REPORT TO

MARY CURTIS

ON

GEOTECHNICAL AND HYDROGEOLOGICAL INVESTIGATION

FOR

PROPOSED ALTERATIONS AND ADDITIONS

AT

68E CALEDONIA STREET, PADDINGTON, NSW

Date: 12 June 2020
Ref: 33171Arpt

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ATTACHMENTS

1 INTRODUCTION

This report presents the results of a geotechnical and hydrogeological investigation for the proposed alterations and additions at 68E Caledonia Street, Paddington, NSW. The location of the site is shown in Figure 1. The investigation was commissioned by Mary Curtis by signed 'Acceptance of Proposal' form dated 4 May 2020. The commission was on the basis of our fee proposal, Ref. P51723A dated 29 April 2020.

Based on the provided architectural drawings prepared by Ecosystem Architecture (Project No. 1001, Drawing Nos. DA 00^{Rev2}, DA-01^{Rev5}, DA-02^{Rev5}, DA-03^{Rev7}, DA-04^{Rev9}, DA-05^{Rev9}, DA-06^{Rev9}, DA-07^{Rev10}, DA-08^{Rev12}, DA-09^{Rev8} & DA-10^{Rev8}), we understand the proposed development will comprise:

- Partial demolition of the existing two-storey terrace house, which shares a party wall with the neighbouring terrace house to the west (68D Caledonia Street).

- Construction of a basement level below the retained portion of the terrace house. The basement floor will be constructed at RL64.915m and will require excavation to depths between approximately 2.3m and 3.3m below existing grade. The survey datum is the Australian Height Datum (AHD). The western wall of the proposed basement will be set back 0.4m from the centreline of the party wall. The eastern wall of the proposed basement will mostly extend below the retained eastern wall of the terrace house and abut the eastern site boundary. At the northern end however, the eastern wall of the proposed basement level will be set back approximately 0.5m from the eastern site boundary, and will be constructed in front of the retained eastern wall of the terrace house. The northern and southern ends of the proposed basement level will generally extend below the footprint of the terrace house, except at the southern end where a 2.79m wide portion will protrude a horizontal distance 1.33m beyond the footprint.

- Construction of a hydraulic lift. The proposed over-run pit will require a slab set-down of 0.2m.

- Internal alterations to the ground floor and first floor levels.

- An attic conversion over the southern two-thirds of the terrace house, and construction of a roof garden over the northern third.

- Excavation for a 5000L rainwater tank below the front parking area. We have assumed that the proposed rainwater tank will require excavation to a maximum depth of about 1.4m.

Structural loads typical for this type of development have been assumed.

In 2011, Jeffery and Katauskas Pty Ltd (now trading as JK Geotechnics) completed a limited scope geotechnical investigation at the site; report Ref. 22472Zrpt dated 7 July 2011 [JKG 2017]. For JKG 2017, two hand augered boreholes (BH2 & BH3) were completed to refusal depths of 0.45m and 0.6m, respectively, and three Dynamic Cone Penetrometer (DCP) tests (DCP1 to DCP3) were completed to refusal depths between 0.18m and 1.78m. For ease of cross reference, the borehole logs and DCP test results sheet from JKG 2017 are attached. The previous borehole and DCP test locations have been plotted on the attached Figure 2.

The purpose of the current investigation was to further assess the subsurface conditions at one cored borehole location and, based on the information obtained, to present our updated comments and recommendations on site preparation, underpinning, excavation conditions, drainage, excavation support, basement walls, new footings, basement floor slab and hydrogeological issues. This report supersedes the advice provided in JKG 2017.

This geotechnical investigation was carried out in conjunction with a 'Preliminary (Stage 1) Site Investigation' by our environmental division, JK Environments (JKE). Reference should be made to the separate report by JKE, Ref. E33171PLrpt.

2 CURRENT INVESTIGATION PROCEDURE

The fieldwork for the investigation was carried out on 11 May 2020 and comprised the drilling of one borehole (BH101), at the location shown on Figure 2, to 5.95m depth below existing grade. Prior to the commencement of the fieldwork, a specialist sub-consultant reviewed available 'Dial Before You Dig' information and electro-magnetically scanned the borehole location for buried services.

The soil profile was hand auger drilled to refusal at 0.95m depth. A buried concrete slab within the soil profile was diatube cored with water flush. A DCP test was completed adjacent to the borehole, also to refusal, to assess the relative compaction/relative density of the soil profile. The underlying bedrock was diamond core drilled using our portable hydraulically powered Melvelle drill rig with a TT56 twin tube core barrel and water flush. The strength of the cored bedrock was assessed by examination of the recovered rock core, together with correlations with subsequent laboratory Point Load Strength Index ($I_{S(50)}$) test results. Groundwater observations were also made in the borehole.

The borehole location was set out by tape measurements from existing surface features. The surface RL indicated on the attached borehole log and DCP test results sheet was interpolated between spot level heights shown on the supplied survey plan (Drawing No. DA-01^{Rev5}), and is therefore approximate. The survey datum is AHD. The survey plan forms the basis of Figure 2.

Further details of the methods and procedures employed in the investigation, including the penetration limitations of the hand auger and DCP, are presented in the attached Report Explanation Notes.

Our geotechnical engineer (Arthur Kourtesis) was present full-time during the fieldwork to set out the borehole, direct the electro-magnetic scanning, nominate testing and sampling, and to prepare the attached borehole log and DCP test results sheet. The Report Explanation Notes define the logging terms and symbols used.

The recovered rock core was photographed and returned to a NATA accredited laboratory, Soil Test Services Pty Ltd (STS), for Point Load Strength Index testing. The rock core photograph is enclosed. The Point Load Strength Index test results are plotted on the borehole log and summarised in the attached STS Table A. The unconfined compressive strengths (UCS), as estimated from the Point Load Strength Index test results, are also summarised in STS Table A.

3 RESULTS OF THE INVESTIGATION

3.1 Site Description

The site is located mid-slope on a north-west-facing hillside, which grades at approximately 1° to 2°. Caledonia Street bounds the site to the south.

At the time of the fieldwork, the site was occupied by a two-storey rendered brick terrace house, which was set back 6m from the southern street frontage and appeared to be in good external condition based on a cursory inspection. The terrace house extended to the eastern and western site boundaries. The front yard was brick paved and provided off street parking. The rear yard was gravel surfaced and contained a timber deck adjacent to the terrace house. The rear yard was surrounded by garden beds, plants and small trees. On the northern and eastern sides of the rear yard were rendered masonry boundary walls. A horizontal crack was present along the northern wall. The eastern wall contained a sub-vertical crack and was slightly leaning. The western side of the rear yard contained a brush boundary fence.

The neighbouring property to the west (68D Caledonia Street) contained a similar two-storey brick terrace, which shared a party wall with the terrace house on the subject site. The neighbouring property to the east (70 Caledonia Street) contained a two-storey brick terrace house, which was set back 1m from the southern street frontage. The neighbouring terrace house to the east abuts the common boundary (and the terrace house on the subject site), but we understand does not share a party wall. The neighbouring terrace houses appeared to be in good condition when viewed from within the subject site and from Caledonia Street. Ground surface levels across the eastern and western site boundaries in the front and rear yards appeared to be similar.

Observations of the neighbouring property to the north (179 Paddington Street) were limited due to the high boundary fence. Notwithstanding, the ground surface level within the neighbouring property is lower in elevation. It is therefore possible that the northern boundary wall is a retaining wall and supports the subject site.

3.2 Subsurface Conditions

The 1:100,000 series geological map of Sydney (Geological Survey of NSW, Geological Series Sheet 9130) indicates the site to be underlain by Hawkesbury Sandstone.

Generally, the previous and current boreholes encountered pavers and/or fill, overlying residual silty sand, then sandstone bedrock at relatively shallow depths. Reference should be made to the attached borehole logs and DCP test results sheets for specific details at each location. A summary of the subsurface conditions encountered in the boreholes and indicated by the DCP test results is provided below:

Pavers

Brick pavers of 50mm and 60mm thickness were encountered in BH2 and BH101, respectively.

Fill

Sandy fill, with sandstone gravel, sandstone cobble, brick and plastic fragment inclusions, was encountered below the pavers in BH2 and BH101, and from the surface in BH3 to depths of at least 0.45m, 0.6m and to at least 0.6m, respectively. In BH101, a 120mm thick buried concrete slab was encountered at 0.25m depth. Based on the DCP test results, the fill was generally assessed to be either poorly or moderately compacted. BH2 and BH3 met refusal at depths of 0.45m and 0.6m, respectively, on obstructions in the fill. Similarly, it is more than likely that DCP1 refusal at 0.18m depth occurred on an obstruction in the fill; possibly the buried concrete slab encountered in BH101.

A hydrocarbon odour was detected in the fill profile in BH101.

Residual Silty Sand

Residual silty sand of loose relative density was encountered below the fill in BH101.

Sandstone Bedrock

Sandstone bedrock was encountered in BH101 at 0.95m depth (RL67.0m). The sandstone was generally highly to moderately weathered and of low, medium and high strength, but contained ‘weaker’ bands. Below 5.2m depth, the sandstone was slightly weathered to fresh, and of high strength.

The sandstone contained few rock defects (ie. joints and extremely weathered seams). However, two ‘no core’ (core loss) zones of 0.21m and 0.16m thickness were encountered between 3.9m and 5.0m depth. ‘No core’ zones are usually the result of ‘weaker’ bands being washed out by the drill flush water.

Based on the results of BH101, it is more than likely that DCP2 and DCP3 refusal at depths of 1.03m and 1.78m, respectively, occurred on, or a short distance above, the bedrock surface. However, due to the limitations of the DCP, the stratum on which refusal occurred cannot be confirmed.

