Proposed Carpark - Geotechnical Assessment

Woodberry Learning Centre, Woodberry Road, Woodberry

NEW21P-0138-AA.rev1
1 July 2022

Qualtest
LABORATORY (NSW) PTY LTD

GEOTECHNICAL I LABORATORY I EARTHWORKS I QUARRY I CONSTRUCTION MATERIAL TESTING

1 July 2022

Metiri Consultants
5/33 The Boulevarde,
TORONTO NSW 2283

Attention: Mr Jamie Russell

Dear Jamie

RE: PROPOSED WOODBERRY LEARNING CENTRE CARPARK
WOODBERRY ROAD, WOODBERRY
GEOTECHNICAL ASSESSMENT

Please find enclosed our geotechnical assessment report for the proposed carpark and extension/fit out of the "D Block" building to be constructed at the Woodberry Learning Centre, located at Woodberry Road, Woodberry.

This report is a revision to the previously issued geotechnical assessment report ref. NEW21P-0138-AA, dated 26 October 2021, for the purpose of including rigid (concrete) pavement design.

The report includes recommendations for site classification in accordance with AS2870-2011, "*Residential Slabs and Footings*", foundation and retaining wall design parameters, pavement thickness design and construction recommendations for the proposed car park area, excavation conditions, suitability of site-won materials for re-use as fill, and site earthworks.

If you have any 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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Attachments:

1.0 Introduction

Qualtest Laboratory NSW Pty Ltd (Qualtest) is pleased to present this geotechnical report on behalf of Metiri Consultants (Metiri), for the proposed Woodberry Learning Centre Carpark project.

Based on the brief and drawings provided by Metiri in an email dated 25 August 2021 (including RFT No. SINSW-21-10605, July 2021), the proposed development is understood to comprise of extension/fit out of the "D Block" building to facilitate a new administration/staff building, and a new carpark.

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

- Site preparation;
- Excavation conditions and depth to rock (where encountered);
- The suitability of the site soils for use as fill and fill construction procedures;
- Preliminary site classification to AS2870-2011, "*Residential Slabs and Footings*";
- Foundation and retaining wall design parameters (within depth of investigation);
- Pavement design and construction to MCC Specifications; and,
- Special requirements for construction procedures and site drainage.

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 9 September 2021, and comprised of:

- DBYD search and scanning of proposed test locations using an accredited professional cable locator engaged by the client to check for the presence of underground services;
- Observations of surface features at the site and in the immediate surrounding area;
- Drilling of 3 boreholes (BH01 to BH03), drilled to depths of between 1.20m and 1.95m, using a hand auger and hand tools;
- Dynamic Cone Penetrometer (DCP) tests were undertaken adjacent to borehole locations carried out to depths of between 1.20m and 1.65m;
- Bulk disturbed samples and undisturbed samples (U50 tubes) were taken for subsequent laboratory testing; and,
- Boreholes were backfilled with the drilling spoil, and compacted using the hand auger and hand tools.

Investigations were carried out by an experienced Geotechnical Engineer from Qualtest who located the boreholes, carried out the sampling and testing, and provided field logs. Engineering logs of the boreholes are presented in Appendix A.

Approximate borehole locations are shown on the attached Figure AA1. Boreholes were located in the field relative to existing site features. Boreholes were surveyed by project surveyors on the day of investigations. GPS coordinates provided by site surveyors are shown in the table as follows.

Test Location	Easting (m)	Northing (m)	Elevation (RL AHD m)
BH1	375630.513	6370990.496	6.18
BH2	375617.828	6370971.233	5.61
BH3	375628.173	6370948.243	4.95

3.0 Site Description

3.1 Surface Conditions

The site is located at Woodberry Learning Centre, at part of Lot 1 DP 557998, Woodberry Road, Woodberry.

Woodberry Learning Centre is bounded to the north by Lawson Avenue, to the east by Woodberry Road, to the south by rural-residential use allotments, and by Woodberry Public School to the west.

The site of the proposed car park is located to the south of the Learning Centre. The "D Block" building and proposed driveway access are located in the south-eastern area of the Learning Centre, as shown on Figure AA1, attached.

Existing development includes four demountable classroom/administration buildings, a sealed carpark area, a paved area for basketball courts and undercover area, and boundary fences.

Areas of the site away from structures are generally vegetated by established grass cover, and a number of scattered established trees, generally close to the lot boundaries, including the eastern side of the proposed carpark and driveway areas.

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

Photograph 1: From near southern end of proposed car park area, near BH3, facing north.

Photograph 2: From west of northern end of proposed car park area, facing northeast. Existing "D Block" visible.

Photograph 3: From west of northern end of proposed car park area, facing east.

Photograph 4: From west of northern end of proposed car park area, facing southeast.

Photograph 5: From northeast of "D Block", beside eastern boundary, facing south.

Photograph 6 From northeast of "D Block", beside eastern boundary, facing west.

3.2 Subsurface Conditions

Reference to the 1:100,000 Newcastle Coalfield Regional Geology Sheets 9231, indicates the site to be underlain by Tomago Coal Measures indicated to generally comprise Siltstone, Sandstone, Coal, Tuff, and Minor Carbonaceous Claystone rock types.

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

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

TABLE 1 – SUMMARY OF GEOTECHNICAL UNITS AND SOIL TYPES

Unit	Soil Type	Description
1A	FILL - Topsoil	Sandy CLAY – low to medium plasticity, brown to dark brown, fine to medium grained sand, root affected.
1B	FILL - Other	CLAY, Sandy CLAY – medium to high plasticity, brown and red-brown to pale orange-brown, fine to medium grained sand.
2	Topsoil	Sandy CLAY – low plasticity, grey-brown to dark brown, fine to medium grained sand, root affected.
3	Colluvium	Sandy CLAY, Sandy CLAY/Clayey SAND – low to medium plasticity, pale grey-brown to grey-brown, fine to coarse grained sand. Trace fine to medium grained sub-rounded to sub-angular gravel in places, with some roots in places, weakly cemented in places.
4	Residual Soil	Sandy CLAY - medium plasticity, pale brown to pale orange-brown with some red-brown, fine to medium grained sand. CLAY – medium to high plasticity, mixtures of pale brown, pale orange-brown, red-brown, pale grey, fine to medium grained sand.

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

Location	Unit 1A FILL - Topsoil	Unit 1B FILL - Other	Unit 2 Topsoil	Unit 3 Colluvium	Unit 4 Residual Soil
	Depth (m)				
BH1	0.00 - 0.10	0.10 - 0.50	0.50 - 0.65	0.65 - 0.75	0.75 - 1.95
BH2	-	-	0.00 - 0.30	0.30 - 0.55	0.55 - 1.20
BH3	-	-	0.00 - 0.30	0.30 - 0.45	0.45 - 1.20

No water inflows or groundwater levels were observed in the boreholes during the limited time that they remained open on the day of investigation.

