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Report on
Geotechnical Investigation

Proposed Aged Care Development
Scaysbrook Drive, Kincumber

Prepared for
Lend Lease Building Pty Ltd

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Integrated Practical Solutions

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The undersigned, on behalf of Douglas Partners Pty Ltd, confirm that this document and all attached drawings, logs and test results have been checked and reviewed for errors, omissions and inaccuracies.

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1. Introduction

This report presents the results of a geotechnical investigation undertaken for a proposed aged care development at Scaysbrook Drive, Kincumber. The investigation was commissioned in an email dated 13 October 2016 by Numa Miller of Lend Lease Building Pty Ltd and was undertaken in accordance with Douglas Partners' proposal NCL160534.P.001.Rev1 dated 18 October 2016.

It is understood that the proposed aged care facility development of the site will include the removal of existing buildings, cut and fill to re-grade the site and new two storey buildings and associated car parking. A geotechnical investigation was undertaken to provide prospective civil contractors with information about the subsurface conditions at the site.

The aim of the investigation was to provide information and comments on the following:

- Subsurface conditions, including depth of fill, depth to groundwater and depth to bedrock;
- Excavation conditions;
- Site classification in accordance with AS 2870 – 2011;
- Shallow footing options and design parameters, including allowable bearing capacities and estimated settlements;
- Internal driveway and car park pavement thickness design;
- Retaining wall parameters;
- Geotechnical suitability of materials for re-use; and
- Earthworks preparation measures.

The investigation included the drilling of eight boreholes and laboratory testing of selected samples. The details of the field work are presented in this report, together with comments and recommendations on the issues listed above.

2. Site Description and Regional Geology

The site is located on the northern side of Scaysbrook Drive, Kincumber, within a former aged care facility. The disused Wayne Williams Crescent runs through the centre of site. The site is bounded by Avoca Drive to the north, grassed properties to the east, Scaysbrook Drive to the south and a residential property / retirement village to the west. The site is shown below in the following Figure 1.

Figure 1: Site (Image Google 2016)

The site is presently occupied by numerous dwellings, garages and storage structures which were part of a former aged care facility that is to be demolished to facilitate redevelopment of the site. The dwellings generally consist of single and double storey brick and tile villas / townhouses.

The internal access roads are generally constructed using concrete pavers, with some concrete pavement sections in parts of the site. A dirt access track runs along the eastern boundary of the site.

Medium to large sized trees were observed in the north eastern part of the site, the eastern boundary of site and at a few other scattered locations.

Photos of site are shown in the following Figures 2 and 3.

Figure 2: Facing south from near Bore 2, showing existing villas and Wayne Williams Crescent

Figure 3: Facing south from near the north western boundary of site, showing large trees

The site slopes down generally towards the south west, with some of the dwellings within the western area of the site located on terraced areas with have been formed by cutting into the hillside. The terraced areas are stepped in multiple levels with retaining walls up to approximately 2 m in height.

An existing block retaining wall up to approximately 3 m in height is located along the western boundary of site. The retaining wall is stepped in multiple levels down towards the south. Residential buildings are located within several metres of the alignment of this wall.

Some of the site slopes are shown in the following Figures 4 and 5.

Figure 4: Facing west from near Bore 2, showing site slope and retaining walls

Figure 5: Facing south from near Bore 1 along western boundary, showing existing retaining walls

Cracking was observed in the brickwork in some of the existing buildings on the western side of the site, particularity around window edges and within the lower courses of brickwork. Articulation joints were not observed on the damaged buildings at these locations.

Clay soils were observed within cuttings observed beneath buildings in the western part of the site, at locations shown as Observation 1 and 2 on the Test Location Plan in Appendix D. Tension cracks with an aperture of about 15 mm were observed within the clay soils of the cutting at Observation 1, possibly indicative of reactive soils. A slip failure of the cutting was observed in the clay soils at Observation 2.

Figures 6 to 8 show the cracking observed in an existing building and cuttings as described above.

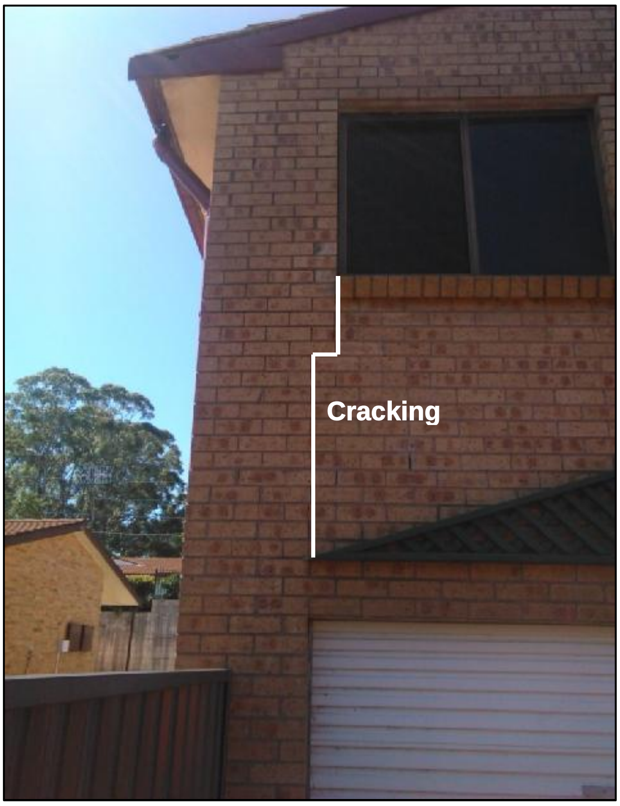


Figure 6: Facing north from near Bore 8, showing cracking in existing building beneath window edge

Figure 7: Facing north beneath building near Bore 8 (Observation 1) showing soil cracking

Figure 8: Facing south east at slip failure beneath building in western part of site (Observation 2)

Reference to the Gosford / Lake Macquarie 1:100,000 Geology Sheet indicates that the site is underlain by the Terrigal Formation of the Narrabeen Group, comprising interbedded laminite, shale and quartz to lithic-quartz sandstone, with possible presence of minor red claystone.

3. Field Work Methods

Field work for the investigation was undertaken on the 24 and 25 October 2016 and comprised the following:

- A site walkover by geotechnical engineer to set out test locations in areas accessible and free from buried services;
- Drilling of eight bores (Bores 1 to 8) using a purpose built geotechnical drilling rig. The bores were drilled to depths ranging from 1.2 m to 7.5 m depth, using solid flight auger and or NMLC rock coring methods;
- Dynamic cone penetrometer (DCP) tests were carried out at selected locations to depths of up to 1.05 m to allow an assessment of the strength of near surface soils;
- The subsurface conditions were logged on site by an geotechnical engineer, who also recovered representative samples for laboratory testing;

- Point load testing was undertaken on recovered rock samples and the results are presented on the borehole logs in Appendix B;
- Each test location position was recorded on site by the DP engineer using a hand held GPS (accuracy of about 10 m). Test location surface levels were interpolated to the nearest 0.5 mAHD from the drawing “Plan Showing Select Features and Levels”, ref 51152DM, Rev0, dated 8/11/2013, supplied by the client.

The test locations are shown on Test Location Plan, Drawing 1, in Appendix D.

4. Field Work Results

The subsurface conditions encountered in the boreholes are presented in the detailed logs in Appendix A, together with notes explaining classification methods and descriptive terms used on the logs. Photographs of recovered core are also included in Appendix B. The results of the DCP tests are presented graphically on the borehole logs.

The subsurface conditions encountered in all the bores can be broadly divided into the geotechnical units summarised as follows in Table 1 below.

Table 1: Summary of Geotechnical Units at Bore Locations

Geotechnical Unit	Soil Type	Description
Unit 1	Filling	Encountered in all bores except Bore 3 and generally comprising pavers overlying gravelly sand and sand filling, with sandy clay/clay filling generally encountered in areas of deep filling (Bores 1, 5, 6 and 8). Filling was encountered up to depths of 2.1m. 50mm of topsoil was encountered in Bore 3.
Unit 2A	Residual Clay – Stiff	Stiff Clay – Encountered in Bores 5 to 7 from depths ranging between 0.1 m and 0.45 m.
Unit 2B	Residual Clay – Very Stiff to Hard	Very Stiff to Hard Clay / Sandy Clay – Encountered in all bores except Bore 6 from depths ranging between 0.05 m and 2.1 m. Generally very stiff and grey mottled orange red with some ironstone gravel/cobbles. Becoming hard with completely weathered rock like properties from 0.8 m in Bore 2 and 6.0 m in Bore 8.
Unit 3A	Weathered Bedrock (ELst to VLst)	Sandstone / Siltstone / Claystone – Encountered in Bores 1 to 5 from depths ranging between 0.7 m and 3.3 m. Some clay bands and high strength iron stained / cobble bands
Unit 3B	Bedrock (Lst or better)	Sandstone / Siltstone – Encountered in Bores 2 and 3 from depths ranging between 2.8 m and 4.8 m. Generally moderately weathered to slightly weathered and fractured to slightly fractured. Also inferred from drilling resistance of auger from depths ranging between 1.6 m and 5.1 m in Bores 1, 4 and 5.

Notes to Table 1:
ELst – Extremely low strength
VLst – Very low strength
Lst – Low strength

It should be noted that the site is spread over a number of existing levels with numerous retaining walls. Filling is anticipated to be present behind such walls and under previously terraced areas. The extent of such filling has not been established during this investigation and should be further assessed.

Slight seepage was observed in Bore 8 at 6.5 m depth whilst drilling. Groundwater was not encountered in the remaining test locations whilst drilling, although drilling fluids prevented groundwater observations below 4.5m and 2.8 m depth in Bores 2 and 3 respectively. It should be noted that groundwater levels are affected by factors such as climatic conditions and soil permeability and will therefore vary with time.

5. Laboratory Testing

Laboratory testing was undertaken on selected samples. Testing was carried out at DP's NATA accredited laboratories. Laboratory testing was undertaken on a selection of samples of soil for the following tests:

- Two Shrink-Swell Index tests;
- Two Standard Compaction / CBR tests.

The detailed results are presented in Appendix C and are further summarised in Table 2 below.

Table 2: Results of Laboratory Testing

Bore	Depth (m)	Description	Geotechnical Unit	FMC (%)	SOMC (%)	SMDD (t/m^3)	CBR (%)	Swell During Soaking (%)	Iss (% per apF)
3	0.5-0.75	Sandy Clay	2B	17.4	-	-	-	-	1.5
5	0.5-0.7	Clay	2A	23.0	-	-	-	-	2.6
6	0.5-0.7	Clay	2A	24.6	20.5	1.66	3.5	2.0	-
7	1.0-1.3	Clay	2B	18.2	17.5	1.78	3.5	2.5	-

Notes to Table 2:

FMC - Field moisture content
SMDD - Standard maximum dry density
Iss - Shrink-Swell Index
SOMC - Standard optimum moisture content
CBR - California bearing ratio (4 day soaked)

6. Proposed Development

It is understood that the proposed aged care facility development of the site will include the removal of existing buildings, cut and fill to re-grade the site, new two storey buildings and a 47 space carpark with associated access road.

The proposed development is shown within a drawing provided by the client and is included in Drawing 2 – Proposed Development in Appendix D.

The final development levels (i.e. cut / fill profile) have not been determined at this stage.

It is understood that cut and fill across the site may typically be in the order of 1 m, to re-grade building platforms. Deeper excavation, in the order of 2 to 3 m depth may be required in the northern part of the site. A cut of up to 6 m in height is proposed at the north western part of the site as part of the proposed car park, which will be supported by an engineer designed retaining wall.

Further, an existing retaining wall of up to approximately 3 m in height, running along the western boundary of the site is to be supported as part of the carpark excavation works. It is noted that residential houses are located near the crest of this retaining wall.

An access road and roundabout is to be constructed along the south western boundary. This access road is to link up with the proposed 47 car space carpark.

7. Comments

7.1 Excavation Conditions

It is understood that excavation for re-grading is typically up to 1 m however, around the proposed car park excavations in the order of 6 m depth are proposed. Excavation up to 3 m depth could be undertaken in the northern part of the site.

Based on the results of the investigation, it is considered that excavation of the Unit 1 to Unit 3A material is expected to be generally achievable using conventional machinery such as a backhoe, hydraulic excavator or elevated scraper. Higher strength ironstained bands / cobbles within the Unit 2B or 3A material may require excavation with a large excavator (at least 20 tonne or 30 tonne) fitted with rock hammer and / or ripping attachments.

High strength sandstone and medium strength siltstone (Unit 3B) was encountered from 4.80 m depth in Bore 2, in the vicinity of the 6 m car park cut. Unit 3B rock was also encountered from 2.80 m depth in Bore 3. Coring of the bedrock indicated that the rock was fractured to slightly fractured.

Increased drilling resistance / slow progress with an auger fitted with a tungsten carbide (TC) drill bit indicated Unit 3B rock from approximately 5.1 m depth in Bore 1, behind the proposed 6 m high car park cut. Increased auger drilling resistance / slow progress with a TC bit was also encountered from 1.6 m and 2.9 m in Bores 4 and 5, indicating Unit 3B rock.

The increased drilling resistance / slow progress with the auger TC bit is likely to correspond to depths at which a 20 tonne excavator may encounter slow production rates and or refusal, within Unit 3B rock (low strength or stronger rock). Where Unit 3B rock is encountered it is anticipated that a large excavator (at least 20 tonne or 30 tonne) fitted with rock hammer and or ripping attachments would be required to excavate, although slow production rates may occur.

Confined and detailed excavations in Unit 3B rock will likely require the use of excavators fitted with rippers, rock hammers and or rock saws.

It is important to note that excavatability of rock is dependent not only on rock strength, but also on the presence, orientation and extent of discontinuities such as jointing / bedding and fracturing of the rock, the presence of favourable and adverse bedding planes, presence of groundwater and other factors. For example, low strength rock with few discontinuities may be more difficult to excavate than highly fractured, high strength rock.

Contractors should be responsible for selecting excavation equipment based on the proposed excavation depths and equipment capabilities, together with the anticipated conditions.

Temporary excavations should be adequately supported, or battered, however it is expected that batters are unlikely to be feasible due to the proposed height of the retaining wall. Therefore, the proposed construction should include support measures that are installed prior to excavation. This is discussed further in the following Section 7.4 of the report.

7.2 Excavation Vibration

It would be prudent to allow for dilapidation surveys to be carried out on nearby buildings and existing services to document their condition prior to the commencement of all work in order to respond to any spurious claims for damage arising from construction activities.

