Douglas Partners
Geotechnics / Environment / Groundwater

Douglas Partners Pty Ltd
ABN 75 053 980 117
www.douglaspartners.com.au
96 Hermitage Road
West Ryde NSW 2114
PO Box 472
West Ryde NSW 1685
Phone (02) 9809 0666

GFM Investment Management Limited as Trustee for GFM Home Trust
Level 4, 255 George Street
Sydney NSW 2000

Project 85301.01
22 September 2023
R.002.Rev3
AA/JDB:gl

Attention: Tony Bellingham

Email: TonyBellingham@homeapartments.com.au

Geotechnical Assessment
Proposed Mixed-Use Development
524-542 Pacific Highway, St Leonards

1. Introduction

This report presents the findings of a preliminary geotechnical assessment for a proposed new mixed-use, high-rise building at 524-542 Pacific Highway, St. Leonards. The assessment was carried out in accordance with Douglas Partners Pty Ltd (DP) proposal dated 13 July 2022.

This preliminary geotechnical assessment has been prepared for submission to the Department of Planning and Environment (DPE) in support of a concurrent State Led Rezoning and State Significant Development Application (SSDA) for a new mixed-use development, comprising build-to-rent housing, commercial and retail land uses at the Telstra Exchange Site at 524-542 Pacific Highway, St Leonards (the site).

Based on the supplied architectural drawings (DKO Architecture drawing set 00013070, revision 3 dated 4 September 2023), and from information provided by Ethos Urban, it is understood that the proposed development will comprise:

- Site preparation and excavation of a single basement level;
- Retention and integration of the existing Telstra Exchange Building;
- Construction of a new 42-storey mixed-use development, comprising:
  - o 21,472 m^2 of build-to-rent housing across 31 storeys, including 272 dwellings;
  - o 3,840 m^2 of non-residential space within an 8 storey podium used for the purposes of short stay accommodation, including;
    - 721 m^2 of Key Worker Housing across one level, within the podium, delivering a total of ten dwellings to be managed as part of the build-to-rent development;
- 2,014 m^2 of community amenity facilities throughout the building, including a swimming pool;
- Residential lobby accessed via Christie Street and separate commercial use lobby accessed via Pacific Highway;
- Podium car parking and loading area with vehicular access via Christie Street, comprising 48 space car stacker;

- Associated landscaping and public domain works; and
- Augmentation of, and connection to, existing utilities services as required.

It is noted that to facilitate the abovementioned development, amendments to the *Lane Cove Local Environmental Plan 2013* are proposed via a concurrent State Led Rezoning to rezone the site from B3 Commercial Core to B4 Mixed Use and to increase the maximum building height of 72 m to 155 m. The maximum FSR of the site will remain as per existing at 17.1:1.

DP previously carried out a preliminary investigation on the site and prepared a geotechnical report (Project 85301.R.001.Rev0, dated 6 April 2016). This current assessment has referenced the previous investigation. The previous investigation comprised a walk-over survey of the site and its surrounds by an experienced geotechnical engineer from DP, drilling, sampling and logging of one borehole and laboratory testing of the recovered rock core.

2. Site Description

The roughly rectangular site is bound to the west by Christie Street, to the north by the Pacific Highway, to the east by a commercial high rise and to the south by a medium-rise commercial property (private property). The western portion of the site is occupied by four, two storey buildings. The Telstra Exchange is two storeys high on the northern side and approximately four storeys high over the southern third. Access to the site is from Christie Street. The existing ground surface within the site slopes from the north-east towards the south-west from about RL 87 m Australian Height Datum (AHD), to about RL 81 m.

The site appears to be located about 30 m to the south of Sydney Metro tunnel, which crosses below Atchison Street and extends towards Pacific Highway following northwest/southeast direction. The actual Sydney Metro alignment should be confirmed for detailed planning and design.

Initial review of the DBYD plans of the site indicates the presence of a 150 mm diameter cast iron cement lined (CICL) water main running close to the site boundary with Pacific Highway.

3. Published Mapping

3.1 Regional Geology

Reference to the Sydney 1:100 000 Geological Sheet indicates that the site is underlain by Triassic aged Ashfield Shale, comprising shale, laminite and carbonaceous shale with minor sandstone close to the boundary with Hawkesbury Sandstone, which comprises medium and coarse grained quartzose sandstone with minor shale and siltstone interbeds. Steeply dipping joints, typically oriented north-south and east-west, are common in the Hawkesbury Sandstone. The Mittagong Formation is sometimes, encountered between the Ashfield Shale and Hawkesbury Sandstone and is typically 3 m to 4 m thick and comprises variable interbedded fine-grained sandstone and laminite.

3.2 Hydrogeology

Review of available information from a WaterNSW registered groundwater well within 500 m of the site (Ref. GW108224) indicates measured groundwater at about 35 m depth, as recorded in 2006.

Review of previous projects by DP in the area indicates measured groundwater in monitoring wells at variable depths of between 10 m to 20 m (RL 62 m to RL 75 m). Groundwater seepage was also encountered at a nearby site at about 12 m depth (RL 68 m) during pile inspections carried out in 2018.

3.3 Acid Sulfate Soils

The NSW Acid Sulfate Soil Risk mapping indicates that the site does not lie within an area known for acid sulfate soils, nor does the site occur within an area known for soil salinity issues.

4. Previous Field Work

4.1 Methods

The previous field work for the investigation in 2016 included the drilling of one cored borehole at the location shown on Drawing 1 attached. A summary of the field work is presented as follows:

- The borehole was drilled using a bobcat-mounted drilling rig fitted with continuous solid flight augers and diamond rock coring equipment on 12 March 2016;
- The borehole was drilled into the weathered bedrock to a depth of 2 m, then extended to the target depth using NMLC (52 mm diameter) diamond coring drilling. Casing was used to a depth of 2 m to protect the bore from caving in during rock coring; and
- The rock cores were transferred to DP's central laboratory in West Ryde to be tested for Point Load Index tests and further logging.

A detailed borehole log sheet is attached together with the explanatory notes explaining descriptive terms and the classification methods used in its preparation. The surface level of the borehole was determined by interpretation of published survey data and is approximate only.

4.2 Results

The borehole encountered the following profile:

- Fill – comprising slightly sandy clay with charcoal, shells, bricks, steel, etc. to a depth of about 1.2 m, underlying the concrete driveway slabs;
- Shale and Laminite – dark grey and grey, very low and low strength shale and laminite was encountered from about 1.2 m to a depth of about 8.4 m. The rock comprised fractured, highly

fractured and fragmented domains with three typical sets of joints (J1: 45°, J2: 65°- 80° and J3: ~90°);

- Sandstone – the borehole encountered fine grained, pale grey sandstone with carbonaceous laminations, highly weathered and of low to medium strength from a depth of about 8.4 m to about 10.65 m. High strength, slightly weathered to fresh sandstone, slightly fractured with some laminations, was encountered from about 10.65 m to the depth of investigation at 13.73 m; and
- Groundwater – no seepage was noted while augering to a depth of 2.0 m. The use of water as a drilling fluid during diamond coring precluded measurement of the groundwater level during the core drilling of the boreholes. Long term groundwater level monitoring was not included in the geotechnical investigation.

