

**PROPOSED HMRI BUILDINGS,
JOHN HUNTER HOSPITAL -
GEOTECHNICAL ASSESSMENT**

APP Corporation

GEOTWARA20576AB-AC
3 June 2009

3 June 2009

APP Corporation
PO Box 1285
NEWCASTLE NSW 2300

Attention: Stuart Diver

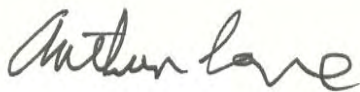
Dear Stuart

**RE: PROPOSED HMRI BUILDINGS
JOHN HUNTER HOSPITAL, NEW LAMBTON HEIGHTS
GEOTECHNICAL ASSESSMENT**

Please find enclosed our final report on the above project.

If you have any further questions regarding this or any other project, please do not hesitate to contact Gavin Tippet or the undersigned.

For and on behalf of Coffey Geotechnics Pty Ltd



Arthur Love

Principal Geotechnical Engineer

CONTENTS

1	INTRODUCTION	1
2	FIELD WORK	1
3	SITE CONDITIONS	2
3.1	Site Location and History	2
3.2	Site Topography and Description	3
3.2.1	Proposed Surgical Skills Training Building	3
3.3	Geology	3
3.4	Subsurface Conditions	4
4	LABORATORY TESTING	6
5	EARTHWORKS AND SITE PREPARATION	7
5.1	General Site Preparation	7
5.2	Excavation Conditions	8
5.3	Suitability of Materials for Reuse	9
5.4	Trafficability	10
5.5	Stability Parameters for Earthworks and Retention of Permanent Excavations	10
6	FOUNDATIONS	11
6.1	Foundation Design Parameters	11
6.2	Site Classification	13
6.3	Site Seismicity	14
7	PAVEMENTS	14
7.1	Design Traffic Loading	14
7.2	Design CBR	14
7.3	Flexible Pavement Design	14
7.4	Drainage	15

CONTENTS

8	SLOPE STABILITY ASSESSMENT	15
8.1	Background Information	15
8.2	Risk of Slope Instability	15
8.3	Recommended Geotechnical Constraints for Development	16
8.4	Summary	17
9	CONSTRUCTION RISK	18
	REFERENCES	18

CSIRO Sheet BTF-18

Attachments 1, 2 and 3

Important Information About Your Coffey Report

Figures

Figure AC1: Borehole Location Plan

Figure AC2: Section A – A

Figure AC3: Section B – B

Appendices

Appendix A: Results of Field Investigations

Appendix B: Results of Laboratory Testing

Appendix C: Pavement Thickness Design Summary

Appendix D: Engineering Logs from Previous Investigations

1 INTRODUCTION

This report presents the results of a geotechnical and environmental investigation carried out by Coffey Geotechnics Pty Ltd (Coffey) for the proposed new Hunter Medical Research Institute (HMRI) buildings, to be located at John Hunter Hospital, New Lambton Heights.

This report was undertaken by Coffey Geotechnics Pty Ltd (Coffey), on behalf of APP Corporation Pty Ltd, for two to three proposed HMRI Buildings, to be located at the John Hunter Hospital, New Lambton Heights. The work was commissioned by Stuart Diver of APP Corporation on 26 March 2009 on behalf of Hunter Medical Research Institute.

The proposed development is understood to consist of two to three HMRI buildings comprising four to five levels, paved car parking and construction of a surgical skills training building at the western end of the existing hospital lecture theatres'. The HMRI site is located east of the existing overflow car park, at the northern end of the John Hunter Hospital as shown on Figure 1.

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

- Site preparation;
- Excavation conditions;
- The suitability of the site soils for use as fill and on fill construction procedures;
- Alternative footing types and founding levels, including recommendations as to allowable bearing pressure and probable settlements;
- Risk of slope instability and associated geotechnical constraints;
- Special requirements for construction procedures and or site drainage.

A compilation of existing reports for work carried out by Coffey at the John Hunter Hospital site has previously been carried out for the proposed HMRI buildings, directly west of the current site location (Coffey Reference: GEOTWARA20576AA-AC, 31 January 2008). Relevant borehole and test pit data from previous studies have been used to supplement information from the current investigations where applicable.