Groundwater

The previous and current boreholes were ‘dry’ during and on completion of hand augering. No meaningful groundwater observations were made on completion of rock coring in BH101 due to the introduced drill flush water. We note that groundwater levels may not have stabilised within the relatively short observation period. No long-term groundwater level monitoring was carried out.

3.3 Laboratory Test Results

The results of the Point Load Strength Index tests carried out on the recovered rock core from BH101 correlated well with our field assessment of bedrock strength. The estimated UCS's, based on the correlation provided in AS1726:2017 'Geotechnical Site Investigations' (ie. UCS = 20 x $I_{S(50)}$), ranged from 2MPa to 24MPa.

4 COMMENTS AND RECOMMENDATIONS

4.1 Site Preparation

Prior to the commencement of any site work, we recommend that detailed dilapidation survey reports be compiled on the neighbouring terrace houses to the west (68D Caledonia Street) and east (70 Caledonia Street). The dilapidation reports can be used as a benchmark against which to set vibration limits for trafficking of plant and rock excavation, and for assessing possible future claims for damage arising from the works.

The respective owners of the neighbouring properties should be asked to confirm in writing that the dilapidation reports present a fair assessment of existing conditions. As dilapidation reports are relied upon for the assessment of potential damage claims, they must be carried out thoroughly by reputable companies with all defects rigorously described (ie. defect type, defect location, crack width, crack length etc) and photographed. The dilapidation survey reports should be reviewed by JK Geotechnics (JKG).

The proposed development will require internal demolition of the existing terrace house and the front brick paving. Following this, any root affected soils, deleterious or contaminated fill should be stripped. Reference should be made to Section 5 for guidance on the offsite disposal of site soils. Care must be taken during demolition, site stripping and excavation not to undermine or remove support from the party wall along the western site boundary and the retained wall along the eastern boundary.

Due to the presence of poorly compacted/loose surface sandy soils which most likely extend beyond the site boundaries, we recommend that movements of all tracked plant be carried out with caution. Sudden stop/start movements may result in ground vibration damage to the neighbouring terrace houses and boundary walls.

We recommend that full-time quantitative vibration monitoring be carried out on the neighbouring terrace houses to the west (68D Caledonia Street) and east (70 Caledonia Street), whenever tracked plant are operating on site, and during rock excavation. The vibrations on the neighbouring terrace houses should be tentatively limited to a peak particle velocity (PPV) of 5mm/sec, subject to review of the dilapidation survey reports. If the dilapidation reports indicate the neighbouring terrace houses to be in fair to poor condition and/or the terrace houses are heritage listed, then the PPV will need to reduce to 3mm/sec. An acoustic consultant will need to be engaged to carry out the vibration monitoring. The monitoring locations should be jointly nominated by the acoustic consultant and JKG (following review of the dilapidation survey reports).

If it is found that transmitted vibrations are excessive, then further advice should be sought from JKG. Notwithstanding, if higher vibrations are recorded, then they should be assessed against the attached Vibration Emission Design Goals as higher vibrations may be acceptable depending on the associated vibration frequency. Reference should be made to Section 4.3.2 below if it is confirmed that transmitted vibrations are excessive during rock excavation.

Immediately following demolition and prior to bulk excavation commencing, we recommend that a number of test pits be excavated to expose the footing details and foundation materials of the party wall along the western site boundary, and of the retained wall along the eastern site boundary. At the northern end of the proposed basement, test pits will also need to be complete on the abutting wall of the neighbouring terrace house to the east. The structural engineer and JKG should be present to inspect the test pits and to provide advice on underpinning/support, as required.

For the purpose of this report, we have assumed that the existing footings mentioned above are uniformly founded on sandstone bedrock. If the test pits prove this assumption to be incorrect, then the advice provided in this report will need to be reviewed and updated as appropriate.

Ground borne vibrations associated with the use of heavy plant and rock excavation can cause subsidence of the poorly compacted sand fill and loose residual sands, which most likely extend beyond the site boundaries. This subsidence will cause settlement of any nearby high level footings (eg. boundary walls) founded in this material and possible cracking of the structures.

If any works are proposed in the rear yard, then we recommend that the retaining wall details along, or immediately outside, the northern site boundary be investigated. We can provide assistance if requested to do so.

4.2 Underpinning

The retained eastern wall must be underpinned if not already founded on bedrock. The underpins must be constructed in ‘hit one, miss three’ sections, with each section being approximately 0.6m long. Advice on the provision of support to the party wall along the western site boundary, and to the abutting neighbouring footing located on the eastern side of the basement at its northern end, will be submitted if required, following inspection of the test pits.

4.3 Excavation

Prior to any excavation commencing, reference should be made to the NSW Government 'Code of Practice, Excavation Work' dated January 2020.

4.3.1 Excavation Conditions

Excavation of the soil profile can be completed using a hydraulic excavator. Excavation of the medium and high strength sandstone for the proposed basement and rainwater tank will require rock hammers for effective removal. Notwithstanding, an attempt should be undertaken to remove as much of this material as possible using the excavator's 'digging' bucket and/or ripping tyne.

Grid rock sawing the sandstone bedrock would facilitate excavation and would also increase the ground borne vibration path to the nearby terrace houses (ie. would reduce vibrations) provided the base of the saw slot is maintained below excavation level. Dust suppression by spraying with water should be carried out whenever rock saws are being used.

Where soil batter slopes descend to the crest of a cut rock face, then a bench at least 0.5m wide should be provided between the crest of the rock face and the toe of the batter slope. This bench width would provide sufficient space for sand bagging, if groundwater seepage is encountered.

4.3.2 Potential Vibration Risks

As discussed in Section 4.1, rock excavation using a rock hammer will need to be strictly controlled as there may be direct transmission of ground vibrations to the adjacent terrace houses. It would therefore be prudent to commence rock excavation using the smallest rock hammer practical. If it is found that transmitted vibrations are excessive, then it would be necessary to change to alternative rock excavation methods such as a tighter grid of rock saw slots and/or a smaller rock hammer, or rotary grinders, or drill and split methods.

The following procedures are recommended to reduce vibrations if rock hammers are used:

- Maintain rock hammer oriented towards the face and enlarge excavation by breaking small wedges off the face. Grid sawing the base would facilitate removal.

- Operate one hammer at a time and in short bursts only, to reduce amplification of vibrations.

- Use excavation contractors with appropriate experience in confined work and a competent supervisor who is aware of vibration damage risks, possible rock face instability issues, etc. The contractor should have all appropriate statutory and public liability insurances and should be provided with a full copy of this report.

4.3.3 Drainage

Groundwater inflows into the excavation may occur as local seepage flows above the soil/bedrock interface, and through joints and bedding partings within the bedrock profile, particularly after heavy rain. We recommend that all cut faces and retaining walls incorporate spoon drains and/or subsoil drains to intercept potential seepage. Seepage volumes into the excavation are expected to be of limited volume and controllable by sump and pump methods. Discharge from the drainage system should be piped to the stormwater system. The excavation should be monitored by the builder and JKG as it progresses to confirm the drainage requirements.

In the long-term, we consider that a drained structure is suitable for the proposed development.

4.4 Excavation Support

On the basis that the existing footings along the eastern and western sides of the proposed basement are founded on bedrock or will be underpinned/supported to bedrock, and that they have adequate lateral restraint to support the soil behind, the soil profile at the northern and southern ends of the proposed basement excavation should be battered back at no steeper than 1 Vertical (V) on 1.5 Horizontal (H). This temporary batter slope also applies for the proposed rainwater tank excavation. Surcharge loads (including soil stockpiles) must be kept well away from the crests of the temporary batter slopes. Where battering cannot be accommodated and/or is not preferred, then JKG should be contacted for further advice.

Assuming all the exposed sandstone bedrock below the existing terrace house is of at least low strength, the faces can be cut vertically. However, based on the defects encountered in BH101, that could isolate potentially unstable rock wedges, we strongly recommend that all vertically cut rock faces (particularly those below existing footings) be incrementally inspected by an experienced geotechnical engineer or engineering geologist at no more than 1.5m depth increments and on completion of excavation to identify features which may require stabilisation (eg. underpinning, rock bolting and/or shotcrete and mesh). Provision must be made in the construction program and budget for the inspections and stabilisation works.

4.5 Basement Walls

4.5.1 Retention Design Parameters

The basement walls and any underpins which are subjected to lateral earth pressures should be designed using the following parameters:

- Underpins and cantilevered block walls which are propped by the ground floor slab of the permanent structure, should be designed using a triangular lateral earth pressure distribution, with an 'at rest' earth pressure coefficient, K_o, of 0.6, for the soil profile, assuming a horizontal retained surface. A bulk unit weight of $19kN/m^3$ should be adopted for the soil profile.

- All surcharge loads affecting the walls/underpins (such as construction loads, live loads, adjacent high level footings, compaction stresses during backfilling, etc) should be taken into account in the design using the above K_0 value.

- The basement walls/underpins should be designed as drained with measures taken to provide permanent and effective drainage of the ground behind the walls/underpins. For cantilevered block walls, subsurface drains should incorporate (1) an appropriately sized 'ag' pipe with filter sock, surrounded by (2) free draining, single size, durable aggregate, such as 'Blue Metal' gravel or crushed concrete aggregate, and encapsulated within (3) a non-woven geotextile filter fabric such as Bidim A34 to control subsoil erosion. For underpins, weep hole outlets (also known as spitter pipes) will need to be provided a short distance above the rock surface at a horizontal spacing no greater than 1.2m and should incorporate a non-woven geotextile filter fabric (at the inserted end) to reduce subsoil erosion. All drainage water should be piped to the stormwater system.