The use of heavy equipment, rock breaking tools and pneumatic equipment has the potential to affect structures adjoining the proposed excavation.

As a guide, the damage threshold due to vibration is dependent on the quality of the building foundations and construction of the building as well as the wavelength of the vibration and the source distance. The longer the wavelength, the more likely a building is to resonate and suffer damage. For construction equipment (generally in the high frequency or short wavelength range), the damage threshold is 40 mm/sec to 50 mm/sec for buildings founded on rock. Most vibration codes set safe limits for building vibrations at lower levels.

The Standards Australia explosives code recommends the maximum peak particle velocities for various structures subjected to blasting vibration (generally a low frequency vibration) as set out in Table 3 below.

Table 3: Recommended Maximum Peak Particle Velocity (from AS 2187.2 – 1993)

Type of Building or Structure	Peak Particle Velocity ($\neg_{p}$) (mm/sec)
Houses and low-rise residential buildings: commercial buildings not included below	10
Commercial and industrial buildings or structures of reinforced concrete or steel construction	25

Notes to Table 3:

1. In a specific instance, where substantiated by careful investigation, a value of peak particle velocity other than that recommended in the Table 3 may be used.
2. The peak particle velocities in the Table 3 have been selected taking into consideration both human discomfort and structural integrity together with the effect on sensitive equipment located within buildings.

For buildings around this site it is suggested that 10 mm/sec be adopted as the upper limit of peak particle velocity.

It should be noted that humans are very sensitive to vibration and consequently may be disturbed by vibration levels which are considered relatively insignificant for buildings. It may therefore be beneficial to carry out vibration monitoring to confirm vibration levels during site works.

7.3 Site Classification

It should be noted that standard designs within AS 2870-2011 (Ref 1) for site classifications which are based on characteristic surface movements only apply to structures of similar size and flexibility to residential buildings and do not strictly apply to larger buildings. Similar principles in design for reactivity / movement, however, should be incorporated into design, construction and maintenance.

Site classification of foundation soil reactivity provides an indication of the propensity of the ground surface to move with seasonal variation in moisture.

Due to the presence of uncontrolled filling greater than 0.4 m depth in Bores 1, 6 and 8 and the removal of buildings, which will potentially lead to adverse soil moisture conditions, the proposed areas affected by such filling and buildings would be classified **Class P** in accordance with the procedures outlined in AS2870-2011.

The results of shrink-swell testing from samples taken from the site returned I_{ss} values ranging from 1.5 to 2.6% per ζpF.

It is noted that cracking was observed in existing building brickwork onsite and tension cracking of the clay soils was observed beneath a building in the north west part of the site, as outlined in Section 2. This indicates that reactive soils are likely present.

The results of the shrink swell testing indicated that, after removal of filling, characteristic surface movements, y_s, were estimated to range from approximately 5 mm to 35 mm under normal seasonal moisture fluctuations, primarily depending on the depth of bedrock across the site. A characteristic surface movement, y_s of up to approximately 50 mm of could occur in areas of deep clay where the surface is cut at least 0.5 m into the natural clays.

The estimated characteristic surface movement above doesn't take into account the effect of trees or the removal of buildings leading to adverse soil moisture conditions. If trees are to remain or be removed from the vicinity of the buildings foundations, then the characteristic surface movement should be revised.

Footings should be founded within the natural stiff or better natural clays, rock, or Level 1 inspected and tested engineered filling and designed in accordance with AS 2870-2011. Footings should not be founded in uncontrolled filling. Where uncontrolled filling is present at foundation level, it should be over excavated and replaced with properly placed and compacted engineered filling in accordance with Section 7.6 of this report.

Articulation joints should be provided within masonry walls in accordance with TN61 (Ref 7) in order to reduce the effects of differential movement.

Site classifications are dependent on proper site maintenance, which should be carried out in accordance with the attached CSIRO Sheet BTF-18, "Foundation Maintenance and Footing Performance: A Homeowners Guide" and with AS 2870- 2011.

The above classification should be revised following earthworks (filling or cutting) as required by AS 2870-2011. The classification would depend on the depth and type of material used as well as the level of compaction and level of quality control.

7.4 Retaining Walls

It is understood that retaining walls are proposed in the north western part of the site as part of the proposed carpark excavation. It is understood that the retaining walls will be up to 6 m in height. Further, existing retaining walls up to approximately 3 m in height, running along the western boundary of site will remain and the adequacy of these walls to support the new loads should be assessed.

It is not currently known what type of retaining system is under consideration by the client. Given the proposed height of the main wall (about 6m) an anchored wall may be required. Detailed analysis and possibly additional investigation may be required during the detailed design of the wall.

One retaining option may be a soldier pile wall with shotcrete in-fill panels. This could involve installation of the soldier piles, prior to excavation, followed by installation of shotcrete infill panels as the excavation proceeds, together with appropriately positioned and designed anchors, where necessary. It is recommended that excavation not exceed 1.5 m depth without shotcrete infill panels being constructed.

Soldier piles are normally drilled with a minimum “toe in” dictated by the retained height and passive resistance of the rock in which the “toe in” is developed. Where high strength or stronger rock is encountered, it may be feasible to terminate the soldier piles above the base of the excavation and provide lateral restraint at the toe of the piles by anchoring. Further investigation of the continuity and degree of fracturing in such high strength rock would be necessary prior to design of this system.

For permanent retaining walls, where the wall will be free to deflect and un anchored, design should be based on “active” (K_a) earth pressure coefficients, assuming a triangular earth pressure distribution. This would comprise any non-propped or laterally unrestrained walls (eg cantilever type walls).

Where structures or services are near the crest, or if the retaining walls are laterally restrained by the structure and not free to deflect, retaining wall design should be based on “at–rest” (K_o) earth pressure coefficients.

The suggested long term (permanent) design soil parameters are shown in Table 4 below. The earth pressure coefficients are for level backfill. Any additional surcharge loads, including those imposed by inclined slopes behind the wall, during or after construction or water pressure should be accounted for in design.

Table 4: Geotechnical Parameters for Retaining Structures

Parameter	Symbol	Uncontrolled Filling (Unit 3A)	Stiff to Hard Clay (Unit 2A/2B)	Extremely low to very low strength rock (Unit 3A)	Low Strength or Better Rock (Unit 3B)
Bulk Density (kN/m^3)	↑	19	20	22	22
Active earth pressure coefficient – cantilever design (free to deflect)	K_a	0.40	0.35	0.30	0.20
At-rest earth pressure coefficient – propped/restrained wall	K_o	0.60	0.53	0.45	0.30
Passive earth pressure coefficient	K_p	2.0	2.5	200 kPa	2000 kPa

The parameters above are unfactored and appropriate factors of safety should be used in design. In terms of passive pressure a factor of safety of at least 2.0 is recommended to limit deflections.

Retaining walls should include adequate subsurface and surface drainage behind the wall to prevent build-up of water pressure. Retaining walls should include free draining single size (10 mm single size gravel or coarser) aggregate backfill at the rear of the wall, with a slotted drainage pipe at the base of the backfill. The pipes should discharge to the stormwater drainage system. The backfill should be encapsulated within geotextile fabric.

A dish drain or impermeable surface should be formed at the top of the wall backfill to prevent stormwater overland flow from surcharging the wall.

It is noted that groundwater was not observed during the investigation except for seepage observed at 6.5 m depth in Bore 8 whilst drilling. These observations however were limited due to the drilling methods and the time that the bores remained open. DP recommends the installation of groundwater monitoring wells to depths below the proposed excavation to confirm the absence of groundwater within the depth of proposed excavation. Retaining walls in areas of groundwater or areas of groundwater level uncertainty must be designed to take hydrostatic pressures into account.

Cantilever walls should not be used to support any adjacent building foundations or underground services.

Once specific retaining wall options and depths are known, analysis of the proposed retaining wall using WALLAP or similar may be required for detailed design. This analysis would assess:

- Pile depth, diameter and spacing;
- Anchor spacing and load requirements;
- Construction sequencing;
- Estimated deflections, bending moments and shear forces.

Geotechnical inspection is also recommended to confirm whether additional support could be required in some areas of the excavation during construction.

7.5 Footing Design Parameters

7.5.1 Shallow Foundations

Pad or strip footings should be founded below any uncontrolled filling within natural stiff or better clays or rock at a depth of at least 0.5 m.

If rock is encountered at footing level in any portion of an individual structure, it is recommended that footings be deepened such that all footings for the structure found on rock to reduce the effects of differential movement.

The recommended maximum allowable bearing pressures for the encountered soil types are presented in Table 5 below.

Table 5: Allowable Bearing Pressure for Shallow Foundations

Founding Strata	Maximum Allowable Bearing Pressure (kPa)
Controlled Filling / Stiff Clay (Unit 2A)	100
Very stiff or better clay (Unit 2B)	200
Extremely low strength to Very Low Strength Rock (Unit 3A)	700
Low strength or better rock (Unit 3B)	1000

For such footing arrangements, it is important that slab panels are not supported on the “uncontrolled” filling but suspended between ground beams / edge beams / strips. This is to avoid potential for cracking due to differential settlement.

Shallow footings should be poured immediately after footing excavation to reduce the risk of softening from rain events / groundwater. Accordingly, footing inspections are recommended during construction to confirm adequate bearing capacity and cleanliness.

7.5.2 Piled Foundations

Depending on the applied loads and subsurface conditions, piles may be required for support of the proposed retaining walls or buildings.

Traditional bored piles are expected to be suitable, where founded on the underlying bedrock. Temporary liners may be required where deep filling is encountered to prevent collapse. The base of the hole should be cleaned of debris and water prior to placement of concrete.

Driven piles are not expected to be suitable due to the proximity of adjacent buildings and the shallow depth of bedrock in parts of the site.

Table 6 below indicates the recommended maximum allowable bearing pressure and shaft adhesion for bored piles.

Table 6: Bored Pile Design Parameters

Founding Strata	Maximum Allowable Bearing Pressure (kPa)	Maximum Allowable Shaft Adhesion (kPa)
Very stiff or better clay (Unit 2B)	400	20
Extremely low strength to Very Low Strength Rock (Unit 3A)	800	50
Low strength or better rock (Unit 3B)	1000	100

The values above assume that the pile is embedded to a depth of at least four pile diameters into the ground. For piles founded in rock the contribution of the shaft in clay should be ignored.

If rock is encountered at the pile toe level in any portion of an individual structure, it is recommended that piles be deepened such that all piles for the structure found on rock to reduce the effects of differential movement.

Estimated settlement of piles is expected to be in the order of 1% of the pile diameter or less.

Prospective piling contractors should confirm the expected rock penetration and pile capacities achievable with their equipment.

Piles should be poured immediately after excavation and inspection to reduce the risk of softening from rain events / groundwater or pile hole collapse. Care should be taken to ensure the base of the bored pile holes are cleaned and free of all loose debris and water at the time of placing concrete. Accordingly, pile hole inspections are recommended during construction to confirm adequate bearing capacity and cleanliness.

Piles should be designed with reference to the piling code AS 2159 (Ref 8). The chemical aggressiveness of soil or groundwater towards buried structures was not assessed as part of this investigation.

DP can assist with pile design for the proposed 6m high retaining wall, if required.

7.6 Pavements

7.6.1 Subgrade Conditions

The anticipated subgrade conditions in the area of the carpark and associated access road are expected to generally comprise stiff or better clay (Unit 2A/2B) based on results encountered in Bores 6, 7 and 8 undertaken in the vicinity of the carpark area. However, in the areas of the proposed 6 m cut on the northern side of the carpark, Unit 3B bedrock is likely to be encountered at subgrade level, as encountered in Bores 1 and 2. Therefore a pavement design has been given for both the clay and bedrock subgrade conditions.

Results of laboratory testing on the Unit 2 clay soil indicated a four-day soaked CBR of 3.5%. The subgrade samples tested indicate a moisture content of 0.7% to 4.1% wet of optimum moisture content and thus will require drying back, to facilitate compaction of overlying pavement materials.

Results of dynamic cone penetrometer testing (DCP) within the clay soils of Bores 6 and 7 returned values of between 6 and 19 blows per 150 mm increment. These results correspond to CBR values of between 8% and 29% as defined in Section 5.5 of Austroads – Guide to Pavement Technology (Ref 3). It is noted that the presence of gravels or cobbles in the clay soils could result in these higher blow count results.

Based on the above results and DPs experience with such soils, a design CBR of 3.0% has been adopted for stiff or better clay soils (Unit 2A / 2B) and design CBR of 10% for bedrock (Unit 3B).

7.6.2 Design Traffic

No traffic data was provided for the purposes of pavement design. Austroads: Part 2 (Ref 3) makes provision for estimating design traffic loading in equivalent standard axle repetitions (ESA) using a number of factors. A design traffic loading for the proposed carpark and access road of 4.0×10^3 DESA has been adopted.

It should be noted that typical service life of asphalt ranges between eight and twenty years for dense graded asphalt.

If the traffic loading is to be significantly different from this value, the pavement thickness designs presented in the following sections should be reviewed.

7.6.3 Flexible Pavement Thickness Design

The pavement designs are based on Austroads design procedures and are in accordance with Austroads – Guide to Pavement Technology (Ref 3). The proposed pavement thickness design is outlined in Table 7 below.

Table 7: Pavement Thickness Design – Sealed Flexible Pavement

Layer	Thickness (mm)	
	Unit 2 - Stiff of better clay subgrade	Unit 3B – Low strength or better bedrock subgrade
Design Subgrade CBR	3.0%	10%
Wearing Course	Two coat spray seal or 30 mm AC14*	Two coat spray seal or 30 mm AC14*
Basecourse	100	200
Subbase	200	-
Total	300(1)	200

Notes to Table 7:

*Where asphalt is to be used as a wearing course a 7 mm or 10 mm prime seal should be placed over the basecourse and the thickness of the AC can be deducted from the basecourse thickness

(1) Additional select may be required dependent on conditions exposed at the time of excavation

Where bedrock is exposed at subgrade level, it should be over excavated and recompacted to a depth of 150 mm to destroy preferential moisture paths.

Subgrade soils should not be allowed to be exposed to water prior to placing the pavement. Water could soften the subgrade, making compaction of the pavement layers difficult, which would likely require a select layer to achieve adequate compaction.

The recommended material quality and compaction requirements for sealed flexible pavement are presented in Table 8, below.

