REPORT

TO
SUN PROPERTY GROUP AUSTRALIA PTY LTD

ON
GEOTECHNICAL INVESTIGATION

FOR
PROPOSED RESIDENTIAL DEVELOPMENT

AT
307 SYDNEY ROAD AND 12 BOYLE STREET,
BALGOWLAH, NSW

21 December 2018
Ref: 31201SBrptRev2

dlcs ISO 9001 certified AS/NZS 4801 certified ISO 14001 certified

JK Geotechnics
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Date: 21 December 2018
Report No: 31201SBrpt
Revision No: 2

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For and on behalf of
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1 INTRODUCTION

This report presents the results of a geotechnical investigation for the proposed residential development at 307 Sydney Road and 12 Boyle Street, Balgowlah, NSW. The location of the site is shown on Figure 1. The investigation was commissioned by Mia Zhoo of Sun Property Group Australia Pty Ltd, in consultation with the project architect, RobertsDay, and was carried out in accordance with our proposal dated 12 December 2017 (Ref: P45783Brev1).

As shown on the supplied architectural drawings by RobertsDay (Ref: SPG SRD, Drawing Nos A200, A201, A203, A204, A300, A301, A302, A303 and A400, Revision A, dated 7/12/18), the existing house at 12 Boyle Street and the garage at 307 Sydney Road will be demolished. The existing house within 307 Sydney Road will be retained and refurbished and incorporated as part of the development. The new residential development will have two above ground levels, over a basement parking level within 12 Boyle Street and the portion of 307 Sydney Road adjacent to 12 Boyle Street. The existing house within 307 Sydney Road will be extended with a first floor addition and will be incorporated into the new development. The basement is proposed at RL43.9m, which will require excavation to depths of about 2m. The basement will partly extend below the southern edge of the existing house within 307 Sydney Road. The buildings within 307 Sydney Road will step up the hillside at the southern end and will also require excavation to maximum depths of about 2m.

The investigation was carried out to assist with the DA stage of the project and as such was limited to a walkover inspection and limited testing using portable hand operated equipment to obtain geotechnical information on subsurface conditions as a basis for comments and recommendations on likely subsurface conditions, slope stability, excavation, retention, footings, acid sulfate soils and additional geotechnical investigation required following demolition.

2 INVESTIGATION PROCEDURE

A walkover inspection of the site was completed by our Senior Associate, Mr Daniel Bliss, on 31 January 2018. Observations made during the inspection are summarised in Section 3.1 below. To supplement the surface observations, boreholes BH1 to BH3 were drilled using a hand auger to refusal at depths ranging from 0.12m to 0.47m below the existing ground surface. Dynamic Cone Penetration (DCP) tests were carried out adjacent to the boreholes to refusal at depths ranging from 0.29m to 0.63m. The purpose of the DCP tests was to assess the apparent compaction of the fill and to probe down to the surface of the underlying sandstone.

The investigation locations, as shown on Figure 2, were set out by taped measurements from existing surface features. The approximate surface levels, as shown on the borehole logs and DCP test results, were estimated by interpolation between spot levels and contours shown on the supplied survey plan by Geosurv (Ref: 170937, Revision B, dated 9/11/17). The datum of the levels is Australian Height Datum (AHD).

Groundwater observations were made during and on completion of drilling of the boreholes. No longer-term monitoring of groundwater levels was carried out.

The borehole logs and DCP tests results are attached, together with a set of explanatory notes, which describe the investigation techniques, and their limitations, and define the logging terms and symbols used.

Laboratory testing, including for soil contamination, was outside the scope of this geotechnical investigation.

3 RESULTS OF INVESTIGATION

3.1 Site Description

Sydney Road, on the northern end of the site, runs along an east to west trending ridgeline, with the site located on the southern side of the ridge on a hill that slopes down towards the south. The ground surface is almost flat for most of 307 Sydney Road, but once this joins 12 Boyle Street the ground surface slopes down to the south at about 5°. At the rear of No. 307 the ground surface is stepped via series of terraces formed by retaining walls and sandstone outcrops.