- Lateral toe restraint may be achieved by concrete-to-rock friction over the base of the underpin/basement wall footing. An effective friction angle of 35° can be adopted. Alternatively, the base of the underpins/basement wall footing could be secured into the bedrock by dowels. For the underpins and basement walls founded on bedrock above bulk excavation level, the dowels should be angled away from the cut rock faces below. Dowels bonded into low strength or stronger sandstone may be designed for a maximum allowable bond stress of 150kPa. Permanent dowels must be designed for corrosion resistance and for long-term durability (for example, hot dipped galvanised with sacrificial thickness). Care must be taken not to undermine the underpins/basement walls when excavating the sandstone below.

4.5.2 Backfilling Behind Basement Walls

Compaction of engineered fill behind basement walls is very difficult, particularly where space is limited. Our preference is to use a single sized durable aggregate, such as 'Blue Metal' gravel or crushed concrete aggregate (free of fines), which do not require significant compactive effort. Such material should be nominally compacted using a hand operated vibrating plate (sled) compactor in maximum 200mm thick loose layers. A non-woven geotextile filter fabric such as Bidim A34 should be placed as a separation layer immediately above the cut batter slope (prior to backfilling) to control subsoil erosion. The geotextile should then be wrapped over the surface of the aggregate backfill and capped with at least a 0.3m thick compacted layer of soil fill.

The advice above also applies to the backfilling of the proposed rainwater tank, if the tank is to be externally drained. If not, then we recommend that the rainwater tank be backfilled with a 'wet' mix of cement stabilised sand (at least 5% cement by dry weight). A vibrating concrete poker should be used during placement. So as to reduce the likelihood of a 'hard spot' developing in the paved surface over the buried tank, we recommend that the cement stabilised sand backfill be capped with at least a 0.3m thick compacted layer of soil fill.

4.6 New Footings

Based on the results of the current investigation, we recommend that new beam and strip footings founded on sandstone bedrock of at least low strength below bulk excavation level can be designed for an allowable bearing pressure of 1,000kPa. New footings and underpins founded behind the crests of vertically cut rock faces, on sandstone of at least low strength, can be designed for an allowable bearing pressure of 600kPa on condition that the rock face below the footing/underpin is progressively inspected by an experienced geotechnical engineer or engineering geologist. We reiterate that adversely oriented rock defects may result in destabilisation of the footing/underpin.

We recommend that all footing/underpin excavations be inspected by a geotechnical engineer (prior to the installation of reinforcement cages) to confirm that a satisfactory bearing stratum has been achieved. This is particularly important for all footings/underpins located outside the basement footprint, which may be founded on potentially unstable bedrock. All footings must be cleaned out prior to inspection and pouring.

The earthquake design parameters are dependent on whether the proposed structure will be isolated away from the poorly compacted/ loose sandy soils. The following parameters can be adopted for earthquake design in accordance with AS1170.4-2007 'Structural Design Actions, Part 4: Earthquake Actions in Australia' (including Amendment Nos. 1 & 2):

- Hazard Factor (Z) = 0.09
- Site Subsoil Class = Class B_e (for a building isolated away from the sandy soils)
- Site Subsoil Class = Class D_e (for a building in contact with the sandy soils)

4.7 Basement Floor Slab

We expect that the basement floor slab will be integrated with, and suspended off, the footing system. The surface of the sandstone bedrock at bulk excavation level will need to be graded to provide good and effective drainage both during construction and in the long-term. The principal aim of the drainage is to promote run-off towards designated sumps by cross-falls and to reduce ponding.

The suspended basement floor slab should be underlain with at least a 100mm thick layer of good quality, durable, single size, crushed rock (free of fines) such as 'Blue Metal' gravel or crushed concrete aggregate, which will also act as underfloor drainage.

The underfloor drainage should include a sump and pump system. The basement wall drains should be connected into the underfloor drainage system. As discussed in Section 4.3.3, groundwater seepage monitoring should be carried out during basement excavation prior to finalising the design of the discharge facility. All sumps should have an automatic level control pump to avoid flooding of the basement level. Outlets into the stormwater system will require Council approval.

4.8 Hydrogeological Issues

Groundwater was not encountered during the previous and current investigations. We anticipate, however, that surface water which infiltrates the ground over the higher lying catchment to the south-east (ie. during and following rainfall events) will flow downslope over the bedrock surface. During these rainfall events, minor groundwater flows through the bedrock (ie. through joints, bedding planes, etc.) may also occur.

The proposed basement will therefore intersect the groundwater flow. However, by providing a drained basement, groundwater throughflow will be permitted, with no build-up of uphill groundwater levels. Also, the proposed drainage will not result in depression of groundwater levels to the extent that the adjoining ground surface will settle or be otherwise adversely affected.

Based on the lack of groundwater encountered during the investigations and provided the recommendations presented in this report with respect to drainage are adopted, the impact of the proposed basement excavation on the groundwater flows at the site and adjoining properties is expected to be negligible.

4.9 Further Geotechnical Input

The following is a summary of the further geotechnical input which is required and which has been detailed in the preceding sections of this report:

- Test pit investigation to confirm the footing details and foundation materials of the party wall along the western site boundary, and of the retained walls along the eastern boundary.
- Additional advice if it is found that the existing footings mentioned above are not uniformly founded on sandstone bedrock.
- Dilapidation survey report on 68D & 70 Caledonia Street.
- Quantitative vibration monitoring on 68D & 70 Caledonia Street during demolition and whenever tracked plant and rock hammers are operating on site.
- Progressive rock face inspections.
- Groundwater seepage monitoring.
- Footing and underpin inspections.

5 GENERAL COMMENTS

The recommendations presented in this report include specific issues to be addressed during the construction phase of the project. In the event that any of the construction phase recommendations presented in this report are not implemented, the general recommendations may become inapplicable and JKG accept no responsibility whatsoever for the performance of the structure where recommendations are not implemented in full and properly tested, inspected and documented.

Occasionally, the subsurface conditions between the completed boreholes and DCP tests may be found to be different (or may be interpreted to be different) from those expected. Variation can also occur with groundwater conditions, especially after climatic changes. If such differences appear to exist, we recommend that you immediately contact this office.

This report provides advice on geotechnical aspects for the proposed civil and structural design. As part of the documentation stage of this project, Contract Documents and Specifications may be prepared based on our report. However, there may be design features we are not aware of or have not commented on for a variety of reasons. The designers should satisfy themselves that all the necessary advice has been obtained. If required, we could be commissioned to review the geotechnical aspects of contract documents to confirm the intent of our recommendations has been correctly implemented.

A waste classification is required for any soil and/or bedrock excavated from the site prior to offsite disposal. Subject to the appropriate testing, material can be classified as Virgin Excavated Natural Material (VENM), Excavated Natural Material (ENM), General Solid, Restricted Solid or Hazardous Waste. Analysis can take up to seven to ten working days to complete, therefore, an adequate allowance should be included in the construction program unless testing is completed prior to construction. If contamination is encountered, then substantial further testing (and associated delays) could be expected. We strongly recommend that this requirement is addressed prior to the commencement of excavation on site.

This report has been prepared for the particular project described and no responsibility is accepted for the use of any part of this report in any other context or for any other purpose. If there is any change in the proposed development described in this report then all recommendations should be reviewed. Copyright in this report is the property of JKG. We have used a degree of care, skill and diligence normally exercised by consulting engineers in similar circumstances and locality. No other warranty expressed or implied is made or intended. Subject to payment of all fees due for the investigation, the client alone shall have a licence to use this report. The report shall not be reproduced except in full.

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SOIL TEST SERVICES

TABLE A
POINT LOAD STRENGTH INDEX TEST REPORT

Client:	JK Geotechnics	**Ref No:**	33171A
Project:	Proposed Alterations and Additions	**Report:**	A
Location:	68E Caledonia Street, Paddington, NSW	**Report Date:**	15/05/2020
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BOREHOLE NUMBER	DEPTH m	$I_{S\ (50)}$ MPa	ESTIMATED UNCONFINED COMPRESSIVE STRENGTH (MPa)
101	1.01 - 1.04	0.5	10
	1.50 - 1.54	1.1	22
	2.00 - 2.04	0.9	18
	2.48 - 2.52	1.0	20
	3.00 - 3.03	0.7	14
	3.49 - 3.52	0.5	10
	4.20 - 4.23	0.1	2
	4.46 - 4.48	0.1	2
	5.05 - 5.08	0.3	6
	5.51 - 5.54	1.2	24

NOTES:

1. In the above table testing was completed in the Axial direction.
2. The above strength tests were completed at the 'as received' moisture content.
3. Test Method: RMS T223.
4. For reporting purposes, the $I_{S(50)}$ has been rounded to the nearest 0.1MPa, or to one significant figure if less than 0.1MPa
5. The Estimated Unconfined Compressive Strength was calculated from the Point Load Strength Index by the following approximate relationship and rounded off to the nearest whole number :
   U.C.S. = 20 $I_{S\ (50)}$

Jeffery and Katauskas Pty Ltd
CONSULTING GEOTECHNICAL AND ENVIRONMENTAL ENGINEERS

BOREHOLE LOG

Borehole No. **2**
1/1

Client: MARY CURTIS
Project: PROPOSED ALTERATIONS AND ADDITIONS
Location: 68E CALEDONIA STREET, PADDINGTON, NSW

Job No. 22472Z **Method:** HAND AUGER **R.L. Surface:** ≈ 67.9m
Date: 9-10-08 **Datum:** ASSUMED
Logged/Checked by: W.W./ A.Z.