Table 8: Material Quality and Compaction Requirements – Sealed Flexible Pavement

Pavement Layer	Material Quality	Compaction Requirements
Basecourse	CBR ≥ 80%, PI ≤ 6%, Grading in accordance with AARB SR41 (Ref 4)	Compact to at least 98% dry density ratio Modified (AS 1289.5.2.1, Ref 6).
Subbase	CBR ≥ 30%, PI ≤ 12%. Grading in accordance with AARB SR41 (Ref 4)	Compact to at least 95% dry density ratio Modified (AS 1289.5.2.1, Ref 6).
Select Subgrade (if required)	Soaked CBR ≥ 15%.	Compact to 100% dry density ratio Standard (AS 1289.5.1.1, Ref 5).
Subgrade	Minimum CBR ≥ 3.0% (clay subgrade) Minimum CBR ≥ 10% (rock subgrade)	Compact to at least 100% dry density ratio Standard (AS 1289.5.1.1, Ref 5).

Notes to Table 8:

CBR – California bearing ratio (4 day soaked)
PI – Plasticity Index

The select subgrade if required should be a well-graded material which is suitable for placement over wet clay soils, and which requires minimal working / rolling to achieve compaction. The maximum particle size of the select should be half the layer thickness.

The pavement thicknesses presented above are dependent on the provision and maintenance of adequate surface and subsurface drainage. Adequate surface drainage should also be provided to avoid water ponding at the surface and minimise the ingress of water in to the pavement materials.

It is recommended that where any new pavement abuts an existing pavement, it should be benched / keyed in a minimum width of 0.3 m. Vertical interface / joints between the new and existing sections of pavements should not be located within wheel paths. Allowance should also be made for the incorporation of intra pavement drainage.

The pavement thickness design presented in this report refers to minimum layer thicknesses, no allowance has been made for construction tolerances and the like.

Geotechnical inspection should be undertaken during construction to confirm the subgrade conditions and the requirements for subgrade improvements (such as select layers, drying back etc.), if required.

7.7 Earthworks

7.7.1 Material Reuse for Engineered Filling

It is understood that proposed cut materials may be reused for filling on site.

The excavated clays and bedrock (Units 2 and 3) are considered geotechnically suitable for re-use as engineered fill provided that they are free of deleterious inclusions such as organics and can be produced in suitable particle sizes (generally with a maximum particle size of less than 100 mm and well-graded distribution).

All proposed fill materials should be screened / sieved or particles broken down by excavation / handling / compaction methods, thus removing / crushing oversized particles greater than 100 mm prior to use as engineered filling.

Topsoil materials are considered suitable for re-use as topsoil.

Field observations from tactile assessment of the clays indicated that the clay soils were typically medium to high plasticity. The results of laboratory testing returned Iss values of 1.5 and 2.6% per ζpF and hence consideration should be given to the effect on final site classification should this material be used as lot filling.

7.7.2 Pavement Subgrade Preparation Measures

Pavement construction procedures should be subject to Level 2 geotechnical inspections and testing as detailed in AS 3798 – 2007 (Ref 2), which requires at least one field density test per layer of filling placed.

A smooth drum roller operating in static mode may be used for test rolling of the subgrade.

The following subgrade preparation measures should be implemented to allow placement and compaction of select material and pavement layers:

- Excavate to design subgrade level to expose stiff or better clay soil or rock. The clay surface should be sloped to ensure water does not pond over the clay;
- Compact the subgrade to at least 100% standard dry density ratio, at a moisture content within the range from 3% (dry) to 1% (wet) of OMC, where OMC is the optimum moisture content as measured by AS 1289.2.1.1;
- Vehicles with rubber tyres should not traffic the clay subgrade;
- The exposed clay surface should be inspected by a geotechnical engineer to check for excessively wet areas or weak zones which may require additional removal;
- Test roll the subgrade using a smooth drum roller. The test roll should be witnessed by a geotechnical engineer who will assess for any soft / heaving areas which may require removal and replacement with select material;
- If required, place a select subgrade in a single 0.2 m layer over the clay, or thicker if required;
- The select subgrade layer should be a gravel material which is suitable for placement over wet clay soils, and which requires minimal working / rolling to achieve compaction. The maximum particle size of the select should be no greater than half the layer thickness;
- The select layer should be compacted by a 8 tonne to 10 tonne roller in static mode;
- The upper half of the select layer should be compacted to at least 100% standard compaction;
- Place pavement materials to the requirements as outlined above.

7.7.3 Building Platform Fill Preparation Measures

The construction of a filling platform under buildings should be carried out in accordance with Level 1 inspection and testing, as defined in AS 3798-2007 (Ref 2) and should include the following:

- Remove topsoil, uncontrolled filling and unsuitable materials to expose Unit 2 to 3 stiff or better clay or rock;
- Remove the tree root zone;
- Inspect the exposed subgrade and remove soft / weak material. Unsuitable material should be replaced with engineered fill;
- Compact the subgrade to at least 98% Standard at a moisture content within the range from 2% (dry) to 1% (wet) of OMC;
- Place and compact the engineered fill (non-reactive materials) to at least 98% Standard dry density ratio, as measured by AS 1289.5.1.1, at a moisture content within the range ± 2% of OMC, where OMC is the optimum moisture content as measured by AS 1289.5.1.1. Non-reactive fill should be used within the upper 1 m of fill profile if possible to control future reactive movements;
- The “non-reactive” fill material should be a low permeability crushed siltstone/sandstone or sandy gravel placed as engineered fill (Level 1 inspection and testing) in accordance with AS 3798 (Ref 2), with shrink-swell index of less than 1%, or a PI of less than 15%;

- Adequate surface drainage should be provided to direct surface water away from engineered filling.

At the completion of earthworks, the surface heave movements and re-classification of the site should be confirmed by site specific laboratory testing and engineering assessment.

7.7.4 General

- Engineered fill should be placed in near horizontal layers not exceeding 300 mm loose thickness, and with a maximum particle size not exceeding two-thirds of the compacted layer thickness;
- Maximum temporary batter slopes of 1.5H:1V (up to 2.0 m depth) during construction are recommended, and batters may need to be 1V:3H or possibly flatter if saturated soils are encountered. For deeper cuts or cuts with groundwater emanating from the face, specific assessment is recommended;
- Each fill layer should be keyed or benched at least 0.3 m into batter slopes;
- Adequate surface drainage should be provided to direct surface water away from engineered filling;
- Excavations should be wide enough to allow access for adequately sized compaction equipment;
- Embankments should be over-filled at the batters and trimmed back to the design batter angle to ensure the filling is compacted for the full design width.

Geotechnical inspection, compaction testing and test rolling of all engineered fill is recommended. Subgrade inspections are also recommended.

Earthworks construction procedures should be in accordance with the Australian Standard AS 3798-2007 (Ref 2).

8. Recommended Additional Investigation

It should be noted that only limited details of the proposed development were available at the time of investigation. Hence subsurface investigation has been preliminary in nature and has included investigation at locations spread throughout the development area. Additional investigation would be prudent when more detailed designs are developed to assess the variability of subsurface conditions, particularly at key structures. The following is a list of items to be investigated in more detail or items that need to be addressed in such investigations:

- Review of the preliminary advice once the bulk earthworks details are finalised;
- Further assessment in the areas of potential filling to determine extent, characteristics and depth of filling;
- Detailed investigation and analysis in the area of the proposed 6 m cut and retaining wall;
- Waste classification assessment of any soils which are to be removed from the site; and
- Routine geotechnical inspection and earthworks testing during construction.

9. References

1. Australian Standard AS2870-2011, 'Residential Slabs and Footings', April 2011, Standards Australia.
2. Australian Standard AS 3798 – 2007 "Guidelines on Earthworks for Commercial and Residential Developments", Standards Australia.
3. Austroads AGPT02-12 "Guide to Pavement Technology, Part 2: Pavement Structural Design", 2012.
4. ARRB Special Report No 41 "A Structural Design Guide for Flexible Residential Sheet Pavements", 1989.
5. Australian Standard AS 1289.5.1.1-2003, "Methods of testing soils for engineering purposes", Standards Australia.
6. Australian Standard AS 1289.5.2.1-2003, "Methods of testing soils for engineering purposes", Standards Australia.
7. Cement Concrete and Aggregates Australia, TN61, "Articulated Walling".
8. Australian Standard AS 2159-2009 "Piling – Design and Installation", 2009 Standards Australia.

10. Limitations

Douglas Partners (DP) has prepared this report for this project at Scaysbrook Drive Kincumber in accordance with DP's proposal NCL160534.P.001.Rev 1 dated 18 October 2016 and email acceptance received from Lend Lease Building Pty Ltd dated 13 October 2016. The work was carried out under DP's Conditions of Engagement. This report is provided for the exclusive use of Lend Lease Building Pty Ltd for this project only and for the purposes as described in the report. It should not be used by or relied upon for other projects or purposes on the same or other site or by a third party. Any party so relying upon this report beyond its exclusive use and purpose as stated above, and without the express written consent of DP, does so entirely at its own risk and without recourse to DP for any loss or damage. In preparing this report DP has necessarily relied upon information provided by the client and/or their agents.

The results provided in the report are indicative of the sub-surface conditions on the site only at the specific sampling and/or testing locations, and then only to the depths investigated and at the time the work was carried out. Sub-surface conditions can change abruptly due to variable geological processes and also as a result of human influences. Such changes may occur after DP's field testing has been completed.

DP's advice is based upon the conditions encountered during this investigation. The accuracy of the advice provided by DP in this report may be affected by undetected variations in ground conditions across the site between and beyond the sampling and/or testing locations. The advice may also be limited by budget constraints imposed by others or by site accessibility.

This report must be read in conjunction with all of the attached and should be kept in its entirety without separation of individual pages or sections. DP cannot be held responsible for interpretations or conclusions made by others unless they are supported by an expressed statement, interpretation, outcome or conclusion stated in this report.

This report, or sections from this report, should not be used as part of a specification for a project, without review and agreement by DP. This is because this report has been written as advice and opinion rather than instructions for construction.

The scope for work for this investigation/report did not include the assessment of surface or sub-surface materials or groundwater for contaminants, within or adjacent to the site. Should evidence of filling of unknown origin be noted in the report, and in particular the presence of building demolition materials, it should be recognised that there may be some risk that such filling may contain contaminants and hazardous building materials.

The contents of this report do not constitute formal design components such as are required, by the Health and Safety Legislation and Regulations, to be included in a Safety Report specifying the hazards likely to be encountered during construction and the controls required to mitigate risk. This design process requires risk assessment to be undertaken, with such assessment being dependent upon factors relating to likelihood of occurrence and consequences of damage to property and to life. This, in turn, requires project data and analysis presently beyond the knowledge and project role respectively of DP. DP may be able, however, to assist the client in carrying out a risk assessment of potential hazards contained in the Comments section of this report, as an extension to the current scope of works, if so requested, and provided that suitable additional information is made available to DP. Any such risk assessment would, however, be necessarily restricted to the geotechnical components set out in this report and to their application by the project designers to project design, construction, maintenance and demolition.

Douglas Partners Pty Ltd

Appendix A

About this Report
Sampling Methods
Soil Descriptions
Symbols and Abbreviations
Rock Descriptions
CSIRO Sheet BTF18

About this Report

Douglas Partners

Introduction

These notes have been provided to amplify DP's report in regard to classification methods, field procedures and the comments section. Not all are necessarily relevant to all reports.

DP's reports are based on information gained from limited subsurface excavations and sampling, supplemented by knowledge of local geology and experience. For this reason, they must be regarded as interpretive rather than factual documents, limited to some extent by the scope of information on which they rely.

Copyright

Borehole and Test Pit Logs

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

Interpretation of the information and its application to design and construction should therefore take into account the spacing of boreholes or pits, the frequency of sampling, and the possibility of other than 'straight line' variations between the test locations.

Groundwater

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

- In low permeability soils groundwater may enter the hole very slowly or perhaps not at all during the time the hole is left open;
- A localised, perched water table may lead to an erroneous indication of the true water table;
- Water table levels will vary from time to time with seasons or recent weather changes. They may not be the same at the time of construction as are indicated in the report; and
- The use of water or mud as a drilling fluid will mask any groundwater inflow. Water has to be blown out of the hole and drilling mud must first be washed out of the hole if water measurements are to be made.

More reliable measurements can be made by installing standpipes which are read at intervals over several days, or perhaps weeks for low permeability soils. Piezometers, sealed in a particular stratum, may be advisable in low permeability soils or where there may be interference from a perched water table.

Reports

The report has been prepared by qualified personnel, is based on the information obtained from field and laboratory testing, and has been undertaken to current engineering standards of interpretation and analysis. Where the report has been prepared for a specific design proposal, the information and interpretation may not be relevant if the design proposal is changed. If this happens, DP will be pleased to review the report and the sufficiency of the investigation work.

Every care is taken with the report as it relates to interpretation of subsurface conditions, discussion of geotechnical and environmental aspects, and recommendations or suggestions for design and construction. However, DP cannot always anticipate or assume responsibility for:

- Unexpected variations in ground conditions. The potential for this will depend partly on borehole or pit spacing and sampling frequency;
- Changes in policy or interpretations of policy by statutory authorities; or
- The actions of contractors responding to commercial pressures.

If these occur, DP will be pleased to assist with investigations or advice to resolve the matter.

About this Report

Site Anomalies

In the event that conditions encountered on site during construction appear to vary from those which were expected from the information contained in the report, DP requests that it be immediately notified. Most problems are much more readily resolved when conditions are exposed rather than at some later stage, well after the event.

Information for Contractual Purposes

Where information obtained from this report is provided for tendering purposes, it is recommended that all information, including the written report and discussion, be made available. In circumstances where the discussion or comments section is not relevant to the contractual situation, it may be appropriate to prepare a specially edited document. DP would be pleased to assist in this regard and/or to make additional report copies available for contract purposes at a nominal charge.

Site Inspection

The company will always be pleased to provide engineering inspection services for geotechnical and environmental aspects of work to which this report is related. This could range from a site visit to confirm that conditions exposed are as expected, to full time engineering presence on site.

Sampling

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

Disturbed samples taken during drilling provide information on colour, type, inclusions and, depending upon the degree of disturbance, some information on strength and structure.

Undisturbed samples are taken by pushing a thin-walled sample tube into the soil and withdrawing it to obtain a sample of the soil in a relatively undisturbed state. Such samples yield information on structure and strength, and are necessary for laboratory determination of shear strength and compressibility. Undisturbed sampling is generally effective only in cohesive soils.

