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Our Ref: PSM5872-006L REV1

21 November 2025

Technical Director
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Level 11
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Attention: Juan Lewis

Dear Juan

**RE: MAMRE ROAD DATA CENTRE CAMPUS (706-752 MAMRE ROAD, KEMPS CREEK)
INTERIM GEOTECHNICAL DESIGN ADVICE**

1. Introduction

This letter presents the interim geotechnical design advice (IGDA) for the proposed data centre development at 706-752 Mamre Road, Kemps Creek NSW (the Site).

The IGDA in this letter has been prepared on the following basis:

- The site is proposed for development under a State Significant Development Application (SSDA) as a data centre campus comprising:
  - Approximately 26 shells across four-storeys data centre buildings (4x four shells and 2x five shells), including six technical office buildings, plus a campus office.
  - Incoming and internal electrical substations and associated infrastructure
  - Site preparation, including earthworks, stormwater, sewer, roads, and associated infrastructure.
- The subsurface conditions encountered are as logged and inferred from the site investigations (ref: PSM5872-005R REV1, dated 21 November 2025)
- The proposed earthworks are completed in accordance with the Bulk Earthworks Specification (ref: PSM5872-007S REV1, dated 21 November 2025).

If any of the above is not applicable, PSM should be requested to confirm that the design advice below is still valid.

Figure 1 presents the site locality plan.

2. Bulk Earthworks

We have been provided with the following documents with regards to the proposed bulk earthworks:

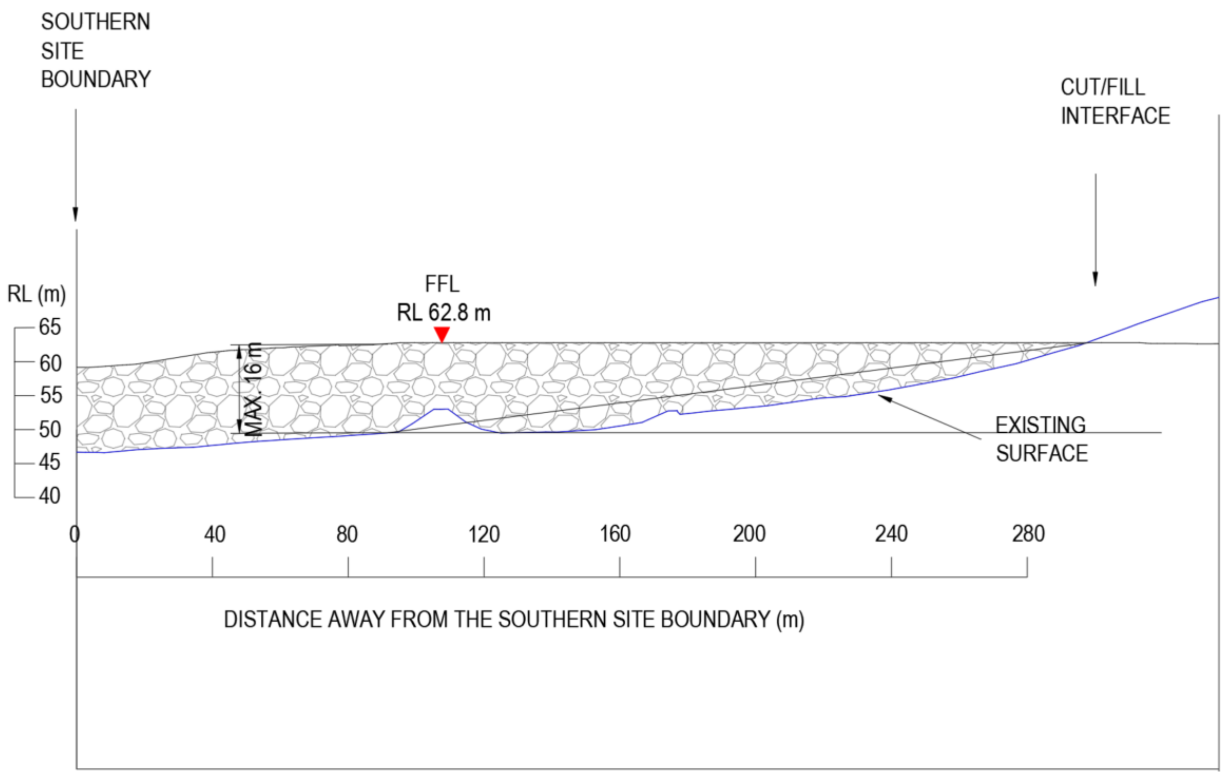
- Masterplan by Greenbox Architecture(ref: SSDA-A-0100 dated 7 November 2025)
- Bulk earthworks cut-fill plan by AT&L (ref: SYD4-SITE-DRG-ATL-CIV-02031 dated 10 November 2025).

