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REPORT ON GEOTECHNICAL SITE INVESTIGATION

for

PROPOSED DEVELOPMENT

at

50 LAWRENCE STREET,
FRESHWATER NSW

Prepared For

Lawrence Street Nominees Pty Ltd

Project No.: 2020-050
March, 2021

Document Revision Record

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0	15th April 2020	Original issue
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Date: 22nd March 2021

Project No: 2020-050

GEOTECHNICAL REPORT FOR CONSTRUCTION OF A NEW THREE STOREY BUILDING WITH UNDERGROUND BASEMENT CARPARK
50 LAWRENCE STREET, FRESHWATER, NSW

1. INTRODUCTION:

This report details the results of a geotechnical investigation carried out for the construction of a new three storey mixed use building with underground basement level carpark at 50 Lawrence Street, Freshwater, NSW. The investigation was undertaken by Crozier Geotechnical Consultants (CGC) at the request of Life Property Group on behalf of client Lawrence Street Nominees Pty Ltd.

With reference to Northern Beaches (Warringah) Councilôs 2011 LEP and DCP states that all building development applications must be accompanied by a geotechnical landslip assessment. That developments within Class ôAô, ôBô and ôDô landslip risk zone may require a preliminary assessment only where excavation/fill is <2.0m depth, however Class ôCô and ôEô require a geotechnical report.

This site is located within landslip risk Class ôBô within the Geotechnical Risk Management Map (Warringah Council Local Environmental Plan, Landslip Risk Map LSR_010). A review of the preliminary checklist and the proposed works identified that the Development Application (DA) involves works which exceed the preliminary assessment guidelines. Therefore, it was required to complete a geotechnical report in support of the DA. Access to the northern end of the site is extremely limited therefore sub-surface investigation was undertaken within the rear southern end of the site only.

This report forms part of a geotechnical investigation requested by the client to provide information for the structural design and construction works in addition to fulfilling DA submission requirements.

The report therefore includes a description of the site and sub-surface conditions, a geotechnical assessment of the development, site mapping/plan, geological sections and provides recommendations for construction.

The site assessment and reporting were undertaken as per the Tender P19-496, Dated: 13th November 2019.

The investigation comprised:

a) A detailed geotechnical inspection and mapping of the entire site and limited inspection of adjacent land, with identification of geotechnical conditions including potential hazards related to the existing site and proposed structures, by a Geotechnical Engineer including a photographic record of site conditions,
b) DBYD plan request and onsite clearing of test locations by an accredited service location contractor.
c) Drilling of three augered boreholes with a restricted access rig at the southern portion of the site along with Dynamic Cone Penetrometer (DCP) testing to investigate the subsurface geology and depth to bedrock.
d) All fieldwork was conducted under the full-time supervision of an experienced Geotechnical Professional who completed logging of soils and ensured the quality of all geotechnical data.

The following plans and drawings were supplied for the work:

- Draft Architectural Drawings ï CKDS Architecture ï Project No.: 19045, Drawing No.: DA-1001, DA-1002, DA-1101, DA-1102, DA-1106, DA-2001, DA-2002, DA-3001 and DA-3002; Issue: D and Dated: 2/3/21.
- Survey Drawing ï Bryne & Associates Pty Ltd ï Plan No.: A1 ï 11008D, Dated: 16/03/2018, Date of Survey: 14/03/2019.

2. PROPOSED DEVELOPMENT:

It is understood that the proposed works involve the demolition of all site structures and construction of a new three storey mixed use building with a basement (Dowling Street access), including a ground floor (Oliver Street access) level carpark. The building will have a combination of apartments and retail/business spaces with a lift shaft located in the centre of the structure for access to residential apartments.

The proposed development will extend to the siteôs northern, western and eastern boundaries and extend to 3.0m off the southern boundary. Major excavation will take place within the southern portion of the site. Excavation down to 4.5m depth will be required at the south-west portion of the site, decreasing to Ò0.5m depth in the northern portion of the site.

3. SITE FEATURES:

3.1. Description:

The site is a near rectangular shaped block located on the high south side of Lawrence Street, on the low east side of Oliver Street and on the high west side of Dowling Street. It has a front north boundary of 10.365m, a rear south boundary of 14.965m, an east boundary of 45.72m, a west boundary of 43.285m and a diagonal north-west boundary of 3.45m with a total area of 590m² as referenced from the supplied survey drawings.

The site is generally within gently (Ò5˚) north dipping topography. The southern portion of the site comprises a raised, gently north dipping ground surface and drops approximately Ò1.5m within the centre of the site via a vertical, west-east striking bedrock cliff face. The northern portion of the site, where the primary structure of the site is situated is gently sloping to the north east. Ground surface levels within the site reduce from a high of approximately RL 32.52m adjacent to No.30 Oliver Street to a low of approximately RL 27.69m at the north east corner of the site.

An aerial photograph of the site and its surrounds is provided below, as sourced from NSW Government Six Map spatial data system, as Photograph 1. General views of the site at the time of investigation are provided in Photograph-2 and Photograph-3

Photograph: 1 – Aerial photo of site and surrounds.

Photograph-2: Front view of the site within the road reserve adjacent to Lawrence Street and Dowling Street intersection. View looking west.

Photograph-3: Rear of the site from the south west corner of the site. View looking north.

3.2. Geology:

Reference to the Sydney 1: 100,000 Geological Series sheet (9130) indicates that the site is underlain by Hawkesbury Sandstone (Rh) which is of Triassic Age. The rock unit typically comprises medium to coarse grained quartz sandstone with minor lenses of shale and laminite.

Morphological features often associated with the weathering of Hawkesbury Sandstone are the formation of near flat ridge tops with steep angular side slopes that consist of sandstone terraces and cliffs in part covered with sandy colluvium. The terraced areas often contain thin sandy clay to clayey sand residual soil profiles with intervening rock (ledge) outcrops. The outline of the cliff areas are often rectilinear in plan view, controlled by large bed thickness and wide spaced near vertical joint patterns. The dominant defect orientations are south-east and north-east. Many cliff areas are undercut by differential weathering along sub-horizontal to gently west dipping bedding defects or weaker sandstone/siltstone/shale horizons. Slopes are often steep (15º to 23°) and are randomly covered by sandstone boulders.

Extract of Sydney: 1:100 000 – Geology underlying the site

4. FIELD WORK:

4.1. Methods:

The field investigation comprised a walk over inspection and mapping of the site and adjacent properties on the 17th March 2020 by a Geotechnical Engineer. It included a photographic record of site conditions as well as geological/geomorphological mapping of the site and adjacent land with examination of the low cliff outcrop within the centre of the site, existing structures and limited inspection of neighbouring properties.

It also included the drilling of three boreholes (BH1 to BH3) using a restricted access drill rig operating solid stem, spiral flight augers in conjunction with a tungsten carbide bit within an existing carpark at the southern portion of the site to investigate sub-surface geology.

DCP testing was carried out from ground surface adjacent to and within the boreholes in accordance with AS1289.6.3.2 ï 1997, ñDetermination of the penetration resistance of a soil ï 9kg dynamic cone penetrometerò to estimate near surface soil conditions and depth to bedrock.

Explanatory notes are included in Appendix: 1. Mapping information and test locations are shown on Figure: 1, along with detailed bore log and DCP sheets in Appendix: 2. A Geological models/section is provided as Figure: 2 and Figure:3, Appendix: 2.

4.2. Field Observations:

Oliver Street is on the high west side of the site, it is formed with a gently (Ò5˚) north dipping bitumen pavement, with low concrete gutter and kerbs adjacent to strip garden beds, power poles, a concrete pathway and significant bedrock outcrop. The bedrock outcrop comprises sandstone with embedded rounded and sub rounded clasts. The sandstone was classified as slightly to moderately weathered and comprised gently west dipping, cross bedding defects (Photograph-5). Significant cracking and undulations were not observed within the road pavement and concrete footpath and they appear in good condition. More sandstone bedrock outcrop comprising similar competency and at slightly higher ground level was observed to the west of Oliver Street (Photograph-6).

Photograph-5: Sandstone bedrock to the east of Oliver Street. View looking north.

Photograph-6: Sandstone bedrock to the west of Oliver Street. View looking west.

Lawrence Street is on the low north side of the site, it is formed with a gently (Ò6˚) east dipping bitumen pavement, with low concrete gutters and kerbs adjacent to a tiled footpath that curved south (Ò5m) to the Dowling Street road reserve.