Test Pits

Test pits are usually excavated with a backhoe or an excavator, allowing close examination of the in-situ soil if it is safe to enter into the pit. The depth of excavation is limited to about 3 m for a backhoe and up to 6 m for a large excavator. A potential disadvantage of this investigation method is the larger area of disturbance to the site.

Large Diameter Augers

Boreholes can be drilled using a rotating plate or short spiral auger, generally 300 mm or larger in diameter commonly mounted on a standard piling rig. The cuttings are returned to the surface at intervals (generally not more than 0.5 m) and are disturbed but usually unchanged in moisture content. Identification of soil strata is generally much more reliable than with continuous spiral flight augers, and is usually supplemented by occasional undisturbed tube samples.

Continuous Spiral Flight Augers

The borehole is advanced using 90-115 mm diameter continuous spiral flight augers which are withdrawn at intervals to allow sampling or in-situ testing. This is a relatively economical means of drilling in clays and sands above the water table. Samples are returned to the surface, or may be collected after withdrawal of the auger flights, but they are disturbed and may be mixed with soils from the sides of the hole. Information from the drilling (as distinct from specific sampling by SPTs or undisturbed samples) is of relatively low reliability, due to the remoulding, possible mixing or softening of samples by groundwater.

Non-core Rotary Drilling

The borehole is advanced using a rotary bit, with water or drilling mud being pumped down the drill rods and returned up the annulus, carrying the drill cuttings. Only major changes in stratification can be determined from the cuttings, together with some information from the rate of penetration. Where drilling mud is used this can mask the cuttings and reliable identification is only possible from separate sampling such as SPTs.

Continuous Core Drilling

A continuous core sample can be obtained using a diamond tipped core barrel, usually with a 50 mm internal diameter. Provided full core recovery is achieved (which is not always possible in weak rocks and granular soils), this technique provides a very reliable method of investigation.

Standard Penetration Tests

Standard penetration tests (SPT) are used as a means of estimating the density or strength of soils and also of obtaining a relatively undisturbed sample. The test procedure is described in Australian Standard 1289, Methods of Testing Soils for Engineering Purposes - Test 6.3.1.

The test is carried out in a borehole by driving a 50 mm diameter split sample tube under the impact of a 63 kg hammer with a free fall of 760 mm. It is normal for the tube to be driven in three successive 150 mm increments and the 'N' value is taken as the number of blows for the last 300 mm. In dense sands, very hard clays or weak rock, the full 450 mm penetration may not be practicable and the test is discontinued.

The test results are reported in the following form.

- In the case where full penetration is obtained with successive blow counts for each 150 mm of, say, 4, 6 and 7 as:

  4,6,7
  N=13

- In the case where the test is discontinued before the full penetration depth, say after 15 blows for the first 150 mm and 30 blows for the next 40 mm as:

  15, 30/40 mm

The results of the SPT tests can be related empirically to the engineering properties of the soils.

Dynamic Cone Penetrometer Tests / Perth Sand Penetrometer Tests

Dynamic penetrometer tests (DCP or PSP) are carried out by driving a steel rod into the ground using a standard weight of hammer falling a specified distance. As the rod penetrates the soil the number of blows required to penetrate each successive 150 mm depth are recorded. Normally there is a depth limitation of 1.2 m, but this may be extended in certain conditions by the use of extension rods. Two types of penetrometer are commonly used.

- Perth sand penetrometer - a 16 mm diameter flat ended rod is driven using a 9 kg hammer dropping 600 mm (AS 1289, Test 6.3.3). This test was developed for testing the density of sands and is mainly used in granular soils and filling.
- Cone penetrometer - a 16 mm diameter rod with a 20 mm diameter cone end is driven using a 9 kg hammer dropping 510 mm (AS 1289, Test 6.3.2). This test was developed initially for pavement subgrade investigations, and correlations of the test results with California Bearing Ratio have been published by various road authorities.

Soil Descriptions

Douglas Partners

Description and Classification Methods

The methods of description and classification of soils and rocks used in this report are based on Australian Standard AS 1726, Geotechnical Site Investigations Code. In general, the descriptions include strength or density, colour, structure, soil or rock type and inclusions.

Soil Types

Soil types are described according to the predominant particle size, qualified by the grading of other particles present:

Type	Particle size (mm)
Boulder	>200
Cobble	63 - 200
Gravel	2.36 - 63
Sand	0.075 - 2.36
Silt	0.002 - 0.075
Clay	<0.002

The sand and gravel sizes can be further subdivided as follows:

Type	Particle size (mm)
Coarse gravel	20 - 63
Medium gravel	6 - 20
Fine gravel	2.36 - 6
Coarse sand	0.6 - 2.36
Medium sand	0.2 - 0.6
Fine sand	0.075 - 0.2

The proportions of secondary constituents of soils are described as:

Term	Proportion	Example
And	Specify	Clay (60%) and Sand (40%)
Adjective	20 - 35%	Sandy Clay
Slightly	12 - 20%	Slightly Sandy Clay
With some	5 - 12%	Clay with some sand
With a trace of	0 - 5%	Clay with a trace of sand

Definitions of grading terms used are:

- Well graded - a good representation of all particle sizes
- Poorly graded - an excess or deficiency of particular sizes within the specified range
- Uniformly graded - an excess of a particular particle size
- Gap graded - a deficiency of a particular particle size with the range

Cohesive Soils

Cohesive soils, such as clays, are classified on the basis of undrained shear strength. The strength may be measured by laboratory testing, or estimated by field tests or engineering examination. The strength terms are defined as follows:

Description	Abbreviation	Undrained shear strength (kPa)
Very soft	vs	<12
Soft	s	12 - 25
Firm	f	25 - 50
Stiff	st	50 - 100
Very stiff	vst	100 - 200
Hard	h	>200

Cohesionless Soils

Cohesionless soils, such as clean sands, are classified on the basis of relative density, generally from the results of standard penetration tests (SPT), cone penetration tests (CPT) or dynamic penetrometers (PSP). The relative density terms are given below:

Relative Density	Abbreviation	SPT N value	CPT qc value (MPa)
Very loose	vl	<4	<2
Loose	l	4 - 10	2 -5
Medium dense	md	10 - 30	5 - 15
Dense	d	30 - 50	15 - 25
Very dense	vd	>50	>25

Soil Descriptions

Soil Origin

It is often difficult to accurately determine the origin of a soil. Soils can generally be classified as:

- Residual soil - derived from in-situ weathering of the underlying rock;
- Transported soils - formed somewhere else and transported by nature to the site; or
- Filling - moved by man.

Transported soils may be further subdivided into:

- Alluvium - river deposits
- Lacustrine - lake deposits
- Aeolian - wind deposits
- Littoral - beach deposits
- Estuarine - tidal river deposits
- Talus - scree or coarse colluvium
- Slopewash or Colluvium - transported downslope by gravity assisted by water. Often includes angular rock fragments and boulders.

Symbols & Abbreviations

Douglas Partners

Introduction

These notes summarise abbreviations commonly used on borehole logs and test pit reports.

Drilling or Excavation Methods

C	Core Drilling
R	Rotary drilling
SFA	Spiral flight augers
NMLC	Diamond core - 52 mm dia
NQ	Diamond core - 47 mm dia
HQ	Diamond core - 63 mm dia
PQ	Diamond core - 81 mm dia

Water

▷	Water seep
▽	Water level

Sampling and Testing

A	Auger sample
B	Bulk sample
D	Disturbed sample
E	Environmental sample
U_{50}	Undisturbed tube sample (50mm)
W	Water sample
pp	pocket penetrometer (kPa)
PID	Photo ionisation detector
PL	Point load strength Is(50) MPa
S	Standard Penetration Test
V	Shear vane (kPa)

Description of Defects in Rock

The abbreviated descriptions of the defects should be in the following order: Depth, Type, Orientation, Coating, Shape, Roughness and Other. Drilling and handling breaks are not usually included on the logs.

Defect Type

B	Bedding plane
Cs	Clay seam
Cv	Cleavage
Cz	Crushed zone
Ds	Decomposed seam
F	Fault
J	Joint
Lam	lamination
Pt	Parting
Sz	Sheared Zone
V	Vein

Orientation

The inclination of defects is always measured from the perpendicular to the core axis.

h	horizontal
v	vertical
sh	sub-horizontal
sv	sub-vertical

Coating or Infilling Term

cln	clean
co	coating
he	healed
inf	infilled
stn	stained
ti	tight
vn	veneer

Coating Descriptor

ca	calcite
cbs	carbonaceous
cly	clay
fe	iron oxide
mn	manganese
slt	silty

Shape

cu	curved
ir	irregular
pl	planar
st	stepped
un	undulating

Roughness

po	polished
ro	rough
sl	slickensided
sm	smooth
vr	very rough

Other

fg	fragmented
bnd	band
qtz	quartz

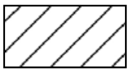
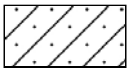
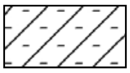
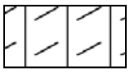
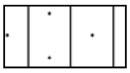
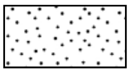
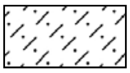
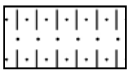
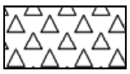
Symbols & Abbreviations

Graphic Symbols for Soil and Rock

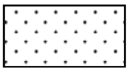
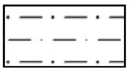
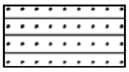
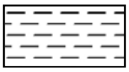
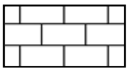
General

	Asphalt
	Road base
	Concrete
	Filling

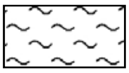
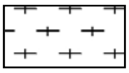
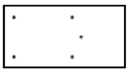
Soils

	Topsoil
	Peat
	Clay
	Silty clay
	Sandy clay
	Gravelly clay
	Shaly clay
	Silt
	Clayey silt
	Sandy silt
	Sand
	Clayey sand
	Silty sand
	Gravel
	Sandy gravel
	Cobbles, boulders
	Talus

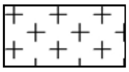
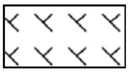
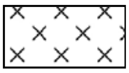
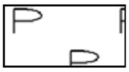
Sedimentary Rocks

	Boulder conglomerate
	Conglomerate
	Conglomeratic sandstone
	Sandstone
	Siltstone
	Laminite
	Mudstone, claystone, shale
	Coal
	Limestone

Metamorphic Rocks

	Slate, phyllite, schist
	Gneiss
	Quartzite

Igneous Rocks

	Granite
	Dolerite, basalt, andesite
	Dacite, epidote
	Tuff, breccia
	Porphyry

Rock Strength

Rock strength is defined by the Point Load Strength Index ($Is_{(50)}$) and refers to the strength of the rock substance and not the strength of the overall rock mass, which may be considerably weaker due to defects. The test procedure is described by Australian Standard 4133.4.1 - 1993. The terms used to describe rock strength are as follows:

Term	Abbreviation	Point Load Index $Is_{(50)}$ MPa	Approx Unconfined Compressive Strength MPa*
Extremely low	EL	<0.03	<0.6
Very low	VL	0.03 - 0.1	0.6 - 2
Low	L	0.1 - 0.3	2 - 6
Medium	M	0.3 - 1.0	6 - 20
High	H	1 - 3	20 - 60
Very high	VH	3 - 10	60 - 200
Extremely high	EH	>10	>200

* Assumes a ratio of 20:1 for UCS to $Is_{(50)}$

Degree of Weathering

The degree of weathering of rock is classified as follows:

Term	Abbreviation	Description
Extremely weathered	EW	Rock substance has soil properties, i.e. it can be remoulded and classified as a soil but the texture of the original rock is still evident.
Highly weathered	HW	Limonite staining or bleaching affects whole of rock substance and other signs of decomposition are evident. Porosity and strength may be altered as a result of iron leaching or deposition. Colour and strength of original fresh rock is not recognisable
Moderately weathered	MW	Staining and discolouration of rock substance has taken place
Slightly weathered	SW	Rock substance is slightly discoloured but shows little or no change of strength from fresh rock
Fresh stained	Fs	Rock substance unaffected by weathering but staining visible along defects
Fresh	Fr	No signs of decomposition or staining

Degree of Fracturing

The following classification applies to the spacing of natural fractures in diamond drill cores. It includes bedding plane partings, joints and other defects, but excludes drilling breaks.

Term	Description
Fragmented	Fragments of <20 mm
Highly Fractured	Core lengths of 20-40 mm with some fragments
Fractured	Core lengths of 40-200 mm with some shorter and longer sections
Slightly Fractured	Core lengths of 200-1000 mm with some shorter and loner sections
Unbroken	Core lengths mostly > 1000 mm

Rock Descriptions

Rock Quality Designation

The quality of the cored rock can be measured using the Rock Quality Designation (RQD) index, defined as:

$$\text{RQD \%} = \frac{\text{cumulative length of 'sound' core sections} \geq 100 \text{ mm long}}{\text{total drilled length of section being assessed}}$$

where 'sound' rock is assessed to be rock of low strength or better. The RQD applies only to natural fractures. If the core is broken by drilling or handling (i.e. drilling breaks) then the broken pieces are fitted back together and are not included in the calculation of RQD.

Stratification Spacing

For sedimentary rocks the following terms may be used to describe the spacing of bedding partings:

Term	Separation of Stratification Planes
Thinly laminated	< 6 mm
Laminated	6 mm to 20 mm
Very thinly bedded	20 mm to 60 mm
Thinly bedded	60 mm to 0.2 m
Medium bedded	0.2 m to 0.6 m
Thickly bedded	0.6 m to 2 m
Very thickly bedded	> 2 m

Foundation Maintenance and Footing Performance: A Homeowner's Guide

CSIRO PUBLISHING

BTF 18-2011 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 2870-2011, 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, which may experience only slight ground movement from moisture changes
M	Moderately reactive clay or silt sites, which may experience moderate ground movement from moisture changes
H1	Highly reactive clay sites, which may experience high ground movement from moisture changes
H2	Highly reactive clay sites, which may experience very high ground movement from moisture changes
E	Extremely reactive sites, which may experience extreme ground movement from moisture changes

Notes

1. Where controlled fill has been used, the site may be classified A to E according to the type of fill used.
2. Filled sites. Class P is used for sites which include soft fills, such as clay or silt or loose sands; landslip; mine subsidence; collapsing soils; soil subject to erosion; reactive sites subject to abnormal moisture conditions or sites which cannot be classified otherwise.
3. Where deep-seated moisture changes exist on sites at depths of 3 m or greater, further classification is needed for Classes M to E (M-D, H1-D, H2-D and E-D).

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.

