REPORT TO

WILLIAM RIVER STEEL GROUP OF COMPANIES

ON

PRELIMINARY GEOTECHNICAL INVESTIGATION

FOR

PROPOSED WELLNESS CENTRE AND SWIMMING SCHOOL

AT

NO.145 OLD PITTWATER ROAD, BROOKVALE, NSW

Date: 23 September 2021

Ref: 34378LCrpt

JKGeotechnics

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Report Reference	Report Status	Report Date
34378LCrpt	Final Report	23 September 2021

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1 INTRODUCTION

This report presents the results of a preliminary geotechnical investigation for a proposed Wellness Centre and Swimming School at No.145 Old Pittwater Road, Brookvale, NSW. The location of the site is shown in Figure 1. The investigation was commissioned by Mr Chris Kelly of The Williams River Steel Group of Companies by email dated 24 August 2021, Purchase Order No. JN132734. The commission was on the basis of our fee proposal dated 23 August 2021, Ref: P54868L.

We have been provided with architectural drawings prepared by Quattro Architecture (Project No. 21-0655, DA-A-000 to DA-A-049, DA-A-050, DA-A-100, DA-A-100 to DA-A-102 all revision A dated 27 July 2021). From these drawings we understand that the existing warehouse structure including portions of the existing on grade car park will be demolished. Following demolition, a new two storey wellness centre and swim training school will be constructed. The proposed ground floor level of the new building is close to existing levels and we anticipate only minor excavations will be required for the purpose of subgrade preparation and site levelling works. Swimming pools are proposed within the northern and north-eastern portions of the building; however, we understand these will be constructed above ground and not require any excavation. A new on grade car park is proposed on the southern side of the proposed building.

The purpose of our preliminary investigation was to carry out some shallow boreholes, laboratory testing and complete a desktop study of geological maps and previous geotechnical investigations carried out by JK Geotechnics in the vicinity of the site, together with a brief walkover inspection. Based on the results of the geotechnical investigation and available information, we have provided herein our comments on the expected subsurface profile and geotechnical issues for the proposed development to assist with the Development Application process. We have Cone Penetration testing (CPT) scheduled for a later date once a rig becomes available and once this has been completed an updated geotechnical report will be provided with more detailed comments and recommendations for use in design.

2 INVESTIGATION PROCEDURE

2.1 Desktop Study

A search of the JK Geotechnics project database was completed to determine likely subsurface conditions based on geotechnical investigations completed in the vicinity of the site. A review of the Sydney 1:100,000 Geological Series Sheet 9130 was also completed.

2.2 Fieldwork

Prior to the commencement of the fieldwork a Dial Before you Dig (DBYD) search was undertaken and the borehole locations were electromagnetically scanned by a specialist buried services subcontractor.

The fieldwork was carried out on 6 September 2021 and comprised the drilling of three boreholes (BH1 to BH3 inclusive). Boreholes were drilled using our Landcruiser mounted EZI probe drilling rig to total depths of 2m below existing ground surface levels. The purpose of the boreholes was to identify the subsurface profile and collect bulk samples for laboratory testing purposes.

The borehole locations, as shown on the attached Figure 2, were set out by taped measurements from existing surface features. The approximate surface levels, as shown on the borehole logs, were estimated by interpolation between spot levels and contours shown on the supplied survey plan by North Western Surveys (Project Reference: 18980, dated 6 July 2021). The datum of the levels is Australian Height Datum (AHD).

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

Our Geotechnical Engineer, Mr Bryan Zheng, set out the borehole locations, nominated the sampling locations, and prepared logs of the strata encountered. The borehole logs are attached to this report together with a set of explanatory notes, which describe the investigation techniques, and their limitations, and define the logging terms and symbols used.

Selected soil samples were returned to Soil Test Services Pty Ltd (STS) a NATA accredited laboratory, for testing to determine standard compaction properties and soaked CBR values. The results of the laboratory testing are presented in the attached STS Table A.

3 RESULTS OF INVESTIGATION

3.3 Site Description

The site is located within relatively low lying and level topography.

The site itself is located within the Westfield Warringah Mall Shopping Complex and at the time of the investigation, comprised an on-grade asphalt concrete (AC) surfaced car park and a two storey high metal warehouse with a saw tooth roof. Several metal awnings were present on the eastern side of the building. A concrete block wall is located on the southern side of the building forming a yard area. Vegetation comprised medium to large sized trees along the northern boundary and shrubs located throughout the kerbside areas of the car park.