Dowling Street is on the low east side of the site, it is formed with a gently (Ò5˚) north dipping bitumen pavement. Itôs road reserve contains the continued tiled footpath, a concrete footpath adjacent to strip garden beds, continued south by a wide garden bed, raised planter beds and bedrock outcrop at the base of the siteôs eastern boundary/retaining wall (Photograph-7). The sandstone outcrop comprises similar consistency as the previously detailed bedrock outcrop. The road reserve also comprises a concrete driveway to the south, which grants vehicular access from Dowling Street to an old rendered brick garage located at the south east corner of the site (Photograph-8 and Photograph-9). Significant cracking and undulations were not observed within the road pavements and road reserves in Lawrence and Dowling Street, similarly the siteôs boundary/retaining wall and concrete driveway within the Dowling Street road reserve appear in good condition. However moderate cracking was observed within the lower portion of the garage façade (Photograph-9), from experience, it appears to be construction/durability related.

Photograph-7: Dowling Street road reserve. View looking north.

Photograph-8: Dowling Street road reserve, concrete driveway. View looking north.

Photograph-9: Rendered brick garage defect. View looking west.

Access to the site was gained from the south west portion of the site, the entrance contains a concrete driveway (that extends east approximately 10.0m) which grants vehicular access from Oliver Street to the southern portion of the site. The southern portion of the site comprises a gravel carpark within the west, a slightly lower (Ò0.5m) grassed lawn within the east (currently used as a carpark extension and is retained by a 0.60m high brick retaining wall to the north and bounded/supported by the brick boundary/retaining wall along the east) and the rendered brick garage at the south east corner of the site. Significant undulations or geotechnical related issues were not observed within the southern portion of the site. The rendered brick garage contains moderate cracking within its external walls and a tree stump adjacent to the base of its west facing wall (Photograph-10). However, these appear to be construction/durability related.

Photograph-10: Rendered brick garage. View looking east

Bedrock outcrop was observed within the southern end of the site (Photograph-11). Similarly, the low (Ò1.50m) bedrock cutting cliff face (Photograph-12) observed to the north (within the centre east portion of the site) is characterised as fine to medium grained, low to medium strength and contains gently north dipping cross bedding defects within the crest along with minor seepage between the soil and bedrock interface. Trace of river-based pebbles were also observed embedded within the sandstone matrix.

Photograph-11: Bedrock outcrop within the southern portion of the site. View looking east.

Photograph-12: Low bedrock cutting cliff face. View looking east.

The northern portion of the site comprises a two-storey brick building of residential and commercial use formed at a lower ground level than the south of the site (approximately 2.0m). The ground floor contains a retail shop with a front tiled façade facing onto Lawrence Street. A concrete pathway surrounds the building along the south and west side. Adjacent to the pathway, along the west, is a brick retaining wall of up to 1.7m height that supports the Oliver Street road reserve above the site (Photograph-13). The outer walls of the building, pathways and retaining wall appear in good condition and did not show any obvious signs of cracks or settlement or underlying any geotechnical issues.

Photograph-13: Brick retaining wall to the west of concrete pathway. View looking north.

Photograph-14: Building outer wall, facing onto Dowling Street. View looking west.

The property to the south (No. 30 Dowling Street), contains a two-storey brick residence with a lawn in the northern portion of the property, a garage below the front east side of the building and a concrete driveway that allows vehicular access from Dowling Street to the property. The house extends north to within 9.50m of the common boundary and it is formed at a higher ground level to the southern portion of the site. A limited inspection of the neighbouring property did not identify any obvious signs of cracks or settlement in the structure, only minor cracks were observed within the concrete driveway (Photograph-15). Bedrock outcrop of similar competency was observed at the base of the boundary wall within the road reserve (Photorgaph-16).

Photograph-15: Cracks within the concrete driveway. View looking west.

Photograph-16: Bedrock outcrop at the base of the boundary wall. View looking west.

Information obtained from available NSW Government SIX Maps identified that Oliver Street between Cavill Street and Lawrence Street, the site and the neighbouring property to the south were undeveloped in 1943.

4.3. Field Testing:

The boreholes (BH1 to BH3) were drilled within the southern portion of the site, using a limited access drill rig with refusal encountered at varying depths from 0.40m (BH1) to 1.20m (BH3) on sandstone bedrock.

Dynamic Cone Penetrometer (DCP) tests were carried out from the ground surface adjacent to the boreholes with refusal at depths varying from 0.25m(DCP1) to 0.85m(DCP3) on interpreted extremely/highly weathered bedrock.

For a detailed description of the ground conditions encountered at each investigation location, the borehole log sheets should be consulted. However, based on the borehole logs and DCP test results, the sub-surface conditions at the project site can be classified as follows:

- **FILL –** this layer was encountered at all boreholes to a maximum of 0.50m depth below the ground surface. It was classified as loose to medium dense, dark brown, fine to medium grained, moist, silty sand with fine to coarse gravel.
- **SAND –** this layer was only encountered in BH3 from 0.50m to 0.90m depth. It was classified as medium dense, pale orange, fine to medium grained, moist, sand with trace of gravel.
- **SANDSTONE –** this unit was encountered underlying the fill within BH1 and BH3 at varying depths from 0.20m (BH1) to 1.20m (BH3). It was classified as fine to medium grained, pale grey mottled orange, extremely low to very low strength sandstone and is considered to be bedrock.

Auger refusal was encountered on sandstone bedrock of at least low strength at varying depths of 0.40m (BH1) to 1.20m (BH3). Based on the geological mapping of the bedrock outcrop over the low cliff face within the centre of the site, it is classified as low grading quickly to medium strength. Seepage was observed within the cliff face at the centre of the site, between the overlying soil and bedrock interface.

5. COMMENTS:

5.1. Geotechnical Assessment:

The site investigation identified the presence of fill to a maximum depth of 0.50m (BH3), which generally comprised silty sand with some fine gravels, underlain by a discontinuous sand layer (up to 0.40m thick) in BH3 only, underlain by a thin layer (Ò0.30m) of extremely weathered sandstone bedrock at varying depths between 0.20m (BH1) to 0.90m (BH3). Underlaying the extremely weathered sandstone and likely underlying the lower northern portion of the site is sandstone bedrock of at least low strength. It appears to be gently north dipping and it is exposed at numerous locations outside and within the site.

The exposed low cliff-face within the centre of the site contained gently north dipping bedding defects within the crest along with massive sandstone within the lower portion. It appeared vertically stable, slightly to moderately weathered with no unfavourable defects or indications of previous failures/instability, at least since the construction of the development. However moderate seepage is expected within the soil rock interface and on rock defects.

It is anticipated that during excavation, up to 1.20m of fill/sand/extremely weathered sandstone will be exposed within the south-west portion of the site and up to 0.35m of fill within the south-east portion of the site, under which sandstone bedrock of at least low strength (likely increasing to medium strength with depth) will be encountered. Investigation within the northern portion of the site was not possible due to access limitations of the existing building, however based on the field investigation (including exposed outcrops), it is anticipated that relatively minor (Ò0.5m depth) excavation will extend through sandstone bedrock. However, this needs to be confirmed directly following demolition of the structure and ground floor slabs.

Due to the extension of the proposed excavation to the siteôs western, eastern and northern boundaries and the likely underlying sand/extremely weathered material within the south-west portion of the site (BH3), support prior to excavation may be required along the western boundary (particularly within the southern portion). This may be necessary to protect the boundary, the underlying soil and services from collapsing into the excavation. Recommended retaining structure parameters are provided in Section 5.3.3. The depth of soil at each boundary can be confirmed at initial site works, potentially allowing variations to support needs.

At the south and east boundaries, where medium to high strength sandstone with no poorly oriented defects is encountered, it will be free standing and can be excavated near vertically without the need for support measures. Where defects are encountered these may be supported during the works (i.e. rock bolts). However, should highly fractured rock mass be encountered then there will be a need for support prior to further excavation. Confirmation of rock strength/conditions prior to excavation would require cored boreholes, drilled in the location of the deep excavation (particularly adjacent to east boundary) to confirm the subsurface conditions (i.e. if any weak zones of rock are identified) prior to final structural design. Where this is not undertaken then regular detail geotechnical inspection during excavation (every 1.5m depth interval) works will be required with potential stop/hold points to allow support installation prior to proceeding.

It is understood that part of the works is to match the existing retaining wall to the west, to the proposed structure. Therefore, to ensure a good service lifetime, as well as protecting the wall from collapse/damage and subsequently impacting the supported road reserve and people. This wall should be assessed by a structural engineer. Similarly, a geotechnical consultant professional should be contacted to inspection the founding strata and footing. This inspection can be done via test pits, to ensure competent founding material. This works should be undertaken prior commencing any excavation works in the northern portion of the site.