5. Proposed Development

From the description provided by Ethos Urban, the proposed development comprises:

- *"Site preparation and excavation;*
- *Retention and integration of the existing Telstra Exchange Building;*
- *Construction of a new 42-storey mixed-use development, comprising:*
  - *21,472 m² of build-to-rent housing across 31 storeys, including 272 dwellings;*
  - *3,840 m² of non-residential space within an 8 storey podium used for the purposes of short stay accommodation, including;*
  - *721 m² of Key Worker Housing across 1 level, within the podium, delivering a total 10 dwellings to be managed as part of the build to rent development; and*
  - *2,014m² of community amenity facilities throughout the building;*
- *Residential lobby accessed via Christie Street and separate commercial use lobby accessed via Pacific Highway;*
- *Podium car parking and loading area with vehicular access via Christie Street, comprising a 48 space car stacker;*
- *Associated landscaping and public domain works; and*
- *Augmentation of, and connection to, existing utilities services as required."*

In addition to the description provided by Ethos Urban, it is further understood from the DKO Architecture drawings (Drawing Set 00013070, revision 3 dated 4 September 2023) that the proposed development includes a single level basement to the west of the Telstra Exchange building.

The lowest basement floor level is RL 75.4 m. It is anticipated that the construction of the basement would require excavations to variable depths in a range of 6 m along the southern boundary and about 12 m along the northern site boundary. Deeper detailed excavation to RL 71.7 m (approximately 10 m to 14 m) is understood to be required for lift shafts.

6. Preliminary Geotechnical Model

A preliminary geotechnical model has been interpreted using the results of BH1 only. Bands of lower and higher strength rock should be expected within the generalised layers.

The rock units have also been classified in accordance with Pells et al (2019) "Classification of Sandstone and Shale in the Sydney Region: A Forty Year Review" Aust. Geomechanics Journal, June, 2019, which use a combination of rock strength and fracture spacing to divide the rock into five classes ranging from Class I (high strength and very few defects) to Class V (very low strength and/or highly fractured). In some cases, the classification for the stronger rock has been downgraded due to fracture spacing and the presence of weaker seams.

A summary of the typical material descriptions for the geotechnical model is summarised in Table 1.

Table 1: Preliminary Geotechnical Model

Unit	Depth Range of Layer (m) / *RL (m, AHD)*	Description
Unit 1 – FILL	0.0 – 1.2 *(RL 80.8 – 82.0)*	Silty Clay FILL with gravel and various proportion of building waste
Unit 2 – Very low to low strength rock (Class V)	1.2 – 8.4 *(RL 73.6 – 80.8)*	Shale and Laminite, very low to low strength, highly weathered, fragmented, fractured and highly fractured (Ashfield Shale).
Unit 3 – Low Strength Rock (Class IV)	8.4 – 9.0 *(RL 73.0 – 73.6)*	Sandstone, low strength, moderately weathered, slightly fractured, (Mittagong Formation)
Unit 4 – Medium Strength Rock (Class III)	9.0 – 10.6 *(RL 71.4 – 73.0)*	Sandstone, medium strength, slightly weathered, slightly fractured, (Mittagong Formation)
Unit 5 – High Strength Rock (Class II)	10.6 – 13.7 *(RL 68.3 – 71.4)*	Sandstone, high strength, slightly weathered to fresh, slightly fractured (Hawkesbury Sandstone)

No groundwater measurements on the site have been carried out to date. It is expected that the regional groundwater table will be relatively deep and within sandstone well below the proposed basement excavation.

Based on DP's experience in the area, perched groundwater seepage should be expected at the interface of existing fill/soil and bedrock, and is also likely to occur within the fractured zones and joints within the rock. Higher seepage flows may also be encountered near the interface of shale and sandstone. Seepage flows are likely to increase following periods of extended wet weather.

Groundwater levels are affected by climatic conditions, rock mass permeability and other factors and will vary with time.

7. Comments

7.1 Site Preparation and Earthworks

7.1.1 Excavation Conditions

Based on the supplied coordination set architectural drawings by DKO Architecture (Ref.: 00013070, dated 4 September 2023) it is understood that a design lower basement level is at RL 75.4 m at the western portion of the site (west of Telstra Exchange building) requiring bulk excavation to depths of between about 4.5 m to 9 m, increasing towards Pacific Highway. Additional, localised, excavation to RL 71.7 m will be required for the lift shafts.

It is anticipated that bulk excavation will generally encounter fill and very low to low strength rock, however, medium to high strength sandstone may be encountered in the deeper parts of the excavation towards Pacific Highway. Fill and highly weathered, very low and low strength, fractured rock should be readily excavated using conventional earthmoving equipment, however, the assistance of rock hammering or ripping will probably be required for effective removal of any medium to high strength rock or ironstone bands within the weathered rock profile. Excavation of the medium and high strength sandstone (Class II and III), if any, will likely require hydraulic rock breakers in conjunction with heavy ripping for effective removal of this material. Rock sawing can be used to reduce over-break and vibration along the boundaries and adjacent to the Telstra Exchange building.

It is likely that the bulk excavation will extend close to the existing footings supporting the Telstra Exchange building that is to be retained. The existing footing levels and dimensions are not known, but given the encountered subsurface condition, it is anticipated that the building may be founded on weathered shale at shallow depth. Any proposed excavation adjacent to the Telstra Building will need to provide both temporary and permanent support to the existing footings.

Exploratory test pits adjacent to the Telstra Building will be required to provide more information on the existing footings and foundation materials for detailed design of shoring.

7.1.2 Vibration

Where rock hammers are required in the vicinity of adjacent structures it would be prudent to monitor and limit vibrations on these structures. Based on DP's experience, and with reference to AS2670, a maximum peak particle velocity of 8 mm/sec (in any component direction) at the foundation level of adjacent structures is suggested for human comfort considerations. However, the excavation methods and tolerable vibration limits should be designed with consideration for the functionality of the equipment housed in the Telstra Exchange. Vibration trials are suggested during initial excavation of rock to verify vibration levels.

7.1.3 Dilapidation Survey

Dilapidation surveys should be carried out on the Telstra Exchange and other nearby buildings and pavements that may be affected by the basement excavation. The dilapidation surveys should be undertaken before the commencement of any excavation work in order to document any existing defects so that any claims for damage due to construction related activities can be accurately assessed.

7.1.4 Disposal of Excavated Material

All material requiring off-site disposal should be classified in accordance with the provisions of the current legislation and guidelines including the Waste Classification Guidelines (NSW EPA, 2014).

7.2 Excavation Support

Temporary slopes in soil and rock can be cut at batter slopes shown in Table 2 below, subject to detailed assessment of rock conditions by a suitably qualified geotechnical engineer/engineering geologist as the excavation progresses. This assumes there are no structures or surcharge loads near the crest of the slope. Shoring walls will generally be required on this site to support excavations.

Where insufficient room exists for the suggested batter slopes, or where underground services, foundations or other structures adjacent to the excavation need protection, support of the excavation face could be provided by soldier pile shoring walls with reinforced shotcrete panels and tie back anchors. Shoring piles are typically spaced at 2-2.4 m however more closely spaced piles may be required to reduce collapse between piles and to reduce deflections close to structures. Contiguous piles may be required where excavation is carried out directly below high level footings (i.e. possibly Telstra Exchange). Shoring piles should be taken to below the bulk excavation level.

