Legend

- Subject site - 4 Vanderspek Place, Frenchville
- Proposed lot
- Proposed amended easement
- Proposed building footprint (300sqm)
- Proposed crossover
- Manhole
- Sewer line
- Cadastral boundary

ROCKHAMPTON REGIONAL COUNCIL
AMENDED PLANS APPROVED
26 July 2024
DATE
These plans are approved subject to the current conditions of approval associated with
Development Permit No.: D/1-2024
Dated: 19 April 2024

REFERENCES

Cadastre - (c) The State of Queensland (DNRM DCDB)

Coordinate System: GDA2020 MGA Zone 56

Images are not orthorectified, overlays are best fit. Features are based on topographical data and may vary.

Nearmap 14 April 2023.

Indicative only.

Date: 9/06/2024

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TITLE

Proposed Subdivision Plan (1 into 2 Lots)

PROJECT

4 Vanderspek Place, Frenchville

REEL PLANNING
Urban & Rural Strategies

Document Set ID: 40835644
Version: 1, Version Date: 18/06/2024

Butler Partners
geotechnical • geo-environmental • groundwater

ROCKHAMPTON REGIONAL COUNCIL

AMENDED PLANS APPROVED

26 July 2024

DATE

These plans are approved subject to the current conditions of approval associated with

Development Permit No.: **D/1-2024**

Dated: **19 April 2024**

Slope Stability Assessment and Geotechnical Investigation Proposed Subdivision

4 Vanderspek Place, Frenchville

Prepared for

Reel Planning Pty Ltd

Project No. RG23-1178A

9 January 2024

Butler Partners
geotechnical • geo–environmental • groundwater

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Slope Stability Assessment and Geotechnical Investigation
Proposed Subdivision
4 Vanderspek Place, Frenchville
Project No. RG23-1178A
9 January 2024

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BUTLER PARTNERS (REGIONAL) PTY LTD
www.butlerpartners.com.au

SECTION 1 INTRODUCTION

1.1 Project

It is understood that a residential development is proposed at 4 Vanderspek in Frenchville, comprising the reconfiguration of the existing lot into two separate lots, allowing for construction of a new dwelling in the south-west corner. The site is within an area of sloping topography and identified on Rockhampton Regional Council's Steep Land Overlay to require the site to be assessed for slope stability.

It is also understood that Rockhampton Regional Council (RRC) requires the slope stability assessment to address the requirements of the Australian Geomechanics Society Landslide Risk Management guidelines and professional geotechnical practice. In conjunction with the slope stability assessment, geotechnical investigation is required as input to detailed structural design of the proposed dwelling.

The location and extent of the site are indicated approximately on Drawing No. 1, attached.

1.2 Proposed Scope of Work

Based on our knowledge of the area from previous investigations, and a desk-top review of published geological mapping (Rockhampton sheet), it was anticipated that the site ground conditions may comprise surface layers of colluvium, underlain by residual soils, overlying weathered bedrock at potentially shallow depth. A shallow groundwater table was not anticipated.

For the scope of the proposed development and the anticipated ground conditions, it was proposed that slope stability assessment be carried out in conjunction with geotechnical investigation, comprising inspection and mapping of the site and the excavation and sampling of three to four test pits to 3m to 4m depth (or prior refusal) at nominated locations across the site, with a medium size track-mounted excavator. A dynamic cone penetrometer test (DCP) would be carried out adjacent to each test pit location to assist with soil strength assessment.

Assessment of slope stability 'risk' for the proposed development would then be carried out using methods published by the Australian Geomechanics Society and to specifically address Council's Steep Land Overlay requirements.

Using the results of the proposed fieldwork and laboratory testing outcomes, an interpretive report including both, the preliminary landslide risk assessment and the geotechnical investigation, would be produced to provide details of the investigation as carried out, as well as geotechnical design information on each of the following:

- subsurface conditions;
- preliminary slope stability assessment;
- earthworks and site preparation;
- temporary and permanent batter slopes;
- erosion and sediment control parameters;
- suitability of cut material for fill;
- site classification to AS2870;
- advice and design parameters for retaining wall selection;
- effect of footing on slope stability;
- suitable alternate foundation types;
- risk control strategies; and
- anticipated construction aspects.

1.3 Commission

Based on the proposed scope of work, a fee to undertake the investigation was presented in a proposal dated 6 June 2023. Butler Partners (Regional) Pty Ltd (Butler Partners) was subsequently commissioned by Reel Planning Pty Ltd (Reel Planning) to conduct the geotechnical investigation as proposed.

This report was issued in draft (for comment) on 31 August 2023.

SECTION 2 THE SITE

2.1 Site Description

The proposed lot to be reconfigured is located approximately 150m north of Frenchville Road within a residential area, and at the time of the investigation the site was temporarily fenced, with access from a concrete roadway running along the western and northern boundaries. Vegetation at the site generally comprised sparse grass, shrubs and small to large native trees, with some patches of bare earth.

Ground surface levels at the site appeared to generally slope downwards from north-east to south-west at approximately 25 to 30 degrees. No evidence of slope instability across the site was observed during fieldwork. Cobbles and some embedded boulders were observed mostly to the southeast section of the proposed subdivision. A relatively recent aerial image indicating the approximate extent of the site (highlighted in blue) is presented in Photograph 1, and two general views of the site, at the time of investigation, are presented in Photograph 2 and Photograph 3.

Photograph 1: Aerial image showing the approximate extent of the proposed subdivision (14 April 2023, Nearmap image)

Photograph 2: General view of the site looking west from adjacent concrete road.

Photograph 3: General view of the site looking south from near Test Pit 1.

2.2 Geology

Reference to the Geological Survey of Queensland 1:100,000 geological series Rockhampton sheet indicates that the site is mapped in an area of Permian aged Lakes Creek Formation, from the Berseker Group, consisting of grey massive, indurated siltstone and lithofeldspathic to quartzolitho feldspathic sandstone derived from felsic to intermediate volcanics.

SECTION 3 FIELDWORK

3.1 Testing and Sampling Methods

The investigation comprised the excavation and sampling of three test pits (Test Pits 1 to 3) with a 12 tonne Hitachi tracked excavator, using a 0.6 m wide bucket. Strata identification and sampling was from inspection of the test pit side walls and disturbed samples recovered from the excavator bucket.

On completion of excavation, all test pits were backfilled with spoil material and track rolled.

3.2 Dynamic Cone Penetrometer Testing

A dynamic cone penetrometer (DCP) test was performed adjacent to each test pit to between 0.3m and 0.8m depth.

3.3 Test Pit Locations and Supervision

Test pit locations were determined in the field by hand-held GPS co-ordinates and are indicated approximately on Drawing No 1, attached. No detailed survey information was available at the time of the investigation, however, approximate ground surface levels at each test location were estimated, based on 1m contour data from the Rockhampton Regional Council's online mapping database, which should be confirmed by detailed survey.

A senior geotechnical engineer set out the test pit locations, logged the stratigraphy encountered in the test pits, supervised the fieldwork and directed the in-situ sampling and testing program.

SECTION 4 INVESTIGATION RESULTS

4.1 Subsurface Conditions

The subsurface conditions encountered in the test pits are given on Test Pit Report sheets included in Appendix A, using classification and descriptive terms defined in the accompanying notes (which are generally in accordance with Australian Standard AS AS1726-1993). It should be noted that rock types indicated on the Test Pit Report sheets are based on visual assessment only; no petrographic analysis has been undertaken for confirmation. The DCP test results are also tabulated with depth in Appendix A.

For a description of the stratigraphy encountered at each test pit location, the Test Pit Report sheets should be consulted. However, in broad summary, the subsurface conditions encountered at the test pit locations generally comprised surface layers of gravelly/sandy silt to between 0.4m and 0.5m depth, except in Test Pit 2 where cobbles and boulders with a gravelly sandy clay matrix were encountered from ground surface level to 0.5m depth, underlain by stiff to hard gravelly/sandy clay to between 0.8m and 1.6m depth. The soils were underlain in turn by sandstone (rock) of medium to very high strength.

It should be noted that 'stronger' rock may exist close below the base of the test pits and potentially at shallower depth at other locations.

4.2 Groundwater

Free groundwater was not encountered during the excavation of any of the test pits. However, it should be noted that groundwater levels can vary both seasonally and with prevailing weather conditions.

4.3 Laboratory Testing

Selected samples of soil and rock recovered from the test pits were submitted to Butler Partners' NATA accredited Rockhampton geotechnical testing laboratory for assessment of erosion and sediment control parameters, particle size distribution, plasticity, 'drained' strength parameters, moisture-density relationship, California Bearing Ratio (CBR) and rock strength using Australian Standards AS1289 test methods. Laboratory test report sheets are included in Appendix B and the test results are summarised in the following sections.

It should be noted that sample descriptions provided in the laboratory results summary tables (and the laboratory test result sheets) are based on the inspection of each individual laboratory test sample only. No allowance has been made in sample descriptions for sampling, sub-sampling or test methodology in determination of the mass material properties. Estimates of mass material properties are provided on each individual Test Pit Report sheet and as such, the laboratory test results should be read in conjunction with the relevant report sheets.

4.3.1 Erosion and Sediment Control Parameters

One selected sample of soil was tested to determine the Emerson Class Number, pH and electrical conductivity. A summary of the reported test results is presented in Table 1, and the Emerson Class Number results indicate that the sample tested had a low potential for dispersion, using distilled water.

Table 1: Summary of Reported Emerson Class, pH and Conductivity Test Results

Test Pit	Depth (m)	Sample Description	Sample Moisture Content (%)	Emerson Class No.	pH	Electrical Conductivity (µS/cm)
2	0.5 – 0.6	Gravelly Sandy Clay	14.2	4	7.2	5.36

4.3.2 Particle Size Distribution

Three selected samples of soil were tested for measurement of particle size distribution analysis using wash sieve grading techniques, and the reported results are summarised in Table 2.

Table 2: Summary of Reported Particle Size Distribution Test Results

Test Pit	Depth (m)	Sample Description	Sample Moisture Content (%)	Cobbles Fraction(1) (%)	Gravel Fraction(2) (%)	Sand Fraction(3) (%)	Silt and Clay Fraction(4) (%)
1	0.6 – 0.8	Gravelly Sandy Clay	20.7	0	26	28	46
2	0.5 – 0.6	Gravelly Sandy Clay	14.2	0	27	27	46
3	0.5 – 0.6	Gravelly Clay	8.2	21	22	11	46

(1) Particle size <200mm, >60mm; (2) Particle size <60mm, >2mm; (3) Particle size (approximately) <2mm, >0.075mm; (4) Particle size (approximately) <0.075mm

4.3.3 Plasticity

Two selected samples of soil were tested for measurement of plasticity using Atterberg limit and linear shrinkage test methods. The reported test results are summarised in Table 3 together with the sample classification and indicate that the clay samples tested were predominantly of low to medium plasticity.

Table 3: Summary of Reported Plasticity Test Results

Test Pit	Depth (m)	Sample Description	Sample Moisture Content (%)	Liquid Limit (%)	Plastic Limit (%)	Plasticity Index (%)	Linear Shrinkage (%)	Inferred Drained Friction Angle – ϕ'(1)	Classification(2)
1	0.6 – 0.8	Gravelly Sandy Clay	20.7	40	18	22	10.5	28	CI
2	0.5 – 0.6	Gravelly Sandy Clay	14.2	31	18	13	7.5	31	CL

(1) Gibson R.E. (1953) *Experimental determination of the true angle of friction in clays* Proc. 3rd I.C.S.M.F.E., Zurich, pp126-132

(2) Australian Standard AS1726-1993, *Geotechnical site investigations*

4.3.4 Drained Strength

One 'remoulded' clay sample was tested for measurement of 'drained' shear strength parameters (c', ø') using staged, consolidated, 'slow' direct shear test methods and a summary of the reported results is presented in Table 4.

Table 4: Reported Direct Shear Strength Parameter Test Results

Test Pit	Depth (m)	Sample Description	Apparent Cohesion – c' (kPa)	Friction Angle – ø' (degrees)
1	0.6 – 0.8	Gravelly Sandy Clay	19	41

4.3.5 Moisture-Density Relationship

One selected bulk sample recovered from the test pits were tested to determine (Standard) laboratory moisture-density relationship and the resulting Maximum Dry Density (MDD) and Optimum Moisture Content (OMC) results for the sample tested are summarised in Table 5.

Table 5: Summary of Moisture-Density Relationship Results

Test Pit	Depth (m)	Sample Description	Sample Moisture Content (%)	Standard Compaction	
				Maximum Dry Density (t/m^3)	Optimum Moisture Content (%)
2	0.2 – 0.5	Gravelly Sandy Clay	16.5	1.65	18.5

The test results indicate that the moisture content of the sample tested (at the time of sampling) was 2% dry of OMC.

4.3.6 California Bearing Ratio

One sub-sample of the sample tested for moisture-density relationship were tested for measurement of soaked CBR using the test method given in Australian Standard AS1289.6.1.1 – 1998. The samples were re-compacted using Standard compactive effort at approximately OMC and soaked under a surcharge loading of 4.5kg for four days prior to testing. A summary of the reported results is presented in Table 6.

Table 6: Summary of California Bearing Ratio Test Results

Test Pit	Depth (m)	Sample Description	Sample Preparation		Swell (%)	CBR (%)
			Moisture Content (%)	Dry Density (%)		
2	0.2 – 0.5	Gravelly Sandy Clay	18.7	1.61	0.5	6

4.3.7 Rock Strength

Selected 'lump' samples of sandstone (rock) recovered from Test Pits 1 and 3 were tested for measurement of rock strength using Point Load Test [Is(50)] methods and the test results are summarised in Table 7.

Table 7: Summary of Reported Point Load Strength Index Test Results

Test Pit	Depth (m)	Sample Description	Point Load Strength (I_s(50)	Rock Strength Category*
1	1.6 – 1.9	Sandstone	0.7	Medium
			5.5	Very High
			6.1	Very High
3	0.8 – 1.0	Sandstone	3.0	Very High
			2.4	High
			3.3	Very High

* Australian Standard AS1726 - 1993, *Geotechnical site investigations*

SECTION 5 GEOTECHNICAL DESIGN DISCUSSION

5.1 Ground Model

The results of the investigation indicate that the subsurface conditions at the test pit locations comprised a surface layer of gravelly/sandy silt (except in one test pit), overlying stiff to hard gravelly/sandy clay, underlain in turn by medium to very high strength sandstone (rock). Cobbles and some embedded boulders were observed across the site. Groundwater was not encountered in any test pit location, however, groundwater levels can change over time. In these ground conditions geotechnical design will need to consider (at least) the following:

- subsurface conditions and variability across the site;
- rock excavatability;
- general slope stability;
- suitability of cut for fill;
- batter slopes;
- classification of the site in accordance with AS2870;
- retaining wall pressures;
- suitable foundation types;
- appropriate founding depths and bearing pressures;
- variations in footing founding depths and founding conditions across the site; and
- possible construction difficulties.

Discussion of geotechnical design parameters, as well as design and construction recommendations and suggestions are detailed in the following sections.

5.2 Earthworks

5.2.1 Excavatability

Based on the results of the fieldwork, excavation for dwelling foundations would be expected to encounter some surface soils (including cobbles and boulders), overlying weathered sandstone (rock). The rock encountered in the test pits ranged from medium to very high strength, and it is considered possible that zones of 'stronger' and/or 'less jointed' rock may also exist below test pit excavation depths (and possibly at shallower depths elsewhere on the site).

Excavation of soil and extremely low to low strength rock should be readily achieved in bulk excavation using a large hydraulic excavator. Bulk excavation of medium to high strength rock will require relatively major use of 'rock breaker' equipment unless joint spacing is moderately close (less than 0.3m). In high strength (or stronger) rock (with relatively few discontinuities), rock breaker excavation methods only would be expected to be very slow and potentially severely damaging to equipment.

In confined (trench, footing, etc.) excavations in medium to high strength rock, heavy rock breaker equipment and slow excavation rates should be allowed for. Due to the inherent jointing and bedding planes contained in the rock, over break should be allowed for in pricing.

Consideration should be given in selecting suitable excavation methods/plant to the potential of encountering 'harder' rock below test pit location termination depths, and at 'shallower' depth intermediate to the test pit locations.

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All confined excavations should be fully supported or battered/benched to a stable angle to ensure personnel safety.

5.2.2 *Use of Cut for Fill*

Organic soils, 'over-wet' soils, 'silts' and soils containing deleterious matter or oversize particles (>75mm size) should be excluded from use as structural fill.

The soils and extremely low to low strength rock should generally be suitable for re-use as 'controlled fill' provided that the excavated material is 'processed' so that it is well mixed and all 'oversize', organic/deleterious and any 'over wet' materials are excluded and expansive movement can be tolerated or designed for. All medium to very high strength rock would be suitable for reuse as fill, but crushing and screening is likely to be required to control particle size for 'hard' rock.

5.2.3 *Fill Compaction*

All fill required to be placed to support settlement sensitive structures/features should be 'controlled', placed in layers not greater than 250mm (loose thickness) and be uniformly compacted to a minimum dry density ratio of 98% (Standard compaction). Where fill is to be placed on sloping sections of the site, the fill must be adequately 'keyed in' at subgrade level. Detailed earthworks and building plans should be carefully reviewed prior to construction to ensure global slope stability can be maintained.

Reactive clay material should be avoided for use as fill, if possible. However if/where any reactive material is to be used as fill, it should be placed and maintained at a moisture content of not drier than Standard Optimum Moisture Content (OMC) in order to reduce potential shrink-swell movements. It should be noted that over-compacting reactive clay fill (particularly at a moisture content below optimum) should be avoided as potentially significant expansion could occur on 'wetting up'. Due allowance must be made in design and detailing for reactive fill movements if reactive fill is used.

To assist with achievement of adequate control of fill placement, 'Level 1' geotechnical supervision and testing as set out in Section 8 of AS3798 – 2007 *Guidelines on earthworks for commercial and residential developments* is recommended.

5.2.4 *Trafficability*

Trafficability for plant will be adversely affected by wet weather and traffic within 'wet' subgrade during and following wet weather would be expected to potentially result in disturbance to the subgrade, with consequent loss of subgrade strength. Consideration should be given to the placement of a coarse granular working platform to those areas where trafficability is critical. The required layer thickness will depend on the type of plant proposed to traffic the site, however, a layer thickness of not less than 150mm is anticipated for 'light' equipment.

5.2.5 *Site Drainage*

During construction, the site should be graded such that water is readily shed and does not collect and pond over the site, otherwise softening of soil and weathered rock subgrade will occur, especially under construction plant traffic and heavy vehicles.

5.3 Reactive Ground Movements

5.3.1 Estimated Magnitude

The magnitude of potential reactive soil movements can be estimated using the following equation (from Australian Standard AS2870-2011 *Residential slabs and footings*) and parameters for the site selected based on recommendations in AS2870:

$$y_s = \frac{1}{100}\sum_{n=1}^{N}\left(\alpha.I_{ss}.\overline{\Delta u}.h\right)_n$$

where y_s is the characteristic surface movement, in millimetres;

α is the lateral restraint factor;

I_{ss} is the shrink-swell index (taken as approximately 2.0% per pF to 3.5% per pF for the site clays, based on a visual/tactile assessment and past experience with soils of similar grading and plasticity);

$\overline{\Delta u}$ is the soil suction change averaged over the thickness of the layer under consideration (estimated as 1.2pF in Rockhampton);

h is the thickness of layer under consideration, in millimetres; and

N is the number of soil layers within the design depth of suction change (H_s), which has been taken as 2.3m in Rockhampton.