Where excavation reduces to Ò0.50m below the road reserve floor level and the road reserve subsurface comprises stable material (subject to geotechnical consultant inspection), it is possible that support prior to excavation would not be required. This might be possible along the north boundary of the site where excavation is expected to have a maximum depth of 0.50m.

Similarly, along the northern portion of the western and eastern boundaries where excavation reduces to Ò0.50m below the ground surface of the road reserve, support prior excavation may not be required. However, this will require inspection by a geotechnical consultant professional during works.

The development is proposed over an excavation that is expected to expose at least low strength sandstone bedrock in its entire base. Appropriate allowable bearing capacities for the range of rock conditions anticipated are provided in Section 5.3.1. Confirmation as well as obtaining higher bearing capacities can be achieved by core drilling investigation.

Medium to high strength sandstone bedrock will require the use of the rock breaking equipment for excavation. This equipment has the potential to create significant ground vibrations which could damage neighbouring structures therefore the selection of suitable excavation equipment is an important factor. Small scale equipment could be utilized to maintain low suitable vibration levels however due to the likely medium to high strength bedrock this will result in significantly slow excavation progress. The geotechnical engineer should be consulted regarding the size and type of equipment proposed for use with an excavation methodology including vibration monitoring requirements determined prior to bulk excavation based on the equipment proposed.

A review of the DBYD service plans obtained from Ausgrid, indicated that a transmission line and power cables are present under the surrounding road reserve. The transmission line (carrying 33KV) runs south-north underlying the Oliver Street road reserve at distances ranging approximately between 2.5m to 4.6m of the western boundary (approximately 0.80m cover), whilst underlying cables (generally low voltage, 230/400 volts) surround the boundaries at the northern portion of the site and connect to a distribution pillar located at the north-east side of the siteœs boundary. It is possible that other cables owned by other utilities (other than Ausgrid) be present underlying the surrounding road reserve. It is extremely important to correctly locate all underground services and contact service provides to confirm their requirements regarding design, construction, including vibration limits, to prevent delays in approvals. Extreme care must be exercised when doing works in these areas, particularly within the south-west portion of the site.

The recommendations and conclusions in this report are based on an investigation utilising only surface observations and isolated. This test equipment provides limited data from small isolated test points across the entire site. Therefore, some minor variation to the interpreted sub-surface conditions is possible, especially between test locations. However, the results of the investigation provide a reasonable basis for the Development Application analysis and subsequent preliminary design of the proposed works.

5.2. Site Specific Risk Assessment:

Based on our site investigation and review of the proposed works we have identified the following credible geological/geotechnical hazards which need to be considered in relation to the existing site and the proposed works. The hazards are:

A. Landslip (earth slide <1m^3) from soils due to the basement excavation of the site.
B. Landslip (rockslide/topple <1m^3) from rocks due to the basement excavation of the site.

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

Hazard A was estimated to have a **Risk to Life** of **3.10×10^{-7}** for a single person, while the **Risk to Property** was considered to be **'Moderate'**.

Hazard B was estimated to have a **Risk to Life** of up to **1.55×10^{-7}** for a single person, while the **Risk to Property** was considered to be **'Moderate'**.

Although the ôModerateö Risk to Property for Hazard A and B is considered to be ôUnacceptableö, the assessments were based on excavations with no support or planning. Provided the recommendations of this report are implemented including geotechnical inspection and installation of engineered support as required the likelihood of any failure becomes ôRareö and as such the consequences reduce and risk reduces further and is within óAcceptableô levels when assessed against the criteria of the AGS. As such the project is considered suitable for the site provided the recommendations of this report are implemented.

5.3. Design & Construction Recommendations:

Design and the construction recommendations are tabulated below:

5.3.1. New Footings:	
Site Classification as per AS2870 ï 2011 for new footing design	Class óAô for footings on bedrock
Type of Footing	Strip/Pad or Slab at base of excavation, piers external to the excavation if required to achieve uniform bearing
Sub-grade material and Maximum Allowable Bearing Capacity for Footing Design	- Very Low Strength Sandstone: 800kPa - Low Strength Sandstone: 1000kPa - Medium Strength Sandstone: 2000kPa*
Site sub-soil classification as per *Structural design actions AS1170.4 – 2007, Part 4: Earthquake actions in Australia*	B_e ï Rock Site
Remarks: * Higher bearing pressures available through core drilling of bedrock All new footings must be inspected by an experienced geotechnical professional before concrete or steel are placed to verify the preliminary maximum bearing capacities provided above and the in-situ nature of the founding strata. This is mandatory to allow them to be ócertifiedô at the end of the project. Individual structures should not be founded on materials with varying bearing and settlement characteristics unless the potential for differential movement has been allowed for in structural design.	

5.3.2. Excavation:	
Depth of Excavation	Up to approximately 4.50m depth within the southern portion, reducing to 0.50m depth at the north-east corner of the site.
Distance of excavation to Neighbouring Properties and Structures.	- No. 30 Lawrence Street ï 3.0m from the southern boundary, building another 9.50m - Oliver Street Road Reserve ï On western boundary, edge of bitumen pavement another 5.50m, retaining wall supporting the road reserve another 1.50m. - Dowling Street Road Reserve ï On eastern boundary, edge of bitumen pavement another 4.00m - Lawrence Street Road Reserve ï On northern boundary, edge of bitumen pavement another 4.00m

Type of Material to be Excavated	Topsoil/fill Ò0.50m
	Sand Ò0.90m
	Sandstone Bedrock ELS VLS Ò1.20m
	Sandstone Bedrock LS to MS from Ò0.40m depth through to the base of the excavation.

ELS= Extremely low strength, VLS = Very low strength, LS= Low strength, MS = Medium strength

Guidelines for batter slopes for this site are tabulated below:

Material	**Safe Batter Slope (H:V)**	
	Short Term/ Temporary	**Long Term/ Permanent**
Fill & Silty Sand natural granular soils	1.5:1	2:1
Very Low strength bedrock or fractured bedrock	0.75:1*	1.25:1*
Low to Medium strength, defect free bedrock	Vertical*	Vertical*

*Dependent on defects and assessment by engineering geologist

Remarks:

Seepage at the bedrock surface or along defects in the rock can also reduce the stability of batter slopes and invoke the need to implement additional support measures.

Where safe batter slopes are not implemented the stability of the excavation cannot be guaranteed until the installation of permanent support measures. This should also be considered with respect to safe working conditions.

Equipment for Excavation	Fill/Sand	Excavator with bucket
	VLS bedrock	Excavator with bucket and ripper
	LS-MS bedrock	Rock hammer and saw

Remarks:

Based on previous testing of ground vibrations created by various rock excavation equipment within medium strength bedrock, to maintain a vibration level below 5mm/s PPV the below hammer weights and buffer distances are required:

Buffer Distance from Structure	Maximum Hammer Weight
2.0m	200kg
4.0m	500kg
5.0m	800kg
8.0m	1000kg

Onsite calibration will provide accurate vibration levels to the site specific conditions and will generally allow for larger excavation machinery or smaller buffers to be used. Calibration of rock excavation

<table>
<tr><td colspan="2">machinery should be carried out prior to commencement of rock excavation works, where Ó250kg rock hammers are proposed for use.
Rock sawing of the excavation perimeter is recommended as it has several advantages. It often reduces the need for rock bolting as the cut faces generally remain more stable and require a lower level of rock support than hammer cut excavations, ground vibrations from rock saws are minimal, the saw cuts will provide a slight increase in buffer distance for use of rock hammers whilst also reducing deflection of rock across boundaries.
The strength of bedrock below the maximum depth achieved during the investigation is unconfirmed and would require cored boreholes using specialist restricted access drilling equipment.
An excavator with bucket will not create excessive vibrations provided it is undertaken with medium scale (<20 tonne excavator) excavation equipment in a sensible manner.</td></tr>
<tr><td>Recommended Vibration Limits (Maximum Peak Particle Velocity (PPV))</td><td>No. 30 Oliver Street= 5mm/s
Services adjacent to the siteôs boundaries = 3mm/s</td></tr>
<tr><td>Vibration Calibration Tests Required</td><td>Yes, recommended for any rock hammer >250kg weight</td></tr>
<tr><td>Full time vibration Monitoring Required</td><td>Pending proposed equipment and vibration calibration testing results</td></tr>
<tr><td>Geotechnical Inspection Requirement</td><td>Yes, recommended that these inspections be undertaken as per below mentioned sequence:
• Following removal of site soils and exposure of bedrock
• During installation of boundary supporting systems
• At 1.50m depth intervals within bedrock excavation
• At completion of the excavation.</td></tr>
<tr><td>Dilapidation Surveys Requirement</td><td>Recommended on neighbouring structures or parts thereof within 10m of the excavation perimeter prior to site work to allow assessment of the recommended vibration limit and protect the client against spurious claims of damage.</td></tr>
<tr><td colspan="2">Remarks:
Water ingress into exposed excavations can result in erosion and stability concerns in both soil and rock portions. Drainage measures will need to be in place during excavation works to divert any surface flow away from the excavation crest and any batter slope, whilst any groundwater seepage must be controlled within the excavation and prevented from ponding or saturating slopes/batters.</td></tr>
</table>

