Proposed Stables Development Geotechnical Assessment

Newcastle Jockey Club, Broadmeadow

NEW20P-0194-AA
12 January 2021

LABORATORY (NSW) PTY LTD

GEOTECHNICAL I LABORATORY I EARTHWORKS I QUARRY I CONSTRUCTION MATERIAL TESTING

12 January 2021

Newcastle Jockey Club
C/- Avid Project Management Pty Ltd
45 Hargrave Street
CARRINGTON NSW 2294

Attention: David Read

Dear David,

RE: PROPOSED STABLES DEVELOPMENT
NEWCASTLE JOCKEY CLUB, BROADMEADOW
GEOTECHNICAL ASSESSMENT

Please find enclosed our Geotechnical Assessment report for the proposed Newcastle Jockey Club stables development to be located at the corner of Darling Street and Chatham Street, Broadmeadow.

The report includes results of the geotechnical investigations, and recommendations for site classification in accordance with AS2870-2011, "*Residential Slabs and Footings*", foundation and retaining wall design parameters, pavement thickness design and construction for internal roads and car parks, infiltration testing results, acid sulfate soil conditions, excavation conditions and earthworks.

If you have any further questions regarding this report, please do not hesitate to contact Ben Bunting, Shannon Kelly or the undersigned.

For and on behalf of Qualtest Laboratory (NSW) Pty Ltd

Jason Lee
Principal Geotechnical Engineer

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Figure AA1: Site Plan and Approximate Test Locations

Appendix A: Results of Field Investigations

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1.0 Introduction

Qualtest Laboratory NSW Pty Ltd (Qualtest) is pleased to present this report to Avid Project Management Pty Ltd (Avid) on behalf of the Newcastle Jockey Club, for the proposed stables development to be located at the corner of Darling Street and Chatham Street, Broadmeadow.

Based on the Brief and Concept Drawings provided in an email dated 11 November 2020, it is understood that the proposed development includes demolition of the existing Race Day Tie-Up Stalls and construction of the new Stables Development catering for approximately 500 horses.

The scope of work is in general accordance with the Geotechnical Brief prepared by MPC Consulting Engineers (MPC), dated 10 November 2020, and as outlined in Qualtest proposal ref. NEW20P-Avid.NJC.01, 16 November 2020.

The scope of work for the geotechnical assessment included providing discussion and recommendations on the following:

- Description of soil profile;
- Groundwater observations;
- Site classification to AS2870-2011, "*Residential Slabs and Footings*";
- High level footing and deep footing recommendations and design parameters (within depth of proposed investigation);
- Retaining wall design parameters;
- Pavement profile design (flexible pavement for heavy vehicle use, rigid pavement for small forklifts and light vehicle use);
- Recommendations for site preparation and excavations, including:
    - Batter slope recommendations (permanent and temporary);
    - Suitability for site won materials to be re-used as controlled fill; and,
    - Compaction recommendations.
- Comment on Acid Sulphate soils (where relevant for the site); and,
- Test for hydraulic conductivity / soil permeability in the location indicated on the sketch by Avid.

This report presents the results of the field work investigations and laboratory testing, and provides recommendations for the scope outlined above.

2.0 Field Work

Field work investigations were carried out on 26 November 2020 and comprised of:

- DBYD search and scanning of proposed test locations using an accredited professional cable locator to check for the presence of underground services;
- Drilling of 12 boreholes (BH01 to BH12) using a 2.7 tonne rubber tracked excavator equipped with a 300mm auger to a depth 2.80m;
- One borehole (BHI01) was drilled using a hand auger to a depth of 1.00m for in-situ permeability testing;

- Dynamic Cone Penetrometer (DCP) Tests were undertaken at the borehole locations to assist in the interpretation of the in-situ density / consistency of the soil to depths ranging from 1.35m to 1.65m.
- Bulk disturbed samples, small bag samples, and undisturbed samples were taken for subsequent laboratory testing; and,
- Boreholes were backfilled with the excavation spoil and compacted by hand tools and excavator tracks and auger.

Investigations were carried out by an experienced Geotechnical Engineer from Qualtest who located the boreholes, carried out the testing and sampling, produced field logs of the boreholes, and made observations of the site surface conditions.

Approximate borehole locations are shown on the attached Figure AA1. Boreholes were located in the field by use of hand held GPS and relative to existing site features including topographic features, lot boundaries, existing developments and trees.

Engineering logs of the boreholes and DCP test results are presented in Appendix A. Indicative density / consistencies of fill / topsoil and granular soil layers shown on logs are generally based upon limited visual / tactile assessment only, with reference to DCP test results where within the depth range of the tests. If needed to be confirmed then further assessment should be undertaken.

3.0 Site Description

3.1 Surface Conditions

The site is located north-east of the corner of Darling Street and Chatham Street at Broadmeadow as shown on Figure AA1. The site is located in the south-western corner of Lot 13 DP227704 which is the broader lot encompassing the Newcastle Racecourse, and on the western parts of Lot 82 DP1138209 and Lot 14 DP227704 which are rectangular lots aligned parallel to Darling Street. The site comprises a roughly trapezoidal shaped area of about 3.0ha.

The site is bounded by the racecourse to the north, NJC property including three nearby existing buildings to the east, Darling Street to the south, and Chatham Street to the west.

The site is situated in an approximately flat alluvial floodplain area, which drains through the stormwater system connected to Darling Street and Chatham Street.

Based on a site plans provided including survey information, surface levels on the site are generally inferred to be in the order of RL6.0m to RL6.5m AHD.

Existing developments are mostly positioned in the north-eastern to eastern parts of the site, and include the current raceday stalls, a horse swim area and a maintenance building.

The south to south-western areas of the site are generally raceday parking areas vegetated by established grass cover, with several mature trees located near to boundaries. Some areas of sealed and unsealed driveways are present, and a sign is positioned beside the western boundary within a raised bed retained by timber walls.

The north-eastern area of the site includes a sealed pavements with a turfed median area and a shed beside the western boundary.

Photographs of the site taken on the day of the site investigations are shown as follows.

Photograph 1: From near south-western corner of site, facing north.

Photograph 2: From near south-western corner of site, facing northeast.

Photograph 3: From near south-eastern corner of site, facing west.

Photograph 4: From near south-eastern corner of site, facing north.

Photograph 5: From near BH06 location, facing south.

Photograph 6: From near BH06 location, facing west.

Photograph 7: From roughly midway along western boundary of site, near BH07, facing north.

Photograph 8: From near BH07, facing northeast.

Photograph 9: From roughly midway along western boundary of site, north of existing raceday stalls, facing southeast.

Photograph 10: From north of existing raceday stalls, facing south.

Photograph 11: From near existing raceday stalls, facing southeast towards north-eastern corner of site.

Photograph 12: From near existing raceday stalls, facing south.

Photograph 13: From near western boundary of site, south of existing raceday stalls, facing northwest.

Photograph 14: From near western boundary of site, south of existing raceday stalls, facing north.

3.2 Subsurface Conditions

Reference to the 1:100,000 Newcastle-Hunter Area Coastal Quaternary Geology Sheet indicates the site to be underlain by Pleistocene undifferentiated estuarine plain which includes clay, silt, fluvial sand, marine sand, and shell soil types.

Table 1 presents a summary of the typical soil types encountered at borehole locations during the field investigation, divided into representative geotechnical units.

TABLE 1 – SUMMARY OF GEOTECHNICAL UNITS AND SOIL TYPES

Unit	Soil Type	Description
1A	FILL – TOPSOIL / ROOT AFFECTED	Silty SAND / Gravelly Silty SAND – fine to medium grained (mostly fine grained), dark brown / brown to grey, fines of low plasticity, fine to coarse grained angular gravel, root affected. Clayey Gravelly SAND – fine to coarse grained, black, fine to medium grained angular gravel, fines of low plasticity, root affected.
1B	FILL – OTHER	SAND / Gravelly SAND – fine to coarse grained, black / pale brown, fine to medium grained angular gravel. Silty Sandy GRAVEL – fine to medium grained, sub-rounded to sub-angular, pale orange-brown, fine to coarse grained sand, fines of low plasticity. Silty SAND / Gravelly Silty SAND – fine to coarse grained, black to dark grey / grey-brown, fines of low plasticity. With some coal chitter / lightweight slag / ash in places. Sandy CLAY (BH01 only) – medium to high plasticity, black, fine to coarse grained sand, with some fine to medium grained angular gravel. ASPHALT (BH08 only) – Up to 30mm thick.
2	TOPSOIL	Silty SAND – fine to medium grained, brown, fines of low plasticity, root affected.
3A	ALLUVIUM (Sandy CLAY / Clayey SAND)	Sandy CLAY – medium / medium to high plasticity, dark grey to grey / pale grey, with some brown, fine to medium grained sand. Sandy CLAY / Clayey SAND – medium plasticity, grey to pale grey with some brown / orange-brown, fine to medium grained sand. Clayey SAND – fine to medium grained, grey to white with some orange-brown, fines of low to medium plasticity.
3B	ALLUVIUM (SAND)	SAND – fine to medium grained, pale grey to white / grey /orange-brown, becoming dark grey to dark brown / black at increasing depth, fines of low plasticity. Weakly cemented layers in places.

Table 2 contains a summary of the distribution of the geotechnical units at the borehole locations.

TABLE 2 – SUMMARY OF GEOTECHNICAL UNITS ENCOUNTERED AT EACH BOREHOLE LOCATION

Location	Unit 1A Fill – Topsoil / Root Affected	Unit 1B Fill – Other	Unit 2 Topsoil	Unit 3A Alluvium (Sandy CLAY / Clayey SAND)	Unit 3B Alluvium (Sand)
BH01	0.00 - 0.25	0.25 - 0.70	-	0.70 – 1.50	1.50 - 2.80
BH02	0.00 - 0.40	-	-	0.40 - 1.90	1.90 - 2.80
BH03	0.00 - 0.15	0.15 - 0.45	-	0.45 - 1.60	1.60 - 2.80
BH04	-	-	0.00 - 0.20	0.20 - 1.40	1.40 - 2.80
BH05	-	0.00 - 0.20	-	0.20 - 0.90	0.90 - 2.80
BH06	-	0.00 - 0.60	-	0.60 - 1.70	1.70 - 2.80
BH07	0.00 - 0.20	0.20 - 1.00	-	1.00 - 2.50	2.50 - 2.80
BH08	-	0.00 - 0.70	-	0.70 - 2.20	2.20 - 2.80
BH09	0.00 - 0.50	0.50 - 0.80	-	0.80 - 2.30	2.30 - 2.80
BH10	0.00 - 0.40	0.40 - 0.80	-	0.80 - 2.20	2.20 - 2.80
BH11	0.00 - 0.20	0.20 - 0.50	-	0.50 - 2.00	2.00 - 2.80
BH12	0.00 - 0.30	0.30 - 0.80	-	0.80 - 1.80	1.80 - 2.80

Slow groundwater inflows were observed at depths of approximately 2.40m beneath existing ground level at borehole locations BH02 to BH05, BH10, and BH12.

No other groundwater levels or inflows were observed in the remaining boreholes during the limited time that they remained open on the day of field work.

It should be noted that groundwater conditions can vary due to rainfall and other influences including regional groundwater flow, temperature, permeability, recharge areas, surface condition, and subsoil drainage.

4.0 Laboratory Testing

Samples collected during the field investigations were returned to our NATA accredited Warabrook Laboratory for testing which comprised of:

- (3 no.) California Bearing Ratio (CBR, 4 day soaked) & Standard Compaction tests;
- (4 no.) Shrink/Swell tests;
- (15 no.) Acid Sulfate Soil (ASS) Field Screening tests; and,
- (1 no.) Chromium Reducible Sulfur Suite test.

Results of the laboratory testing are presented in Appendix B, with CBR and Shrink / Swell test results summarised in Table 3 and Table 4, respectively. The results of Acid Sulfate Soil testing are discussed in Section 5.0.

TABLE 3 – SUMMARY OF CBR TESTING RESULTS

Location	Sample Depth (m)	Field Moisture Content (%)	Optimum Moisture Content (%)	Relationship of Field MC to OMC (%)	CBR (%)
BH02	0.40 – 0.80	25.9	22.5	3.4 WET	2.5
BH04	0.20 – 0.70	14.8	15.0	0.2 DRY	4.0
BH09	0.90 – 1.20	25.6	20.5	5.1 WET	2.5

TABLE 4 – SUMMARY OF SHRINK / SWELL TESTING RESULTS

Location	Depth (m)	Material Description	I_{ss} (%)
BH01	0.80 – 1.10	(CH) Sandy CLAY	1.7
BH05	0.40 – 0.60	(CI) Sandy CLAY	0.6
BH10	0.80 – 1.10	(CH) Sandy CLAY	0.6
BH12	0.90 – 1.10	(CH) Sandy CLAY	0.3

5.0 Acid Sulfate Soils

5.1 Risk Map

The 1:25,000 Acid Sulfate Risk Map for Wallsend (Edition Two, 1997) indicates the site is located in an area with a 'low probability' of acid sulfate soils greater than 3m below ground surface (bgs) within an Aeolian sandplain at over 4m AHD elevation.

5.2 Occurrence

Acid sulfate soils can form in a number of geologic and geomorphic landscapes provided there is a source of iron, sulfate and soil bacteria. Coastal Acid Sulfate Soils (CASS) have formed along the east coast of Australia, since the last glacial period (19,000 to 18,000 years ago), when sea levels were around 120m to 130m below today's levels.

Sea levels rose rapidly to about 7,000 years ago, reaching a height about 1.0m above the present day mean sea level (0.0m AHD), at which time they stabilised. Since that time there has been a slow accumulation of coastal sediments within the intertidal zone, including saline wetlands, salt marshes and as bottom sediments in embayments, coastal rivers, estuaries and coastal lakes. This accumulation is still occurring today.

CASS are found along most of the coast of mainland Australia, generally found below about 5m AHD where tidal ranges are large, such as northern Queensland. Along coastal areas with smaller tidal ranges, it is rare to find significant accumulations of CASS above about 2m AHD (Simpson et al 2018).

The formation of sulfidic sediments is a natural part of the sulfur cycle where sulfates from sea water, in combination with iron and sulfate reducing bacteria (SRB), combine to produce reduced inorganic sulphides (RIS). RIS can include iron disulfides (FeS2), pyrite and marcasite, monosulfides (FeS) and elemental sulfur (S8) (Sullivan et al 2018). Provided these sediments remain in an anoxic state (saturated) they are benign (Dear et al 2014, Sullivan et al 2018).

5.3 Action Criteria

In order to assess the presence of ASS, the laboratory results were compared to Action Criteria from ASSMAC (1998) Acid Sulfate Soil Manual.

The ASSMAC (1998) action levels are based on oxidisable sulfur concentrations for three differing soil textures. There are separate action levels depending on the amount of soil disturbed as a result of the proposed works. For the purposes of this assessment it has been assumed that greater than 1000 tonnes of ASS would be disturbed, or that the ASS would be Coarse texture. The applicable action levels are indicated below in Table 5.

TABLE 5 – ASSMAC (1998) ACTION CRITERIA

Texture category	Approximate clay content (%)	Action Levels	
		Net Acidity (S_{CR} or S_{POS}) (%)	Net Acidity (moles H+/tonne)
Coarse	< 5% clay*	0.03	18
Medium	5 to 40% clay*	0.06	36
Fine	> 40% clay*	0.10	62
If >1000 tonnes to be disturbed.		0.03	18
* Refer to ASSMAC, 1998 for more detailed soil texture definitions.			

5.4 Screening Tests

Screening of the twenty-two samples collected was carried out by an experienced Qualtest Environmental Scientist, at our Warabrook laboratory. The screening test report sheets are attached, and a summary of the results provided in Table 6 below.

TABLE 6 – RESULTS OF FIELD SCREENING TESTS

Sample ID	pH_F	pH_{FOX}	Reaction
BH04 0.5 - 0.6	6.09	4.80	None Observed
BH04 0.8 - 0.9	5.52	4.71	None Observed
BH04 1.0 - 1.1	5.65	4.70	None Observed
BH05 0.5 - 0.6	5.06	4.51	None Observed
BH05 0.9 - 1.0	6.04	5.18	None Observed
BH05 1.5 - 1.6	5.75	5.05	None Observed
BH05 2.0 - 2.1	5.70	3.01	Slight
BH06 0.6 - 0.7	5.88	4.88	Slight
BH06 1.0 - 1.1	5.73	4.90	None Observed
BH06 1.3 - 1.4	5.94	4.96	None Observed

Sample ID	pH_F	pH_{FOX}	Reaction
BH08 0.7 - 0.8	6.29	5.00	Slight
BH08 1.0 - 1.1	6.16	4.81	Slight
BH08 2.0 - 2.1	6.19	4.25	Slight
BH08 2.3 - 2.4	6.79	4.63	None Observed
BH10 1.2 - 1.3	6.03	4.81	None Observed

A pH_{FOX} around 3.5 or lower, can sometimes indicate a potential for reduced inorganic sulphides (RIS) to be present within the soils. Sample BH05 2.0-2.1 recorded a pH_{FOX} below 3.5.

5.5 Laboratory Results

Based on the results of the field screening, one sample was selected for laboratory analysis. The sample was dispatched to NATA accredited laboratory Eurofins MGT for Chromium Reducible Sulfur (CRS) testing. The laboratory reports are attached. A summary of the ASS laboratory results compared to action criteria are provided in Table 7, below

TABLE 7 – LABORATORY RESULTS

Sample ID	Description	pH_{KCL}	TAA (mol H+/t)	Scr (%S)	S_{NAS} Sulfur (molH+/t)	Net Acidity (%S)
BH05 2.0-2.1	SAND – fine to grained, dark brown.	5.4	6.7	0.030	N/A	0.04
Action Criteria*		**-**	**18**	**0.03**	**-**	**0.03**

*****ASSMAC (1998), Acid Sulfate Soil Manual, Table 4.4 – Action Criteria for coarse textured soil,>1000 tonnes*

S_{NAS} – Net Acid Soluble Sulphur
Scr – Chromium Reducible Sulphur
TAA – Titratable Actual Acidity
N/A – Not Applicable

The laboratory results showed that Chromium Reducible Sulfur (Scr) and net acidity were reported slightly above the adopted criteria of 0.03%S in BH05_2.0-2.1.

5.6 Conclusion and Recommendations

Based on the results of the field screening and laboratory testing, it is considered that ASS are present in some of the soils below 1.6m to 2.3m bgs. ASS are not considered to be present in the soils above 1.6m, based on field observations and results of the field screening.

Based on the results of the assessment, an ASS Management Plan would be required if excavations below 1.5m are proposed. Excavations above 1.5m would not require an ASSMP.

6.0 Discussion and Recommendations

6.1 Pavement Design

6.1.1 Design Subgrade CBR Value

Subgrade laboratory CBR test results from the current investigation at the site ranged from 2.5% to 4.0%. Based on the results of the field work and laboratory testing, and previous experience in the surrounding area, the following design California Bearing Ratio (CBR) value has been adopted for the subgrade soils for pavement thickness design.

- Design Subgrade CBR = 2.5%

Subgrade should be prepared in accordance with the site preparation requirements presented in Section 6.8. Subgrade should be compacted in accordance with the recommendations of this report.