No. 12 Boyle Street contains a two storey brick house, which appeared to be in good external condition. No. 307 Sydney Road contains a one storey rendered house, with a partial sub floor level at the southern end. A separate single garage is located to the north of the main house. The house and garage appeared to be in fair to good external condition. The houses in both properties are surrounded by concrete paths, grassed areas and gardens.

At the rear of No. 307 a flat grass area is supported by a sandstone cobble retaining wall of about 1.1m to 1.2m in height, with small areas of sandstone exposed at the base of this wall. Below this wall the site slopes down to the south, with the slope broken by a sandstone cliff of about 1.5m in height, but it appeared that parts of this cliff comprised sandstone boulders. Due to the vegetation covering the cliff the exact nature of these suspected boulders could not be confirmed. Where the sandstone was exposed it was assessed to be distinctly weathered and of at least low to medium strength. Some water seepage was observed within parts of the exposed sandstone. At the rear of No. 307 a sandstone cobble retaining wall supports the subject site along the boundary with the property to the south.

The site is surrounded by one to three storey brick and rendered houses and unit buildings. The ground surface across the boundaries generally follows the hillside slope, with the properties along Boyle Street stepping down the hillside, with the changes in level supported by mainly sandstone block retaining walls of less than 1m in height. The surrounding buildings generally appeared to be in good external condition.

3.2 Subsurface Conditions

The boreholes encountered fill, with BH1 and BH3 terminating within the fill at depths of 0.47m and 0.12m, respectively. In BH2 the base of the fill was encountered at a depth of 0.17m. The fill comprised sand and silty sand topsoil, with sandstone gravel and silt. Based on the DCP test results the fill was assessed to be moderately compacted.

In BH2, natural silty sand was encountered at a depth of 0.17m and extended to the refusal depth of 0.28m. Based on the DCP test results the silty sand is of loose relative density. The silty sand also contained sandstone gravel.

The DCP tests refused at depths ranging from 0.29m to 0.63m and this is inferred to have occurred on the surface of the underlying sandstone, particularly where BH1 and BH2 refused at similar depths to the hand auger, which was found to scrape on what felt to be sandstone in the base of the boreholes. However, since DCP tests do not provide sample recovery and penetration into the sandstone using hand operated equipment is not possible, the presence of the sandstone could not be confirmed.

No groundwater was encountered within the boreholes during or on completion of drilling.

Reference should be made to the borehole logs and DCP test results for detailed descriptions of the subsurface conditions encountered.

4 COMMENTS AND RECOMMENDATIONS

4.1 Subsurface Profile and Additional Geotechnical Investigation

This investigation was limited to the use of portable hand operated equipment, which refused at shallow depths. Due to the exposed sandstone on site the refusal has been inferred to have occurred on the surface of the underlying sandstone. Therefore, the subsurface profile for this site is expected to comprise a generally shallow soil cover over sandstone bedrock. However, some boulders may be present within parts of the site, which appeared to be present as part of the sandstone cliff that crosses the site towards the southern end.

The comments and recommendations provided herein are based on the above inferred subsurface profile. However, we recommend that the profile be confirmed by the drilling of boreholes using a drilling rig in order to penetrate into the sandstone bedrock. Access for a drilling rig would be required to drill the boreholes, which at present is only possible within the northern portion of 307 Sydney Road. Therefore, the investigation would need to be carried out following demolition of the existing house within 12 Boyle Street so access is possible for a drilling rig to the entire site. We recommend the drilling of three to four boreholes to depths below the base of the proposed basement. In order to reach these depths and optimise the information obtained, we recommend that these boreholes involve core drilling of the sandstone. The comments and recommendations provided herein must be reviewed and amplified as part of the additional geotechnical investigation.

4.2 Slope Stability

The majority of the site is on a gentle slope, with the major change in level being the existing retaining walls and the sandstone cliff that cross the site within the southern portion. Based on this and the shallow depth of the sandstone we consider that the likelihood of slope instability within the majority of the site would be unlikely.

The main geotechnical hazard for the site would be instability of sandstone boulders within the existing cliff line or exposed during excavation. Provided geotechnical inspections are carried out during excavation as recommended below and any stability measures required, such as retaining walls, are constructed to good engineering standards we consider that the risk to property of such hazards would be very low.