The potential characteristic surface movement values for the stratigraphy encountered in the test pits have been calculated to be approximately 20mm to 35mm using the methods and parameters discussed above, assuming normal seasonal moisture/suction variations. Based on the magnitude of the calculated characteristic surface movement, the site in its current condition would be classified as 'Class M' (Moderately Reactive).

5.3.2 Design Considerations

The clays encountered at the site are expected to generally have low to moderate potential for shrink-swell movements associated with change in moisture content. If recompacted, the potential reactivity of these materials will increase, which will need to be allowed for in the estimation of future shrink-swell movement.

Use of reactive materials for fill should be avoided, however, if their use cannot be avoided then the calculated characteristic surface movement value would increase significantly. It should be carefully noted that the calculated surface movement values given above do not include any allowance for 'abnormal' influences such as vegetation effects. It is strongly recommended that the estimated characteristic surface movement values for the site be recalculated once site earthworks design is completed and fill sources are known. It is considered that the following issues must be carefully considered in design:

- Where reactive fill is placed over a natural soil subgrade, higher characteristic movements than those nominated above could potentially occur (as the ratio of lateral restrained to unrestrained movement will increase), particularly if the fill reactivity is greater than that of the existing site soils. If filling of the site is proposed, a revised site classification should be considered, which takes into account the actual reactivity, compaction and depth of fill used.

- Vegetation (particularly large trees) has the potential to significantly increase soil suction change magnitude and depth, which leads to a significant increase in potential reactive soil movements adjacent to any (proposed) tree locations. If trees are to be planted 'close' to proposed footings in the future, consideration should be given to constructing root barriers around the trees, and footing design must allow for potentially (significantly) higher reactive soil movements than are nominated above.
- Abnormal subgrade moisture variations could potentially result in adverse, non-uniform reactive movements that are significantly greater than those nominated above for 'normal' seasonal moisture changes. The risk of 'abnormal' movement occurring could be reduced by ensuring over-watering of gardens, ponding water, broken/leaking pipes, 'close' planting of trees/shrubs, etc. does not occur.

'Good practice' should be adopted in project design and detailing if control of reactive ground movement is desired. In particular, the following are recommended:

- trees/shrubs should not be planted closer than their mature height to movement sensitive features (unless significantly greater reactive movements than those estimated above are designed for);
- subgrade moisture content should not be allowed to change during or following construction;
- site grades should be designed to readily shed water and prevent ponding around footings and other movement sensitive areas;
- services should be designed to be flexible and to prevent any leakage and to rapidly promote removal of fluid if leakage does occur; and
- proposed structures should be made as flexible as possible, with regular full height movement control joints, flexible in-fill above windows and doors etc.

5.4 Slope Stability Assessment

At the time of the investigation, there was no observable evidence of instability at the proposed dwelling location or nearby surrounds. Some minor scouring and erosion was observed along the southern boundary, where the driveway is proposed, however, was not noted in surrounding areas where grass cover exists. Outcropping sandstone and large boulders (up to approximately 1.1m, which appeared to be well embedded) were observed across the site. The undeveloped area to the west of the proposed dwelling location was generally dense bushland comprising sparse grasses, shrubs and small to large trees showing no observable signs of movement/creep of the soils (e.g. bent trees, etc.).

5.4.1 Analysis Method

Preliminary slope stability analysis has been undertaken using the commercially available geotechnical analysis software package Slope/W, which uses limit equilibrium methods to assess the Factor of Safety (FOS) against slope instability. The analysis carried out has adopted the following:

- Approximate slope geometry based on ground surface levels interpolated from Rockhampton Regional Council's online mapping database;
- Subsurface profiles based on the results of the test pits;
- Mohr-Coulomb strength model for soils;
- 'Long term' analysis carried out using assumed effective stress soil strength parameters.

5.4.2 Interpretation of Calculated Factor of Safety Values

In the 'long term' it is typical to adopt a minimum calculated FOS in the range of 1.4 to 1.5, depending on the level of uncertainty in input parameters. Where detailed investigation has been carried out and applied loads are well defined, a FOS at the low end of the range could be considered, however, as the degree of uncertainty in parameters, geometry, applied loads, groundwater conditions and variability increases the acceptable FOS limit from slope stability analysis should increase.

5.4.3 Material Properties

The stratigraphy and soil/rock strength properties adopted in the analysis are given in Table 8.

Table 8: Summary of soil/rock strength properties adopted

Material	Strength	Colour	Unit Weight (kN/m^3)	Apparent Cohesion (kPa)	Friction Angle (degrees)
Gravelly/Sandy Silt	very stiff		19	2	24
Gravelly/Sandy Clay	very stiff/hard		19	5	28
Sandstone	medium to very high		22	30	38

5.4.4 Analysis Results

An automated search of potential circular failure surfaces was carried out to assess the failure surface with the lowest calculated FOS. The analysis has been undertaken for an idealised ground condition, generally based on the results from the test pits. The stratigraphy and geometry adopted are presented in Figure 1, with results of the analysis presented in Figure 2 showing the failure surface with the lowest calculated FOS.

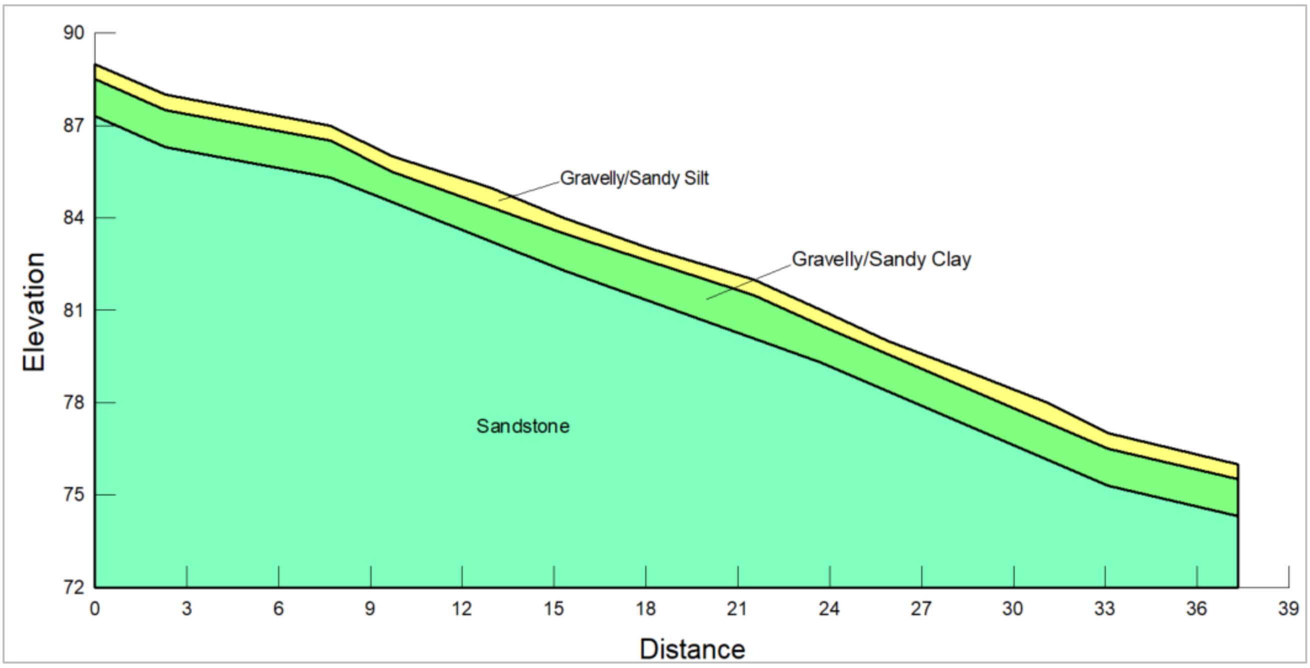

Figure 1: Stratigraphy and geometry adopted.

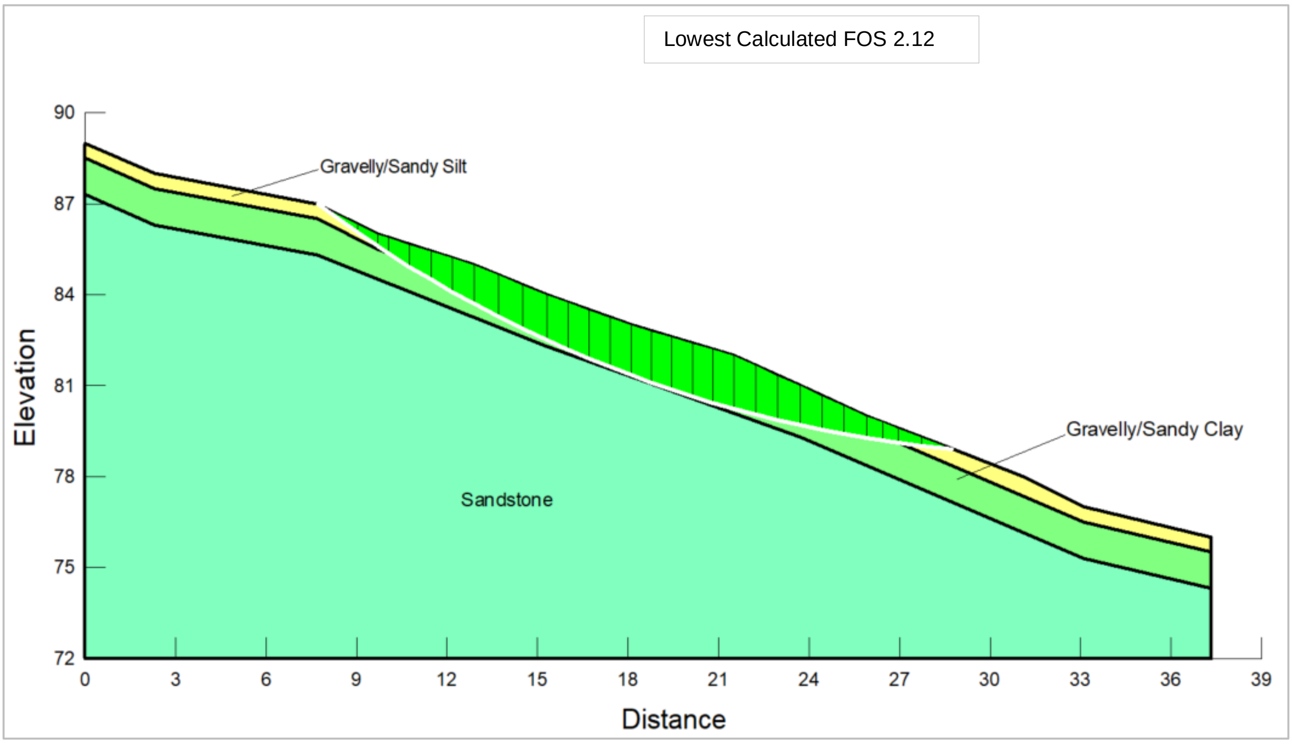

Figure 2: Lowest calculated FOS with 'Long Term' conditions.

5.4.5 Landslide Risk

Based on the preliminary stability analysis results, the site in its current condition would be considered **'low'** risk. However, it must be noted that the slope geometry used in the analysis is approximate only, and that a detail survey of ground surface levels must be obtained to reassess slope stability. In addition, future earthworks and construction at the site have the potential to affect the likelihood of instability occurring as well as the consequences if a failure were to occur and specific design and analysis for the proposed development will also be required to confirm that instability risk remains low for the development as constructed.

It is anticipated that, provided the recommendations included in this report are followed, and detailed design and location specific analysis (using more accurate survey information) is carried out, a development with low landslide risk is feasible.

5.5 General Good Practice for Slope Stability

As a broad preliminary guide to the placement of future buildings, it is suggested that buildings be founded into low strength (or stronger) sandstone. If any fill is proposed to be placed as part of site development, it must be 'keyed-in' to the natural slope at no greater than 0.25m high 'benches'.

To minimise the potential for the proposed development being adversely affected by potential slope instability, it is considered that the design and construction procedures described below, and in the following sections of this report, should be adopted:

- care must be taken to minimise disturbance to the site by cutting and filling, unless areas to be cut/filled are appropriately battered/retained and slope stability assessment is carried out on a case-by-case basis to confirm adequate FOS;
- adequate site slope drainage must be provided and maintained to minimise the potential for groundwater induced instability;

- stormwater run-off and sewage effluent (e.g. septic tanks, etc.) should be piped away from these areas; and
- all vegetation destroyed as a result of construction activities should be restored wherever possible and densified with new plantings, as soon as practicable after completion of earthworks.

Excavations for in-ground services should be kept to a minimum or avoided if possible; but where necessary, they should be backfilled with properly compacted materials and capped with an impervious layer to minimise potential surface water ingress into the backfill and potentially the subsurface profile. It is also recommended that the alignment of service trenches be perpendicular to site ground surface contours, if possible.

Any alterations to the existing topography at the site will need to be analysed in detail and a re-assessment of soil instability hazard carried out.

Figure 3 and Figure 4 illustrate methods of good hillside practice that should be adopted for design.

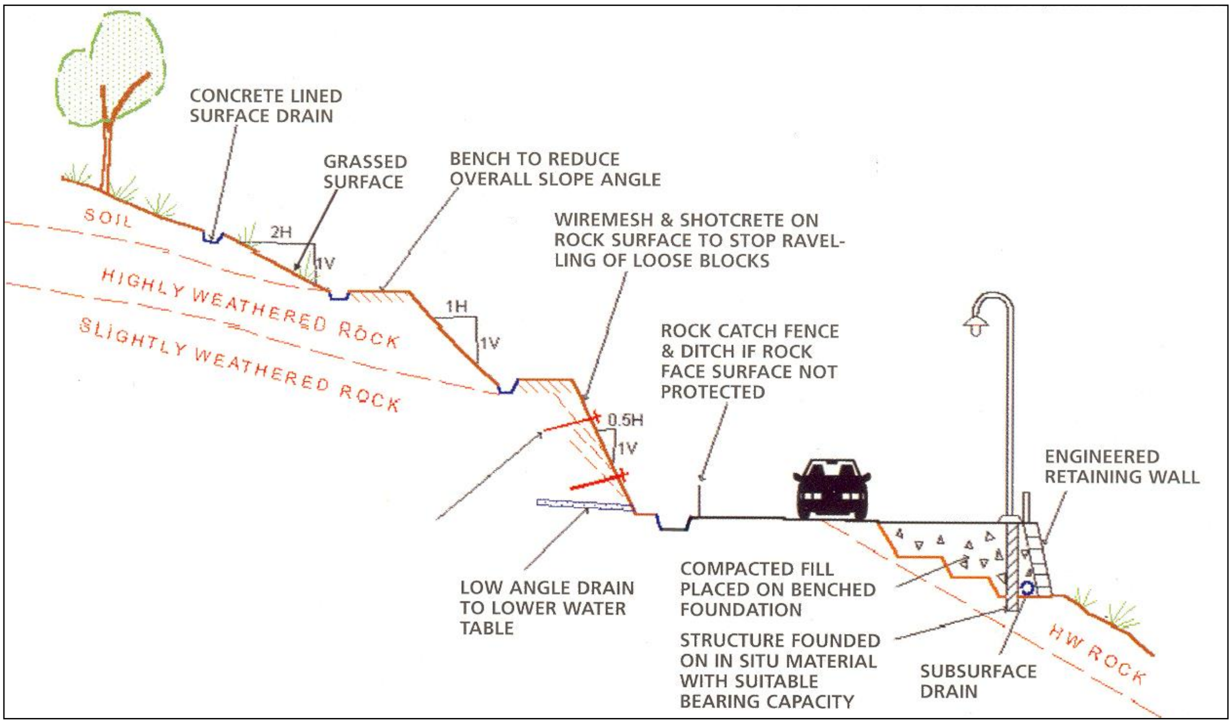

Figure 3: Possible methods of maintaining stability in cut and fill developments.

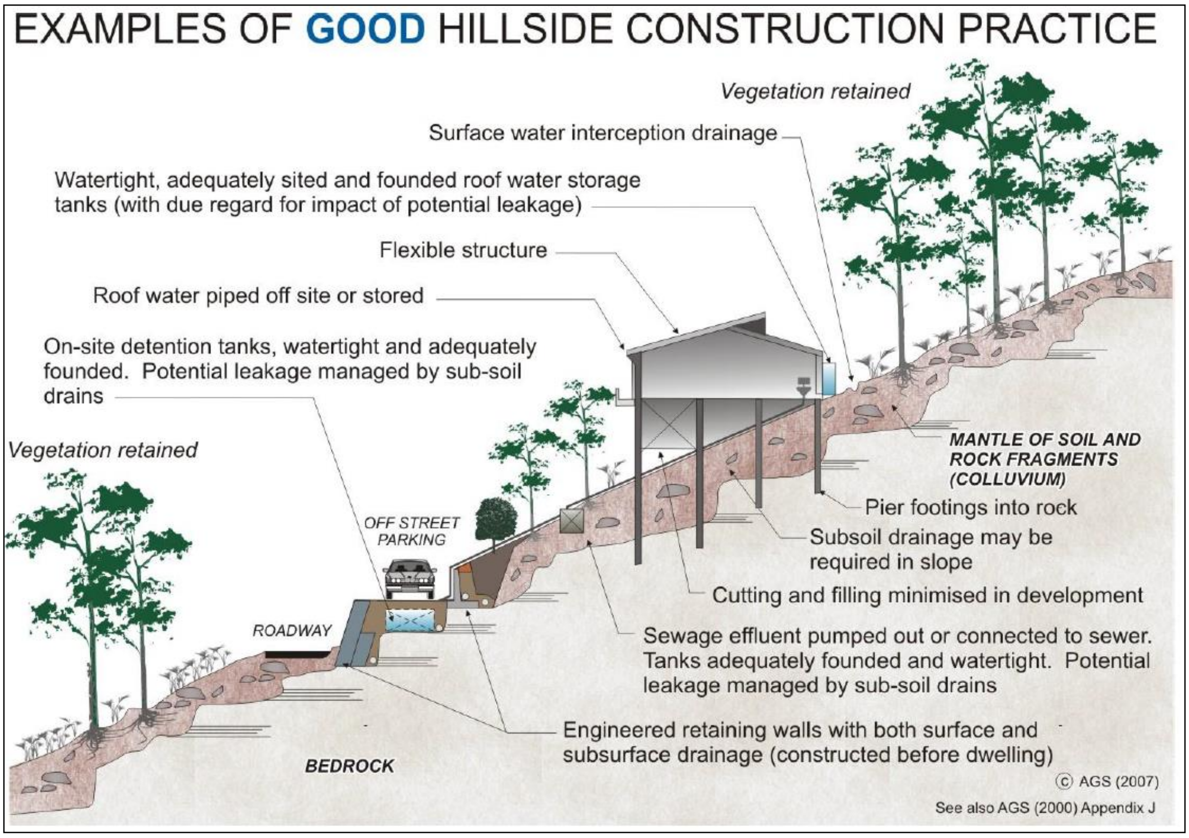

Figure 4: Good hillside practice.

5.6 Batter Slopes

If movement sensitive features/sections are not located 'close' to excavations, and geometry permits, battered slopes may be adopted. Provided slopes are protected from groundwater or surface water effects, the preliminary maximum cut slope angles given in the Table 9 may be used with a relatively low risk of instability for unsurcharged batters up to approximately 2m to 3m in height. Where batters exceed 3m in height mid-slope benches (not less than 3m wide) may be required and will require detailed stability assessment on a location by location basis.