Based on the provided drawings, we understand the following bulk earthworks details:

- Max cut depth: 21.0 m
- Max fill depth: 16.0 m
- The bulk earthworks level ranges between RL 51 m and 67 m.

We understand the gradient of the existing landform is relatively low, e.g. approx. 6H to 20H:1V. Some retention structures (i.e. up to 12 m high) are to be developed along the southern boundary to the adjacent site.

Inset 1 presents the sketch of cross section outlining the variation in ENGINEERED FILL thickness in relation to the Site. The section location is presented in Figure 1.



Inset 1: Typical north-south section across the deep fill area of the Site showing the maximum thickness of ENGINEERED FILL to be placed during the proposed earthworks

The design intent is for the bulk earthworks on site to be completed in accordance with the PSM Specification PSM5872-007S (the Specification). The Specification sets out clearly the roles and responsibilities of the earthworks contractor and its Geotechnical Inspection and Testing Authority (GITA). The Specification will only be varied with the consent of PSM to ensure that this interim design advice is able to be confirmed at the completion of the earthworks.

The Specification allows for a broad range of fill to be incorporated into the earthworks. The Specification requires close inspection and frequent testing to provide a high level of confidence that the completed work complies with the Specification.

We have based our assessment of moduli on numerous plate load tests (PLTs) completed on VENM/ENM fills by PSM. Fill placed in accordance with such a specification is referred to herein as ENGINEERED FILL. It is our opinion that the majority of the cut material would be suitable for reuse on the site as ENGINEERED FILL without the requirement for crushing. The criteria for and selection of acceptable material is set out in Clause 3.3 of The Specification.

If the structural or civil engineer requires engineering properties different to those provided in Section 3, then the Specification can be modified such that these properties will be obtained in the final earthworks.

This allows the additional cost of the earthworks to be balanced against any economies achieved in other parts of the works.

3. Interim Geotechnical Design Advice

3.1 General

The design of the data storage centre should consider the following factors:

- Fill settlement
- Load induced settlement (slab loads, surcharges, etc.)
- Characteristic surface movement due to moisture changes.

3.2 Fill Settlement (Deep Fill)

The following advice regarding settlements that may be experienced by the structures founded on the deep fill area (more than 10 m deep) as initial advice. We recommend that once details of the design are advanced further specific advice be sought. This advice is additional to the advice provided in previous sections.

The civil and structural designers need to consider the settlements due to the fill depths in their design. Three settlement mechanisms are addressed in this letter.

3.2.1 Immediate Settlement during Filling

This refers to the ‘immediate’ settlement experienced during the filling.

The controlling factors to this settlement are:

- The thickness of fill
- The stiffness (or Young’s Modulus).

We have estimated the during construction settlement at an 16 m deep ENGINEERED FILL area and its various components. The calculation considers 16 m of ENGINEERED FILL, see Inset 1. The estimated “immediate” settlement during filling is up to 170 mm. Depending on the rate of fill construction, the majority of this settlement is expected to occur during filling, with at most 30% of the immediate settlement occurring within 6 months after the completion of filling; i.e. that is settlement of up to 50 mm can be expected to occur between completion of filling and 6 months after the earthworks are completed.

Table 1 – “Immediate Settlement During Filling” Predictions

Fill Thickness (m)	Estimated “Immediate Settlement” (mm)	30% of Remaining Immediate Settlement to occur after the pad reaches BEL (mm)	Expected “Immediate” Settlement to complete after the pad reaches BEL
10	65	20	Up to 3 months
13	115	34	Up to 3 months
16	170	50	3 – 6 months

On this basis, we advise the following:

- OPTION 1 - Design of the structures within this fill pad will need to allow for the remaining immediate settlement of the fill

  Or

- OPTION 2 - Construction within the pad be delayed until the immediate settlement is completed.

  We expect this to be completed between 3 and 6 months.