4.3 Excavation

Excavation to the required depths of about 2m will encounter surface soils, but for the most part sandstone bedrock.

Prior to the start of excavation we recommend that dilapidation surveys be completed on the existing structures within the adjoining properties at 10 and 14 Boyle Street and 305 Sydney Road. The dilapidation surveys should comprise detailed inspections of the adjoining structures, both externally and internally, with all defects rigorously described, i.e. defect location, defect type, crack width, crack length, etc. The respective owners of the adjoining properties should be asked to confirm that the dilapidation reports represent a fair record of actual conditions. The preparation of the dilapidation reports will also help to guard against opportunistic claims for damage that was present prior to the start of excavation.

Care must be taken during excavation that any boulders that are present are not undermined or rendered unstable. The initial works on site should involve clearing of vegetation that covers the existing cliff at the southern end of the site to allow a geotechnical inspection to confirm if boulders are present. The geotechnical engineer can then advise if any stabilisation measures are required prior to or during excavation.

Care must also be taken during excavation below the edge of the existing house at 307 Sydney Road. Temporary propping, underpinning or other support must be provided as recommended by the structural engineer prior to bulk excavation. We understand that inspection has confirmed that the footings below the existing house are founded on exposed sandstone, but regular inspections will be required during excavation to assess if the sandstone supporting the footings is of good quality and any underpinning or additional support required and further areas are exposed.

Excavation of soil and possible any upper extremely weathered sandstone would be achievable using conventional excavation equipment, such as the buckets of hydraulic excavators. Excavation of the sandstone will require assistance with rock excavation equipment, such as hydraulic rock hammers, ripping hooks, rotary grinders or rock saws.

Hydraulic rock hammers must be used with care due to the risk of damage to existing structures from the vibrations generated by such equipment. If hydraulic rock hammers are used we recommend that the vibrations transmitted to the adjoining structures be quantitatively monitored at all times during rock hammer operation, by attaching vibration monitors to the adjoining structures. The vibrations monitors should also be attached to flashing warning lights, or other suitable warning devices, so the operator is aware when vibration limits have been reached so work can cease. Reference should be made to the attached Vibration Emission Design Goals sheet for acceptable limits of transmitted vibrations.

Where the transmitted vibrations are excessive alternative excavation equipment will be required, such as a smaller rock hammer, rotary grinders, ripping hooks or rock saws. Consideration could be given to using a rock saw to cut a slot around the perimeter of the excavation and then using a rock hammer to break out the rock from between the saw cuts in order to limit the transmitted vibrations. However, the effectiveness of this must be confirmed from the results of vibration monitoring.

Where the excavations extend to the existing house at 307 Sydney Road the use of rock hammers will not be appropriate and excavation close to the existing structure will need to be carried out using non-percussive equipment, such as rock saws and rotary grinders.

4.4 Groundwater

Given the location of the site towards the top of a ridge line we do not expect that groundwater will be an issues for the proposed development. However, some seepage into the excavation may be experienced, which would tend to occur along the soil/rock interface and through bedding partings and joints within the sandstone, particularly during and following rainfall. Given the slope of the site any such seepage that does occur should be able to be controlled during construction using gravity drainage. In the long term, drainage should be provided behind any retaining walls and at the base of any exposed sandstone cut faces to direct and control the collected seepage.

4.5 Retention

Most of the excavations will be offset from the site boundaries, apart from parts of the basement excavation within No. 12 Boyle Street and the eastern side of the basement within No. 307 Sydney Road. Therefore, given the shallow depth of the sandstone it appears that for the majority of the site temporary batters within the soils and vertical excavations within the sandstone are likely to be feasible. However, this must be further assessed by the drilling of cored boreholes to determine the quality of the sandstone and its ability to stand unsupported.