Fill placed at road subgrade level should be assessed by the geotechnical authority. If the fill is assessed to have CBR different to that of the design CBR, then a revised pavement design will be required for that section.

6.1.2 Design Traffic Loadings

The proposed development is understood to be a private facility with pavement areas expected to include the maintenance, equine and goods drop off – pick up zones connected to Chatham Street and Darling Street with commercial and heavy vehicle traffic, and the staff car park connected to Darling Street with no large commercial or heavy vehicle traffic.

In the absence of detailed traffic data for the site, an estimate has been made in terms of equivalent standard axles (ESA's) based on the proposed use of the site by horse transport and delivery vehicles plus a separate area for car parking, with respect to Newcastle City Council (NCC) / AUS-SPEC specifications. A summary of the design traffic loadings adopted for the proposed drop off – pick up zones and car parking areas is provided in Table 8.

TABLE 8 – PAVEMENT DESIGN TRAFFIC LOADING

Road Section	Equivalent Classification (Newcastle City Council / AUS-SPEC)	Design Traffic (ESA's)
Car park	Access Place	7×10^4
Maintenance, equine and Goods drop off – pick up zones	Collector Road	1×10^6
TBC	Commercial and Industrial	1×10^7

Car park areas estimated traffic of 7×10^4 ESA's generally allows for regular light vehicle traffic, up to about 10 small (two axle) heavy vehicles per day and 2 medium sized heavy vehicle (e.g. delivery / garbage truck) per day.

Based upon a 30 year design life, the estimated traffic of 1 x 10^6 ESA's comprises an average of about 90 ESA's per day if the site were to operate 7 days per week. Based on an assumption of about 2.5 ESA's per horse transport/goods truck e.g. mostly 4 axles or less; the design traffic of 1 x 10^6 ESA's could be equated to about 35 similar heavy vehicles per day, plus light vehicles including cars and short towing vehicles such as horse floats.

The design traffic loading assumes unloading of goods trucks and using small forklifts and plant with tyre pressures and loads which do not exceed normal public road limits, and numbers of passes that do not exceed the number of trucks assumed as described above.

If the pavements are expected to be trafficked by more heavy vehicles or forklift traffic than described above, then a higher design traffic loading should be adopted. Design based upon the traffic loading from NCC specifications for Commercial and Industrial pavements is provided in case it is required.

Rigid pavement design to Austroads is based on Design Traffic NDT in terms of Heavy Vehicle Axle Groups (HVAG). The design traffic has been converted from ESA's based on an adopted ESA per Heavy Vehicle Axle Group of 0.2 to 0.3, as shown in Table 9.

TABLE 9 – RIGID PAVEMENT DESIGN TRAFFIC LOADING

Road Section	Equivalent Classification (Newcastle City Council / AUS-SPEC)	Design Traffic (HVAG)
Car park	Access Place	3.1 x 10^5
Maintenance, equine and Goods drop off – pick up zones	Collector Road	3.5 x 10^6
TBC	Commercial and Industrial	3.5 x 10^7

In the event that different design traffic design loadings are applicable, then the pavement thickness designs presented in this report should be reviewed.

6.1.3 Flexible Pavement Thickness Design

Flexible pavement thickness design has been based on the procedures outlined in:

- Austroads, "Guide to Pavement Technology, Part 2: Pavement Structural Design";
- Newcastle Development Control Plan 2012, 7.04 Movement Networks, A. Road Design;
- Newcastle Technical Manual Subdivision, June 2012;
- AUS-SPEC - DDSS D2; and,
- Australian Road Research Board, Special Report No. 41 (ARRB-SR41).

Flexible Pavement Thickness Designs are presented in Table 10.

Flexible Pavement Material Specification and Compaction Requirements are presented in Table 11.

It is recommended that each construction length be boxed out to the minimum subgrade level required by the relevant pavement thickness design. Prior to pavement construction, the exposed subgrade should be assessed by the geotechnical authority to confirm the pavement thickness requirement for that section.

TABLE 10 – FLEXIBLE PAVEMENT THICKNESS DESIGN SUMMARY

Road Section	Car Park	Maintenance, Equine and Goods Drop Off – Pick Up Zones	TBC
Equivalent Road Classification	Access Place	Collector Road	Commercial / Industrial
Design Traffic Loading (ESA's)	7×10^4	1×10^6	1×10^7
Subgrade Material	Natural Clay	Natural Clay	Natural Clay
Design Subgrade CBR (%)	2.5	2.5	2.5
Wearing Course (mm)	30 (AC10)	45 AC [(1)] (Dense Graded)	45 AC [(1)] (Dense Graded)
Base Course (mm)	120	150	150
Subbase (mm)	250	375	515
Select Fill (mm)			
Total Thickness (mm)	400	570	710

Notes:

1) Recommended to be AC14 dense graded asphalt wearing course with A15E PMB, or alternatively it is recommended that a hard wearing asphalt, such as a 'gilsonite' or 'portmix' be adopted to provide additional wearing resistance for concentrated (heavy) axle loads and/or turning (screwing) loads.
2) A 7mm primer seal should be placed over the base course prior to placement of the asphaltic concrete wearing course.
3) An allowance for subgrade replacement and/or bridging layers should be anticipated where road pavements cross gullies and in any areas where poor, wet or saturated subgrade conditions are encountered.
4) The requirement for, and extent of any subgrade replacement / select filling, should be confirmed by the geotechnical authority at the time of construction.
5) Prior to pavement construction, the exposed subgrade should be assessed by the geotechnical authority to confirm the pavement thickness requirement for that section.

TABLE 11 – PAVEMENT MATERIAL SPECIFICATION AND COMPACTION REQUIREMENTS

Pavement Course	Material Specification	Compaction Requirements
Wearing Course (AC)	DG14 AR450, Newcastle City Council requirements, Construction Specification C245.	3% to 7% Air Voids, Newcastle City Council Specification.
Base Course	CBR ≥ 80%, PI ≤ 6%	98% Modified (AS1289 5.2.1)
Subbase	CBR ≥ 30%, PI ≤ 12%	95% Modified (AS1289 5.2.1)
Select Fill *	2% cement stabilised subbase material Or CBR ≥ 15%, PI ≤ 15%, max particle size 75mm Or Stabilised Subgrade - lime stabilised with either 3% quicklime or 4% hydrated lime to achieve CBR ≥ 10%	95% Modified (AS1289 5.2.1)
Subgrade (top 300mm)	Minimum CBR = 2.5%	100% Standard (AS1289 5.1.1)
Subgrade / Fill Below	Minimum CBR = 2.5%	95% Standard (AS1289 5.1.1)

Notes:

1) All flexible road pavement materials shall be supplied to comply with requirements of AUS-SPEC, Subgroup 114 – Pavement;
2) Pavement materials for base course and subbase shall also comply with Construction Specification, C242 Flexible Pavements, Tables C242.3 and C242.4.
3) CBR = California Bearing Ratio, PI = Plasticity Index.
4) Select Fill / Subgrade Replacement options if required and/or adopted will be dependent on subgrade CBR and moisture conditions at time of construction.

6.1.4 Rigid (Concrete) Pavement Thickness Design

Rigid (concrete) pavement design has been carried out in accordance with:

- Newcastle Development Control Plan 2012, 7.04 Movement Networks, A. Road Design;
- Newcastle Technical Manual Subdivision, June 2012;
- Austroads, "Guide to Pavement Technology, Part 2: Pavement Structural Design".

Rigid Pavement Thickness Design is presented in Table 12 to Table 14.

Pavement Material Specification and Compaction Requirements are presented in Table 15.

It is recommended that each construction length be boxed out to the minimum subgrade level required by the relevant pavement thickness design. Prior to pavement construction, the exposed subgrade should be assessed by the geotechnical authority to confirm the pavement thickness requirement for that section.

Concrete Base:

The design assumes steel reinforced concrete. Dowels are required at all transverse contraction joints which should be designed by an experienced structural engineer.

In general accordance with Austroads, for areas with 1 x 10^6 HVAG or more (i.e. the maintenance, equine and goods drop off – pick up zones) the base should comprise concrete with a 28-day characteristic compressive strength of not less than 40 MPa, and flexural strength of not less than 4.5MPa.

In Car Park areas the base should comprise concrete with a 28-day characteristic compressive strength of not less than 32 MPa, and flexural strength of not less than 4.0MPa.

Areas with odd-shaped and acute cornered slabs requiring increased resistance to cracking should be designed for construction with fibre-reinforced concrete base. The base should be of flexural strength of not less than 5.5MPa, with a minimum 50kg/m^3 of steel fibre.

Subbase Options:

In the maintenance, equine and goods drop off – pick up zones, the concrete should be constructed over lightly bound sub-base (5% cement or equivalent), or Lean Concrete Sub-base (LCS) as specified in the designs.

Options have been provided for bound and unbound sub-base beneath the concrete base layer for car park pavements. Some improvement in performance may result from having a bound sub-base in all areas; however, unbound material is accepted to generally be sufficient for lightly trafficked areas (about 1 x 10^6 HVAG or less).

> Austroads Publication No. AGPT02-12 states '*while erosion of subgrade/subbase is an important distress mode for more heavily-trafficked roads, erosion is not normally of concern for lightly-trafficked roads due to the combination of low axle repetitions and low vehicle speeds which reduces the likelihood of pumping of subbase and subgrade materials*', and that in most cases a granular subbase – typically crushed rock – will provide the remaining functions for lightly-trafficked concrete streets.

It is recommended that a sub-base with higher resistance to erosion and pumping is used in areas subject to higher traffic loads including turning and braking loads such as the entrance / exit and turning circle areas. Bound sub-base material or Lean Concrete Sub-base (LCS) is recommended in those areas.

The unbound sub-base layer may be replaced by bound Subbase or Lean Concrete Sub-base (LCS) as outlined in the Pavement Thickness Design Tables.

TABLE 12 – RIGID PAVEMENT THICKNESS DESIGN SUMMARY – CAR PARK

Equivalent Road Classification	Access Place	
Design Traffic Loading (HVAG)	3.1×10^5	3.1×10^5
Sub-base Option	Unbound	Bound
Design Subgrade CBR (%)	2.5	2.5
Concrete Base (mm)	195	180
Sub-base (mm)	125 unbound	125 bound
Total Thickness (mm)	320	305

Notes:

1) The requirement for, and extent of any subgrade replacement / select filling, should be confirmed by the geotechnical authority at the time of construction.
2) The 125mm bound sub-base layer may be replaced by 100mm thickness of Lean Concrete Sub-base (LCS), with total thickness reduced accordingly where applicable.

TABLE 13 – RIGID PAVEMENT THICKNESS DESIGN SUMMARY – MAINTENANCE, EQUINE AND GOODS DROP OFF – PICK UP ZONES

Road Classification	Collector Road	
Design Traffic Loading (HVAG)	3.5×10^6	
Sub-base Option	Bound	LCS
Design Subgrade CBR (%)	2.5	2.5
Concrete Base (mm)	195	190
Sub-base (mm)	150 bound	125 LCS
Total Thickness (mm)	345	315

Notes:

1) The requirement for, and extent of any subgrade replacement / select filling, should be confirmed by the geotechnical authority at the time of construction.

TABLE 14 – RIGID PAVEMENT THICKNESS DESIGN SUMMARY – COMMERCIAL / INDUSTRIAL

Road Classification	Commercial / Industrial
Design Traffic Loading (HVAG)	3.5×10^7
Design Subgrade CBR (%)	2.5
Concrete Base (mm)	195
Sub-base (mm)	150 LCS
Total Thickness (mm)	345

Notes:

1) The requirement for, and extent of any subgrade replacement / select filling, should be confirmed by the geotechnical authority at the time of construction.

TABLE 15 – RIGID PAVEMENT MATERIAL SPECIFICATION AND COMPACTION REQUIREMENTS

Pavement Course	**Material Specification**	**Compaction Requirements**
Concrete Base – Steel Reinforced Concrete Pavement with dowelled joints.	Concrete with minimum characteristic compressive strength, fc = 40 MPa. Reinforced with SL92, or steel fibre reinforcement as specified in Section 6.1.4.	Newcastle City Council Specification
Lean Concrete Sub-base (LCS)	Concrete with minimum characteristic compressive strength, fc = 5 MPa (with fly ash) or 7MPa (without fly ash).	Newcastle City Council Specification
Bound Sub-base	CBR > 30%, PI < 12%, bound with 5% cementitious binder	95% Modified (AS1289 5.2.1)
Unbound Sub-base	CBR ≥ 30%, PI ≤ 12%	95% Modified (AS1289 5.2.1)
Select Fill / Stabilised Subgrade	2% cement stabilised subbase material Or Select, CBR ≥ 15%, PI ≤ 15%, max particle size 75mm	95% Modified (AS1289 5.2.1)
Subgrade (top 300mm)	Minimum CBR = 2.5%	100% Standard (AS1289 5.1.1)
Subgrade / Fill Below	Minimum CBR = 2.5%	95% Standard (AS1289 5.1.1)

6.1.5 Construction Considerations

Care should be taken to follow recommended construction practices when constructing new pavement adjacent to existing, including:

- A clean, vertical perpendicular surface at full depth should be cut for both transverse and longitudinal jointing. This will reduce the risk of plating and heaving effects on the pavement;
- Ensuring joints are not in wheel paths;
- Ensuring joints in sub-base / select layers are offset to joints in the base layer; and,
- Ramping between layers, and at the entry and exit points to the pavement, must be removed at all times. During construction, any temporary access ramps to properties or driveways must also be removed.

Inspection should be carried out by a geotechnical authority during construction to confirm the conditions assumed in this report and in the design.

6.2 Preliminary Site Classification to AS2870-2011

Site Classification to AS2870 is not strictly applicable to this site due to it being a series of equestrian stable buildings rather than a residential development. However, the principles of footing design and site maintenance presented therein may be taken into account for buildings such as those proposed for the site.

Based on the results of the field work and laboratory testing, the site of the proposed stable development to be located at the Newcastle Jockey Club at Broadmeadow, as shown on Figure AA1, is classified in its current condition in accordance with AS2870-2011 '*Residential Slabs and Footings*', as shown in Table 16.

TABLE 16 – SITE CLASSIFICATION TO AS2870-2011

Location	Site Classification
Locations affected by uncontrolled filling and/or topsoil of depths of greater than 0.4m. The subsurface profile encountered in boreholes was variable, from no fill observed in some boreholes, to a fill depth of up to 1.00m at BH07.	**P**
Locations with natural Soil Profile / Fill or Topsoil depth up to a maximum depth of 0.40m. Other locations within proposed development area not affected by uncontrolled fill, abnormal moisture conditions, or possible inadequate bearing capacity.	**H1**

Part of the site has been classified as **Class 'P'** in its existing condition due to the presence of uncontrolled fill and topsoil to depths of greater than 0.4m. No records of the placement or compaction of the fill material have been provided; therefore, it has been assessed to be uncontrolled fill.

The approximate extent of fill was inferred based on limited information including observation of surface features and boreholes conducted. If the depth and extent of fill needs to be known more accurately for planning, design or other purposes, then it should be investigated further.

If site re-grading works involving cutting or filling are performed after the date of this assessment the classification may change and further advice should be sought.

It is envisaged that if uncontrolled fill, topsoil and slopewash depths are reduced to less than 0.4m, witnessed and documented by a geotechnical authority, then it is likely that those areas could be re-classified as **Class 'H1'**. This should be confirmed by the geotechnical authority following fill / topsoil removal.

Alternatively, provided structures on those areas classified as **Class 'P'** due to the presence of uncontrolled fill and topsoil / slopewash to depths of greater than 0.4m are supported on engineered footings founded in stiff or better natural clay soils or medium dense or better natural sand beneath uncontrolled fill, topsoil and slopewash, they may be proportioned based on the characteristic free surface movement equivalent to that of a site classification of **Class 'H1'**.

A characteristic free surface movement of 40mm to 60mm is estimated for areas classified as **Class 'H1'**.

The effects of changes to the soil profile by additional cutting and filling and the effects of past and future trees should be considered in selection of the design value for differential movement.

Footings for the proposed development should be designed and constructed in accordance with the requirements of AS2870-2011 and/or sound engineering principles.

The classification presented above assumes that:

- All footings are founded in controlled fill (if applicable) or in the residual clayey soils or rock below all non-controlled fill, topsoil material and root zones, and fill under slab panels meets the requirements of AS2870-2011, in particular, the root zone must be removed prior to the placement of fill materials beneath slabs;
- The performance expectations set out in Appendix B of AS2870-2011 are acceptable, and that site foundation maintenance is undertaken to avoid extremes of wetting and drying;
- Footings are to be founded outside of or below all zones of influence resulting from existing or future service trenches and other excavations;
- The constructional and architectural requirements for reactive clay sites set out in AS2870-2011 are followed;
- Adherence to the detailing requirement outlined in Section 5 of AS2870-2011 '*Residential Slabs and Footings*' is essential, in particular Section 5.6, '*Additional requirements for Classes M, H1, H2 and E sites*' including architectural restrictions, plumbing and drainage requirements; and,
- Site maintenance complies with the provisions of CSIRO Sheet BTF 18, "*Foundation Maintenance and Footing Performance: A Homeowner's Guide*", a copy of which is attached in Appendix C.

All structural elements on all lots should be supported on footings founded beneath all uncontrolled fill, layers of inadequate bearing capacity, soft/loose, wet or other potentially deleterious material.

If any localised areas of uncontrolled fill of depths greater than 0.4m are encountered during construction, footings should be designed in accordance with engineering principles for Class 'P' sites.

6.3 Foundations

6.3.1 Shallow Footings

Footings should be founded in suitable material beneath all uncontrolled fill, or the fill should be removed and replaced under engineering supervision. Shallow footings may not be appropriate for some areas of the site which are affected by uncontrolled fill unless the fill is removed, or the fill is removed and replaced with approved controlled fill.

Suitability for shallow footings will be dependent on the type of fill and level of supervision carried out, and should be confirmed by the geotechnical authority at the time of construction following any site regrade works.

Shallow footings founded on stiff or better alluvial clay, medium dense or better alluvial sands, or approved controlled fill (placed under Level 1 supervision in accordance with AS3798-2007) may be proportioned for a maximum allowable bearing pressure of 100kPa, provided they are founded below any existing uncontrolled fill, topsoil, deleterious material, or very soft to firm material.

The recommended allowable bearing pressures assume that elastic settlements will be less than about 1% of least footing width; although, relevant ground movements related to reactive clay would also apply.

Inspection should be carried out by a geotechnical authority during construction to confirm the conditions assumed in this report and in the design.

6.3.2 Deep Foundation Design Parameters

Footing options for the proposed development may include piles such as progressively cased bored piles, screw piles, grout injected continuous flight auger (CFA) piles, or driven piles, founded in natural soils.

Conventional bored piers may be problematic due to the presence of sands and possibly groundwater at depth. Allowance would need to be made to progressively case the holes during drilling.

There is a risk of causing vibration-induced damage to adjacent buildings or structures with driven displacement piles. Vibration monitoring and dilapidation survey on nearby structures prior to any pile driving are recommended if driven piles are to be used.

Driven piles and steel screw piles may need to be pre-bored through the upper fill in some places depending on pile type, and allowance for this should be made.