  Surface settlement monitoring (eg. survey points) shall be installed at the completed surface to monitor the settlement of the pad for at least 3 months and up to six months to confirm that the immediate settlement is complete prior to construction of the slab. The surface settlement shall be monitored at lines across the pad with points installed at 20 m spacing.

  If this option (i.e. delaying the construction) is adopted, then the design of the structures would not need to account for this immediate settlement.

3.2.2 Post Construction Long term Settlement Predictions

This refers to the settlement that occurs from the end of the immediate settlement is often referred to as creep settlement. The characteristic of this settlement is that it slows down with time with the settlement between six months (0.5 year) and five years, being the same as the settlement between five years and 50 years.

The controlling factors to this creep settlement are:

- The thickness of fill
- The creep coefficient, which is in turn a function of thickness of the fill and the fill material. We have adopted a creep coefficient linearly increasing with fill thickness up to 0.3%/log cycles of time at 16 m thickness
- The timing of construction of the slab.

We have estimated the post construction settlement a slab constructed on a of a 10 m, and 16 m deep fill 6 months after completion of the earthworks from construction (6 months) to 50 years after construction (typical building design life), i.e. 2 log cycles of time and its various components. These are presented in Table 4.

The estimated creep settlement (over 50 years) is up to 168 mm.

Table 2 - Creep Settlement

Fill Thickness (m)	Estimated Creep settlement from 6 months to 50 years – 2 log cycle of time (mm)
10	38
13	63
16	96

On this basis, we advise the following:

- The design of the structures within this fill pad will need to allow for the above predicted post construction settlement (i.e. creep settlement)
- The designer (Civil and Structural) shall also consider the differential settlement resulting from the creep. Given that the existing natural landform is relatively flat, it is unlikely to be an issue. Based on the above and an original ground surface sloping at 15H:1V (see Inset 1) we would expect the differential settlements (tilts) to be in the range of 1:1000.

Should the post construction settlement be an issue for the proposed development, further geotechnical advice should be sought. Alternatives such as suspended slab system (for critical structures) or the use of crushed sandstone fill at the bottom of the fill to reduce the creep settlement at depth could be considered.

The magnitude of creep settlement will decrease with a delay between end of filling and start of construction. The above estimates allow for a 6 month delay. Should this time be less (say 3 months) the creep settlements may be up to 25% greater. Similarly a larger delay in construction results in smaller creep settlements being experienced by the structure.

3.2.3 Inundation/Collapse Settlement

At high stresses, wetting up of loosely placed fill can result in inundation/collapse settlements. The literature and our experience indicate that provided the fill is compacted to at least 98% SMDD the magnitude of collapse settlements is negligible.

It thus becomes critical that the material is placed in layers no thicker than 300 mm and compacted to above 98% SMDD throughout the layer. We have varied the requirements in the Specification in this area to increase the control of layer thickness and the degree of compaction of the fill in each layer.

3.3 Site Classification

While the proposed development (i.edata centre campus) is out of scope of AS2870-2011 "Residential slabs and footings", we assess that for natural clay soil placed in accordance with the Specification, the characteristic surface movement, y_s, would be in the range 40 mm to 60 mm and thus would classify the site as Class H1. The civil and structural engineers should consider likely heave / settlement due to the effect of climatic factors in their designs.

We recommend that all structures and services be detailed such that they preclude any local wetting up or drying out of the subgrade after initial equilibrium is reached following construction of the slab and that the subgrade be within specification at the time of construction of the slab. We note that normal mounding or sagging away from the perimeter of covered areas will still occur and perimeters, or open joints, will still respond to environmental changes.

For effectively sealed areas away from the perimeter, the design should allow for the following:

- Differential mound movement, y_m = 20 mm. We note that this is not the total heave or settlement but the estimated local heave or settlement due to FILL variability
- Tilts of up to approximately 1 in 400.

Mounds at perimeters or penetrations of slabs open to the environment can be taken to be as per AS2870-2011 for y_s = 55 mm.

3.4 Earthquake Site Classification

Earthquake loading and site classification is to be determined by the structural engineering designer in accordance with AS1170.4-2007. Based on the expected ground conditions following proposed bulk earthworks:

- Areas where the bedrock is within the upper 3.0 m of the bulk earthworks level, the sub-soil classification will be Class B_e (rock)
- Areas where the bedrock is below 3.0 m of the bulk earthworks level, the sub-soil classification will be Class C_e(shallow soil).