Groundwater Record	SAMPLES ES	U50	DB	DS	Field Tests	Depth (m)	Graphic Log	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel. Density	Hand Penetrometer Readings (kPa.)	Remarks
DRY ON COMPLETION					REFER TO DCP TEST RESULTS	0			BRICK PAVERS: 50mm.t				
								-	FILL: Sand, fine to medium grained, brown, with fine to medium grained sandstone gravel, and a trace of plastic.	M	-	-	APPEARS MODERATELY COMPACTED
						0.5			END OF BOREHOLE AT 0.45m				HAND AUGER REFUSAL IN FILL
						1							
						1.5							
						2							
						2.5							
						3							
						3.5							

Jeffery and Katauskas Pty Ltd

CONSULTING GEOTECHNICAL AND ENVIRONMENTAL ENGINEERS

BOREHOLE LOG

Borehole No. 3

1/1

Client: MARY CURTIS

Project: PROPOSED ALTERATIONS AND ADDITIONS

Location: 68E CALEDONIA STREET, PADDINGTON, NSW

Job No. 22472Z **Method:** HAND AUGER **R.L. Surface:** ≈ 67.2m

Date: 9-10-08 **Datum:** ASSUMED

Logged/Checked by: W.W./ A.Z.

Groundwater Record	SAMPLES ES	U50	DB	DS	Field Tests	Depth (m)	Graphic Log	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel. Density	Hand Penetrometer Readings (kPa.)	Remarks
DRY ON COMPLETION					REFER TO DCP TEST RESULTS	0			FILL: Silty clayey sand, fine to medium grained, dark brown, with roots and root fibres, with a trace of fine to medium grained sandstone gravel and crushed bricks and plastic fragments.	M			GARDEN BED APPEARS POORLY COMPACTED
						0.5			FILL: Silty sand, fine to medium grained, brown and light brown, with fine to medium grained sandstone gravel and crushed bricks.	D-M			
									END OF BOREHOLE AT 0.6m				HAND AUGER REFUSAL IN FILL
						1							
						1.5							
						2							
						2.5							
						3							
						3.5							

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BOREHOLE LOG

Borehole No.
101

Client: MARY CURTIS
Project: PROPOSED ALTERATIONS & ADDITIONS
Location: 68E CALEDONIA STREET, PADDINGTON, NSW

Job No.: 33171A | **Method:** HAND AUGER | **R.L. Surface:** ~67.9 m
Date: 11/5/20 | **Datum:** AHD
Plant Type: | **Logged/Checked By:** A.C.K./A.J.

Groundwater Record	SAMPLES ES	U50	DB	DS	Field Tests	RL (m AHD)	Depth (m)	Graphic Log	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel Density	Hand Penetrometer Readings (kPa)	Remarks
DRY ON COMPLETION OF AUGERING					REFER TO DCP TEST RESULTS				-	BRICK PAVERS: 60mm.t	M			HYDROCARBON ODOUR
									-	FILL: Sand, fine to medium grained, light grey brown, with medium to coarse grained sandstone gravel and cobbles.				NO OBSERVED REINFORCEMENT
									-	FILL: Silty sand, fine to medium grained, grey, with medium to coarse grained sandstone gravel.	M			
									SM	CONCRETE: 120mm.t	M	L		RESIDUAL
						67	1			FILL: Silty sand, fine to medium grained, grey, with medium to coarse grained sandstone gravel and clay fines.				HAND AUGER REFUSAL ON SANDSTONE BEDROCK
										Silty SAND: fine to medium grained, light grey mottled light brown.				
										REFER TO CORED BOREHOLE LOG				
						66	2							
						65	3							
						64	4							
						63	5							
						62	6							
						61								

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CORED BOREHOLE LOG

Borehole No.
101
2 / 2

Client: MARY CURTIS
Project: PROPOSED ALTERATIONS & ADDITIONS
Location: 68E CALEDONIA STREET, PADDINGTON, NSW

Job No.: 33171A **Core Size:** TT56 **R.L. Surface:** ~67.9 m
Date: 11/5/20 **Inclination:** VERTICAL **Datum:** AHD
Plant Type: MELVELLE **Bearing:** N/A **Logged/Checked By:** A.C.K./A.J.

Water Loss\Level	RL (m AHD)	Depth (m)	Core Description (Rock Type, grain characteristics, colour, texture and fabric, features, inclusions and minor components)	Weathering	Strength	Point Load Strength Index $I_s(50)$	Defect Details – Description (Type, orientation, defect shape and roughness, defect coatings and seams, openness and thickness) – Specific	Formation
	67		START CORING AT 0.95m					
100% RETURN		1	SANDSTONE: fine to medium grained, light grey and light brown, with red brown and dark brown bands, bedded at 0-20°.	MW	M - H	0.50	(1.16m) J, 15°, P, R, Cn	Hawkesbury Sandstone
							(1.40m) J, 20°, P, R, Cn	
	66					1.1	(1.81m) XWS, 5°, 1 mm.t	
		2				0.90		
					M	1.0		
	65	3				0.70	(3.21m) XWS, 5°, 50 mm.t	
			SANDSTONE: fine to medium grained, light grey and light brown, bedded at 0-20°.			0.50		
	64			HW	VL - L			
		4	NO CORE 0.21m					
			SANDSTONE: fine to medium grained, light grey and light brown, bedded at 0-20°.	HW	VL - L	0.10	(4.30m) XWS, 20°, 30 mm.t	Hawkesbury Sandstone
						0.10		
			as above, but dark brown and red brown.	MW	L - M			
	63		NO CORE 0.16m					
		5	SANDSTONE: fine to medium grained, light grey and red brown, bedded at 0-20°.	MW	L - M	0.30		
			SANDSTONE: fine to medium grained, light grey and light brown, bedded at 0-20°.	SW	H	1.2		
			SANDSTONE: fine to medium grained, light grey.	FR				
	62	6	END OF BOREHOLE AT 5.95 m					
	61							

FRACTURES NOT MARKED ARE CONSIDERED TO BE DRILLING AND HANDLING BREAKS

JK 9.02.4 LIB.GLB Log JK CORED BOREHOLE - MASTER 33171A PADDINGTON.GPJ <<DrawingFile>> 02/06/2020 18:48 10.01.00.01 Datgel Lab and In Situ Tool - DGD | Lib: JK 9.02.4 2019-05-31 Prj: JK 9.01.0 2018-03-20


33171A BH101 START CORING AT 0.95m
1
2
3
4
NO CORE: 0.21m
NO CORE: 0.16m
5
END OF BOREHOLE AT 5.95m

Jeffery and Katauskas Pty Ltd

CONSULTING GEOTECHNICAL AND ENVIRONMENTAL ENGINEERS

DYNAMIC CONE PENETRATION TEST RESULTS

Client:	MARY CURTIS		
Project:	PROPOSED ALTERATIONS AND ADDITIONS		
Location:	68E CALEDONIA STREET, PADDINGTON, NSW		
Job No.	22472Z	Hammer Weight & Drop: 9kg/510mm	
Date:	9-10-08	Rod Diameter: 16mm	
Tested By:	W.W.	Point Diameter: 20mm	

	Number of Blows per 100mm Penetration						
Test Location	RL ~67.9m	RL ~67.9m	RL ~67.2m				
Depth (mm)	**1**	**2**	**3**				
0 - 100	VOID	VOID	SUNK				
100 - 200	10/80mm	2					
200 - 300	REFUSAL	4					
300 - 400		7	1				
400 - 500		7	2				
500 - 600		12					
600 - 700		11	2				
700 - 800		13	2				
800 - 900		15					
900 - 1000		16	3				
1000 - 1100		28/30mm	3				
1100 - 1200		REFUSAL	3				
1200 - 1300			2				
1300 - 1400			3				
1400 - 1500			3				
1500 - 1600			3				
1600 - 1700			3				
1700 - 1800			13/80mm				
1800 - 1900			REFUSAL				
1900 - 2000							
2000 - 2100							
2100 - 2200							
2200 - 2300							
2300 - 2400							
2400 - 2500							
2500 - 2600							
2600 - 2700							
2700 - 2800							
2800 - 2900							
2900 - 3000							

Remarks:
1. The procedure used for this test is similar to that described in AS1289.6.3.2-1997, Method 6.3.2.
2. Usually 8 blows per 20mm is taken as refusal
3. Survey datum is assumed.

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DYNAMIC CONE PENETRATION TEST RESULTS

Client:	MARY CURTIS		
Project:	PROPOSED ALTERATIONS & ADDITIONS		
Location:	68E CALEDONIA STREET, PADDINGTON, NSW		
Job No.	33171A	Hammer Weight & Drop: 9kg/510mm	
Date:	11-5-20	Rod Diameter: 16mm	
Tested By:	A.C.K.	Point Diameter: 20mm	

Test Location	**101**						
Surface RL	≈67.9m						
Depth (mm)	Number of Blows per 100mm Penetration						
0 - 100	PAVER						
100 - 200	3						
200 - 300	CORED						
300 - 400	↓						
400 - 500	SUNK						
500 - 600	2						
600 - 700	↓						
700 - 800	4						
800 - 900	3						
900 - 1000	7/50mm						
1000 - 1100	REFUSAL						
1100 - 1200							
1200 - 1300							
1300 - 1400							
1400 - 1500							
1500 - 1600							
1600 - 1700							
1700 - 1800							
1800 - 1900							
1900 - 2000							
2000 - 2100							
2100 - 2200							
2200 - 2300							
2300 - 2400							
2400 - 2500							
2500 - 2600							
2600 - 2700							
2700 - 2800							
2800 - 2900							
2900 - 3000							

Remarks:
1. The procedure used for this test is described in AS1289.6.3.2-1997 (R2013)
2. Usually 8 blows per 20mm is taken as refusal
3. Datum of levels is AHD