Table 17 presents a summary of ultimate pile design parameters for deep footings (founding depth greater than 3 times maximum footing width) that have been adopted for the relevant site materials. Elastic soil parameters are also provided for use in elastic analysis of foundations.

TABLE 17 – SUMMARY OF ULTIMATE PILE DESIGN PARAMETERS

Soil Description	E (MPa)	ν	Displacement Piles		Non Displacement Piles	
			f_b (kPa)	f_s (kPa)	f_b (kPa)	f_s (kPa)
Fill	-	-	-	-	-	-
Topsoil	-	-	-	-	-	-
CLAY – Stiff or better	10	0.4	450	40	450	40
SAND – Medium Dense or better	20	0.35	900	50	900	35
f_b = Ultimate End Bearing Capacity E = Young's Modulus			f_s = Ultimate Shaft Adhesion ν = Poisson's Ratio			

Notes:

- Ultimate values occur at large settlements (>5% of minimum footing dimensions).
- The ultimate pile parameters presented in Table 17 should be used in limit state pile design in accordance with Australian Standard AS 2159-2009, *Piling – Design and Installation*.
- A geotechnical strength reduction factor should be adopted for use with the above ultimate soil and rock parameters. A geotechnical strength reduction factor of 0.45 is recommended based on available information at this stage.
- With the exception of steel "Screw-Piles", it is expected that the settlement of deep footings proportioned as recommended above should be less than about 1% of the effective pile diameter.
- Where the founding stratum is underlain by a weaker layer, the pile toe should be located at least three pile diameters above the top of the weaker layer.
- Piles should be no closer than 2.5 pile diameters apart. If closer than this, interaction effects between piles should be taken into account and pile group settlement assessed.
- More accurate ultimate bearing capacities and settlement estimates can be obtained by undertaking static load tests on trial piles.
- These recommendations do not preclude the use of established correlations for specific pile types and may be upgraded by carrying out pile load testing.

The values presented in Table 17 are for the purposes of calculating geotechnical capacities. These values may be exceeded in site soils, particularly in sand layers during activities such as pile driving. It is recommended that pile driving equipment and piles have some additional capacity to allow piles to be driven to the design depths if higher resistance is encountered.

Softwood timber mini-piles of 125mm toe diameter driven to a design set in dense sands generally achieve working loads of about 75kN. A test pile may be carried out to assess the depth at which the design capacity may be achieved. Potential vibration effects should be considered.

As screw pile dimensions, configurations and installation procedures vary between piling contractors, pile design optimisation is usually best conducted by the piling contractor proposed to undertake the installation work. The piling contractors typically have established performance data from load testing and experience, specifically for their pile types and configurations.

These recommendations do not preclude the use of established correlations for specific pile types and may be upgraded by carrying out pile load testing.

Inspection should be carried out by a geotechnical authority during construction to confirm the conditions assumed in this report and in the design.

6.4 Retaining Wall Design Parameters

All structural retaining walls and all landscaping walls in excess of 1.0m should be designed by an experienced engineer familiar with the site conditions. All retaining walls should be designed for surcharge loading from slopes, structures and other existing/future improvements in the vicinity of the wall. Adequate subsurface and surface drainage should be provided behind all retaining walls.

Retaining walls backfilled with a free draining granular material may be designed for an active earth pressure coefficient (k_a) of 0.33 and a passive earth pressure coefficient (k_p) of 3.0 and a total density of 1.9 t/m^3.

Stiff or better clay may be designed for an active earth pressure coefficient (k_a) of 0.4 and a passive earth pressure coefficient (k_p) of 2.5 and a total density of 1.9 t/m^3.

During progressive placement of fill behind the retaining wall it may displace outwards slightly. An at rest earth pressure coefficient (k_o) should be used instead of an active earth pressure coefficient (K_a) behind the retaining structures for any walls that are relatively rigid and/or propped. A modified at rest earth pressure coefficient (k_o) of 0.5 may be used for walls that can tolerate a small amount of movement (about 0.1% to 0.3% of wall height).

Allowance should be made for in the design of retention measures to resist hydrostatic pressures due to groundwater build-up in addition to earth pressures.

Indicative parameters for generalised site materials are provided in Table 18.

The values provided in Table 18 may be adopted if applicable to the adopted design methods subject to appropriate engineering judgement. Appropriate reduction factors should be applied. Due to the potential for variability of the soil parameters, appropriately conservative parameters should be selected based on the particular application.

In applications where potential variation in the parameters is critical, further testing should be undertaken on representative materials based on trials or similar.

TABLE 18 – GEOTECHNICAL SOIL PARAMETERS

Unit	Soil / Rock Description	γ (kN/m^3)	S_u (kPa)	c' (kPa)	ϕ' (°)	E_v (MPa)	E_h (MPa)	ν
N/A	Compacted Fill - Cohesive	19	50	3	27	10	7.5	0.4
N/A	Compacted Fill - Granular	20	-	0	35	15 to 30	11 to 22	0.3
1, 2	Topsoil, Uncontrolled Fill	-	-	-	-	-	-	-
3A	Alluvium – Stiff or better Clay, Sandy Clay	19	50	5	27	10	7.5	0.4
3B	Alluvium – Medium Dense or better Sand	20	-	0	35	15 to 30	11 to 22	0.3

Note:

γ = Unit Weight

S_u = Undrained Shear Strength

c' = Effective Cohesion

ϕ' = Effective Friction Angle

E_v = Vertical Young's Modulus

E_h = Young's Modulus

ν = Poisson's Ratio

6.5 Excavation Stability and Support Requirements

Temporary earthworks may be battered at the maximum recommended batters as outlined below (or flatter), or supported by shoring.

- 1V:2H - Uncontrolled Fill materials or topsoils;
- 1V:1.5H - Non-cohesive soils (e.g. sands and gravels with minimal fines, clays with sand layers);
- 1V:1H - Cohesive soils (e.g. clays).

Possible methods of supporting deeper or steeper excavations include cantilevered piles with panels in between the piles, a retaining wall installed prior to bulk excavation or temporary shoring installed during excavation.

Temporary excavations to depths of up to 1.2m in competent compact material with sufficient cohesion, such as clay of stiff consistency or better may be battered vertically, subject to inspection during excavation by the geotechnical authority.

Temporary earthworks in any wet soils will require shallow batters or shoring to prevent slumping and/or collapse.

Visual assessment for signs of instability should be made prior to carrying out any work in the trench. If any deflection or excavation instability is observed, the excavation should be backfilled and further geotechnical advice sought.

Surcharge loads such as stockpiles of excavated soils and vehicle traffic should not be applied within a 1V:1.5H projection from the toe of any excavations or embankments, or within a 1m offset from the crest of the excavation or embankments, unless specific assessment is undertaken.

Care must be taken not to cause relaxation of ground supporting nearby structures during excavations on site.

Drainage measures should be implemented above and behind all temporary and permanent batter slopes to avoid concentrated water flows on the face or infiltration into the soil/rock profile behind the face. Surface water flows from upslope areas should be diverted away from the face.

The safe working procedures of Work Cover NSW *Excavation work code of practice*, dated January 2020 should be followed.

Longer term excavations or embankments should be supported by properly designed and constructed retaining walls or else battered at 1V:2H or flatter and protected against erosion.

Selection of batter slopes should consider access requirements for future maintenance activities, and elements at risk in the possible event of slope instability.

Shoring and retention measures may be designed based on the parameters provided in Section 6.4.

6.6 Infiltration Rates

Values of hydraulic conductivity, K, were assessed for the soil profiles at the test locations using the following equation (Porchet, from Kessler & Oosterbaan, 1974, p292):

$$K = 1.15xRxF$$

$$where\ F = \frac{Log\left(h_1 + {}^{R}/_{2}\right) - Log\left(h_2 + {}^{R}/_{2}\right)}{t_2 - t_1}$$

Where, K = hydraulic conductivity (m/s);
h_i = height of water column at a time t_i (m);
t_i = time at which measurement h_i was taken (s); and,
R = radius of borehole (m).

The results of falling head permeability testing are summarised in Table 19.

TABLE 19 – PERMEABILITY TEST RESULTS

Test Location	Depth (m)	R (m)	h_1 (m)	h_2 (m)	t_1 (s)	t_2 (s)	K (m/s)	K (m/day)	K (mm/hr)
BHI01	1.00	0.05	0.510	0.505	14400	16200	1.303×10^{-7}	0.01126	0.469

Based on the soil profiles encountered, interpretation of the results of in situ permeability testing, and previous experience in the area, it is recommended that a permeability value of 1.303×10^{-7} metres per second (~0.011 metres per day) be adopted for the Unit 3A Alluvial Clay at this location.

For design purposes it is recommended that a reduction factor be applied to this value to obtain the long-term infiltration rate for design of on-site storm water infiltration systems. This factor may be specified by the consenting authority, or in the absence thereof, a reduction factor of 0.33 (or factor of safety of 3) is recommended.

It is noted that sand (Unit 3B) was encountered at a depth of 1.50m to the termination depth of 2.50m at the borehole carried out adjacent to the infiltration test location (BH01). It is expected that infiltration rates within the sand layers would be significantly greater.

Based on past experience in the area and reference values for similar materials, it is expected that a permeability value in the range of 1.0 x 10^{-2} to 1.0 x 10^{2} metres per day would be applicable.

Based on previous experience in the region with soil similar to that encountered in BH01 below a depth of 1.50m, permeability values in the order of 2 metres per day (with reduction factor applied) are common. Testing should be carried out to obtain a site specific value if it is proposed to utilise higher permeability Unit 3B sand for infiltration systems.

6.7 Excavation Conditions and Depth to Rock

The depths of fill, topsoil, and alluvial soils, together with depths of practical refusal of the 2.7 tonne excavator's 300mm auger where encountered are summarised in Table 2.

Based upon the borehole logs, it is anticipated that Weathered Rock (Unit 4) materials are unlikely to be encountered within 2.80m of existing surface level, and that soils could be excavated by conventional excavator or equivalent at least to the depths indicated on the appended borehole logs.

No groundwater levels were observed in the boreholes during the limited time that they remained open on the day of field work. Slow groundwater inflows were observed at depths of approximately 2.40m beneath existing ground level at borehole locations BH02 to BH05, BH10, and BH12. This may be indicative of a water table depth at or near the depths of encountered inflows. Groundwater levels may change due to rainfall and other influences including regional groundwater flow, temperature, permeability, recharge areas, surface condition, and subsoil drainage.

Groundwater inflows are likely to occur if excavations proceed below the water table. These inflows are likely to cause collapse of unsupported excavations in sandy soils.

If encountered, groundwater inflows are likely to be rapid due to the relatively high permeability of the sand soils. It is recommended that further assessment is carried out to assist plans for shoring and dewatering if excavation below the water table is proposed.

Groundwater may exist at shallow depths in localised areas of the site such as within the topsoil profile, from water perched above the alluvial clay. It is possible that slow water inflow may be encountered from such layers, particularly if earthworks are carried out during or following periods of wet weather. If perched groundwater is encountered, it is generally expected to be manageable by de-watering by sump and pump methods.

Care should be taken not to disturb or destabilise existing underground services or structures.

6.8 Site Preparation

Site preparation suitable for structures, pavement support and site re-grading should consist of:

- Following any bulk excavation to proposed subgrade level, all areas of proposed structures, pavement construction or site re-grading should be stripped to remove all existing uncontrolled fill, vegetation, topsoil, root affected or other potentially deleterious materials.
- Stripping depths are expected to be variable due to variable depths of existing fill, with stripping of fill and topsoil generally expected to be in the range of about 0.20m to 1.00m based on the depths encountered within the boreholes;
- Following stripping, the exposed subgrade should be proof rolled (minimum 10 tonne static roller), to identify any wet or excessively deflecting material. Any such areas should be over excavated and backfilled with an approved select material;
- The moisture content of the subgrade materials and therefore the need for moisture conditioning or over-excavation and replacement, will be largely dependent on pre-existing and prevailing weather conditions at the time of construction;
- Protect the area after subgrade preparation to maintain moisture content as far as practicable. The placement of subbase gravel would normally provide adequate protection.
- Site preparation should include provision of drainage and erosion control as required as well as sedimentation control measures.

It should be anticipated that some moisture conditioning of the subgrade may be necessary prior to compaction and placement of fill materials.

The required time period to prepare the subgrade is likely to be dependent on the prevailing weather conditions at the time of construction.

If over wet subgrades exist at the time of construction or deleterious fill materials are encountered at subgrade level, these materials should be over-excavated and be replaced with a minimum depth of 250mm of well graded granular select material with CBR of 15% or greater. The requirement for, and extent of subgrade replacement / select filling, should be confirmed by the geotechnical authority at the time of construction.

6.9 Fill Construction Procedures

Earthworks for pavement construction or support of foundations should consist of the following measures:

- At design subgrade level for pavements or structures, the surface should be compacted for a depth of at least 1.0m to a minimum density index of 70% (AS1289 5.6.1) in granular soils. Compaction should be confirmed by penetrometer testing prior to placement of pavement materials or pouring of concrete for footings;
- Approved fill beneath structures and pavements should be compacted in layers not exceeding 300mm loose thickness;
- Approved fill for pavements should be compacted to the compaction requirements provided in Section 6.1;

- Approved fill beneath pavements should be compacted in layers not exceeding 300mm loose thickness to a minimum density ratio of 95% Standard Compaction within ±2% of OMC in cohesive soils, or to a minimum density index of 70% (AS1289 5.6.1 for granular soils;
- The top 300mm of natural subgrade below pavements or the final 300mm of road subgrade fill should be compacted to a minimum density ratio of 100% Standard Compaction within the moisture range of 60% to 90% of Optimum Moisture Content (OMC) in cohesive soils, or to a minimum density index of 80% (AS1289 5.6.1 for granular soils;
- Site fill beneath structures should be compacted to a minimum density ratio of 98% Standard Compaction within ±2% of OMC in cohesive soils, or to a minimum density index of 80% (AS1289 5.6.1 for granular soils;
- All fill should be supported by properly designed and constructed retaining walls or else battered at 1V:2H or flatter and protected against erosion;
- If fill is to be placed on slopes in excess of 1V:8H (7°), a prepared surface should be benched or stepped into the slope;
- Earthworks should be carried out in accordance with the recommendations outlined in AS3798-2007 '*Guidelines for Earthworks for Commercial and Residential Developments*'.

6.10 Suitability of Site Materials for Re-Use as Fill

The following comments are made with respect to suitability of site materials for re-use as fill:

- Unit 1A Fill-Topsoil materials are expected to be suitable for landscaping purposes only;
- Unit 1B Fill materials may be variable. Some fill material may be suitable for landscaping purposes only due to the presence of roots and organics. If fill material is not affected by roots or other deleterious material, it is generally expected to be suitable for re-use as general fill for engineering purposes, this should be confirmed during construction;
- Unit 2 Topsoil materials are expected to be suitable for landscaping purposes only;
- Unit 3 Alluvium materials are generally expected to be suitable for re-use as general fill for engineering purposes.

These materials may require some moisture conditioning. Final selection of fill materials should consider properties such as and reactivity which is typically low to moderate for site won Unit 3A Alluvial Soils.

The suitability of material for re-use should be assessed and confirmed by the geotechnical authority at the time of construction.

6.11 Special Construction Requirements and Site Drainage

Inspection should be carried out by a geotechnical authority during construction to confirm the conditions assumed in this report and in the design.

Adequate surface and subsurface drainage should be installed and connected to the stormwater disposal system.

Pavement thickness designs should allow for the provision of adequate surface and subsurface drainage of the pavement and adjacent areas to prevent moisture ingress into the pavement materials and subgrade. It is recommended that subsoil drains be installed:

- Along the high side of roads aligned across site slopes;
- Along both sides of roads aligned down slope.

It is recommended that surface and subsoil drainage be installed in line with the above advice, and in accordance with Newcastle City Council (NCC) specifications.

Care should be taken during backfilling of any depressions to reduce the risk of leaving a preferential underground drainage path which could result in softening of the surrounding area, piping erosion and/or localised seepage.

Potential effects of slope modifications on groundwater flowing from upslope should also be considered, with provision of subsurface drainage to intercept and redirect groundwater where assessed to be necessary.

7.0 Limitations

The findings presented in the report and used as the basis for recommendations presented herein were obtained using normal, industry accepted geotechnical design practices and standards. To our knowledge, they represent a reasonable interpretation of the general conditions of the site.

The extent of testing associated with this assessment is limited to discrete test locations. It should be noted that subsurface conditions between and away from the test locations may be different to those observed during the field work and used as the basis of the recommendations contained in this report.

If subsurface conditions encountered during construction differ from those given in this report, further advice should be sought without delay.

Data and opinions contained within the report may not be used in other contexts or for any other purposes without prior review and agreement by Qualtest. If this report is reproduced, it must be in full.

If you have any further questions regarding this report, please do not hesitate to contact Ben Bunting, Shannon Kelly, or the undersigned.

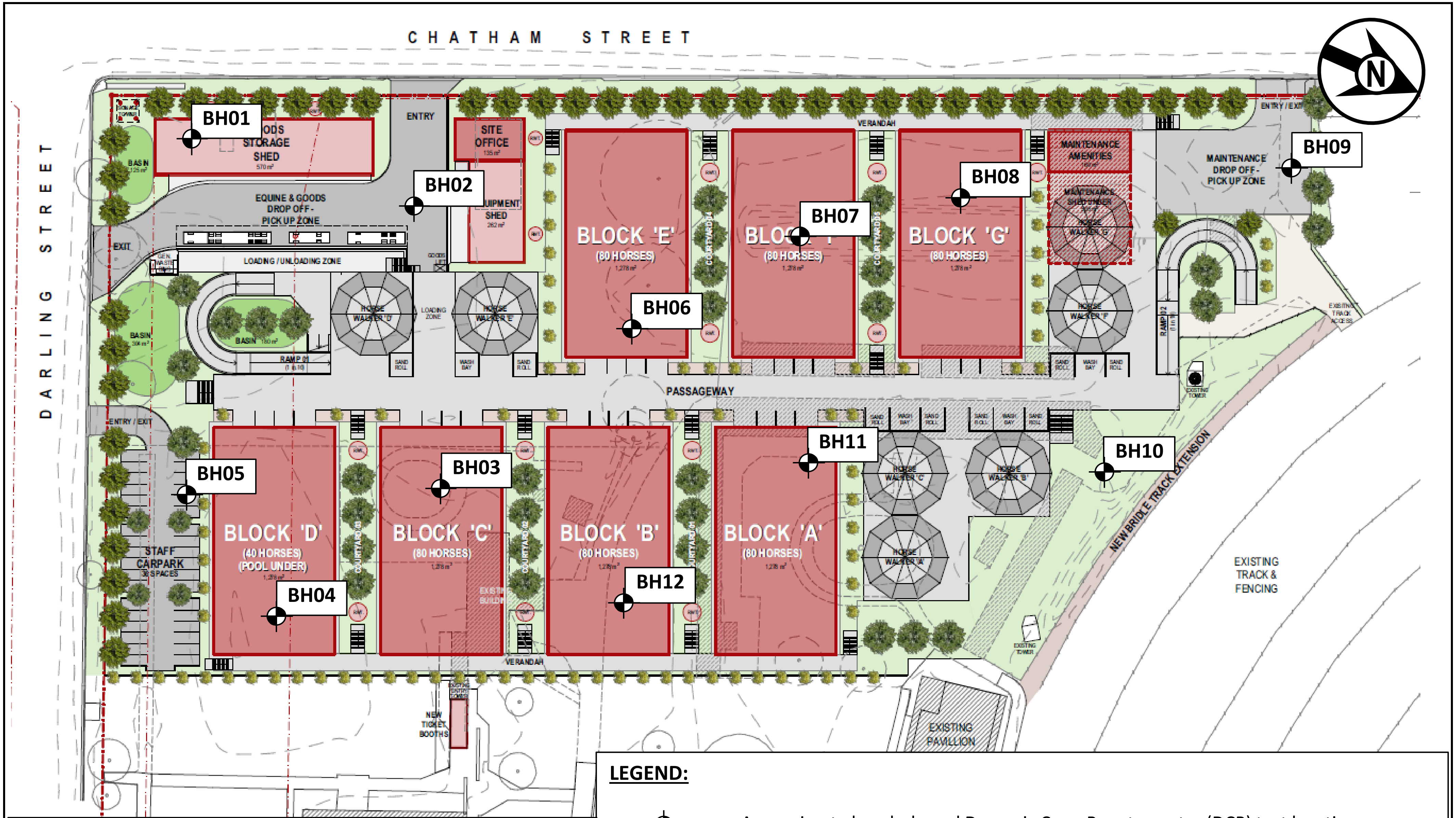
For and on behalf of Qualtest Laboratory (NSW) Pty Ltd.