The Hazard Factor (Z) depends on the geographic location of the Site. The Hazard Factor for Sydney is 0.09 and is taken from Figure 3.2(A) of AS1170.4-2007.

3.5 Foundations

3.5.1 Pad Footings

Pad footings can be proportioned based on an allowable bearing pressure (ABP) for centric vertical loads provided in Table 3. Higher ABPs in soil units may be available, but these depend on the size, depth, loads, etc., and would be subject to specific advice. The ABP needs to be confirmed by a geotechnical engineer during an inspection.

Settlements in soil units can be estimated using the elastic parameters provided in Table 6. We note that allowable bearing pressures presented in assume a settlement of approximately 1% (or less) of the least footing dimension for footings in the BEDROCK units.

Table 3 – Engineering Parameters of Inferred Geotechnical Units

Inferred Unit	Bulk Unit Weight (kN/m^3)	Soil Effective Strength Parameters		Ultimate Bearing Pressure Under Vertical Centric Loading (kPa)	Allowable Bearing Pressure Under Vertical Centric Loading (kPa)	Ultimate Shaft Adhesion (kPa)	Elastic Parameters	
		c' (kPa)	*ϕ'* (deg)				Young's Modulus (MPa)	Poisson's Ratio
ENGINEERED FILL, NATURAL SOIL	18	0	30	420 (1)	150 (1)	N/A	10	0.3
BEDROCK A	22	N/A	N/A	3,000 (2)	700 (3)	50	100	0.25
BEDROCK B	24	N/A	N/A	6,000 (2)	1,500 (3)	350	350	0.25
BEDROCK C	24	N/A	N/A	30,000 (2)	6,000 (3)	600	1200	0.2

Note:

(1) Pad footings in soil unit should have a minimum horizontal dimension of 1.0 m and a minimum embedment depth of 0.5 m.

(2) Ultimate bearing pressure for bedrock assumes a settlement of approximately 5% of the least footing dimension for footings in rock.

(3) Allowable bearing pressure assumes a settlement of approximately 1% of the least footing dimension for footings in rock.

3.5.2 Piled Foundations

Piled foundation should be founded within the BEDROCK units.

Piles should be designed in accordance with the requirements in AS 2159 (2009), *Piling – Design and Installation.* The parameters provided in Table 3 may be adopted in the design of piles founded in the BEDROCK unit.

The foundation designer should note the following with regards to the pile design:

- The ABP needs to be confirmed by a geotechnical engineer during a pile inspection
- Under permanent load, the contribution of side adhesion for soils including soil units should be ignored
- Pile settlement needs to be checked using the recommended elastic parameters in Table 3.

The bearing capacities provided are contingent on piles or footings being vertically and centrally loaded. Further advice should be sought if the footings are not vertically centrically loaded. Should higher bearing capacities be required in Table 3 further advice should be sought from PSM.

With regards to the pile design, we recommend that:

- A geotechnical strength reduction factor, Φ_g = 0.60 (AS2159 CL. 4.3.2) be adopted for a high redundancy system for an assessed average risk rating (ARR) between 2.5 and 3.0. This should be reviewed to suit the specific design and appropriate pile testing proposed by the structural designers in accord with the requirements of AS 2159
- It may be possible to increase the pile reduction factors, if the details of the proposed pile installation procedures indicate a high level of quality control with regards to concrete placement, base cleanliness, etc
- If a geotechnical strength reduction factor, Φ_g = 0.40 is adopted then no pile testing will be required (AS2159 Clause 8.2.4 (b)).

Foundation conditions at the proposed pile locations should be inspected by a suitably qualified geotechnical engineer prior to pouring concrete to confirm this advice.

3.5.3 Slab on Ground

In general, we advise the slab on ground design can be based Young's moduli in Table 3. We note that the environmental effects (e.g. drying or wetting up of the finished surface) affecting the land prior to the development should be considered by the various designers of any development.

The Designers should also consider the fill settlements in deep fill area (See Section 3.2).

We note that the final bulk earthworks subgrade will require proof rolling and plate load testing to confirm the properties provided and may require some boxing out and refilling, etc. Plate load testing during the filling will be required where blended topsoil has been used.

We understand that the structural engineer should be able to design efficient slabs. If assessed deformation and settlement are an issue, our advice can be further refined if required.