To the north of the site is a neighbouring car park lot which comprises an on grade car park. The surface levels within the western portion of the boundary were similar, however towards the eastern end, the neighbouring site is approximately 1.2m higher and is retained by a timber koppers log wall.

On the western side of the site is a two storey concrete block warehouse set back about 2m from the common boundary. A brick fence with a height of about 2.5m is positioned along the boundary and appeared to be in good condition. The ground surface levels were similar across the boundary and the neighbouring warehouse appeared to be in good condition upon a cursory inspection.

To the south of the site is the accessway from Old Pittwater Road which comprises an on grade roadway below an elevated concrete overpass which provides access to the shopping complex. The elevated overpass is supported on concrete columns. The surface level of the roadway beneath the overpass is approximately 1m below the subject site's car park with a shallow vegetated batter along the western portion of the boundary, and a concrete ramp along the eastern portion of the boundary facilitating the differences in surface levels.

The area to the east of the site comprises two-storey brick and concrete structures of the shopping complex which all appeared to be in good condition upon a cursory inspection. The ground floor level of the adjacent buildings appeared to be about 1m lower than the subject. It was not possible to determine the type of retaining structure along this section of the boundary. However, it is likely internal walls support the subject site.

3.4 Subsurface Conditions

3.4.1 Desktop Review

Reference to the Sydney 1:100,000 Geological Series Sheet 9130 indicates that the site is underlain by Quaternary alluvial deposits overlying the Hawkesbury Sandstone.

We have completed a geotechnical investigation for the neighbouring site at No.113 Old Pittwater Road which bounds the site to the south-west. That investigation was for a warehouse building and the results of that previous investigation is summarised below.

The previous investigation was carried out in 1986 and comprised the auger drilling of six boreholes within the nearby site to depths ranging from 3m to 17.4m. The boreholes closest to the subject site were BH3 and BH5 which encountered natural sand and sandy fill from the surface, respectively. Alluvial soils were encountered in both boreholes and comprised alternating layers of clayey sand, sandy clay and sand with varying amounts of organic content. Residual soils were encountered at a depth of 12.3m and comprised sandy clay of very stiff strength. Based on Standard Penetration test (SPT) 'N' and 'NC' (solid cone) values the clayey material ranged in strength from firm to stiff and the sandy layers ranged in relative density from very loose to medium dense. BH2 and BH4 were extended to the underlying sandstone bedrock which was encountered at depths of about 14.4m and 15.3m, respectively.

Groundwater seepage was encountered during auger drilling at about 2m depth.

3.4.2 Geotechnical Boreholes

Boreholes BH1 and BH2 encountered fill comprising gravelly sand and sand to depths of 0.8m in BH1 and 0.9m in BH2. BH3 encountered a cemented gravelly fill to a depth of 0.4m. Alluvial soils were encountered below the fill in all three boreholes and comprised sand and sandy clay.

Groundwater seepage was encountered in BH2 and BH3 at a depth of 1.8m.

3.5 Laboratory Test Results

The Four Day Soaked CBR tests carried out on samples of the sandy fill and alluvial sandy clay resulted in soaked CBR values of 25% and 9% when compacted to 98% of Standard Maximum Dry Density (SMDD) and at their respective Standard Optimum Moisture Contents (SOMC).

4 COMMENTS AND RECOMMENDATIONS

The following comments and recommendations are preliminary only and have been based on the limited subsurface information obtained to date and review of nearby subsurface investigation data. Further investigations are required to confirm and amplify these preliminary comments and recommendation.

4.1 Inferred Subsurface Conditions

Based on the results of our limited geotechnical investigation and the previous geotechnical investigation carried out to the south-west of the subject site, we expect that the subsurface conditions below the site will comprise predominantly sandy and gravelly fill overlying quaternary age alluvial sands and clays of variable relative densities and strength.

The above inferred subsurface profile may be used for planning purposes, but will need to be confirmed to allow detailed design. A detailed geotechnical investigation of the site must be carried out to determine the actual subsurface conditions and allow detailed design. This should include the completion of Continuous Penetration Testing (CPT) as this gives a better assessment of relative density/material strength of the anticipated alluvial soils. If structural loads are expected to be high then cored boreholes would be beneficial to prove the bedrock and optimise bearing pressures for piled footings.