5.3.3. Retaining Structures:	
Required	New retaining structures appear required at the northern, western and eastern boundaries, as part of the proposed development.
Types	Steel reinforced concrete/concrete block wall post excavation where safe temporary batters can be formed or soldier bored pile wall where support prior to excavation is required.
Parameters for calculating un-surcharged pressures on retaining walls for the materials likely to be retained:	

Material	Unit Weight (kN/m^3)	Long Term (Drained)	Earth Pressure Coefficients		Passive Earth Pressure Coefficient *
			Active (Ka)	At Rest (K_0)	
Sandy Fill/Loose Sand	18	$\phi' = 28°$	0.35	0.52	N/A
LS bedrock (fractured)	23	$\phi' = 40°$	0.10	0.15	300kPa
MS bedrock (defect free)	24	$\phi' = 40°$	0.00	0.01	600kPa

Remarks:

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

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

5.3.4. Drainage and Hydrogeology		
Groundwater Table or Seepage identified in Investigation		No
Excavation likely to intersect	Water Table	No
	Seepage	Minor (<1L/min), on defects and at soil/rock interface
Site Location and Topography		High south side of Lawrence Street within gently north dipping topography.
Impact of development on local hydrogeology		Negligible

Onsite Stormwater Disposal	Not required or recommended.
Remarks: As the excavation faces are expected to encounter some seepage, an excavation trench should be installed at the base of excavation cuts to below floor slab levels to reduce the risk of resulting dampness issues. Trenches, as well as all new building gutters, down pipes and stormwater intercept trenches should be connected to a stormwater system designed by a Hydraulic Engineer which discharges to the Councilôs stormwater system off site.	

5.4. Conditions Relating to Design and Construction Monitoring:

To allow certification at the completion of the project it will be necessary for Crozier Geotechnical Consultants to:

1. Review and approve the structural design drawings, including the retaining structure design and construction methodology, for compliance with the recommendations of this report prior to construction,
2. Inspection/investigation in the north end of the site, following removal of existing building to confirm sub-surface geological conditions,
3. Inspect any medium strength bedrock and the proposed excavation equipment prior to its excavation,
4. Inspect all new footings to confirm compliance to design assumptions with respect to allowable bearing pressure, basal cleanness and stability prior to the placement of steel or concrete,
5. Inspect completed works to ensure no new landslip hazards have been created by site works and that all required stabilisation and drainage measures are in place.

Crozier Geotechnical Consultants cannot provide certification for the Occupation Certificate if it has not been called to site to undertake the required inspections.

6. CONCLUSION:

The site investigation indicated the presence of fill (Ò0.50m) underlain by a sand layer (Ò0.40m thick), overlying extremely sandstone bedrock, quickly grading to sandstone bedrock of at least low strength at depths between 0.35m (BH2) to 1.20m (BH3) below the existing ground surface. The bedrock is expected to grade to medium strength at shallow depth. However, this will require confirmation at the north end of the site, following demolition of the structure and existing ground floor slabs.

The proposed works will require an excavation up to 4.5m depth within the southern portion of the site decreasing to 0.50m depth within the north-east corner of the site. The excavation is expected to mainly intersect sandstone bedrock. As such a crucial part of the works will be to ensure ground vibrations produced by the rock excavation equipment do not damage the neighbouring properties (including nearby services). The geotechnical engineer should approve the proposed excavation equipment and methodologies.

Based on the extension of the excavation to the siteǿs boundaries and the depth of soils identified within the southern portion, support prior to excavation may be required along the west boundary (particularly the south-west corner of the proposed new basement) of the site. However, it might not be required along the south, east and north boundaries of the site. This should be confirmed by geotechnical inspection following demolition and prior to bulk excavation.

The risks associated with the proposed development can be maintained within óAcceptableô levels with negligible impact to neighbouring properties or structures provided the recommendations of this report and any future geotechnical directive are implemented. As such the site is considered suitable for the proposed construction works provided that the recommendations outlined in this report are followed.

Prepared By:	Reviewed By:
Marvin Lujan	Troy Crozier
Geotechnical Engineer	Principal
	MAIG, RPGeo ï Geotechnical and Engineering
	Registration No.: 10197

7. REFERENCES:

1. Australian Geomechanics Society 2007, ñLandslide Risk Assessment and Managementò, Australian Geomechanics Journal Vol. 42, No 1, March 2007.
2. Geological Society Engineering Group Working Party 1972, ñThe preparation of maps and plans in terms of engineering geologyò Quarterly Journal Engineering Geology, Volume 5, Pages 295 - 382.
3. E. Hoek & J.W. Bray 1981, ñRock Slope Engineeringò By The Institution of Mining and Metallurgy, London.
4. C. W. Fetter 1995, ñApplied Hydrologyò by Prentice Hall.
5. V. Gardiner & R. Dackombe 1983, ñGeomorphological Field Manualò by George Allen & Unwin.
6. Australian Standard AS1170.4 ï 2007, Part 4: Earthquake actions in Australia

Appendix 1

CROZIER GEOTECHNICAL CONSULTANTS

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Unit 12/ 42-46 Wattle Road Phone: (02) 9939 1882
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Crozier Geotechnical Consultants, a division of PJC Geo-Engineering Pty Ltd

NOTES RELATING TO THIS REPORT

Introduction

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

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

Description and classification Methods

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

Standard Penetration Tests

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

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

The test results are reported in the following form.

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

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

Cone Penetrometer Testing and Interpretation

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

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

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

The information provided on the plotted results comprises: -

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

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

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

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

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

Q_c = (12 to 18) C_u

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

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

Dynamic Penetrometers

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

Two relatively similar tests are used.

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

Laboratory Testing

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

Borehole Logs

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

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

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

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

Ground Water

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

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

Engineering Reports

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

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

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

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

Site Anomalies

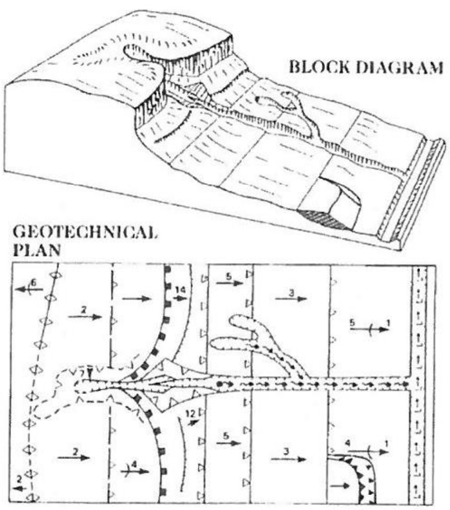
In the event that conditions encountered on site during construction appear to vary from those which were expected from the information contained in the report, the Company requests that it immediately be notified. Most problems are much more readily resolved when conditions are exposed than at some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

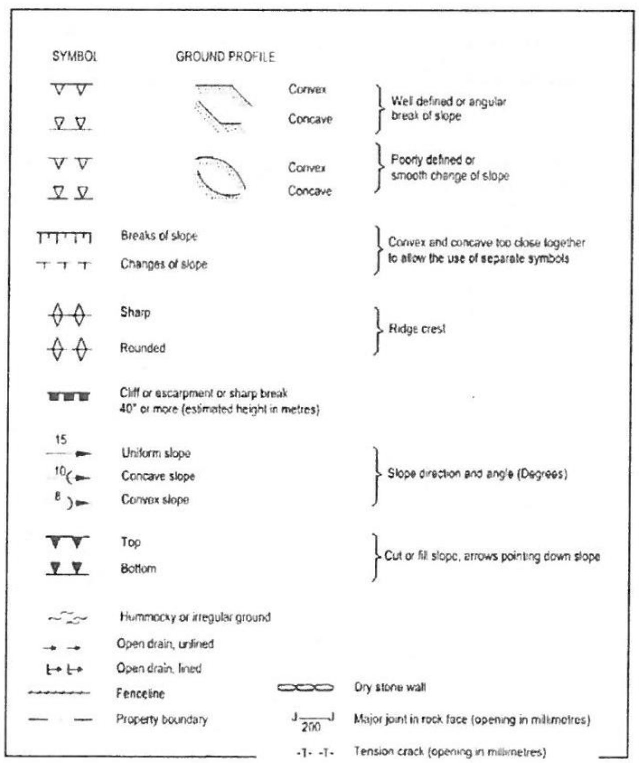
Reproduction of Information for Contractual Purposes