AERIAL IMAGE SOURCE: MAPS.AU.NEARMAP.COM

Title: **SITE LOCATION PLAN**

Location: 68E CALEDONIA STREET, PADDINGTON, NSW

Report No: 33171A

Figure No: 1

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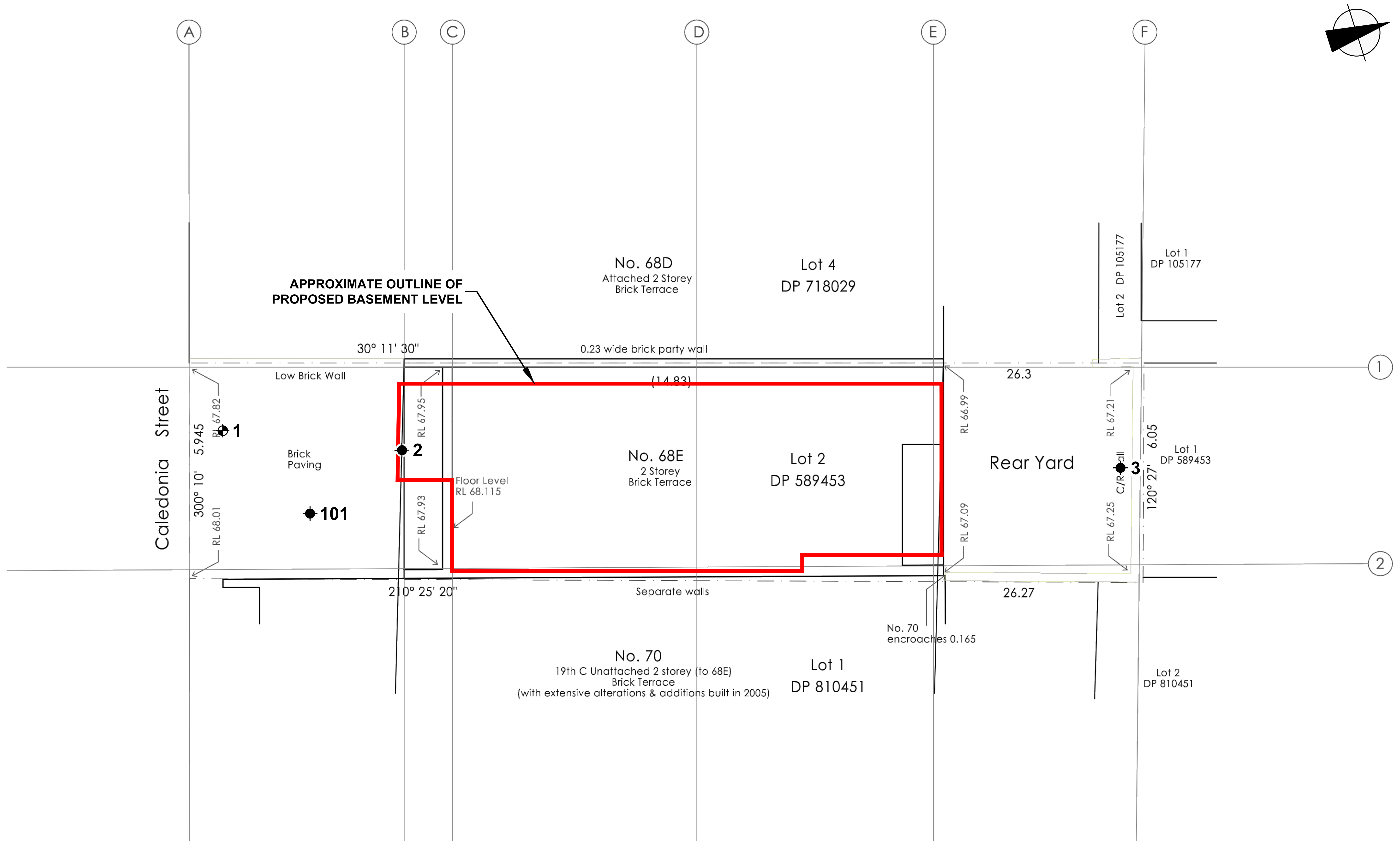
This plan should be read in conjunction with the JK Geotechnics report.

PLOT DATE: 11/06/2020 9:35:39 AM DWG FILE: Y:\33000'S\33171A PADDINGTON\CAD\33171A.DWG



LEGEND

- BOREHOLE AND DCP TEST
- DCP TEST

NOTES:
1. DCP1, BH2 & BH3 ARE FROM OUR PREVIOUS GEOTECHNICAL INVESTIGATION REPORT, REF. 22472Zrpt DATED 7 JULY 2011.
2. BH101 IS FROM OUR CURRENT INVESTIGATION.

0 1 2 3 4 5
SCALE 1:100 @A3 METRES

This plan should be read in conjunction with the JK Geotechnics report.

Title: **TEST LOCATION PLAN**

Location: 68E CALEDONIA STREET, PADDINGTON, NSW

Report No: 33171A

Figure No: 2

JKGeotechnics

PLOT DATE: 11/06/2020 9:28:33 AM DWG FILE: Y:\33000'S\33171A PADDINGTON\CAD\33171A.DWG

VIBRATION EMISSION DESIGN GOALS

German Standard DIN 4150 – Part 3: 1999 provides guideline levels of vibration velocity for evaluating the effects of vibration in structures. The limits presented in this standard are generally recognised to be conservative.

The DIN 4150 values (maximum levels measured in any direction at the foundation, OR, maximum levels measured in (x) or (y) horizontal directions, in the plane of the uppermost floor), are summarised in Table 1 below.

It should be noted that peak vibration velocities higher than the minimum figures in Table 1 for low frequencies may be quite 'safe', depending on the frequency content of the vibration and the actual condition of the structure.

It should also be noted that these levels are 'safe limits', up to which no damage due to vibration effects has been observed for the particular class of building. 'Damage' is defined by DIN 4150 to include even minor non-structural effects such as superficial cracking in cement render, the enlargement of cracks already present, and the separation of partitions or intermediate walls from load bearing walls. Should damage be observed at vibration levels lower than the 'safe limits', then it may be attributed to other causes. DIN 4150 also states that when vibration levels higher than the 'safe limits' are present, it does not necessarily follow that damage will occur. Values given are only a broad guide.

Table 1: DIN 4150 – Structural Damage – Safe Limits for Building Vibration

Group	Type of Structure	Peak Vibration Velocity in mm/s			
		At Foundation Level at a Frequency of:			Plane of Floor of Uppermost Storey
		Less than 10Hz	10Hz to 50Hz	50Hz to 100Hz	All Frequencies
1	Buildings used for commercial purposes, industrial buildings and buildings of similar design.	20	20 to 40	40 to 50	40
2	Dwellings and buildings of similar design and/or use.	5	5 to 15	15 to 20	15
3	Structures that because of their particular sensitivity to vibration, do not correspond to those listed in Group 1 and 2 and have intrinsic value (eg. buildings that are under a preservation order).	3	3 to 8	8 to 10	8

Note: For frequencies above 100Hz, the higher values in the 50Hz to 100Hz column should be used.

REPORT EXPLANATION NOTES

INTRODUCTION

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

The ground is a product of continuing natural and man-made processes and therefore exhibits a variety of characteristics and properties which vary from place to place and can change with time. Geotechnical engineering involves gathering and assimilating limited facts about these characteristics and properties in order to understand or predict the behaviour of the ground on a particular site under certain conditions. This report may contain such facts obtained by inspection, excavation, probing, sampling, testing or other means of investigation. If so, they are directly relevant only to the ground at the place where and time when the investigation was carried out.

DESCRIPTION AND CLASSIFICATION METHODS

The methods of description and classification of soils and rocks used in this report are based on Australian Standard 1726:2017 *'Geotechnical Site Investigations'*. In general, descriptions cover the following properties – soil or rock type, colour, structure, strength or density, and inclusions. Identification and classification of soil and rock involves judgement and the Company infers accuracy only to the extent that is common in current geotechnical practice.

Soil types are described according to the predominating particle size and behaviour as set out in the attached soil classification table qualified by the grading of other particles present (eg. sandy clay) as set out below:

Soil Classification	Particle Size
Clay	< 0.002mm
Silt	0.002 to 0.075mm
Sand	0.075 to 2.36mm
Gravel	2.36 to 63mm
Cobbles	63 to 200mm
Boulders	> 200mm

Non-cohesive soils are classified on the basis of relative density, generally from the results of Standard Penetration Test (SPT) as below:

Relative Density	SPT 'N' Value (blows/300mm)
Very loose (VL)	< 4
Loose (L)	4 to 10
Medium dense (MD)	10 to 30
Dense (D)	30 to 50
Very Dense (VD)	> 50

Cohesive soils are classified on the basis of strength (consistency) either by use of a hand penetrometer, vane shear, laboratory testing and/or tactile engineering examination. The strength terms are defined as follows.

Classification	Unconfined Compressive Strength (kPa)	Indicative Undrained Shear Strength (kPa)
Very Soft (VS)	≤ 25	≤ 12
Soft (S)	> 25 and ≤ 50	> 12 and ≤ 25
Firm (F)	> 50 and ≤ 100	> 25 and ≤ 50
Stiff (St)	> 100 and ≤ 200	> 50 and ≤ 100
Very Stiff (VSt)	> 200 and ≤ 400	> 100 and ≤ 200
Hard (Hd)	> 400	> 200
Friable (Fr)	Strength not attainable – soil crumbles	

Rock types are classified by their geological names, together with descriptive terms regarding weathering, strength, defects, etc. Where relevant, further information regarding rock classification is given in the text of the report. In the Sydney Basin, 'shale' is used to describe fissile mudstone, with a weakness parallel to bedding. Rocks with alternating inter-laminations of different grain size (eg. siltstone/claystone and siltstone/fine grained sandstone) is referred to as 'laminite'.