Where the basement excavation is close to the boundaries, a retention systems could be installed prior to the start of excavation, such as piled retaining walls, but if the sandstone is shallow it may be more practical to form retaining walls within trenches prior to bulk excavation. This would involve excavation of a shallow trench to the top of the sandstone and placement of steel and concrete immediately following excavation to form the wall. Bulk excavation could then be carried out in front of the wall once cured and continue below the base of the wall into the sandstone. Geotechnical inspection of the base of the wall once exposed and the sandstone below would be required during excavation to assess if any additional support is required. Following demolition we recommend that a series of test pits be excavated where the basement will extend close to the boundaries to confirm the depth of the sandstone and assess if such a retention system is feasible.

Temporary batters within the soils of no more than 3m in height should be no steeper than 1 Vertical in 1 Horizontal (1V:1H). Such batters should remain stable in the short term provided all surcharge loads, including construction loads, are kept well clear of the crest of the batters.

Permanent batters, if required, should be no steeper than 1V:2H, but flatter batters of the order of 1V:3H may be preferred to allow access for maintenance of vegetation. Permanent batters should be covered with topsoil and planted with a deep rooted runner grass, or other suitable coverings, to reduce erosion. All stormwater runoff should be directed away from all temporary and permanent batters to also reduce erosion.

Provided the cored boreholes drilled following demolition confirm that the sandstone is of good quality, vertical unsupported excavation of the sandstone would be feasible. However, geotechnical inspections of the cut faces must be carried out at depth intervals of 1m to 1.5m to check for any inclined joints or weak seams that require additional support. Any support recommended by the geotechnical engineer, such as rock bolts and/or shotcrete and mesh, must be installed prior to further excavation.

If the above batters cannot be accommodated or if the sandstone is too deep to allow construction of retaining walls within trenches, further geotechnical advice on suitable retaining system should be obtained. Such system may comprise contiguous or secant pile retaining walls.

Permanent retaining walls constructed at the toe of temporary batters may be designed as cantilevered walls based on a triangular earth pressure distribution using an active earth pressure coefficient, K_a, of 0.33 and a bulk unit weight of 20kN/m^3. Where walls are restrained from some lateral movement, such as by other structural elements in front of the wall, an ‘at rest’ earth pressure coefficient, K_0, of 0.6 should be used. Lower coefficients may be used for the support of rock, but these would need to be assessed once boreholes extending into the rock can be drilled.

The above coefficients assume horizontal backfill surfaces and where inclined backfill is proposed the coefficients would need to be increased or the inclined backfill taken as a surcharge load. All surcharge loads must be allowed for in the design, plus full hydrostatic pressures, unless measures are undertaken to provide complete and permanent drainage behind the wall.

4.6 Footings

Since sandstone will be encountered within the excavations all footings should be founded within the sandstone to provide uniform support and reduce the risk of differential settlement. Pad or strip footings founded within the sandstone should be appropriate.

Footings may be provisionally designed based on an allowable bearing pressure of 1000kPa. However, higher bearing pressures are likely to be appropriate within the sandstone, but this will need to be assessed based on the results of the cored boreholes. The footing excavations should be inspected by a geotechnical engineer following excavation to confirm that the appropriate quality rock has been encountered.

4.7 Acid Sulfate Soils

Our environmental division, Environmental Investigation Services (EIS), have undertaken a preliminary Acid Sulfate Soil (ASS) assessment to establish whether there is a risk of ASS occurrence at the site. Based on this assessment, EIS are of the opinion that ASS or potential ASS (PASS) are unlikely to be present at the site and that an ASS management plan is not required for the proposed development. This is based on the following lines of evidence:

- The site is mapped as being within an area of “no known occurrence” of ASS based on the risk map prepared by the Department of Land and Water Conservation;
- The site is within a Class 5 ASS risk area based on the Manly Council Local Environmental Plan 2013. Works that trigger a more detailed assessment of ASS or preparation of an ASS management plan for a Class 5 site include works within 500m of adjacent Class 1, 2, 3, or 4 land which are likely to lower the water table below 1m AHD on the adjacent land. The proposed development works do not trigger this requirement;
- The site lies at an elevation of approximately RL45m AHD, within a geological landscape characterised by shallow sandstone bedrock outcrops. ASS and PASS are typically associated with low-lying, alluvial soils at elevations below RL10m AHD; and
- The boreholes drilled for the geotechnical investigation did not identify any soils that were suspected to be ASS or PASS.

