ACT Geotechnical Engineers Pty Ltd
ACN 063 673 530

5/9 Beaconsfield Street, Fyshwick, ACT, 2609
PO Box 9225, Deakin, ACT, 2600
Ph: (02) 6285 1547
Email: jeremy.murray@actgeoeng.com.au

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Merimbula Central Pty Ltd
via email: merimbulacentral@gmail.com

Attention: Robert Green

Dear Sir

PROPOSED RETAIL/RESIDENTIAL DEVELOPMENT - 29 to 43 MARKET STREET, MERIMBULA, NSW

PRELIMINARY GEOTECHNICAL ASSESSMENT

At the request of Merimbula Central Pty Ltd, ACT Geotechnical Engineers Pty Ltd carried out a desk-top preliminary geotechnical assessment as part of a proposed retail/residential development, in Merimbula, NSW.

It has been indicated that the development may comprise the construction of a six-storey retail/residential building, which will be founded close to existing grade.

It is understood the study is required to provide preliminary geotechnical information and identify possible constraints for redevelopment of the site.

This report summarises expected site geotechnical and geological conditions based on research of available geological and topographical maps, past and recent aerial photographs, and available geotechnical investigations conducted in the area.

1 SITE DESCRIPTION & GEOLOGY

The site ~45m x 75m site located at 29 to 43 Market Street, in Merimbula, NSW, and is bounded by Market Street to the east, Monaro Street to the south, Palmer Lane to the west, and existing retail buildings to the north. Figure 1 shows the site locality.

The site is presently occupied by one and two-storey retail buildings that cover the entire site. The groundsurface is relatively flat, dipping gently south. An old creek alignment used to drain E/SE across the southern tip of the site, into Merimbula Lake, which is about 40m E/SE of the site. This creek was presumably backfilled and replaced by a stormwater drain during development of the Merimbula town centre. Figure 2 is a recent aerial photograph showing the present site layout, while Figure 3 is a topographic plan of the area.

The 1:250,000 Bega-Mallacoota Geology map documents the site to be covered by Tertiary age sediments of fluvial sands and lacustrine clays, underlain by Devonian age Merimbula Group bedrock, which includes sandstone, siltstone, and shale.

Figure 4 is an extract of the 1:250,000 Bega-Mallacoota Geology map , showing the above mentioned geological features.

3 EXPECTED SUBSURFACE CONDITIONS

ACT Geotechnical Engineers have conducted many investigations and been involved in various feature constructions in and around the Merimbula Town Centre area. Such projects include an apartment development at 95 Main Street, shade structure at Club Sapphire, supermarket development at 107 to 133 Main Street, foreshore redevelopment on Beach Street, several retain developments on Market Street, and an investigation for a proposed retail development (that never proceeded) in the carpark to the immediate west of this site.

Information was also provided by Mr John Moffatt, who was the structural engineer for the construction of the newsagent building located at 33 Market Street. He indicated that there was a~6m wide concrete box culvert present under the site, which collects stormwater from the west of the site, to discharge into Lake Merimbula to the east of the site. The ground either side of the culvert was very soft with a low bearing capacity. Therefore, the newsagency building was supported on driven steel piles (250mm Universal Beams), founded in weathered mudstone bedrock at a depth of about 15m.

Based on these past involvements, as well as reference to past and present topographical maps and local soil and geology maps, the subsurface profile at the site is expected to comprise the following:

Depth	Geological Profile
0m to ~0.5m/1m	FILL; Sand, Silty Sand, & Clayey Sand, fine to coarse sand, low plasticity clay, grey-brown, dark grey, some gravel to 60mm size, dry to moist, loose to medium dense.
0.5m/1m - 9m/12m	FLUVIAL/COASTAL SANDS: Sand & Clayey Sand; fine to coarse sand, some low plasticity silty clay fines, yellow-brown, yellow, grey, dry to moist (but wet/saturated below the groundwater table), medium dense, becoming very loose with depth.
below 9m/15m	SANDSTONE or MUDSTONE BEDROCK: Extremely weathered (EW), extremely to highly weathered (EW/HW), and highly weathered (HW), and extremely weak to weak rock, pale grey, some yellow and pale red.