SAMPLING

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

Disturbed samples taken during drilling provide information on plasticity, grain size, colour, moisture content, minor constituents and, depending upon the degree of disturbance, some information on strength and structure. Bulk samples are similar but of greater volume required for some test procedures.

Undisturbed samples are taken by pushing a thin-walled sample tube, usually 50mm diameter (known as a U50), into the soil and withdrawing it with a sample of the soil contained in a relatively undisturbed state. Such samples yield information on structure and strength, and are necessary for laboratory determination of shrink-swell behaviour, strength and compressibility. Undisturbed sampling is generally effective only in cohesive soils.

Details of the type and method of sampling used are given on the attached logs.

INVESTIGATION METHODS

The following is a brief summary of investigation methods currently adopted by the Company and some comments on their use and application. All methods except test pits, hand auger drilling and portable Dynamic Cone Penetrometers require the use of a mechanical rig which is commonly mounted on a truck chassis or track base.

Test Pits: These are normally excavated with a backhoe or a tracked excavator, allowing close examination of the insitu soils and 'weaker' bedrock if it is safe to descend into the pit. The depth of penetration is limited to about 3m for a backhoe and up to 6m for a large excavator. Limitations of test pits are the problems associated with disturbance and difficulty of reinstatement and the consequent effects on close-by structures. Care must be taken if construction is to be carried out near test pit locations to either properly recompact the backfill during construction or to design and construct the structure so as not to be adversely affected by poorly compacted backfill at the test pit location.

Hand Auger Drilling: A borehole of 50mm to 100mm diameter is advanced by manually operated equipment. Refusal of the hand auger can occur on a variety of materials such as obstructions within any fill, tree roots, hard clay, gravel or ironstone, cobbles and boulders, and does not necessarily indicate rock level.

Continuous Spiral Flight Augers: The borehole is advanced using 75mm to 115mm diameter continuous spiral flight augers, which are withdrawn at intervals to allow sampling and 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 by the flights or may be collected after withdrawal of the auger flights, but they can be very disturbed and layers may become mixed. Information from the auger sampling (as distinct from specific sampling by SPTs or undisturbed samples) is of limited reliability due to mixing or softening of samples by groundwater, or uncertainties as to the original depth of the samples. Augering below the groundwater table is of even lesser reliability than augering above the water table.

Rock Augering: Use can be made of a Tungsten Carbide (TC) bit for auger drilling into rock to indicate rock quality and continuity by variation in drilling resistance and from examination of recovered rock cuttings. This method of investigation is quick and relatively inexpensive but provides only an indication of the likely rock strength and predicted values may be in error by a strength order. Where rock strengths may have a significant impact on construction feasibility or costs, then further investigation by means of cored boreholes may be warranted.

Wash Boring: The borehole is usually 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 assessed from the cuttings, together with some information from "feel" and rate of penetration.

Mud Stabilised Drilling: Either Wash Boring or Continuous Core Drilling can use drilling mud as a circulating fluid to stabilise the borehole. The term 'mud' encompasses a range of products ranging from bentonite to polymers. The mud tends to mask the cuttings and reliable identification is only possible from intermittent intact sampling (eg. from SPT and U50 samples) or from rock coring, etc.

Continuous Core Drilling: A continuous core sample is obtained using a diamond tipped core barrel. Provided full core recovery is achieved (which is not always possible in very low strength rocks and granular soils), this technique provides a very reliable (but relatively expensive) method of investigation. In rocks, NMLC or HQ triple tube core barrels, which give a core of about 50mm and 61mm diameter, respectively, is usually used with water flush. The length of core recovered is compared to the length drilled and any length not recovered is shown as NO CORE. The location of NO CORE recovery is determined on site by the supervising engineer; where the location is uncertain, the loss is placed at the bottom of the drill run.

Standard Penetration Tests: Standard Penetration Tests (SPT) are used mainly in non-cohesive soils, but can also be used in cohesive soils, as a means of indicating density or strength and also of obtaining a relatively undisturbed sample. The test procedure is described in Australian Standard 1289.6.3.1–2004 (R2016) *'Methods of Testing Soils for Engineering Purposes, Soil Strength and Consolidation Tests – Determination of the Penetration Resistance of a Soil – Standard Penetration Test (SPT)'*.

The test is carried out in a borehole by driving a 50mm diameter split sample tube with a tapered shoe, under the impact of a 63.5kg 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 blows, as

  N = 13
  4, 6, 7

- In a 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, as

  N > 30
  15, 30/40mm

The results of the test can be related empirically to the engineering properties of the soil.

A modification to the SPT is where the same driving system is used with a solid 60° tipped steel cone of the same diameter as the SPT hollow sampler. The solid cone can be continuously driven for some distance in soft clays or loose sands, or may be used where damage would otherwise occur to the SPT. The results of this Solid Cone Penetration Test (SCPT) are shown as 'N_c' on the borehole logs, together with the number of blows per 150mm penetration.

Cone Penetrometer Testing (CPT) and Interpretation: The cone penetrometer is sometimes referred to as a Dutch Cone. The test is described in Australian Standard 1289.6.5.1–1999 (R2013) *'Methods of Testing Soils for Engineering Purposes, Soil Strength and Consolidation Tests – Determination of the Static Cone Penetration Resistance of a Soil – Field Test using a Mechanical and Electrical Cone or Friction-Cone Penetrometer'*.

In the tests, a 35mm or 44mm diameter rod with a conical tip is pushed continuously into the soil, the reaction being provided by a specially designed truck or rig which is fitted with a hydraulic ram system. Measurements are made of the end bearing resistance on the cone and the frictional resistance on a separate 134mm or 165mm long sleeve, immediately behind the cone. Transducers in the tip of the assembly are electrically connected by wires passing through the centre of the push rods to an amplifier and recorder unit mounted on the control truck. The CPT does not provide soil sample recovery.

As penetration occurs (at a rate of approximately 20mm per second), the information is output as incremental digital records every 10mm. The results given in this report have been plotted from the digital data.

The information provided on the charts comprise:

- Cone resistance – the actual end bearing force divided by the cross sectional area of the cone – expressed in MPa. There are two scales presented for the cone resistance. The lower scale has a range of 0 to 5MPa and the main scale has a range of 0 to 50MPa. For cone resistance values less than 5MPa, the plot will appear on both scales.
- 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 as a percentage.

  The ratios of the sleeve resistance to cone resistance will vary with the type of soil encountered, with higher relative friction in clays than in sands. Friction ratios of 1% to 2% are commonly encountered in sands and occasionally very soft clays, rising to 4% to 10% in stiff clays and peats. Soil descriptions based on cone resistance and friction ratios are only inferred and must not be considered as exact.

Correlations between CPT and SPT values can be developed for both sands and clays but may be site specific.

Interpretation of CPT values can be made to empirically derive modulus or compressibility values to allow calculation of foundation settlements.

Stratification can be inferred from the cone and friction traces and from experience and information from nearby boreholes etc. Where shown, this information is presented for general guidance, but must be regarded as interpretive. The test method provides a continuous profile of engineering properties but, where precise information on soil classification is required, direct drilling and sampling may be preferable.

There are limitations when using the CPT in that it may not penetrate obstructions within any fill, thick layers of hard clay and very dense sand, gravel and weathered bedrock. Normally a 'dummy' cone is pushed through fill to protect the equipment. No information is recorded by the 'dummy' probe.

Flat Dilatometer Test: The flat dilatometer (DMT), also known as the Marchetti Dilometer comprises a stainless steel blade having a flat, circular steel membrane mounted flush on one side.

The blade is connected to a control unit at ground surface by a pneumatic-electrical tube running through the insertion rods. A gas tank, connected to the control unit by a pneumatic cable, supplies the gas pressure required to expand the membrane. The control unit is equipped with a pressure regulator, pressure gauges, an audio-visual signal and vent valves.

The blade is advanced into the ground using our CPT rig or one of our drilling rigs, and can be driven into the ground using an SPT hammer. As soon as the blade is in place, the membrane is inflated, and the pressure required to lift the membrane (approximately 0.1mm) is recorded. The pressure then required to lift the centre of the membrane by an additional 1mm is recorded. The membrane is then deflated before pushing to the next depth increment, usually 200mm down. The pressure readings are corrected for membrane stiffness.

The DMT is used to measure material index (I_D), horizontal stress index (K_D), and dilatometer modulus (E_D). Using established correlations, the DMT results can also be used to assess the 'at rest' earth pressure coefficient (K_o), over-consolidation ratio (OCR), undrained shear strength (C_u), friction angle (ϕ), coefficient of consolidation (C_h), coefficient of permeability (K_h), unit weight (γ), and vertical drained constrained modulus (M).

The seismic dilatometer (SDMT) is the combination of the DMT with an add-on seismic module for the measurement of shear wave velocity (V_s). Using established correlations, the SDMT results can also be used to assess the small strain modulus (G_o).

Portable Dynamic Cone Penetrometers: Portable Dynamic Cone Penetrometer (DCP) tests are carried out by driving a 16mm diameter rod with a 20mm diameter cone end with a 9kg hammer dropping 510mm. The test is described in Australian Standard 1289.6.3.2–1997 (R2013) *'Methods of Testing Soils for Engineering Purposes, Soil Strength and Consolidation Tests – Determination of the Penetration Resistance of a Soil – 9kg Dynamic Cone Penetrometer Test'*.

The results are used to assess the relative compaction of fill, the relative density of granular soils, and the strength of cohesive soils. Using established correlations, the DCP test results can also be used to assess California Bearing Ratio (CBR).

Refusal of the DCP can occur on a variety of materials such as obstructions within any fill, tree roots, hard clay, gravel or ironstone, cobbles and boulders, and does not necessarily indicate rock level.