Jason Lee

Jason Lee
Principal Geotechnical Engineer

FIGURE AA1:

Site Plan and Approximate Test Locations



Based on Proposed Site Plan prepared by EJE Architecture
(Dwg No. 11553-SK-A03.5.11.2020-Rev. E)

LEGEND:

⊕ Approximate borehole and Dynamic Cone Penetrometer (DCP) test location

Qualtest LABORATORY (NSW) PTY LTD

Client:	NEWCASTLE JOCKEY CLUB	Drawing No:	FIGURE AA1
Project:	PROPOSED STABLES DEVELOPMENT	Project No:	NEW20P-0194
Location:	CNR DARLING STREET & CHATHAM STREET, BROADMEADOW	Scale:	N.T.S.
Title:	SITE PLAN AND APPROXIMATE TEST LOCATIONS	Date:	12/01/2021

APPENDIX A:

Results of Field Investigations

ENGINEERING LOG - BOREHOLE

Qualtest LABORATORY (NSW) PTY LTD

CLIENT:	NEWCASTLE JOCKEY CLUB	**BOREHOLE NO:**	**BH01**
PROJECT:	PROPOSED STABLES DEVELOPMENT	**PAGE:**	1 OF 1
LOCATION:	CNR DARLING STREET & CHATHAM STREET, BROADMEADOW	**JOB NO:**	NEW20P-0194
		LOGGED BY:	BB
		DATE:	26/11/20

DRILL TYPE: 2.7 TONNE EXCAVATOR WITH AUGER
BOREHOLE DIAMETER: 300 mm
SURFACE RL:
DATUM:

METHOD	WATER	SAMPLES	RL (m)	DEPTH (m)	CLASSIFICATION SYMBOL	MATERIAL DESCRIPTION: Soil type, plasticity/particle characteristics, colour, minor components	MOISTURE CONDITION	CONSISTENCY DENSITY	Field Test: Test Type	Field Test: Result	Structure and additional observations
AD/T	Not Encountered			0.00–0.25m	SM	FILL-TOPSOIL: Gravelly Silty SAND - fine to medium grained (mostly fine grained), dark brown, fines of low plasticity, fine to coarse grained angular gravel, root affected in top 0.10m. Black.	D - M				FILL - TOPSOIL
				0.25–0.40m	SP	FILL: SAND - fine to medium grained, pale brown.	M				FILL
				0.40–0.50m	SP	FILL: Gravelly SAND - fine to coarse grained, black, fine to medium grained angular gravel.	M				
				0.50–0.70m	CH	FILL: Sandy CLAY - medium plasticity, black, fine to coarse grained sand, with some fine to medium grained angular gravel.			HP	180	
		0.80m U50 1.10m		0.70–1.50m	CH	Sandy CLAY - medium to high plasticity, dark grey with some brown, fine to medium grained sand.	M > W_P	St	HP HP	100 150	ALLUVIUM / POSSIBLE FILL
						Medium plasticity.	M ~ W_P	St - VSt	HP	220	
				1.50–2.80m	SP	SAND - fine to medium grained, grey with some orange-brown, with some fines of low plasticity. Orange-brown. Dark grey. Weakly cemented, dark brown.	M	D			ALLUVIUM
						Hole Terminated at 2.80 m Limit Of Reach					

LEGEND:

Water
- Water Level (Date and time shown)
- Water Inflow
- Water Outflow

Strata Changes
- Gradational or transitional strata
- Definitive or distict strata change

Notes, Samples and Tests

U_{50}	50mm Diameter tube sample
CBR	Bulk sample for CBR testing
E	Environmental sample (Glass jar, sealed and chilled on site)
ASS	Acid Sulfate Soil Sample (Plastic bag, air expelled, chilled)
B	Bulk Sample

Field Tests

PID	Photoionisation detector reading (ppm)
DCP(x-y)	Dynamic penetrometer test (test depth interval shown)
HP	Hand Penetrometer test (UCS kPa)

Consistency		UCS (kPa)
VS	Very Soft	<25
S	Soft	25 - 50
F	Firm	50 - 100
St	Stiff	100 - 200
VSt	Very Stiff	200 - 400
H	Hard	>400
Fb	Friable	

Moisture Condition	
D	Dry
M	Moist
W	Wet
W_p	Plastic Limit
W_L	Liquid Limit

Density		
V	Very Loose	Density Index <15%
L	Loose	Density Index 15 - 35%
MD	Medium Dense	Density Index 35 - 65%
D	Dense	Density Index 65 - 85%
VD	Very Dense	Density Index 85 - 100%

ENGINEERING LOG - BOREHOLE

Qualtest LABORATORY (NSW) PTY LTD

CLIENT:	NEWCASTLE JOCKEY CLUB	**BOREHOLE NO:**	**BH02**
PROJECT:	PROPOSED STABLES DEVELOPMENT	**PAGE:**	1 OF 1
LOCATION:	CNR DARLING STREET & CHATHAM STREET, BROADMEADOW	**JOB NO:**	NEW20P-0194
		LOGGED BY:	BB
		DATE:	26/11/20

DRILL TYPE: 2.7 TONNE EXCAVATOR WITH AUGER **SURFACE RL:**
BOREHOLE DIAMETER: 300 mm **DATUM:**

METHOD	WATER	SAMPLES	RL (m)	DEPTH (m)	CLASSIFICATION SYMBOL	MATERIAL DESCRIPTION: Soil type, plasticity/particle characteristics,colour,minor components	MOISTURE CONDITION	CONSISTENCY DENSITY	Test Type	Result	Structure and additional observations
AD/T				0.00–0.40m	SC	FILL-TOPSOIL: Clayey Gravelly SAND - fine to coarse grained, black, fine to medium grained angular gravel, fines of low plasticity, root affected in top 0.10m.	D - M				FILL - TOPSOIL
		0.40m–0.80m CBR		0.40–0.80m	CH	Sandy CLAY - medium to high plasticity, dark grey with some brown, fine to medium grained sand.	M > W_P	St	HP HP	130 150	ALLUVIUM / POSSIBLE FILL
				0.80–1.40m	CI	Sandy CLAY - medium plasticity, grey with some brown, fine to medium grained sand.	M > W_P	St - VSt	HP HP	210 200	ALLUVIUM
				1.40–1.90m	CI	Sandy CLAY / Clayey SAND - medium plasticity, grey with some brown, fine to medium grained sand.	M ~ W_P	St - VSt	HP	180	
	Very slow inflow (<1L/min) at ~2.4m			1.90–2.80m	SP	SAND - fine to medium grained, pale grey to white. Dark grey to black. Dark brown.	M - W	MD - D			
				2.80m		Hole Terminated at 2.80 m Limit Of Reach					

LEGEND:

Water
- Water Level (Date and time shown)
- Water Inflow
- Water Outflow

Strata Changes
- Gradational or transitional strata
- Definitive or distict strata change

Notes, Samples and Tests

U_{50}	50mm Diameter tube sample
CBR	Bulk sample for CBR testing
E	Environmental sample (Glass jar, sealed and chilled on site)
ASS	Acid Sulfate Soil Sample (Plastic bag, air expelled, chilled)
B	Bulk Sample

Field Tests

PID	Photoionisation detector reading (ppm)
DCP(x-y)	Dynamic penetrometer test (test depth interval shown)
HP	Hand Penetrometer test (UCS kPa)

Consistency		UCS (kPa)
VS	Very Soft	<25
S	Soft	25 - 50
F	Firm	50 - 100
St	Stiff	100 - 200
VSt	Very Stiff	200 - 400
H	Hard	>400
Fb	Friable	

Moisture Condition	
D	Dry
M	Moist
W	Wet
W_p	Plastic Limit
W_L	Liquid Limit

Density		
V	Very Loose	Density Index <15%
L	Loose	Density Index 15 - 35%
MD	Medium Dense	Density Index 35 - 65%
D	Dense	Density Index 65 - 85%
VD	Very Dense	Density Index 85 - 100%

ENGINEERING LOG - BOREHOLE

Qualtest LABORATORY (NSW) PTY LTD

CLIENT:	NEWCASTLE JOCKEY CLUB	BOREHOLE NO:	BH03
PROJECT:	PROPOSED STABLES DEVELOPMENT	PAGE:	1 OF 1
LOCATION:	CNR DARLING STREET & CHATHAM STREET, BROADMEADOW	JOB NO:	NEW20P-0194
		LOGGED BY:	BB
		DATE:	26/11/20

DRILL TYPE: 2.7 TONNE EXCAVATOR WITH AUGER **SURFACE RL:**

BOREHOLE DIAMETER: 300 mm **DATUM:**

METHOD	WATER	SAMPLES	RL (m)	DEPTH (m)	CLASSIFICATION SYMBOL	MATERIAL DESCRIPTION: Soil type, plasticity/particle characteristics, colour, minor components	MOISTURE CONDITION	CONSISTENCY DENSITY	Test Type	Result	Structure and additional observations
AD/T				0.00–0.15m	SM	FILL-TOPSOIL: Silty SAND - fine to medium grained, brown, fines of low plasticity, root affected.	D - M				FILL - TOPSOIL
				0.15–0.45m	GM	FILL: Silty Sandy GRAVEL - fine to medium grained, sub-rounded to sub-angular, pale orange-brown, fine to coarse grained sand, fines of low plasticity.	M				FILL
				0.45–0.90m	CH	Sandy CLAY - medium to high plasticity, grey with some brown, fine to medium grained sand.	M > W$_P$	VSt	HP HP	300 250	ALLUVIUM
				0.90–1.50m	CI	Sandy CLAY - medium plasticity, grey with some orange-brown, fine to medium grained sand.	M ~ W$_P$	VSt	HP HP	350 300	
				1.50–1.60m	CI	Sandy CLAY / Clayey SAND - medium plasticity, grey with some orange-brown, fine to medium grained sand.	M ~ W$_P$	MD - D			
	Very slow inflow (<1L/min) at ~2.4m			1.60–2.80m	SP	SAND - fine to medium grained, pale grey to white. Dark brown to dark grey-brown.	M - W	MD - D			
				2.80m		Hole Terminated at 2.80 m Limit Of Reach					

LEGEND:

Water
- Water Level (Date and time shown)
- Water Inflow
- Water Outflow

Strata Changes
- Gradational or transitional strata
- Definitive or distict strata change

Notes, Samples and Tests

U_{50}	50mm Diameter tube sample
CBR	Bulk sample for CBR testing
E	Environmental sample (Glass jar, sealed and chilled on site)
ASS	Acid Sulfate Soil Sample (Plastic bag, air expelled, chilled)
B	Bulk Sample

Field Tests

PID	Photoionisation detector reading (ppm)
DCP(x-y)	Dynamic penetrometer test (test depth interval shown)
HP	Hand Penetrometer test (UCS kPa)

Consistency		UCS (kPa)
VS	Very Soft	<25
S	Soft	25 - 50
F	Firm	50 - 100
St	Stiff	100 - 200
VSt	Very Stiff	200 - 400
H	Hard	>400
Fb	Friable	

Moisture Condition	
D	Dry
M	Moist
W	Wet
W_p	Plastic Limit
W_L	Liquid Limit

Density		
V	Very Loose	Density Index <15%
L	Loose	Density Index 15 - 35%
MD	Medium Dense	Density Index 35 - 65%
D	Dense	Density Index 65 - 85%
VD	Very Dense	Density Index 85 - 100%

ENGINEERING LOG - BOREHOLE

CLIENT:	NEWCASTLE JOCKEY CLUB	**BOREHOLE NO:**	**BH04**
PROJECT:	PROPOSED STABLES DEVELOPMENT	**PAGE:**	1 OF 1
LOCATION:	CNR DARLING STREET & CHATHAM STREET, BROADMEADOW	**JOB NO:**	NEW20P-0194
		LOGGED BY:	BB
		DATE:	26/11/20

DRILL TYPE: 2.7 TONNE EXCAVATOR WITH AUGER — **SURFACE RL:**

BOREHOLE DIAMETER: 300 mm — **DATUM:**

Drilling and Sampling | Material description and profile information | Field Test

METHOD	WATER	SAMPLES	RL (m)	DEPTH (m)	CLASSIFICATION SYMBOL	MATERIAL DESCRIPTION: Soil type, plasticity/particle characteristics,colour,minor components	MOISTURE CONDITION	CONSISTENCY DENSITY	Test Type	Result	Structure and additional observations
AD/T	Very slow inflow (<1L/min)	0.20m			SM	TOPSOIL: Silty SAND - fine to medium grained, brown, fines of low plasticity, root affected.	D - M				TOPSOIL
		CBR 0.50m; D 0.60m; 0.70m		0.5	CH	0.20m Sandy CLAY - medium to high plasticity, dark grey, fine to medium grained sand.	M > W_p	St	HP HP	160 180	ALLUVIUM
					CI	0.70m Sandy CLAY - medium plasticity, pale grey with some orange-brown, fine to medium grained sand.	M ~ W_p	VSt	HP	230	
		1.00m D 1.10m		1.0	SC	0.90m Clayey SAND / Sandy CLAY - medium plasticity, pale grey with some orange-brown, fine to medium grained sand.	M	Fb			
				1.5 2.0 2.5	SP	1.40m SAND - fine to medium grained, grey, with some fines of low plasticity. Pale grey to white. Dark grey-brown to dark brown. Dark brown.	M M - W	MD - D			
						2.80m Hole Terminated at 2.80 m Limit Of Reach					

LEGEND:

Water
- Water Level (Date and time shown)
- Water Inflow
- Water Outflow

Strata Changes
- Gradational or transitional strata
- Definitive or distict strata change

Notes, Samples and Tests

U_{50}	50mm Diameter tube sample
CBR	Bulk sample for CBR testing
E	Environmental sample (Glass jar, sealed and chilled on site)
ASS	Acid Sulfate Soil Sample (Plastic bag, air expelled, chilled)
B	Bulk Sample

Field Tests

PID	Photoionisation detector reading (ppm)
DCP(x-y)	Dynamic penetrometer test (test depth interval shown)
HP	Hand Penetrometer test (UCS kPa)

Consistency		**UCS (kPa)**
VS	Very Soft	<25
S	Soft	25 - 50
F	Firm	50 - 100
St	Stiff	100 - 200
VSt	Very Stiff	200 - 400
H	Hard	>400
Fb	Friable	

Moisture Condition	
D	Dry
M	Moist
W	Wet
W_p	Plastic Limit
W_L	Liquid Limit

Density		
V	Very Loose	Density Index <15%
L	Loose	Density Index 15 - 35%
MD	Medium Dense	Density Index 35 - 65%
D	Dense	Density Index 65 - 85%
VD	Very Dense	Density Index 85 - 100%

ENGINEERING LOG - BOREHOLE

Qualtest LABORATORY (NSW) PTY LTD

CLIENT:	NEWCASTLE JOCKEY CLUB	**BOREHOLE NO:**	**BH05**
PROJECT:	PROPOSED STABLES DEVELOPMENT	**PAGE:**	1 OF 1
LOCATION:	CNR DARLING STREET & CHATHAM STREET, BROADMEADOW	**JOB NO:**	NEW20P-0194
		LOGGED BY:	BB
		DATE:	26/11/20

DRILL TYPE: 2.7 TONNE EXCAVATOR WITH AUGER **SURFACE RL:**

BOREHOLE DIAMETER: 300 mm **DATUM:**

METHOD	WATER	SAMPLES	RL (m)	DEPTH (m)	CLASSIFICATION SYMBOL	MATERIAL DESCRIPTION: Soil type, plasticity/particle characteristics,colour,minor components	MOISTURE CONDITION	CONSISTENCY DENSITY	Test Type	Result	Structure and additional observations
AD/T					GM	FILL: Silty Sandy GRAVEL - fine to medium grained, angular to sub-angular, black, fine to coarse grained sand, fines of low plasticity.	D - M				FILL
				0.20m	SC	Clayey SAND - fine to medium grained, grey with some brown, fines of low plasticity.	M				ALLUVIUM
		0.40m U50 0.60m; 0.80m		0.40m, 0.5	CI	Sandy CLAY - medium plasticity, grey with some brown to orange-brown, fine to medium grained sand.	M ~ w_P	St	HP	150	
		0.90m D 0.90m		0.80m	SC	Clayey SAND - fine to medium grained, grey and pale grey, fines of low to medium plasticity.					
		D 1.00m; 1.50m D 1.60m		0.90m, 1.0, 1.5	SP	SAND - fine to medium grained, pale grey to white. Grey and orange-brown.	M	MD - D			
	Very slow inflow (<1L/min)	2.00m D 2.10m		2.0, 2.5, 2.80m	SP	Dark brown.	M - W				
						Hole Terminated at 2.80 m Limit Of Reach					

LEGEND:

Water
- Water Level (Date and time shown)
- Water Inflow
- Water Outflow

Strata Changes
- Gradational or transitional strata
- Definitive or distict strata change

Notes, Samples and Tests

U_{50}	50mm Diameter tube sample
CBR	Bulk sample for CBR testing
E	Environmental sample (Glass jar, sealed and chilled on site)
ASS	Acid Sulfate Soil Sample (Plastic bag, air expelled, chilled)
B	Bulk Sample

Field Tests

PID	Photoionisation detector reading (ppm)
DCP(x-y)	Dynamic penetrometer test (test depth interval shown)
HP	Hand Penetrometer test (UCS kPa)

Consistency		UCS (kPa)
VS	Very Soft	<25
S	Soft	25 - 50
F	Firm	50 - 100
St	Stiff	100 - 200
VSt	Very Stiff	200 - 400
H	Hard	>400
Fb	Friable	