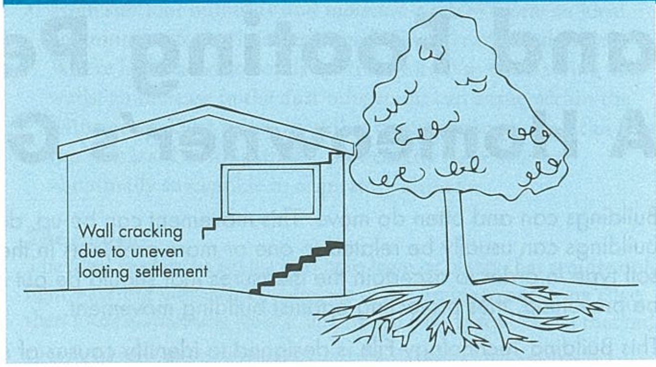
Seasonal swelling/shrinkage in clay
Swelling foundation soil due to rainy periods first lifts the most exposed extremities of the footing system, then the remainder of the perimeter footings while gradually permeating inside the building footprint to lift internal footings. This swelling first tends 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.

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

Trees can cause shrinkage and damage



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 causes 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 2870-2011.

AS 2870-2011 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

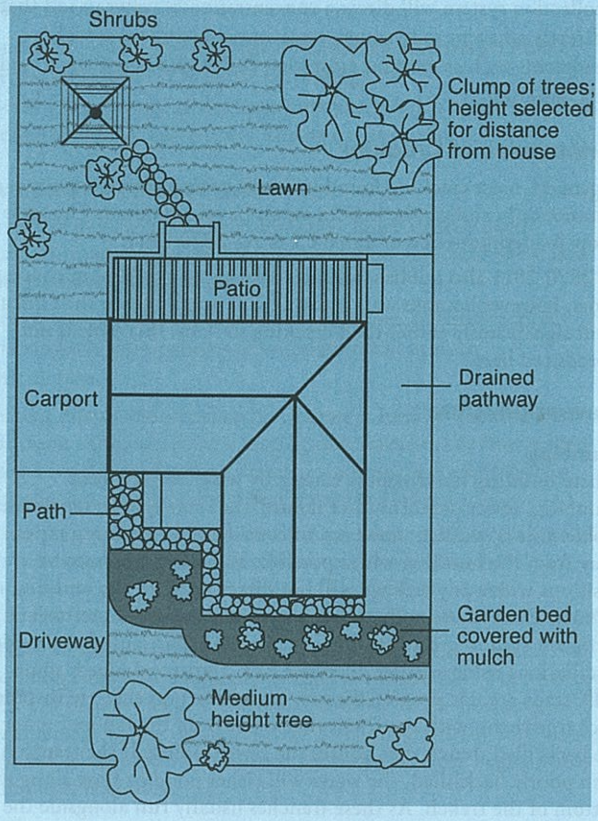
It is essential to remember that the soil that affects footings extends well beyond the actual building line. Watering of garden plants, shrubs and trees causes some of the most serious water problems.

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 should

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 depends on number of cracks	4

Gardens for a reactive site



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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Appendix B

Borehole Logs 1 to 8
Core Photoplates
Results of Dynamic Penetrometer Testing

BOREHOLE LOG

CLIENT: Lend Lease Building Pty Ltd
PROJECT: Proposed Aged Care Development
LOCATION: Scaysbrook Drive, Kincumber

SURFACE LEVEL: 41.0 AHD*
EASTING: 351890
NORTHING: 6295770
DIP/AZIMUTH: 90°/--

BORE No: BH1
PROJECT No: 91006.00
DATE: 24/10/2016
SHEET 1 OF 2

RL	Depth (m)	Description of Strata	Graphic Log	Type	Depth	Sample	Results & Comments	Water	Well Construction Details
	0.06	FILLING - Pavers							
		FILLING - Generally comprising brown, fine to coarse grained sand filling, moist		D	0.1				
	0.3	FILLING - Generally comprising brown, fine to medium grained sandy clay filling, some fine to coarse sized gravel, M>Wp		D	0.5				
1				D	1.0				1
				S	1.0–1.45		pp = 150 2,2,5 N = 7		
2	2.0	CLAY - Very stiff, grey red clay, M>Wp							2
				S	2.5–2.95		pp = 280 3,6,11 N = 17		
3									3
	3.3	SANDSTONE - Extremely low strength, extremely weathered red orange sandstone, some clay bands and ironstaining							
		From 3.70m, (extremely low to very low strength) with clay like bands and soil like properties		S	3.7–3.81		pp = 600 25/110mm,-,- refusal		
4									4

RIG: Nissan Patrol Mounted Drill Rig **DRILLER:** FICO (Currie) **LOGGED:** Parkinson **CASING:** Nil

TYPE OF BORING: Solid flight auger v-bit to 3.7m refusal, TC bit to 5.4m

WATER OBSERVATIONS: No free groundwater obserbed

REMARKS: ( ) Strength inferred from drilling resistance. *RLs interpolated from client supplied plan to nearest 0.5m.

SAMPLING & IN SITU TESTING LEGEND

A	Auger sample	G	Gas sample	PID	Photo ionisation detector (ppm)
B	Bulk sample	P	Piston sample	PL(A)	Point load axial test Is(50) (MPa)
BLK	Block sample	U_x	Tube sample (x mm dia.)	PL(D)	Point load diametral test Is(50) (MPa)
C	Core drilling	W	Water sample	pp	Pocket penetrometer (kPa)
D	Disturbed sample	▷	Water seep	S	Standard penetration test
E	Environmental sample	▼	Water level	V	Shear vane (kPa)

Douglas Partners
Geotechnics / Environment / Groundwater

BOREHOLE LOG

CLIENT: Lend Lease Building Pty Ltd
PROJECT: Proposed Aged Care Development
LOCATION: Scaysbrook Drive, Kincumber

SURFACE LEVEL: 41.0 AHD*
EASTING: 351890
NORTHING: 6295770
DIP/AZIMUTH: 90°/--

BORE No: BH1
PROJECT No: 91006.00
DATE: 24/10/2016
SHEET 2 OF 2

RL	Depth (m)	Description of Strata	Graphic Log	Type	Depth	Sample	Results & Comments	Water	Well Construction Details
		SANDSTONE - Extremely low strength, extremely weathered red orange sandstone, some clay bands and ironstaining *(continued)* From 5.1m, increased drilling resistance							
	5.4	Bore discontinued at 5.4m , slow progress							
	6								6
	7								7
	8								8
	9								9

RIG: Nissan Patrol Mounted Drill Rig **DRILLER:** FICO (Currie) **LOGGED:** Parkinson **CASING:** Nil
TYPE OF BORING: Solid flight auger v-bit to 3.7m refusal, TC bit to 5.4m
WATER OBSERVATIONS: No free groundwater obserbed
REMARKS: ( ) Strength inferred from drilling resistance. *RLs interpolated from client supplied plan to nearest 0.5m.

SAMPLING & IN SITU TESTING LEGEND

A	Auger sample	G	Gas sample	PID	Photo ionisation detector (ppm)
B	Bulk sample	P	Piston sample	PL(A)	Point load axial test Is(50) (MPa)
BLK	Block sample	U_x	Tube sample (x mm dia.)	PL(D)	Point load diametral test Is(50) (MPa)
C	Core drilling	W	Water sample	pp	Pocket penetrometer (kPa)
D	Disturbed sample	▷	Water seep	S	Standard penetration test
E	Environmental sample	▼	Water level	V	Shear vane (kPa)

Douglas Partners
Geotechnics / Environment / Groundwater

BOREHOLE LOG

CLIENT: Lend Lease Building Pty Ltd
PROJECT: Proposed Aged Care Development
LOCATION: Scaysbrook Drive, Kincumber

SURFACE LEVEL: 40.5 AHD*
EASTING: 351920
NORTHING: 6295741
DIP/AZIMUTH: 90°/--

BORE No: BH2
PROJECT No: 91006.00
DATE: 24/10/2016
SHEET 1 OF 3

RL	Depth (m)	Description of Strata	Degree of Weathering (EW HW MW SW FS FR)	Graphic Log	Rock Strength (Ex Low, Very Low, Low, Medium, High, Very High, Ex High)	Water	Fracture Spacing (m) (0.01, 0.05, 0.10, 0.50, 1.00)	Discontinuities (B - Bedding, J - Joint, S - Shear, F - Fault)	Sampling & In Situ Testing: Type	Core Rec. %	RQD %	Test Results & Comments
		FILLING - Pavers										
	0.08	FILLING - Generally comprising brown grey gravelly sand filling gravel, medium to coarse sized and subrounded, moist							D			
	0.25	CLAY - Very stiff, orange brown clay, trace fine to medium grained sand, M>Wp							S			pp = 350 2,11,13 N = 24
	0.8	CLAY - Hard grey mottled orange red clay, with some fine to medium grained (low to medium strength) ironstained gravel / cobble bands, M<Wp (completely weathered rock)										
	1											
	2								S			pp = 600 10,20/120mm refusal

RIG: Nissan Patrol Mounted Drill Rig **DRILLER:** FICO **LOGGED:** Parkinson **CASING:** HQ to 4.5m

TYPE OF BORING: Solid flight auger vbit to 1.1m refusal, TC bit to 4.5m, NMLC core to 7.50m

WATER OBSERVATIONS: No free groundwater in top 4.5m, observations obscured below due to drilling fluids

REMARKS: Vbit refusal at 1.1m on probable ironstained band. () strength inferred from drilling resistance. *RLs interpolated from client supplied plan to nearest 0.5m

SAMPLING & IN SITU TESTING LEGEND

A	Auger sample	G	Gas sample	PID	Photo ionisation detector (ppm)
B	Bulk sample	P	Piston sample	PL(A)	Point load axial test Is(50) (MPa)
BLK	Block sample	U_x	Tube sample (x mm dia.)	PL(D)	Point load diametral test Is(50) (MPa)
C	Core drilling	W	Water sample	pp	Pocket penetrometer (kPa)
D	Disturbed sample	▷	Water seep	S	Standard penetration test
E	Environmental sample	▼	Water level	V	Shear vane (kPa)

Douglas Partners
Geotechnics / Environment / Groundwater

BOREHOLE LOG

CLIENT: Lend Lease Building Pty Ltd
PROJECT: Proposed Aged Care Development
LOCATION: Scaysbrook Drive, Kincumber

SURFACE LEVEL: 40.5 AHD*
EASTING: 351920
NORTHING: 6295741
DIP/AZIMUTH: 90°/--

BORE No: BH2
PROJECT No: 91006.00
DATE: 24/10/2016
SHEET 2 OF 3

RL	Depth (m)	Description of Strata	Degree of Weathering (EW HW MW SW FS FR)	Graphic Log	Rock Strength (Ex Low, Very Low, Low, Medium, High, Very High, Ex High)	Water	Fracture Spacing (m) (0.01, 0.05, 0.10, 0.50, 1.00)	Discontinuities (B - Bedding, J - Joint, S - Shear, F - Fault)	Type	Core Rec. %	RQD %	Test Results & Comments
		CLAY - Hard grey mottled orange red clay, with some fine to medium grained (low to medium strength) ironstained gravel / cobble bands, M<Wp (completely weathered rock) *(continued)*										
	3.5	SILTSTONE - Extremely low strength, brown red siltstone										
	4								S			pp = 600 20,15/40mm refusal
	4.5	CORE LOSS - 0.30m - probable siltstone						4.5m: CORE LOSS: 300mm				
	4.8	SANDSTONE - High strength, moderately weathered grey stained orange red, fine to medium grained sandstone, some sub horizontal ironstained healed partings, fractured to slightly fractured From 4.90m to 5.15m, ironstained band						4.85m: J, 40°, pl, ro, vn, clay 4.88m: J, 40°, pl, ro, vn, clay	C	80	61	
	5							5.15m: PP, sh, pl, ro, stn, fe 5.3m: J, 70°, ir, ro, stn, fe 5.34m: PL, sh, pl, ro, stn, fe 5.42m: PL, sh, pl, ro, stn, fe 5.6m: PL, sh, pl, ro, stn, fe 5.65m: PL, sh, pl, ro, stn, fe 5.83m: PL, sh, pl, ro, vn clay 5.86m: PL, sh, pl, ro, un, clay				PL(A) = 1.08 PL(D) = 0.44 PL(A) = 1.06

RIG: Nissan Patrol Mounted Drill Rig **DRILLER:** FICO **LOGGED:** Parkinson **CASING:** HQ to 4.5m

TYPE OF BORING: Solid flight auger vbit to 1.1m refusal, TC bit to 4.5m, NMLC core to 7.50m

WATER OBSERVATIONS: No free groundwater in top 4.5m, observations obscured below due to drilling fluids

REMARKS: Vbit refusal at 1.1m on probable ironstained band. () strength inferred from drilling resistance. *RLs interpolated from client supplied plan to nearest 0.5m

SAMPLING & IN SITU TESTING LEGEND

A	Auger sample	G	Gas sample	PID	Photo ionisation detector (ppm)
B	Bulk sample	P	Piston sample	PL(A)	Point load axial test Is(50) (MPa)
BLK	Block sample	U_x	Tube sample (x mm dia.)	PL(D)	Point load diametral test Is(50) (MPa)
C	Core drilling	W	Water sample	pp	Pocket penetrometer (kPa)
D	Disturbed sample	▷	Water seep	S	Standard penetration test
E	Environmental sample	▼	Water level	V	Shear vane (kPa)

Douglas Partners
Geotechnics / Environment / Groundwater

BOREHOLE LOG

CLIENT: Lend Lease Building Pty Ltd
PROJECT: Proposed Aged Care Development
LOCATION: Scaysbrook Drive, Kincumber