Table 9: Preliminary Maximum Unsurcharged Batter Slopes for 'Dry' Slopes up to 2m to 3m high.

Material	Strength	Temporary Batter(1) (2)	Permanent Batter(1) (2)
Level 1 Controlled Fill (refer Section 5.2.3)	–	1V:1H	1V:2.5H
Clays/Silts	stiff to hard	1V:1H	1V:2H
Sandstone (rock)	low medium high	1V:1H(3) 1V:0.75H(3) 1V:0.5H(3)	1V:1.5H(3) 1V:1H(3) 1V:1H(3)

(1) Not underlain by 'softer' materials and subject to confirmation by engineering analysis and inspection during construction

(2) Flatter if 'wet'

(3) Depends on jointing

If insufficient space exists for the construction of batters at the maximum slopes given above, mechanical excavation support will be required in order to prevent excavation instability. At the batter angles nominated above there may be some localised slumping of batter slopes and it will be necessary to ensure that the faces are protected from any surface water or groundwater seepage effects.

Detailed stability analysis, with specific ground surface levels, prior to bulk earthworks design finalisation will be required to confirm stable batter slopes and detailed inspection by an experienced geotechnical engineer will be required at the time of construction to confirm the stability of batter faces and the need for any supplementary mechanical support.

5.7 Retaining Wall Pressures

An estimate of 'unsurcharged' retaining wall pressures for 'flexible' and 'rigid' walls can be obtained for drained conditions and a horizontal retained surface, using a triangular pressure distribution in conjunction with the parameters given in Table 10.

Table 10: Retaining Wall Design Parameters

Material Type	Strength/ Density	Total Weight (t/m^3)	Flexible Wall 'Active' Pressure Coefficient (k_a)	Rigid Wall 'At Rest' Pressure Coefficient (k_o)
Level 1 Controlled Fill (refer Section 5.2.3) – 100% Standard	–	1.9	0.40	0.60
Clays/Silts	stiff to hard	1.9	0.40	0.60
Sandstone (rock)	low medium high	2.3	0.35 0.30 0.20	0.50 0.40 0.30

Due allowance must also be included in the calculation of wall pressure for groundwater pressure, back fill compaction, surcharge effects from adjacent structures and/or construction loading, the effects of sloping retained materials, reactive soil/fill pressures, etc.

If a drainage system is installed behind retaining walls, it would still be prudent to allow for elevated water pressures as elevated groundwater levels may occur during or following prolonged 'wet' weather, or from blocked drainage etc. Drain design should incorporate free draining backfill and slotted pipe discharging into a sealed disposal system.

5.8 Foundations

Design of pad/strip footings or 'short' bored piles could be based on the maximum allowable working bearing pressures nominated in Table 11. Ultimate bearing stress design values can be obtained by multiplying the working stress bearing pressure values given in Table 11 by 2.5.

Table 11: Working Bearing Pressure for Strip/Pad Footings and 'Short' Bored Piles

Material	Strength	Allowable Working Bearing Pressure (kPa)*
Gravelly/Sandy Silt	-	not recommended
Gravelly/Sandy Clay	stiff very stiff hard	100 150 250
Sandstone (rock)	extremely low very low low medium (or stronger)	300 600 1000 2500

* Not underlain by any 'softer' material

It is recommended that the above strengths be confirmed by an experienced geotechnical engineer prior to the casting of foundation elements. It should be carefully noted that the potential presence of 'strength inversions' in the rock will require careful consideration in foundation design and the selection of maximum bearing pressures/founding depths.

It is considered that local variations in rock strength could be expected to occur over the site and it is suggested that a 'flexible' approach be adopted to the foundation design, construction methodology and costing, so that footing sizes/founding depths can be readily adjusted as required during construction, without cost/time penalties being incurred.

It is recommended that in order to minimise potential differential footing performance that all footings be extended to found in similar stratigraphy (i.e. footings for a particular structure should not found partly in soil and partly in weathered rock).

5.9 On-Ground Slab and Pavement Properties

5.9.1 Insitu Estimates of CBR

The correlation between DCP results and insitu CBR given by AUSTROADS[1], is reproduced in Figure 5 and can be used to estimate the CBR of proposed subgrade materials. Caution should be exercised with the interpretation of the DCP values as they are only relevant for the moisture conditions existing at the time of testing and 'false' interpreted CBR values can result from the presence of gravels etc. contained with otherwise 'clayey' soils.

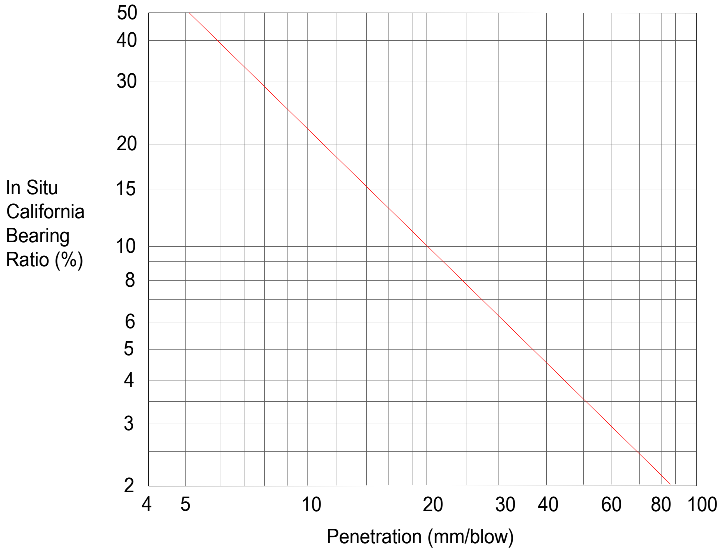

Figure 5: Correlation of DCP Results and Insitu CBR

5.9.2 On-Ground Slabs and Pavements

The design of on-ground slabs and pavements, cast over natural soil or controlled fill subgrade could be based on the 'soaked' parameters presented in Table 12, which are based on the results of the investigation and past experience with similar soil/rock and on the assumption that the subgrade is prepared in accordance with Section 5.2.

The subgrade design values will be significantly influenced by the properties of any compacted fill used.

[1] AUSTROADS' Publication No. AP-17/92 (1992) Pavement Design: A Guide to the Structural Design of Road Pavements – Figure 5.2.

Table 12: Preliminary Subgrade Design Values

Subgrade Material	CBR (%)	Modulus of Subgrade Reaction (kPa/mm)[1]
Gravelly/Sandy Clay (natural or controlled fill)	3 – 6	30 - 40
Sandstone (extremely low to very low strength)[2]	5 – 10 (undisturbed) 3 – 7 (disturbed)	40 – 50 30 – 45
Sandstone (medium strength or stronger)	20 – 35	70 – 110

(1) For transient loading only

(2) May breakdown under compaction, leading to degraded properties

If reactive ground movement can occur, it is suggested that on-ground slabs be fully dowelled (and joints between slabs sealed to control differential movements and minimise under-slab moisture changes) and should be detailed to enable movement, independent of foundations, fixtures, etc.

Reactive subgrade materials should not be allowed to 'dry out', otherwise significant softening and soil-swell movements on 'wetting up' could potentially occur.

BUTLER PARTNERS (REGIONAL) PTY LTD

JENNY SALAS
Senior Geotechnical Engineer

NICK BLOXSOM
Senior Geotechnical Engineer

Important Information about Your

Geotechnical Engineering Report

Subsurface problems are a principal cause of construction delays, cost overruns, claims, and disputes.

While you cannot eliminate all such risks, you can manage them. The following information is provided to help.

Geotechnical Services Are Performed for Specific Purposes, Persons, and Projects

Geotechnical engineers structure their services to meet the specific needs of their clients. A geotechnical engineering study conducted for a civil engineer may not fulfill the needs of a construction contractor or even another civil engineer. Because each geotechnical engineering study is unique, each geotechnical engineering report is unique, prepared *solely* for the client. No one except you should rely on your geotechnical engineering report without first conferring with the geotechnical engineer who prepared it. *And no one — not even you* — should apply the report for any purpose or project except the one originally contemplated.

Read the Full Report

Serious problems have occurred because those relying on a geotechnical engineering report did not read it all. Do not rely on an executive summary. Do not read selected elements only.

A Geotechnical Engineering Report Is Based on A Unique Set of Project-Specific Factors

Geotechnical engineers consider a number of unique, project-specific factors when establishing the scope of a study. Typical factors include: the client's goals, objectives, and risk management preferences; the general nature of the structure involved, its size, and configuration; the location of the structure on the site; and other planned or existing site improvements, such as access roads, parking lots, and underground utilities. Unless the geotechnical engineer who conducted the study specifically indicates otherwise, do not rely on a geotechnical engineering report that was:

- not prepared for you,
- not prepared for your project,
- not prepared for the specific site explored, or
- completed before important project changes were made.

Typical changes that can erode the reliability of an existing geotechnical engineering report include those that affect:

- the function of the proposed structure, as when it's changed from a parking garage to an office building, or from a light industrial plant to a refrigerated warehouse,
- elevation, configuration, location, orientation, or weight of the proposed structure,
- composition of the design team, or
- project ownership.

As a general rule, *always* inform your geotechnical engineer of project changes—even minor ones—and request an assessment of their impact. *Geotechnical engineers cannot accept responsibility or liability for problems that occur because their reports do not consider developments of which they were not informed.*

Subsurface Conditions Can Change

A geotechnical engineering report is based on conditions that existed at the time the study was performed. *Do not rely on a geotechnical engineering report* whose adequacy may have been affected by: the passage of time; by man-made events, such as construction on or adjacent to the site; or by natural events, such as floods, earthquakes, or groundwater fluctuations. *Always* contact the geotechnical engineer before applying the report to determine if it is still reliable. A minor amount of additional testing or analysis could prevent major problems.

Most Geotechnical Findings Are Professional Opinions

Site exploration identifies subsurface conditions only at those points where subsurface tests are conducted or samples are taken. Geotechnical engineers review field and laboratory data and then apply their professional judgment to render an opinion about subsurface conditions throughout the site. Actual subsurface conditions may differ—sometimes significantly—from those indicated in your report. Retaining the geotechnical engineer who developed your report to provide construction observation is the most effective method of managing the risks associated with unanticipated conditions.

A Report's Recommendations Are *Not* Final

Do not overrely on the construction recommendations included in your report. *Those recommendations are not final,* because geotechnical engineers develop them principally from judgment and opinion. Geotechnical engineers can finalize their recommendations only by observing actual subsurface conditions revealed during construction. *The geotechnical engineer who developed your report cannot assume responsibility or liability for the report's recommendations if that engineer does not perform construction observation.*

A Geotechnical Engineering Report Is Subject to Misinterpretation

Other design team members' misinterpretation of geotechnical engineering reports has resulted in costly problems. Lower that risk by having your geotechnical engineer confer with appropriate members of the design team after submitting the report. Also retain your geotechnical engineer to review pertinent elements of the design team's plans and specifications. Contractors can also misinterpret a geotechnical engineering report. Reduce that risk by having your geotechnical engineer participate in prebid and preconstruction conferences, and by providing construction observation.

Do Not Redraw the Engineer's Logs

Geotechnical engineers prepare final boring and testing logs based upon their interpretation of field logs and laboratory data. To prevent errors or omissions, the logs included in a geotechnical engineering report should *never* be redrawn for inclusion in architectural or other design drawings. Only photographic or electronic reproduction is acceptable, *but recognize that separating logs from the report can elevate risk.*

Give Contractors a Complete Report and Guidance

Some owners and design professionals mistakenly believe they can make contractors liable for unanticipated subsurface conditions by limiting what they provide for bid preparation. To help prevent costly problems, give contractors the complete geotechnical engineering report, *but* preface it with a clearly written letter of transmittal. In that letter, advise contractors that the report was not prepared for purposes of bid development and that the report's accuracy is limited; encourage them to confer with the geotechnical engineer who prepared the report (a modest fee may be required) and/or to conduct additional study to obtain the specific types of information they need or prefer. A prebid conference can also be valuable. *Be sure contractors have sufficient time* to perform additional study. Only then might you be in a position to give contractors the best information available to you, while requiring them to at least share some of the financial responsibilities stemming from unanticipated conditions.

Read Responsibility Provisions Closely

Some clients, design professionals, and contractors do not recognize that geotechnical engineering is far less exact than other engineering disciplines. This lack of understanding has created unrealistic expectations that have led to disappointments, claims, and disputes. To help reduce the risk of such outcomes, geotechnical engineers commonly include a variety of explanatory provisions in their reports. Sometimes labeled "limitations" many of these provisions indicate where geotechnical engineers' responsibilities begin and end, to help others recognize their own responsibilities and risks. *Read these provisions closely.* Ask questions. Your geotechnical engineer should respond fully and frankly.

Geoenvironmental Concerns Are Not Covered

The equipment, techniques, and personnel used to perform a *geoenvironmental* study differ significantly from those used to perform a *geotechnical* study. For that reason, a geotechnical engineering report does not usually relate any geoenvironmental findings, conclusions, or recommendations; e.g., about the likelihood of encountering underground storage tanks or regulated contaminants. *Unanticipated environmental problems have led to numerous project failures.* If you have not yet obtained your own geoenvironmental information, ask your geotechnical consultant for risk management guidance. *Do not rely on an environmental report prepared for someone else.*

Obtain Professional Assistance To Deal with Mold

Diverse strategies can be applied during building design, construction, operation, and maintenance to prevent significant amounts of mold from growing on indoor surfaces. To be effective, all such strategies should be devised for the *express purpose* of mold prevention, integrated into a comprehensive plan, and executed with diligent oversight by a professional mold prevention consultant. Because just a small amount of water or moisture can lead to the development of severe mold infestations, a number of mold prevention strategies focus on keeping building surfaces dry. While groundwater, water infiltration, and similar issues may have been addressed as part of the geotechnical engineering study whose findings are conveyed in this report, the geotechnical engineer in charge of this project is not a mold prevention consultant; ***none of the services performed in connection with the geotechnical engineer's study were designed or conducted for the purpose of mold prevention. Proper implementation of the recommendations conveyed in this report will not of itself be sufficient to prevent mold from growing in or on the structure involved.***

Rely, on Your ASFE-Member Geotechncial Engineer for Additional Assistance

Membership in ASFE/The Best People on Earth exposes geotechnical engineers to a wide array of risk management techniques that can be of genuine benefit for everyone involved with a construction project. Confer with your ASFE-member geotechnical engineer for more information.

ASFE
THE BEST PEOPLE ON EARTH

8811 Colesville Road/Suite G106, Silver Spring, MD 20910
Telephone: 301/565-2733 Facsimile: 301/589-2017
e-mail: info@asfe.org www.asfe.org

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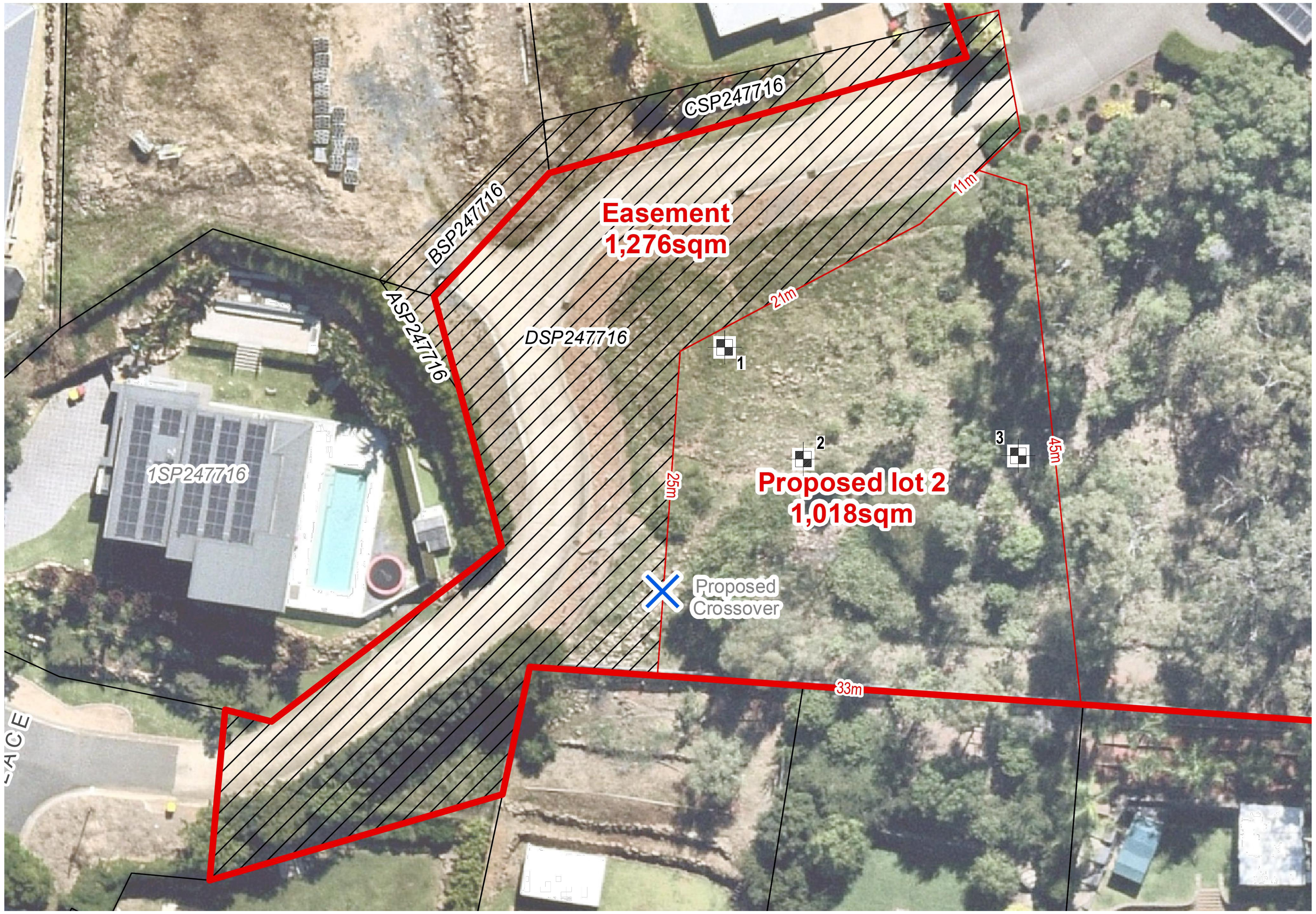

LEGEND

1 Test Pit

Butler Partners
geotechnical • geo-environmental • groundwater

Butler Partners (Regional) Pty Ltd
ABN 43 603 849 305
246 Kent Street
Rockhampton Qld 4700
PO Box 1400
Rockhampton Qld 4700

Ph: 61 7 4927 1400
Fx: 61 7 4927 1800
enquiries@butlerpartners.com.au

Site

Reference: Nearmap, 2023

Slope Stability Assessment and Geotechnical Investigation Proposed Subdvision

4 Vanderspek Place, Frenchville

Locality Plan and Test Locations

CLIENT: **Reel Planning Pty Ltd**

SCALE AT A3:

Not to Scale

DATE:	AUGUST 2023
DRAWN:	LA
APPROVED:	NB
PROJECT No:	RG23-1178A
DRAWING No: 1	REV: A

File Location: P:\Projects 2023\RG23-1178A - Michael Swann - 4 Vanderspek Place, Frenchville\Graphics\Drawings\RG23-1178A Dwg1 RevA 240823.dwg

Reference: Reel Planning Pty Ltd, 'Proposed Subdivision Plan', 4 Vanderspek Place, Frenchville, dated 30/05/2023.