The following report presents the results of field investigations and laboratory testing and provides discussion and recommendations relevant to the above scope of work.

2 FIELD WORK

Field work was carried out from 6 to 24 April 2009 and comprised of:

- Six boreholes (BH20 to BH25) drilled using a truck mounted rig (only used for BH22) or a trailer mounted rig (all other boreholes) equipped with continuous spiral flight augers and core barrel for coring in rock. Boreholes (excluding BH22) were drilled to V-Bit refusal on weathered rock at depths varying from 0.50m to 7.60m, and cored for additional depths of about 3m or until 2m of suitable founding material had been recovered. BH22 was drilled to V-bit refusal at a depth of 6.17m HW casing advanced to 15.31m with coring through the Borehole Seam coal to a depth of 92.74m for the purpose of mine subsidence assessment. Standard penetration tests were carried out at nominally 1.5m intervals;
- Thirteen test pits were (TP09-1 to TP09-13) excavated using a small rubber tracked excavator in the area of the proposed car park, with samples taken for subsequent laboratory testing;

- Four test pits were (TP09-14 to TP09-17) excavated using a small rubber tracked excavator in the area of the surgical skills training building;
- Observation and mapping of relevant site features;
- Borehole and test pit locations were pegged in the field to be accurately located and levelled by the Project Surveyor (CR Hutchison & Co Pty Ltd).

All field work was carried out in the full time presence of an Engineering Geologist from Coffey who located the boreholes, directed the sampling and testing, and produced field logs for the boreholes. The work was carried out in general accordance with the site specific Job Safety Analysis and Safe Work Procedure Statement (Safety Plans) prepared by Coffey, and site induction and safety briefing held prior to the commencement of site works.

Engineering logs of the boreholes and test pits are presented in Appendix A, together with explanation sheets defining the terms and symbols used in their preparation. Approximate borehole test locations are shown on Figure 2. Co-ordinate locations and reduced levels of the boreholes to Australian Height Datum (AHD) are given on the appended borehole logs.

3 SITE CONDITIONS

3.1 Site Location and History

The site is located at the John Hunter Hospital, off Lookout Road, New Lambton Heights. The area for the proposed development is situated on a ridge located east of the overflow car park situated at the northern end of John Hunter Hospital, refer Figure 1.

Historical information for the study area has been compiled from a Land Titles Search, interpretation of aerial photographs, site observations and a review of existing reports carried out at or nearby the site.

Results of the Land Titles Search indicate that the study area is located within a parcel of land identified as, Lot 13, DP876245, that was transferred to the Hunter Area Health Service on 15 April 2002.

Aerial photographs flown in 1979 were obtained from the Department of Lands, NSW and examined for any features which may affect the site. The approximate scale of the photographs was 1:16000. The following features were noted:

- John Hunter Hospital site comprised of undeveloped bushland, with only minor access tracks visible;
- A lineament running approximately east-west across the north-western end of the site was noted, previous investigations indicating this to be an old, disused and overgrown roadway, possibly associated with coal mining;
- Buildings associated with the Rankin Park Hospital adjacent to Lookout Road had been constructed.

Aerial photographs flown in 1998 indicate the main hospital, the overflow car park, the Club Med building and associated car park and tennis court, were constructed prior to this date, the Site remaining as undeveloped bush land, car park and Club Med building.

Review of available information indicate that all structures adjacent to and at the site are associated with the development of the John Hunter Hospital, with construction occurring over a period of time since the early to mid 1980's.

At the time of the fieldwork, the site appeared to be in similar condition as that shown in the 1998 photographs, with the exception of minor clearing associated with construction of walking paths, gravel access tracks, and installation of a sewer main and high voltage underground cable.

3.2 Site Topography and Description

The site is situated within an area of gently to moderately undulating topography, on the mid to upper northern and eastern facing slopes of a prominent north trending ridge. Topographic relief across the site varies from about RL 90m AHD adjacent to Kookaburra Crescent, to about RL 60m in the base of the existing gully / watercourse, located on the lower, eastern portion of the site.