Moisture Condition	
D	Dry
M	Moist
W	Wet
W_p	Plastic Limit
W_L	Liquid Limit

Density		
V	Very Loose	Density Index <15%
L	Loose	Density Index 15 - 35%
MD	Medium Dense	Density Index 35 - 65%
D	Dense	Density Index 65 - 85%
VD	Very Dense	Density Index 85 - 100%

ENGINEERING LOG - BOREHOLE

Qualtest LABORATORY (NSW) PTY LTD

CLIENT:	NEWCASTLE JOCKEY CLUB	BOREHOLE NO:	BH06
PROJECT:	PROPOSED STABLES DEVELOPMENT	PAGE:	1 OF 1
LOCATION:	CNR DARLING STREET & CHATHAM STREET, BROADMEADOW	JOB NO:	NEW20P-0194
		LOGGED BY:	BB
		DATE:	26/11/20

DRILL TYPE: 2.7 TONNE EXCAVATOR WITH AUGER **SURFACE RL:**
BOREHOLE DIAMETER: 300 mm **DATUM:**

METHOD	WATER	SAMPLES	RL (m)	DEPTH (m)	CLASSIFICATION SYMBOL	MATERIAL DESCRIPTION: Soil type, plasticity/particle characteristics,colour,minor components	MOISTURE CONDITION	CONSISTENCY DENSITY	Test Type	Result	Structure and additional observations
AD/T	Not Encountered			0.05m	GP	FILL: Sandy GRAVEL - fine grained, grey, fine to coarse grained sand.	D - M				FILL
				0.15m	GM	FILL: Silty Sandy GRAVEL - fine to medium grained sub-angular to sub-rounded, pale orange-brown, fine to coarse grained sand, fines of low plasticity.	D - M				
				0.60m	GM	FILL: Silty Sandy GRAVEL - fine to medium grained angular, black, fine to coarse grained sand, fines of low plasticity, with some coal chitter and lightweight slag / ash.	M				
		0.60m D 0.70m		1.00m	CH	Sandy CLAY - medium to high plasticity, grey to dark grey with some orange-brown, fine to medium grained sand.	M > Wp	VSt	HP	220	ALLUVIUM
		1.00m D 1.10m		1.30m	CI	Sandy CLAY - medium plasticity, pale grey and orange-brown, fine to medium grained sand.	M ~ Wp	VSt	HP	300	
		1.30m D 1.40m		1.70m	CI	Clayey SAND / Sandy CLAY - medium plasticity, pale grey to orange-brown, fine to medium grained sand.	M				
				2.80m	SP	SAND - fine to medium grained, pale grey. Pale grey to white. Dark brown.	M	MD - D			

Hole Terminated at 2.80 m
Limit Of Reach

LEGEND:

Water
- Water Level (Date and time shown)
- Water Inflow
- Water Outflow

Strata Changes
- Gradational or transitional strata
- Definitive or distict strata change

Notes, Samples and Tests

U_{50}	50mm Diameter tube sample
CBR	Bulk sample for CBR testing
E	Environmental sample (Glass jar, sealed and chilled on site)
ASS	Acid Sulfate Soil Sample (Plastic bag, air expelled, chilled)
B	Bulk Sample

Field Tests

PID	Photoionisation detector reading (ppm)
DCP(x-y)	Dynamic penetrometer test (test depth interval shown)
HP	Hand Penetrometer test (UCS kPa)

Consistency		UCS (kPa)
VS	Very Soft	<25
S	Soft	25 - 50
F	Firm	50 - 100
St	Stiff	100 - 200
VSt	Very Stiff	200 - 400
H	Hard	>400
Fb	Friable	

Moisture Condition	
D	Dry
M	Moist
W	Wet
W_p	Plastic Limit
W_L	Liquid Limit

Density		
V	Very Loose	Density Index <15%
L	Loose	Density Index 15 - 35%
MD	Medium Dense	Density Index 35 - 65%
D	Dense	Density Index 65 - 85%
VD	Very Dense	Density Index 85 - 100%

ENGINEERING LOG - BOREHOLE

CLIENT:	NEWCASTLE JOCKEY CLUB	**BOREHOLE NO:**	**BH07**
PROJECT:	PROPOSED STABLES DEVELOPMENT	**PAGE:**	1 OF 1
LOCATION:	CNR DARLING STREET & CHATHAM STREET, BROADMEADOW	**JOB NO:**	NEW20P-0194
		LOGGED BY:	BB
		DATE:	26/11/20

Qualtest LABORATORY (NSW) PTY LTD

DRILL TYPE: 2.7 TONNE EXCAVATOR WITH AUGER **SURFACE RL:**

BOREHOLE DIAMETER: 300 mm **DATUM:**

METHOD	WATER	SAMPLES	RL (m)	DEPTH (m)	CLASSIFICATION SYMBOL	MATERIAL DESCRIPTION: Soil type, plasticity/particle characteristics,colour,minor components	MOISTURE CONDITION	CONSISTENCY DENSITY	Test Type	Result	Structure and additional observations
AD/T	Not Encountered			0.00 – 0.20m	SM	FILL-TOPSOIL: Silty SAND - fine to medium grained, dark brown, fines of low plasticity, with some fine to coarse grained rounded to sub-angular gravel, root affected.	D - M				FILL - TOPSOIL
				0.20 – 0.60m	SM	FILL: Silty SAND - fine to coarse grained (mostly fine to medium grained), black to dark grey, fines of low plasticity.	M				FILL
				0.60 – 1.00m	SP	Gravelly SAND - fine to coarse grained, black, fine to medium grained (mostly fine grained) angular gravel, with some coal chitter.	M				
				1.00 – 1.30m	CH	Sandy CLAY - medium to high plasticity, dark grey, fine to medium grained sand.	M > W_P	St	HP	130	ALLUVIUM
				1.30 – 2.10m	CI	Sandy CLAY - medium plasticity, grey with some orange-brown, fine to medium grained sand.	M > W_P / M ~ W_P	St / VSt	HP HP	150 300	
				2.10 – 2.50m	CI	Sandy CLAY / Clayey SAND - medium plasticity, grey with some orange-brown, fine to medium grained sand.	M ~ W_P	VSt			
				2.50 – 2.80m	SP	SAND - fine to medium grained, pale grey to white with some pale orange-brown.	M - W				

Hole Terminated at 2.80 m
Limit Of Reach

LEGEND:

Water
- Water Level (Date and time shown)
- Water Inflow
- Water Outflow

Strata Changes
- Gradational or transitional strata
- Definitive or distict strata change

Notes, Samples and Tests

U_{50}	50mm Diameter tube sample
CBR	Bulk sample for CBR testing
E	Environmental sample (Glass jar, sealed and chilled on site)
ASS	Acid Sulfate Soil Sample (Plastic bag, air expelled, chilled)
B	Bulk Sample

Field Tests

PID	Photoionisation detector reading (ppm)
DCP(x-y)	Dynamic penetrometer test (test depth interval shown)
HP	Hand Penetrometer test (UCS kPa)

Consistency		UCS (kPa)
VS	Very Soft	<25
S	Soft	25 - 50
F	Firm	50 - 100
St	Stiff	100 - 200
VSt	Very Stiff	200 - 400
H	Hard	>400
Fb	Friable	

Moisture Condition	
D	Dry
M	Moist
W	Wet
W_p	Plastic Limit
W_L	Liquid Limit

Density		
V	Very Loose	Density Index <15%
L	Loose	Density Index 15 - 35%
MD	Medium Dense	Density Index 35 - 65%
D	Dense	Density Index 65 - 85%
VD	Very Dense	Density Index 85 - 100%

ENGINEERING LOG - BOREHOLE

Qualtest LABORATORY (NSW) PTY LTD

CLIENT:	NEWCASTLE JOCKEY CLUB	**BOREHOLE NO:**	**BH08**
PROJECT:	PROPOSED STABLES DEVELOPMENT	**PAGE:**	1 OF 1
LOCATION:	CNR DARLING STREET & CHATHAM STREET, BROADMEADOW	**JOB NO:**	NEW20P-0194
		LOGGED BY:	BB
		DATE:	26/11/20

DRILL TYPE: 2.7 TONNE EXCAVATOR WITH AUGER **SURFACE RL:**

BOREHOLE DIAMETER: 300 mm **DATUM:**

METHOD	WATER	SAMPLES	RL (m)	DEPTH (m)	CLASSIFICATION SYMBOL	MATERIAL DESCRIPTION: Soil type, plasticity/particle characteristics,colour,minor components	MOISTURE CONDITION	CONSISTENCY DENSITY	Test Type	Result	Structure and additional observations
	Not Encountered			0.03m		ASPHALT					ASPHALT
				0.03m–0.35m	GM	FILL: Silty Sandy GRAVEL - fine to coarse grained (mostly fine to medium grained), rounded to sub-angular, pale brown, fine to coarse grained sand, fines of low plasticity.	M	D - VD			FILL - PAVEMENT
				0.35m–0.70m	GP	FILL: Sandy GRAVEL - fine grained angular, black, fine to coarse grained sand.	M - W	L - MD			FILL
		0.70m D 0.80m		0.70m–1.00m	CH	Sandy CLAY - medium to high plasticity, grey, fine to medium grained sand.	$M > w_P$	St	HP	110	ALLUVIUM
		1.00m D 1.10m		1.00m–2.00m	CI	Sandy CLAY - medium plasticity, grey with some pale orange-brown, fine to medium grained sand.	$M \sim w_P$	VSt	HP	250	
		2.00m D 2.10m		2.00m–2.20m	SC	Clayey SAND - fine to medium grained, grey and pale orange-brown, fines of low to medium plasticity.	M				
		2.30m D 2.40m		2.20m–2.80m	SP	SAND - fine to medium grained, pale grey to white. Grey-brown.	M				
				2.80m		Hole Terminated at 2.80 m Limit Of Reach					

LEGEND:

Water
- Water Level (Date and time shown)
- Water Inflow
- Water Outflow

Strata Changes
- Gradational or transitional strata
- Definitive or distict strata change

Notes, Samples and Tests

U_{50}	50mm Diameter tube sample
CBR	Bulk sample for CBR testing
E	Environmental sample (Glass jar, sealed and chilled on site)
ASS	Acid Sulfate Soil Sample (Plastic bag, air expelled, chilled)
B	Bulk Sample

Field Tests

PID	Photoionisation detector reading (ppm)
DCP(x-y)	Dynamic penetrometer test (test depth interval shown)
HP	Hand Penetrometer test (UCS kPa)

Consistency		UCS (kPa)
VS	Very Soft	<25
S	Soft	25 - 50
F	Firm	50 - 100
St	Stiff	100 - 200
VSt	Very Stiff	200 - 400
H	Hard	>400
Fb	Friable	

Moisture Condition	
D	Dry
M	Moist
W	Wet
W_p	Plastic Limit
W_L	Liquid Limit

Density		
V	Very Loose	Density Index <15%
L	Loose	Density Index 15 - 35%
MD	Medium Dense	Density Index 35 - 65%
D	Dense	Density Index 65 - 85%
VD	Very Dense	Density Index 85 - 100%

ENGINEERING LOG - BOREHOLE

Qualtest LABORATORY (NSW) PTY LTD

CLIENT:	NEWCASTLE JOCKEY CLUB	BOREHOLE NO:	BH09
PROJECT:	PROPOSED STABLES DEVELOPMENT	PAGE:	1 OF 1
LOCATION:	CNR DARLING STREET & CHATHAM STREET, BROADMEADOW	JOB NO:	NEW20P-0194
		LOGGED BY:	BB
		DATE:	26/11/20

DRILL TYPE: 2.7 TONNE EXCAVATOR WITH AUGER **SURFACE RL:**
BOREHOLE DIAMETER: 300 mm **DATUM:**

METHOD	WATER	SAMPLES	RL (m)	DEPTH (m)	CLASSIFICATION SYMBOL	MATERIAL DESCRIPTION: Soil type, plasticity/particle characteristics, colour, minor components	MOISTURE CONDITION	CONSISTENCY DENSITY	Test Type	Result	Structure and additional observations
AD/T	Not Encountered			0.00–0.50m	SM	FILL-TOPSOIL: Silty SAND - fine to medium grained, dark brown, fines of low plasticity, root affected.	M				FILL - TOPSOIL
				0.50–0.80m	SP	FILL: Gravelly SAND - fine to coarse grained, black, fine to medium grained angular gravel.	M - W				FILL
		0.90m–1.20m CBR		0.80–2.00m	CH	Sandy CLAY - medium to high plasticity, dark grey, fine to medium grained sand.	$M > W_P$	F	HP	70	ALLUVIUM
						Pale grey with some pale orange-brown.		St	HP	120	
									HP	150	
				2.00–2.30m	SC	Clayey SAND - fine to medium grained, grey with some pale orange-brown, fines of low plasticity.	M				
				2.30–2.80m	SP	SAND - fine to medium grained, dark grey. Pale grey with some pale orange-brown.					
				2.80m		Hole Terminated at 2.80 m Limit Of Reach					

LEGEND:

Water
- Water Level (Date and time shown)
- Water Inflow
- Water Outflow

Strata Changes
- Gradational or transitional strata
- Definitive or distict strata change

Notes, Samples and Tests

U_{50}	50mm Diameter tube sample
CBR	Bulk sample for CBR testing
E	Environmental sample (Glass jar, sealed and chilled on site)
ASS	Acid Sulfate Soil Sample (Plastic bag, air expelled, chilled)
B	Bulk Sample

Field Tests

PID	Photoionisation detector reading (ppm)
DCP(x-y)	Dynamic penetrometer test (test depth interval shown)
HP	Hand Penetrometer test (UCS kPa)

Consistency		UCS (kPa)
VS	Very Soft	<25
S	Soft	25 - 50
F	Firm	50 - 100
St	Stiff	100 - 200
VSt	Very Stiff	200 - 400
H	Hard	>400
Fb	Friable	

Moisture Condition	
D	Dry
M	Moist
W	Wet
W_p	Plastic Limit
W_L	Liquid Limit

Density		
V	Very Loose	Density Index <15%
L	Loose	Density Index 15 - 35%
MD	Medium Dense	Density Index 35 - 65%
D	Dense	Density Index 65 - 85%
VD	Very Dense	Density Index 85 - 100%

ENGINEERING LOG - BOREHOLE

Qualtest LABORATORY (NSW) PTY LTD

CLIENT:	NEWCASTLE JOCKEY CLUB	BOREHOLE NO:	**BH10**
PROJECT:	PROPOSED STABLES DEVELOPMENT	PAGE:	1 OF 1
LOCATION:	CNR DARLING STREET & CHATHAM STREET, BROADMEADOW	JOB NO:	NEW20P-0194
		LOGGED BY:	BB
		DATE:	26/11/20

DRILL TYPE: 2.7 TONNE EXCAVATOR WITH AUGER **SURFACE RL:**
BOREHOLE DIAMETER: 300 mm **DATUM:**

METHOD	WATER	SAMPLES	RL (m)	DEPTH (m)	GRAPHIC LOG	CLASSIFICATION SYMBOL	MATERIAL DESCRIPTION: Soil type, plasticity/particle characteristics,colour,minor components	MOISTURE CONDITION	CONSISTENCY DENSITY	Test Type	Result	Structure and additional observations
AD/T				0.00 - 0.40m		SM	FILL-TOPSOIL: Silty SAND - fine to medium grained, dark grey-brown, fines of low plasticity, root affected.	M				FILL - TOPSOIL
				0.40 - 0.80m		SP	FILL: Gravelly SAND - fine to coarse grained, black to dark grey, fine grained angular gravel.	M				FILL
		0.80m - 1.10m U50		0.80 - 1.30m		CH	Sandy CLAY - medium to high plasticity, grey with some brown, fine to medium grained sand.	M > Wp	F	HP	70	ALLUVIUM
		1.20m - 1.30m D							St	HP	130	
				1.30 - 2.00m		CI	Sandy CLAY - medium plasticity, pale grey and pale orange-brown, fine to medium grained sand.	M > Wp	VSt	HP	210	
				2.00 - 2.20m		SC	Clayey SAND - fine to medium grained, grey and pale orange-brown, fines of medium plasticity.	M				
	Very slow inflow (<1L/min)			2.20 - 2.80m		SP	SAND - fine to medium grained, pale grey to white. Grey to grey-brown.	M - W				
							Hole Terminated at 2.80 m Limit Of Reach					

LEGEND:

Water
- Water Level (Date and time shown)
- Water Inflow
- Water Outflow

Strata Changes
- — — Gradational or transitional strata
- —— Definitive or distict strata change

Notes, Samples and Tests

U_{50}	50mm Diameter tube sample
CBR	Bulk sample for CBR testing
E	Environmental sample (Glass jar, sealed and chilled on site)
ASS	Acid Sulfate Soil Sample (Plastic bag, air expelled, chilled)
B	Bulk Sample

Field Tests

PID	Photoionisation detector reading (ppm)
DCP(x-y)	Dynamic penetrometer test (test depth interval shown)
HP	Hand Penetrometer test (UCS kPa)

Consistency		UCS (kPa)
VS	Very Soft	<25
S	Soft	25 - 50
F	Firm	50 - 100
St	Stiff	100 - 200
VSt	Very Stiff	200 - 400
H	Hard	>400
Fb	Friable	

Moisture Condition	
D	Dry
M	Moist
W	Wet
W_p	Plastic Limit
W_L	Liquid Limit

Density		
V	Very Loose	Density Index <15%
L	Loose	Density Index 15 - 35%
MD	Medium Dense	Density Index 35 - 65%
D	Dense	Density Index 65 - 85%
VD	Very Dense	Density Index 85 - 100%

ENGINEERING LOG - BOREHOLE

Qualtest LABORATORY (NSW) PTY LTD

BOREHOLE NO: BH11
PAGE: 1 OF 1
CLIENT: NEWCASTLE JOCKEY CLUB
JOB NO: NEW20P-0194
PROJECT: PROPOSED STABLES DEVELOPMENT
LOGGED BY: BB
LOCATION: CNR DARLING STREET & CHATHAM STREET, BROADMEADOW
DATE: 26/11/20

DRILL TYPE: 2.7 TONNE EXCAVATOR WITH AUGER
SURFACE RL:
BOREHOLE DIAMETER: 300 mm
DATUM:

METHOD	WATER	SAMPLES	RL (m)	DEPTH (m)	CLASSIFICATION SYMBOL	MATERIAL DESCRIPTION: Soil type, plasticity/particle characteristics, colour, minor components	MOISTURE CONDITION	CONSISTENCY DENSITY	Test Type	Result	Structure and additional observations
AD/T	Not Encountered			0.00–0.20m	CL	FILL-TOPSOIL: Sandy CLAY - low plasticity, dark brown, fine to medium grained sand, root affected.	$M \sim W_P$				FILL - TOPSOIL
				0.20–0.50m	SP	FILL: Gravelly SAND - fine to coarse grained, black, fine grained angular gravel.	M				FILL
				0.50–1.00m	CH	Sandy CLAY - medium to high plasticity, grey to dark grey, fine to medium grained sand.	$M > W_P$	F - St	HP HP	80 100	ALLUVIUM
				1.00–1.70m	CI	Sandy CLAY - medium plasticity, pale grey with some pale orange-brown, fine to medium grained sand.	$M \sim W_P$	VSt	HP HP	200 300	
				1.70–2.00m	SC	Clayey SAND - fine to medium grained, grey and pale orange-brown, fines of low plasticity.	M				
				2.00–2.80m	SP	SAND - fine to medium grained, pale grey to white. Dark brown to dark grey-brown. Dark grey.	M - W				
				2.80m		Hole Terminated at 2.80 m Limit Of Reach					