Based on the investigation conducted in the carpark to the immediate west of this site, the depth to bedrock is expected to be about 9m at the northern end, deepening to about 12m at the southern end. As indicated by Mr John Moffatt, the depth to bedrock is expected at about 15m depth in the centre of the site. The proposed development is expected to be founded close to existing grade, and the foundation is expected to generally comprise medium dense, coastal/fluvial sands.

Below the groundwater level, the fluvial sands could be very loose/very soft, and any excavations are expected to be unstable and prone to collapse.

4 EXPECTED GROUNDWATER CONDITIONS

The permanent groundwater table is expected to be present at about at about 1m/3m depth, and will correspond closely to sea level/the water level in nearby Lake Merimbula. The groundwater level will fluctuate with tidal fluctuations and in response to rainfall. Perched groundwater may also be present at shallower depth within the more pervious soils. Groundwater inflows into excavations below the groundwater table are expected to be rapid.

5 PRELIMINARY ENGINEERING ASSESSMENT

5.1 Anticipated Excavation Conditions & Use of Excavated Material

The site soils to at least 10m depth are diggable by backhoe or excavator. Permanent groundwater is expected to be at about 1m/3m depth below existing surface levels and excavations below this depth may be problematic due to rapid groundwater inflows and collapse of excavation sides in the saturated, loose fluvial/coastal sands. A de-watering system such as sheet-piling with spear-points or other suitable retaining wall would be required prior to bulk excavations into these strata, and liners would be required for bored pier holes.

Existing sandy fill and the sandy fluvial/coastal sands can be used as engineered fill in building and pavement areas if required.

5.2 Structure Footings

The proposed development is expected to be founded close to existing grade, and the foundation is expected to generally comprise loose to medium dense, coastal/fluvial sands. The coastal/fluvial soils are expected to generally be medium dense, however, they will become saturated and very loose/very soft below groundwater level. Given the expected high loads, and to avoid differential settlement, it is suggested that all footings be founded in the weathered sandstone bedrock.

Possible building footing options include:

- Driven steel H-piles or driven pre-cast concrete piles terminating in the underlying bedrock.
- Bored cast in-situ reinforced concrete piers, end-bearing on the underlying bedrock. Bored cast in-situ piers would require permanent or temporary liners to stabilise the hole sides against collapse of saturated fill and loose granular sands.
- Continuous flight auger (CFA) piers terminating in the underlying bedrock.
- Screw-in steel piles terminating in the underlying bedrock.

For these pile systems, the vertical resistance to imposed footing loads would be provided by a combination of shaft adhesion in the overburden soils and end-bearing on weathered bedrock. As a guide, the allowable end-bearing and shaft adhesion values given in Table 2 can be used in the design of driven and bored piles.

TABLE 2
Recommended Allowable End-Bearing Pressures & Shaft Adhesion for Piles

Foundation Material	Depth Range	Driven Steel or Pre-cast Concrete Piles		Bored Cast In-situ & CFA Piers	
		Allowable End-Bearing Pressure	Allowable Shaft Adhesion	Allowable End-Bearing Pressure	Allowable Shaft Adhesion
Existing Fill	0m - 0.5m/1m	N.A.	0kPa	N.A.	0kPa
Fluvial/Coastal Sands	0.5m/1m - 10m/11m	N.A.	10kPa	N.A.	10kPa
Weathered Bedrock*	9m/12m	2000kPa	50kPa	1000kPa	100kPa

* - The depth to bedrock, as well as strength and weathering of the bedrock needs to be accurately determined by a detailed borehole investigation.

Screw-in piles consist of hollow steel pipe sections with one or more helixes attached to the lower end of the pile. The piles can be screwed through the fill and weak/collapsible sands to end-bear on the weathered sandstone bedrock below. Like driven piles, they are installed from ground level and avoid problems associated with groundwater inflows and collapsing soils, which may need to be overcome in open excavations such as bored pier holes. Groups of screw-in piles can be integrated into a reinforced concrete capping block or slab to form the required column footing. Vertical capacity of the piles is determined by the specialist installation contractor, based on previous test performance data, and/or site trial load-testing. Single pile point-load capacities range from 2 to 65 tonnes, while pile clusters can typically carry maximum loads up to 200 tonnes.

Alternatively, a rigid raft slab footing option could be considered. For this, an overall average allowable bearing pressure of 100kPa can be assumed for a raft footing founding in the medium dense fluvial/coastal sands. A structural raft-foundation interaction analysis to determine the settlement distribution, and the bending moments and shear forces in the raft would need to be carried out. For this, a Young's Modulus of 15MPa could be assumed for the foundation.