APPENDIX A

TEST PIT REPORT SHEETS WITH EXPLANATORY NOTES

TEST PIT REPORT

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TEST PIT 1

Client: Reel Planning Pty Ltd
Project: Proposed Subdivision
Location: 4 Vanderspek Place, Frenchville
Project No: RG23-1178A

Page No: 1 of 1
Date: 21/07/2023
Ground Surface Level: RL 83.0m*

Depth (m)	Description	RL (m)	Lithology	Sample Type	Depth (m)	Test Results
		83.0				
0	***GRAVELLY SANDY SILT (ML)*** - very stiff, dark brown, fine to coarse angular to subangular gravel, fine to coarse grained sand, with cobbles, with tree roots and organics (topsoil)					
	GRAVELLY SANDY CLAY (CI) -stiff, brown-red, fine to medium angular gravel, fine to coarse grained sand, with cobbles and boulders (up to 600 mm long)				0.4	PP=350
	- stiff to very stiff, orange-red, fine to coarse angular to subangular gravel			B	0.6	PP=200
					0.8	PP=300
1		82.0			1.0	PP=350
				B	1.4	
	SANDSTONE (HW) - medium to very high strength, pale brown mottled green and pink, fine to medium grained.			HS	1.6	Is(50)=0.7-6.1
	End of Test Pit at 1.9 m (Bucket Refusal)				1.9	
2		81.0				

D	**Disturbed Sample**	**E**	**Environmental Sample**	**pp**	**Pocket Penetrometer Test (kPa)**
B	**Bulk Sample**	**U**	**Undisturbed Tube (50mm diameter)**	**Is(50)**	**Point Load Test Result (MPa)**
V	**Vane Shear Strength (Uncorrected)kPa**	**A**	**Asbestos**	**HS**	**Hand Sample**

Excavator: Hitachi 12 tonne Tracked Excavator
Logged by: JS/MG
Bucket Size: 600 mm
Groundwater: No free groundwater encountered during excavation
Remarks: * Approx. ground surface level interpolated from Rockhampton Regional Council's online mapping database, viewed on 10 August 2023.

TEST PIT REPORT

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Client: Reel Planning Pty Ltd
Project: Proposed Subdivision
Location: 4 Vanderspek Place, Frenchville
Project No: RG23-1178A

TEST PIT 2

Date: 21/07/2023
Ground Surface Level: RL 80.0m*

Depth (m)	Description	RL (m)	Lithology	Sample Type	Depth (m)	Test Results
0	***COBBLES AND BOULDERS*** - gravelly sandy clay matrix, dark brown, 63 mm - 200 mm sized angular cobbles, 200 mm - 1100 mm sized angular to subangular boulders (with organics)	80.0		B	0.2	
	GRAVELLY SANDY CLAY (CL) - hard, orange-red, fine to coarse angular gravel, fine to coarse grained sand, with cobbles and boulders			D	0.5	PP>600
					0.6	PP>600
	End of Test Pit at 0.7 m (Bucket Refusal on Rock)					
1		79.0				
2		78.0				

D	**Disturbed Sample**	**E**	**Environmental Sample**	**pp**	**Pocket Penetrometer Test (kPa)**
B	**Bulk Sample**	**U**	**Undisturbed Tube (50mm diameter)**	**Is(50)**	**Point Load Test Result (MPa)**
V	**Vane Shear Strength (Uncorrected)kPa**	**A**	**Asbestos**	**HS**	**Hand Sample**

Excavator: Hitachi 12 tonne Tracked Excavator
Logged by: JS/MG
Bucket Size: 600 mm
Groundwater: No free groundwater encountered during excavation
Remarks: * Approx. ground surface level interpolated from Rockhampton Regional Council's online mapping database, viewed on 10 August 2023.

TEST PIT REPORT

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TEST PIT 3

Client: Reel Planning Pty Ltd
Project: Proposed Subdivision
Location: 4 Vanderspek Place, Frenchville
Project No: RG23-1178A

Page No: 1 of 1
Date: 21/07/2023
Ground Surface Level: RL 81.0m*

Depth (m)	Description	RL (m)	Lithology	Sample Type	Depth (m)	Test Results
0	***SANDY SILT (ML)*** - very stiff, dark brown, fine to coarse grained sand, with fine to coarse angular gravel, with cobbles and boulders (with organics)	81.0			0.2	PP=400
	GRAVELLY CLAY (CI) - hard, pale brown, fine to coarse angular gravel, trace fine to coarse grained sand, with cobbles and boulders			B	0.5 0.6	PP>600
	SANDSTONE (MW) - high to very high strength, pale brown mottled green and pink, fine to medium grained			HS	0.8 1.0	Is(50)=2.4-3.3
1	End of Test Pit at 1 m (Bucket Refusal)	80.0				
2		79.0				

D	**Disturbed Sample**	**E**	**Environmental Sample**	**pp**	**Pocket Penetrometer Test (kPa)**
B	**Bulk Sample**	**U**	**Undisturbed Tube (50mm diameter)**	**Is(50)**	**Point Load Test Result (MPa)**
V	**Vane Shear Strength (Uncorrected)kPa**	**A**	**Asbestos**	**HS**	**Hand Sample**

Excavator: Hitachi 12 tonne Tracked Excavator
Logged by: JS/MG
Bucket Size: 600 mm
Groundwater: No free groundwater encountered during excavation
Remarks: * Approx. ground surface level interpolated from Rockhampton Regional Council's online mapping database, viewed on 10 August 2023.

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Rockhampton Laboratory
246 Kent Street
Rockhampton Queensland 4700
Telephone : 61 (07) 4927 1400

Dynamic Cone Penetrometer

Test Method: AS1289.6.3.2

Client:	Reel Planning Pty Ltd	**Report No.:**	**RG23-1178A-001**
Project:	Proposed Subdivision	Tested by:	MG
Location:	4 Vanderspek Place, Frenchville	Date:	21/07/23
		Checked by:	NB
Project No.:	RG23-1178A	Date:	24/07/23

THIS DOCUMENT SHALL NOT BE REPRODUCED EXCEPT IN FULL

PENETRATION RESISTANCE - BLOWS / 100mm

Depth (m):	Test Pit No.					
	1	**2**	**3**			
0.0-0.1	*14*	*4*	*5*			
0.1-0.2	*14*	*4*	*15*			
0.2-0.3	*21*	*4*	*6*			
0.3-0.4	*Refusal*	*21*	*6*			
0.4-0.5		*Refusal*	*8*			
0.5-0.6			*13*			
0.6-0.7			*14*			
0.7-0.8			*21*			
0.8-0.9			*Refusal*			
0.9-1.0						
1.0-1.1						
1.1-1.2						
1.2-1.3						
1.3-1.4						
1.4-1.5						
1.5-1.6						
1.6-1.7						
1.7-1.8						
1.8-1.9						
1.9-2.0						
2.0-2.1						
2.1-2.2						
2.2-2.3						
2.3-2.4						
2.4-2.5						
2.5-2.6						
2.6-2.7						
2.7-2.8						
2.8-2.9						
2.9-3.0						
3.0-3.1						
3.1-3.2						
3.2-3.3						
3.3-3.4						

Comments:

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Notes on Description and Classification of Soil

The methods of description and classification of soils used in this report are generally based on Australian Standard AS1726-1993 Geotechnical Site Investigations.

Soil description is based on an assessment of disturbed samples, as recovered from bores and excavations, or from undisturbed materials as seen in excavations and exposures or in undisturbed samples. Descriptions given on report sheets are an interpretation of the conditions encountered at the time of investigation.

In the case of cone or piezocone penetrometer tests, actual soil samples are not recovered and soil description is inferred based on published correlations, past experience and comparison with bore and/or test pit data (if available).

Soil classification is based on the particle size distribution of the soil and the plasticity of the portion of the material finer than 0.425mm. The description of particle size distribution and plasticity is based on the results of visual field estimation, laboratory testing or both. When assessed in the field, the properties of the soil are estimated; precise description will always require laboratory testing to define soil properties.

Where soil can be clearly identified as FILL this will be noted as the main soil type followed by a description of the composition of the fill (e.g. FILL – yellow-brown, fine to coarse grained gravelly clay fill with concrete rubble). If the soil is assessed as possibly being fill this will be noted as an additional observation.

Soils are generally described using the following sequence of terms. In certain instances, not all of the terms will be included in the soil description.

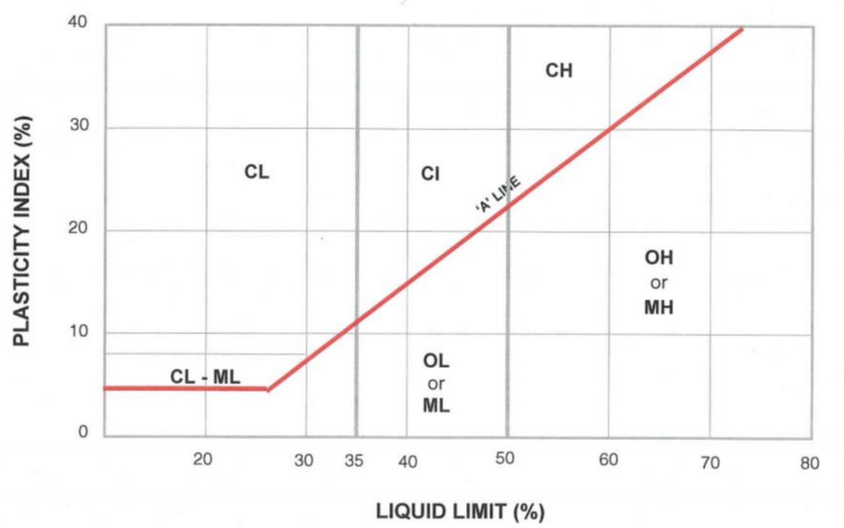
MAIN SOIL TYPE (CLASSIFICATION GROUP SYMBOL)
- strength/density, colour, structure/grain size, secondary and minor components, additional observations

Information on the definition of descriptive and classification terms follows.

<u>SOIL TYPE and CLASSIFICATION GROUP SYMBOLS</u>

	Major Divisions	Particle Size	Classification Group Symbol	Typical Names
COARSE GRAINED SOILS (more than half of material is larger than 0.075mm)	BOULDERS	>200mm		
	COBBLES	63 – 200mm		
	GRAVELS (more than half of coarse fraction is larger than 2.36mm)	Coarse: 20 – 63mm Medium: 6 – 20mm Fine: 2.36 – 6mm	GW	Well graded gravels, gravel-sand mixtures, little or no fines.
			GP	Poorly graded gravels and gravel-sand mixtures, little or no fines, uniform gravels.
			GM	Silty gravels, gravel-sand-silt mixtures.
			GC	Clayey gravels, gravel-sand-clay mixtures.
	SANDS (more than half of coarse fraction is smaller than 2.36mm)	Coarse: 0.6 – 2.36mm Medium: 0.2 – 0.6mm Fine: 0.075 – 0.2mm	SW	Well graded sands, gravelly sands, little or no fines.
			SP	Poorly graded sands and gravelly sands; little or no fines, uniform sands.
			SM	Silty sands, sand-silt mixtures.
			SC	Clayey sands, sand-clay mixtures.
FINE GRAINED SOILS (more than half of material is smaller than 0.075mm)	SILTS & CLAYS (liquid limit <50%)		ML	Inorganic silts and very fine sands, silty/clayey fine sands or clayey silts with low plasticity.
			CL and CI	Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays.
			OL	Organic silts and organic silty clays of low plasticity.
	SILTS & CLAYS (liquid limit >50%)		MH	Inorganic silts, micaceous or diatomaceous fine sandy or silty soils.
			CH	Inorganic clays of high plasticity.
			OH	Organic clays of medium to high plasticity, organic silts.
	HIGHLY ORGANIC SOILS		Pt	Peat and other highly organic soils.

PLASTICITY CHART FOR CLASSIFICATION OF FINE GRAINED SOILS



(Reference: Australian Standard AS1726-1993 Geotechnical site investigations)

DESCRIPTIVE TERMS FOR MATERIAL PROPORTIONS

Coarse Grained Soils		Fine Grained Soils	
% Fines	Modifier	% Coarse	Modifier
<5	Omit, or use 'trace'	<15	Omit, or use trace.
5 – 12	Describe as 'with clay/silt' as applicable.	15 – 30	Describe as 'with sand/gravel' as applicable.
>12	Prefix soil as 'silty/clayey' as applicable	>30	Prefix soil as 'sandy/gravelly' as applicable.

STRENGTH TERMS – COHESIVE SOILS

Strength Term	Undrained Shear Strength	Field Guide to Strength
Very soft	<12kPa	Exudes between the fingers when squeezed in hand.
Soft	12 – 25kPa	Can be moulded by light finger pressure.
Firm	25 – 50kPa	Can be moulded by strong finger pressure.
Stiff	50 – 100kPa	Cannot be moulded by fingers, can be indented by thumb.
Very stiff	100 – 200kPa	Can be indented by thumb nail.
Hard	>200kPa	Can be indented with difficulty by thumb nail.

DENSITY TERMS – NON COHESIVE SOILS

Density Term	Density Index	SPT "N"	CPT Cone Resistance
Very loose	<15%	0 – 5	0 – 2MPa
Loose	15 – 35%	5 – 10	2 – 5MPa
Medium dense	35 – 65%	10 – 30	5 – 15MPa
Dense	65 – 85%	30 – 50	15 – 25MPa
Very dense	>85%	>50	>25MPa

COLOUR

The colour of a soil will generally be described in a 'moist' condition using simple colour terms (e.g. black, grey, red, brown etc.) modified as necessary by "pale", "dark", "light" or "mottled". Borderline colours will be described as a combination of colours (e.g. grey-brown).

EXAMPLE

e.g. CLAYEY SAND (SC) – medium dense, grey-brown, fine to medium grained with silt.

Indicates a medium dense, grey-brown, fine to medium grained clayey sand with silt.

Notes on Description and Classification of Rock

The methods of description and classification of rock used in this report are generally based on Australian Standard AS1726-1993 *Geotechnical site investigations*.

Rock description is based on an assessment of disturbed samples, as recovered from bores and excavations, or from undisturbed materials as seen in excavations and exposures, or in core samples. Descriptions given on report sheets are an interpretation of the conditions encountered at the time of investigation.

Notes outlining the method and terminology adopted for the description of rock defects are given below, however, detailed information on defects can generally only be determined where rock core is taken, or excavations or exposures allow detailed observation and measurement.

Rocks are generally described using the following sequence of terms. In certain instances not all of the terms will be included in the rock description.

ROCK TYPE (WEATHERING SYMBOL), strength, colour, grain size, defect frequency

Information on the definition of descriptive and classification terms follows.

ROCK TYPE

In general, simple rock names are used rather than precise geological classifications.

ROCK MATERIALS WEATHERING CLASSIFICATION

Term	Weathering Symbol	Definition
Residual soil	RS	Soil developed from extremely weathered rock; the mass structure and substance fabrics are no longer evident; there is a large change in volume but the soil has not been significantly transported.
Extremely weathered	XW	Rock is weathered to such an extent that it has 'soil' properties, i.e. it either disintegrates or can be remoulded in water.
Distinctly weathered *	DW	Rock strength usually changed by weathering. The rock may be highly discoloured, usually by ironstaining. Porosity may be increased by leaching, or may be decreased due to deposition of weathering products in pores.
- Highly weathered	HW	Rock substance affected by weathering to the extent that limonite staining or bleaching affects the whole of the rock substance and other signs of chemical or physical decomposition are evident. Porosity and strength may be increased or decreased compared to the fresh rock, usually as a result of iron leaching or deposition. The colour and strength of the original fresh rock substance is no longer recognisable.
- Moderately weathered	MW	Rock substance affected by weathering to the extent that staining extends throughout the whole of the rock substance and the original colour of the fresh rock may be no longer recognisable.
Slightly weathered	SW	Rock is slightly discoloured but shows little or no change of strength from fresh rock.
Fresh	FR	Rock shows no sign of decomposition or staining.

** Subdivision of this weathering grade into highly and moderately may be used where applicable.*

STRENGTH OF ROCK MATERIAL

Term	Symbol	Point Load Index I_s (50)	Field Guide To Strength
Extremely low	EL	<0.03MPa	Easily remoulded by hand to a material with soil properties.
Very low	VL	0.03 – 0.1MPa	Material crumbles under firm blows with sharp end of pick; can be peeled with knife; too hard to cut a triaxial sample by hand. Pieces up to 30mm thick can be broken by finger pressure.
Low	L	0.1 – 0.3MPa	Easily scored with a knife; indentations 1mm to 3mm show in the specimen with firm blows of the pick point; has dull sound under hammer. A piece of core 150mm long 50mm diameter may be broken by hand. Sharp edges of core may be friable and break during handling.
Medium	M	0.3 – 1.0MPa	Readily scored with a knife; a piece of core 150mm long by 50mm diameter can be broken by hand with difficulty.
High	H	1.0 – 3.0MPa	A piece of core 150mm long by 50mm diameter cannot be broken by hand but can be broken by a pick with a single firm blow; rock rings under hammer.
Very high	VH	3.0 – 10.0MPa	Hand specimen breaks with pick after more than one blow; rock rings under hammer.
Extremely high	EH	>10MPa	Specimen requires many blows with geological pick to break through intact material; rock rings under hammer.

Notes:

1. *These terms refer to the strength of the rock material and not to the strength of the rock mass which may be considerably weaker due to the effect of rock defects.*
2. *The field guide visual assessment for rock strength may be used for preliminary assessment or when point load testing is not available.*
3. *Anisotropy of rock may affect the field assessment of strength.*

COLOUR

The colour of a rock will generally be described in a 'moist' condition using simple colour terms (e.g. black, grey, red, brown, etc) modified as necessary by 'pale', 'dark', 'light' or 'mottled'. Borderline colours will be described as a combination of colours (e.g. grey-brown).

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GRAIN SIZE

Descriptive Term	Particle Size Range
Coarse grained	0.6 – 2.0mm
Medium grained	0.2 – 0.6mm
Fine grained	0.06 – 0.2mm

DEFECT FREQUENCY

Where appropriate, a defect frequency may be recorded as part of the rock description and will be expressed as the number of natural (or interpreted natural) defects present in an equivalent one metre length of core; by use of the following defect frequency descriptive terms; or both. The descriptive terms refer to the spacing of all types of natural defects along which the rock is discontinuous and include, bedding plane partings, joints and other rock defects, but excludes known artificial fractures such as drilling breaks.

Defect Frequency	Description
Fragmented	Rock core is comprised primarily of fragments of length less than 20mm, and mostly of width less than the core diameter.
Highly Fractured	Core lengths are generally less than 20mm to 40mm with occasional fragments.
Fractured	Core lengths are mainly 30mm to 100mm with occasional shorter and longer sections.
Fractured to Slightly Fractured	Core lengths are mainly 100mm to 300mm with occasional shorter to longer sections.
Slightly Fractured	Core lengths are generally 300mm to 1,000mm with occasional longer sections and occasional sections of 100mm to 300mm.
Unbroken	The core does not contain any fractures.

EXAMPLE

e.g. SANDSTONE (XW) – low strength, pale brown, fine to coarse grained, slightly fractured.