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 Newcastle Laboratory for testing which comprised:

- (2 no.) California Bearing Ratio (CBR, 4 day soaked) & Standard Compaction; and,
- (1 no.) Shrink/Swell test.

Results of the laboratory testing are presented in Appendix B, with a summary of the Shrink/Swell and CBR test results presented in Table 3 and Table 4, respectively.

TABLE 3 – SUMMARY OF SHRINK / SWELL TESTING RESULTS

Location	Depth (m)	Material Description	I_{ss} (%)
BH1	0.80 – 0.95	(CH) CLAY	2.3

TABLE 4 – SUMMARY OF CBR TESTING RESULTS

Location	Sample Depth (m)	Field Moisture Content (%)	Optimum Moisture Content (%)	Relationship of Field MC to OMC (%)	CBR (%)
BH2	0.60 – 0.70	27.4	26.2	1.2 WET	5
BH3	0.45 – 0.60	13.7	20.7	7.0 DRY	8

5.0 Discussion and Recommendations

5.1 Preliminary Site Classification to AS2870-2011

Site Classification to AS2870 is not strictly applicable to this site due to it being a proposed school building rather than a residential development. However, the principles of footing design and site maintenance presented therein may be taken into account for structures if proposed for the site.

The following site classification is a preliminary assessment based upon limited test holes and one laboratory shrink/swell test, based upon the proposed "D Block" development outlined in the 'Request for Tender' document. It is recommended that further assessment be undertaken if needed.

The proposed development area at the Woodberry Learning Centre, as shown on Figure AA1, is preliminarily classified in its current condition in accordance with AS2870-2011 '*Residential Slabs and Footings*', as shown in Table 5.

TABLE 5 – PRELIMINARY SITE CLASSIFICATION TO AS2870-2011

Site Location	Site Classification
Areas with greater than 0.40m Uncontrolled Filling and/or Topsoil BH1 profile indicated 0.50m uncontrolled filling, overlying buried Topsoil to a depth of 0.65m.	**P**
Natural Soil Profile BH2 and BH3 indicated natural profile.	**H1**

Areas of the site have been classified as **Class 'P'** due to the assessed presence of uncontrolled filling and buried topsoil to depths of greater than 0.40m (depths of up to 0.65m at BH1). No records of the placement or compaction of this material have been provided, and it is not normal engineering practice for controlled fill to place fill over existing topsoil; therefore, it has been assessed to be uncontrolled fill for the purpose of this assessment.

Based upon the borehole logs and site observations it was inferred that depths of fill are generally greater to the south of the existing "D Block" structure, i.e. at and near BH1, and are likely to reduce at the northern end of the existing structure. Fill is expected to be encountered to the north of the existing structure in the existing car park area.

Despite the fill having been in place for a number of years during which it is likely to have undergone the majority of potential settlement, there may be some layers/zones which are insufficiently compacted. It is also noted that a layer of buried topsoil material was underlying the fill, which would be considered unsuitable for support of structural foundations. Additional loading at the site may increase the magnitude of potential settlement, or the fill may be subject to some ongoing settlement which is independent of any additional loading.

There is a risk that poor sections within the fill may exist that could lead to unplanned settlement or displacement. Therefore, for low risk engineering construction, it is recommended that settlement sensitive structures be founded beneath the fill.

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

A characteristic free surface movement in the range 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.

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.

Final site classification will be dependent on a number of factors, including depth of topsoil, depth of fill and residual soil, reactivity of the natural soil and any fill material placed, and the level of supervision carried out. Re-classification of site areas should be confirmed by the geotechnical authority at the time of construction following any site re-grade works.

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.
- 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.
- 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.

5.2 Foundations

Footings should be founded in suitable material beneath all uncontrolled fill, or the fill should be removed and replaced under engineering supervision.

5.2.1 Shallow Footings

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

Shallow footings founded on stiff or better residual clay, 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.

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.

5.2.2 Deep Footings

Table 6 presents a summary of founding parameters for deep footings (founding depth greater than 3 times maximum footing width) that have been adopted for the relevant materials.

TABLE 6 – SUMMARY OF DEEP FOOTING DESIGN PARAMETERS

Soil Description	Serviceability (Allowable) End Bearing Capacity (kPa)	Serviceability (Allowable) Shaft Adhesion (kPa)	Serviceability (Allowable) Lateral Bearing Capacity (kPa)	Ultimate End Bearing Capacity (kPa)	Ultimate Shaft Adhesion (kPa)
Uncontrolled Fill, Topsoil	-	-	-	-	-
Unit 3 - Colluvium / Unit 4 - Residual Soil (stiff or better)	150	25	75	450	50

Notes:

1) Ultimate values occur at large settlements (>5% of minimum footing dimensions);
2) The ultimate pile parameters presented should be used in limit state pile design in accordance with Australian Standard AS 2159-2009, '*Piling – Design and Installation*';
3) 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;
4) 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;
5) Piles should be no closer than 2.5 pile diameters apart; and,
6) It is expected that the settlement of deep footings proportioned as recommended above should be less than about 1% of the effective pile diameter.

Foundation design parameters should be confirmed by the geotechnical authority at the time of construction.

Greater capacities may be achieved in weathered rock if encountered; however, rock was not observed within the maximum depth of investigations (1.95m at BH01) carried out during this assessment.

The base of piles should be cleaned using a suitable bucket to remove spoil, as open flight augers usually cannot remove sufficient spoil to expose the majority of the pile base.

A suitably experienced geotechnical engineer should inspect the pile excavations prior to pouring concrete.

5.2.3 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.

Unit 3 or Unit 4 Stiff or better clay material 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 7.

TABLE 7 – 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	-	-	-	-	-	-	-
3,4	Colluvium / Residual Soil – Stiff or better Clay	19	50	5	27	10	7.5	0.4

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

The values provided in Table 7 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. E.g. it is recommended that the design does not allow for any geotechnical strength for any Topsoil & Uncontrolled Fill, if present; however, these materials should be allowed for (where present) when assessing loads.

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

5.3 Pavement Design

5.3.1 Design Subgrade CBR Values

Subgrade CBR test results from the investigations at the site returned results of 5% and 8%.

Based on the results of the field work, laboratory testing, and previous experience in the surrounding area, the following design California Bearing Ratio (CBR) values have been adopted for pavement thickness design for the proposed carpark pavements.