4.2 Dilapidation Reports

We recommend that detailed internal and external dilapidation reports be completed on the neighbouring buildings to the west prior to commencement of the site works (including demolition). Consideration should also be given to completing dilapidation reports on the existing buildings and structures to the south and east (within the Westfield complex) however if these buildings are owned by the subject site owners then this can be assessed by the site owners. The dilapidation surveys should include a detailed inspection of the internal and external portions of the properties. All defects should be rigorously described, mapped and photographed. The respective owners of the adjoining property should be asked to confirm that the dilapidation report presents a fair record of existing conditions by signing a copy of the report. The dilapidation report can then be used as a benchmark for assessing possible future damage claims. The dilapidation reports should be reviewed by the geotechnical and structural engineers (the engineers) prior to any works commencing on site.

4.3 Demolition and Site Works

All demolition should only be carried out by appropriately qualified, experienced and insured demolition contractors. We recommend that a demolition methodology be prepared by the demolition contractor and that it be approved by the engineers prior to demolition commencing. The aim of the demolition methodology would be to ensure that support is maintained to boundary walls and adjoining structures during all phases of the demolition works, and that consideration is being given to the use of heavy plant and their vibrations during the works.

Due to the presence of sandy soils, which will extend onto the adjoining sites, and sensitive boundary structures, we recommend that tracking of plant and breaking up of concrete slabs etc. be carried out with caution. Sudden start/stop movements may result in ground vibration damage to neighbouring structures and boundary walls. The use of hydraulic impact hammers to break up concrete slabs or other structural elements should be avoided. Concrete elements should be saw cut into smaller pieces before being removed with the buckets of an excavator.

During demolition, earthworks and footing construction, we recommend at least some quantitative vibration monitoring be carried out. The monitors should be set to record peak particle velocities and vibration frequencies. Depending on the results of the dilapidation reports and the sensitive nature of the adjoining structures and boundary walls, full-time quantitative monitoring may be warranted. Where full-time vibration monitoring is carried out, the monitors should also have flashing and audible warning signals to warn when vibrations exceed tolerable levels. If tolerable levels of vibration are exceeded, then works will need to cease and a review of the methods and procedures being adopted will need to be undertaken by the geotechnical engineers and structural engineers. Following review of the dilapidation reports on adjoining properties by the engineers, the engineers can determine the extent of monitoring and set the tolerable vibration limits. The engineers can also provide an assessment of any additional works that may be required prior to any works commencing, such as assessment of adjoining footing systems, and/or assessment of the structural integrity of adjoining structures.

4.4 Earthworks & Subgrade Preparation

The proposed floor slab and pavement levels will be close to the existing ground surface and assuming excavation and replacement of the existing fill is not carried out, the earthworks will predominantly comprise minor site levelling.

If the building floor slabs are designed as fully suspended floor slabs no particular subgrade preparation measures will be required below the buildings other than stripping of any existing pavements, vegetation and root affected soils. Any fill placed should be approved by the geotechnical engineers before it is transported to site. Non-reactive granular fill should be used to avoid the risk of high plasticity clay fill swelling and producing unacceptable uplift pressures on the underside of the slab.

Within the pavement areas and where slabs on grade are being considered, further assessment of the subgrade conditions, including provision or proposed floor loads will be required, so that the extent and risk of ground settlement can be assessed.

For pavements areas, and if slabs on grade are determined by further assessment to be feasible the following subgrade preparation is likely to be required;

- Existing pavements, vegetation and root affected soils should be stripped,
- The exposed subgrade should then be proof rolled with at least 7 passes of a minimum 8 tonne dead weight, smooth drum, **non-vibratory roller**. Subject to the results of dilapidation surveys and vibration monitoring requirements, smaller (lighter) compaction equipment may need to be used.
- The final pass of the proof rolling should be carried out in the presence of a geotechnical engineer to detect any weak subgrade areas.
- Any weak areas detected should be locally excavated to a sound base and the excavated material replaced with engineered fill, or as advised by the geotechnical engineer during the proof rolling.