Preliminary design of the retaining walls and excavation support with a single row of anchors can be based on a triangular earth pressure distribution using the parameters provided in Table 3. Active earth pressure coefficient (K_a) values may be used where some wall movement is acceptable. At rest earth pressure (K_o) values should be used where wall movement needs to be limited.

Table 2: Safe Batter Slopes

Material type	Maximum Temporary Batter Slope (H : V)	Maximum Permanent Batter Slope (H : V)
Fill (Unit 1)	2 : 1	2 : 1
Very low and low strength shale, laminite and sandstone (Units 2 and 3)	1 : 1	1.5 : 1
Medium strength sandstone (Unit 4)	Vertical*	Vertical*
High strength sandstone (Unit 5)	Vertical*	Vertical*

Note: * Subject to discontinuity assessment by experienced Geotechnical Engineer/Engineering Geologist. Bolting, possibly with shotcrete, will be required if the rock is adversely affected by defects.

Table 3: Preliminary Design Parameters for Shoring Systems

Material Type	Unit Weight (kN/m^3)	Earth Pressure Coefficient	
		Active (K_a)	At Rest (K_o)
Very low strength shale and laminite (Unit 2)	22	0.25	0.30
Low strength sandstone (Unit 3)	22	0.20	0.25
Medium and High strength sandstone (Units 4 and 5)	24	*	*

Note: * Subject to discontinuity assessment by experienced Geotechnical Engineer/Engineering Geologist. Bolting, possibly with shotcrete, will be required if the rock is adversely affected by defects.

All surcharge loads should be allowed for in the shoring design, including Telstra Exchange building footings, inclined slopes behind the wall, traffic and construction related loads. Shoring walls should be designed for the full hydrostatic pressure, unless drainage of the ground behind impermeable walls can be provided. Drainage could comprise suitable strip drains, pinned to the rock face at 1.5 m centres. Water from the strip drains is typically allowed to run into the underfloor drainage system.

For preliminary design of anchors, the maximum ultimate bond stresses shown in Table 4 could be adopted. The parameters given in Table 4 assume that the drill holes are clean and adequately flushed. The rockbolts/anchors should be bonded behind a line drawn up at 45 degrees from the base of the shoring or top of Unit 5 sandstone. Testing should be carried out to confirm the anchor capacities.

It is anticipated that the buildings will prop/support the shoring walls over the long term and therefore ground anchors are expected to be temporary only. The use of permanent anchors, if required, would require careful attention to corrosion protection and further geotechnical advice should be sought.

Table 4: Bond Stresses for Preliminary Anchor Design

Material Description	Ultimate Bond Stress (kPa)
Very low to low strength rock (Units 2 and 3)	250
Medium strength sandstone (Unit 4)	1,000
High strength sandstone (Unit 5)	2,000

These values should be confirmed by pull-out tests prior to installation of support elements. Ultimately, it is the contractor's responsibility to ensure that the correct design values (specific to the support system and method of installation) are used and that the support element holes are carefully cleaned prior to grouting.

Based on the anticipated bulk excavation depths, it is expected that the medium strength or stronger rock may only be encountered in deeper parts of the excavation and some detailed excavations. Rock faces in medium strength and stronger Hawkesbury Sandstone are generally self-supporting if not affected by adversely dipping discontinuities. Regular rock face inspections will be required during excavation (recommended at about every 1.0 m drop) to determine whether conditions are as anticipated.

7.3 Acid Sulfate Soils

The site is in an area mapped as having no known occurrence of acid sulfate soils. The relatively shallow residual clay soils encountered in the previous borehole are not consistent with acid sulphate soils. On this basis, it is considered that the site is unlikely to be underlain by acid sulfate soils and therefore an acid sulfate soil management plan is not relevant for this site.

7.4 Groundwater

The bulk excavation level is not expected to intersect the regional groundwater table. DP has been involved with similar and deeper basements nearby that have excavated through Ashfield Shale and into the Hawkesbury Sandstone. Some seepage occurred at varying depths but significant ongoing groundwater flows were not observed.

During construction and in the long term, it is anticipated that seepage into the excavation should generally be readily controlled by perimeter drains connected to a “sump-and-pump” system. A drained basement is technically feasible and will require permanent subfloor drainage below the basement floor slab to direct seepage to the stormwater drainage system. However, a drained basement and pumping to the stormwater system will be subject to approval from Council and relevant authorities. Given that the measured water levels on nearby sites have been within competent sandstone, it is expected that minor inflows associated with perched seepage in the shale would present minimal impact to surrounding groundwater systems and property. This could be assessed with permeability testing of the rock mass and numerical modelling.

It is possible that iron oxides will precipitate from any seepage, possibly leading to a build-up of an iron-oxide sludge. Allowance for periodic cleaning of such sludge should be made in the long-term maintenance requirements.

7.5 Foundations

The bulk excavation may expose variable foundations ranging from very low strength shale to medium and high strength sandstone (to be confirmed with further investigation). At this stage, given the high column loads it is anticipated that all structures will be supported on piles founded into the medium strength or stronger sandstone to provide uniform foundations and reduce differential settlements. Pad footings may be adopted if deeper bulk excavation exposes medium strength or stronger sandstone in some areas. Typical maximum allowable pressures for the various anticipated foundation materials, based on the foundation classification methods of Pells et al. (2019), are shown in Table 5. All footings in jointed rock should be founded below a line drawn upwards at 45 degrees from adjacent excavations. All footings should preferably be founded on the same foundation unit to minimise risks associated with differential settlements, however different rock foundations may be utilised provided differential settlements are properly assessed and considered.

Table 5: Preliminary Design Parameters for Footings

Material	Allowable End Bearing Pressure (kPa)	Ultimate End Bearing Pressure* (kPa)	Allowable Shaft Adhesion (kPa)	Ultimate Shaft Adhesion (kPa)
Class V – Very low to low strength shale and laminite (Unit 2)	700	3,000	50	50
Class IV – Low strength sandstone (Unit 3)	1,000	4,000	150	250
Class III – Medium strength sandstone (Unit 4)	3,500	30,000	350	800
Class II – High strength sandstone (Unit 5)	6,000	60,000	600	1,500

Note: * Ultimate values occur at large settlements (>5% of minimum footing dimension).

Foundations proportioned on the basis of the allowable parameters would be expected to experience total settlements of less than 1% of the minimum footing width/pile diameter under the applied working load, with differential settlement between adjacent columns expected to be less than half this value. All footing excavations should be inspected by a geotechnical engineer to confirm that foundation conditions are suitable for the design parameters. Spoon testing/proof coring will be required for foundations with allowable bearing pressures of, or in excess of, 3500 kPa.

Piles may be socketed into the medium to high strength rock (Class II or better), which was encountered below a depth of 9.0 m. Casing or support will be required for the filling and residual clays, as well as for the extremely to highly weathered shale to a depth of about 1.7 m. The casing will need to be extended into the very low to low strength shale and laminite if weak or fragmented zones are encountered during the piling.

8. Additional Investigation

For detailed design purposes, once the buildings are demolished, additional boreholes must be completed to confirm the soil, rock, and groundwater conditions across the site.

Groundwater monitoring wells should be installed on site to allow for a long-term groundwater level monitoring and permeability tests.