TABLE 8 – DESIGN SUBGRADE CBR VALUES

Design Subgrade	Design CBR (%)
Residual Soil	5.0
Lower CBR Material (if encountered)	3.0

Notes:

1) Design subgrade CBR values should be confirmed at the time of construction by the geotechnical authority for each relevant road section.
2) 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.

Based upon the test results from the site, and previous experience, is anticipated that:

- **Design subgrade CBR of 5% may apply to the majority of proposed car park and access driveway pavement**;
- Design subgrade CBR of 3% may apply to some pavement sections if lower CBR material is encountered or imported to the site during construction;
- Localised areas may be affected by uncontrolled fill, buried topsoil or poor subgrade which should be removed, and replaced if necessary, prior to pavement construction.

Subgrade should be prepared in accordance with the site preparation requirements presented in Section 5.5.

5.3.2 Design Traffic Loadings

The proposed development is understood to be a private facility with internal pavement areas including access driveways and car parking spaces with no large commercial or heavy vehicle traffic.

In the absence of detailed traffic data for the internal site pavement areas, design traffic in terms of equivalent standard axles (ESA's) has been provided based on the proposed use of the site for a private carpark with access driveway, as shown in Table 9.

TABLE 9 – PAVEMENT DESIGN TRAFFIC LOADING

Road Section	Equivalent Classification Maitland City Council (MCC)	Design Traffic (ESA's)
Car park and Access Driveways	Local - Access	1×10^5

Car park areas estimated traffic of 1×10^5 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.

Rigid pavement design to Austroads is based on Design Traffic NDT in terms of Heavy Vehicle Axle Groups (HVAG). Based on an adopted ESA per Heavy Vehicle Axle Group of 0.3, the design traffic for internal pavements is assessed to be as shown in Table 10.

TABLE 10 – RIGID PAVEMENT DESIGN TRAFFIC LOADING

Road Section	Equivalent Classification Maitland City Council (MCC)	Heavy Vehicle Axle Groups (HVAG)
Car park and Access Driveways	Local - Access	3.33×10^5

If the pavements are expected to be trafficked by more heavy vehicles than described above, then a higher design traffic loading should be adopted.

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

5.3.3 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";
- MCC – Manual of Engineering Standards 2014 – Pavement Design;
- ARRB Special Report No 41;
- APRG Report No 21.

Flexible Pavement Thickness Designs are presented in Table 11.

Pavement Material Specification and Compaction Requirements are presented in Table 12.

Select fill or bridging layer should be allowed for beneath the pavement in any areas where poor, wet or saturated subgrade conditions are encountered. This is discussed in Section 5.5.

If rock subgrade materials are encountered, the rock should be ripped and re-compacted for a minimum depth of 300mm to break-up preferential drainage paths and provide a dense homogenous surface on which to construct the pavement.

Any areas of uncontrolled fill should be replaced as controlled fill in accordance with AS3798-2007 prior to pavement construction.

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 11 – FLEXIBLE PAVEMENT THICKNESS DESIGN SUMMARY - CAR PARK (LOCAL - ACCESS)

Road Classification	Car Park / Local – Access	
Design Traffic Loading (ESA's)	1×10^5	
Design Subgrade CBR (%)	3.0	**5.0**
Wearing Course (mm)	30 AC10	**30 AC10**
Base Course (mm)	120	**120**
Subbase (mm)	240	**180**
Select Fill (mm) *	-	**-**
Total Thickness (mm)	390	**330**

Notes:

1) A 10mm primer seal should be placed over the base course prior to placement of the asphaltic concrete wearing course.
2) An allowance for additional subgrade replacement should be anticipated in any areas where poor, wet or saturated subgrade conditions are encountered. The requirement for, and depth and extent of any subgrade replacement / select filling, should be confirmed by the geotechnical authority at the time of construction.
3) Prior to pavement construction, the exposed subgrade should be assessed by the geotechnical authority to confirm the pavement thickness requirement for that section.
4) Due to construction practicalities when tying in with depth of kerb and gutter construction, the basecourse layer depth may be increased from 120mm to 150mm. The subbase thickness may be reduced accordingly, by up to 30mm, provided that this does not result in a minimum subbase thickness of less than 125mm.

TABLE 12 – PAVEMENT MATERIAL SPECIFICATION AND COMPACTION REQUIREMENTS

Pavement Course	Material Specification	Compaction Requirements
Wearing Course (AC)	Maitland City Council Spec.	Maitland City Council Spec.
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 / Stabilised Subgrade	Select Fill, CBR ≥ 15%, PI ≤ 15%, max particle size 75mm Or 2% cement stabilised subbase material 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 = Design CBR	100% Standard (AS1289 5.1.1)
Subgrade / Fill Below	Minimum CBR = Design CBR	95% Standard (AS1289 5.1.1)

Notes:

1) Pavement materials for base course and subbase shall also comply with Maitland City Council (MCC) Manual of Engineering Standards Appendix D – Pavement Material Properties

2) CBR = California Bearing Ratio, PI = Plasticity Index.

3) Select Fill / Stabilised Subgrade option adopted will be dependent on subgrade moisture conditions.

5.3.4 Rigid (Concrete) Pavement Thickness Design

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

- Austroads, "Guide to Pavement Technology, Part 2: Pavement Structural Design" (AGPT02-12);
- MCC – Manual of Engineering Standards 2014 – Pavement Design;

Rigid Pavement Thickness Design is presented in Table 13.

Rigid Pavement Material Specification and Compaction Requirements are presented in Table 14.

The depths of base and subbase shown in Table 11 to Table 14 are based upon Austroads assuming the pavements are a private development rather than a public road dedication. The minimum thickness of Concrete Base (170mm minimum) and Sub-base (150mm minimum) specified in the MCC – Manual of Engineering Standards should be adopted if required by Council.

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.

If rock subgrade materials are encountered, the rock should be ripped and re-compacted for a minimum depth of 300mm to break-up any preferential drainage paths and provide a dense homogenous surface on which to construct the pavement.

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 lightly trafficked areas with less than 1×10^6 HVAG 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:

Options have been provided for bound and unbound sub-base beneath the new concrete base layer for lightly trafficked carpark and access driveway pavements. It would be preferred to have a bound sub-base in all areas; however, unbound material is accepted to generally be sufficient for lightly trafficked areas (less than about 1×10^6 HVAG).

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 loading 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 Lean Concrete Sub-base (LCS) or bound Subbase provided that the base and subbase thicknesses specified in the Pavement Thickness Design Tables are achieved.

