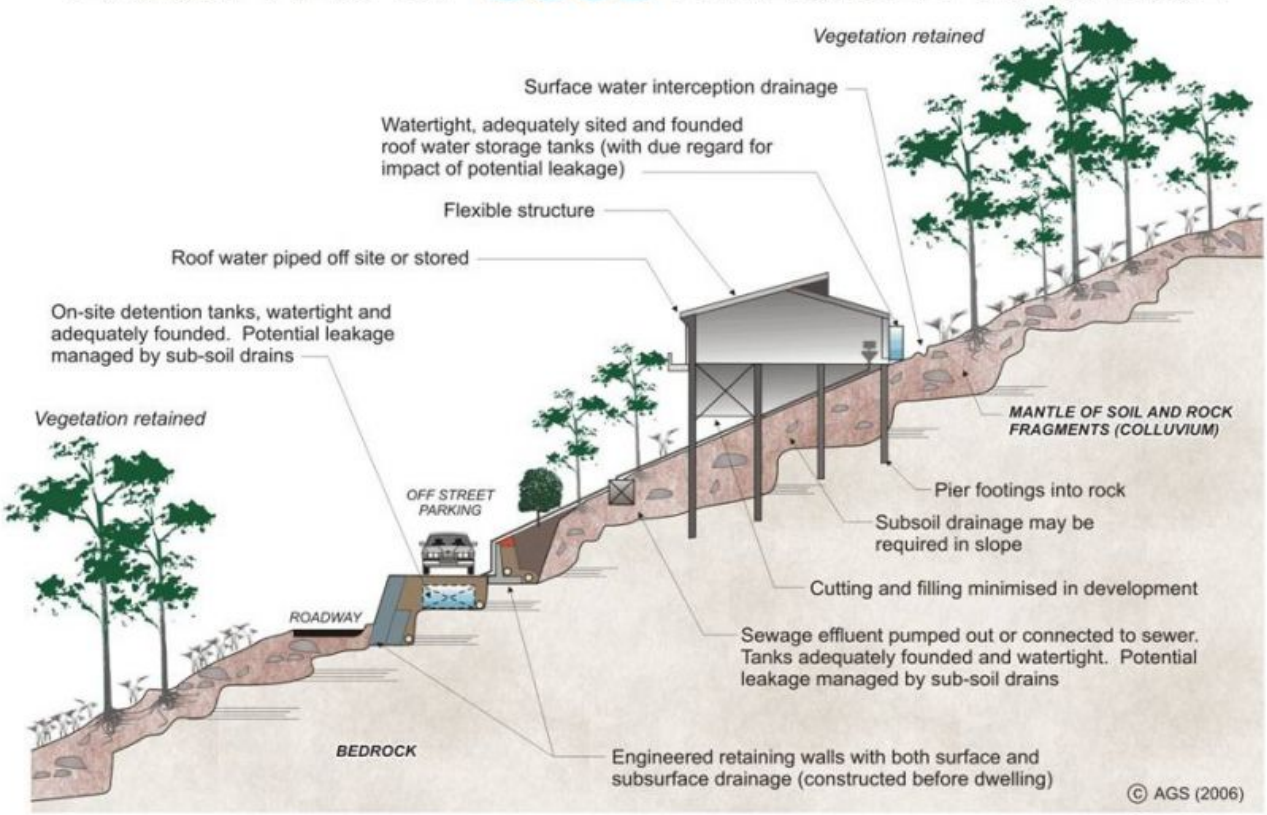
North:

Drawing No: DA10 | Rev: E

JANE EDWARDS ARCHITECTURE

ABN: 79628347350
Nominated Architect: Jane Edwards NSW ARB 7050

EXAMPLES OF **GOOD** HILLSIDE PRACTICE



EXAMPLES OF **POOR** HILLSIDE PRACTICE