SURFACE LEVEL: 40.5 AHD*
EASTING: 351920
NORTHING: 6295741
DIP/AZIMUTH: 90°/--

BORE No: BH2
PROJECT No: 91006.00
DATE: 24/10/2016
SHEET 3 OF 3

RL	Depth (m)	Description of Strata	Degree of Weathering (EW HW MW SW FS FR)	Graphic Log	Rock Strength (Ex Low, Very Low, Low, Medium, High, Very High, Ex High)	Water	Fracture Spacing (m) (0.01, 0.05, 0.10, 0.50, 1.00)	Discontinuities (B - Bedding, J - Joint, S - Shear, F - Fault)	Type	Core Rec. %	RQD %	Test Results & Comments
		SANDSTONE - High strength, moderately weathered grey stained orange red, fine to medium grained sandstone, some sub horizontal ironstained healed partings, fractured to slightly fractured *(continued)*						6.18m: PL, sh, pl, ro, stn, fe				
		From 6.23m, slightly weathered										PL(A) = 0.06
		From 6.30m, very low strength										
		From 6.40m to 6.55m, extremely low strength						6.42m: PT, sh, ti, fe, stn, fe 6.44m: PL, ir, ro, stn, fe				
		From 6.55m, medium strength						6.62m: PL, sh, ti, he, stn, fe	C	100	81	
	7							6.89m: PL, sh, pl, ro, stn, fe				PL(A) = 0.82 PL(D) = 0.64
								7.02m: J, 20°, ir, ro, stn, fe				
	7.1	SILTSTONE - Medium strength, slightly weathered, dark grey siltstone, slightly fractured						7.05m: PT, sh, pl, ti, he, stn, fe				PL(A) = 0.35
	7.5	Bore discontinued at 7.5m , limit of investigation						7.47m: Cz, sh, pl, ti, he, stn, fe				
	8											

RIG: Nissan Patrol Mounted Drill Rig **DRILLER:** FICO **LOGGED:** Parkinson **CASING:** HQ to 4.5m
TYPE OF BORING: Solid flight auger vbit to 1.1m refusal, TC bit to 4.5m, NMLC core to 7.50m
WATER OBSERVATIONS: No free groundwater in top 4.5m, observations obscured below due to drilling fluids
REMARKS: Vbit refusal at 1.1m on probable ironstained band. () strength inferred from drilling resistance. *RLs interpolated from client supplied plan to nearest 0.5m

SAMPLING & IN SITU TESTING LEGEND

A	Auger sample	G	Gas sample	PID	Photo ionisation detector (ppm)
B	Bulk sample	P	Piston sample	PL(A)	Point load axial test Is(50) (MPa)
BLK	Block sample	U_x	Tube sample (x mm dia.)	PL(D)	Point load diametral test Is(50) (MPa)
C	Core drilling	W	Water sample	pp	Pocket penetrometer (kPa)
D	Disturbed sample	▷	Water seep	S	Standard penetration test
E	Environmental sample	▼	Water level	V	Shear vane (kPa)

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Geotechnics / Environment / Groundwater

BOREHOLE LOG

CLIENT: Lend Lease Building Pty Ltd
PROJECT: Proposed Aged Care Development
LOCATION: Scaysbrook Drive, Kincumber

SURFACE LEVEL: 40.0 AHD*
EASTING: 351953
NORTHING: 6295697
DIP/AZIMUTH: 90°/--

BORE No: BH3
PROJECT No: 91006.00
DATE: 25/10/2016
SHEET 1 OF 2

RL	Depth (m)	Description of Strata	Degree of Weathering (EW HW MW SW FS FR)	Graphic Log	Rock Strength (Ex Low, Very Low, Low, Medium, High, Very High, Ex High)	Water	Fracture Spacing (m) (0.01, 0.05, 0.10, 0.50, 1.00)	Discontinuities (B - Bedding, J - Joint, S - Shear, F - Fault)	Type	Core Rec. %	RQD %	Test Results & Comments
	0.05	TOPSOIL - Generally comprising dark grey clayey silt topsoil, some rootlets, moist										
		SANDY CLAY - Very stiff, light grey mottled orange red, fine to medium grained sandy clay, M<Wp							U_{50}			
	1								S			pp = 350 8,13,9 N = 22
	2											
	2.4	SANDSTONE - Extremely low strength, extremely weathered, grey stained orange red, fine to medium grained sandstone							S			25/140mm refusal
	2.8							2.86m: PT, sh, pl, ro, stn, fe 2.93m: Cs, sh, pl, inf,	C	100	57	

RIG: Nissan Patrol Mounted Drill Rig **DRILLER:** FICO **LOGGED:** Parkinson **CASING:** HQ to 2.8m
TYPE OF BORING: Solid flight auger vbit to 2.4m refusal, TC bit to 2.8m, NMLC core to 5.0m
WATER OBSERVATIONS: No free groundwater in top 2.8m, observations obscured below 2.8m due to drilling fluids
REMARKS: *RLs interpolated from client supplied plan to nearest 0.5m

SAMPLING & IN SITU TESTING LEGEND

A	Auger sample	G	Gas sample	PID	Photo ionisation detector (ppm)
B	Bulk sample	P	Piston sample	PL(A)	Point load axial test Is(50) (MPa)
BLK	Block sample	U_x	Tube sample (x mm dia.)	PL(D)	Point load diametral test Is(50) (MPa)
C	Core drilling	W	Water sample	pp	Pocket penetrometer (kPa)
D	Disturbed sample	▷	Water seep	S	Standard penetration test
E	Environmental sample	▼	Water level	V	Shear vane (kPa)

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BOREHOLE LOG

CLIENT: Lend Lease Building Pty Ltd
PROJECT: Proposed Aged Care Development
LOCATION: Scaysbrook Drive, Kincumber

SURFACE LEVEL: 40.0 AHD*
EASTING: 351953
NORTHING: 6295697
DIP/AZIMUTH: 90°/--

BORE No: BH3
PROJECT No: 91006.00
DATE: 25/10/2016
SHEET 2 OF 2

RL	Depth (m)	Description of Strata	Discontinuities (B - Bedding, J - Joint, S - Shear, F - Fault)	Type	Core Rec. %	RQD %	Test Results & Comments
		SANDSTONE - Medium strength, moderately weathered, grey stained orange red, fine to medium grained sandstone, with some dark grey siltstone bands fractured *(continued)* From 3.0m to 3.11m, dark grey siltstone band From 3.24m to 3.39m, dark grey siltstone band	5mm clay 3.11m: PT, sh, pl, stn, fe 3.17m: PT, sh, pl, ro, stn, fe 3.24m: PT, sh, pl, ro, stn, fe From 3.42m to 3.55m, Cs, 45°, pl, inf, 10mm clay	C	100	57	PL(A) = 0.3 PL(A) = 0.73
	3.55	CLAYSTONE - Extremely low strength, extremely weathered grey claystone					
	3.65	SILTSTONE - Medium strength, slightly weathered, dark grey siltstone	3.68m: PT, sh, pl, ro				PL(A) = 0.46
	4		3.94m: PT, sh, pl, ro, stn, fe 4.02m: PT, sh, pl, ro, stn, fe 4.19m: PT, sh, pl, ro, stn, fe 4.34m: J, 20°, pl, ro, stn, fe 4.46m: PT, sh, pl, ro, stn, fe 4.55m: PT, sh, pl, ti, he, stn, fe	C	100	86	PL(A) = 0.93
	4.7	SANDSTONE - Medium strength, slightly weathered, grey, fine to medium grained sandstone, fractured to slightly fractured	4.67m: Cs, 10°, pl, inf, 10mm clay				PL(A) = 0.52
	5 5.0	Bore discontinued at 5.0m , limit of investigation					

RIG: Nissan Patrol Mounted Drill Rig **DRILLER:** FICO **LOGGED:** Parkinson **CASING:** HQ to 2.8m

TYPE OF BORING: Solid flight auger vbit to 2.4m refusal, TC bit to 2.8m, NMLC core to 5.0m

WATER OBSERVATIONS: No free groundwater in top 2.8m, observations obscured below 2.8m due to drilling fluids

REMARKS: *RLs interpolated from client supplied plan to nearest 0.5m

SAMPLING & IN SITU TESTING LEGEND

A	Auger sample	G	Gas sample	PID	Photo ionisation detector (ppm)
B	Bulk sample	P	Piston sample	PL(A)	Point load axial test Is(50) (MPa)
BLK	Block sample	U_x	Tube sample (x mm dia.)	PL(D)	Point load diametral test Is(50) (MPa)
C	Core drilling	W	Water sample	pp	Pocket penetrometer (kPa)
D	Disturbed sample	▷	Water seep	S	Standard penetration test
E	Environmental sample	▼	Water level	V	Shear vane (kPa)

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Geotechnics / Environment / Groundwater

BOREHOLE LOG

CLIENT: Lend Lease Building Pty Ltd
PROJECT: Proposed Aged Care Development
LOCATION: Scaysbrook Drive, Kincumber

SURFACE LEVEL: 39.5 AHD*
EASTING: 351972
NORTHING: 6295654
DIP/AZIMUTH: 90°/--

BORE No: BH4
PROJECT No: 91006.00
DATE: 24/10/2016
SHEET 1 OF 1

RL	Depth (m)	Description of Strata	Graphic Log	Sampling & In Situ Testing: Type	Depth	Sample	Results & Comments	Water	Well Construction Details
	0.06	FILLING - Pavers		D	0.1				
	0.2	FILLING - Generally comprising brown, fine to medium grained sand filling, some fine to medium sized gravel, humid to moist							
	0.35	FILLING - Generally comprising brown orange, fine to medium grained sand filling, slightly cemented		D	0.5				
		CLAY - Very stiff, orange brown clay, some fine to medium grained sand and some silt, M≫Wp		U_{50}	0.5–0.7		pp = 480		
	0.7	SANDSTONE - Extremely low strength, extremely weathered, grey stained orange, fine to medium grained sandstone, some (low strength) ironstained bands, M<Wp (soil like properties)							
1				S	1.0–1.36		pp = 600 11,11,20/60mm refusal		
		From 1.6m, increased drilling resistance							
2	2.0	Bore discontinued at 2.0m , slow progress TC bit							
3									
4									

RIG: Nissan Patrol Mounted Drill Rig **DRILLER:** FICO (Currie) **LOGGED:** Parkinson **CASING:** Nil
TYPE OF BORING: Solid flight auger v-bit to 0.8m, TC bit to 2.0m
WATER OBSERVATIONS: No free groundwater obserbed
REMARKS: ( ) Strength inferred from drilling resistance. *RLs interpolated from client supplied plan to nearest 0.5m.

SAMPLING & IN SITU TESTING LEGEND

A	Auger sample	G	Gas sample	PID	Photo ionisation detector (ppm)
B	Bulk sample	P	Piston sample	PL(A)	Point load axial test Is(50) (MPa)
BLK	Block sample	U_x	Tube sample (x mm dia.)	PL(D)	Point load diametral test Is(50) (MPa)
C	Core drilling	W	Water sample	pp	Pocket penetrometer (kPa)
D	Disturbed sample	▷	Water seep	S	Standard penetration test
E	Environmental sample	▼	Water level	V	Shear vane (kPa)

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Geotechnics / Environment / Groundwater

BOREHOLE LOG

CLIENT: Lend Lease Building Pty Ltd
PROJECT: Proposed Aged Care Development
LOCATION: Scaysbrook Drive, Kincumber

SURFACE LEVEL: 43.5 AHD*
EASTING: 351970
NORTHING: 6295746
DIP/AZIMUTH: 90°/--

BORE No: BH5
PROJECT No: 91006.00
DATE: 25/10/2016
SHEET 1 OF 1

Depth (m)	Description of Strata	Sampling Type	Sampling Depth	Results & Comments
0.1	FILLING - Generally comprising brown grey, fine to medium grained sandy clay filling, with some medium to coarse sized subangular gravel			
1.5	CLAY - Stiff, orange brown clay, M>Wp	U	0.5–0.7	pp = 190
	From 1.0m, very stiff light grey mottled orange, M≫Wp	S	1.0–1.45	pp = 380-400 2,4,7 N = 11
	From 1.4m, slightly fine to medium grained sandy			
2.9	SANDSTONE - (Extremely low to very low strength) grey stained red, fine to medium grained sandstone, with some clay bands, probable low strength ironstained bands	S	2.5–2.73	28,25/80mm refusal
4.1	SANDSTONE - (Medium strength) orange brown, fine to medium grained sandstone, some red ironstaining	D	3.5	
5.0	SILTSTONE - (Medium strength) dark grey siltstone	D	4.5	

Bore discontinued at 5.0m , limit of investigation

RIG: Nissan Patrol Mounted Drill Rig **DRILLER:** FICO (Currie) **LOGGED:** Parkinson **CASING:** Nil

TYPE OF BORING: Solid flight auger v-bit to 1.5m refusal, TC bit to 5.0m

WATER OBSERVATIONS: No free groundwater obserbed

REMARKS: ( ) Strength inferred from drilling resistance. *RLs interpolated from client supplied plan to nearest 0.5m.

SAMPLING & IN SITU TESTING LEGEND

A	Auger sample	G	Gas sample	PID	Photo ionisation detector (ppm)
B	Bulk sample	P	Piston sample	PL(A)	Point load axial test Is(50) (MPa)
BLK	Block sample	U_x	Tube sample (x mm dia.)	PL(D)	Point load diametral test Is(50) (MPa)
C	Core drilling	W	Water sample	pp	Pocket penetrometer (kPa)
D	Disturbed sample	▷	Water seep	S	Standard penetration test
E	Environmental sample	▼	Water level	V	Shear vane (kPa)

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Geotechnics / Environment / Groundwater

BOREHOLE LOG

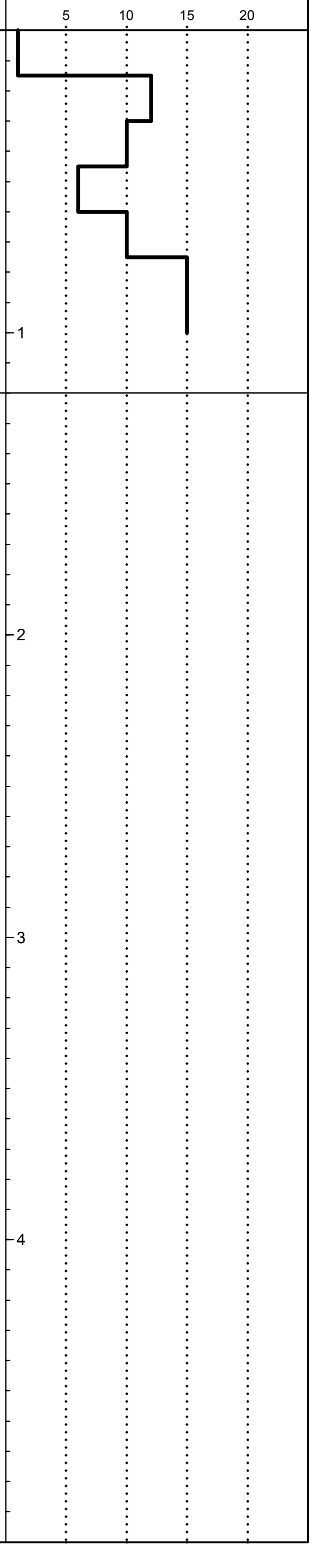
CLIENT: Lend Lease Building Pty Ltd
PROJECT: Proposed Aged Care Development
LOCATION: Scaysbrook Drive, Kincumber

SURFACE LEVEL: 38.0 AHD*
EASTING: 351940
NORTHING: 6295669
DIP/AZIMUTH: 90°/--

BORE No: BH6
PROJECT No: 91006.00
DATE: 25/10/2016
SHEET 1 OF 1

RL	Depth (m)	Description of Strata	Graphic Log	Sampling & In Situ Testing: Type	Depth	Sample	Results & Comments	Water
	0.06	FILLING - Generally comprising pavers						
	0.2	FILLING - Generally comprising brown, fine to medium grained sand filling, moist From 0.1m, grey with some gravel						
	0.45	FILLING - Generally comprising dark grey gravelly sandy clay filling						
		CLAY - Stiff brown orange clay, M>Wp		B	0.5 0.6 0.7		pp = 180	
	1				1.0		pp = 180	
	1.2	Bore discontinued at 1.2m , limit of investigation						
	2							
	3							
	4							

Dynamic Penetrometer Test (blows per 150mm)



RIG: Nissan Patrol Mounted Drill Rig **DRILLER:** FICO (Currie) **LOGGED:** Parkinson **CASING:** Nil
TYPE OF BORING: Solid flight auger 250mm diameter
WATER OBSERVATIONS: No free groundwater obserbed
REMARKS: *RLs interpolated from client supplied plan to nearest 0.5m.