TABLE 13 – RIGID PAVEMENT THICKNESS DESIGN SUMMARY – CAR PARK (LOCAL - ACCESS)

Road Section	Car Park / Access Driveway							
Design Traffic Loading (HVAG)	3.33×10^5							
Sub-base Option	Unbound		Bound		**Unbound**		**Bound**	
With Shoulder or No Shoulder	With	No	With	No	**With**	**No**	**With**	**No**
Design Subgrade CBR (%)	3	3	3	3	**5**	**5**	**5**	**5**
Concrete Base (mm)	165	195	155	180	**160**	**185**	**150**	**170**
Sub-base (mm)	125	125	125 bound	125 bound	**125**	**125**	**125 bound**	**125 bound**
Total Thickness (mm)	290	320	280	305	**285**	**310**	**275**	**295**

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) Where design is based on the option of "with" shoulder, the concrete shoulder must be either integral or structural in accordance with the requirements of Austroads.
3) 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.
4) If CBR < 3.0% subgrade is encountered, thicknesses for Design Subgrade CBR of 5% may be used over a 250mm select fill bridging layer.

TABLE 14 – 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 = 32MPa reinforced with suitable steel mesh, or steel fibre reinforcement.	
Lean Concrete Sub-base (LCS)	Concrete with minimum characteristic compressive strength, fc = 5 MPa (with fly ash) or 7MPa (without fly ash).	
Bound Sub-base	Conforming to ARRB SR41, CBR > 30%, PI < 12%, bound with 5% cementitious binder	95% Modified (AS1289 5.2.1)
Select Fill	2% cement stabilised subbase material, or; CBR ≥ 15%, PI ≤ 15%, max particle size 75mmOr ; 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 = 3%	100% Standard (AS1289 5.1.1)
Subgrade / Fill Below	Minimum CBR = 3%	95% Standard (AS1289 5.1.1)

5.3.5 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.

5.4 Excavation Conditions

The depths of fill, topsoil, colluvium, residual soils, and weathered rock, together with depths of practical refusal of the hand auger where encountered are summarised in Table 2.

In terms of excavation conditions, site materials can generally be divided into:

- Clayey and Granular Soils (Units 1, 2, 3, & 4). It is anticipated that these materials could be excavated by a conventional excavator or backhoe bucket;
- Weathered Rock (not encountered during current investigation). Rippability is dependent on rock strength, depth, degree of weathering and number of defects within the rock mass which can vary significantly.

Weathered Rock (Unit 5) materials were not encountered within the depths of investigation at BH1 to BH3 (1.95m, 1.20m, and 1.20m respectively). It is assessed that it is unlikely to encounter these materials during the proposed development, and site subsoils will be generally excavatable by conventional excavator or backhoe equipped with toothed bucket to at least the depths reached in the appended borehole logs.

It is recommended that targeted investigations are carried out if significant excavations are proposed where bedrock depth or excavatability is important to design or construction.

Groundwater may exist at localised areas of the site such as within the topsoil / colluvium profile, from water perched above the residual clay / bedrock profile, or in areas of former drainage channels. 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 encountered, in most cases shallow groundwater is generally expected to be manageable by de-watering by sump and pump methods.

Excavations should be supported by properly designed and constructed retaining walls or else battered at 1V:2H or flatter and protected from erosion.

Temporary excavations should be battered at 1V:1H or flatter in cohesive soils, or 1V:1.5H or flatter in granular soils, and protected from erosion. Steeper excavations may be supported by means of temporary shoring.

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.

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

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

5.5 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 pavement construction or site re-grading should be stripped to remove all existing vegetation, topsoil, root affected or other potentially deleterious materials;
- Stripping is generally expected to be required to depths of about 0.3m to remove topsoil and root affected material from areas not affected by filling.
- Stripping of greater depths of fill material in addition to topsoil and root affected material is anticipated in areas affected by surface filling (e.g. Fill and Topsoil to a depth of 0.65m at BH1);

- Additional stripping may be required in any areas where poor, wet or saturated subgrade conditions are encountered. Excavation of over-wet sandy material may be required in areas of deep colluvium/slopewash prior to placement of fill depending upon conditions encountered at the time of construction;
- 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 clean sand or 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;
- Subgrade preparation should be carried out using a tracked excavator equipped with a smooth sided ('gummy') bucket to minimise the risk of over-disturbance of soils;
- Protect the area after subgrade preparation to maintain moisture content as far as practicable. The placement of subbase gravel would normally provide adequate protection; and,
- Site preparation should include provision of drainage and erosion control as required, as well as sedimentation control measures.

At the time of the field investigations, moisture content for the two samples of subgrade material tested varied from 7.0% dry to 1.2% wet of standard Optimum Moisture Content (OMC). It should be anticipated that moisture conditioning of the subgrade may be necessary prior to compaction and placement of pavement 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 materials are encountered at subgrade level, these materials should be over-excavated and be replaced with a minimum depth of 300mm 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.

5.6 Fill Construction Procedures

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

- Approved fill beneath structures and pavements should be compacted in layers not exceeding 300mm loose thickness;
- Approved fill beneath pavements should be compacted to the compaction requirements provided in Table 11;
- The top 300mm of natural subgrade below pavements or the final 300mm of road subgrade fill should be compacted to provide a subgrade that is within the moisture range of 60% to 90% of Optimum Moisture Content (OMC);
- Site fill beneath structures should be compacted to a minimum density ratio of 98% Standard Compaction within ±2% of OMC in cohesive 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; and,
- Earthworks should be carried out in accordance with the recommendations outlined in AS3798-2007 '*Guidelines for Earthworks for Commercial and Residential Developments*'.

5.7 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:

- Units 1A Fill-Topsoil and Unit 2 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. These materials may require some moisture conditioning sorting and/or blending. Suitability for re-use should be confirmed prior to, or at the time of construction;
- Unit 3 – Colluvium soils are generally expected to be suitable for re-use as general fill for engineering purposes; and,
- Unit 4 – Residual Soils are generally expected to be suitable for re-use as general fill for engineering purposes.

Final selection of fill materials should consider properties such as reactivity which is likely to be low to moderate for site won Unit 3 – Colluvium soils, and moderate to high for site won Unit 4 – Residual Soils.

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

5.8 Special Requirements for Construction Procedures and Drainage

Adequate surface and subsurface drainage should be provided to the pavement and adjacent areas in accordance with Council specifications.

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 Maitland City Council (MCC) specifications.

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

6.0 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.

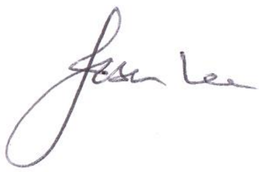
The extent of testing associated with this assessment is limited to discrete test locations. It should be noted that subsurface conditions between and away from the test locations may be different to those observed during the field work and used as the basis of the recommendations contained in this report.

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.