LEGEND:

Water
- Water Level (Date and time shown)
- Water Inflow
- Water Outflow

Strata Changes
- Gradational or transitional strata
- Definitive or distict strata change

Notes, Samples and Tests

U_{50}	50mm Diameter tube sample
CBR	Bulk sample for CBR testing
E	Environmental sample (Glass jar, sealed and chilled on site)
ASS	Acid Sulfate Soil Sample (Plastic bag, air expelled, chilled)
B	Bulk Sample

Field Tests

PID	Photoionisation detector reading (ppm)
DCP(x-y)	Dynamic penetrometer test (test depth interval shown)
HP	Hand Penetrometer test (UCS kPa)

Consistency		UCS (kPa)
VS	Very Soft	<25
S	Soft	25 - 50
F	Firm	50 - 100
St	Stiff	100 - 200
VSt	Very Stiff	200 - 400
H	Hard	>400
Fb	Friable	

Moisture Condition	
D	Dry
M	Moist
W	Wet
W_p	Plastic Limit
W_L	Liquid Limit

Density		
V	Very Loose	Density Index <15%
L	Loose	Density Index 15 - 35%
MD	Medium Dense	Density Index 35 - 65%
D	Dense	Density Index 65 - 85%
VD	Very Dense	Density Index 85 - 100%

ENGINEERING LOG - BOREHOLE

Qualtest LABORATORY (NSW) PTY LTD

CLIENT:	NEWCASTLE JOCKEY CLUB	**BOREHOLE NO:**	**BH12**
PROJECT:	PROPOSED STABLES DEVELOPMENT	**PAGE:**	1 OF 1
LOCATION:	CNR DARLING STREET & CHATHAM STREET, BROADMEADOW	**JOB NO:**	NEW20P-0194
		LOGGED BY:	BB
		DATE:	26/11/20

DRILL TYPE: 2.7 TONNE EXCAVATOR WITH AUGER **SURFACE RL:**
BOREHOLE DIAMETER: 300 mm **DATUM:**

METHOD	WATER	SAMPLES	RL (m)	DEPTH (m)	CLASSIFICATION SYMBOL	MATERIAL DESCRIPTION: Soil type, plasticity/particle characteristics, colour, minor components	MOISTURE CONDITION	CONSISTENCY DENSITY	Test Type	Result	Structure and additional observations
				0.00–0.30m	SM	FILL-TOPSOIL: Silty SAND - fine to coarse grained (mostly fine to medium grained), dark brown, fines of low plasticity, root affected.	M				FILL - TOPSOIL
				0.30–0.80m	SM	FILL: Gravelly Silty SAND - fine to coarse grained, grey-brown, fine grained rounded to sub-angular gravel, fines of low plasticity.	M				FILL
				0.80–0.90m	CL	Sandy CLAY - low plasticity, dark grey-brown, fine to medium grained sand.	M ~ W_p	F	HP	90	ALLUVIUM / POSSIBLE FILL
		U50 0.90m–1.10m		0.90–1.80m	CH	Sandy CLAY - medium to high plasticity, grey, fine to medium grained sand.	M > W_p	St	HP	180	ALLUVIUM
							M ~ W_p	VSt	HP	250	
									HP	270	
				1.80–2.00m	SP	SAND - fine to medium grained, grey and pale orange-brown, with some fines of low plasticity.	M				
	Very slow inflow (<1L/min)			2.00–2.80m	SP	SAND - fine to medium grained, grey-brown. Pale grey to white with some pale orange-brown. Pale brown to brown.	M - W				
				2.80m		Hole Terminated at 2.80 m Limit Of Reach					

LEGEND:

Water
- Water Level (Date and time shown)
- Water Inflow
- Water Outflow

Strata Changes
- Gradational or transitional strata
- Definitive or distict strata change

Notes, Samples and Tests

U_{50}	50mm Diameter tube sample
CBR	Bulk sample for CBR testing
E	Environmental sample (Glass jar, sealed and chilled on site)
ASS	Acid Sulfate Soil Sample (Plastic bag, air expelled, chilled)
B	Bulk Sample

Field Tests

PID	Photoionisation detector reading (ppm)
DCP(x-y)	Dynamic penetrometer test (test depth interval shown)
HP	Hand Penetrometer test (UCS kPa)

Consistency		**UCS (kPa)**
VS	Very Soft	<25
S	Soft	25 - 50
F	Firm	50 - 100
St	Stiff	100 - 200
VSt	Very Stiff	200 - 400
H	Hard	>400
Fb	Friable	

Moisture Condition	
D	Dry
M	Moist
W	Wet
W_p	Plastic Limit
W_L	Liquid Limit

Density		
V	Very Loose	Density Index <15%
L	Loose	Density Index 15 - 35%
MD	Medium Dense	Density Index 35 - 65%
D	Dense	Density Index 65 - 85%
VD	Very Dense	Density Index 85 - 100%

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DYNAMIC CONE PENETROMETER - TEST REPORT

Client:	**NEWCASTLE JOCKEY CLUB**	**Project Number:**	*NEW20P-0194*
Principal:		**Sheet No:**	*1 of 2*
Project:	**PROPOSED STABLES DEVELOPMENT**	**Test Date:**	26/11/2020
Location:	**CNR DARLING STREET & CHATHAM STREET, BROADMEADOW**	**Tested By:**	BB

Test Method: **AS1289 6.3.2** ☑ **Cone Tip**

Drop Height: **510 ± 5mm** ☐ **Blunt Tip**

Depth Below Surface (mm)	BH01	BH02	BH03	BH04	BH05	BH06	BH07	BH08	Test Location / Comments
150	5	6	11	7	12	16	8	-	DCP locations as per attached Figure AA1.
300	7	10	20	8	10	15	14	-	
450	10	5	-	5	4	9	20	-	
600	2	2	2	4	3	9	14	2	At BH03 location, DCP test attempted from surface, encountered high blow counts and discontinued at 0.30m. Test resumed at 0.45m deep after borehole passed this depth.
750	1	2	2	3	4	2	8	2	
900	6	3	2	8	10	3	8	4	
1050	9	6	2	9	13	5	10	5	
1200	14	7	4	11	20	10	3	8	
1350	19	14	6	10	25	16	4	9	
1500	19	17	9	11		18	6	14	
1650	19	20	12	15		20	8	17	
1800									
1950									
2100									
2250									
2400									
2550									
2700									
2850									
3000									
3150									
3300									
3450									
3600									
3750									
3900									
4050									
4200									
4350									
4500									

Comments: Readings recorded in blows per 150mm increments.

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DYNAMIC CONE PENETROMETER - TEST REPORT

Client:	**NEWCASTLE JOCKEY CLUB**	**Project Number:**	*NEW20P-0194*
Principal:		**Sheet No:**	*2 of 2*
Project:	**PROPOSED STABLES DEVELOPMENT**	**Test Date:**	26/11/2020
Location:	**CNR DARLING STREET & CHATHAM STREET, BROADMEADOW**	**Tested By:**	BB

Test Method: AS1289 6.3.2 ☑ Cone Tip
Drop Height: 510 ± 5mm ☐ Blunt Tip

Depth Below Surface (mm)	Test Number								Test Location / Comments
	BH09	BH10	BH11	BH12					
150	2	3	2	7					DCP locations as per attached Figure AA1.
300	2	15	7	7					
450	3	13	6	3					
600	8	14	2	3					
750	13	11	2	3					
900	4	2	3	4					
1050	2	5	4	3					
1200	3	6	6	4					
1350	6	10	14	3					
1500	8	13	16	5					
1650	10	14	16	7					
1800									
1950									
2100									
2250									
2400									
2550									
2700									
2850									
3000									
3150									
3300									
3450									
3600									
3750									
3900									
4050									
4200									
4350									
4500									

Comments: Readings recorded in blows per 150mm increments.

APPENDIX B:

Results of Laboratory Testing

QUALTEST Laboratory (NSW) Pty Ltd (20708)
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California Bearing Ratio Test Report

Report No: CBR:NEW20W-4121--S02
Issue No: 1

Client:	Newcastle Jockey Club Darling Street Broadmeadow NSW 2292
Project No.:	NEW20P-0194
Project Name:	Stables Development

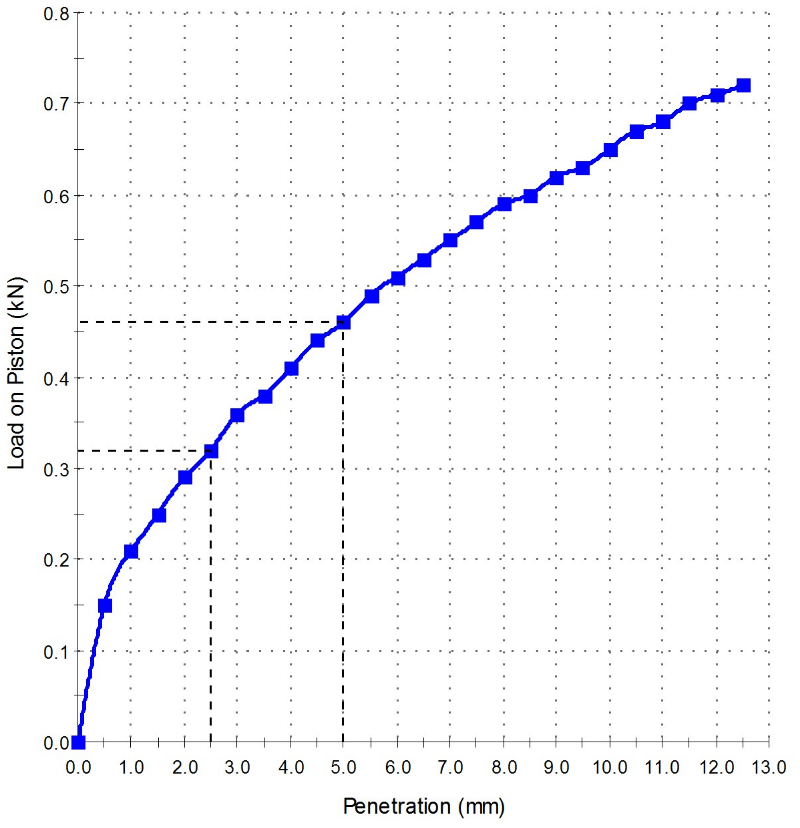
NATA
WORLD RECOGNISED ACCREDITATION

Approved Signatory: Brent Cullen
(Senior Geotechnician)
NATA Accredited Laboratory Number: 18686
Date of Issue: 8/12/2020

Sample Details

Sample ID:	NEW20W-4121--S02	**Date Sampled:**	26/11/2020
Sampling Method:	Sampled by Engineering Department		
Specification:	No Specification	**Source:**	On-Site
Location:	BH02 - (0.4 - 0.8m)	**Material:**	Insitu
Project Location:	Darling Street, Broadmeadow	**Date Tested:**	3/12/2020

Load vs Penetration



Test Results

AS 1289.6.1.1

CBR At 2.5mm (%):	**2.5**
Maximum Dry Density (t/m³):	1.54
Optimum Moisture Content (%):	22.5
Dry Density before Soaking (t/m³):	1.54
Density Ratio before Soaking (%):	100.0
Moisture Content before Soaking (%):	22.5
Moisture Ratio before Soaking (%):	101.0
Dry Density after Soaking (t/m³):	1.53
Density Ratio after Soaking (%):	99.5
Swell (%):	0.5
Moisture Content of Top 30mm (%):	24.4
Moisture Content of Remaining Depth (%):	20.4
Compactive Effort:	Standard
	AS 1289.5.1.1
Surcharge Mass (kg):	9.00
Period of Soaking (Days):	4
Oversize Material (%):	0
CBR Moisture Content Method:	AS 1289.2.1.1
Field Moisture Content (%):	25.9
Curing Time (hrs) :	48

Comments

Method of establishing plasticity level: Visual Assessment
The results outlined above apply to the sample as received

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ABN: 98 153 268 896

Report No: CBR:NEW20W-4121--S03
Issue No: 1

California Bearing Ratio Test Report

Client:	Newcastle Jockey Club Darling Street Broadmeadow NSW 2292
Project No.:	NEW20P-0194
Project Name:	Stables Development

Accredited for compliance with ISO/IEC 17025-Testing.
The results of the tests, calibrations and/or measurements included in this document are traceable to Australian/national standards.
Results provided relate only to the items tested or sampled.
This report shall not be reproduced except in full.

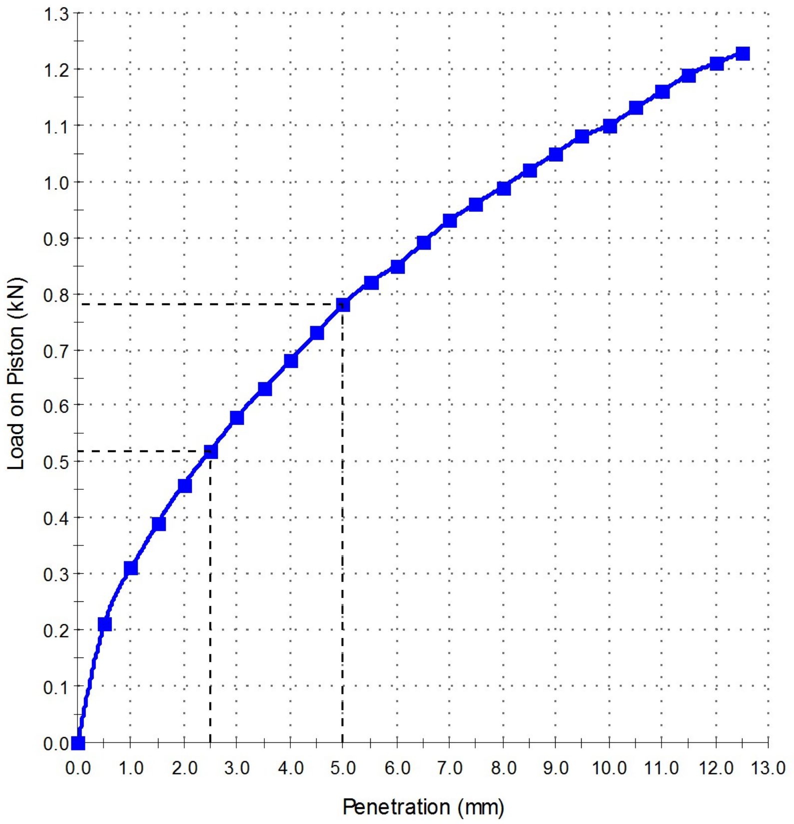
WORLD RECOGNISED ACCREDITATION

B. Cullen
Approved Signatory: Brent Cullen
(Senior Geotechnician)
NATA Accredited Laboratory Number: 18686
Date of Issue: 8/12/2020

Sample Details

Sample ID:	NEW20W-4121--S03	**Date Sampled:**	26/11/2020
Sampling Method:	Sampled by Engineering Department		
Specification:	No Specification	**Source:**	On-Site
Location:	BH04 - (0.2 - 0.7m)	**Material:**	Insitu
Project Location:	Darling Street, Broadmeadow	**Date Tested:**	3/12/2020

Load vs Penetration



Test Results

AS 1289.6.1.1

CBR At 2.5mm (%):	**4.0**
Maximum Dry Density (t/m³):	1.84
Optimum Moisture Content (%):	15.0
Dry Density before Soaking (t/m³):	1.84
Density Ratio before Soaking (%):	100.0
Moisture Content before Soaking (%):	14.5
Moisture Ratio before Soaking (%):	97.5
Dry Density after Soaking (t/m³):	1.84
Density Ratio after Soaking (%):	100.0
Swell (%):	0.0
Moisture Content of Top 30mm (%):	16.6
Moisture Content of Remaining Depth (%):	14.8
Compactive Effort:	Standard
	AS 1289.5.1.1
Surcharge Mass (kg):	9.00
Period of Soaking (Days):	4
Oversize Material (%):	0
CBR Moisture Content Method:	AS 1289.2.1.1
Field Moisture Content (%):	14.8
Curing Time (hrs) :	48

Comments

Method of establishing plasticity level: Visual Assessment
The results outlined above apply to the sample as received

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California Bearing Ratio Test Report

Report No: CBR:NEW20W-4121--S05
Issue No: 1

Client:	Newcastle Jockey Club Darling Street Broadmeadow NSW 2292
Project No.:	NEW20P-0194
Project Name:	Stables Development

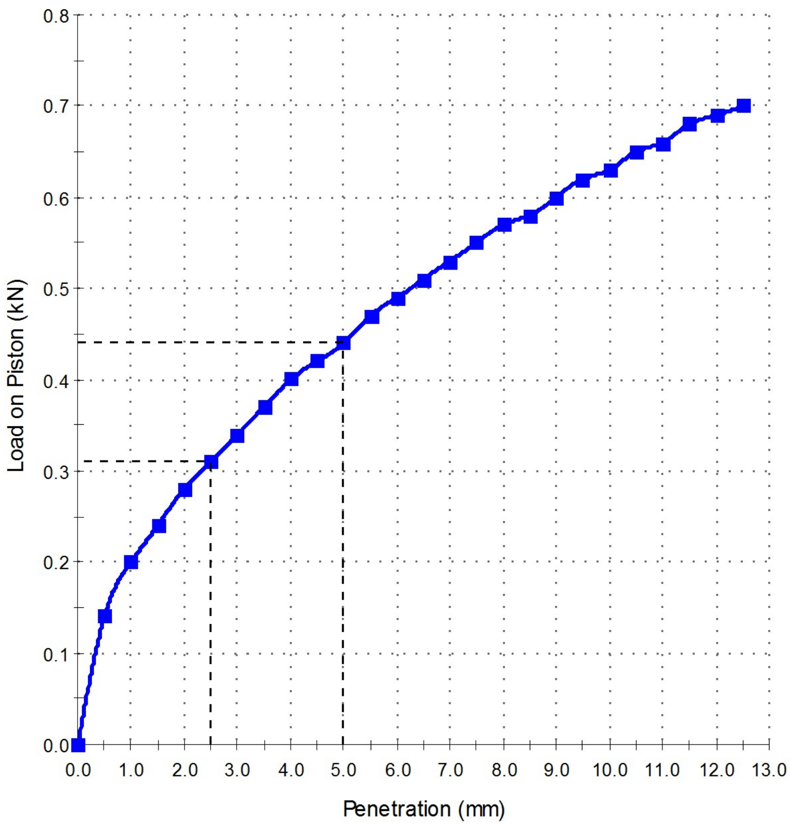
NATA
WORLD RECOGNISED ACCREDITATION

B. Cullen
Approved Signatory: Brent Cullen
(Senior Geotechnician)
NATA Accredited Laboratory Number: 18686
Date of Issue: 8/12/2020

Sample Details

Sample ID:	NEW20W-4121--S05	**Date Sampled:**	26/11/2020
Sampling Method:	Sampled by Engineering Department		
Specification:	No Specification	**Source:**	On-Site
Location:	BH09 - (0.9 - 1.2m)	**Material:**	Insitu
Project Location:	Darling Street, Broadmeadow	**Date Tested:**	3/12/2020