Natural surface slopes are typically in the order of about 10° to 15°, with some localised steeper slopes along the side of the main gully to the east.

Drainage of the site is generally assessed to be via infiltration into the near surface soils and overland flow downslope towards the existing gullies on either side of the site. Concentrated flows were observed in the area directly to the north of the existing car park and have formed shallow erosion gullies through the loose soils into stiff clays; further concentrations of flows are likely to occur through the existing gullies, discharging to the north east.

The site is currently occupied by existing bush land, paved car park, a paved cycle and walking path and a brick clad building, Club Med. Some clearing has taken place associated with the construction of gravel access tracks and car park overflow, fire access tracks, and for services along the southern portion of the site.

Vegetation typically comprised of a moderate cover of mature eucalypts. Some areas of bare ground were observed associated with the site clearing noted above. Vegetation including tree cover, and an undergrowth of native shrubs, plants and grasses became denser on the downslope areas.

The main site features are shown on the detail survey plans provided by the project surveyors, CR Hutchison & Co Pty Ltd, which has been used as the basis for Figure 2 attached.

3.2.1 Proposed Surgical Skills Training Building

The site is located on a small grassed hillock between three existing buildings with elevations varying by approximately two meters over the site. The buildings to the north south and east of the site are all in good condition. Current drainage on site is by either overland flow or infiltration into the soils.

3.3 Geology

Reference to the BHP Newcastle Coalfields Surface Geology Map indicates the site is located in the Permian Aged Newcastle Coal Measures. The study area is judged to be underlain by the Merewether Conglomerate Member of the Kotara Formation, comprising of conglomerate, pebbly sandstone and sandstone overlying the Victoria Tunnel Seam and the Shepherds Hill Formation comprising coal, siltstone and sandstone.

The Borehole Seam, which has been extensively mined, underlies the site at about RL -20m AHD (i.e.: approximately 80m to 100m below the existing ground surface). The seam dips at approximately 2° to the south-west and has a thickness of approximately 3.0m.

3.4 Subsurface Conditions

The subsurface profiles encountered at borehole locations are presented on the appended engineering logs, with a summary of the interpreted profiles encountered presented in Table 1 on the basis of geotechnical units.

TABLE 1 – SUMMARY OF GEOTECHNICAL UNITS AT BOREHOLE LOCATIONS

UNIT	MATERIAL	MATERIAL DESCRIPTION
UNIT 1	FILL	Silty Sandy GRAVEL, fine to medium grained. Sand fine to medium grained, with some clay fines.
UNIT 2	TOPSOIL / SLOPEWASH	Silty Clayey SAND and Gravelly Sandy SILT. Sand fine to medium grained, clay of low to medium plasticity, gravel fine grained. With some root fibres.
UNIT 3	RESIDUAL	Sandy CLAY and CLAY, of medium to high plasticity. Sand fine to medium grained. Generally of very stiff to hard consistency. With some extremely weathered and carbonaceous coal, claystone and mudstone (excavating as a soil).
UNIT 4A	WEATHERED ROCK (Class IV)	Interbedded SANDSTONE, TUFF, SILTSTONE, CLAYSTONE, and COAL. Extremely to distinctly weathered. Fractured. Estimated to be typically of low to medium rock strength.
UNIT 4B	WEATHERED ROCK (Class III)	Interbedded SANDSTONE, TUFF and SILTSTONE. Distinctly to slightly weathered. Estimated to be typically of high to very high rock strength.

Groundwater or water inflows were not observed in any of the boreholes during the limited time that they remained open during the field work investigations.

It should be noted that fluctuations in groundwater levels may occur as a results of seasonal variations, temperatures, rainfall or other similar factors, the influence of which may not have been apparent at the time of the assessment.

Table 2 provides a summary of the distribution of the above geotechnical units at each of the boreholes locations.