Vane Shear Test: The vane shear test is used to measure the undrained shear strength (C_u) of typically very soft to firm fine grained cohesive soils. The vane shear is normally performed in the bottom of a borehole, but can be completed from surface level, the bottom and sides of test pits, and on recovered undisturbed tube samples (when using a hand vane).

The vane comprises four rectangular blades arranged in the form of a cross on the end of a thin rod, which is coupled to the bottom of a drill rod string when used in a borehole. The size of the vane is dependent on the strength of the fine grained cohesive soils; that is, larger vanes are normally used for very low strength soils. For borehole testing, the size of the vane can be limited by the size of the casing that is used.

For testing inside a borehole, a device is used at the top of the casing, which suspends the vane and rods so that they do not sink under self-weight into the 'soft' soils beyond the depth at which the test is to be carried out. A calibrated torque head is used to rotate the rods and vane and to measure the resistance of the vane to rotation.

With the vane in position, torque is applied to cause rotation of the vane at a constant rate. A rate of 6° per minute is the common rotation rate. Rotation is continued until the soil is sheared and the maximum torque has been recorded. This value is then used to calculate the undrained shear strength. The vane is then rotated rapidly a number of times and the operation repeated until a constant torque reading is obtained. This torque value is used to calculate the remoulded shear strength. Where appropriate, friction on the vane rods is measured and taken into account in the shear strength calculation.

LOGS

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

The terms and symbols used in preparation of the logs are defined in the following pages.

Interpretation of the information shown on the logs, and its application to design and construction, should therefore take into account the spacing of boreholes or test pits, the method of drilling or excavation, the frequency of sampling and testing and the possibility of other than 'straight line' variations between the boreholes or test pits. Subsurface conditions between boreholes or test pits may vary significantly from conditions encountered at the borehole or test pit locations.

GROUNDWATER

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

- Although groundwater may be present, in low permeability soils it 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 and may not be the same at the time of construction.
- 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 be washed out of the hole or 'reverted' chemically if reliable water observations are to be made.

More reliable measurements can be made by installing standpipes which are read after the groundwater level has stabilised at intervals ranging from several days to 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 perched water tables or surface water.

FILL

The presence of fill materials can often be determined only by the inclusion of foreign objects (eg. bricks, steel, etc) or by distinctly unusual colour, texture or fabric. Identification of the extent of fill materials will also depend on investigation methods and frequency. Where natural soils similar to those at the site are used for fill, it may be difficult with limited testing and sampling to reliably assess the extent of the fill.

The presence of fill materials is usually regarded with caution as the possible variation in density, strength and material type is much greater than with natural soil deposits. Consequently, there is an increased risk of adverse engineering characteristics or behaviour. If the volume and quality of fill is of importance to a project, then frequent test pit excavations are preferable to boreholes.

LABORATORY TESTING

Laboratory testing is normally carried out in accordance with Australian Standard 1289 *'Methods of Testing Soils for Engineering Purposes'* or appropriate NSW Government Roads & Maritime Services (RMS) test methods. Details of the test procedure used are given on the individual report forms.

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.

Reasonable care is taken with the report as it relates to interpretation of subsurface conditions, 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 be partially dependent on borehole spacing and sampling frequency as well as investigation technique.
- Changes in policy or interpretation of policy by statutory authorities.
- The actions of persons or contractors responding to commercial pressures.
- Details of the development that the Company could not reasonably be expected to anticipate.

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

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 rather than at some later stage, well after the event.

REPRODUCTION OF INFORMATION FOR CONTRACTUAL PURPOSES

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 specially 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.

Copyright in all documents (such as drawings, borehole or test pit logs, reports and specifications) provided by the Company shall remain the property of Jeffery and Katauskas Pty Ltd. Subject to the payment of all fees due, the Client alone shall have a licence to use the documents provided for the sole purpose of completing the project to which they relate. Licence to use the documents may be revoked without notice if the Client is in breach of any obligation to make a payment to us.

REVIEW OF DESIGN

Where major civil or structural developments are proposed <u>or</u> where only a limited investigation has been completed <u>or</u> where the geotechnical conditions/constraints are quite complex, it is prudent to have a joint design review which involves an experienced geotechnical engineer/engineering geologist.

SITE INSPECTION

The Company will always be pleased to provide engineering inspection services for geotechnical aspects of work to which this report is related.

Requirements could range from:

i) a site visit to confirm that conditions exposed are no worse than those interpreted, to

ii) a visit to assist the contractor or other site personnel in identifying various soil/rock types and appropriate footing or pile founding depths, or

iii) full time engineering presence on site.

SYMBOL LEGENDS

SOIL

FILL
TOPSOIL
CLAY (CL, CI, CH)
SILT (ML, MH)
SAND (SP, SW)
GRAVEL (GP, GW)
SANDY CLAY (CL, CI, CH)
SILTY CLAY (CL, CI, CH)
CLAYEY SAND (SC)
SILTY SAND (SM)
GRAVELLY CLAY (CL, CI, CH)
CLAYEY GRAVEL (GC)
SANDY SILT (ML, MH)
PEAT AND HIGHLY ORGANIC SOILS (Pt)

ROCK

CONGLOMERATE
SANDSTONE
SHALE/MUDSTONE
SILTSTONE
CLAYSTONE
COAL
LAMINITE
LIMESTONE
PHYLLITE, SCHIST
TUFF
GRANITE, GABBRO
DOLERITE, DIORITE
BASALT, ANDESITE
QUARTZITE

OTHER MATERIALS

BRICKS OR PAVERS
CONCRETE
ASPHALTIC CONCRETE

CLASSIFICATION OF COARSE AND FINE GRAINED SOILS

Major Divisions		Group Symbol	Typical Names	Field Classification of Sand and Gravel	Laboratory Classification	
Coarse grained soil (more than 65% of soil excluding oversize fraction is greater than 0.075mm)	GRAVEL (more than half of coarse fraction is larger than 2.36mm	GW	Gravel and gravel-sand mixtures, little or no fines	Wide range in grain size and substantial amounts of all intermediate sizes, not enough fines to bind coarse grains, no dry strength	≤ 5% fines	$C_u > 4$ $1 < C_c < 3$
		GP	Gravel and gravel-sand mixtures, little or no fines, uniform gravels	Predominantly one size or range of sizes with some intermediate sizes missing, not enough fines to bind coarse grains, no dry strength	≤ 5% fines	Fails to comply with above
		GM	Gravel-silt mixtures and gravel-sand-silt mixtures	'Dirty' materials with excess of non-plastic fines, zero to medium dry strength	≥ 12% fines, fines are silty	Fines behave as silt
		GC	Gravel-clay mixtures and gravel-sand-clay mixtures	'Dirty' materials with excess of plastic fines, medium to high dry strength	≥ 12% fines, fines are clayey	Fines behave as clay
	SAND (more than half of coarse fraction is smaller than 2.36mm)	SW	Sand and gravel-sand mixtures, little or no fines	Wide range in grain size and substantial amounts of all intermediate sizes, not enough fines to bind coarse grains, no dry strength	≤ 5% fines	$C_u > 6$ $1 < C_c < 3$
		SP	Sand and gravel-sand mixtures, little or no fines	Predominantly one size or range of sizes with some intermediate sizes missing, not enough fines to bind coarse grains, no dry strength	≤ 5% fines	Fails to comply with above
		SM	Sand-silt mixtures	'Dirty' materials with excess of non-plastic fines, zero to medium dry strength	≥ 12% fines, fines are silty	N/A
		SC	Sand-clay mixtures	'Dirty' materials with excess of plastic fines, medium to high dry strength	≥ 12% fines, fines are clayey	

Major Divisions		Group Symbol	Typical Names	Field Classification of Silt and Clay			Laboratory Classification
				Dry Strength	Dilatancy	Toughness	% < 0.075mm
ine grained soils (more than 35% of soil excluding oversize fraction is less than 0.075mm)	SILT and CLAY (low to medium plasticity)	ML	Inorganic silt and very fine sand, rock flour, silty or clayey fine sand or silt with low plasticity	None to low	Slow to rapid	Low	Below A line
		CL, CI	Inorganic clay of low to medium plasticity, gravelly clay, sandy clay	Medium to high	None to slow	Medium	Above A line
		OL	Organic silt	Low to medium	Slow	Low	Below A line
	SILT and CLAY (high plasticity)	MH	Inorganic silt	Low to medium	None to slow	Low to medium	Below A line
		CH	Inorganic clay of high plasticity	High to very high	None	High	Above A line
		OH	Organic clay of medium to high plasticity, organic silt	Medium to high	None to very slow	Low to medium	Below A line
	Highly organic soil	Pt	Peat, highly organic soil	–	–	–	–

Laboratory Classification Criteria

A well graded coarse grained soil is one for which the coefficient of uniformity $Cu > 4$ and the coefficient of curvature $1 < C_c < 3$. Otherwise, the soil is poorly graded. These coefficients are given by:

$$C_U = \frac{D_{60}}{D_{10}} \quad \text{and} \quad C_C = \frac{(D_{30})^2}{D_{10}\,D_{60}}$$

Where D_{10}, D_{30} and D_{60} are those grain sizes for which 10%, 30% and 60% of the soil grains, respectively, are smaller.

NOTES:

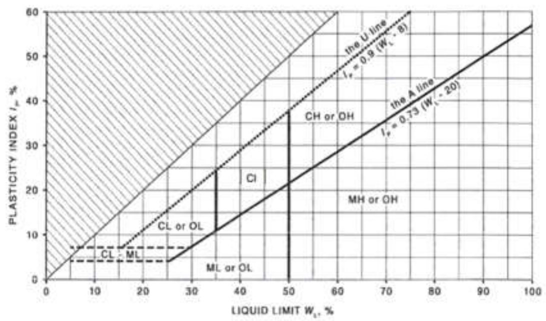
1 For a coarse grained soil with a fines content between 5% and 12%, the soil is given a dual classification comprising the two group symbols separated by a dash; for example, for a poorly graded gravel with between 5% and 12% silt fines, the classification is GP-GM.