All footing excavations should be inspected and approved by a geotechnical engineer to confirm the foundation material and design values, and to ensure the excavations are clean and stable.

Groundslabs can be constructed directly on natural soil or controlled fill. Cut subgrades should be proof-rolled by a vibratory pad-foot roller, and any wet or deforming subgrades replaced to their full depth. Suitable replacement fill can be placed in not thicker than 150mm layers to not less than 98%StdMDD.

5.3 Temporary Batter Stability

For temporary stability during construction, excavations above groundwater level can be formed at a slope of 1(H):1(V). If space restrictions prevent battering back at this slope, excavations should be supported by retaining walls, or in the case of trenches, by timber shoring or movable safety cages or shields. It is advisable that all unsupported temporary excavation batters be inspected by a geotechnical engineer to confirm their stability or to advise on stabilisation treatment. Excavations below the groundwater table will be prone to collapse.

5.4 Permanent Batter Stability

Any permanent unsupported batters should be formed no steeper than 1(H):1(V). Permanent soil batters would need to be protected against erosion, either by stone pitching, shotcreting, or other suitable means. If steeper slopes are required, structural retaining walls will be required.

5.5 Retaining Wall Design Parameters

Low retaining walls (<3m high) constructed in open excavation, with the gap between the excavation face and the wall backfilled later, can be designed for a triangular earth/rock pressure distribution given by:

$$\sigma_h = (K\gamma' h) + 0.6q$$

where,

- σ_h is the horizontal earth pressure acting on the back of the wall, in kPa
- K is the dimensionless coefficient of earth pressure; this can be assumed to be 0.4 when the top of the wall is unrestrained horizontally, and 0.6 when the top of the wall is restrained (i.e. by building slabs etc.)
- γ' is the effective unit weight of the backfill, and can be assumed to be $20kN/m^3$ for a lightly compacted soil backfill
- h is the height of the backfill, in metres
- q is any uniform distributed vertical surcharge acting on the top of the backfill, in kPa.

Free-draining granular backfill or geotextile drains should be placed behind all walls. These should connect to weep holes and/or a collector drain, and into the stormwater system. Granular backfill should be wrapped in a suitable filter fabric to minimise infiltration of silt/clay fines into the drains.

Apart from structural restraints such as floor slabs, resistance to overturning and sliding of retaining walls is provided by frictional and adhesive resistance on their base, and by passive resistance of the soil at the toe of the wall. For a soil foundation, an ultimate base friction factor (tanδ) of 0.4, base adhesion (c) of 10kPa, and allowable passive earth pressure coefficient Kp=2.5 can be used for calculation of sliding resistance.

5.6 Pavement & Groundslab Subgrades

Existing filled ground and natural fluvial/coastal sands above the groundwater level can be improved to provide adequate support for new carpark pavements, and it is recommended that:

- Subgrades of existing fill/natural soils be initially removed to a depth of 0.5m below required subgrade level.
- The exposed ground be proof-rolled by a heavy vibrating smooth drum-roller until deformation of the surface ceases.
- The previously excavated sandy soils and other suitable soils be replaced and compacted to final subgrade levels in not thicket than 150mm layers to not less than 98% of standard maximum dry density, or a Density Index of 70%.

A small roller or vibratory plate compactor may need to be used near block boundaries and existing roads to avoid causing settlement of sands beneath existing structures and pavements. A subgrade design CBR value of 5% can be used for properly compacted existing subgrades of site sandy soils.

5.7 Drainage & Groundwater Control

Surface drainage measures should ensure that rainwater or seepage water does not pond against structure footings or pavements. The permanent groundwater table is expected to be present at about at about 1m/3m depth, and will correspond closely to sea level/the water level in nearby Lake Merimbula. The groundwater level will fluctuate with tidal fluctuations and in response to rainfall. Perched groundwater may also be present at shallower depth within the more pervious soils. Groundwater inflows into excavations below the groundwater table are expected to be rapid. This groundwater advice would need to be better established during a site-specific geotechnical investigation.