TABLE 2 –DISTRIBUTION OF GEOTECHNICAL UNITS AT BOREHOLE AND TEST PIT LOCATIONS

BOREHOLE LOCATION	SURFACE RL(m) AHD	UNIT 1 FILL	UNIT 2 TOPSOIL / SLOPEWASH	UNIT 3 RESIDUAL	UNIT 4A WEATHERED ROCK	UNIT 4B WEATHERED ROCK
TP09-1	86.17	0.00-0.30	0.30-0.50	0.50-0.70	0.70->0.80	-
TP09-2	82.48	-	0.00-0.10	0.10-0.50	0.50->0.6	-
TP09-3	82.48	0.00-0.40	-	0.40-1.10	1.10->1.20	-
TP09-4	79.03	0.00-1.40	-	1.40->1.80	-	-
TP09-5	71.47	0.00-0.20	-	0.20-1.80	1.80->1.90	-
TP09-6	75.65	0.00-1.30	-	1.30-1.80	1.80->1.90	-
TP09-7	66.02	-	0.00-0.80	0.80->2.00	-	-
TP09-8	61.43	-	0.00-0.20	0.20-1.00	1.00->1.20	-
TP09-9	70.13	-	0.00-0.10	0.10-1.40	1.40->1.5	-
TP09-10	62.96	-	0.00-0.20	0.20->2.00	-	-
TP09-11	73.34	-	0.00-0.20	0.20-1.20	1.20->1.3	-
TP09-12	66.35	-	0.00-0.10	0.10-0.90	0.90->1.00	-
TP09-13	61.25	-	0.00-0.10	0.10-0.50	0.50->0.60	-
TP09-14*	96.91	0.00-0.70	-	0.70-0.80	0.80->0.90	-
TP09-15*	94.57	0.00-0.25	-	0.25-0.70	0.70->0.75	-
TP09-16*	95.13	0.00->1.50	-	-	-	-
TP09-17*	95.75	0.00-0.40	-	0.40-1.30	1.30->1.40	-
(1)TP4[#]	64.1	-	0.00-0.80	0.80-1.40	1.40->1.80	-
(1)TP5[#]	67.2	-	0.00-0.30	0.30-1.50	1.50->1.90	-

BOREHOLE LOCATION	SURFACE RL(m) AHD	UNIT 1 FILL	UNIT 2 TOPSOIL / SLOPEWASH	UNIT 3 RESIDUAL	UNIT 4A WEATHERED ROCK	UNIT 4B WEATHERED ROCK
(1)TP6 [#]	74.8	-	0.00-0.30	0.30-1.40	1.40->2.40	-
(8)29 [#]	78.3	-	0.00-0.10	0.10-0.55	0.55->1.00	-
(8)30 [#]	77.5	-	0.00-0.20	0.20-1.30	1.30->1.70	-
(8)34 [#]	77.0	-	0.00-0.10	0.10-0.95	0.95->1.20	-
(1)BH4 [#]	72.4	-	0.00-0.20	0.20-0.90	0.90-5.00	5.00->6.60
BH6 [#]	72.8	-	0.00-0.10	0.10-1.20	1.20-5.1	5.1->9.05
(1)BH7 [#]	75.5	-	0.00-0.30	0.30-1.30	1.30-6.80	6.80->8.70
BH20	85.32	-	0.00-0.50	-	0.50->6.40	-
BH21	77.13	0.00-0.90	-	0.90-1.40	1.40-8.10	8.10->13.7
BH22	74.89	0.00-0.50	-	0.50-1.90	1.90-13.5	13.5->18.30
BH23	73.23	-	0.00-0.20	0.20-1.30	1.30-2.50	2.50->12.40
BH24	70.22	-	0.00-0.20	0.20-0.70	0.70-2.40	2.40->5.73
BH25	70.10	-	0.00-0.40	0.40-1.30	1.30-6.00	6.00->7.20
* Test Pits conducted in the area of the proposed surgical skills training building						
[#] Boreholes and Test Pits from previous investigations						

4 LABORATORY TESTING

Samples obtained during the field investigations were returned to Coffey's NATA registered Newcastle Laboratory for testing.

Results of testing are outlined in Table 3 below.