Based on DP's experience with similar developments in the area, some authorities will likely require further assessments for DA consent at later stages of the application process. This may include but not necessarily limited to the following:

- Groundwater Inflow Assessment and Dewatering Management Plan by WaterNSW and North Sydney Council;
- Impact Assessment and Geotechnical Monitoring Plan by TfNSW (Pacific Highway);
- Impact assessment by Sydney Water to assess the movement of existing 150 mm CICL water main due the excavation; and
- Based on the review of the available information, it is inferred that the site is more than 25 m away from the Sydney Metro tunnels. However, the available setback of the site boundary to the Sydney Metro protection zones are not clearly known. Sydney Metro would likely require additional assessment and geotechnical monitoring plan to address the effects of the proposed excavations to Sydney Metro assets.

9. Limitations

Douglas Partners (DP) has prepared this report for this project at 524 – 542 Pacific Highway, St Leonards in accordance with DP's proposal dated 13 July 2022 and acceptance received dated 13 July 2022. The work was carried out under DP's Conditions of Engagement. This report is provided for the exclusive use of GFM Investment Management Limited as Trustee for GFM Home Trust for this project only and for the purposes as described in the report. It should not be used by or relied upon for other projects or purposes on the same or other site or by a third party. Any party so relying upon this report beyond its exclusive use and purpose as stated above, and without the express written consent of DP, does so entirely at its own risk and without recourse to DP for any loss or damage. In preparing this report DP has necessarily relied upon information provided by the client and/or their agents.

The results provided in the report are indicative of the sub-surface conditions on the site only at the specific sampling and/or testing locations, and then only to the depths investigated and at the time the work was carried out. Sub-surface conditions can change abruptly due to variable geological processes and also as a result of human influences. Such changes may occur after DP's field testing has been completed.

DP's advice is based upon the conditions encountered during this investigation. The accuracy of the advice provided by DP in this report may be affected by undetected variations in ground conditions across the site between and beyond the sampling and/or testing locations. The advice may also be limited by budget constraints imposed by others or by site accessibility.

The assessment of atypical safety hazards arising from this advice is restricted to the components set out in this report and based on known project conditions and stated design advice and assumptions. While some recommendations for safe controls may be provided, detailed 'safety in design' assessment is outside the current scope of this report and requires additional project data and assessment.

This report must be read in conjunction with all of the attached and should be kept in its entirety without separation of individual pages or sections. DP cannot be held responsible for interpretations or conclusions made by others unless they are supported by an expressed statement, interpretation, outcome or conclusion stated in this report.

This report, or sections from this report, should not be used as part of a specification for a project, without review and agreement by DP. This is because this report has been written as advice and opinion rather than instructions for construction.

The scope of work for previous DP investigation in 2016 did not include the assessment of surface or sub-surface materials or groundwater for contaminants, within or adjacent to the site. Should evidence of filling of unknown origin be noted in the report, and in particular the presence of building demolition materials, it should be recognised that there may be some risk that such filling may contain contaminants and hazardous building materials.

Please contact the undersigned if you have any questions on this matter.

Yours faithfully
Douglas Partners Pty Ltd

Joshua Bendit
Geotechnical Engineer

Reviewed by

Scott Easton
Principal

Attachments:	Consultant Declaration
	About this Report
	Drawing 1: Site Layout and Previous Borehole Location
	Previous Borehole Log and Core Photographs

About this Report

Douglas Partners

Introduction

These notes have been provided to amplify DP's report in regard to classification methods, field procedures and the comments section. Not all are necessarily relevant to all reports.

DP's reports are based on information gained from limited subsurface excavations 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.

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This report is the property of Douglas Partners Pty Ltd. The report may only be used for the purpose for which it was commissioned and in accordance with the Conditions of Engagement for the commission supplied at the time of proposal. Unauthorised use of this report in any form whatsoever is prohibited.

Borehole and Test Pit Logs

The borehole and test pit logs presented in this report 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 or excavation. 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 and test pits 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 or pits, the frequency of sampling, and the possibility of other than 'straight line' variations between the test locations.

Groundwater

Where groundwater levels are measured in boreholes there are several potential problems, namely:

- In low permeability soils groundwater may enter the hole very slowly or perhaps not at all during the time the hole 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; and
- The use of water or mud as a drilling fluid will mask any groundwater inflow. Water has to be blown out of the hole and drilling mud must first be washed out of the hole if water measurements 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 advisable in low permeability soils or where there may be interference from a perched water table.

Reports

The report has been prepared by qualified personnel, is based on the information obtained from field and laboratory testing, and has been undertaken to current engineering standards of interpretation and analysis. Where the report has been prepared for a specific design proposal, the information and interpretation may not be relevant if the design proposal is changed. If this happens, DP 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 conditions, discussion of geotechnical and environmental aspects, and recommendations or suggestions for design and construction. However, DP cannot always anticipate or assume responsibility for:

- Unexpected variations in ground conditions. The potential for this will depend partly on borehole or pit spacing and sampling frequency;
- Changes in policy or interpretations of policy by statutory authorities; or
- The actions of contractors responding to commercial pressures.

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

About this Report

Site Anomalies

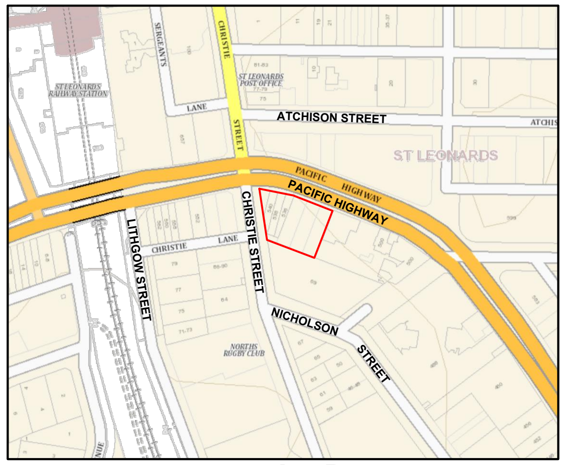
In the event that conditions encountered on site during construction appear to vary from those which were expected from the information contained in the report, DP requests that it be immediately notified. Most problems are much more readily resolved when conditions are exposed rather than at some later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

Information for Contractual Purposes

Where information obtained from this report 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 specially edited document. DP 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 and environmental 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.



Locality Plan

LEGEND

- Approximate Site Boundary
- Approximate Basement Outline
- Previous Borehole Location (DP, 85301.00, 2016)
- Geological Cross Section

NOTE:
1: Base image from MetroMap (Dated 09.02.2022)
2: Basement outline is adapted from DKO Architecture (NSW) Pty Ltd, Project No. 00013070, Drawing No. DA200, Revision 01 (Dated 18.07.2022)

0 1 2 4 6 8 10 15 20 30m
1:300 @ A3

Douglas Partners
Geotechnics / Environment / Groundwater

CLIENT: GFM Investment Management Limited as Trustee for GFM Home Trust		TITLE: **Previous Test Location Plan** **Proposed Mixed-Use Development** **524-542 Pacific Highway, St Leonards**
OFFICE: Sydney	DRAWN BY: MG	
SCALE: 1:300 @ A3	DATE: 22.07.2022	

N

PROJECT No:	85301.01
DRAWING No:	1
REVISION:	0

Sampling

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

Disturbed samples taken during drilling provide 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 it to obtain 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.