6 FURTHER INVESTIGATION

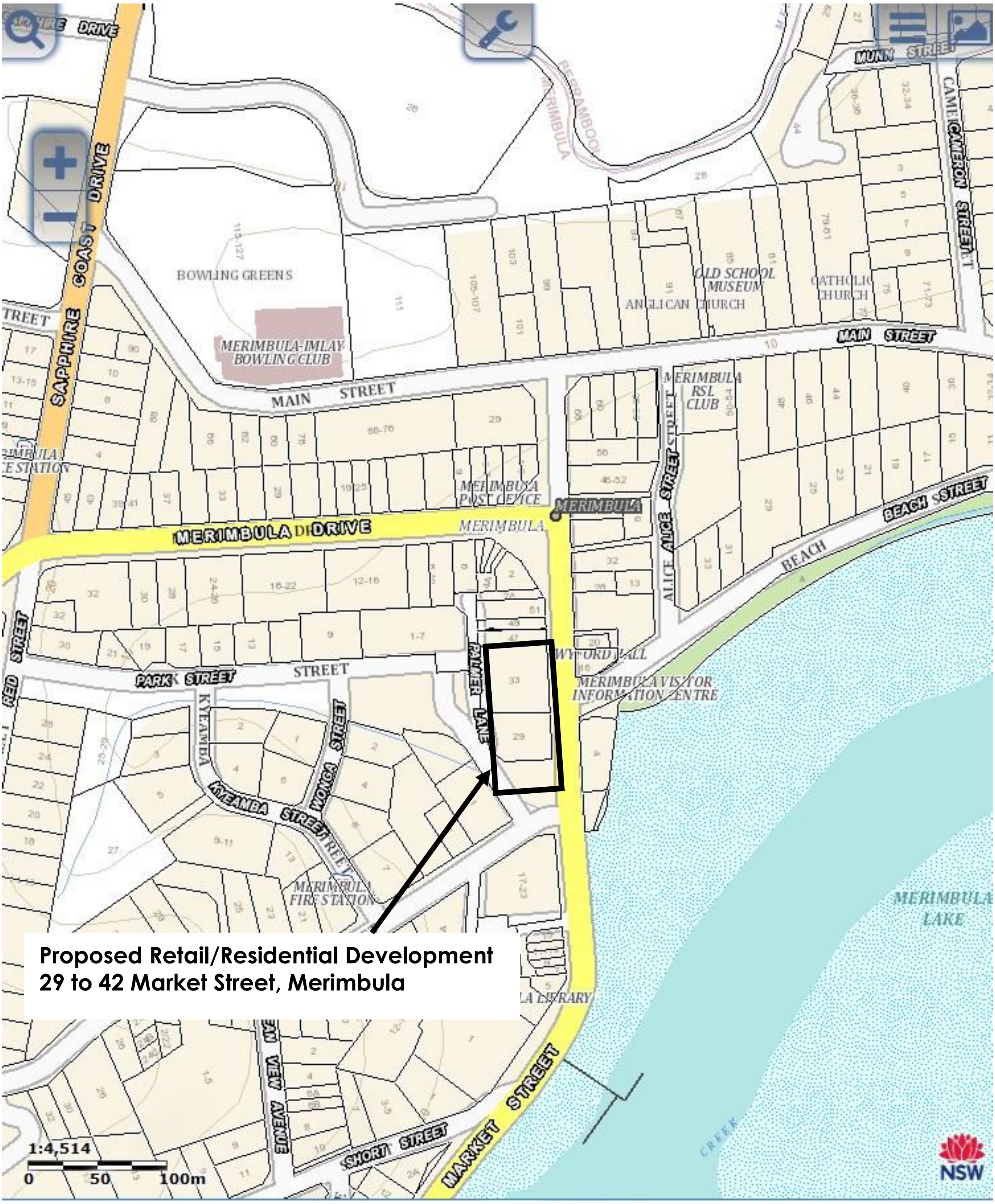
This report is of a preliminary nature, based on limited information from past investigations, and on our knowledge of the various geological formations.

Given the expected large column loads, and potentially deep bedrock, a comprehensive site-specific, detailed geotechnical investigation by cored boreholes will be required for any significant proposed development, to properly assess the depth of the bedrock, groundwater conditions, and material properties.

Should you require any further information, please contact our office.