5 GENERAL COMMENTS

The recommendations presented in this report include specific issues to be addressed during the detailed design and construction phases of the project. In the event that any of the detailed design and construction phase recommendations presented in this report are not implemented, the general recommendations may become inapplicable and JK Geotechnics 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 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 will need to be assigned to any soil excavated from the site prior to offsite disposal. Subject to the appropriate testing, material can be classified as Virgin Excavated Natural Material (VENM), General Solid, Restricted Solid or Hazardous Waste. Analysis takes seven to 10 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) should be expected. We strongly recommend that this issue 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 JK Geotechnics. 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.

JK Geotechnics

GEOTECHNICAL AND ENVIRONMENTAL ENGINEERS

BOREHOLE LOG

Borehole No.
1
1/1

Client: SUN PROPERTY GROUP AUSTRALIA PTY LTD

Project: PROPOSED RESIDENTIAL DEVELOPMENT

Location: 307 SYDNEY ROAD AND 12 BOYLE STREET, BALGOWLAH, NSW

Job No. 31201SB **Method:** HAND AUGER **R.L. Surface:** ~ 45.6m

Date: 31/01/18 **Datum:** AHD

Logged/Checked by: D.B./P.S.

Groundwater Record	SAMPLES ES	SAMPLES U50	SAMPLES DB	SAMPLES 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: Sand, fine grained, dark brown, with coarse grained sandstone gravel, trace of silt.	D			GRASS COVER APPEARS MODERATELY COMPACTED
						0.5			END OF BOREHOLE AT 0.47m				HAND AUGER REFUSAL
						1							
						1.5							
						2							
						2.5							
						3							
						3.5							

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BOREHOLE LOG

Borehole No. **2**

1/1

Client: SUN PROPERTY GROUP AUSTRALIA PTY LTD

Project: PROPOSED RESIDENTIAL DEVELOPMENT

Location: 307 SYDNEY ROAD AND 12 BOYLE STREET, BALGOWLAH, NSW

Job No. 31201SB **Method:** HAND AUGER **R.L. Surface:** ~ 46.7m

Date: 31/01/18 **Datum:** AHD

Logged/Checked by: D.B./P.S.

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 sand topsoil, fine to medium grained, dark brown with root fibres.	M			GRASS COVER
								SM	SILTY SAND: fine to medium grained, brown and orange brown with sandstone gravel.	M	L		
									END OF BOREHOLE AT 0.28m				HAND AUGER REFUSAL
						0.5							
						1							
						1.5							
						2							
						2.5							
						3							
						3.5							

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GEOTECHNICAL AND ENVIRONMENTAL ENGINEERS

BOREHOLE LOG

Borehole No.
3
1/1

Client: SUN PROPERTY GROUP AUSTRALIA PTY LTD
Project: PROPOSED RESIDENTIAL DEVELOPMENT
Location: 307 SYDNEY ROAD AND 12 BOYLE STREET, BALGOWLAH, NSW

Job No. 31201SB **Method:** HAND AUGER **R.L. Surface:** ~ 45.5m
Date: 31/01/18 **Datum:** AHD
Logged/Checked by: D.B./P.S.

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 COMPLET-ION					REFER TO DCP TEST RESULTS	0			FILL: Sand, fine grained, with sandstone gravel, trace of silt.	D			APPEARS MODERATELY COMPACTED
									END OF BOREHOLE AT 0.12m				HAND AUGER REFUSAL ON GRAVEL IN FILL
						0.5							
						1							
						1.5							
						2							
						2.5							
						3							
						3.5							

JK Geotechnics

GEOTECHNICAL AND ENVIRONMENTAL ENGINEERS

DYNAMIC CONE PENETRATION TEST RESULTS

Client:	SUN PROPERTY GROUP AUSTRALIA PTY LTD
Project:	PROPOSED RESIDENTIAL DEVELOPMENT
Location:	307 SYDNEY ROAD AND 12 BOYLE STREET, BALGOWLAH, NSW

Job No.	31201SB	Hammer Weight & Drop: 9kg/510mm
Date:	31-1-18	Rod Diameter: 16mm
Tested By:	D.B.	Point Diameter: 20mm