Test Pits

Test pits are usually excavated with a backhoe or an excavator, allowing close examination of the in-situ soil if it is safe to enter into the pit. The depth of excavation is limited to about 3 m for a backhoe and up to 6 m for a large excavator. A potential disadvantage of this investigation method is the larger area of disturbance to the site.

Large Diameter Augers

Boreholes can be drilled using a rotating plate or short spiral auger, generally 300 mm or larger in diameter commonly mounted on a standard piling rig. The cuttings are returned to the surface at intervals (generally not more than 0.5 m) 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 samples.

Continuous Spiral Flight Augers

The borehole is advanced using 90-115 mm diameter continuous spiral flight augers which are withdrawn at intervals to allow sampling or in-situ testing. This is a relatively economical means of drilling in clays and sands above the water table. Samples are returned to the surface, or may be collected after withdrawal of the auger flights, but they are disturbed and may be mixed with soils from the sides of the hole. Information from the drilling (as distinct from specific sampling by SPTs or undisturbed samples) is of relatively low reliability, due to the remoulding, possible mixing or softening of samples by groundwater.

Non-core Rotary Drilling

The borehole is advanced using a rotary bit, with water or drilling mud 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 the rate of penetration. Where drilling mud is used this can mask the cuttings and reliable identification is only possible from separate sampling such as SPTs.

Continuous Core Drilling

A continuous core sample can be obtained using a diamond tipped core barrel, usually with a 50 mm internal diameter. Provided full core recovery is achieved (which is not always possible in weak rocks and granular soils), this technique provides a very reliable method of investigation.

Standard Penetration Tests

Standard penetration tests (SPT) are used as a means of estimating the density or strength of soils and also of obtaining a relatively undisturbed sample. The test procedure 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 50 mm diameter split sample tube under the impact of a 63 kg hammer with a free fall of 760 mm. It is normal for the tube to be driven in three successive 150 mm increments and the 'N' value is taken as the number of blows for the last 300 mm. In dense sands, very hard clays or weak rock, the full 450 mm 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 150 mm of, say, 4, 6 and 7 as:

  4,6,7
  N=13

- In the case where the test is discontinued before the full penetration depth, say after 15 blows for the first 150 mm and 30 blows for the next 40 mm as:

  15, 30/40 mm

The results of the SPT tests can be related empirically to the engineering properties of the soils.

Dynamic Cone Penetrometer Tests / Perth Sand Penetrometer Tests

Dynamic penetrometer tests (DCP or PSP) are carried out by driving a steel rod into the ground using a standard weight of hammer falling a specified distance. As the rod penetrates the soil the number of blows required to penetrate each successive 150 mm depth are recorded. Normally there is a depth limitation of 1.2 m, but this may be extended in certain conditions by the use of extension rods. Two types of penetrometer are commonly used.

- Perth sand penetrometer - a 16 mm diameter flat ended rod is driven using a 9 kg hammer dropping 600 mm (AS 1289, Test 6.3.3). This test was developed for testing the density of sands and is mainly used in granular soils and filling.
- Cone penetrometer - a 16 mm diameter rod with a 20 mm diameter cone end is driven using a 9 kg hammer dropping 510 mm (AS 1289, Test 6.3.2). This test was developed initially for pavement subgrade investigations, and correlations of the test results with California Bearing Ratio have been published by various road authorities.

Description and Classification Methods

The methods of description and classification of soils and rocks used in this report are generally based on Australian Standard AS1726:2017, Geotechnical Site Investigations. In general, the descriptions include strength or density, colour, structure, soil or rock type and inclusions.

Soil Types

Soil types are described according to the predominant particle size, qualified by the grading of other particles present:

Type	Particle size (mm)
Boulder	>200
Cobble	63 - 200
Gravel	2.36 - 63
Sand	0.075 - 2.36
Silt	0.002 - 0.075
Clay	<0.002

The sand and gravel sizes can be further subdivided as follows:

Type	Particle size (mm)
Coarse gravel	19 - 63
Medium gravel	6.7 - 19
Fine gravel	2.36 – 6.7
Coarse sand	0.6 - 2.36
Medium sand	0.21 - 0.6
Fine sand	0.075 - 0.21

Definitions of grading terms used are:

- Well graded - a good representation of all particle sizes
- Poorly graded - an excess or deficiency of particular sizes within the specified range
- Uniformly graded - an excess of a particular particle size
- Gap graded - a deficiency of a particular particle size with the range

The proportions of secondary constituents of soils are described as follows:

In fine grained soils (>35% fines)

Term	Proportion of sand or gravel	Example
And	Specify	Clay (60%) and Sand (40%)
Adjective	>30%	Sandy Clay
With	15 – 30%	Clay with sand
Trace	0 - 15%	Clay with trace sand

In coarse grained soils (>65% coarse)
- with clays or silts

Term	Proportion of fines	Example
And	Specify	Sand (70%) and Clay (30%)
Adjective	>12%	Clayey Sand
With	5 - 12%	Sand with clay
Trace	0 - 5%	Sand with trace clay

In coarse grained soils (>65% coarse)
- with coarser fraction

Term	Proportion of coarser fraction	Example
And	Specify	Sand (60%) and Gravel (40%)
Adjective	>30%	Gravelly Sand
With	15 - 30%	Sand with gravel
Trace	0 - 15%	Sand with trace gravel

The presence of cobbles and boulders shall be specifically noted by beginning the description with ‘Mix of Soil and Cobbles/Boulders’ with the word order indicating the dominant first and the proportion of cobbles and boulders described together.

Soil Descriptions

Cohesive Soils

Cohesive soils, such as clays, are classified on the basis of undrained shear strength. The strength may be measured by laboratory testing, or estimated by field tests or engineering examination. The strength terms are defined as follows:

Description	Abbreviation	Undrained shear strength (kPa)
Very soft	VS	<12
Soft	S	12 - 25
Firm	F	25 - 50
Stiff	St	50 - 100
Very stiff	VSt	100 - 200
Hard	H	>200
Friable	Fr	-

Cohesionless Soils

Cohesionless soils, such as clean sands, are classified on the basis of relative density, generally from the results of standard penetration tests (SPT), cone penetration tests (CPT) or dynamic penetrometers (PSP). The relative density terms are given below:

Relative Density	Abbreviation	Density Index (%)
Very loose	VL	<15
Loose	L	15-35
Medium dense	MD	35-65
Dense	D	65-85
Very dense	VD	>85

Soil Origin

It is often difficult to accurately determine the origin of a soil. Soils can generally be classified as:

- Residual soil - derived from in-situ weathering of the underlying rock;
- Extremely weathered material – formed from in-situ weathering of geological formations. Has soil strength but retains the structure or fabric of the parent rock;
- Alluvial soil – deposited by streams and rivers;
- Estuarine soil – deposited in coastal estuaries;
- Marine soil – deposited in a marine environment;
- Lacustrine soil – deposited in freshwater lakes;
- Aeolian soil – carried and deposited by wind;
- Colluvial soil – soil and rock debris transported down slopes by gravity;
- Topsoil – mantle of surface soil, often with high levels of organic material.
- Fill – any material which has been moved by man.