ROCK DEFECT LOGGING

Defects are discontinuities in the rock mass and include joints, sheared zones, cleavages and bedding partings. The ability to observe and log defects will depend on the investigation methodology. Defects logged in core are described using the abbreviations noted in the following tables.

The *depth* noted in the description is measured in metres from the ground surface, the *defect angle* is measured in degrees from horizontal, and the defect *thickness* is measured normal to the plane of the defect and is in millimetres (unless otherwise noted).

Defects are generally described using the following sequence of terms:

Depth, Defect Type, Defect Angle (dip), Surface Roughness, Infill, Thickness

DEFECT TYPE

B	– Bedding
J	– Joint
S	– Shear Zone
C	– Crushed Zone

SURFACE ROUGHNESS

i	- rough or irregular, stepped
ii	- smooth, stepped
iii	- slickensided, stepped
iv	- rough or irregular, undulating
v	- smooth, undulating
vi	- slickensided, undulating
vii	- rough or irregular, planar
viii	- smooth planar
ix	- slickensided, planar

INFILL

Infill refers to secondary minerals or other materials formed on the surface of the defect and some common descriptions are given in the following table together with their abbreviations.

Ls	- limonite staining
Fe	- iron staining
Cl	- clay
Mn	- manganese staining
Qtz	- quartz
Ca	- calcite
Clean	- no visible infill

EXAMPLE

3.59m, J, 90, vii, Ls, 1mm

indicates a joint at 3.59m depth that is at 90° to horizontal (i.e. vertical), is rough or irregular and planar, limonite stained and 1mm thick.

APPENDIX B

LABORATORY TEST RESULT REPORT SHEETS

Material Test Report

Report Number:	**RG23-1178A-1**
Issue Number:	1
Date Issued:	31/08/2023
Client:	Michael Swann 4 Vanderspek Place, Frenchville Qld 4701
Contact:	Michael Swann
Project Number:	RG23-1178A
Project Name:	Proposed Subdivision and Dwelling
Project Location:	4 Vanderspek Place, Frenchville
Work Request:	21511
Sample Number:	R23-21511A
Date Sampled:	31/07/2023
Dates Tested:	31/07/2023 - 14/08/2023
Sampling Method:	AS 1289.1.2.1 6.5.4 - Machine excavated pit or trench
Preparation Method:	In accordance with the test method
Sample Location:	**Test Pit 1 , Depth: 0.6 - 0.8**
Material Source:	Insitu

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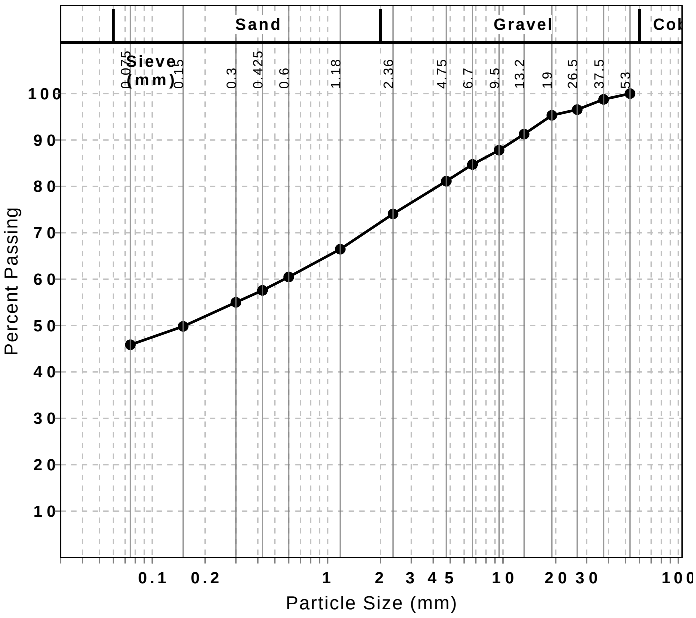
Laboratory Manager
NATA Accredited Laboratory Number: 19665

Moisture Content (AS 1289 2.1.1)		Min	Max
Moisture Content (%)	20.7		

Particle Size Distribution (AS1289 3.6.1)			
Sieve	Passed %	Passing Limits	
53 mm	**100**		
37.5 mm	**99**		
26.5 mm	**97**		
19 mm	**95**		
13.2 mm	**91**		
9.5 mm	**88**		
6.7 mm	**85**		
4.75 mm	**81**		
2.36 mm	**74**		
1.18 mm	**66**		
0.6 mm	**60**		
0.425 mm	**58**		
0.3 mm	**55**		
0.15 mm	**50**		
0.075 mm	**46**		

Particle Size Distribution



Material Test Report

Report Number:	**RG23-1178A-1**
Issue Number:	1
Date Issued:	31/08/2023
Client:	Michael Swann 4 Vanderspek Place, Frenchville Qld 4701
Contact:	Michael Swann
Project Number:	RG23-1178A
Project Name:	Proposed Subdivision and Dwelling
Project Location:	4 Vanderspek Place, Frenchville
Work Request:	21511
Sample Number:	R23-21511A
Date Sampled:	31/07/2023
Dates Tested:	31/07/2023 - 11/08/2023
Sampling Method:	AS 1289.1.2.1 6.5.4 - Machine excavated pit or trench
Preparation Method:	In accordance with the test method
Sample Location:	**Test Pit 1 , Depth: 0.6 - 0.8**
Material Source:	Insitu

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Atterberg Limit (AS1289 3.1.2 & 3.2.1 & 3.3.1 & Q252)		Min	Max
Sample History	Oven Dried		
Preparation Method	Dry Sieve		
Passing 0.425 (%)	58		
Liquid Limit (%)	40		
Plastic Limit (%)	18		
Plasticity Index (%)	**22**		
Weighted Plasticity Index (%)	1267		

Linear Shrinkage (AS1289 3.4.1)		Min	Max
Moisture Condition Determined By	AS 1289.3.1.2		
Linear Shrinkage (%)	**10.5**		
Cracking Crumbling Curling	Curling		

Material Test Report

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Approved Signatory: Travis Driver
Laboratory Manager

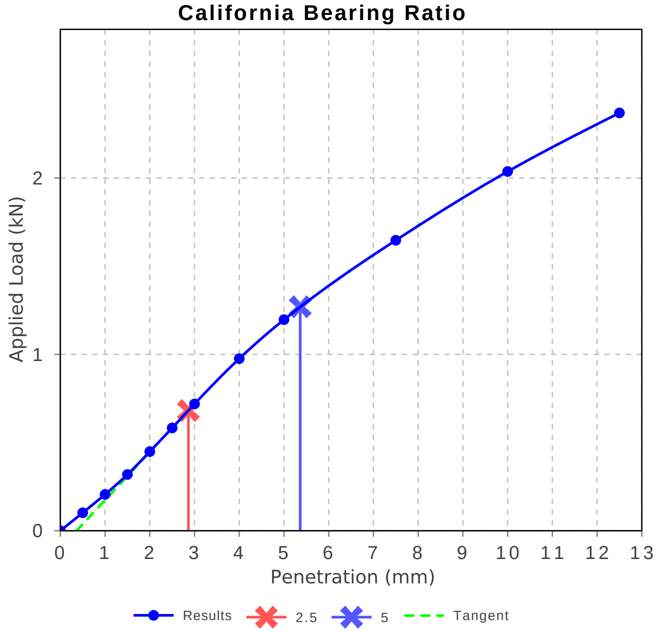
NATA Accredited Laboratory Number: 19665

Report Number:	**RG23-1178A-1**
Issue Number:	1
Date Issued:	31/08/2023
Client:	Michael Swann 4 Vanderspek Place, Frenchville Qld 4701
Contact:	Michael Swann
Project Number:	RG23-1178A
Project Name:	Proposed Subdivision and Dwelling
Project Location:	4 Vanderspek Place, Frenchville
Work Request:	21511
Sample Number:	R23-21511B
Date Sampled:	31/07/2023
Dates Tested:	31/07/2023 - 07/08/2023
Sampling Method:	AS 1289.1.2.1 6.5.4 - Machine excavated pit or trench
Preparation Method:	In accordance with the test method
Sample Location:	**Test Pit 2 , Depth: 0.2 - 0.5**
Material Source:	Insitu

Moisture Content (AS 1289 2.1.1)		Min	Max
Moisture Content (%)	16.5		

California Bearing Ratio (AS 1289 6.1.1 & 2.1.1)		Min	Max
CBR taken at	5 mm		
CBR %	**6**		
Method of Compactive Effort	Standard		
Method used to Determine MDD	AS 1289 5.1.1 & 2.1.1		
Method used to Determine Plasticity	Visual / Tactile		
Maximum Dry Density (t/m^3)	1.65		
Optimum Moisture Content (%)	18.5		
Laboratory Density Ratio (%)	97.5		
Laboratory Moisture Ratio (%)	100.5		
Dry Density after Soaking (t/m^3)	1.61		
Field Moisture Content (%)	15.0		
Moisture Content at Placement (%)	18.7		
Moisture Content Top 30mm (%)	21.5		
Moisture Content Rest of Sample (%)	20.2		
Mass Surcharge (kg)	4.5		
Soaking Period (days)	4		
Curing Hours	72.6		
Swell (%)	0.5		
Oversize Material (mm)	19		
Oversize Material Included	Excluded		
Oversize Material (%)	20.4		

California Bearing Ratio



Material Test Report

Report Number:	**RG23-1178A-1**
Issue Number:	1
Date Issued:	31/08/2023
Client:	Michael Swann 4 Vanderspek Place, Frenchville Qld 4701
Contact:	Michael Swann
Project Number:	RG23-1178A
Project Name:	Proposed Subdivision and Dwelling
Project Location:	4 Vanderspek Place, Frenchville
Work Request:	21511
Sample Number:	R23-21511C
Date Sampled:	31/07/2023
Dates Tested:	31/07/2023 - 03/08/2023
Sampling Method:	AS 1289.1.2.1 6.5.4 - Machine excavated pit or trench
Preparation Method:	In accordance with the test method
Sample Location:	**Test Pit 2 , Depth: 0.5 - 0.6**
Material Source:	Insitu

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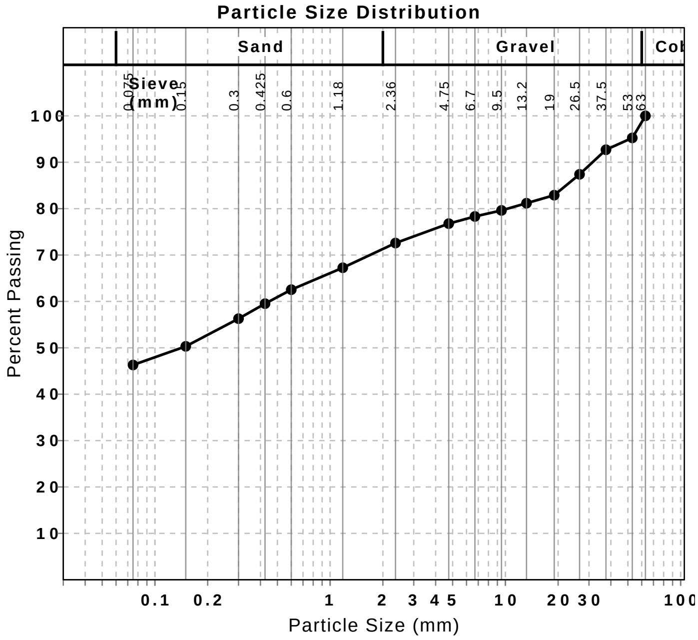
Moisture Content (AS 1289 2.1.1)		Min	Max
Moisture Content (%)	14.2		

Particle Size Distribution (AS1289 3.6.1)			
Sieve	Passed %	Passing Limits	
63 mm	**100**		
53 mm	**95**		
37.5 mm	**93**		
26.5 mm	**87**		
19 mm	**83**		
13.2 mm	**81**		
9.5 mm	**80**		
6.7 mm	**78**		
4.75 mm	**77**		
2.36 mm	**73**		
1.18 mm	**67**		
0.6 mm	**63**		
0.425 mm	**60**		
0.3 mm	**56**		
0.15 mm	**50**		
0.075 mm	**46**		



Material Test Report

Report Number:	**RG23-1178A-1**
Issue Number:	1
Date Issued:	31/08/2023
Client:	Michael Swann 4 Vanderspek Place, Frenchville Qld 4701
Contact:	Michael Swann
Project Number:	RG23-1178A
Project Name:	Proposed Subdivision and Dwelling
Project Location:	4 Vanderspek Place, Frenchville
Work Request:	21511
Sample Number:	R23-21511C
Date Sampled:	31/07/2023
Dates Tested:	31/07/2023 - 08/08/2023
Sampling Method:	AS 1289.1.2.1 6.5.4 - Machine excavated pit or trench
Preparation Method:	In accordance with the test method
Sample Location:	**Test Pit 2 , Depth: 0.5 - 0.6**
Material Source:	Insitu

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Laboratory Manager

NATA Accredited Laboratory Number: 19665

Atterberg Limit (AS1289 3.1.2 & 3.2.1 & 3.3.1 & Q252)		Min	Max
Sample History	Oven Dried		
Preparation Method	Dry Sieve		
Passing 0.425 (%)	60		
Liquid Limit (%)	31		
Plastic Limit (%)	18		
Plasticity Index (%)	**13**		
Weighted Plasticity Index (%)	774		

Linear Shrinkage (AS1289 3.4.1)		Min	Max
Moisture Condition Determined By	AS 1289.3.1.2		
Linear Shrinkage (%)	**7.5**		
Cracking Crumbling Curling	Cracking		

Emerson Class Number of a Soil (AS 1289 3.8.1)		Min	Max
Emerson Class	**4 ***		
Soil Description	Gravelly Sandy Clay		
Nature of Water	Distilled		
Temperature of Water (^{o}C)	20		
* Mineral Present	Carbonate and Gypsum		

Material Test Report

Report Number: **RG23-1178A-1**
Issue Number: 1
Date Issued: 31/08/2023
Client: Michael Swann
4 Vanderspek Place, Frenchville Qld 4701
Contact: Michael Swann
Project Number: RG23-1178A
Project Name: Proposed Subdivision and Dwelling
Project Location: 4 Vanderspek Place, Frenchville
Work Request: 21511
Sample Number: R23-21511C
Date Sampled: 31/07/2023
Dates Tested: 31/07/2023 - 04/08/2023
Sampling Method: AS 1289.1.2.1 6.5.4 - Machine excavated pit or trench
Preparation Method: In accordance with the test method
Sample Location: **Test Pit 2 , Depth: 0.5 - 0.6**
Material Source: Insitu

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pH Value of Soil (AS 1289 4.3.1)		Min	Max
Air Temp (^{o}C)	20		
Distilled Water pH	7.19		
Depth	0.5 - 0.6		
Moisture Condition	Natural		
pH	7.2		
Electrical Conductivity (µS/cm)	5.36		
For Conductivity - 1 dS/m = 1 mS/cm = 1000 µS/cm			
Electrical Conductivity not covered by accreditation.			

Material Test Report

Report Number:	**RG23-1178A-1**
Issue Number:	1
Date Issued:	31/08/2023
Client:	Michael Swann 4 Vanderspek Place, Frenchville Qld 4701
Contact:	Michael Swann
Project Number:	RG23-1178A
Project Name:	Proposed Subdivision and Dwelling
Project Location:	4 Vanderspek Place, Frenchville
Work Request:	21511
Sample Number:	R23-21511D
Date Sampled:	31/07/2023
Dates Tested:	31/07/2023 - 03/08/2023
Sampling Method:	AS 1289.1.2.1 6.5.4 - Machine excavated pit or trench
Preparation Method:	In accordance with the test method
Sample Location:	**Test Pit 3 , Depth: 0.5 - 0.6**
Material Source:	Insitu

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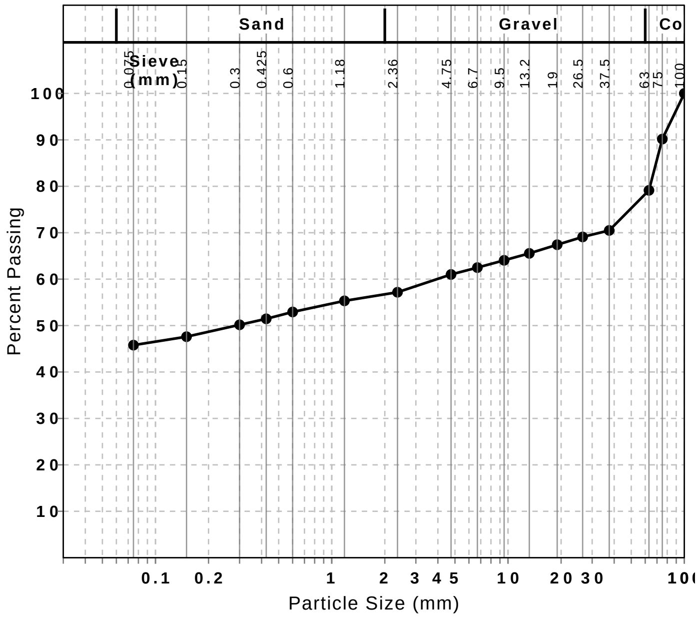
Moisture Content (AS 1289 2.1.1)		Min	Max
Moisture Content (%)	8.2		

Particle Size Distribution (AS1289 3.6.1)			
Sieve	Passed %	Passing Limits	
100 mm	**100**		
75 mm	**90**		
63 mm	**79**		
37.5 mm	**70**		
26.5 mm	**69**		
19 mm	**67**		
13.2 mm	**66**		
9.5 mm	**64**		
6.7 mm	**62**		
4.75 mm	**61**		
2.36 mm	**57**		
1.18 mm	**55**		
0.6 mm	**53**		
0.425 mm	**51**		
0.3 mm	**50**		
0.15 mm	**48**		
0.075 mm	**46**		

Particle Size Distribution



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POINT LOAD STRENGTH INDEX TEST REPORT

Test Method: AS4133.4.1 AS1726

Client:	Reel Planning Pty Ltd	**Report No.:**	RG23-1178A_1A
Project:	Proposed Subsivision	**Tested by:**	SC
Location:	4 Vanderspek Place, Frenchville	**Date:**	8/15/2023
		Checked by:	NB
Project No:	RG23-1178A	**Date:**	8/16/2023

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0	Depth (m)	Test Type	Sample Description	Point Load Strength $[I_s(50)]$ (MPa)	Rock Strength Category*
TP1	1.6-1.9	Irregular	Sandstone	0.7	Medium
	1.6-1.9	Irregular		5.5	Very high
	1.6-1.9	Irregular		6.1	Very high
TP3	0.8-1.0	Irregular	Sandstone	3.0	Very high
	0.8-1.0	Irregular		2.4	High
	0.8-1.0	Irregular		3.3	Very high

*Australian Standard AS1726-1993 *Geotechnical site investigation*

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DIRECT SHEAR STRENGTH OF A SOIL (SHEAR BOX) TEST REPORT

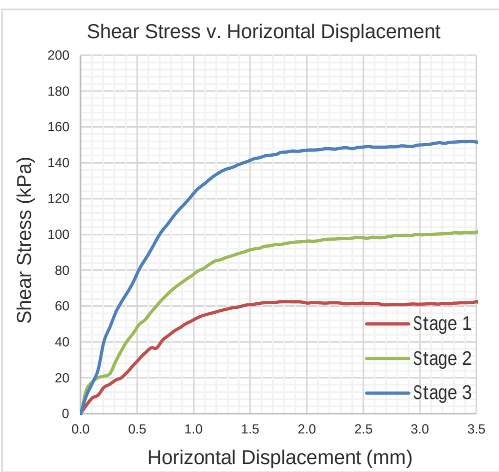
Test Procedure: AS1289.6.2.2 AS1289.2.1.1