	Number of Blows per 100mm Penetration						
Test Location	RL~45.6m	RL~46.7m	RL~45.5m				
Depth (mm)	**1**	**2**	**3**				
0 - 100	8	4	8				
100 - 200	7	6	24				
200 - 300	8	28/90mm	17				
300 - 400	7	REFUSAL	13				
400 - 500	23/90mm		9				
500 - 600	REFUSAL		17				
600 - 700			10/30mm				
700 - 800			REFUSAL				
800 - 900							
900 - 1000							
1000 - 1100							
1100 - 1200							
1200 - 1300							
1300 - 1400							
1400 - 1500							
1500 - 1600							
1600 - 1700							
1700 - 1800							
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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. Datum of levels is AHD

AERIAL IMAGE SOURCE: GOOGLE EARTH PRO 7.1.5.1557
AERIAL IMAGE ©: 2015 GOOGLE INC.

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

Title: **SITE LOCATION PLAN**

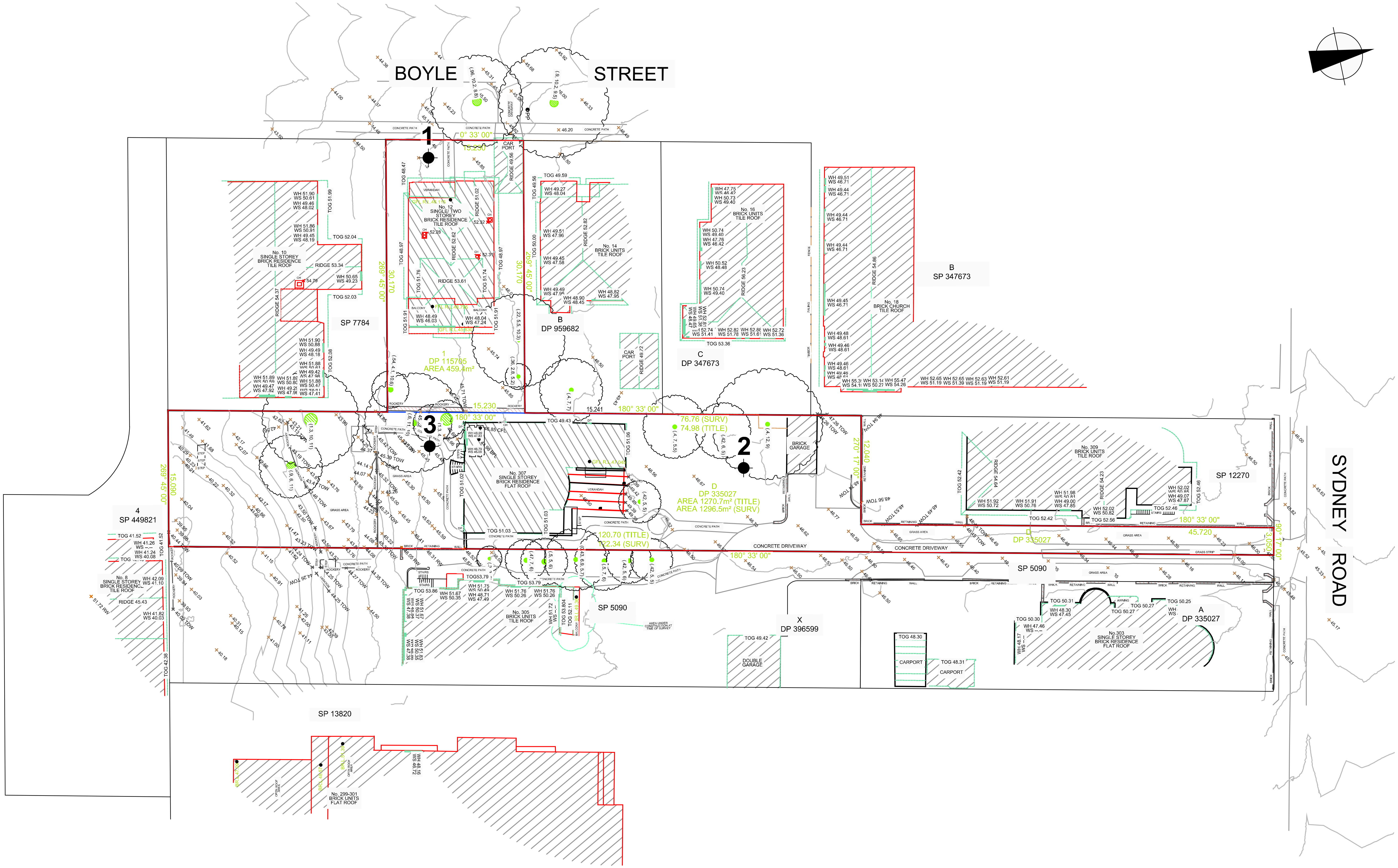
Location: 307 SYDNEY ROAD & 12 BOYLE STREET BALGOWLAH, NSW