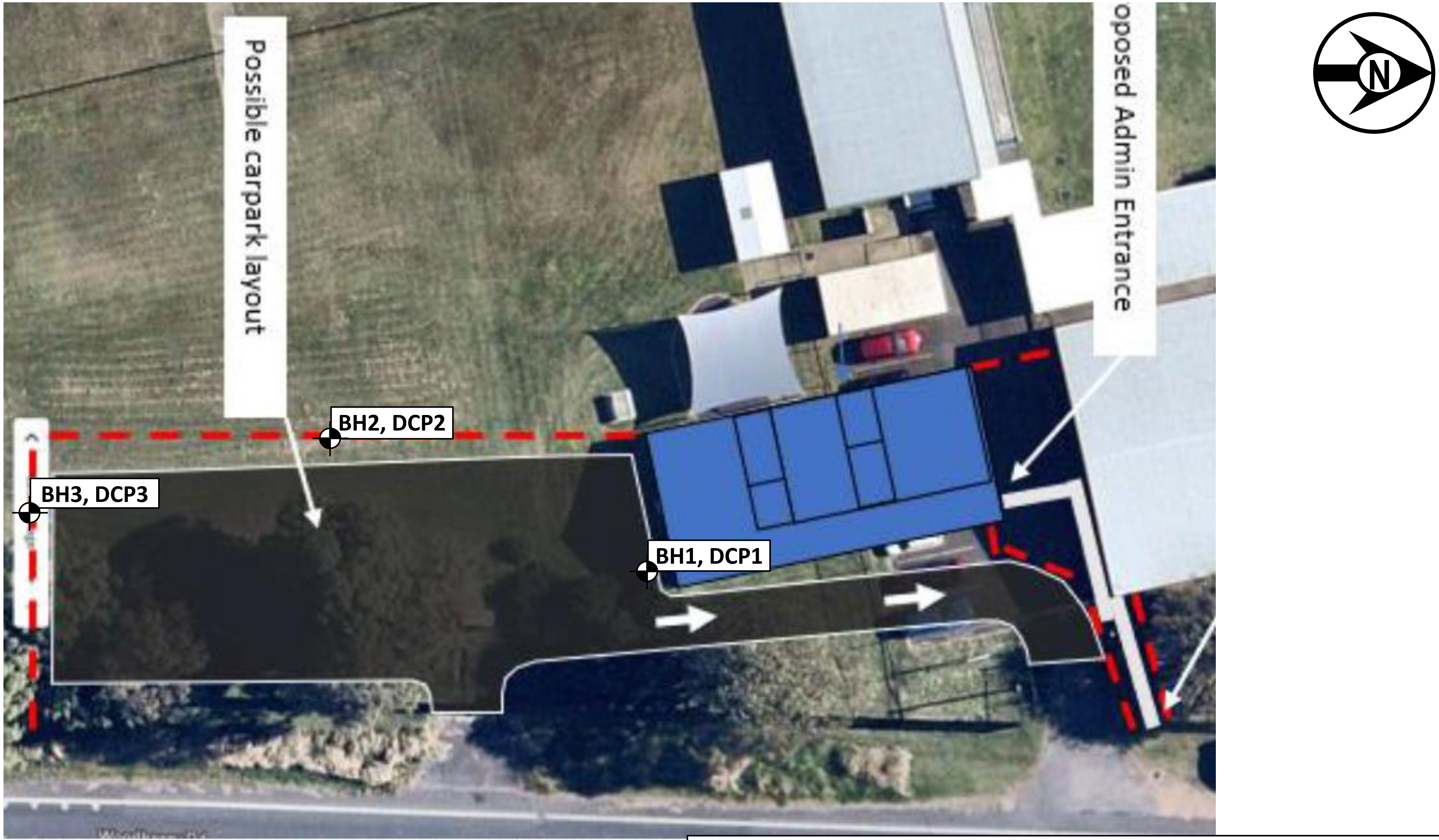
For and on behalf of Qualtest Laboratory (NSW) Pty Ltd.



Jason Lee
Principal Geotechnical Engineer

FIGURE AA1:

Site Plan and Approximate Test Locations



Based on site plan provided in School Infrastructure NSW request for tender
(RFT No. SINSW-21-10605, dated July 2021)

LEGEND:

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

Qualtest LABORATORY (NSW) PTY LTD

Client:	METIRI CONSULTANTS PTY LTD	Drawing No:	FIGURE AA1
Project:	PROPOSED WOODBERRY LEARNING CENTRE CARPARK	Project No:	NEW21P-0138
Location:	WOODBERRY ROAD, WOODBERRY	Scale:	N.T.S.
Title:	SITE PLAN AND APPROXIMATE TEST LOCATIONS	Date:	26/10/2021

APPENDIX A:

Results of Field Investigations

ENGINEERING LOG - HAND AUGER

Qualtest LABORATORY (NSW) PTY LTD

CLIENT:	METIRI CONSULTANTS PTY LTD	**HAND AUGER NO:**	**BH1**
PROJECT:	WOODBERRY LEARNING CENTRE CARPARK	**PAGE:**	1 OF 1
LOCATION:	WOODBERRY ROAD, WOODBERRY	**JOB NO:**	NEW21P-0138
		LOGGED BY:	BB
		DATE:	9/9/21

DRILL TYPE:	HAND AUGER	**SURFACE RL:**	6.2 m
BOREHOLE DIAMETER:	100 mm	**DATUM:**	AHD

Drilling and Sampling: METHOD: HA; WATER: Not Encountered; SAMPLES: U50 (0.80m – 0.95m)

RL (m): 6.0 (0.2 m depth), 5.5 (0.7 m depth), 5.0 (1.2 m depth), 4.5 (1.7 m depth)

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 – 0.10m	CL	FILL-TOPSOIL: Sandy CLAY - low to medium plasticity, brown to dark brown, fine to medium grained sand, root affected.	M ~ W_P				FILL - TOPSOIL
0.10 – 0.30m	CH	FILL: CLAY - medium to high plasticity, brown red-brown and pale orange-brown, with some fine to medium grained sand.	M < W_P - M ~ W_P		HP	300	FILL
0.30 – 0.50m	CH	FILL: Sandy CLAY - medium to high plasticity, brown red-brown and pale orange-brown, fine to medium grained sand.	M < W_P - M ~ W_P		HP	250	
0.50 – 0.65m	CL	TOPSOIL: Sandy CLAY - low plasticity, grey-brown, fine to medium grained sand, with some rootlets.	M < W_P				BURIED TOPSOIL
0.65 – 0.75m	CL	Sandy CLAY - low to medium plasticity, grey-brown to pale brown, fine to medium grained sand.	M < W_P	VSt			COLLUVIUM
0.75 – 1.95m	CH	CLAY - medium to high plasticity. pale brown and pale orange-brown to red-brown, with some fine to medium grained sand.	M < W_P	H	HP	500	RESIDUAL SOIL
					HP	550	
					HP	>600	
					HP	410	
		Pale orange-brown to red-brown with some pale grey.	M ~ W_P	VSt	HP	350	
		Pale grey and red-brown with some pale orange-brown.			HP	320	