2 Where the grading is determined from laboratory tests, it is defined by coefficients of curvature (C_c) and uniformity (C_u) derived from the particle size distribution curve.

3 Clay soils with liquid limits > 35% and ≤ 50% may be classified as being of medium plasticity.

4 The U line on the Modified Casagrande Chart is an approximate upper bound for most natural soils.

Modified Casagrande Chart for Classifying Silts and Clays according to their Behaviour



LOG SYMBOLS

Log Column	Symbol	Definition
Groundwater Record	▼	Standing water level. Time delay following completion of drilling/excavation may be shown.
	C	Extent of borehole/test pit collapse shortly after drilling/excavation.
	▶	Groundwater seepage into borehole or test pit noted during drilling or excavation.
Samples	ES	Sample taken over depth indicated, for environmental analysis.
	U50	Undisturbed 50mm diameter tube sample taken over depth indicated.
	DB	Bulk disturbed sample taken over depth indicated.
	DS	Small disturbed bag sample taken over depth indicated.
	ASB	Soil sample taken over depth indicated, for asbestos analysis.
	ASS	Soil sample taken over depth indicated, for acid sulfate soil analysis.
	SAL	Soil sample taken over depth indicated, for salinity analysis.
Field Tests	N = 17 4, 7, 10	Standard Penetration Test (SPT) performed between depths indicated by lines. Individual figures show blows per 150mm penetration. 'Refusal' refers to apparent hammer refusal within the corresponding 150mm depth increment.
	N_c = 5 7 3R	Solid Cone Penetration Test (SCPT) performed between depths indicated by lines. Individual figures show blows per 150mm penetration for 60° solid cone driven by SPT hammer. 'R' refers to apparent hammer refusal within the corresponding 150mm depth increment.
	VNS = 25	Vane shear reading in kPa of undrained shear strength.
	PID = 100	Photoionisation detector reading in ppm (soil sample headspace test).
Moisture Condition (Fine Grained Soils)	w > PL	Moisture content estimated to be greater than plastic limit.
	w ≈ PL	Moisture content estimated to be approximately equal to plastic limit.
	w < PL	Moisture content estimated to be less than plastic limit.
	w ≈ LL	Moisture content estimated to be near liquid limit.
	w > LL	Moisture content estimated to be wet of liquid limit.
(Coarse Grained Soils)	D	DRY – runs freely through fingers.
	M	MOIST – does not run freely but no free water visible on soil surface.
	W	WET – free water visible on soil surface.
Strength (Consistency) Cohesive Soils	VS	VERY SOFT – unconfined compressive strength ≤ 25kPa.
	S	SOFT – unconfined compressive strength > 25kPa and ≤ 50kPa.
	F	FIRM – unconfined compressive strength > 50kPa and ≤ 100kPa.
	St	STIFF – unconfined compressive strength > 100kPa and ≤ 200kPa.
	VSt	VERY STIFF – unconfined compressive strength > 200kPa and ≤ 400kPa.
	Hd	HARD – unconfined compressive strength > 400kPa.
	Fr	FRIABLE – strength not attainable, soil crumbles.
	( )	Bracketed symbol indicates estimated consistency based on tactile examination or other assessment.
Density Index/ Relative Density (Cohesionless Soils)		**Density Index (I_D) Range (%)** / **SPT 'N' Value Range (Blows/300mm)**
	VL	VERY LOOSE ≤ 15 / 0 – 4
	L	LOOSE > 15 and ≤ 35 / 4 – 10
	MD	MEDIUM DENSE > 35 and ≤ 65 / 10 – 30
	D	DENSE > 65 and ≤ 85 / 30 – 50
	VD	VERY DENSE > 85 / > 50
	( )	Bracketed symbol indicates estimated density based on ease of drilling or other assessment.
Hand Penetrometer Readings	300 250	Measures reading in kPa of unconfined compressive strength. Numbers indicate individual test results on representative undisturbed material unless noted otherwise.

Log Column	Symbol	Definition
Remarks	'V' bit	Hardened steel 'V' shaped bit.
	'TC' bit	Twin pronged tungsten carbide bit.
	T_{60}	Penetration of auger string in mm under static load of rig applied by drill head hydraulics without rotation of augers.
	Soil Origin	The geological origin of the soil can generally be described as:
		RESIDUAL – soil formed directly from insitu weathering of the underlying rock. No visible structure or fabric of the parent rock.
		EXTREMELY WEATHERED – soil formed directly from insitu weathering of the underlying rock. Material is of soil strength but retains the structure and/or fabric of the parent rock.
		ALLUVIAL – soil deposited by creeks and rivers.
		ESTUARINE – soil deposited in coastal estuaries, including sediments caused by inflowing creeks and rivers, and tidal currents.
		MARINE – soil deposited in a marine environment.
		AEOLIAN – soil carried and deposited by wind.
		COLLUVIAL – soil and rock debris transported downslope by gravity, with or without the assistance of flowing water. Colluvium is usually a thick deposit formed from a landslide. The description 'slopewash' is used for thinner surficial deposits.
		LITTORAL – beach deposited soil.

Classification of Material Weathering

<table>
<tr><th colspan="2">Term</th><th colspan="2">Abbreviation</th><th>Definition</th></tr>
<tr><td colspan="2">Residual Soil</td><td colspan="2">RS</td><td>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.</td></tr>
<tr><td colspan="2">Extremely Weathered</td><td colspan="2">XW</td><td>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.</td></tr>
<tr><td>Highly Weathered</td><td rowspan="2">Distinctly Weathered (Note 1)</td><td>HW</td><td rowspan="2">DW</td><td>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.</td></tr>
<tr><td>Moderately Weathered</td><td>MW</td><td>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.</td></tr>
<tr><td colspan="2">Slightly Weathered</td><td colspan="2">SW</td><td>Rock is partially discoloured with staining or bleaching along joints but shows little or no change of strength from fresh rock.</td></tr>
<tr><td colspan="2">Fresh</td><td colspan="2">FR</td><td>Rock shows no sign of decomposition of individual minerals or colour changes.</td></tr>
</table>

NOTE 1: The term 'Distinctly Weathered' is used where it is not practicable to distinguish between 'Highly Weathered' and 'Moderately Weathered' rock. 'Distinctly Weathered' is defined as follows: *'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 weathering products in pores'*. There is some change in rock strength.

Rock Material Strength Classification

<table>
<tr><th rowspan="2">Term</th><th rowspan="2">Abbreviation</th><th rowspan="2">Uniaxial Compressive Strength (MPa)</th><th colspan="2">Guide to Strength</th></tr>
<tr><th>Point Load Strength Index $Is_{(50)}$ (MPa)</th><th>Field Assessment</th></tr>
<tr><td>Very Low Strength</td><td>VL</td><td>0.6 to 2</td><td>0.03 to 0.1</td><td>Material crumbles under firm blows with sharp end of pick; can be peeled with knife; too hard to cut a triaxial sample by hand. Pieces up to 30mm thick can be broken by finger pressure.</td></tr>
<tr><td>Low Strength</td><td>L</td><td>2 to 6</td><td>0.1 to 0.3</td><td>Easily scored with a knife; indentations 1mm to 3mm show in the specimen with firm blows of the pick point; has dull sound under hammer. A piece of core 150mm long by 50mm diameter may be broken by hand. Sharp edges of core may be friable and break during handling.</td></tr>
<tr><td>Medium Strength</td><td>M</td><td>6 to 20</td><td>0.3 to 1</td><td>Scored with a knife; a piece of core 150mm long by 50mm diameter can be broken by hand with difficulty.</td></tr>
<tr><td>High Strength</td><td>H</td><td>20 to 60</td><td>1 to 3</td><td>A piece of core 150mm long by 50mm diameter cannot be broken by hand but can be broken by a pick with a single firm blow; rock rings under hammer.</td></tr>
<tr><td>Very High Strength</td><td>VH</td><td>60 to 200</td><td>3 to 10</td><td>Hand specimen breaks with pick after more than one blow; rock rings under hammer.</td></tr>
<tr><td>Extremely High Strength</td><td>EH</td><td>> 200</td><td>> 10</td><td>Specimen requires many blows with geological pick to break through intact material; rock rings under hammer.</td></tr>
</table>

Abbreviations Used in Defect Description

Cored Borehole Log Column		Symbol Abbreviation	Description
Point Load Strength Index		• 0.6	Axial point load strength index test result (MPa)
		x 0.6	Diametral point load strength index test result (MPa)
Defect Details	– Type	Be	Parting – bedding or cleavage
		CS	Clay seam
		Cr	Crushed/sheared seam or zone
		J	Joint
		Jh	Healed joint
		Ji	Incipient joint
		XWS	Extremely weathered seam
	– Orientation	Degrees	Defect orientation is measured relative to normal to the core axis (ie. relative to the horizontal for a vertical borehole)
	– Shape	P	Planar
		C	Curved
		Un	Undulating
		St	Stepped
		Ir	Irregular
	– Roughness	Vr	Very rough
		R	Rough
		S	Smooth
		Po	Polished
		Sl	Slickensided
	– Infill Material	Ca	Calcite
		Cb	Carbonaceous
		Clay	Clay
		Fe	Iron
		Qz	Quartz
		Py	Pyrite
	– Coatings	Cn	Clean
		Sn	Stained – no visible coating, surface is discoloured
		Vn	Veneer – visible, too thin to measure, may be patchy
		Ct	Coating ≤ 1mm thick
		Filled	Coating > 1mm thick
	– Thickness	mm.t	Defect thickness measured in millimetres