Report No: 31201SB

Figure No: 1

JK **Geotechnics**

PLOT DATE: 31/01/2018 5:03:42 PM DWG FILE: S:\16 GEOTECHNICAL\6F GEOTECHNICAL JOBS\31000'S\31201SB BALGOWLAH\CAD\31201SB.DWG



LEGEND

BOREHOLE AND DCP TEST

0 4 8 12 16 20

SCALE 1:400 @A3 METRES

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

Title: **INVESTIGATION LOCATION PLAN**

Location: 307 SYDNEY ROAD & 12 BOYLE STREET BALGOWLAH, NSW

Report No: 31201SB

Figure No: 2

JK **Geotechnics**

PLOT DATE: 16/02/2018 12:55:40 PM DWG FILE: S:\6 GEOTECHNICAL\6F GEOTECHNICAL JOBS\31000's\31201SB BALGOWLAH\CAD\31201SB.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:

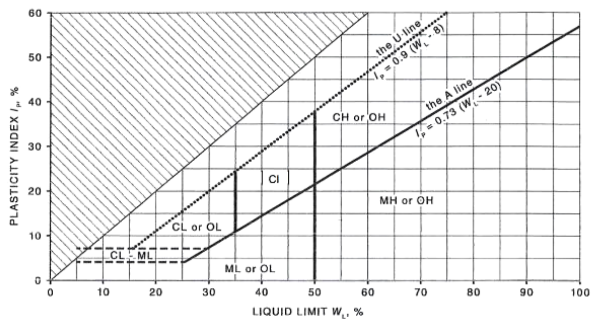
1 For a coarse grained soil with a fines content between 5% and 12%, the soil is given a dual classification comprising the two group symbols separated by a dash; for example, for a poorly graded gravel with between 5% and 12% silt fines, the classification is GP-GM.

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



Jeffery & Katauskas Pty Ltd, trading as JK Geotechnics

LOG SYMBOLS

Log Column	Symbol	Definition
Groundwater Record		Standing water level. Time delay following completion of drilling/excavation may be shown.
		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.

Log Column	Symbol	Definition	Density Index (I_D) Range (%)	SPT 'N' Value Range (Blows/300mm)
Density Index/ Relative Density (Cohesionless Soils)	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.		

Log Column	Symbol	Definition
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

			Guide to Strength	
Term	Abbreviation	Uniaxial Compressive Strength (MPa)	Point Load Strength Index $I_{s(50)}$ (MPa)	Field Assessment
Very Low Strength	VL	0.6 to 2	0.03 to 0.1	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.
Low Strength	L	2 to 6	0.1 to 0.3	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.
Medium Strength	M	6 to 20	0.3 to 1	Scored with a knife; a piece of core 150mm long by 50mm diameter can be broken by hand with difficulty.
High Strength	H	20 to 60	1 to 3	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.
Very High Strength	VH	60 to 200	3 to 10	Hand specimen breaks with pick after more than one blow; rock rings under hammer.
Extremely High Strength	EH	> 200	> 10	Specimen requires many blows with geological pick to break through intact material; rock rings under hammer.

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 $\leq$ 1mm thick
		Filled	Coating > 1mm thick
	– Thickness	mm.t	Defect thickness measured in millimetres