TABLE 3 – SUMMARY OF RESULTS OF CBR TESTING

LOCATION	SAMPLE DEPTH (m)	FIELD MOISTURE CONTENT (%)	OPTIMUM MOISTURE CONTENT (OMC) (%)	CBR (%)
TP09-4*	0.2 – 0.4	10.9	11.6	45.0
TP09-5	0.25 – 0.5	22.3	22.8	4.5
TP09-6*	0.3 – 0.5	11.6	11.4	40.0
TP09-9	0.2 – 0.4	29.3	29.7	5.0
TP09-10	0.8 – 1.0	27.7	30.4	5.0
TP09-13	0.2 – 0.4	17.4	17.3	6.0
* Fill Material Associated With Existing Car Park (Suitable for Reuse)				

Point load index tests were undertaken on core recovered from the investigation at approximately 1.0m intervals or as appropriate for the materials encountered. Photographs of rock core are included in Appendix A with the engineering logs.

5 EARTHWORKS AND SITE PREPARATION

5.1 General Site Preparation

The following comments and recommendations are presented for earthworks and general site preparation.

- Prior to construction of any structure, the proposed built-up and pavement areas should be stripped to remove all vegetation, topsoil, fill, root affected or any other potentially deleterious material. Care will need to be exercised when working in the vicinity of the existing service mains that pass through the sites;
- Following stripping, the exposed subgrade materials should be proof rolled to identify any wet or excessively deflecting material. Any such areas should be over-excavated and backfilled with approved clean granular material. The moisture content of the subgrade materials and therefore the need for moisture conditioning or over-excavation and replacement, will be largely dependent on pre-existing and prevailing weather conditions at the time of construction;
- It is recommended that all exposed subgrade (beneath floor slabs) be immediately covered with a layer of blinding concrete or working platform of gravel or similar approved material to minimise risk of degradation due to exposure to rain;
- Where fill is to be placed on slopes exceeding 1V:8H (about 7°) a surface should be prepared by benching or stepping into the natural slope;

- Where fill is placed across an existing watercourse, a culvert of adequate size should be installed to accommodate flood flows. Subsoil drains should also be installed along the low point of the gullies at culvert crossings;
- Approved fill beneath roads should be placed in layers not exceeding 300mm loose thickness and be compacted to a minimum density ratio of 95% Standard in accordance with AS1289 5.1.1 or equivalent. Clay fill should be placed and maintained at 60% to 90% of standard Optimum Moisture Content (OMC);
- The natural subgrade and fill within 300mm of design subgrade level for road pavements should be compacted to a minimum density ratio of 100% Standard in accordance with AS1289 5.1.1 or equivalent within the above stated moisture content range;
- Fill beneath structures and floor slabs should be placed in layers not exceeding 300mm loose thickness and be compacted to a minimum density ratio of 95% Standard in accordance with AS1289 5.1.1 or equivalent, within $\pm 2\%$ of OMC;
- Controlled fill should be placed and compacted to the specifications outlined above unless it can be shown that the specified compaction can be obtained for a thicker layer. Alternative layer thicknesses and compaction standards may be permitted at the time of construction under geotechnical supervision. The maximum particle size should be less than $2/3$ of the compacted layer thickness;
- Fill batters should be finished at a maximum slope of 1V:2H and be vegetated to protect against erosion. Slopes intended to be grassed and maintained may require relaxed batters of around 1V:4H;
- Embankment materials shall be uniformly watered, tined and rolled to produce homogenous layers over the full width of the embankment. Embankments shall be overfilled and trimmed back to proper grade to expose materials compacted to the minimum standards specified above;
- Foundations materials will consist of stiff to very stiff clay or weathered rock. The foundation conditions are therefore not considered to be prone to excessive settlement, lateral displacement or slip circle development. Any proposed embankments are not likely to undergo significant consolidation settlements, provided they are constructed to the specifications outlined in this report;
- Earthworks should be carried out in accordance with the recommendations for "Level 1" controlled fill as outlined in Australian Standard AS3798-2007 *"Guidelines on Earthworks for Commercial and Residential Developments"*.