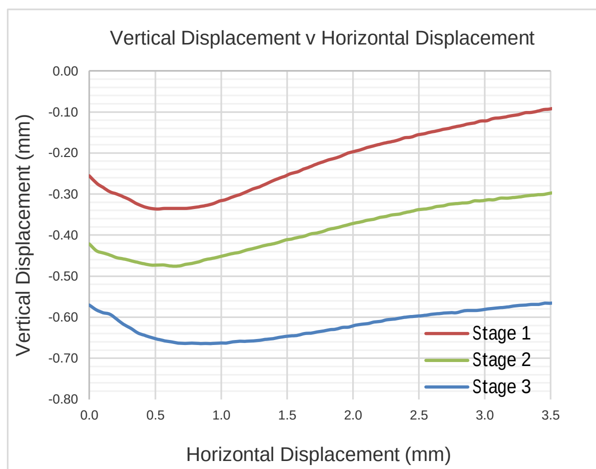
Client:	Michael Swann	**Report No.:**	RG23-1178A_DS_R23-21511A
Project:	Proposed Subdivision and Dwelling	**Tested by:**	AC
Location:	4 Vanderspek Place,	**Date:**	8/10/2023
	Frenchville	**Checked by:**	NB
Project No:	RG23-1178A	**Date:**	8/14/2023

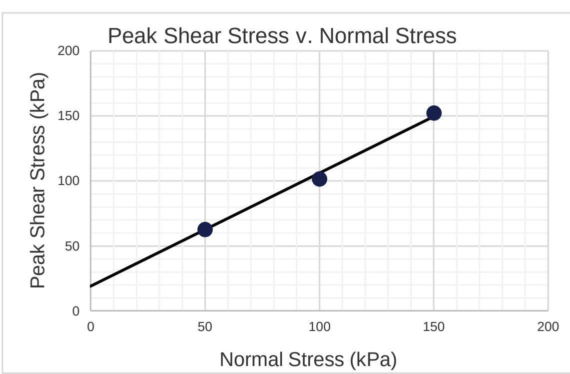
THIS DOCUMENT SHALL NOT BE REPRODUCED EXCEPT IN FULL

Stage	Moisture Content (%)		Initial Dry Density (t/m^3)	Shearing Rate (mm/min)	Normal Stress (kPa)	Peak Shear Stress (kPa)
	Initial	Final				
1	19.6	20.7	1.72	0.005	50.0	62.5
2	19.7	20.2	1.71	0.005	100.0	101.5
3	19.4	20.1	1.72	0.005	150.0	152.1

Type of Specimen	Remoulded	**Size of Shear Box (mm)**	45
Conditions	Submerged	**Sample Shape**	Circle







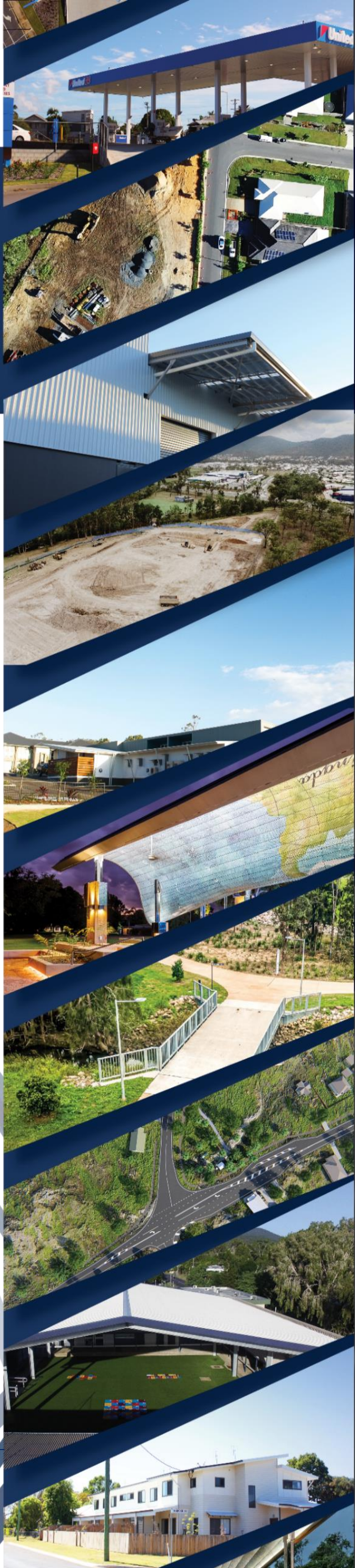
Sample No.:	R23-21511A
Sampling Method:	AS1289.1.2.1
	Clause 6.5.4
Date Sampled:	7/31/2023
Test Pit:	1
Depth (m):	0.6 -0.8
Sample Description:	Gravely Sandy Clay
Apparent Cohesion (kPa)	**Friction Angle (degrees)**
19	**41**

Values for cohesion and friction angle are interpetations only

Comments

Authorised Signatory

Nick Bloxom **Date:** 14/08/23



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ROCKHAMPTON REGIONAL COUNCIL

AMENDED PLANS APPROVED

26 July 2024

DATE

These plans are approved subject to the current conditions of approval associated with

Development Permit No.: D/1-2024

Dated: 19 April 2024

Mr Michael Swann

Site Based Stormwater Management Plan

1 into 2 Lot Residential Development

4 Vanderspek Place, FRENCHVILLE QLD

9 June 2023

FP/001.CE23062 Rev A

Contract No. CE23062

A/ PO BOX 3203 RED HILL ROCKHAMPTON Q 4701

MOLONEY & SONS

ABN: 39 133 970 689

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2023

DOCUMENT CONTROL:

Issue	*Date*	*Issue Description*	*Author*	*Checked*	*Approved*
A	*9/06/23*	*Development Approval*	*LM*	*LM/PAJ*	*PAJ* RPEQ: 21524

TABLE OF CONTENTS

RPEQ: 21524

EXECUTIVE SUMMARY

Moloney & Sons Engineering (MSE) have been engaged by Mr Michael Swann to prepare a Site Based Stormwater Management Plan (SBSWMP) suitable for submission to Rockhampton Regional Council (RRC) in support of a Reconfiguration of Lot (ROL) development application (i.e.1 into 2 lots) for Lot 4 SP 247716, known as 4 Vanderspek Place FRENCHVILLE QLD.

The stormwater quantity objective was to demonstrate that there is a no nett increase in peak discharges from the subject site. This objective included storm events up to and including the 1% AEP storm event. The purpose is to ensure that the existing infrastructure and/or downstream properties are not adversely affected. The above-mentioned objectives are achieved through the use of detention measures.

As a result of the site proposed developed condition no on-site detention will be required in order to mitigate any post-developed stormwater runoff from the site and all roofwater runoff will be directed to rainwater tanks and/or to the existing underground roofwater network beneath the access easement.

All relevant standards and guidelines are addressed including criteria from Rockhampton Regional Council, Queensland Urban Development Manual (QUDM) and the State Planning Policy.

RPEQ

1. INTRODUCTION AND BACKGROUND

Moloney & Sons Engineering (MSE) have been engaged by Mr Michael Swann to prepare the following Site Based Stormwater Management Plan (SBSWMP) in support of a Development Application for the proposed reconfiguration of 1 into 2 lots, at 4 Vanderspek Place FRENCHVILLE QLD.

The intent of the SBSWMP is to provide guidelines and recommendations to be incorporated into future Operational Works detailed design documentation, to minimise the impact of the development on the surrounding environment, infrastructure, and property owners.

Figure 1 Site Locality

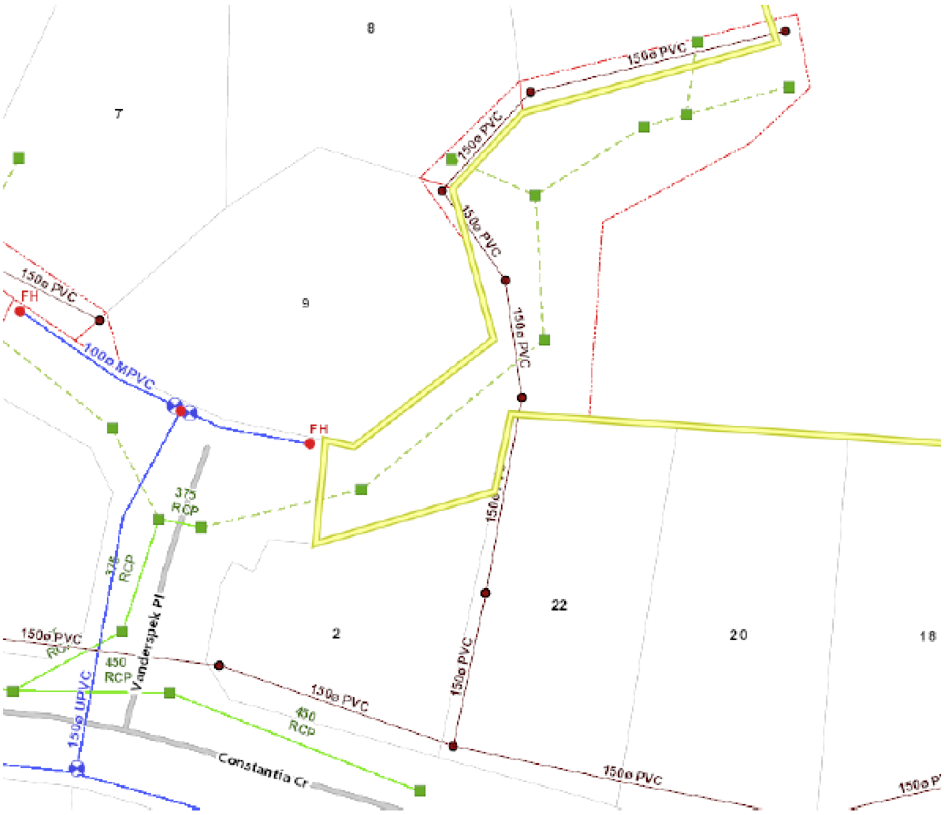
1.1. Site Characteristics

The existing site, known as 4 Vanderspek Place FRENCHVILLE (Lot 4 SP 247716), comprises a total site area of 16,600m^2 (Lot 4 SP 247716). The topography of the proposed residential development site is based on survey data obtained from the QLD Governments ELVIS Online Mapping, utilising a 1m Digital Elevation Model.

The existing site pertains to a predominantly heavily treed and grassed site with little clearing with assumed Bushland (healthy, unburnt) Group C (Loamy Clay) soils with a low to moderate infiltration capacity. Usually consists of moderately fine clay loams, or loamy clays, or more porous soils that are impeded by poor surface conditions, shallow depth or a low porosity subsoil horizon.

The site forms a single contributing catchment, where-by all overland runoff falls south towards the rear of the allotments fronting Constantia Cresent, performing at the present as the current Lawful Point of Discharge (LPoD). All site stormwater runoff is currently conveyed via overland flow. It is noted there is an existing underground roofwater network beneath the existing concrete driveway.

Please refer to Moloney and Sons sketch CE23062-101-SK, included as APPENDIX A for the pre-developed site



characteristics.

Figure 2 RRC Existing Infrastructure

2. STORMWATER QUANTITY ASSESSMENT

The aim of the stormwater quantity assessment is to ensure that the development shall impose no adverse effects on downstream properties or receiving water bodies and that the conveyance of flows will be in a safe manner with minimal risk of human endangerment as well as the following objectives:

- Address the need for stormwater quantity control measures.
- Ensure there is no nett increase in peak discharges from the subject site for events up to and including the 1% AEP event; and
- Ensure proposed quantity control measures detain and convey flows in accordance with QUDM minimum freeboard recommendations.

2.1. Proposed Development and Associated Issues

One of the implications of an increase in impervious area is that the total volume and flow rate of stormwater runoff from the catchment will increase as a result of typical increase in coefficient factors of discharge. It is essential that these increases are mitigated such that post-developed peak flows do not exceed those for the pre-developed case.

2.2. Stormwater Mitigation

In the event of increased stormwater runoff within the site, said flows will be directed to a detention system before being discharged via staged outlets to the respective Lawful Point of Discharge (LPoD). Stormwater flow generated from the new development will be discharged to the LPoDs at flow rates equal to or below predevelopment rates.

Major & Minor Systems have been classified and adopted in accordance with CMDG Stormwater Drainage Design Guidelines under "Urban Residential" Table D05-04.1 as 1% AEP & 50% AEP (2yr ARI) respectively.

2.3. External Catchments

The site has no contributing external catchments, all stormwater runoff falls away from the subject site.

2.4. Lawful Point of Discharge

It is proposed that the existing LPoD will be maintained as per the Pre-Developed site conditions, which the exception of all roofwater of the proposed new dwelling being directed to the existing roofwater network within the current access easement, as shown in Figure 2.

Please refer to Moloney and Sons sketch, CE23062-102-SK, contained in APPENDIX A for the post-developed site characteristics.

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2.5. ICM1/XP-STORM Rainfall Parameters

An ICM One/XP-STORM Hydraulic and Runoff model was created to analyse the pre-development and post-development scenarios. The models include a typical 1D node-link connectivity identifying the catchments and hydraulic parameters.

The IFD data for the Blackwater Region has been produced by the Bureau of Meteorology and was obtained from the following site:

(http://www.bom.gov.au/water/designRainfalls/revised-ifd/?design=ifds&sdmin=true&sdhr=true&sdday=true&nsd%5B%5D=16&nsdunit%5B%5D=m&coordinate_type=dd&latitude=-23.341028&longitude=150.5529&user_label=&values=intensities&update=)

The location of the site is: Latitude -23.341028 and Longitude 150.5529.

This data was used for the hydrologic analysis for the determination of the ICM One/XP-STORM.

In accordance with the AR&R & TMR Hydrologic & Hydraulic Modelling Guidelines, for each AEP event the full range of storm durations with associated temporal pattern ensembles were assessed in order to determine the critical durations, flow rates and temporal patterns.

2.6. Pre-Development Hydrology

The subject site maintains an average landfall of 31%, as previously mentioned the sites topographic landfalls south towards the rear boundaries of the properties fronting Constantia Crescent, refer below table for key catchment parameters.

The pre-development catchment plan (CE23062-101-SK) for the subject site is demonstrated in Appendix A of this report.

The hydrology of the pre-developed catchments has been assessed in accordance with QUDM Section 4.0 using the rational method. From QUDM Section 4.6.6, the theoretical calculated time of concentrations and peak discharge for storm events ranging from the 50% to 1% AEP has been calculated.

The Coefficient of discharge (C_{10}) values were derived from QUDM Table 4.05.3 (a) and Table 4.05.3 (b).

Table 1 Pre-Development Catchment Parameters

Catchment ID	Area (ha)	Avg Slope (%)	Fraction Imp (f_i)	Co-efficient of Runoff (C_{10})	Time of Concentration (Tc)
A	1.189	31	0.09	0.44	16 min

A/ Po Box 3203 Red Hill Rockhampton QLD 4701
Moloney & Sons
ABN: 39 133 970 689

2.7. Post-Development Hydrology

The proposed development will in effect maintain the pre-development scenario catchment extents. The only noted change is a slight increase (4%) in impervious area, however due to location of the proposed dwelling and the catchments Tc will remain @ 16min as there will be no increase or change to the post-developed runoff coefficient.

The post-development catchment plan (CE23062-102-SK) is attached within Appendix A for further information.

A time of concentration of 16 minutes as per Friends equation for overland flow paths was obtained for the post-development time of concentration for each catchment.

Table 2 Post-Development Catchment Parameters

Catchment ID	Area (ha)	Avg Slope (%)	Fraction Imp (f_i)	Co-efficient of Runoff (C_{10})	Time of Concentration (Tc)
A1	1.189	31	0.13	0.44	16 min

2.8. 1D Modelling Assumptions & Methodology

The following modelling assumptions were used to create the ICM One/XP-STORM Models

- Two (2) separate scenarios were generated, which were:
  - Pre-Development (which included all points of discharge); and
  - Post-Development (which included all points of discharge).
- Each model included runoff nodes for each contributing sub-catchment
- The sub-catchment areas were split into Urban Residential Land Uses with absolute values adopted for percentages impervious as specified in the above Tables 1 & 2.
- Infiltration uniform losses were applied to the pervious areas of the sub-catchments, SCS Soil Type C.
- The models were run at various durations for a constant ARI to determine the critical storm event.

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2.9. Design Flow Verification

As the pre & post developed scenarios confirm the same Tc's & runoff coefficients with the only comparable difference being a 4% increase in impervious area, therefore the peak discharge values obtained for the 1% through to the 50% AEP storm events using the Rational Method and the ICM1/XP-STORM model will demonstrate zero to negligible increases in peak discharges from subject contributing catchment.

Furthermore, it is proposed that all roofwater runoff from the newly proposed dwelling will be directed to the existing underground roofwater network beneath the existing access easement, further mitigating any impacts on neighbouring and surrounding downstream properties.

3. CONCLUSION AND RECOMMENDATIONS

As outlined in Section 2 of this report, the proposed development will not introduce any nett increase in post-developed stormwater runoff from the site and no detention is required. It should be noted the requirement for all roofwater to be directed to a rainwater tank and/or to the existing underground roofwater network beneath the existing access easement.

Therefore, in accordance with the conditions and recommendations outlined in this report and referenced documents (i.e. Engineering Drawings), Moloney and Sons are of the opinion that the proposed development will not create any actionable external to the site or any net increase in flows or nuisance to surrounding properties.

It is our opinion that if the abovementioned recommendations are implemented, the proposed development will comply with the intent of Rockhampton Regional Council, Queensland Urban Drainage Manual & AR&R requirements for stormwater quantity management.