Yours faithfully
ACT Geotechnical Engineers Pty Ltd

Jeremy Murray
Director
Senior Geotechnical Engineer
FIEAust CPEng EngExec NER APEC Engineer IntPE(Aust)



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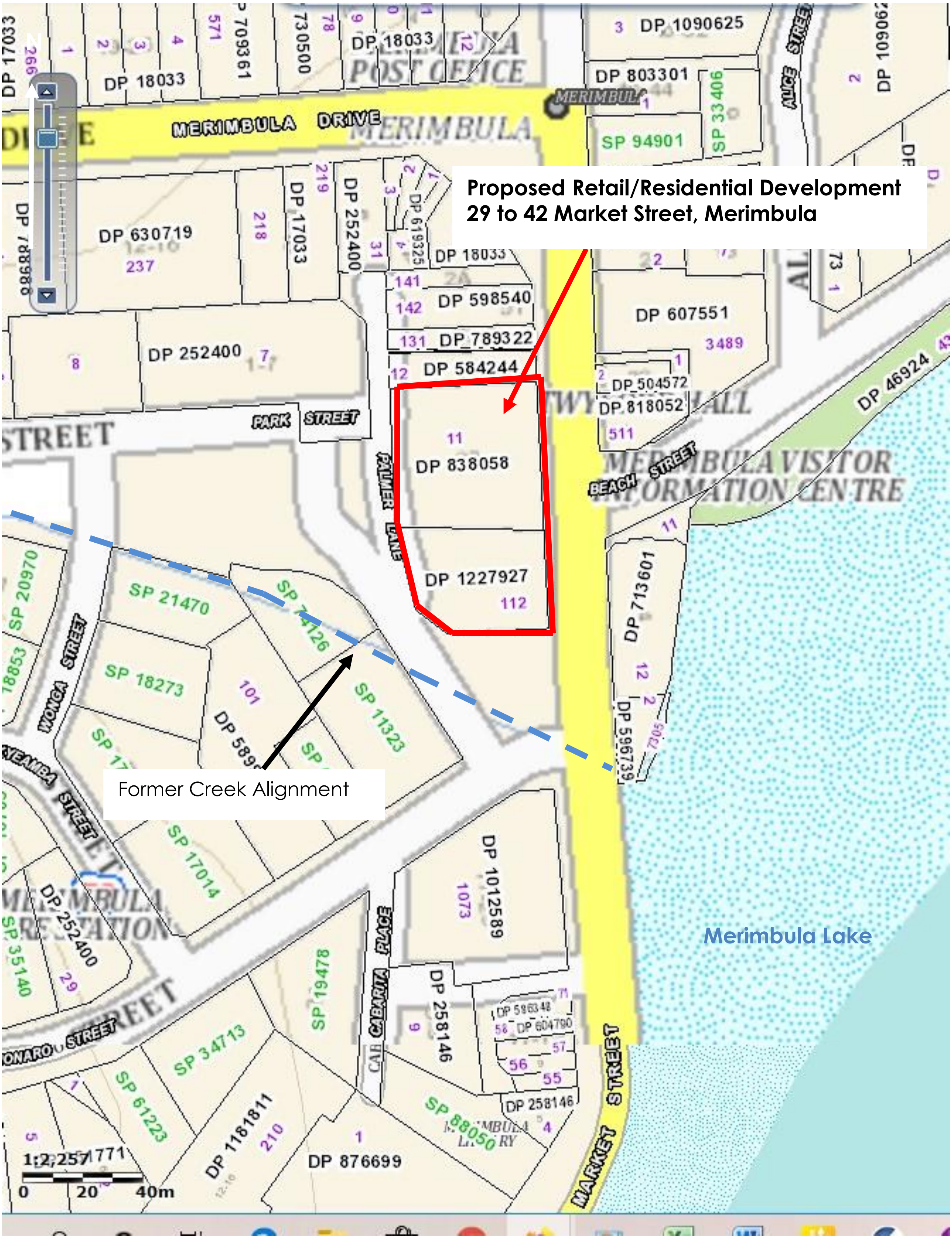
RETAIL/RESIDENTIAL DEVELOPMENT – 28 to 43 MARKET ST, MERIMBULA