Hole Terminated at 1.95 m

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 - HAND AUGER

Qualtest LABORATORY (NSW) PTY LTD

CLIENT:	METIRI CONSULTANTS PTY LTD	**HAND AUGER NO:**	**BH2**
PROJECT:	WOODBERRY LEARNING CENTRE CARPARK	**PAGE:**	1 OF 1
LOCATION:	WOODBERRY ROAD, WOODBERRY	**JOB NO:**	NEW21P-0138
		LOGGED BY:	BB
		DATE:	9/9/21

DRILL TYPE:	HAND AUGER	**SURFACE RL:**	5.6 m
BOREHOLE DIAMETER:	100 mm	**DATUM:**	AHD

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
HA	Not Encountered		5.5	0.00–0.30m	CL	TOPSOIL: Sandy CLAY - low plasticity, dark brown, fine to medium grained sand, root affected.	M > W_P				TOPSOIL
				0.30–0.55m	CL	Sandy CLAY / Clayey SAND - low plasticity, pale grey-brown, fine to medium grained sand.	M > W_P	St			COLLUVIUM
		0.60m CBR 0.70m	5.0	0.55–0.80m	CI	Sandy CLAY - medium plasticity, pale brown with some red-brown, fine to medium grained sand.	M > W_P	St - VSt	HP	220	RESIDUAL SOIL
			4.5	0.80–1.20m	CH	CLAY - medium to high plasticity, pale grey and red-brown with some pale orange-brown, with some fine to medium grained sand.	M < W_P - M ~ W_P	VSt - H	HP HP	410 380	
			4.0			Hole Terminated at 1.20 m					

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 - HAND AUGER

CLIENT:	METIRI CONSULTANTS PTY LTD	**HAND AUGER NO:**	**BH3**
PROJECT:	WOODBERRY LEARNING CENTRE CARPARK	**PAGE:**	1 OF 1
LOCATION:	WOODBERRY ROAD, WOODBERRY	**JOB NO:**	NEW21P-0138
		LOGGED BY:	BB
		DATE:	9/9/21

DRILL TYPE: HAND AUGER **SURFACE RL:** 5.0 m
BOREHOLE DIAMETER: 100 mm **DATUM:** AHD

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
HA	Not Encountered			0.00–0.30m	CL	TOPSOIL: Sandy CLAY - low plasticity, pale grey-brown, fine to medium grained (mostly fine grained) sand, root affected.	M < W_P				TOPSOIL
		0.45m	4.5	0.30–0.45m	CL	Sandy CLAY - low plasticity, grey-brown, fine to coarse grained (mostly fine grained) sand, trace fine to medium grained sub-rounded to sub-angular gravel, with some roots, weakly cemented.		H	HP	>600	COLLUVIUM
		CBR 0.45m–0.60m		0.45–0.80m	CI	Sandy CLAY - medium plasticity, pale brown to pale orange-brown trace red-brown, fine to medium grained sand.			HP HP	>600 >600	RESIDUAL SOIL
			4.0	0.80–1.20m	CH	CLAY - medium to high plasticity, pale grey and red-brown with some pale orange-brown, with some fine to medium grained sand.			HP HP HP	500 550 580	
				1.20m		Hole Terminated at 1.20 m					

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:	**METIRI CONSULTANTS PTY LTD**	**Project Number:**	*NEW21P-0138*
Principal:		**Sheet No:**	*1 of 1*
Project:	**PROPOSED LEARNING CENTRE CARPARK**	**Test Date:**	9/09/2021
Location:	**WOODBERRY ROAD, WOODBERRY**	**Tested By:**	BB

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

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

Depth Below Surface (mm)	DCP1	DCP2	DCP3						Test Location / Comments
150	4	2	5						DCP locations as per attached Figure AA1.
300	3	1	9						
450	5	3	11						
600	6	6	11						
750	7	9	9						
900	11	15	9						
1050	9	19	7						
1200	8	24	8						
1350	5								
1500	6								
1650	6								
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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Report No: SSI:NEW21W-4110-S01
Issue No: 1

Shrink Swell Index Report

Client: Metiri Consultants Pty Ltd
5/33 The Boulevarde
Toronto NSW 2283

Project No.: NEW21P-0138
Project Name: Woodberry Learning Centre Carpark

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.

Approved Signatory: Brent Cullen
(Senior Geotechnician)
NATA Accredited Laboratory Number: 18686
Date of Issue: 24/09/2021

Sample Details

Sample ID:	NEW21W-4110-S01		
Sampling Method:	The results outlined below apply to the sample as received		
Material:	Insitu	**Date Sampled:**	9/09/2021
Source:	On-Site	**Date Submitted:**	9/09/2021
Specification:	No Specification		
Project Location:	Woodberry Road, Woodberry		
Sample Location:	BH1 - (0.8 - 0.95m)		
Date Tested:	17/09/2021		