APPENDIX A – Engineering Drawings

LEGEND

EXISTING SURVEY

- Existing Lot Boundary
- Existing Lawful Point of Discharge Line
- Existing to be Demolished
- Existing Edge of Bitumen
- Existing Fence Line
- Existing Overhead Electrical
- Existing Electrical Line (DBYD Location Only)
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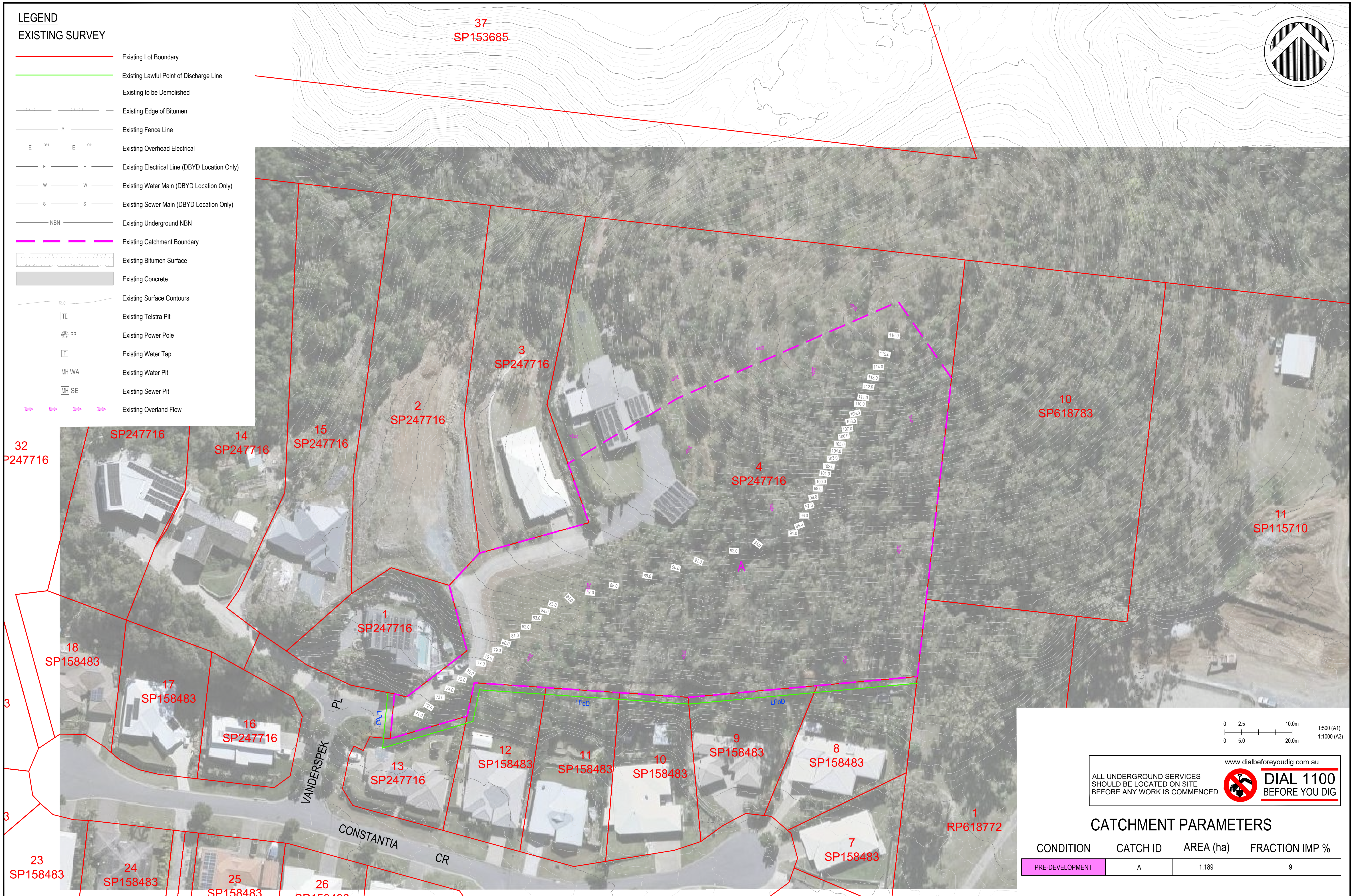

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0 5.0 20.0m 1:1000 (A3)

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CATCHMENT PARAMETERS

CONDITION	CATCH ID	AREA (ha)	FRACTION IMP %
PRE-DEVELOPMENT	A	1.189	9

ISSUE	FIRST CALCS	DRAWN	DATE	AMENDMENT DETAILS
	LM	AB	06/23	
A	LM	AB	09/06/23	PRELIMINARY ISSUED FOR DEVELOPMENT APPROVAL

DESIGN CHECK
DRAWN CHECK

DATUM

PROJECT No. CE23062

ISSUED FOR APPROVAL

APPROVED
FOR & ON BEHALF OF MOLONEY & SONS ENGINEERING

CLIENT: MR MICHAEL SWANN

PROJECT: 4 VANDERSPEK PL
FRENCHVILLE QLD
1 INTO 2 SUBDIVISION

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ENGINEERING
EXCELLENCE - INTEGRITY - INNOVATION
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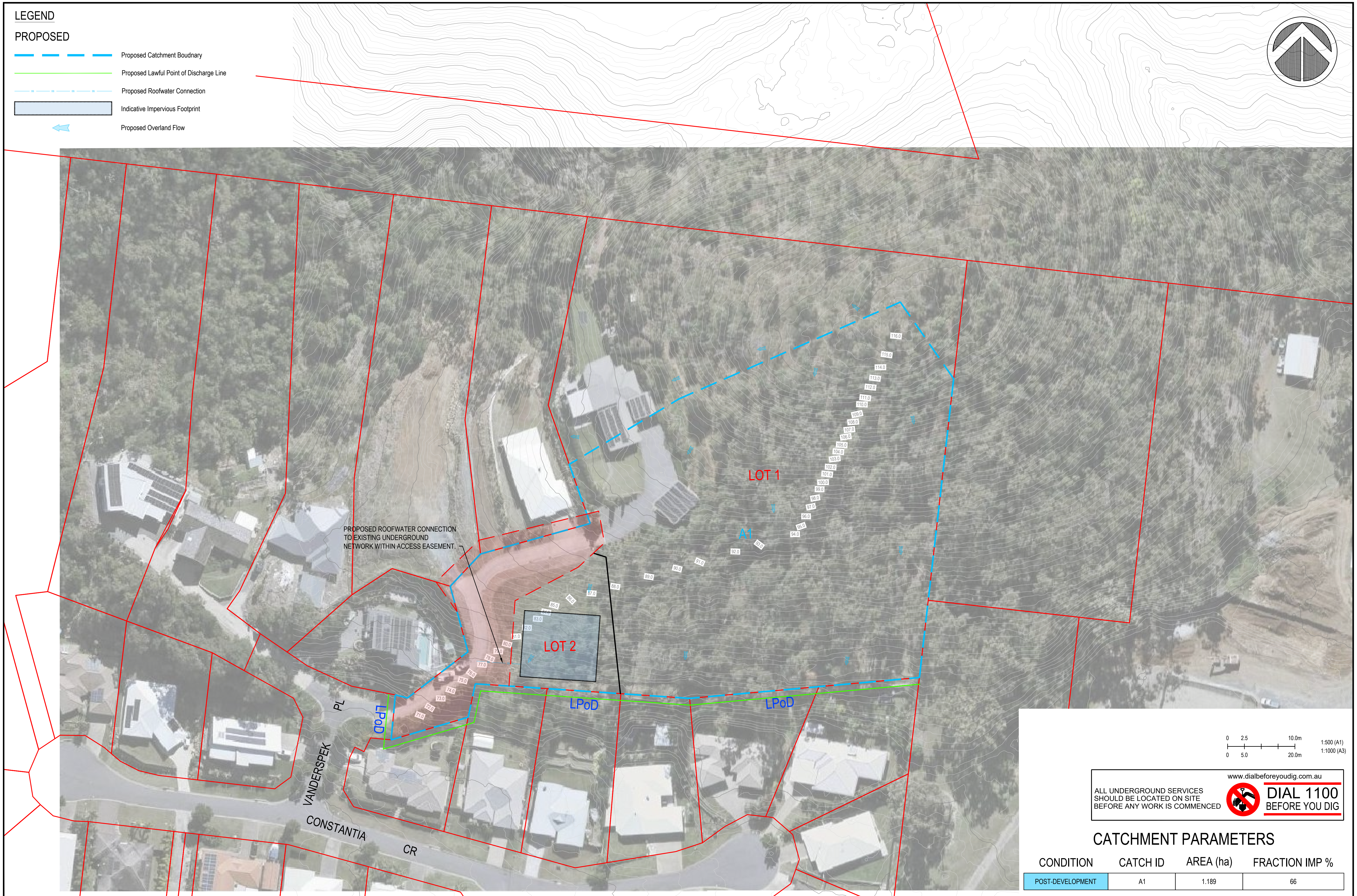
DRAWING TITLE: GENERAL ARRANGEMENT
PRE-DEVELOPMENT

DRAWING NUMBER: CE23062-101-SK
ISSUE: A



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CATCHMENT PARAMETERS

CONDITION	CATCH ID	AREA (ha)	FRACTION IMP %
POST-DEVELOPMENT	A1	1.189	66

FIRST ISSUE	CALCS	DRAWN	DATE	AMENDMENT DETAILS
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DRAWING TITLE: GENERAL ARRANGEMENT POST-DEVELOPMENT

DRAWING NUMBER: CE23062-102-SK

ISSUE: A

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Pre-Qualified Supplier

LOCAL GOVERNMENT PROCUREMENT
APPROVED CONTRACTOR

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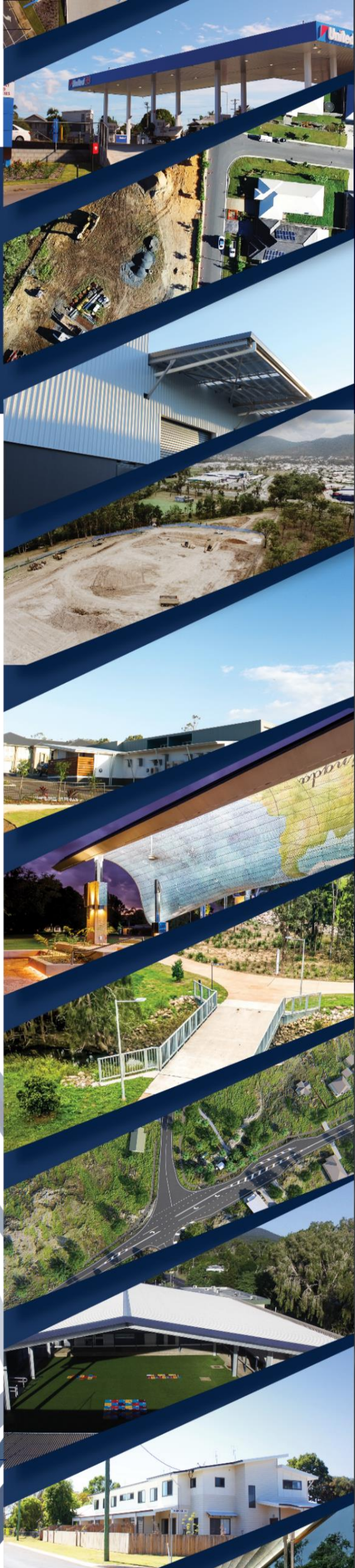

Mr Michael Swann

Site Based Stormwater Management Plan

1 into 2 Lot Residential Development

4 Vanderspek Place, FRENCHVILLE QLD

3 November 2023

FP/001.CE23062 Rev A

Contract No. CE23062

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MOLONEY & SONS
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COMMERCIAL IN CONFIDENCE

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A	*09/06/2023*	*Development Approval*	*LM*	*LM/PAJ*	*PAJ*
B	*26/10/2023*	*Additional Information included for Council*	*LM*	*LM/PAJ*	*PAJ* RPEQ: 21524

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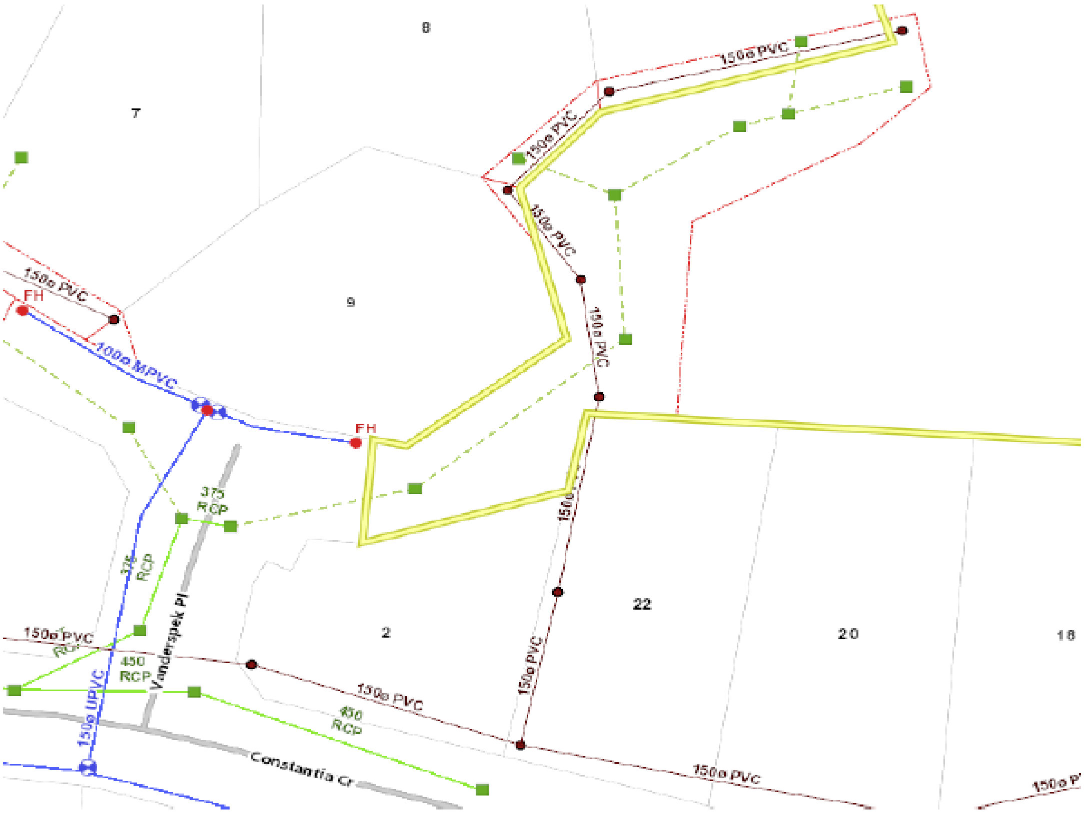

Figure 2 RRC Existing Infrastructure

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2.5. ICM1/XP-STORM Rainfall Parameters

An ICM One/XP-STORM Hydraulic and Runoff model was created to analyse the pre-development and post-development scenarios. The models include a typical 1D node-link connectivity identifying the catchments and hydraulic parameters.

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In accordance with the AR&R & TMR Hydrologic & Hydraulic Modelling Guidelines, for each AEP event the full range of storm durations with associated temporal pattern ensembles were assessed in order to determine the critical durations, flow rates and temporal patterns.

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The pre-development catchment plan (CE23062-101-SK) for the subject site is demonstrated in Appendix A of this report.

The hydrology of the pre-developed catchments has been assessed in accordance with QUDM Section 4.0 using the rational method. From QUDM Section 4.6.6, the theoretical calculated time of concentrations and peak discharge for storm events ranging from the 50% to 1% AEP has been calculated.

The Coefficient of discharge (C_{10}) values were derived from QUDM Table 4.05.3 (a) and Table 4.05.3 (b).

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Catchment ID	Area (ha)	Avg Slope (%)	Fraction Imp (f_i)	Co-efficient of Runoff (C_{10})	Time of Concentration (Tc)
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2.7. Post-Development Hydrology

The proposed development will in effect maintain the pre-development scenario catchment extents. The only noted change is a slight increase (4%) in impervious area, however due to location of the proposed dwelling and the catchments Tc will remain @ 16 minutes, as there will be no increase or change to the post-developed runoff coefficient.

The post-development catchment plan (CE23062-102-SK) is attached within **Appendix A** for further information.

A time of concentration of 16 minutes, as per Friends equation for overland flow paths, was obtained for the post-development time of concentration for each catchment.

Table 2 Post-Development Catchment Parameters

Catchment ID	Area (ha)	Avg Slope (%)	Fraction Imp (f_i)	Co-efficient of Runoff (C_{10})	Time of Concentration (Tc)
A1	1.189	31	0.13	0.44	16 min

2.8. 1D Modelling Assumptions & Methodology

MSE used the following modelling assumptions, to create the ICM One/XP-STORM Models for the proposed development:

- Two (2) separate scenarios were generated, which were:
  - Pre-Development (which included all points of discharge); and
  - Post-Development (which included all points of discharge).
- Each model included runoff nodes for each contributing sub-catchment
- The sub-catchment areas were split into Urban Residential Land Uses with absolute values adopted for percentages impervious as specified in the above Tables 1 & 2.
- Infiltration uniform losses were applied to the pervious areas of the sub-catchments, SCS Soil Type C.
- The models were run at various durations for a constant ARI to determine the critical storm event.

2.9. Design Flow Verification

As the Pre and Post developed scenarios confirmed the same Times of Concentration (Tc's) & runoff coefficients, the only comparable difference being a minor increase (i.e. 4%) in the impervious area.

Based upon this, the peak discharge values obtained for the 1% through to the 50% AEP storm events using the Rational Method and the ICM1/XP-STORM model have demonstrated zero to negligible increases in peak discharges from the contributing catchment.

Furthermore, it is proposed that all roofwater runoff from the newly proposed dwelling will be directed to the existing underground roofwater network beneath the existing access easement.

This will further mitigate any impacts on neighbouring and surrounding downstream properties.

Table 3 Pre Development and Post-Development Difference ICM1/XP-STORM model Results

PRE-DEVELOPMENT					POST-DEVELOPMENT				Difference (m3/s)
Event (ARI)	Event (AEP %)	Critical storm & temp pattern	Catch id	ICM1/XP-Storm (m3/s)	Critical storm & temp pattern	Catch id	Rational method (m3/s)	ICM1/XP-Storm (m3/s)	
100	1	ECN_1pct_10min_6	A	0.0328	ECN_1pct_10min_4	A1	0.0000	0.0350	0.0022
50	2	ECN_2pct_10min_5	A	0.0281	ECN_2pct_15min_9	A1	0.0000	0.0305	0.0024
20	5	ECN_5pct_10min_4	A	0.0224	ECN_5pct_10min_8	A1	0.0000	0.0245	0.0021
10	10	ECN_10pct_10min_4	A	0.0178	ECN_10pct_10min_4	A1	0.0000	0.0199	0.0021
5	20	ECN_20pct_15min_8	A	0.0124	ECN_20pct_10min_5	A1	0.0000	0.0146	0.0022
2	50	ECN_50pc_10min_5	A	0.0060	ECN_50pc_15min_2	A1	0.0000	0.0077	0.0017

3. CONCLUSION AND RECOMMENDATIONS

As outlined in Section 2 of this report, the proposed development will not introduce any nett increase in post-developed stormwater runoff from the site, where actionable damage will be created.