The structural designer or builder may wish to employ a surface layer of road base/crushed sandstone/concrete for trafficability or structural purposes. This is not required to achieve the properties in this design advice.

3.6 Pavements

A total of three (3) CBR tests were undertaken. The test results indicate a soaked CBR value of between 1.5% and 2.5%.

Based on our experience with typical clay VENM in western Sydney, the soaked CBR value could be as low as 1%. We advise that a CBR of 2% can be adopted for subgrade and FILL formed in bulk earthworks constructed in accordance with the Specification.

Higher values may be provided on completion of testing on the finished bulk earthworks or if, on request, the Specification is varied to obtain such higher values on ENGINEERED FILL.

We recommend that specific CBR testing be undertaken at the subgrade level when pavement layouts are finalised.

3.7 Permanent and Temporary Batters

The batter slope angles in Table 4 are recommended for the design of batters up to a nominal 3 m height**,** subject to the following recommendations:

1. All batters shall be protected from erosion.
2. Permanent batters shall be drained.
3. Temporary batters shall not be left unsupported for more than 1 month without further advice, and inspection by a geotechnical engineer should be undertaken following significant rain events.
4. Where loads are imposed or structures/services are located within one batter height of the crest of the batter, further advice should be sought.

If the conditions above cannot be met, further advice should be sought.

Steeper batters may be possible, subject to further advice. This could include the requirement for soil nails or rock bolts. The length and spacing of soil nails and rock bolts are a matter of design.

The batters should be inspected by an experienced geotechnical engineer or engineering geologist during excavation to confirm the batter advice provided and assess the need for localised support.

Proper and suitable safe work method statements and OHS documents need to be developed for works to be undertaken in the vicinity of the crest and toe of batters.

Table 4 – Batter Slope Angles

Unit	Temporary	Permanent
ENGINEERED FILL	1.5H : 1V	2.0H : 1V
NATURAL SOIL	1.5H : 1V	2.0H : 1V
BEDROCK A	1H : 1V*	1.5H : 1V*
BEDROCK B and C	0.5H : 1V (subject to local support)	1H : 1V (subject to design)

Note:
* See above requirements regarding inspections and local support.

3.8 Retention Systems

Permanent cuts in the NATURAL SOIL and BEDROCK unit's steeper than the recommended permanent batter slopes in Table **4** will need to be supported by some form of retaining structure.

The design of retaining structures should be based on the following:

- Effective soil strength parameters in Table 3
- Water pressure (depending on the type of the structure)
- With regards to the BEDROCK units, the designer shall allow a minimum lateral pressure of 10 kPa for the BEDROCK units when cut vertical. This is to allow for blocks and rock wedges formed due to adverse defects that may exist within the unit. These loads may be able to be reduced by specifying inspections during the works and provision of additional support (rock bolts, shotcrete etc.) should the inspection indicate that support is required. In any case excavation in BEDROCK units will need to be inspected during the works to confirm/dismiss the presence of defects/structure in the unit that may result in higher loads than anticipated in this design. The designer of the wall should consider including inspection requirements in their design at no more than 2 m intervals in the excavation.

Note that design of retention systems may be based on either K_a or K_p earth pressures. Design using active earth pressures provides the minimum lateral earth pressure that must be supported to avoid failure and requires wall that can rotate or translate to allow the pressures to reduce to these values (vertical and lateral movements up to 2% of height may occur, typical movements will be much less).

Where the design is based on K_o pressures, construction should be carefully controlled to avoid unwanted effects. It should be noted that designing for K_o pressures does not, of itself, ensure that movement does not occur. Movements are controlled by the construction method, especially sequence.

Both surface and sub-surface drainage needs to be designed and constructed properly to prevent pore water pressures from building up behind the retaining walls or appropriate water pressures must be included in the design.