Load vs Penetration



Test Results

AS 1289.6.1.1

CBR At 2.5mm (%):	**2.5**
Maximum Dry Density (t/m³):	1.62
Optimum Moisture Content (%):	20.5
Dry Density before Soaking (t/m³):	1.63
Density Ratio before Soaking (%):	100.5
Moisture Content before Soaking (%):	20.0
Moisture Ratio before Soaking (%):	97.5
Dry Density after Soaking (t/m³):	1.60
Density Ratio after Soaking (%):	98.5
Swell (%):	2.0
Moisture Content of Top 30mm (%):	24.2
Moisture Content of Remaining Depth (%):	20.4
Compactive Effort:	Standard AS 1289.5.1.1
Surcharge Mass (kg):	9.00
Period of Soaking (Days):	4
Oversize Material (%):	0
CBR Moisture Content Method:	AS 1289.2.1.1
Field Moisture Content (%):	25.6
Curing Time (hrs) :	48

Comments

Method of establishing plasticity level: Visual Assessment
The results outlined above apply to the sample as received

QUALTEST Laboratory (NSW) Pty Ltd (20708)
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Report No: SSI:NEW20W-4121--S01
Issue No: 1

Shrink Swell Index Report

Client: Newcastle Jockey Club
Darling Street
Broadmeadow NSW 2292

Project No.: NEW20P-0194
Project Name: Stables Development

NATA
WORLD RECOGNISED ACCREDITATION

Approved Signatory: Brent Cullen
(Senior Geotechnician)
NATA Accredited Laboratory Number: 18686
Date of Issue: 2/12/2020

Sample Details

Sample ID:	NEW20W-4121--S01	**Sampling Method:**	Sampled by Engineering Department
Material:	Insitu	**Date Sampled:**	26/11/2020
Source:	On-Site	**Date Submitted:**	27/11/2020
Specification:	No Specification		
Project Location:	Darling Street, Broadmeadow		
Sample Location:	BH01 - (0.8 - 1.1m)		
Date Tested:	27/11/2020		

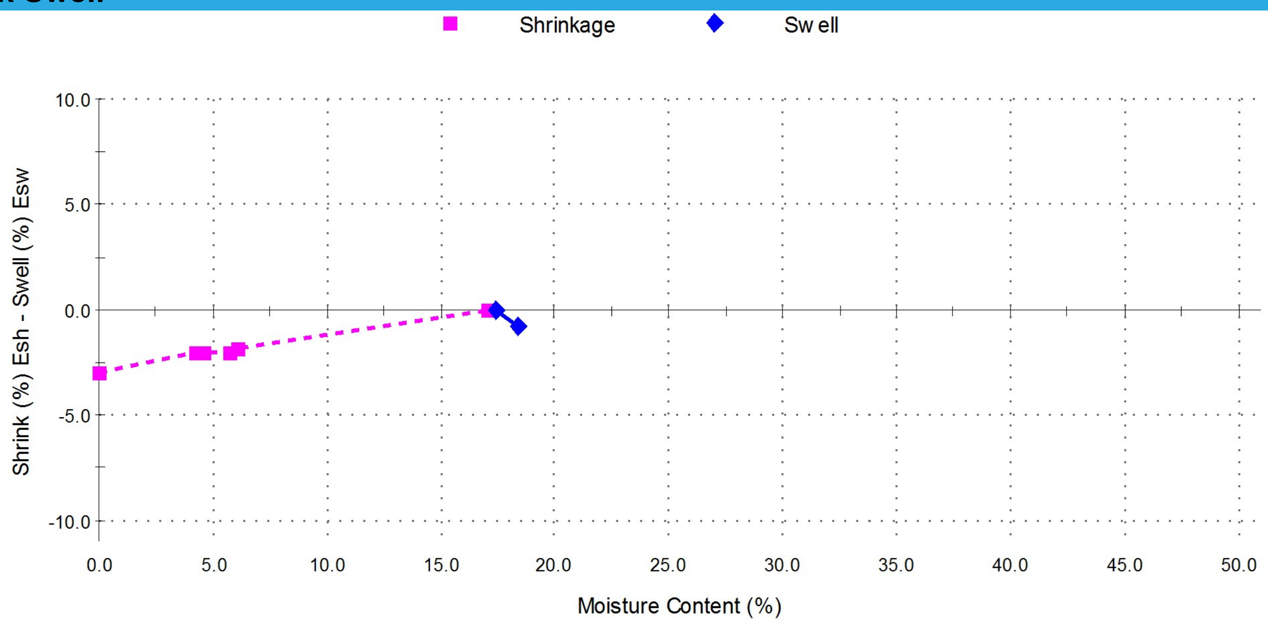
Swell Test — AS 1289.7.1.1

Swell on Saturation (%):	-0.8
Moisture Content before (%):	17.4
Moisture Content after (%):	18.4
Est. Unc. Comp. Strength before (kPa):	140
Est. Unc. Comp. Strength after (kPa):	170

Shrink Test — AS 1289.7.1.1

Shrink on drying (%):	3.0
Shrinkage Moisture Content (%):	17.1
Est. inert material (%):	1%
Crumbling during shrinkage:	Minor
Cracking during shrinkage:	Nil

Shrink Swell



Shrink Swell Index - Iss (%): 1.7

Comments

The results outlined above apply to the sample as received

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ABN: 98 153 268 896

Report No: SSI:NEW20W-4121--S04
Issue No: 1

Shrink Swell Index Report

Client: Newcastle Jockey Club
Darling Street
Broadmeadow NSW 2292

Project No.: NEW20P-0194
Project Name: Stables Development

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WORLD RECOGNISED ACCREDITATION

Approved Signatory: Brent Cullen
(Senior Geotechnician)
NATA Accredited Laboratory Number: 18686
Date of Issue: 2/12/2020

Sample Details

Sample ID:	NEW20W-4121--S04	**Sampling Method:**	Sampled by Engineering Department
Material:	Insitu	**Date Sampled:**	26/11/2020
Source:	On-Site	**Date Submitted:**	27/11/2020
Specification:	No Specification		
Project Location:	Darling Street, Broadmeadow		
Sample Location:	BH05 - (0.4 - 0.6m)		
Date Tested:	27/11/2020		

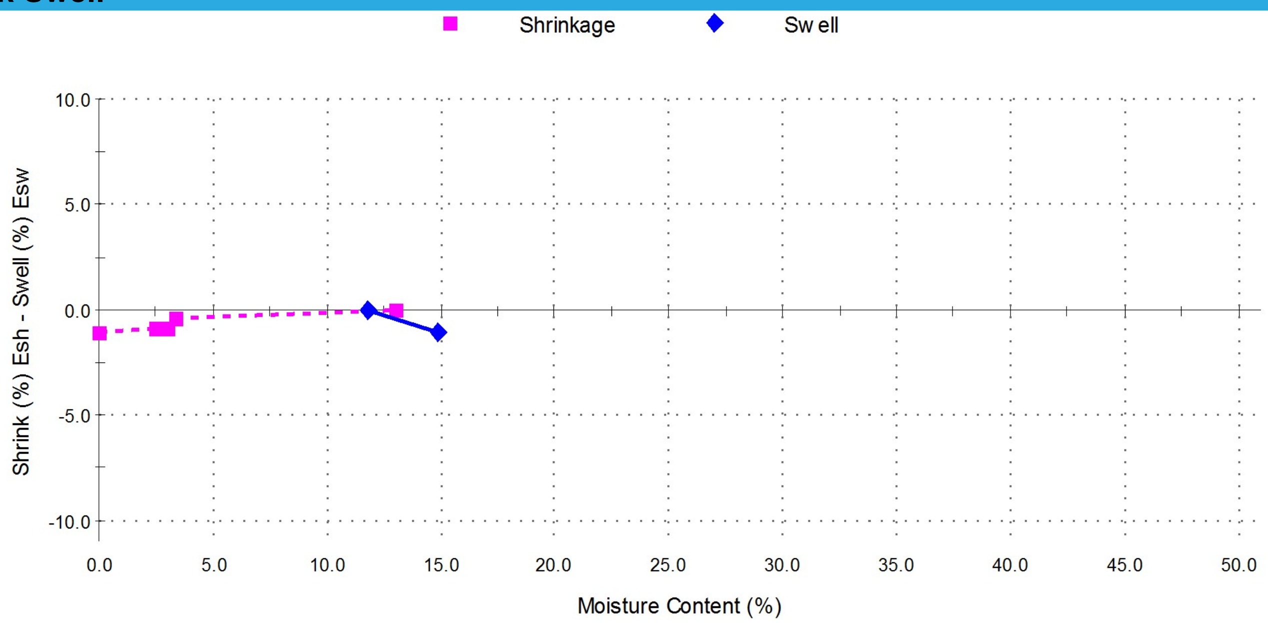
Swell Test — AS 1289.7.1.1

Swell on Saturation (%):	-1.1
Moisture Content before (%):	11.8
Moisture Content after (%):	14.9
Est. Unc. Comp. Strength before (kPa):	210
Est. Unc. Comp. Strength after (kPa):	>600

Shrink Test — AS 1289.7.1.1

Shrink on drying (%):	1.1
Shrinkage Moisture Content (%):	13.0
Est. inert material (%):	10%
Crumbling during shrinkage:	Nil
Cracking during shrinkage:	Nil

Shrink Swell



Shrink Swell Index - Iss (%): 0.6

Comments

The results outlined above apply to the sample as received

QUALTEST Laboratory (NSW) Pty Ltd (20708)
8 Ironbark Close Warabrook NSW 2304
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ABN: 98 153 268 896

Report No: SSI:NEW20W-4121--S06
Issue No: 1

Shrink Swell Index Report

Client: Newcastle Jockey Club
Darling Street
Broadmeadow NSW 2292

Project No.: NEW20P-0194
Project Name: Stables Development

NATA WORLD RECOGNISED ACCREDITATION

Approved Signatory: Brent Cullen
(Senior Geotechnician)
NATA Accredited Laboratory Number: 18686
Date of Issue: 2/12/2020

Sample Details

Sample ID:	NEW20W-4121--S06	**Sampling Method:**	Sampled by Engineering Department
Material:	Insitu	**Date Sampled:**	26/11/2020
Source:	On-Site	**Date Submitted:**	27/11/2020
Specification:	No Specification		
Project Location:	Darling Street, Broadmeadow		
Sample Location:	BH10 - (0.8 - 1.1m)		
Date Tested:	27/11/2020		

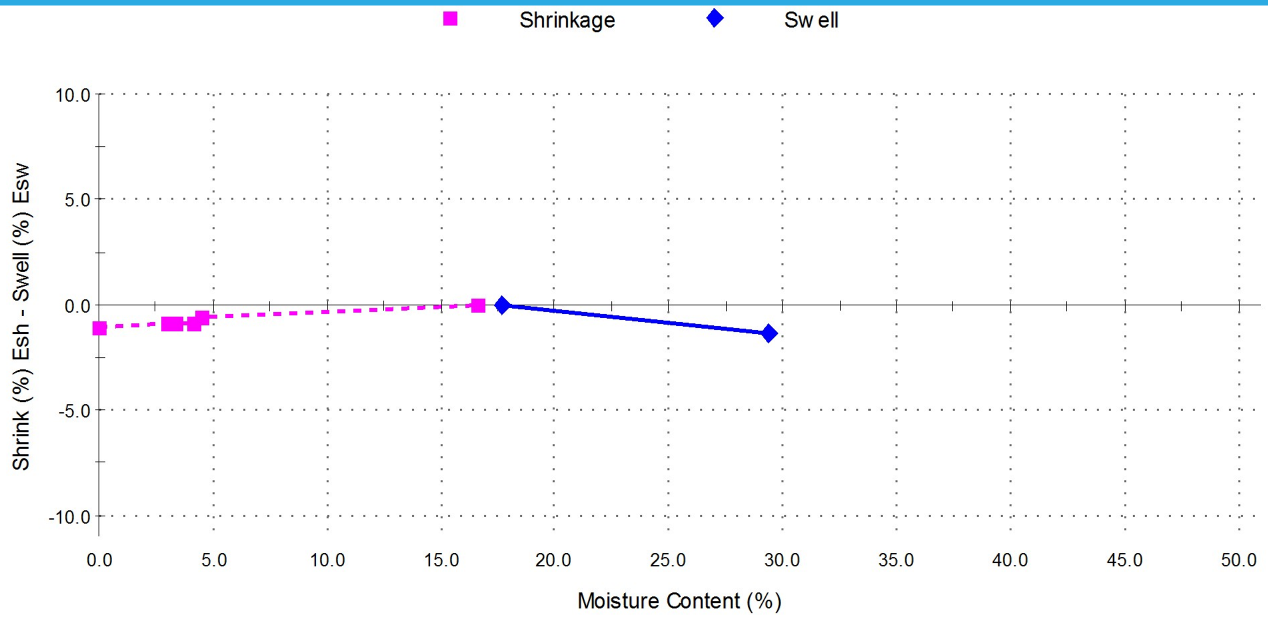
Swell Test — AS 1289.7.1.1

Swell on Saturation (%):	-1.4
Moisture Content before (%):	17.6
Moisture Content after (%):	29.4
Est. Unc. Comp. Strength before (kPa):	170
Est. Unc. Comp. Strength after (kPa):	70

Shrink Test — AS 1289.7.1.1

Shrink on drying (%):	1.1
Shrinkage Moisture Content (%):	16.6
Est. inert material (%):	1%
Crumbling during shrinkage:	Minor
Cracking during shrinkage:	Nil

Shrink Swell



Shrink Swell Index - Iss (%): 0.6

Comments

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Report No: SSI:NEW20W-4121--S07
Issue No: 1

Shrink Swell Index Report

Client: Newcastle Jockey Club
Darling Street
Broadmeadow NSW 2292

Project No.: NEW20P-0194
Project Name: Stables Development

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NATA
WORLD RECOGNISED ACCREDITATION

Approved Signatory: Brent Cullen
(Senior Geotechnician)
NATA Accredited Laboratory Number: 18686
Date of Issue: 2/12/2020

Sample Details

Sample ID:	NEW20W-4121--S07	**Sampling Method:**	Sampled by Engineering Department
Material:	Insitu	**Date Sampled:**	26/11/2020
Source:	On-Site	**Date Submitted:**	27/11/2020
Specification:	No Specification		
Project Location:	Darling Street, Broadmeadow		
Sample Location:	BH12 - (0.9 - 1.1m)		
Date Tested:	27/11/2020		

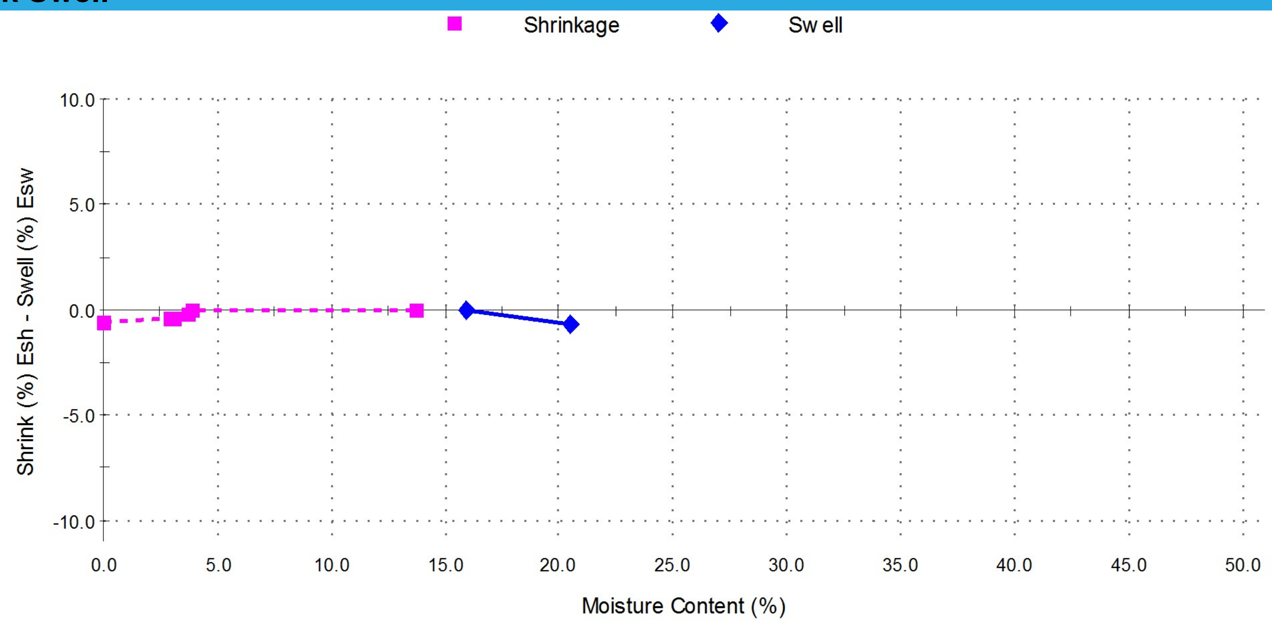
Swell Test — AS 1289.7.1.1

Swell on Saturation (%):	-0.7
Moisture Content before (%):	15.9
Moisture Content after (%):	20.5
Est. Unc. Comp. Strength before (kPa):	300
Est. Unc. Comp. Strength after (kPa):	160

Shrink Test — AS 1289.7.1.1

Shrink on drying (%):	0.6
Shrinkage Moisture Content (%):	13.7
Est. inert material (%):	1%
Crumbling during shrinkage:	Minor
Cracking during shrinkage:	Nil

Shrink Swell



Shrink Swell Index - Iss (%): 0.3

Comments

The results outlined above apply to the sample as received

APPENDIX C:

CSIRO Sheet BTF 18

Foundation Maintenance and Footing Performance: A Homeowner's Guide

Foundation Maintenance and Footing Performance: A Homeowner's Guide

CSIRO

BTF 18 replaces Information Sheet 10/91

Buildings can and often do move. This movement can be up, down, lateral or rotational. The fundamental cause of movement in buildings can usually be related to one or more problems in the foundation soil. It is important for the homeowner to identify the soil type in order to ascertain the measures that should be put in place in order to ensure that problems in the foundation soil can be prevented, thus protecting against building movement.

This Building Technology File is designed to identify causes of soil-related building movement, and to suggest methods of prevention of resultant cracking in buildings.

Soil Types

The types of soils usually present under the topsoil in land zoned for residential buildings can be split into two approximate groups – granular and clay. Quite often, foundation soil is a mixture of both types. The general problems associated with soils having granular content are usually caused by erosion. Clay soils are subject to saturation and swell/shrink problems.

Classifications for a given area can generally be obtained by application to the local authority, but these are sometimes unreliable and if there is doubt, a geotechnical report should be commissioned. As most buildings suffering movement problems are founded on clay soils, there is an emphasis on classification of soils according to the amount of swell and shrinkage they experience with variations of water content. The table below is Table 2.1 from AS 2870, the Residential Slab and Footing Code.