LOCALITY PLAN

ACT Geotechnical Engineers Pty Ltd	**C10929**	**FIGURE 1**

SPUNGROW PTY LTD
RETAIL/RESIDENTIAL DEVELOPMENT – 28 to 43 MARKET ST, MERIMBULA
AERIAL PHOTOGRAPH

ACT Geotechnical Engineers Pty Ltd	C10929 FIGURE 2



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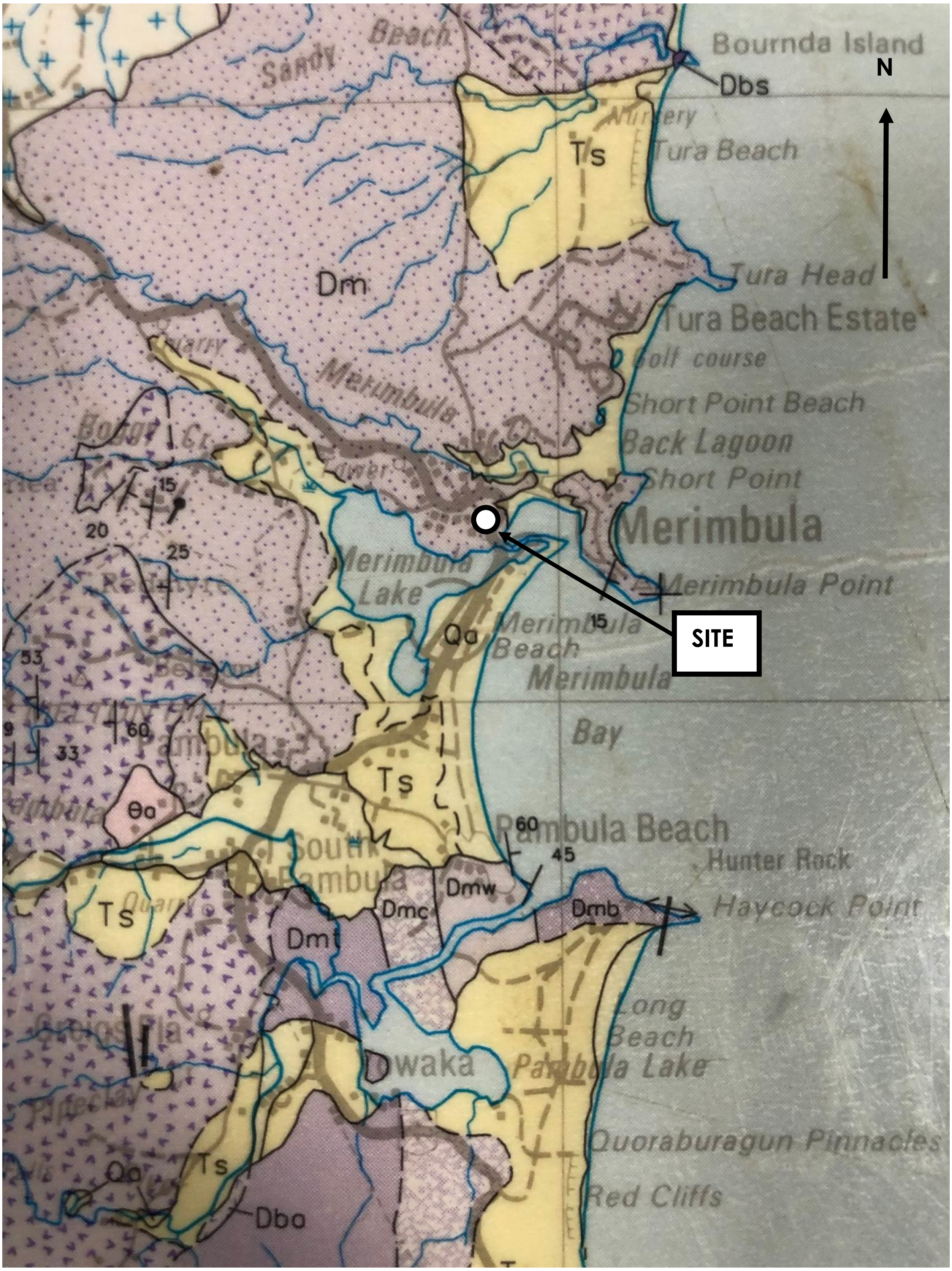
TOPOGRAPHIC PLAN

ACT Geotechnical Engineers Pty Ltd	C10929	FIGURE 3



Ts – Fluvial Sand & Lacustrine Clays
Dm – Merimbula Group bedrock

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GEOLOGY MAP EXTRACT

ACT Geotechnical Engineers Pty Ltd	C10929	FIGURE 4