Following treatment of any weak areas, engineered fill may be placed in horizontal layers to the required level. Engineered fill should preferably comprise well graded granular materials, such as crushed sandstone, free of deleterious substances and having a maximum particle size not exceeding 75mm. Such fill should be compacted in horizontal layers of not greater than 300mm loose thickness, to a density of at least 98% of Standard Maximum Dry Density (SMDD). For backfilling confined excavations such as service trenches, a similar compaction to engineered fill should be adhered to, but if light compaction equipment is used then the layer thickness should be limited to 100mm loose thickness. Density tests should be regularly carried out on the fill to confirm the above specifications are achieved.

4.5 Batters and Retaining Walls

Any batters or retaining walls, if any, will be of low height (less than 2m) and the following general advice should be followed. If more substantial batters or retaining walls are proposed then specific geotechnical advice would be required.

Temporary batters should be no steeper than 1 Vertical in 1.5 Horizontal (1V:1.5H). Such batters should remain stable in the short term provided all surcharge loads, including construction loads and adjoining footing loads, are kept well clear of the crest of the batters (at least twice the height of the batter from the crest). Where batters are to be excavated within 5m of adjoining footings, then a specific assessment should be made of the adjoining footing type, its depth and its founding stratum, so that further specific advice on the batter slope can be made by the geotechnical engineers.

Permanent batters should be no steeper than 1V:2H, but flatter batters of the order of 1V:3H may be preferred to allow 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 run-off should be directed away from all temporary and permanent batters to also reduce erosion.

Where free-standing retaining walls are required, they may be designed for a triangular earth pressure distribution. Where movement of these walls is acceptable (i.e. movement sensitive structures are not located within the zone of influence of the excavation, which is defined as a distance extending 2H back from the crest of the retaining wall, where H is the total retained height), a coefficient of active lateral earth pressure, k_a, of 0.35 may be adopted, assuming a level backfill surface. A bulk unit weight of $20kN/m^3$ should also be used for the soil. Appropriate surcharge and hydrostatic loads should be added to the above pressures.

4.6 Footings

Given the building size and anticipated upper profile of sandy fill and alluvial sands and clays of very loose relative density and stiff strength, high level footings are unlikely to be appropriate. Therefore, we consider piled footings to be the most suitable footing design.

Our preferred option would be to support the building on piles founded within the underlying sandstone bedrock. However, it may be possible to support the building on piles within the underlying alluvial sands of at least medium dense relative density. However, this cannot be determined until CPT testing has been completed to determine the depth and thickness of such material, if present beneath the site.

Considering the nature of the sandy soils and high water table, a suitable pile type would include machine drilled CFA piles. Piles socketed into the underlying sandstone bedrock may be designed based on an allowable bearing pressure (ABP) of 1000kPa.

4.7 Floor Slabs and Pavement Design

4.7.1 Floor Slabs

Where the ground floor slab is to be cast on soils, it should be isolated from the structural footings and the subgrade will need to be inspected by a geotechnical engineer who will need to observe the proof rolling of the subgrade to detect any weak subgrade areas. If any weak areas are detected they should be treated as advised by the geotechnical engineer. The final subgrade preparation measures will depend on the ground floor slab design and should be determined once the design is known. Due to the potential shrink-swell nature of the clay soils, If slabs are to be cast on clayey soils further advice should be sought from this office.

A granular layer of at least 100mm thickness should be placed below the ground floor slab to provide a uniform base below the slab and to assist compaction of the sands below.

4.7.2 Pavement Design

The design of new pavements will depend on subgrade preparation, subgrade drainage, the nature and composition of the natural soils exposed or fill excavated or imported to the site that forms the subgrade, as well as vehicle loadings and use. Various alternative types of construction could be used for the pavements. Concrete construction would undoubtedly be the best in areas where heavy vehicles manoeuvre such as trucks turning and manoeuvring. Flexible pavements may have a lower initial cost but maintenance will be higher. These factors should be considered when making the final choice.

Prior to the placement of pavements, the recommendations set out above in *Section 4.4 Earthworks and Subgrade Preparation* should be closely followed.

For the design of flexible pavements formed over the clayey subgrade we recommend that a CBR value of 5% be adopted. Where flexible pavements are formed over sandy subgrades we recommend that a CBR value of 10% be adopted.

Concrete pavements should be supported on at least a 100mm thick sub-base of good quality fine crushed rock such as RTA QA Specification 3051 unbound base (e.g. DGB20), and compacted to a minimum density ratio of 98% of Modified Maximum Dry Density (MMDD). Slab joints should be designed to resist shear forces but not bending moments by providing dowelled or keyed joints.