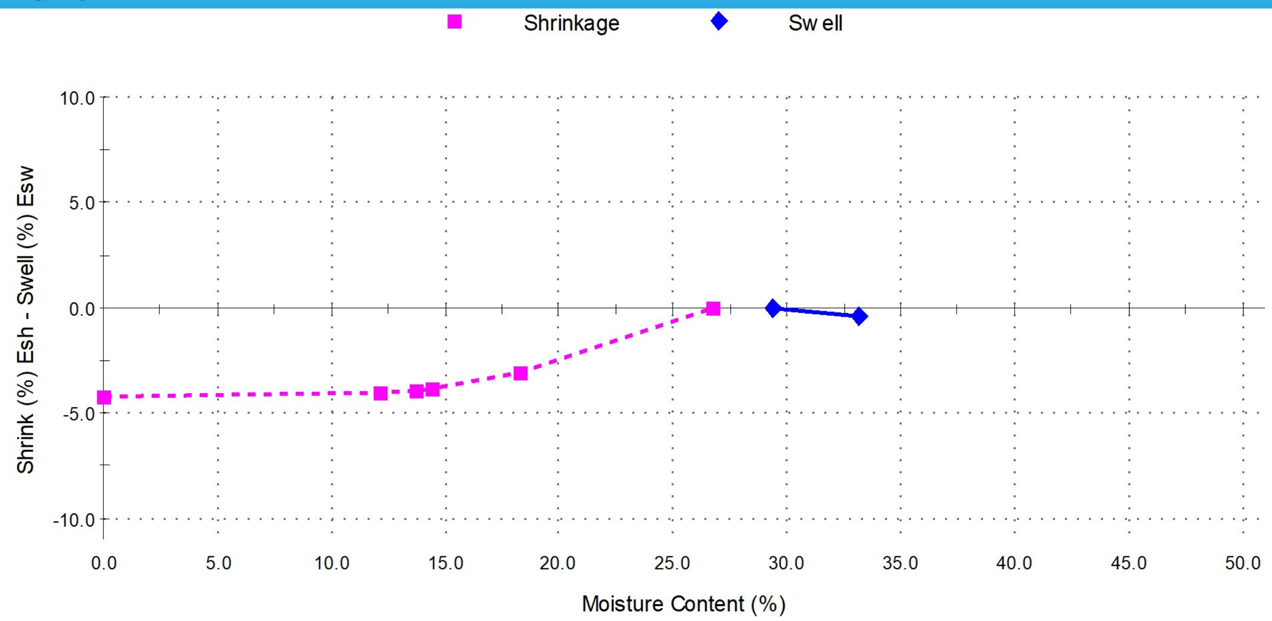
Swell Test — AS 1289.7.1.1

Swell on Saturation (%):	-0.4
Moisture Content before (%):	29.4
Moisture Content after (%):	33.1
Est. Unc. Comp. Strength before (kPa):	>600
Est. Unc. Comp. Strength after (kPa):	550

Shrink Test — AS 1289.7.1.1

Shrink on drying (%):	4.2
Shrinkage Moisture Content (%):	26.7
Est. inert material (%):	1%
Crumbling during shrinkage:	Nil
Cracking during shrinkage:	Nil

Shrink Swell



Shrink Swell Index - Iss (%): 2.3

Comments

QUALTEST Laboratory (NSW) Pty Ltd (20708)
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Report No: CBR:NEW21W-4110-S02
Issue No: 1

California Bearing Ratio Test Report

Client:	Metiri Consultants Pty Ltd 5/33 The Boulevarde Toronto NSW 2283
Project No.:	NEW21P-0138
Project Name:	Woodberry Learning Centre Carpark

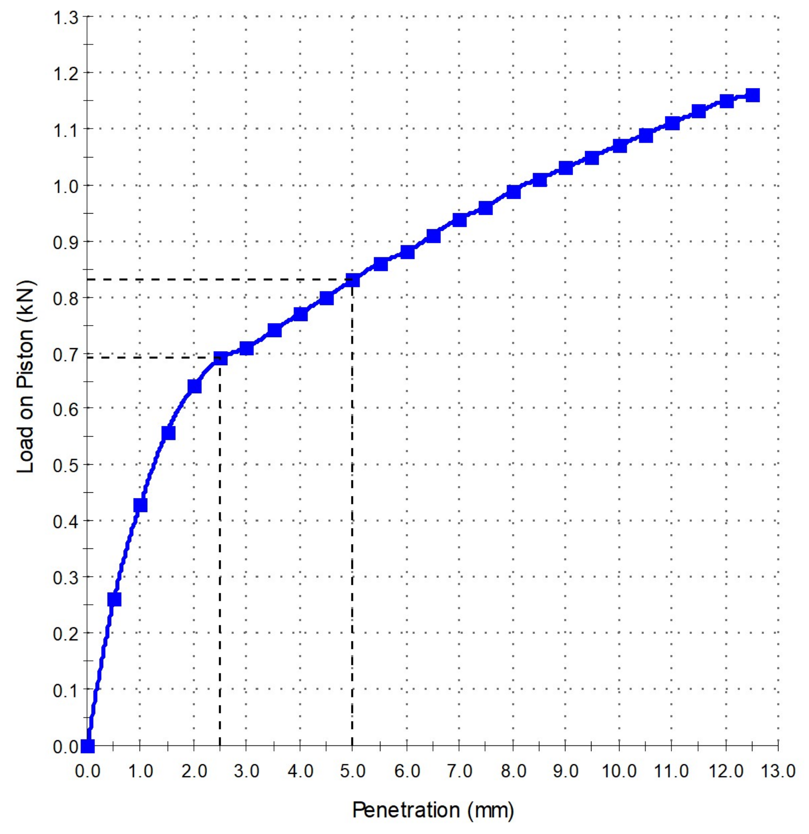
Accredited for compliance with ISO/IEC 17025-Testing.
The results of the tests, calibrations and/or measurements included in this document are traceable to Australian/national standards.
Results provided relate only to the items tested or sampled.

Approved Signatory: Brent Cullen
(Senior Geotechnician)
NATA Accredited Laboratory Number: 18686
Date of Issue: 23/09/2021

Sample Details

Sample ID:	NEW21W-4110-S02	**Date Sampled:**	9/09/2021
Sampling Method:	The results outlined below apply to the sample as received		
Specification:	No Specification	**Source:**	On-Site
Location:	BH2 - (0.6 - 0.7m)	**Material:**	Insitu
Project Location:	Woodberry Road, Woodberry	**Date Tested:**	17/09/2021

Load vs Penetration



Test Results

AS 1289.6.1.1

CBR at 2.5mm (%):	**5**
Maximum Dry Density(t/m³):	1.53
Optimum Moisture Content(%):	26.2
Dry Density before Soaking (t/m³):	1.53
Density Ratio before Soaking (%):	100.5
Moisture Content before Soaking (%):	26.0
Moisture Ratio before Soaking (%):	99.0
Dry Density after Soaking (t/m³):	1.53
Density Ratio after Soaking (%):	100.0
Swell (%):	0.5
Moisture Content of Top 30mm (%):	26.4
Moisture Content of Remaining Depth (%):	22.1
Compaction Hammer Used:	Standard AS 1289.5.1.1
Surcharge Mass (kg):	9.00
Period of Soaking (Days):	4
Retained on 19 mm Sieve (%):	0
CBR Moisture Content Method:	AS 1289.2.1.1
Sample Curing Time (h):	72
Plasticity Determination Method:	Visual/Tactile

——— AS1289.2.1.1 ———

In Situ (Field) Moisture Content (%):	27.4

Comments

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California Bearing Ratio Test Report

Report No: CBR:NEW21W-4110-S03
Issue No: 1

Client:	Metiri Consultants Pty Ltd 5/33 The Boulevarde Toronto NSW 2283
Project No.:	NEW21P-0138
Project Name:	Woodberry Learning Centre Carpark