SAMPLING & IN SITU TESTING LEGEND

A	Auger sample	G	Gas sample	PID	Photo ionisation detector (ppm)
B	Bulk sample	P	Piston sample	PL(A)	Point load axial test Is(50) (MPa)
BLK	Block sample	U_x	Tube sample (x mm dia.)	PL(D)	Point load diametral test Is(50) (MPa)
C	Core drilling	W	Water sample	pp	Pocket penetrometer (kPa)
D	Disturbed sample	▷	Water seep	S	Standard penetration test
E	Environmental sample	▼	Water level	V	Shear vane (kPa)

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BOREHOLE LOG

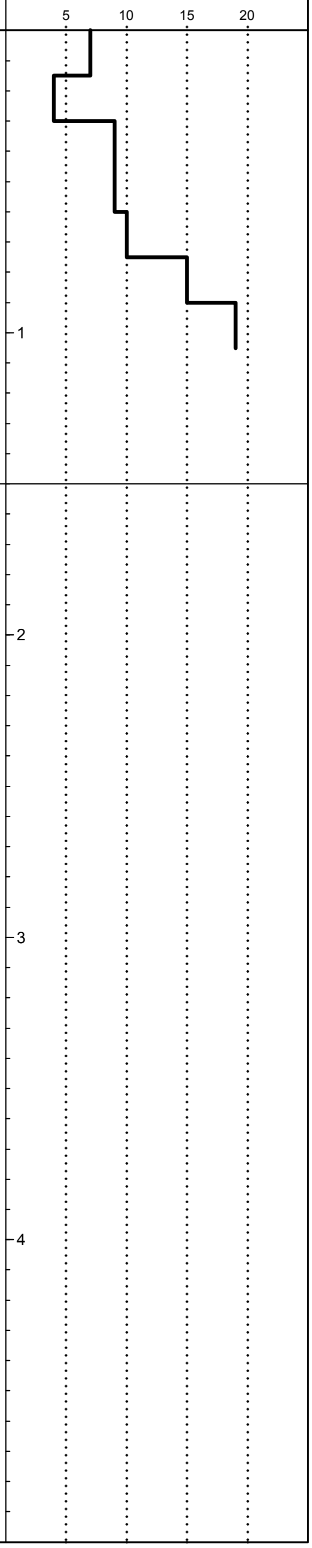
CLIENT: Lend Lease Building Pty Ltd
PROJECT: Proposed Aged Care Development
LOCATION: Scaysbrook Drive, Kincumber

SURFACE LEVEL: 35.0 AHD*
EASTING: 351902
NORTHING: 629704
DIP/AZIMUTH: 90°/--

BORE No: BH7
PROJECT No: 91006.00
DATE: 25/10/2016
SHEET 1 OF 1

RL	Depth (m)	Description of Strata	Graphic Log	Sampling & In Situ Testing: Type	Depth	Sample	Results & Comments	Water
	0.06	FILLING - Generally comprising pavers		D	0.1			
	0.1	FILLING - Generally comprising brown, fine to medium grained sand filling						
	0.2	FILLING - Generally comprising dark grey gravelly sandy clay filling						
		CLAY - Stiff, orange mottled red clay, M>Wp			0.5		pp = 180	
				B				
					0.8			
	1	From 1.0m, very stiff, light grey mottled orange red, trace coarse ironstone gravel and cobbles		D	1.0		pp = 220	
		At 1.2m, ironstone cobble band		B				
					1.3			
	1.5	Bore discontinued at 1.5m , slow progress on probable ironstone cobble band						
	2							
	3							
	4							

Dynamic Penetrometer Test (blows per 150mm)



RIG: Nissan Patrol Mounted Drill Rig **DRILLER:** FICO (Currie) **LOGGED:** Parkinson **CASING:** Nil
TYPE OF BORING: Solid flight auger 250mm diameter
WATER OBSERVATIONS: No free groundwater obserbed
REMARKS: *RLs interpolated from client supplied plan to nearest 0.5m.

SAMPLING & IN SITU TESTING LEGEND

A	Auger sample	G	Gas sample	PID	Photo ionisation detector (ppm)
B	Bulk sample	P	Piston sample	PL(A)	Point load axial test Is(50) (MPa)
BLK	Block sample	U_x	Tube sample (x mm dia.)	PL(D)	Point load diametral test Is(50) (MPa)
C	Core drilling	W	Water sample	pp	Pocket penetrometer (kPa)
D	Disturbed sample	▷	Water seep	S	Standard penetration test
E	Environmental sample	▼	Water level	V	Shear vane (kPa)

Douglas Partners
Geotechnics / Environment / Groundwater

BOREHOLE LOG

CLIENT: Lend Lease Building Pty Ltd
PROJECT: Proposed Aged Care Development
LOCATION: Scaysbrook Drive, Kincumber

SURFACE LEVEL: 36.5 AHD*
EASTING: 351883
NORTHING: 6295739
DIP/AZIMUTH: 90°/--

BORE No: BH8
PROJECT No: 91006.00
DATE: 25/10/2016
SHEET 1 OF 2

RL	Depth (m)	Description of Strata	Graphic Log	Sampling & In Situ Testing: Type	Depth	Sample	Results & Comments	Water	Well Construction Details
	0.06	FILLING - Generally comprising pavers							
	0.1	FILLING - Generally comprising brown, fine to medium grained sand, moist		D	0.1				
		FILLING - Generally comprising dark grey sandy gravel filling, moist		D	0.2				
	0.5	FILLING - Generally comprising dark grey silty clay filling, trace fine to medium sized gravel		D	0.5				
	1			S	1.0–1.45		pp = 80 2,2,2 N = 4		1
	2								2
	2.1	CLAY - Very stiff orange brown clay, some silt, trace fine to medium grained sand, M>Wp							
				S	2.5–2.95		pp = 350 4,7,9 N = 16		
	3								3
	4	From 4.0m, grey mottled red some ironstained gravel, M ≫ Wp		S	4.0–4.45		pp = 380-400 5,13,16 N = 29		4

RIG: Nissan Patrol Mounted Drill Rig **DRILLER:** FICO (Currie) **LOGGED:** Parkinson **CASING:** Nil
TYPE OF BORING: Solid flight auger v-bit to 6.0m refusal, TC bit to 7m
WATER OBSERVATIONS: Trace seepage observed at 6.5m, whilst bore remained open
REMARKS: ( ) Strength inferred from drilling resistance. *RLs interpolated from client supplied plan to nearest 0.5m.

SAMPLING & IN SITU TESTING LEGEND

A	Auger sample	G	Gas sample	PID	Photo ionisation detector (ppm)
B	Bulk sample	P	Piston sample	PL(A)	Point load axial test Is(50) (MPa)
BLK	Block sample	U_x	Tube sample (x mm dia.)	PL(D)	Point load diametral test Is(50) (MPa)
C	Core drilling	W	Water sample	pp	Pocket penetrometer (kPa)
D	Disturbed sample	▷	Water seep	S	Standard penetration test
E	Environmental sample	▼	Water level	V	Shear vane (kPa)

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BOREHOLE LOG

CLIENT: Lend Lease Building Pty Ltd
PROJECT: Proposed Aged Care Development
LOCATION: Scaysbrook Drive, Kincumber

SURFACE LEVEL: 36.5 AHD*
EASTING: 351883
NORTHING: 6295739
DIP/AZIMUTH: 90°/--

BORE No: BH8
PROJECT No: 91006.00
DATE: 25/10/2016
SHEET 2 OF 2

RL	Depth (m)	Description of Strata	Graphic Log	Sampling & In Situ Testing: Type	Depth	Sample	Results & Comments	Water	Well Construction Details
		CLAY - Very stiff orange brown clay, some silt, trace fine to medium grained sand, M>Wp *(continued)*							
		From 5.5m, hard		S	5.5–5.95		pp = 600 3,11,21 N = 32		
	6	From 6.0m, ironstained in parts (rock like properties)							
	7			S	7.0–7.22		14,25/70mm refusal		
	7.22	Bore discontinued at 7.22m , limit of investigation							
	8								
	9								

RIG: Nissan Patrol Mounted Drill Rig **DRILLER:** FICO (Currie) **LOGGED:** Parkinson **CASING:** Nil
TYPE OF BORING: Solid flight auger v-bit to 6.0m refusal, TC bit to 7m
WATER OBSERVATIONS: Trace seepage observed at 6.5m, whilst bore remained open
REMARKS: ( ) Strength inferred from drilling resistance. *RLs interpolated from client supplied plan to nearest 0.5m.

SAMPLING & IN SITU TESTING LEGEND

A	Auger sample	G	Gas sample	PID	Photo ionisation detector (ppm)
B	Bulk sample	P	Piston sample	PL(A)	Point load axial test Is(50) (MPa)
BLK	Block sample	U_x	Tube sample (x mm dia.)	PL(D)	Point load diametral test Is(50) (MPa)
C	Core drilling	W	Water sample	pp	Pocket penetrometer (kPa)
D	Disturbed sample	▷	Water seep	S	Standard penetration test
E	Environmental sample	▼	Water level	V	Shear vane (kPa)

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DOUGLAS PARTNERS PTY LTD

SCAYSBROOK DRIVE, KINCUMBER

BORE 2 PROJECT 91006.00 2016

4.50 m – 7.50 m

DOUGLAS PARTNERS PTY LTD

SCAYSBROOK DRIVE, KINCUMBER

BORE 3 PROJECT 91006.00 2016

2.8 m – 5.0 m

Douglas Partners Geotechnics / Environment / Groundwater	Proposed Aged Care Development Scaysbrook Drive, Kincumber	PROJECT:	91006.00
		PLATE No:	1
		REV:	0
	CLIENT: Lend Lease Building Pty Ltd	DATE:	15-Nov-16

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Results of Dynamic Penetrometer Tests

Client	Lend Lease Building Pty Ltd	**Project No.**	91006.00
Project	Proposed Aged Care Development	**Date**	25/10/16
Location	Scaysbrook Drive, Kincumber	**Page No.**	1 of 1

Test Location	6	7									
RL of Test (AHD)											
Depth (m)	**Penetration Resistance** Blows/150 mm										
0 - 0.15	1	7									
0.15 - 0.30	12	4									
0.30 - 0.45	10	9									
0.45 - 0.60	6	9									
0.60 - 0.75	10	10									
0.75 - 0.90	15	15									
0.90 - 1.05	15/100 mm	19									
1.05 - 1.20											
1.20 - 1.35											
1.35 - 1.50											
1.50 - 1.65											
1.65 - 1.80											
1.80 - 1.95											
1.95 - 2.10											
2.10 - 2.25											
2.25 - 2.40											
2.40 - 2.55											
2.55 - 2.70											
2.70 - 2.85											
2.85 - 3.00											
3.00 - 3.15											
3.15 - 3.30											
3.30 - 3.45											
3.45 - 3.60											

Test Method	AS 1289.6.3.2, Cone Penetrometer	☑	**Tested By**	MJP
	AS 1289.6.3.3, Sand Penetrometer	☐	**Checked By**	MJP

Remarks Ref = Refusal, 24/110 indicates 25 blows for 110 mm penetration

Appendix C

Laboratory Test Results

Material Test Report

Report Number: **91006.00-1**
Issue Number: 1
Date Issued: 09/11/2016
Client: Lend Lease Building Pty Ltd
Level 4, Millers Point NSW 2000
Project Number: 91006.00
Project Name: Proposed Aged Care Development
Project Location: Scaysbrook Drive, Kincumber
Work Request: 221
Sample Number: 16-221A
Date Sampled: 25/10/2016
Sampling Method: Sampled by Engineering Department
Sample Location: **Bore 3 (0.50 - 0.75m)**
Material: Sandy CLAY - Light grey mottled orange red

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NATA
WORLD RECOGNISED ACCREDITATION

Accredited for compliance with ISO/IEC 17025 - Testing

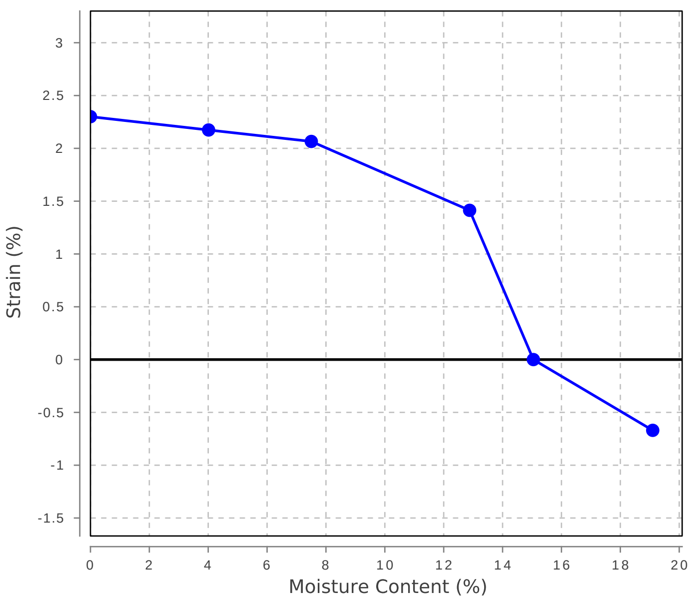
Approved Signatory: Dave Millard
Nata Accredited Laboratory Number: 828

Shrink Swell Index (AS 1289 7.1.1 & 2.1.1)	
Iss (%)	**1.5**
Visual Description	Sandy CLAY - Light grey mottled orange red
* Shrink Swell Index (Iss) reported as the percentage vertical strain per pF change in suction.	