For flexible pavements we recommend that all base course materials comprise DGB20 in accordance with RTA QA Specification 3051 unbound base. The DGB20 material should be compacted in maximum 200mm thick loose layers to at least 98% of MMDD.

4.8 Further Geotechnical Investigations

- Continuous penetration tests (CPT) to a minimum depth of about 20m or prior refusal.
- Cored boreholes if column loads are high and it would be advantageous to achieve pile end bearing pressures greater than 1000kPa
- Inspection of subgrade proof rolling for floor slabs and pavements to identify any weak areas.

5 GENERAL COMMENTS

The recommendations presented in this report are based on an inferred subsurface profile based on previous geotechnical investigations carried out on nearby sites. A site-specific geotechnical investigation will be required. The comments and recommendations provided herein must be confirmed and amplified as part of the detailed geotechnical investigation.

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. If the natural soil has been stockpiled, classification of this soil as Excavated Natural Material (ENM) can also be undertaken, if requested. However, the criteria for ENM are more stringent and the cost associated with attempting to meet these criteria may be significant. 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.

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SOIL TEST SERVICES

ABN 43 002 145 173

TABLE A
FOUR DAY SOAKED CALIFORNIA BEARING RATIO TEST REPORT

Client:	JK Geotechnics	**Report No.:**	34378LC - A
Project:	Proposed Wellness Centre	**Report Date:**	20/09/2021
Location:	Westfield Shopping Complex - 145 Old Pittwater Road, Brookvale, NSW	**Page 1 of 1**	

BOREHOLE NUMBER	BH 1	BH 3
DEPTH (m)	0.10 - 0.70	0.40 - 1.00
Surcharge (kg)	9.0	9.0
Maximum Dry Density (t/m^3)	1.92 STD	1.83 STD
Optimum Moisture Content (%)	12.6	15.6
Moulded Dry Density (t/m^3)	1.87	1.79
Sample Density Ratio (%)	98	98
Sample Moisture Ratio (%)	103	105
Moisture Contents		
Insitu (%)	7.3	20.8
Moulded (%)	13.0	16.4
After soaking and		
After Test, Top 30mm(%)	15.2	17.8
Remaining Depth (%)	14.2	17.5
Material Retained on 19mm Sieve (%)	0	0
Swell (%)	0.0	0.0
C.B.R. value:		
@5.0mm penetration	25	9

NOTES: Sampled and supplied by client. Samples tested as received.

- Refer to appropriate Borehole logs for soil descriptions
- Test Methods : AS 1289 6.1.1, 5.1.1 & 2.1.1.
- Date of receipt of sample: 08/09/2021.
- BH 3 dried back prior to testing as the sample was too saturated.

NATA

NATA Accredited Laboratory
Number:1327

Accredited for compliance with ISO/IEC 17025 - Testing.
This document shall not be reproduced except
In full without approval of the laboratory. Results relate only to
the items tested or sampled.

20/09/2021
Authorised Signature / Date
(D. Treweek)

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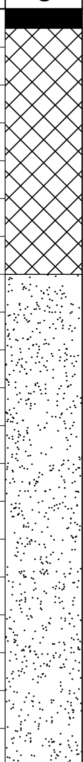
BOREHOLE LOG

Borehole No.
1

Client: WILLIAMS RIVER STEEL
Project: PROPOSED WELLNESS CENTRE
Location: WESTFIELD SHOPPING COMPLEX - 145 OLD PITTWATER ROAD, BROOKVALE, NSW

Job No.: 34378LC **Method:** SPIRAL AUGER **R.L. Surface:** ≈ 11.7m
Date: 6/9/21 **Datum:** AHD
Plant Type: EZI-PROBE **Logged/Checked by:** B.Z./T.C.