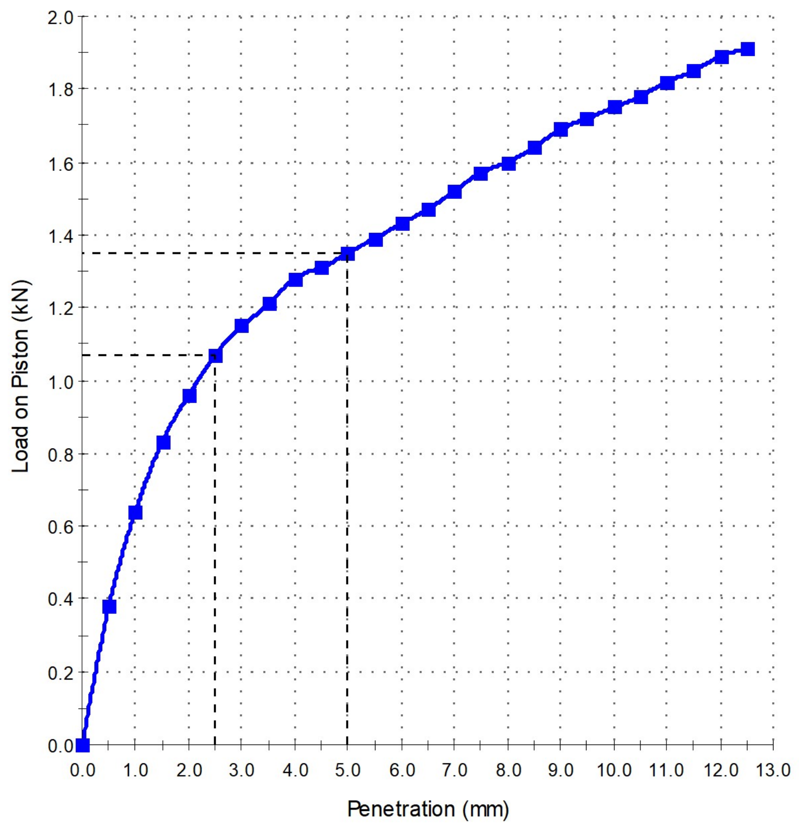
Accredited for compliance with ISO/IEC 17025-Testing.
The results of the tests, calibrations and/or measurements included in this document are traceable to Australian/national standards.
Results provided relate only to the items tested or sampled.

Approved Signatory: Brent Cullen
(Senior Geotechnician)
NATA Accredited Laboratory Number: 18686
Date of Issue: 23/09/2021

Sample Details

Sample ID:	NEW21W-4110-S03	**Date Sampled:**	9/09/2021
Sampling Method:	The results outlined below apply to the sample as received		
Specification:	No Specification	**Source:**	On-Site
Location:	BH3 - (0.45 - 0.6m)	**Material:**	Insitu
Project Location:	Woodberry Road, Woodberry	**Date Tested:**	17/09/2021

Load vs Penetration



Test Results

AS 1289.6.1.1

CBR at 2.5mm (%):	**8**
Maximum Dry Density(t/m³):	1.61
Optimum Moisture Content(%):	20.7
Dry Density before Soaking (t/m³):	1.61
Density Ratio before Soaking (%):	100.0
Moisture Content before Soaking (%):	20.8
Moisture Ratio before Soaking (%):	100.5
Dry Density after Soaking (t/m³):	1.60
Density Ratio after Soaking (%):	99.5
Swell (%):	0.5
Moisture Content of Top 30mm (%):	22.9
Moisture Content of Remaining Depth (%):	19.9
Compaction Hammer Used:	Standard AS 1289.5.1.1
Surcharge Mass (kg):	9.00
Period of Soaking (Days):	4
Retained on 19 mm Sieve (%):	0
CBR Moisture Content Method:	AS 1289.2.1.1
Sample Curing Time (h):	96
Plasticity Determination Method:	Visual/Tactile

——— AS1289.2.1.1 ———

In Situ (Field) Moisture Content (%):	13.7

Comments

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).

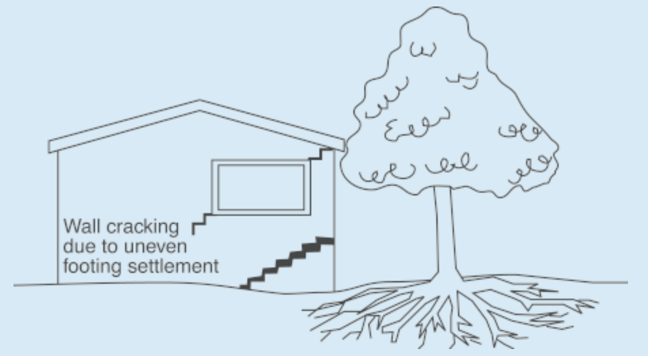
Isolated piers affected by erosion or saturation of foundations will eventually lose contact with the bearers they support and may tilt or fall over. The floors that have lost this support will become bouncy, sometimes rattling ornaments etc.

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.

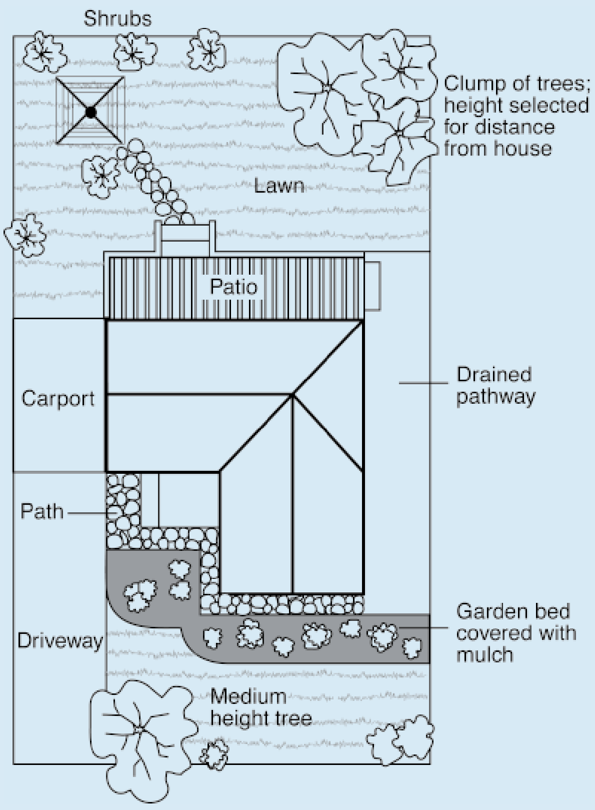
Protection of the building perimeter
It is essential to remember that the soil that affects footings extends well beyond the actual building line. Watering of garden plants, shrubs and trees causes some of the most serious water problems.

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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