Causes of Movement

Settlement due to construction

There are two types of settlement that occur as a result of construction:

- Immediate settlement occurs when a building is first placed on its foundation soil, as a result of compaction of the soil under the weight of the structure. The cohesive quality of clay soil mitigates against this, but granular (particularly sandy) soil is susceptible.
- Consolidation settlement is a feature of clay soil and may take place because of the expulsion of moisture from the soil or because of the soil's lack of resistance to local compressive or shear stresses. This will usually take place during the first few months after construction, but has been known to take many years in exceptional cases.

These problems are the province of the builder and should be taken into consideration as part of the preparation of the site for construction. Building Technology File 19 (BTF 19) deals with these problems.

Erosion

All soils are prone to erosion, but sandy soil is particularly susceptible to being washed away. Even clay with a sand component of say 10% or more can suffer from erosion.

Saturation

This is particularly a problem in clay soils. Saturation creates a bog-like suspension of the soil that causes it to lose virtually all of its bearing capacity. To a lesser degree, sand is affected by saturation because saturated sand may undergo a reduction in volume – particularly imported sand fill for bedding and blinding layers. However, this usually occurs as immediate settlement and should normally be the province of the builder.

Seasonal swelling and shrinkage of soil

All clays react to the presence of water by slowly absorbing it, making the soil increase in volume (see table below). The degree of increase varies considerably between different clays, as does the degree of decrease during the subsequent drying out caused by fair weather periods. Because of the low absorption and expulsion rate, this phenomenon will not usually be noticeable unless there are prolonged rainy or dry periods, usually of weeks or months, depending on the land and soil characteristics.

The swelling of soil creates an upward force on the footings of the building, and shrinkage creates subsidence that takes away the support needed by the footing to retain equilibrium.

Shear failure

This phenomenon occurs when the foundation soil does not have sufficient strength to support the weight of the footing. There are two major post-construction causes:

- Significant load increase.
- Reduction of lateral support of the soil under the footing due to erosion or excavation.
- In clay soil, shear failure can be caused by saturation of the soil adjacent to or under the footing.

GENERAL DEFINITIONS OF SITE CLASSES

Class	Foundation
A	Most sand and rock sites with little or no ground movement from moisture changes
S	Slightly reactive clay sites with only slight ground movement from moisture changes
M	Moderately reactive clay or silt sites, which can experience moderate ground movement from moisture changes
H	Highly reactive clay sites, which can experience high ground movement from moisture changes
E	Extremely reactive sites, which can experience extreme ground movement from moisture changes
A to P	Filled sites
P	Sites which include soft soils, such as soft clay or silt or loose sands; landslip; mine subsidence; collapsing soils; soils subject to erosion; reactive sites subject to abnormal moisture conditions or sites which cannot be classified otherwise

Tree root growth
Trees and shrubs that are allowed to grow in the vicinity of footings can cause foundation soil movement in two ways:

- Roots that grow under footings may increase in cross-sectional size, exerting upward pressure on footings.
- Roots in the vicinity of footings will absorb much of the moisture in the foundation soil, causing shrinkage or subsidence.

Unevenness of Movement

The types of ground movement described above usually occur unevenly throughout the building's foundation soil. Settlement due to construction tends to be uneven because of:

- Differing compaction of foundation soil prior to construction.
- Differing moisture content of foundation soil prior to construction.

Movement due to non-construction causes is usually more uneven still. Erosion can undermine a footing that traverses the flow or can create the conditions for shear failure by eroding soil adjacent to a footing that runs in the same direction as the flow.

Saturation of clay foundation soil may occur where subfloor walls create a dam that makes water pond. It can also occur wherever there is a source of water near footings in clay soil. This leads to a severe reduction in the strength of the soil which may create local shear failure.

Seasonal swelling and shrinkage of clay soil affects the perimeter of the building first, then gradually spreads to the interior. The swelling process will usually begin at the uphill extreme of the building, or on the weather side where the land is flat. Swelling gradually reaches the interior soil as absorption continues. Shrinkage usually begins where the sun's heat is greatest.

Effects of Uneven Soil Movement on Structures

Erosion and saturation
Erosion removes the support from under footings, tending to create subsidence of the part of the structure under which it occurs. Brickwork walls will resist the stress created by this removal of support by bridging the gap or cantilevering until the bricks or the mortar bedding fail. Older masonry has little resistance. Evidence of failure varies according to circumstances and symptoms may include:

- Step cracking in the mortar beds in the body of the wall or above/below openings such as doors or windows.
- Vertical cracking in the bricks (usually but not necessarily in line with the vertical beds or perpends).

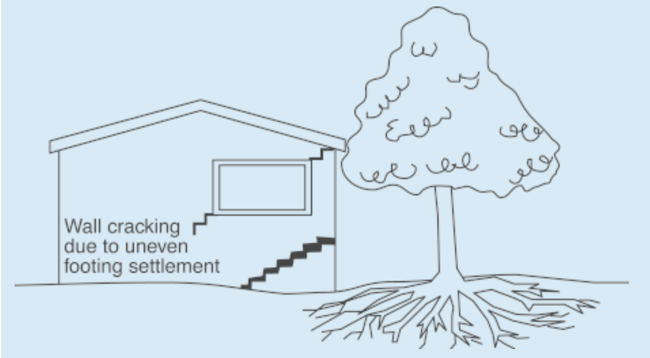
Isolated piers affected by erosion or saturation of foundations will eventually lose contact with the bearers they support and may tilt or fall over. The floors that have lost this support will become bouncy, sometimes rattling ornaments etc.

Seasonal swelling/shrinkage in clay
Swelling foundation soil due to rainy periods first lifts the most exposed extremities of the footing system, then the remainder of the perimeter footings while gradually permeating inside the building footprint to lift internal footings. This swelling first tends to create a dish effect, because the external footings are pushed higher than the internal ones.

The first noticeable symptom may be that the floor appears slightly dished. This is often accompanied by some doors binding on the floor or the door head, together with some cracking of cornice mitres. In buildings with timber flooring supported by bearers and joists, the floor can be bouncy. Externally there may be visible dishing of the hip or ridge lines.

As the moisture absorption process completes its journey to the innermost areas of the building, the internal footings will rise. If the spread of moisture is roughly even, it may be that the symptoms will temporarily disappear, but it is more likely that swelling will be uneven, creating a difference rather than a disappearance in symptoms. In buildings with timber flooring supported by bearers and joists, the isolated piers will rise more easily than the strip footings or piers under walls, creating noticeable doming of flooring.

Trees can cause shrinkage and damage



As the weather pattern changes and the soil begins to dry out, the external footings will be first affected, beginning with the locations where the sun's effect is strongest. This has the effect of lowering the external footings. The doming is accentuated and cracking reduces or disappears where it occurred because of dishing, but other cracks open up. The roof lines may become convex.

Doming and dishing are also affected by weather in other ways. In areas where warm, wet summers and cooler dry winters prevail, water migration tends to be toward the interior and doming will be accentuated, whereas where summers are dry and winters are cold and wet, migration tends to be toward the exterior and the underlying propensity is toward dishing.

Movement caused by tree roots
In general, growing roots will exert an upward pressure on footings, whereas soil subject to drying because of tree or shrub roots will tend to remove support from under footings by inducing shrinkage.

Complications caused by the structure itself
Most forces that the soil causes to be exerted on structures are vertical – i.e. either up or down. However, because these forces are seldom spread evenly around the footings, and because the building resists uneven movement because of its rigidity, forces are exerted from one part of the building to another. The net result of all these forces is usually rotational. This resultant force often complicates the diagnosis because the visible symptoms do not simply reflect the original cause. A common symptom is binding of doors on the vertical member of the frame.

Effects on full masonry structures
Brickwork will resist cracking where it can. It will attempt to span areas that lose support because of subsided foundations or raised points. It is therefore usual to see cracking at weak points, such as openings for windows or doors.

In the event of construction settlement, cracking will usually remain unchanged after the process of settlement has ceased.

With local shear or erosion, cracking will usually continue to develop until the original cause has been remedied, or until the subsidence has completely neutralised the affected portion of footing and the structure has stabilised on other footings that remain effective.

In the case of swell/shrink effects, the brickwork will in some cases return to its original position after completion of a cycle, however it is more likely that the rotational effect will not be exactly reversed, and it is also usual that brickwork will settle in its new position and will resist the forces trying to return it to its original position. This means that in a case where swelling takes place after construction and cracking occurs, the cracking is likely to at least partly remain after the shrink segment of the cycle is complete. Thus, each time the cycle is repeated, the likelihood is that the cracking will become wider until the sections of brickwork become virtually independent.

With repeated cycles, once the cracking is established, if there is no other complication, it is normal for the incidence of cracking to stabilise, as the building has the articulation it needs to cope with the problem. This is by no means always the case, however, and monitoring of cracks in walls and floors should always be treated seriously.

Upheaval caused by growth of tree roots under footings is not a simple vertical shear stress. There is a tendency for the root to also exert lateral forces that attempt to separate sections of brickwork after initial cracking has occurred.

The normal structural arrangement is that the inner leaf of brickwork in the external walls and at least some of the internal walls (depending on the roof type) comprise the load-bearing structure on which any upper floors, ceilings and the roof are supported. In these cases, it is internally visible cracking that should be the main focus of attention, however there are a few examples of dwellings whose external leaf of masonry plays some supporting role, so this should be checked if there is any doubt. In any case, externally visible cracking is important as a guide to stresses on the structure generally, and it should also be remembered that the external walls must be capable of supporting themselves.

Effects on framed structures
Timber or steel framed buildings are less likely to exhibit cracking due to swell/shrink than masonry buildings because of their flexibility. Also, the doming/dishing effects tend to be lower because of the lighter weight of walls. The main risks to framed buildings are encountered because of the isolated pier footings used under walls. Where erosion or saturation cause a footing to fall away, this can double the span which a wall must bridge. This additional stress can create cracking in wall linings, particularly where there is a weak point in the structure caused by a door or window opening. It is, however, unlikely that framed structures will be so stressed as to suffer serious damage without first exhibiting some or all of the above symptoms for a considerable period. The same warning period should apply in the case of upheaval. It should be noted, however, that where framed buildings are supported by strip footings there is only one leaf of brickwork and therefore the externally visible walls are the supporting structure for the building. In this case, the subfloor masonry walls can be expected to behave as full brickwork walls.

Effects on brick veneer structures
Because the load-bearing structure of a brick veneer building is the frame that makes up the interior leaf of the external walls plus perhaps the internal walls, depending on the type of roof, the building can be expected to behave as a framed structure, except that the external masonry will behave in a similar way to the external leaf of a full masonry structure.

Water Service and Drainage

Where a water service pipe, a sewer or stormwater drainage pipe is in the vicinity of a building, a water leak can cause erosion, swelling or saturation of susceptible soil. Even a minuscule leak can be enough to saturate a clay foundation. A leaking tap near a building can have the same effect. In addition, trenches containing pipes can become watercourses even though backfilled, particularly where broken rubble is used as fill. Water that runs along these trenches can be responsible for serious erosion, interstrata seepage into subfloor areas and saturation.

Pipe leakage and trench water flows also encourage tree and shrub roots to the source of water, complicating and exacerbating the problem.
Poor roof plumbing can result in large volumes of rainwater being concentrated in a small area of soil:

- Incorrect falls in roof guttering may result in overflows, as may gutters blocked with leaves etc.
- Corroded guttering or downpipes can spill water to ground.
- Downpipes not positively connected to a proper stormwater collection system will direct a concentration of water to soil that is directly adjacent to footings, sometimes causing large-scale problems such as erosion, saturation and migration of water under the building.

Seriousness of Cracking

In general, most cracking found in masonry walls is a cosmetic nuisance only and can be kept in repair or even ignored. The table below is a reproduction of Table C1 of AS 2870.

AS 2870 also publishes figures relating to cracking in concrete floors, however because wall cracking will usually reach the critical point significantly earlier than cracking in slabs, this table is not reproduced here.

Prevention/Cure

Plumbing
Where building movement is caused by water service, roof plumbing, sewer or stormwater failure, the remedy is to repair the problem. It is prudent, however, to consider also rerouting pipes away from the building where possible, and relocating taps to positions where any leakage will not direct water to the building vicinity. Even where gully traps are present, there is sometimes sufficient spill to create erosion or saturation, particularly in modern installations using smaller diameter PVC fixtures. Indeed, some gully traps are not situated directly under the taps that are installed to charge them, with the result that water from the tap may enter the backfilled trench that houses the sewer piping. If the trench has been poorly backfilled, the water will either pond or flow along the bottom of the trench. As these trenches usually run alongside the footings and can be at a similar depth, it is not hard to see how any water that is thus directed into a trench can easily affect the foundation's ability to support footings or even gain entry to the subfloor area.

Ground drainage
In all soils there is the capacity for water to travel on the surface and below it. Surface water flows can be established by inspection during and after heavy or prolonged rain. If necessary, a grated drain system connected to the stormwater collection system is usually an easy solution.

It is, however, sometimes necessary when attempting to prevent water migration that testing be carried out to establish watertable height and subsoil water flows. This subject is referred to in BTF 19 and may properly be regarded as an area for an expert consultant.

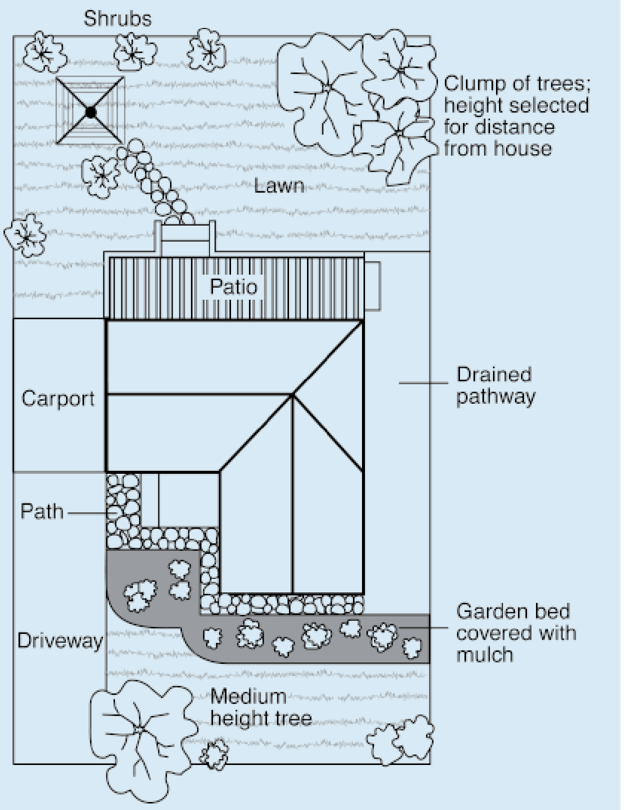
Protection of the building perimeter
It is essential to remember that the soil that affects footings extends well beyond the actual building line. Watering of garden plants, shrubs and trees causes some of the most serious water problems.

For this reason, particularly where problems exist or are likely to occur, it is recommended that an apron of paving be installed around as much of the building perimeter as necessary. This paving

CLASSIFICATION OF DAMAGE WITH REFERENCE TO WALLS

Description of typical damage and required repair	Approximate crack width limit (see Note 3)	Damage category
Hairline cracks	<0.1 mm	0
Fine cracks which do not need repair	<1 mm	1
Cracks noticeable but easily filled. Doors and windows stick slightly	<5 mm	2
Cracks can be repaired and possibly a small amount of wall will need to be replaced. Doors and windows stick. Service pipes can fracture. Weathertightness often impaired	5–15 mm (or a number of cracks 3 mm or more in one group)	3
Extensive repair work involving breaking-out and replacing sections of walls, especially over doors and windows. Window and door frames distort. Walls lean or bulge noticeably, some loss of bearing in beams. Service pipes disrupted	15–25 mm but also depend on number of cracks	4

Gardens for a reactive site



should extend outwards a minimum of 900 mm (more in highly reactive soil) and should have a minimum fall away from the building of 1:60. The finished paving should be no less than 100 mm below brick vent bases.

It is prudent to relocate drainage pipes away from this paving, if possible, to avoid complications from future leakage. If this is not practical, earthenware pipes should be replaced by PVC and backfilling should be of the same soil type as the surrounding soil and compacted to the same density.

Except in areas where freezing of water is an issue, it is wise to remove taps in the building area and relocate them well away from the building – preferably not uphill from it (see BTF 19).

It may be desirable to install a grated drain at the outside edge of the paving on the uphill side of the building. If subsoil drainage is needed this can be installed under the surface drain.

Condensation

In buildings with a subfloor void such as where bearers and joists support flooring, insufficient ventilation creates ideal conditions for condensation, particularly where there is little clearance between the floor and the ground. Condensation adds to the moisture already present in the subfloor and significantly slows the process of drying out. Installation of an adequate subfloor ventilation system, either natural or mechanical, is desirable.

Warning: Although this Building Technology File deals with cracking in buildings, it should be said that subfloor moisture can result in the development of other problems, notably:

- Water that is transmitted into masonry, metal or timber building elements causes damage and/or decay to those elements.
- High subfloor humidity and moisture content create an ideal environment for various pests, including termites and spiders.
- Where high moisture levels are transmitted to the flooring and walls, an increase in the dust mite count can ensue within the living areas. Dust mites, as well as dampness in general, can be a health hazard to inhabitants, particularly those who are abnormally susceptible to respiratory ailments.

The garden

The ideal vegetation layout is to have lawn or plants that require only light watering immediately adjacent to the drainage or paving edge, then more demanding plants, shrubs and trees spread out in that order.

Overwatering due to misuse of automatic watering systems is a common cause of saturation and water migration under footings. If it is necessary to use these systems, it is important to remove garden beds to a completely safe distance from buildings.

Existing trees

Where a tree is causing a problem of soil drying or there is the existence or threat of upheaval of footings, if the offending roots are subsidiary and their removal will not significantly damage the tree, they should be severed and a concrete or metal barrier placed vertically in the soil to prevent future root growth in the direction of the building. If it is not possible to remove the relevant roots without damage to the tree, an application to remove the tree should be made to the local authority. A prudent plan is to transplant likely offenders before they become a problem.

Information on trees, plants and shrubs

State departments overseeing agriculture can give information regarding root patterns, volume of water needed and safe distance from buildings of most species. Botanic gardens are also sources of information. For information on plant roots and drains, see Building Technology File 17.

Excavation

Excavation around footings must be properly engineered. Soil supporting footings can only be safely excavated at an angle that allows the soil under the footing to remain stable. This angle is called the angle of repose (or friction) and varies significantly between soil types and conditions. Removal of soil within the angle of repose will cause subsidence.

Remediation

Where erosion has occurred that has washed away soil adjacent to footings, soil of the same classification should be introduced and compacted to the same density. Where footings have been undermined, augmentation or other specialist work may be required. Remediation of footings and foundations is generally the realm of a specialist consultant.

Where isolated footings rise and fall because of swell/shrink effect, the homeowner may be tempted to alleviate floor bounce by filling the gap that has appeared between the bearer and the pier with blocking. The danger here is that when the next swell segment of the cycle occurs, the extra blocking will push the floor up into an accentuated dome and may also cause local shear failure in the soil. If it is necessary to use blocking, it should be by a pair of fine wedges and monitoring should be carried out fortnightly.

This BTF was prepared by John Lewer FAIB, MIAMA, Partner, Construction Diagnosis.

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