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			0			ASPHALTIC CONCRETE: 50mm.t				
					-	FILL: GRavelly sand, fine to coarse grained, dark brown and grey, fine to medium grained igneous and sandstone gravel.	M			
			0.5			FILL: Sand, fine to medium grained, dark brown and brown.				
					SP	SAND: fine to medium grained, light brown.	M			ALLUVIAL
			1							
						as above, but brown and yellow brown, with with medium plasticity clay.				
			1.5							
			2			END OF BOREHOLE AT 2.0m				
			2.5							
			3							
			3.5							

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BOREHOLE LOG

Borehole No. 2

1/1

Client: WILLIAMS RIVER STEEL

Project: PROPOSED WELLNESS CENTRE

Location: WESTFIELD SHOPPING COMPLEX - 145 OLD PITTWATER ROAD, BROOKVALE, NSW

Job No.: 34378LC **Method:** SPIRAL AUGER **R.L. Surface:** ≈ 11.5m

Date: 6/9/21 **Datum:** AHD

Plant Type: EZI-PROBE **Logged/Checked by:** B.Z./T.C.

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
			0			ASPHALTIC CONCRETE: 50mm.t				
					-	FILL: Gravelly sand, fine to medium grained, dark brown, fine to medium grained igneous gravel.	M			
			0.5			FILL: Sand, fine to medium grained, grey and light brown.				
	DS		1		SP	SAND: fine to medium grained, brown and grey, with medium plasticity clay bands.	M			ALLUVIAL
			1.5							
▶							M-W			EXTRA MOISTURE ENCOUNTERED AT 1.8m DEPTH
			2			END OF BOREHOLE AT 2.0m				
			2.5							
			3							
			3.5							

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BOREHOLE LOG

Borehole No.
3
1/1

Client: WILLIAMS RIVER STEEL
Project: PROPOSED WELLNESS CENTRE
Location: WESTFIELD SHOPPING COMPLEX - 145 OLD PITTWATER ROAD, BROOKVALE, NSW

Job No.: 34378LC **Method:** SPIRAL AUGER **R.L. Surface:** ≈ 11.3m
Date: 6/9/21 **Datum:** AHD
Plant Type: EZI-PROBE **Logged/Checked by:** B.Z./T.C.

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
			0			ASPHALTIC CONCRETE: 50mm.t				
					-	CEMENTED GRAVEL: 350mm.t				
			0.5		CI	Silty sandy CLAY: medium plasticity, dark brown, fine to medium grained sand.	w>PL			ALLUVIAL
			1		SP	SAND: fine to medium grained, dark brown and grey.	M			
			1.5							
▶							M-W			EXTRA MOISTURE ENCOUNTERED AT 1.8m DEPTH
			2			END OF BOREHOLE AT 2.0m				
			2.5							
			3							
			3.5							

AERIAL IMAGE SOURCE: MAPS.AU.NEARMAP.COM

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

Title:	**SITE LOCATION PLAN**
Location:	145 OLD PITTWATER ROAD, BROOKVALE, NSW
Report No: 34378LC	Figure No: 1

JKGeotechnics

PLOT DATE: 23/09/2021 2:20:55 PM DWG FILE: Y:\34000'S\34378L BROOKVALE\CAD\34378LC.DWG

0 4 8 12 16 20 SCALE 1:400 @A3 METRES This plan should be read in conjunction with the JK Geotechnics report.	Title: **BOREHOLE LOCATION PLAN** Location: 145 OLD PITTWATER ROAD, BROOKVALE, NSW Report No: 34378LC Figure No: 2	

JKGeotechnics

PLOT DATE: 20/09/2021 10:47:23 AM DWG FILE: Y:\34000'S\34378L BROOKVALE\CAD\34378LC.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:

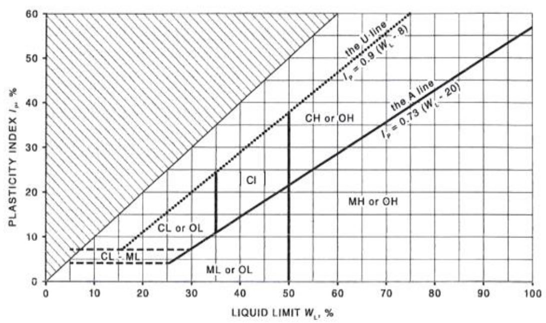
1 For a coarse grained soil with a fines content between 5% and 12%, the soil is given a dual classification comprising the two group symbols separated by a dash; for example, for a poorly graded gravel with between 5% and 12% silt fines, the classification is GP-GM.

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

Term	Abbreviation	Uniaxial Compressive Strength (MPa)	Guide to Strength	
			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 ≤ 1mm thick
		Filled	Coating > 1mm thick
	– Thickness	mm.t	Defect thickness measured in millimetres