Moisture Condition – Coarse Grained Soils

For coarse grained soils the moisture condition should be described by appearance and feel using the following terms:

- Dry (D) Non-cohesive and free-running.
- Moist (M) Soil feels cool, darkened in colour.

  Soil tends to stick together.

  Sand forms weak ball but breaks easily.
- Wet (W) Soil feels cool, darkened in colour.

  Soil tends to stick together, free water forms when handling.

Moisture Condition – Fine Grained Soils

For fine grained soils the assessment of moisture content is relative to their plastic limit or liquid limit, as follows:

- 'Moist, dry of plastic limit' or 'w <PL' (i.e. hard and friable or powdery).
- 'Moist, near plastic limit' or 'w ≈ PL (i.e. soil can be moulded at moisture content approximately equal to the plastic limit).
- 'Moist, wet of plastic limit' or 'w >PL' (i.e. soils usually weakened and free water forms on the hands when handling).
- 'Wet' or 'w ≈LL' (i.e. near the liquid limit).
- 'Wet' or 'w >LL' (i.e. wet of the liquid limit).

Rock Descriptions

Rock Strength

Rock strength is defined by the Unconfined Compressive Strength and it refers to the strength of the rock substance and not the strength of the overall rock mass, which may be considerably weaker due to defects.

The Point Load Strength Index $Is_{(50)}$ is commonly used to provide an estimate of the rock strength and site specific correlations should be developed to allow UCS values to be determined. The point load strength test procedure is described by Australian Standard AS4133.4.1-2007. The terms used to describe rock strength are as follows:

Strength Term	Abbreviation	Unconfined Compressive Strength MPa	Point Load Index * $Is_{(50)}$ MPa
Very low	VL	0.6 - 2	0.03 - 0.1
Low	L	2 - 6	0.1 - 0.3
Medium	M	6 - 20	0.3 - 1.0
High	H	20 - 60	1 - 3
Very high	VH	60 - 200	3 - 10
Extremely high	EH	>200	>10

* Assumes a ratio of 20:1 for UCS to $Is_{(50)}$. It should be noted that the UCS to $Is_{(50)}$ ratio varies significantly for different rock types and specific ratios should be determined for each site.

Degree of Weathering

The degree of weathering of rock is classified as follows:

Term	Abbreviation	Description
Residual Soil	RS	Material is weathered to such an extent that it has soil properties. Mass structure and material texture and fabric of original rock are no longer visible, but the soil has not been significantly transported.
Extremely weathered	XW	Material is weathered to such an extent that it has soil properties. Mass structure and material texture and fabric of original rock are still visible
Highly weathered	HW	The whole of the rock material is discoloured, usually by iron staining or bleaching to the extent that the colour of the original rock is not recognisable. Rock strength is significantly changed by weathering. Some primary minerals have weathered to clay minerals. Porosity may be increased by leaching, or may be decreased due to deposition of weathering products in pores.
Moderately weathered	MW	The whole of the rock material is discoloured , usually by iron staining or bleaching to the extent that the colour of the original rock is not recognisable, but shows little or no change of strength from fresh rock.
Slightly weathered	SW	Rock is partially discoloured with staining or bleaching along joints but shows little or no change of strength from fresh rock.
Fresh	FR	No signs of decomposition or staining.
Note: If HW and MW cannot be differentiated use DW (see below)		
Distinctly weathered	DW	Rock strength usually changed by weathering. The rock may be highly discoloured, usually by iron staining. Porosity may be increased by leaching or may be decreased due to deposition of weathered products in pores.

Rock Descriptions

Degree of Fracturing

The following classification applies to the spacing of natural fractures in diamond drill cores. It includes bedding plane partings, joints and other defects, but excludes drilling breaks.

Term	Description
Fragmented	Fragments of <20 mm
Highly Fractured	Core lengths of 20-40 mm with occasional fragments
Fractured	Core lengths of 30-100 mm with occasional shorter and longer sections
Slightly Fractured	Core lengths of 300 mm or longer with occasional sections of 100-300 mm
Unbroken	Core contains very few fractures

Rock Quality Designation

The quality of the cored rock can be measured using the Rock Quality Designation (RQD) index, defined as:

$$\text{RQD \%} = \frac{\text{cumulative length of 'sound' core sections > 100 mm long}}{\text{total drilled length of section being assessed}}$$

where 'sound' rock is assessed to be rock of low strength or stronger. The RQD applies only to natural fractures. If the core is broken by drilling or handling (i.e. drilling breaks) then the broken pieces are fitted back together and are not included in the calculation of RQD.

Stratification Spacing

For sedimentary rocks the following terms may be used to describe the spacing of bedding partings:

Term	Separation of Stratification Planes
Thinly laminated	< 6 mm
Laminated	6 mm to 20 mm
Very thinly bedded	20 mm to 60 mm
Thinly bedded	60 mm to 0.2 m
Medium bedded	0.2 m to 0.6 m
Thickly bedded	0.6 m to 2 m
Very thickly bedded	> 2 m

Symbols & Abbreviations

Douglas Partners

Introduction

These notes summarise abbreviations commonly used on borehole logs and test pit reports.

Drilling or Excavation Methods

C	Core drilling
R	Rotary drilling
SFA	Spiral flight augers
NMLC	Diamond core - 52 mm dia
NQ	Diamond core - 47 mm dia
HQ	Diamond core - 63 mm dia
PQ	Diamond core - 81 mm dia

Water

▷	Water seep
▽	Water level

Sampling and Testing

A	Auger sample
B	Bulk sample
D	Disturbed sample
E	Environmental sample
U_{50}	Undisturbed tube sample (50mm)
W	Water sample
pp	Pocket penetrometer (kPa)
PID	Photo ionisation detector
PL	Point load strength Is(50) MPa
S	Standard Penetration Test
V	Shear vane (kPa)

Description of Defects in Rock

The abbreviated descriptions of the defects should be in the following order: Depth, Type, Orientation, Coating, Shape, Roughness and Other. Drilling and handling breaks are not usually included on the logs.

Defect Type

B	Bedding plane
Cs	Clay seam
Cv	Cleavage
Cz	Crushed zone
Ds	Decomposed seam
F	Fault
J	Joint
Lam	Lamination
Pt	Parting
Sz	Sheared Zone
V	Vein

Orientation

The inclination of defects is always measured from the perpendicular to the core axis.