Core Shrinkage Test	
Shrinkage Strain - Oven Dried (%)	**2.3**
Estimated % by volume of significant inert inclusions	5
Cracking	Moderately Cracked
Crumbling	Yes
Moisture Content (%)	15.0

Swell Test	
Initial Pocket Penetrometer (kPa)	>600
Final Pocket Penetrometer (kPa)	510
Initial Moisture Content (%)	17.4
Final Moisture Content (%)	19.1
Swell (%)	**0.7**
* NATA Accreditation does not cover the performance of pocket penetrometer readings.	

Shrink Swell



Material Test Report

Report Number: **91006.00-1**
Issue Number: 1
Date Issued: 09/11/2016
Client: Lend Lease Building Pty Ltd
Level 4, Millers Point NSW 2000
Project Number: 91006.00
Project Name: Proposed Aged Care Development
Project Location: Scaysbrook Drive, Kincumber
Work Request: 221
Sample Number: 16-221B
Date Sampled: 25/10/2016
Sampling Method: Sampled by Engineering Department
Sample Location: **Bore 5 (0.50 - 0.70m)**
Material: CLAY - Orange brown

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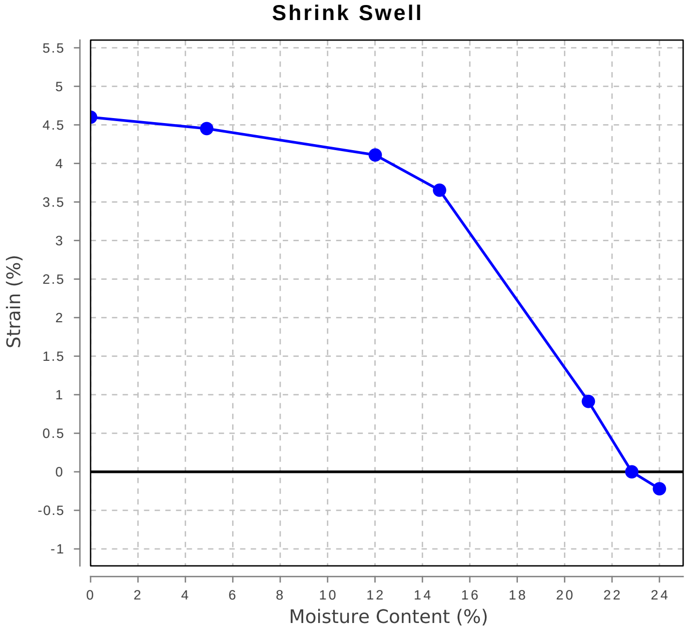
Nata Accredited Laboratory Number: 828

Shrink Swell Index (AS 1289 7.1.1 & 2.1.1)	
Iss (%)	**2.6**
Visual Description	CLAY - Orange brown
* Shrink Swell Index (Iss) reported as the percentage vertical strain per pF change in suction.	

Core Shrinkage Test	
Shrinkage Strain - Oven Dried (%)	**4.6**
Estimated % by volume of significant inert inclusions	0
Cracking	Slightly Cracked
Crumbling	No
Moisture Content (%)	22.8

Swell Test	
Initial Pocket Penetrometer (kPa)	350
Final Pocket Penetrometer (kPa)	260
Initial Moisture Content (%)	23.0
Final Moisture Content (%)	24.0
Swell (%)	**0.2**
* NATA Accreditation does not cover the performance of pocket penetrometer readings.	



Material Test Report

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NATA WORLD RECOGNISED ACCREDITATION

Approved Signatory: Dave Millard
Nata Accredited Laboratory Number: 828

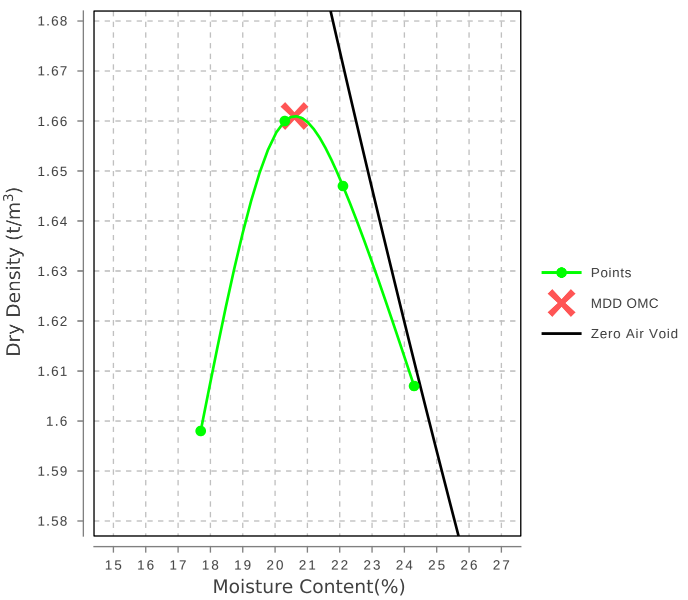
Report Number:	**91006.00-1**
Issue Number:	1
Date Issued:	09/11/2016
Client:	Lend Lease Building Pty Ltd Level 4, Millers Point NSW 2000
Project Number:	91006.00
Project Name:	Proposed Aged Care Development
Project Location:	Scaysbrook Drive, Kincumber
Work Request:	221
Sample Number:	16-221C
Date Sampled:	25/10/2016
Sampling Method:	Sampled by Engineering Department
Sample Location:	**Bore 6 (0.50 - 0.70m)**
Material:	CLAY - Brown orange

Moisture Content (AS 1289 2.1.1)	
Moisture Content (%)	24.6

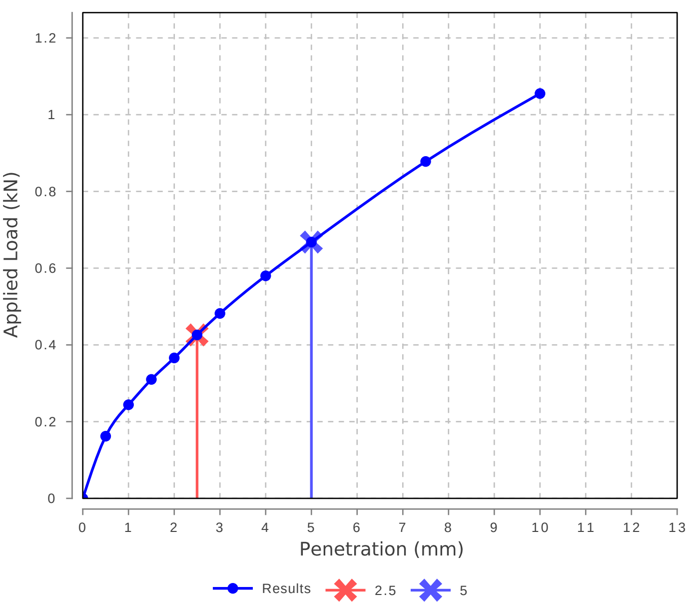
Moisture Density Relationship (AS 1289 5.1.1 & 2.1.1)	
Mould Type	1 LITRE MOULD A
Compaction	Standard
No. Layers	3
No. Blows / Layer	25
Maximum Dry Density (t/m^3)	1.66
Optimum Moisture Content (%)	20.5
Oversize Material (%)	0

California Bearing Ratio (AS 1289 6.1.1 & 2.1.1)		Min	Max
CBR taken at	5 mm		
CBR %	**3.5**		
Method of Compactive Effort	Standard		
Method used to Determine MDD	AS 1289 5.1.1 & 2.1.1		
Maximum Dry Density (t/m^3)	1.66		
Dry Density after Soaking (t/m^3)	1.71		
Optimum Moisture Content (%)	20.5		
Laboratory Moisture Ratio (%)	99.5		
Laboratory Density Ratio (%)	100.5		
Moisture Content at Placement (%)	20.5		
Moisture Content Top 30mm (%)	24.2		
Moisture Content Rest of Sample (%)	22.3		
Mass Surcharge (kg)	4.5		
Soaking Period (days)	4		
Swell (%)	2.0		
Oversize Material (mm)	19		
Oversize Material Included	Excluded		
Oversize Material (%)	0		

Moisture Density Relationship



California Bearing Ratio



Material Test Report

Report Number:	**91006.00-1**
Issue Number:	1
Date Issued:	09/11/2016
Client:	Lend Lease Building Pty Ltd
	Level 4, Millers Point NSW 2000
Project Number:	91006.00
Project Name:	Proposed Aged Care Development
Project Location:	Scaysbrook Drive, Kincumber
Work Request:	221
Sample Number:	16-221D
Date Sampled:	25/10/2016
Sampling Method:	Sampled by Engineering Department
Sample Location:	**Bore 7 (1.00 - 1.30m)**
Material:	CLAY - Light grey mottled orange red

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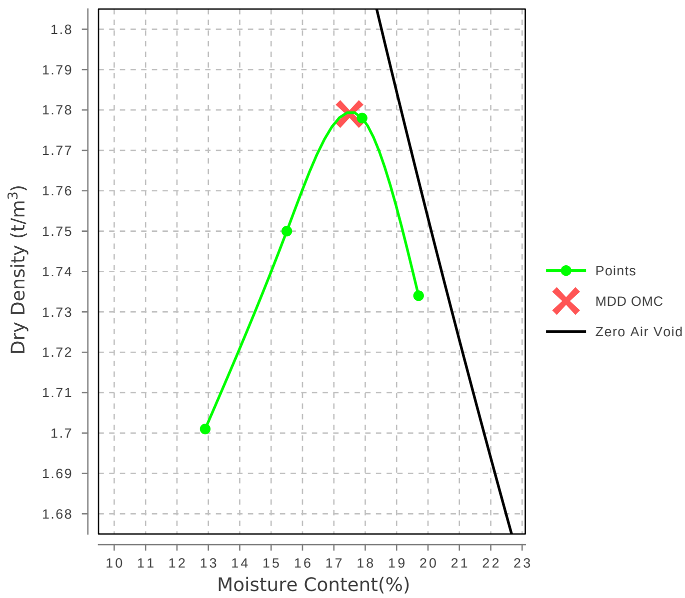
Moisture Content (AS 1289 2.1.1)	
Moisture Content (%)	18.2

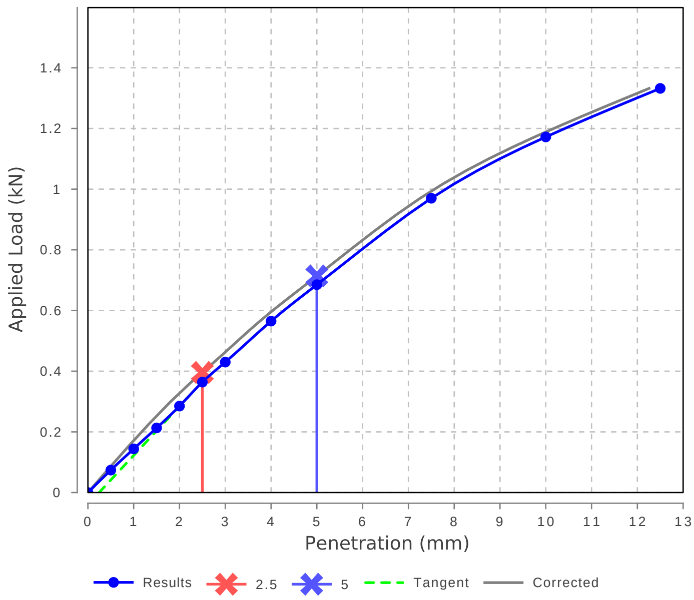
Moisture Density Relationship (AS 1289 5.1.1 & 2.1.1)	
Mould Type	1 LITRE MOULD A
Compaction	Standard
No. Layers	3
No. Blows / Layer	25
Maximum Dry Density (t/m^3)	1.78
Optimum Moisture Content (%)	17.5
Oversize Material (%)	7

California Bearing Ratio (AS 1289 6.1.1 & 2.1.1)		Min	Max
CBR taken at	5 mm		
CBR %	**3.5**		
Method of Compactive Effort	Standard		
Method used to Determine MDD	AS 1289 5.1.1 & 2.1.1		
Maximum Dry Density (t/m^3)	1.78		
Dry Density after Soaking (t/m^3)	1.82		
Optimum Moisture Content (%)	17.5		
Laboratory Moisture Ratio (%)	99.5		
Laboratory Density Ratio (%)	100.0		
Moisture Content at Placement (%)	17.4		
Moisture Content Top 30mm (%)	22.2		
Moisture Content Rest of Sample (%)	19.1		
Mass Surcharge (kg)	4.5		
Soaking Period (days)	4		
Swell (%)	2.5		
Oversize Material (mm)	19		
Oversize Material Included	Excluded		
Oversize Material (%)	7		

Moisture Density Relationship



California Bearing Ratio



Appendix D

Drawing 1 – Test Location Plan
Drawing 2 – Proposed Development

Locality Plan

0 10 20 30 40 50 75 100m

1:1000 @ A3

NOTES

1. Drawing adapted from Nearmap Image dated May 2016
2. Test locations are approximate only and are shown with reference to existing site features.

LEGEND

- Borehole Location
- Ob1 Observation Points
- Approximate Carpark and Access Road Location
- Proposed 6m High Retaining Wall
- Site Boundary

Douglas Partners
Geotechnics / Environment / Groundwater

CLIENT: Lend Lease Building Pty Ltd		TITLE: **Test Location Plan**	PROJECT No: 91006.00
OFFICE: Newcastle	DRAWN BY: PLH	**Proposed Aged Care Development**	DRAWING No: 1
SCALE: As shown	DATE: 15.11.2016	**Scaysbrook Drive, Kincumber**	REVISION: 0

NOTE: Base drawing from plan provided by client

Douglas Partners Geotechnics / Environment / Groundwater	**Proposed Development**	PROJECT No: 91006.00
	Proposed Aged Care Development	DRAWING No: 2
	Scaysbrook Drive, Kincumber	REVISION: 0
	CLIENT: Lend Lease Building Pty Ltd	DATE: 15.11.2016