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

Site Inspection

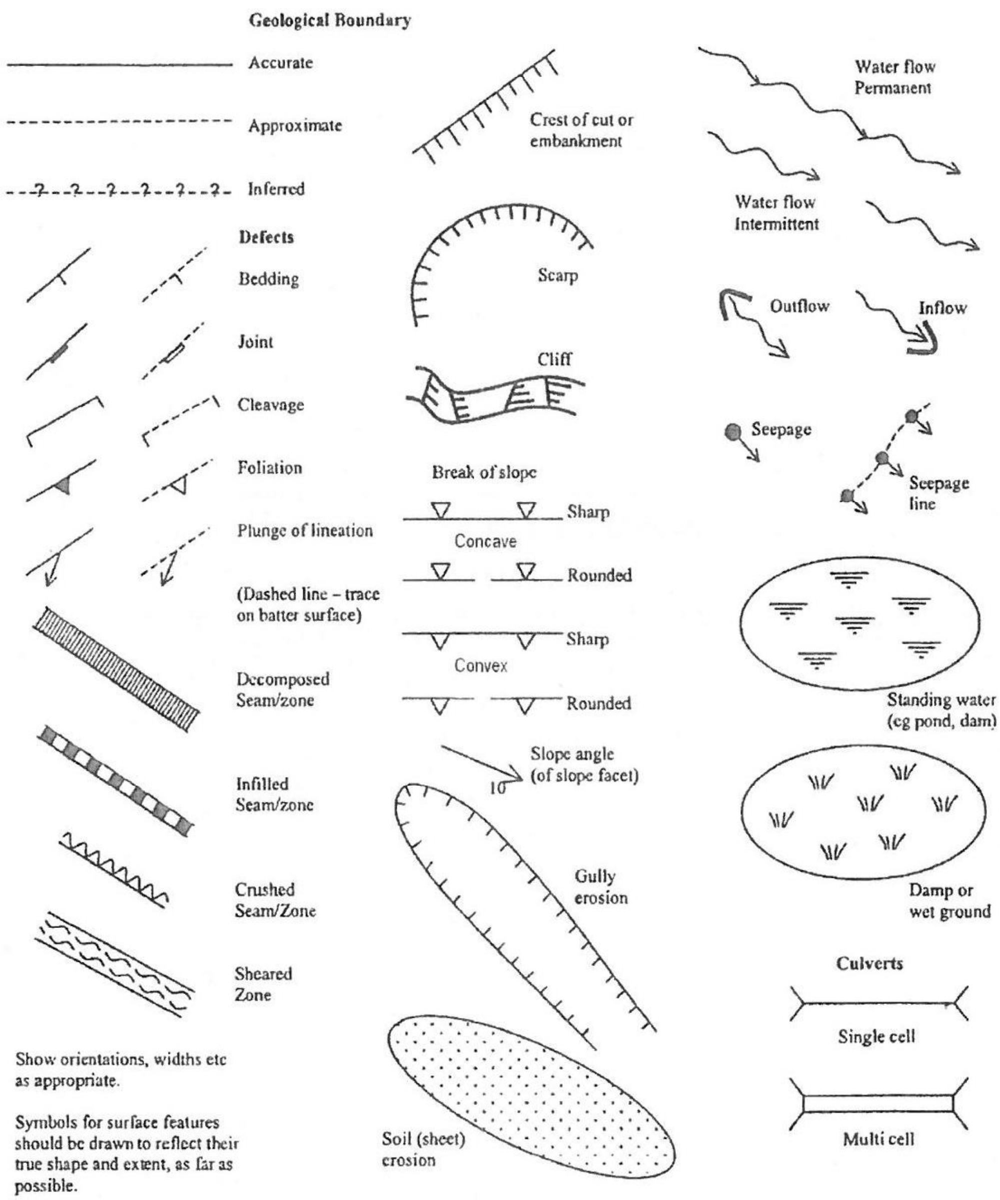
The Company will always be pleased to provide engineering inspection services for geotechnical aspects of work to which this report is related. This could range from a site visit to confirm that conditions exposed are as expected, to full time engineering presence on site.





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

APPENDIX E - GEOLOGICAL AND GEOMORPHOLOGICAL MAPPING SYMBOLS AND TERMINOLOGY



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

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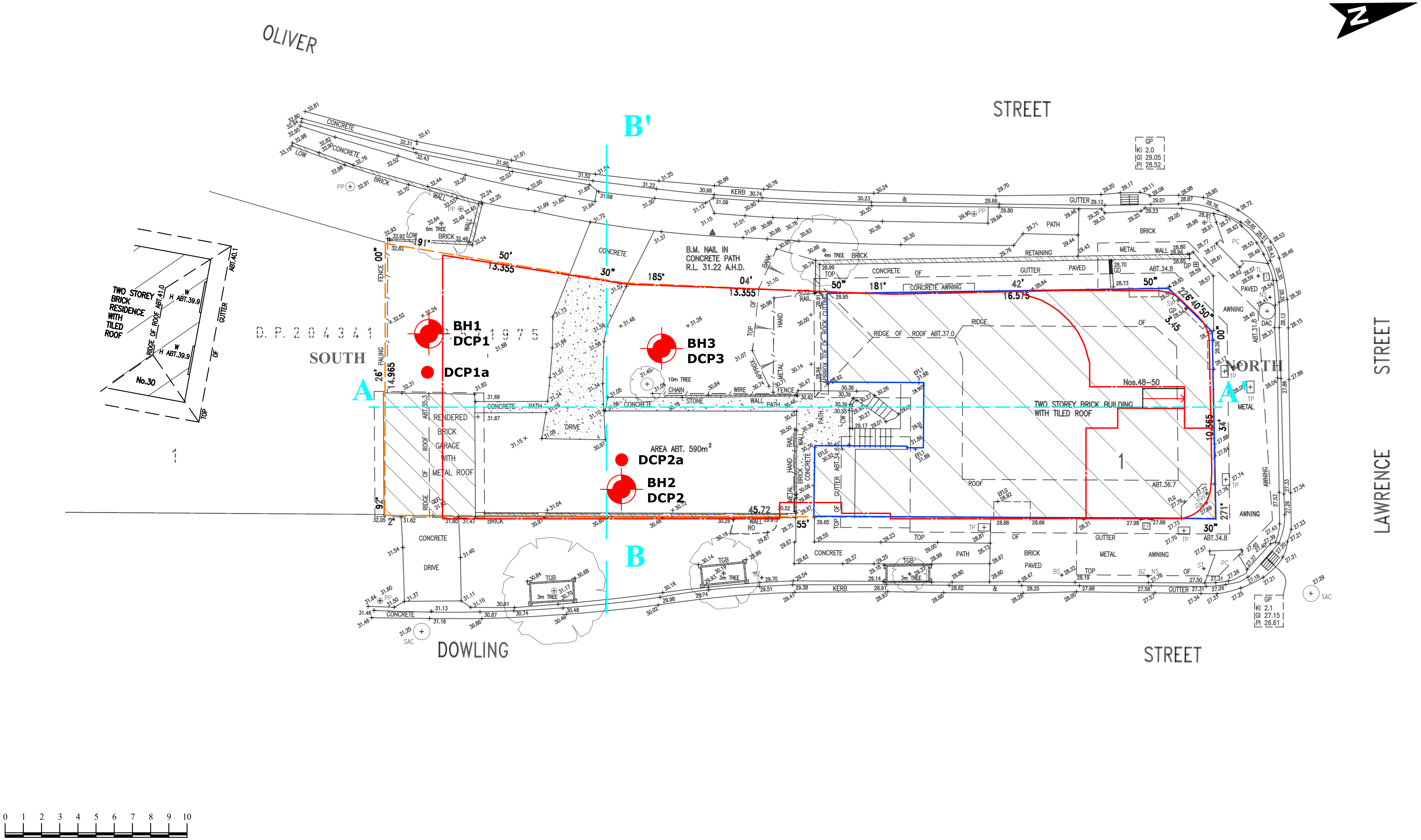
Appendix 2



SITE PLAN & TEST LOCATIONS FIGURE 1.