h	horizontal
v	vertical
sh	sub-horizontal
sv	sub-vertical

Coating or Infilling Term

cln	clean
co	coating
he	healed
inf	infilled
stn	stained
ti	tight
vn	veneer

Coating Descriptor

ca	calcite
cbs	carbonaceous
cly	clay
fe	iron oxide
mn	manganese
slt	silty

Shape

cu	curved
ir	irregular
pl	planar
st	stepped
un	undulating

Roughness

po	polished
ro	rough
sl	slickensided
sm	smooth
vr	very rough

Other

fg	fragmented
bnd	band
qtz	quartz

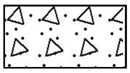
Symbols & Abbreviations

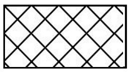
Graphic Symbols for Soil and Rock

General

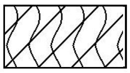
 Asphalt

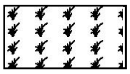
 Road base

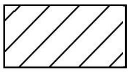
 Concrete

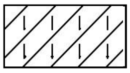
 Filling

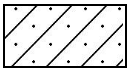
Soils

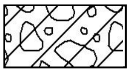
 Topsoil

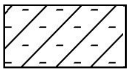
 Peat

 Clay

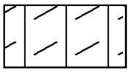
 Silty clay

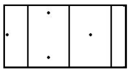
 Sandy clay

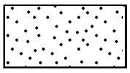
 Gravelly clay

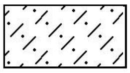
 Shaly clay

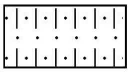
 Silt

 Clayey silt

 Sandy silt

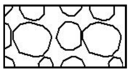
 Sand

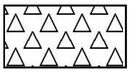
 Clayey sand

 Silty sand

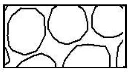
 Gravel

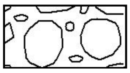
 Sandy gravel

 Cobbles, boulders

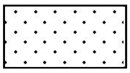
 Talus

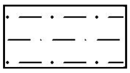
Sedimentary Rocks

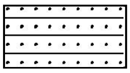
 Boulder conglomerate

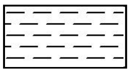
 Conglomerate

 Conglomeratic sandstone

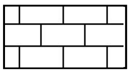
 Sandstone

 Siltstone

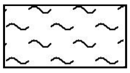
 Laminite

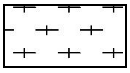
 Mudstone, claystone, shale

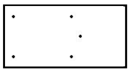
 Coal

 Limestone

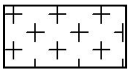
Metamorphic Rocks

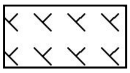
 Slate, phyllite, schist

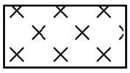
 Gneiss

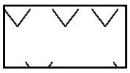
 Quartzite

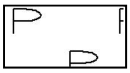
Igneous Rocks

 Granite

 Dolerite, basalt, andesite

 Dacite, epidote

 Tuff, breccia

 Porphyry

BOREHOLE LOG

CLIENT: GFM Investment Management Limited
PROJECT: Proposed Mixed-Use Development
LOCATION: 524 - 542 Pacific Highway, St Leonards

SURFACE LEVEL: 82m AHD*
EASTING: 333025
NORTHING: 6255932
DIP/AZIMUTH: 90°/--

BORE No: 1
PROJECT No: 85301.00
DATE: 12/3/2016
SHEET 1 OF 2

Column headings: RL | Depth (m) | Description of Strata | Degree of Weathering (EW, HW, MW, SW, FS, FR) | Graphic Log | Rock Strength (Ex Low, Very Low, Low, Medium, High, Very High, Ex High) | Water | Fracture Spacing (m) (0.01, 0.05, 0.10, 0.50, 1.00) | Discontinuities (B - Bedding, J - Joint, S - Shear, F - Fault) | Sampling & In Situ Testing: Type, Core Rec. %, RQD %, Test Results & Comments

Depth scale: 1, 2, 3, 4, 5, 6, 7, 8, 9

Depth (m)	Description of Strata
	CONCRETE - 2 x 100mm slabs
0.2	FILLING - brown, slightly sandy, silty clay filling with some medium to coarse slag gravel and a trace of brick, steel, charcoal and shells, moist
0.7	FILLING - grey, slightly clayey, shale cobble filling
1.2	SHALE - extremely low to very low strength, grey shale 1.7m: very low to low strength
2.0	SHALE - low to medium strength, highly weathered, fractured, dark grey shale with very low strength bands
3.95	SILTSTONE - very low and low strength, highly weathered, fragmented to fractured, pale grey siltstone
5.6	LAMINITE - very low strength, highly weathered, fractured, pale grey to grey laminite with approximately 25% fine sandstone laminations
8.4	SANDSTONE - low then medium strength, moderately weathered, slightly fractured to fractured, pale grey, fine grained sandstone

Discontinuities:

Note: Unless otherwise stated, rock is fractured along rough planar bedding dipping 0°- 10°

- 2.0-2.07m: fg
- 2.09m: J90°, pl, ro, cln
- 2.1-2.17m: B (x9) 0°- 10°, pl-un, ro, cly vn-cln
- 2.15m: J45°, pl, ro, cln
- 2.33-2.52m: J (x4) 25°- 35°, pl, sm, fe
- 2.52-2.55m: fg
- 2.58m: J60°- 75°, cu, ro, cly vn
- 2.7-2.73m: fg
- 3.1-3.21m: B (x6) 0°, pl, sm, cly vn
- 3.43m: J60°, pl-un, sm, cln
- 3.49m: J (x2) 65°- 80°, un, ro, cln
- 3.94m: J90°, pl, ro, cln
- 4.62m: J45°, pl, sm, cly
- 5.64-5.71m: J (x2) 45°, pl, ro, cln
- 6.44m: J45°, pl, ro, cln
- 6.55m: J40°, pl, ro, cly (possible fault)
- 7.85m: J45°, pl, ro, cln
- 9.14-9.21m: B (x3) 5°- 10°, pl-un, ro, cln-cly vn
- 9.25m: B (x3) 5°, ti, cly, 2-4mm
- 9.27-9.53m: B (x5) pl-un, ro, cln

Sampling & In Situ Testing:

Type	Core Rec. %	RQD %	Test Results & Comments
D			
A			
C	100	35	PL(A) = 0.3 PL(A) = 0.4 PL(A) = 0.2
C	100	13	PL(A) = 0.1 PL(A) = 0.1 PL(A) = 0.1
C	100	40	PL(A) = 0.2 PL(A) = 0.4

RIG: Bobcat **DRILLER:** GM **LOGGED:** KM/SI **CASING:** HW to 2.0m
TYPE OF BORING: NDD t o1.2m; Solid flight auger to 2.0m; NMLC-Coring to 13.73m
WATER OBSERVATIONS: No free groundwater observed whilst augering
REMARKS: *Surface level determined by interpolation of published survey data

SAMPLING & IN SITU TESTING LEGEND

A	Auger sample	G	Gas sample	PID	Photo ionisation detector (ppm)
B	Bulk sample	P	Piston sample	PL(A)	Point load axial test Is(50) (MPa)
BLK	Block sample	U_x	Tube sample (x mm dia.)	PL(D)	Point load diametral test Is(50) (MPa)
C	Core drilling	W	Water sample	pp	Pocket penetrometer (kPa)
D	Disturbed sample	▷	Water seep	S	Standard penetration test
E	Environmental sample	▼	Water level	V	Shear vane (kPa)

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BOREHOLE LOG

CLIENT: GFM Investment Management Limited
PROJECT: Proposed Mixed-Use Development
LOCATION: 524 - 542 Pacific Highway, St Leonards