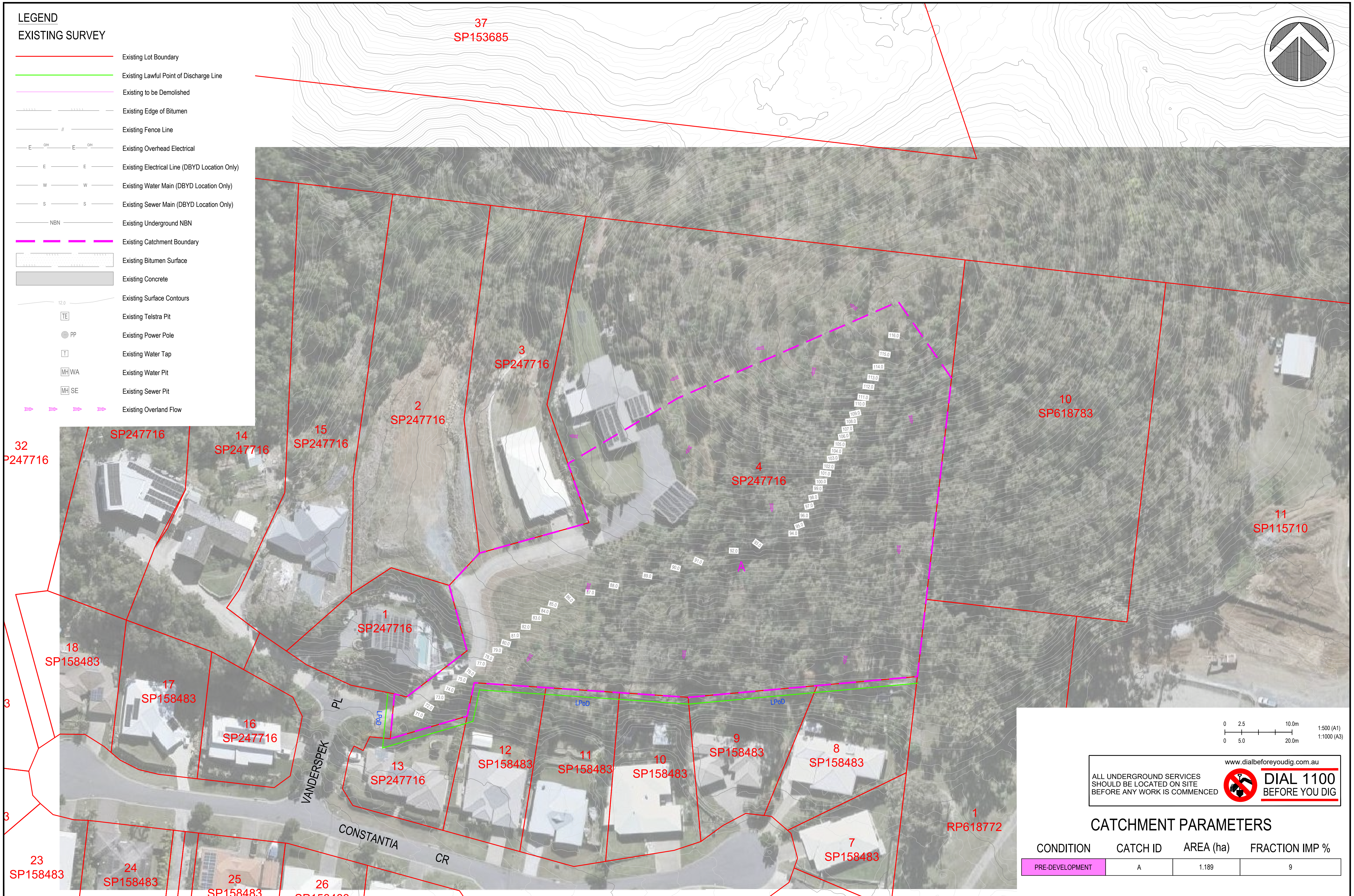
It should be noted, that MSE recommend that all roofwater be directed to a rainwater tank and/or to the existing underground roofwater network beneath the existing access easement.

With this measure being in place, no additional detention will be required.

Therefore, in accordance with the conditions and recommendations outlined in this report and referenced documents (i.e. Engineering Drawings), Moloney and Sons are of the opinion that the proposed development will not create any actionable damage external to the site or any net increase in flows or nuisance to surrounding properties.

It is our opinion that if the abovementioned recommendations are implemented, the proposed development will comply with the intent of Rockhampton Regional Council, Queensland Urban Drainage Manual & AR&R requirements for stormwater quantity management.

APPENDIX A – Engineering Drawings



LEGEND

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JAS-ANZ ISO 9001 CERTIFIED

DATUM

PROJECT No. CE23062

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FOR & ON BEHALF OF MOLONEY & SONS ENGINEERING

CLIENT: MR MICHAEL SWANN

PROJECT: 4 VANDERSPEK PL FRENCHVILLE QLD 1 INTO 2 SUBDIVISION

MOLONEY & SONS ENGINEERING
EXCELLENCE - INTEGRITY - INNOVATION
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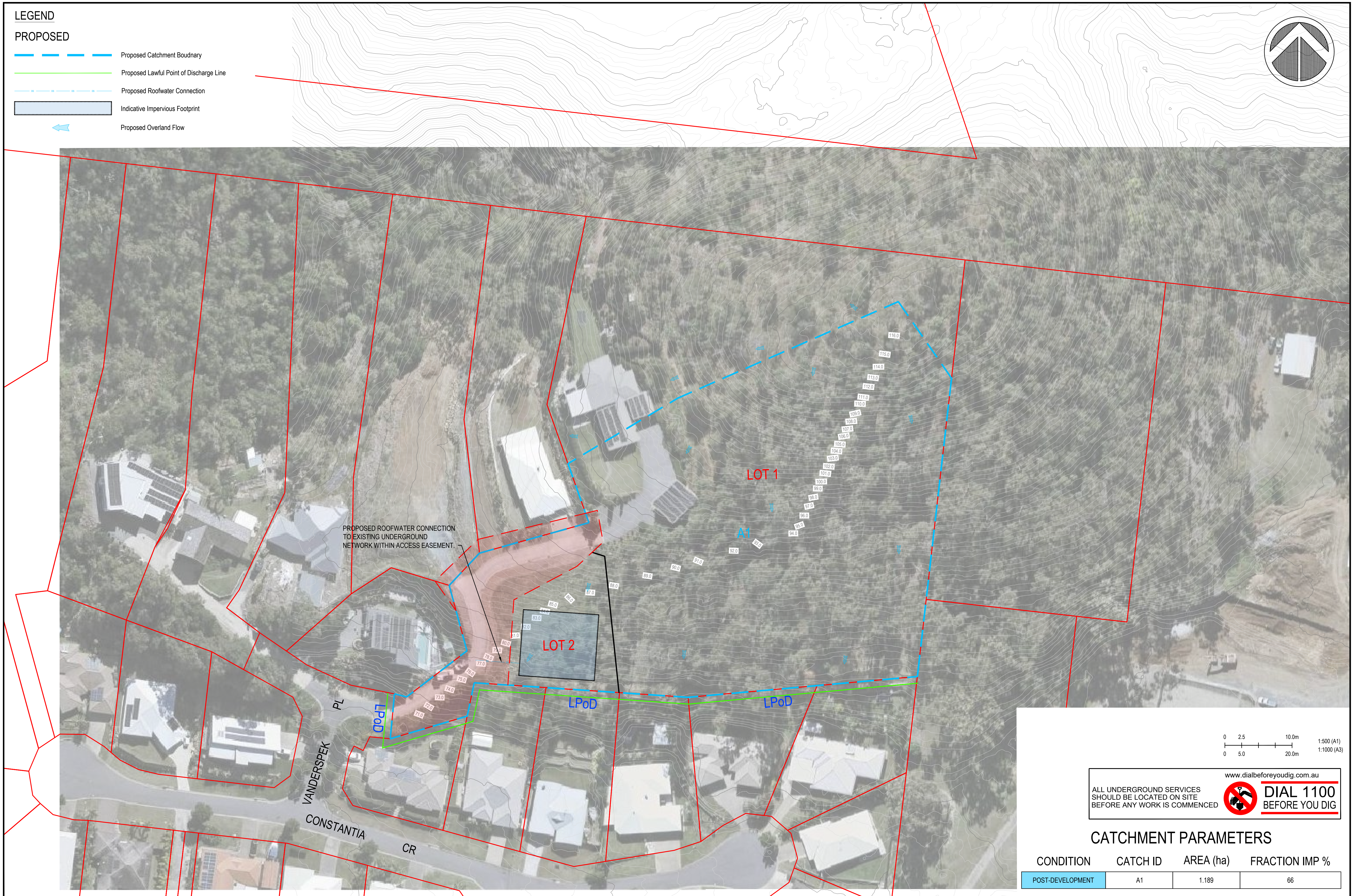
DRAWING TITLE: GENERAL ARRANGEMENT PRE-DEVELOPMENT

DRAWING NUMBER: CE23062-101-SK

ISSUE: A



www.dialbeforeyoudig.com.au

ALL UNDERGROUND SERVICES SHOULD BE LOCATED ON SITE BEFORE ANY WORK IS COMMENCED

DIAL 1100 BEFORE YOU DIG

CATCHMENT PARAMETERS

CONDITION	CATCH ID	AREA (ha)	FRACTION IMP %
POST-DEVELOPMENT	A1	1.189	66

ISSUE	CALCS	DRAWN	DATE	AMENDMENT DETAILS
FIRST ISSUE	LM	AB	06/23	
A	LM	AB	09/06/23	PRELIMINARY ISSUE FOR DEVELOPMENT APPROVAL

DESIGN CHECK

DRAWN CHECK

DATUM

PROJECT No. CE23062

ISSUED FOR APPROVAL

APPROVED

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ISSUE: A

local buy
Pre-Qualified Supplier

LOCAL GOVERNMENT PROCUREMENT
APPROVED CONTRACTOR

MODUS
Transport and Traffic Engineering

Traffic Engineering Letter

To	Jacob Dalton, Reel Planning	Date	29 September 2023
Prepared by	Bradley Fuller, Modus Traffic Engineer	Approved by	Harj Singh, Modus Director (RPEQ 22364)
Location	4 Vanderspek Place, Frenchville		
Subject	Proposed Access Arrangements – Traffic Engineering Letter		
Status	Final	Attachments	**Appendix A**: Traffic Concept Plans **Appendix B**: Swept Path Assessment

1 Introduction

1.1 Overview

Modus has been commissioned by Reel Planning to provide traffic and transport advice in relation to the access arrangements for the proposed Reconfigure of Lot (ROL) development located at 4 Vanderspek Place, Frenchville.

1.2 Project Context

The proposed RoL development will consist of a one (1) Lot into two (2) Lot subdivision off the existing Lot 4 on SP247716. The proposed Lot 1 will retain the existing residential dwelling where proposed Lot 2 is anticipated to accommodate a new residential dwelling.

The project area and proposed subdivision plans are illustrated on Figure 1 below.

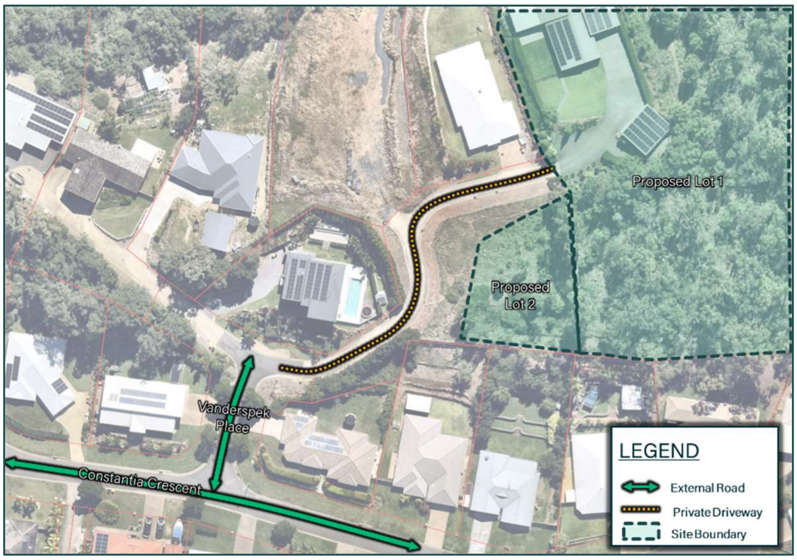
Figure 1 Project Area Context

2 Existing Access Conditions

Existing Lot 4 on SP247716 is currently accessed via a private concrete driveway that stems off Vanderspek Place, with a typical width of 5.2m along the driveway extent. The private driveway also provides access to Lot 2 and Lot 3 on SP247716, in total providing access to three (3) lots. Given the site topography surrounding the project site, the private driveway slopes down from the north. It is also noted that the private driveway currently provides heavy vegetation to the east of the driveway extent.

An aerial and street view perspective of the private driveway is provided in Table 1 below.

Table 1 Private Driveway Conditions

Aerial Perspective	
	
Google Street View Perspective	
	

3 Proposed Access Conditions

3.1 Council Formal Advice

The client has provided Modus with Rockhampton Regional Council formal advice regarding the access arrangements for the private driveway for the proposed RoL, as detailed below:

"Ensuring the access driveway is of a sufficient width to cater for the four lots benefitting from it (Council preference 5.5m wide)".

While it is understood that Rockhampton Regional Council's preference is to widen the driveway to 5.5m in line with industry standards for two-way laneways, Modus is of the opinion that widening the full driveway extent is not the most practical nor feasible solution with respect to constructability.

3.2 Proposed Passing Bay

Therefore, Modus recommends that a passing bay be provided along the driveway extent to accommodate two-way movements as opposed to widening the driveway to 5.5m along its extent. Provided that the Rockhampton Regional Council planning scheme does not stipulate requirements for a passing bay / easement conditions, the Brisbane City Council City Plan TAPS PSP has been referenced to inform the passing bay design and feasibility.

For a driveway that provides access to four (4) lots (proposed development scenario) for sites that are more than 40m away from dedicated road, the easement passing bay / easement conditions as per the Brisbane City Council City Plan TAPS PSP is outlined on Figure 2.

Figure 2 Brisbane City Council City Plan TAPS PSP Passing Bay / Easement Requirements

Table 11—Internal access requirements for rear residential lots

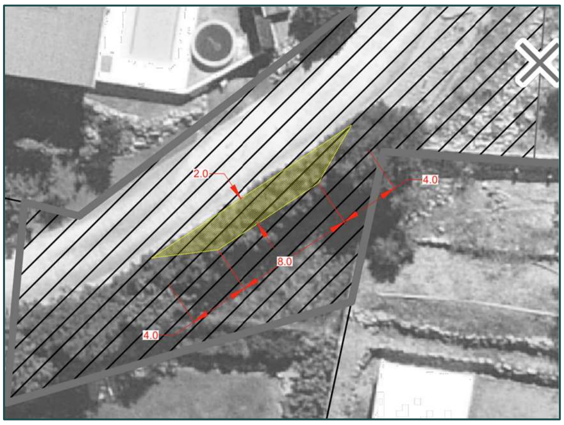
No of dwelling units	Distance from dedicated road	Easement width	Minimum requirements
1–3	≤40m	3.5m	Grade N25 concrete driveway: 2.5m wide, 125mm thick, F72 reinforcing mesh
4–5	≤40m	4.0m	Grade N25 concrete driveway: 3.1m wide, 125mm thick, F72 reinforcing mesh
≥6	≤40m	6.5m	Grade N25 concrete driveway: 5.5m wide, 160mm thick, F82 reinforcing mesh
1–5	>40m	6.0m	Grade N25 concrete driveway: 3.1m wide, 125mm thick, F72 reinforcing mesh Grade N25 concrete passing lanes: 2.0m wide x 6.0m length, 1 in 2 taper at 60m centres (1) Alternative asphalt driveway: 3.1m wide, nominal traffic loading 1.5 x 104 ESA (1) Alternative asphalt passing lane: 2.0m wide x 6.0m length, 1 in 2 taper at 60m centres
≥6	>40m	6.5m	Grade N25 concrete driveway: 5.5m wide, 160mm thick, F82 reinforcing mesh (1) Alternative asphalt driveway: 5.5m wide, nominal traffic loading 1.5 x 104 ESA

Therefore, based on the abovementioned criteria the passing bay / easement conditions are as follows:

- Concrete Driveway: Minimum width 3.1m,
- Passing Bays: 2.0m wide by 8.0m in length with 1 in 2 tapers.

Noting the existing driveway width of 5.2m exceeds the minimum width outlined above, the passing bay design to achieve compliance with the Brisbane City Council City Plan TAPS PSP is outlined in Table 2 (the traffic concept plan demonstrating the required passing bay is provided at Appendix A).

Table 2 Proposed Passing Bay Design



The proposed passing bay design and location is considered acceptable on the following basis:

- The passing bay location allows for outbound vehicles to pull in and give-way to inbound vehicles, where the proposed location provides sufficient visibility for an outbound vehicle to observe an inbound vehicle,
- A swept path assessment has been conducted which confirms that an inbound and outbound B99 design vehicle (VAN) is able to passing along the driveway utilising the passing bay, whilst ensuring a consistent 600mm clearance between both vehicles. The swept path assessment is provided at Appendix B,
- The private driveway will accommodate four (4) residential dwellings in total, which corresponds to a peak hour trip generation of four (4) vehicles per hour:
    - Adopting typical 80% / 20% inbound and outbound directional distributions in the AM and PM peak hour indicates that up to three (3) vehicles will travel in one direction while only one (1) vehicle will travel in the opposing direction,
    - This indicates that a conflict between an inbound and outbound vehicle will occur at most once in the peak hour period (every 60 minutes), of which conservatively assumes the inbound / outbound vehicle trip movements occur at the same time.

4 Construction Vehicle Parking

Furthermore, the client has informed that the construction vehicles will park / store in the areas illustrated on Figure 3, and detailed below:

- The portion of land directly north of the proposed building footprint,
- The existing driveway located within Lot 4 on SP247716.

Figure 3 Construction Vehicle Parking Areas

Modus considers these parking areas acceptable and will not have a substantial impact on the operations of the surrounding Residential Dwellings. Modus also recommends that the tenants of the surrounding Residential Dwellings be informed prior to construction vehicles accessing the site, such that there is awareness of the forthcoming increase in construction vehicles.

5 Summary

Therefore, Modus is of the opinion that the proposed passing bay provision is acceptable in ensuring two-way movements along the private driveway to accommodate the proposed RoL development located at 4 Vanderspek Place, Frenchville.

Should there be any issue with the above, please contact the undersigned.

Yours sincerely,

MODUS TRANSPORT AND TRAFFIC ENGINEERING

HSingh

Harj Singh
Director
RPEQ 22364

APPENDIX A

Traffic Concept Plans

Legend

- Subject site - 4 Vanderspek Place, Frenchville
- Proposed lot
- Proposed building footprint (300sqm)
- Proposed crossover
- Cadastral boundary
- Easement

REFERENCES

Cadastre - (c) The State of Queensland (DNRM DCDB)

Coordinate System: GDA2020 MGA Zone 56

Images are not orthorectified, overlays are best fit. Features are based on topographical data and may vary.

Nearmap 14 April 2023.

Indicative only.

Date: 27/09/2023

N

1:600

0 5 10 20 M

TITLE

Proposed Subdivision Plan (1 into 2 Lots)

PROJECT

4 Vanderspek Place, Frenchville

REEL PLANNING
Urban & Rural Strategies

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APPENDIX B

Swept Path Assessment

N

0 1 2 3 4 5 6 7 8 9 10m
2 1.5 1 0.5
SCALE 1:200

PROJECT

4 VANDERSPEK PLACE

CLIENT

REEL PLANNING

DRAWING TITLE

PASSING BAY DESIGN

DRAWING NUMBER

MOD221244QLD - SK01

DATE	REVISION
29 SEPT 2023	B

REV	DRAWN BY	APPROVED	DATE	AMENDMENT DETAILS

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GENERAL NOTES:

1.PRELIMINARY DESIGN ISSUED FOR COMMENT ONLY.

2.ALL DIMENSIONS ARE PROVIDED IN METERS UNLESS NOTED OTHERWISE.

Document Set ID: 40692070
Version: 1, Version Date: 10/01/2024

VEHICLE USED IN SIMULATION

B99

5.20

0.95 3.05

Meters

Width	: 1.94
Track	: 1.84
Lock to Lock Time	: 6.0
Steering Angle	: 33.5

SWEPT PATH LEGEND

- VEHICLE BODY
- FRONT TIRES
- VEHICLE PATH
- VEHICLE CLEARANCE (300mm)
- 10KM/H DESIGN SPEED
- VEHICLE

N

SCALE 1:200

PROJECT

4 VANDERSPEK PLACE

CLIENT

REEL PLANNING

DRAWING TITLE

B99 SWEPT PATH ASSESSMENT

DRAWING NUMBER

MOD221244QLD - SK02

DATE	REVISION
29 SEPT 2023	B

REV	DRAWN BY	APPROVED	DATE	AMENDMENT DETAILS

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