LEGEND

A — A'	CROSS-SECTION REFERENCE LINE
BH / DCP	AUGER / DYNAMIC CONE PENETROMETER LOCATION
●	DYNAMIC PENETROMETER TEST
	PROPOSED STRUCTURE
	EXISTING STRUCTURE
	PROPERTY BOUNDARY
	PROPERTY EXCAVATION

CROZIER GEOTECHNICAL CONSULTANTS
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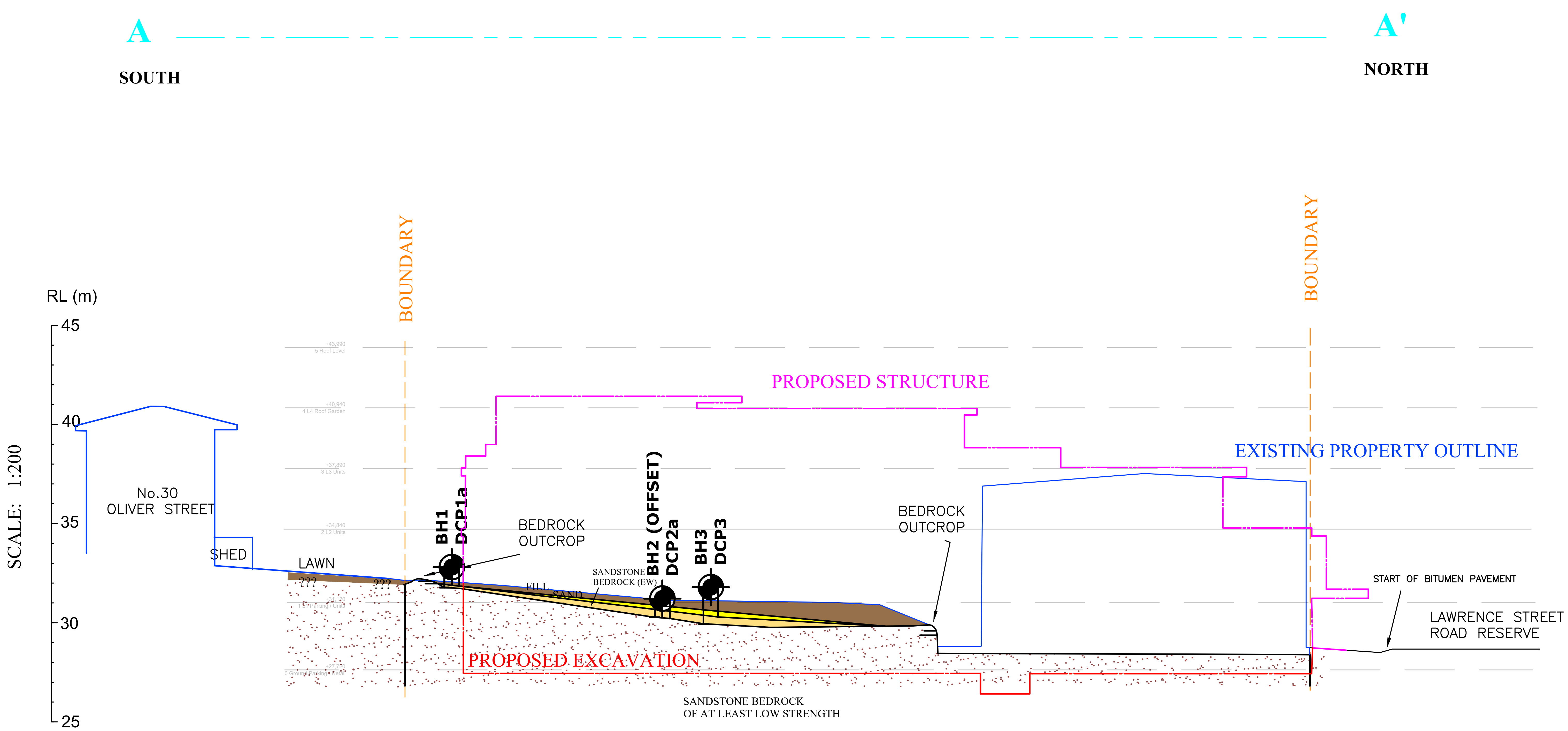
Fax: (02) 9939 1883
Crozier Geotechnical is a division of PJC Geo-Engineering Pty Ltd

SCALE:	1:200
DRAWING:	FIGURE 1
DATE:	22/03/2021
APPROVED BY:	TMC
DRAWN BY:	ML
PROJECT:	2020-050

PREPARED FOR:
LAWRENCE STREET NOMINEES PTY LTD

ADDRESS:
50 LAWRENCE STREET, FRESHWATER



0 1 2 3 4 5 6 7 8 9 10

SCALE: 1:200

VL - Very Loose	VS - Very Soft	ELS - Extremely Low Strength	EW - Extremely Weathered	fg - Fine Grained
L - Loose	S - Soft	VLS - Very Low Strength	HW - Highly Weathered	mg - Medium Grained
MD - Medium Dense	F - Firm	LS - Low Strength	DW - Distinctly Weathered	cg - Coarse Grained
D - Dense	St - Stiff	MS - Medium Strength	MW - Moderately Weathered	MAS - Massive
VD - Very Dense	VSt - Very Stiff	HS - High Strength	SW - Slightly Weathered	BD - Bedded
	H - Hard	VHS - Very High Strength	FR - Fresh	OC - Outcrop

NB. FOR LOCATION OF SECTION A-A', PLEASE REFER TO FIGURE 1. SITE PLAN AND TEST LOCATIONS

GEOLOGICAL MODEL FIGURE 2.

CROZIER
GEOTECHNICAL CONSULTANTS

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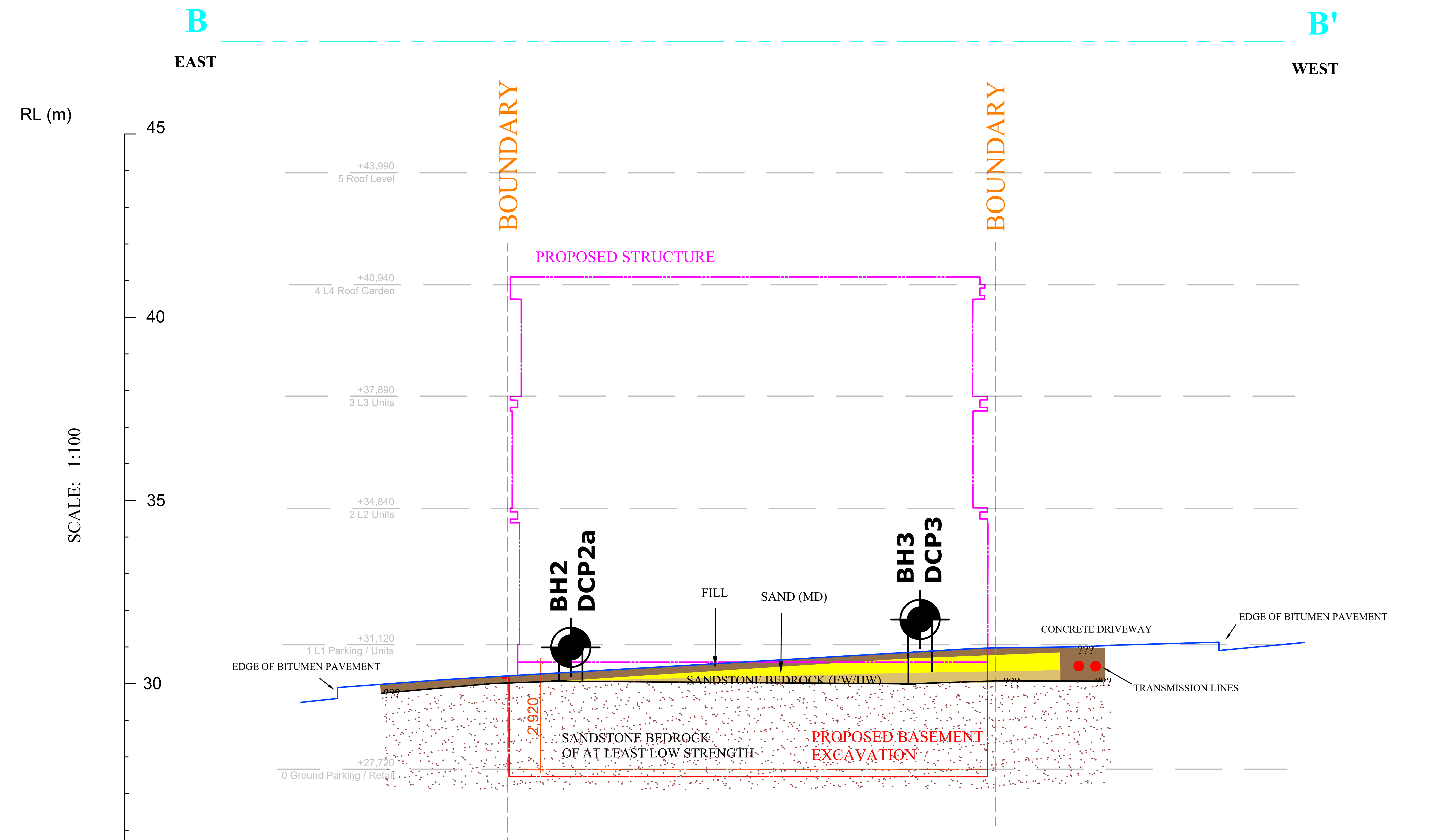
LEGEND