SURFACE LEVEL: 82m AHD*
EASTING: 333025
NORTHING: 6255932
DIP/AZIMUTH: 90°/--

BORE No: 1
PROJECT No: 85301.00
DATE: 12/3/2016
SHEET 2 OF 2

RL	Depth (m)	Description of Strata	Degree of Weathering (EW HW MW SW FS FR)	Graphic Log	Rock Strength (Ex Low, Very Low, Low, Medium, High, Very High, Ex High)	Water	Fracture Spacing (m) (0.01, 0.05, 0.10, 0.50, 1.00)	Discontinuities (B - Bedding, J - Joint, S - Shear, F - Fault)	Type	Core Rec. %	RQD %	Test Results & Comments
		SANDSTONE - low then medium strength, moderately weathered, slightly fractured to fractured, pale grey, fine grained sandstone *(continued)*						10.15m: B0°, cly co, 3mm	C	100	40	PL(A) = 0.5
	10.65	SANDSTONE - high strength, slightly weathered to fresh, slightly fractured, pale grey and pale brown, fine to medium grained sandstone, trace carbonaceous laminations on crossbeds						10.62m: B0°, cly, 30mm 10.67m: B0°, pl, ro, fe 10.94-11.53m: B (x2) 0°, pl, ro, cln 12.01m: B5°, ti, cly, 20mm 12.24-12.48m: B (x3) 0°-4°, pl, ro, cln-fg, 5mm	C	100	95	PL(A) = 1.3 PL(A) = 1.8
	12.4	SANDSTONE - high strength, fresh, slightly fractured, pale grey and pale brown, medium grained sandstone						13.15-13.2m: Cs 13.37m: B0°, cly, 5mm & J (x2) 80°, pl, ro, cln				PL(A) = 1.5
	13.73	Bore discontinued at 13.73m										

Depth scale: 11, 12, 13, 14, 15, 16, 17, 18, 19

RIG: Bobcat **DRILLER:** GM **LOGGED:** KM/SI **CASING:** HW to 2.0m
TYPE OF BORING: NDD t o1.2m; Solid flight auger to 2.0m; NMLC-Coring to 13.73m
WATER OBSERVATIONS: No free groundwater observed whilst augering
REMARKS: *Surface level determined by interpolation of published survey data

SAMPLING & IN SITU TESTING LEGEND

A	Auger sample	G	Gas sample	PID	Photo ionisation detector (ppm)
B	Bulk sample	P	Piston sample	PL(A)	Point load axial test Is(50) (MPa)
BLK	Block sample	U_x	Tube sample (x mm dia.)	PL(D)	Point load diametral test Is(50) (MPa)
C	Core drilling	W	Water sample	pp	Pocket penetrometer (kPa)
D	Disturbed sample	▷	Water seep	S	Standard penetration test
E	Environmental sample	▼	Water level	V	Shear vane (kPa)

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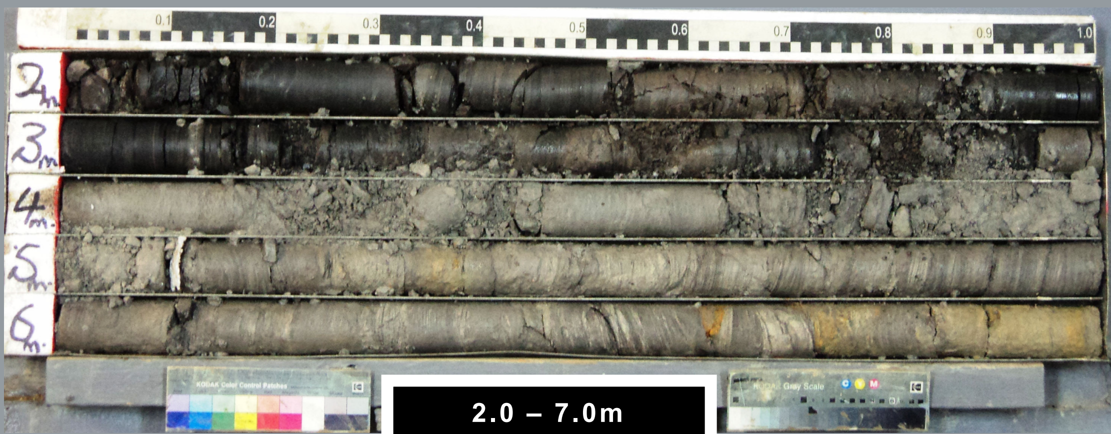
Proposed Mixed-Use Development – ST. LEONARDS

BORE 1 PROJECT 85301 MAR 2016



2.0 – 7.0m

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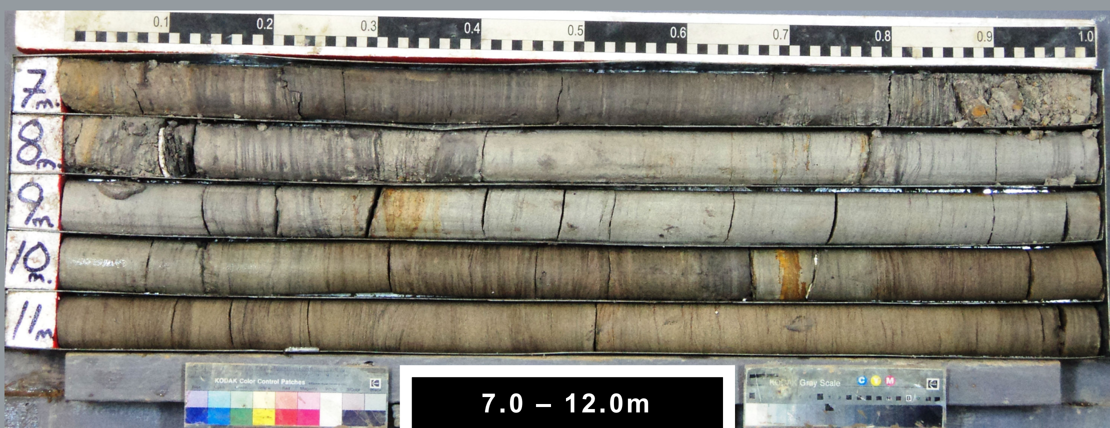
BORE 1 PROJECT 85301 MAR 2016



7.0 – 12.0m

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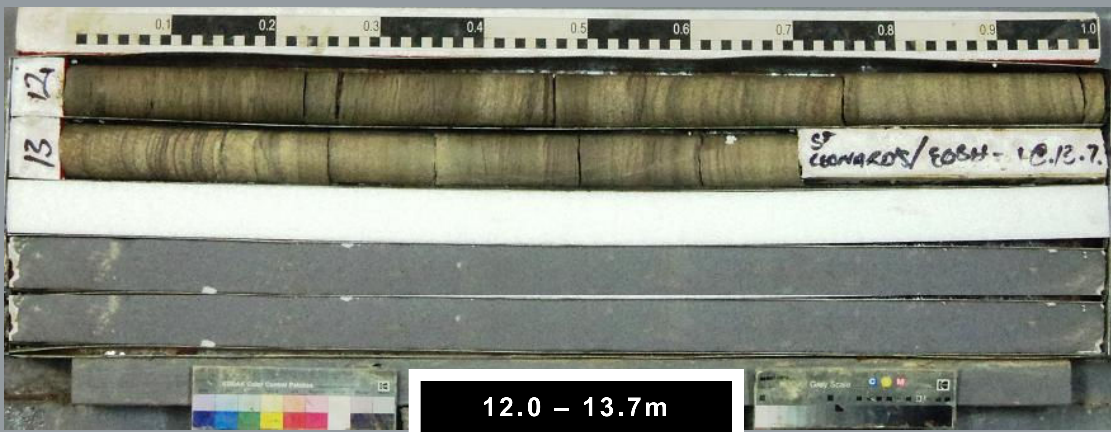
BORE 1 PROJECT 85301 MAR 2016



12.0 – 13.7m