Yours Sincerely

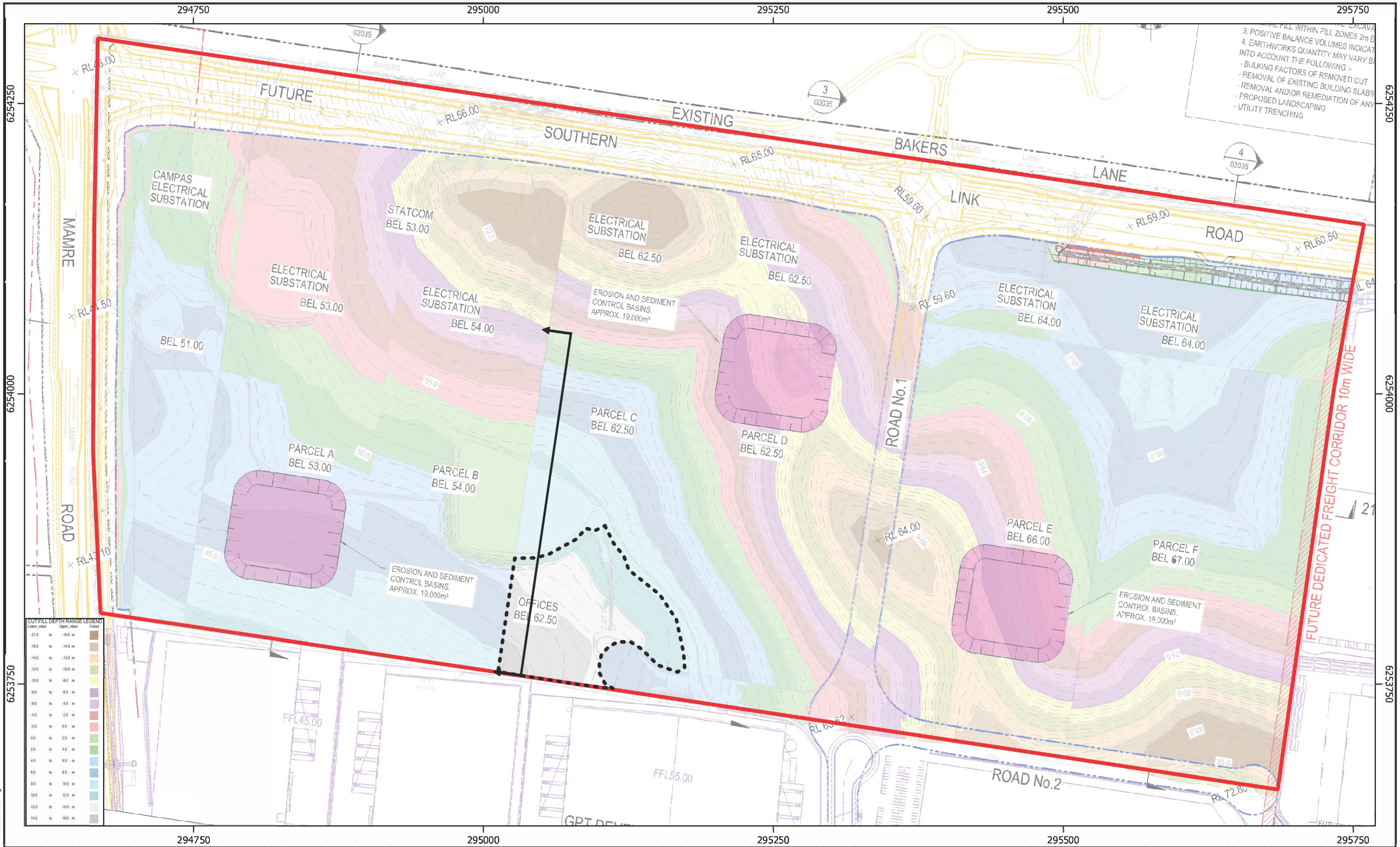
BRENDAN TA
GEOTECHNICAL ENGINEER

JUNO LIANG
ASSOCIATE GEOTECHNICAL ENGINEER

AGUSTRIA SALIM
PRINCIPAL

Encl.

Figure 1 Site Locality Plan



Legend

Site Boundary

Section Line

Area comprising deep FILL (>10m)

Notes:
1. Bulk earthworks cut-fill plan by AT&L (ref. SYD4-SITE-DRG-ATL-CIV-02031, dated 10 November 2025).
2. Indicative Section Line for Inset 1.

N

Scale 1:3,000

0 20 40 60 80 100 m

EPSG:7856
GDA2020 / MGA zone 56

PSM

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706-752 Mamre Road, Kemps Creek
Interim Geotechnical Design Advice

SITE LOCALITY PLAN

PSM5872-006L

Figure 1

M:\PSM5872\Eng\QGIS\QGIS\PSM5872.qgz Layout: PSM5872-006L Figure 1