AUGER LOCATIONS	A — A' CROSS-SECTION REFERENCE LINE	PROPOSED STRUCTURE	SAND	SANDSTONE BEDROCK
	PROPERTY BOUNDARY	EXISTING STRUCTURE	FILL/ POSSIBLE FILL	SANDSTONE BEDROCK (EW/HW)

SCALE:	1:200
DRAWING:	FIGURE 2
DATE:	22/03/2021
APPROVED BY:	TMC
DRAWN BY:	ML
PROJECT:	2020-050

PREPARED FOR:
LAWRENCE STREET NOMINEES PTY LTD

ADDRESS:
50 LAWRENCE STREET, FRESHWATER



VL - Very Loose	VS - Very Soft	ELS - Extremely Low Strength	EW - Extremely Weathered	fg - Fine Grained
L - Loose	S - Soft	VLS - Very Low Strength	HW - Highly Weathered	mg - Medium Grained
MD - Medium Dense	F - Firm	LS - Low Strength	DW - Distinctly Weathered	cg - Coarse Grained
D - Dense	St - Stiff	MS - Medium Strength	MW - Moderately Weathered	MAS - Massive
VD - Very Dense	VSt - Very Stiff	HS - High Strength	SW - Slightly Weathered	BD - Bedded
	H - Hard	VHS - Very High Strength	FR - Fresh	OC - Outcrop

NB. FOR LOCATION OF SECTION B-B', PLEASE REFER TO FIGURE 1. SITE PLAN AND TEST LOCATIONS

GEOLOGICAL MODEL FIGURE 3.

CROZIER GEOTECHNICAL CONSULTANTS

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LEGEND

- AUGER LOCATIONS
- A — A' CROSS-SECTION REFERENCE LINE
- PROPERTY BOUNDARY
- EXCAVATION OUTLINE
- SANDSTONE BEDROCK
- SANDSTONE BEDROCK (EW/HW)
- AUGER LOCATIONS
- EXISTING TRANSMISSION LINES
- PROPOSED STRUCTURE
- EXISTING GROUND
- FILL/ POSSIBLE FILL
- SAND(MD)

SCALE:	1:100 @ A3
DRAWING:	FIGURE 3
DATE:	22/03/2021
APPROVED BY:	TMC
DRAWN BY:	ML
PROJECT:	2020-050

PREPARED FOR:
LAWRENCE STREET NOMINEES PTY LTD

ADDRESS:
50 LAWRENCE STREET, FRESHWATER

BOREHOLE LOG

CLIENT: Lawrence Street Nominees Pty Ltd | **DATE:** 17/03/2020 | **BORE No.:** 1

PROJECT: Demolition of existing site structures and construction of three storey mix use building | **PROJECT No.:** 2020-050 | **SHEET:** 1 of 1

LOCATION: 50 Lawrence Street, Freshwater | **SURFACE LEVEL:** RL= 32.28m

Depth (m)	Classification	Description of Strata **PRIMARY SOIL** - consistency / density, colour, grainsize or plasticity, moisture condition, soil type and secondary constituents, other remarks	Sampling		In Situ Testing	
			Type	Tests	Type	Results
0.00						
0.20		FILL/TOPSOIL: Loose, dark brown, fine to medium grained, moist, silty sand with some plant roots, silty sand with fine to coarse gravel.				
0.40		SANDSTONE (EW/HW): Fine to medium grained, pale brown mottled orange, extremely low to very low strength	D	0.25 0.35		
		AUGER REFUSAL at 0.40m depth on sandstone bedrock of at least low strength				
1.00						
2.00						

RIG: Dingo Restricted Access

DRILLER: AC

METHOD: Solid stem, spiral flight auger in conjunction with a tungsten carbide bit

LOGGED: ML

GROUND WATER OBSERVATIONS: None

REMARKS:

CHECKED: TMC

BOREHOLE LOG

CLIENT:	Lawrence Street Nominees Pty Ltd	**DATE:**	17/03/2020	**BORE No.:**	2
PROJECT:	Demolition of existing site structures and construction of three storey mix use building	**PROJECT No.:**	2020-050	**SHEET:**	1 of 1
LOCATION:	50 Lawrence Street, Freshwater	**SURFACE LEVEL:**	RL= 30.70m		

Depth (m)	Classification	Description of Strata **PRIMARY SOIL** - consistency / density, colour, grainsize or plasticity, moisture condition, soil type and secondary constituents, other remarks	Sampling		In Situ Testing	
			Type	**Tests**	**Type**	**Results**
0.00		FILL/TOPSOIL: Loose, dark brown, fne to medium grained, moist, silty sands with fine to coarse gravel	D	0.15 0.25		
0.35		AUGER REFUSAL at 0.35m depth on sandstone bedrock of at least low strength				
1.00						
2.00						

RIG:	Dingo Restricted Access	DRILLER:	AC
METHOD:	Solid stem, spiral flight auger in conjunction with a tungsten carbide	LOGGED:	ML
GROUND WATER OBSERVATIONS:	None		
REMARKS:		CHECKED:	TMC

BOREHOLE LOG

CLIENT: Lawrence Street Nominees Pty Ltd **DATE:** 17/03/2020 **BORE No.:** 3

PROJECT: Demolition of existing site structures and construction of three storey mix use building **PROJECT No.:** 2020-050 **SHEET:** 1 of 1

LOCATION: 50 Lawrence Street, Freshwater **SURFACE LEVEL:** RL= 31.20m

Depth (m)	Classification	Description of Strata **PRIMARY SOIL** - consistency / density, colour, grainsize or plasticity, moisture condition, soil type and secondary constituents, other remarks	Sampling Type	Sampling Tests	In Situ Testing Type	In Situ Testing Results
0.00		FILL: Medium dense, dark brown, fine to medium grained, moist, silty sand with some fine gravels				
0.50	SM	SAND: Medium dense, pale orange, fine to medium grained, moist, sands with trace of gravel	D	0.50 0.60		
0.80		é pale orange mottled brown		0.80		
0.85		é pale grey mottled brown	D	0.90		
0.90		SANDSTONE (EW): Fine to medium grained, pale grey mottled orange, extremely low to very low strength bedrock				
1.00			D	1.10 1.20		
1.20		AUGER REFUSAL at 1.20m depth on sandstone bedrock of at least low strength				
2.00						

RIG: Dingo Restricted Access DRILLER: AC

METHOD: Solid stem, spiral flight auger in conjunction with a tungsten carbide LOGGED: ML

GROUND WATER OBSERVATIONS: None

REMARKS: CHECKED: TMC

DYNAMIC PENETROMETER TEST SHEET

CLIENT: Lawrence Street Nominees Pty Ltd **DATE:** 17/03/2020

PROJECT: Demolition of existing site structures and construction of three storey mix use building **PROJECT No.:** 2020-050

LOCATION: 50 Lawrence Street, Freshwater **SHEET:** 1 of 1

	Test Location							
Depth (m)	DCP1	DCP1a	DCP2	DCP2a	DCP3			
0.00 - 0.15	2	2	3	3	5			
0.15 - 0.30	2 (B) Refusal at 0.25m depth	6	4 (B) Refusal at 0.30m depth	4	4			
0.30 - 0.45		6 (B) Refusal at 0.45m depth		2 (B) Refusal at 0.40m depth	4			
0.45 - 0.60					3			
0.60 - 0.75					6			
0.75 - 0.90					15 (B) Refusal at 0.85m depth			
0.90 - 1.05								
1.05 - 1.20								
1.20 - 1.35								
1.35 - 1.50								
1.50 - 1.65								
1.65 - 1.80								
1.80 - 1.95								
1.95 - 2.10								
2.10 - 2.25								
2.25 - 2.40								
2.40 - 2.55								
2.55 - 2.70								
2.70 - 2.85								
2.85 - 3.00								

TEST METHOD: AS 1289. F3.2, CONE PENETROMETER

REMARKS:

(B) Test hammer bouncing upon refusal on solid object

-- No test undertaken at this level due to prior excavation of soils

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Appendix 3

TABLE : A

Landslide risk assessment for Risk to life

HAZARD	Description	Impacting	Likelihood of Slide	Spatial Impact of Slide		Occupancy	Evacuation	Vulnerability	Risk to Life
A	Landslip (earth slide <1m³) from soils due to basement excavation along		Excavation of up to ≤1.20m depth of fill/soil, decreasing north to 0.60m of soil	a) House, ≤12.50m from the Ò1.20m deep soil excavation b) Lawn, Ò3.0m from the southern side of the Ò1.20m deep soil excavation c) Footpath is on western side of the Ò1.20m deep soil excavation d) Transmission line, Ò2.40 from the Ò1.20m deep soil excavation e)Pavement is 5.0m from the Ò1.20m deep soil excavation f) Footpath is on the northern side of the Ò0.50m deep excavation g) Pavement is 4.0m from the Ò0.50m deep excavation h) Grass lawn/pathway is on the eastern side of the Ò1.20m deep soil excavation i) Pavement is 4.0m from the Ò1.20m deep soil excavation		a) Person in house 10hrs/day avge. b) Person in the lawn 1hr/day avge. c) Person on footpath 7hrs/day avge. d) Person on footpath 7hrs/day avge. e) Person in car 16hrs/day avge. f) Person on footpath 7hrs/day avge. g) Person in car 16hrs/day avge. h) Person on the grass lawn/footpath 5hrs/day avge. i) Person in car 10hrs/day avge.	a) Almost certain to not evacuate b) Unlikely to not evacuate c) Unlikely to not evacuate d) Unlikely to not evacuate e) Almost certain to not evacuate f) Unlikely to not evacuate g) Almost certain to not evacuate h) Unlikely to not evacuate i) Almost certain to not evacuate	a) Person in building, minor damage only b) Person in lawn, minor damage only c) Person in footpath, minor damage only d) Person in footpath, impacted by tansmission line e) Person in the car, minor damage only f) Person in footpath, minor damage only g) Person in the car, minor damage only h) Person in the grass lawn/pathway, minor damage only i) Person in the car, minor damage only	
			Possible	**Prob. of Impact**	**Impacted**				
		a) No. 30 Dowling Street (house)	0.001	0.001	0.0001	0.4167	0.9	0.01	**3.75E-13**
		b) No.30 Dowling Street (lawn)	0.001	0.010	0.0001	0.0417	0.25	0.05	**5.21E-13**
		c) Road reserve, footpath (Oliver St)	0.001	0.850	0.1000	0.2917	0.25	0.05	**3.10E-07**
		d) Transmission lines (Oliver St)	0.001	0.001	0.1000	0.2917	0.25	1.00	**7.29E-09**
		e) Road reserve, pavement (Oliver St)	0.001	0.001	0.0001	0.6667	0.9	0.01	**6.00E-13**
		f) Road reserve, footpath (Lawrence St)	0.001	0.500	0.0500	0.2917	0.25	0.01	**1.82E-08**
		g) Road reserve, pavement (Lawrence St)	0.001	0.001	0.0001	0.6667	0.9	0.01	**6.00E-13**
		h) Road reserve, grass lawn (Dowling St)	0.001	0.500	0.0500	0.2083	0.25	0.05	**6.51E-08**
		i) Road reserve, pavement (Dowling St)	0.001	0.001	0.0001	0.4167	0.9	0.01	**3.75E-13**
B	Landslip (rock slide/topple <1m³) within rock excavation - Ground Level & Basement Level		Rock excavations up to 3.3m depth expected, decreasing north up to 0.60m depth excavation	a) House, ≤12.50m from the Ò3.30m deep rock excavation b) Lawn Ò3.0m from the southern side of the Ò3.30m deep rock excavation c) Footpath is on western side of the Ò4.20m deep rock excavation d) Transmission line, Ò2.40 from the Ò4.20m deep rock excavation e) Pavement is 5.0m from the Ò4.20m deep rock excavation f) Footpath is on the northern side of the Ò0.60m deep excavation g) Pavement is 4.0m from the Ò0.60m deep excavation h) Grass lawn/pathway is on the eastern side of the Ò4.20m deep rock excavation i) Pavement is 4.0m from the Ò4.20m deep rock excavation		a) Person in house 10hrs/day avge. b) Person in the lawn 1hr/day avge. c) Person on footpath 7hrs/day avge. d) Person on footpath 7hrs/day avge. e) Person in car 16hrs/day avge. f) Person on footpath 7hrs/day avge. g) Person in car 16hrs/day avge. h) Person on the grass lawn/footpath 5hrs/day avge. i) Person in car 10hrs/day avge.	a) Almost certain to not evacuate b) Unlikely to not evacuate c) Unlikely to not evacuate d) Unlikely to not evacuate e) Almost certain to not evacuate f) Unlikely to not evacuate g) Almost certain to not evacuate h) Unlikely to not evacuate i) Almost certain to not evacuate	a) Person in building, minor damage only b) Person on lawn, likely to be impacted by fall c) Person on footpath, possibly impacted by fall d) Person on footpath, impacted by transmission line e) Person in the car, minor damage only f) Person on footpath, minor damage only g) Person in the car, minor damage only h) Person on the grass lawn/pathway, possibly impacted by fall i) Person in the car, minor damage only	
			Unlikely	**Prob. of Impact**	**Impacted**				
		a) No. 30 Dowling Street (house)	0.0001	0.001	0.0001	0.4167	0.9	0.01	**3.75E-14**
		b) No.30 Dowling Street (lawn)	0.0001	0.010	0.0001	0.0417	0.25	0.70	**7.29E-13**
		c) Road reserve, footpath (Oliver St)	0.0001	0.850	0.0500	0.2917	0.25	0.50	**1.55E-07**
		d) Transmission lines (Oliver St)	0.0001	0.001	0.1000	0.2917	0.25	1.00	**7.29E-10**
		e) Road reserve, pavement (Oliver St)	0.0001	0.001	0.0001	0.6667	0.9	0.01	**6.00E-14**
		f) Road reserve, footpath (Lawrence St)	**Rare** 0.00001	0.850	0.0500	0.2917	0.25	0.01	**3.10E-10**
		g) Road reserve, pavement (Lawrence St)	**Rare** 0.00001	0.001	0.0001	0.6667	0.9	0.01	**6.00E-15**
		h) Road reserve, grass lawn (Dowling St)	0.0001	0.850	0.0500	0.2083	0.25	0.5	**1.11E-07**
		i) Road reserve, pavement (Dowling St)	0.0001	0.001	0.0001	0.4167	0.9	0.01	**3.75E-14**

* hazards considered in current condition and/or without remedial/stabilisation measures or poor support systems
* likelihood of occurrence for design life of 100 years
* Spatial Impact - Probaility of Impact refers to slide impacting structure/area expressed as a % (i.e. 1.00 = 100% probability of slide impacting area if slide occurs).
  Impacted refers to expected % of area/structure damaged if slide impacts (i.e. small, slow earth slide will damage small portion of house structure such as 1 bedroom (5%), where as large boulder roll may damage/destroy >50%)
* neighbouring houses considered for impact of slide to bedroom unless specified, due to high occupancy and lower potential for evacuation.
* considered for person most at risk, where multiple people occupy area then increased risk levels
* for excavation induced landslip then considered for adjacent premises/buildings founded off shallow footings, unless indicated
* evacuation scale from Almost Certain to not evacuate (1.0), Likely (0.75), Possible (0.5), Unlikely (0.25), Rare to not evacuate (0.01). Based on likelihood of person knowing of landslide and completely evacuating area prior to landslide impact.
* vulnerability assessed using Appendix F - AGS Practice Note Guidelines for Landslide Risk Management 2007

Appendix 4

APPENDIX A

DEFINITION OF TERMS

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

APPENDIX C: LANDSLIDE RISK ASSESSMENT

QUALITATIVE TERMINOLOGY FOR USE IN ASSESSING RISK TO PROPERTY

QUALITATIVE MEASURES OF LIKELIHOOD

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

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

QUALITATIVE MEASURES OF CONSEQUENCES TO PROPERTY

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

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

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

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

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

QUALITATIVE RISK ANALYSIS MATRIX – LEVEL OF RISK TO PROPERTY

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

Notes: (5) For Cell A5, may be subdivided such that a consequence of less than 0.1% is Low Risk.
(6) When considering a risk assessment it must be clearly stated whether it is for existing conditions or with risk control measures which may not be implemented at the current time.

RISK LEVEL IMPLICATIONS

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

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