5.2 Excavation Conditions

Details of bulk excavation levels are unknown at this stage, however it is assumed that excavations will take place into the hillslope, into the residual clays and weathered rock. In terms of rippability, the site materials can generally be divided into:

- Clayey and Granular Soils (Unit 1, 2 and 3). It is anticipated that these materials could be excavated by a conventional bulldozer blade or excavator;
- Weathered Rock (Unit 4A and 4B). Rippability is dependent on rock strength, degree of weathering and number of defects within the rock mass. It is assessed that generally the Unit 4A weathered rock encountered in the boreholes will be rippable. However, some excavatability problems may occur in the high strength Unit 4B weathered rock or if iron indurated zones are encountered. Such areas may require the use of hydraulic rock breakers. Higher strength rock or randomly occurring hard bands within the rock mass if encountered, are likely to occur towards the base of deeper cuts.

Table 4 provides an indication of the assessed excavatability of each geotechnical unit.

TABLE 4 - SUMMARY OF INDICATIVE EXCAVATABILITY OF GEOTECHNICAL UNITS

GEOTECHNICAL UNIT	LIKELY TO BE EXCAVATABLE USING
UNIT 1, 2 and 3	Conventional bulldozer blade or excavator bucket.
UNIT 4A	Conventional bulldozer (Caterpillar D8 or equivalent) equipped with ripping tynes. Large excavator equipped with sharp toothed bucket and/or rock-breaker.
UNIT 4B	Conventional bulldozer (Caterpillar D9 or equivalent) equipped with ripping tynes. Large excavator equipped with rock-breaker. Stronger, less jointed zones do occur and may require the use of a D10 (or equivalent) or blasting to loosen prior to ripping.

5.3 Suitability of Materials for Reuse

Table 5 presents comments with regard to the suitability of site materials for reuse as fill from a geotechnical viewpoint.

TABLE 5 - SUMMARY OF SUITABILITY OF MATERIALS FOR RE-USE

GEOTECHNICAL UNIT	SUITABILITY FOR RE-USE
UNIT 1	Clean Sand, Clay and Gravel fill materials may be suitable for reuse as general site fill, however this will be dependent on inspection by the geotechnical authority at the time of construction. Fill containing deleterious materials should be disposed of to spoil.
UNIT 2	The surface layers of topsoil and root affected vegetation or other potentially deleterious materials should be disposed of to spoil or stockpiled for later landscaping purposes only. Slopewash sand, gravel and clay may be suitable for use as engineering fill, however this is likely to be dependent on material moisture content and should be confirmed by the geotechnical authority at the time of construction.
UNIT 3	The residual clay soils may be used for engineering fill beneath structures or for embankment fill. High plasticity clay should be blended with granular material or soil of lower plasticity prior to placement as fill.
UNIT 4A and 4B	Weathered rock may be used for engineering fill beneath structures or for embankment fill. Select fill should be chosen from the better quality, less weathered rock.

5.4 Trafficability

Generally, the site is assessed to be moderately to well drained and trafficability by 4WD vehicle at the time of the field investigations was fair to good under the prevailing weather conditions, which included periods of prolonged rain.

It should be noted that some trafficability problems may occur in the lower lying areas of the site and in the vicinity of the existing gully if UNIT 1, 2 or 3 soils are exposed during periods of wet weather.

In such instances, the provision of a granular bridging layer over the affected materials should be adequate to provide suitable vehicular access.

The extent of such areas, if they occur, will be largely dependent on pre-existing and prevailing weather conditions at the time of construction.

5.5 Stability Parameters for Earthworks and Retention of Permanent Excavations

Table 6 and Table 7 provide recommended parameters for design of retaining measures and earthworks within each of the inferred geotechnical units.

TABLE 6 - RECOMMENDED BATTER SLOPES BY GEOTECHNICAL UNIT

GEOTECHNICAL UNIT	MAXIMUM SLOPE OF EXCAVATED UNSUPPORTED BATTER	
	Temporary Excavations *	Permanent Excavations
UNIT 1	1V:1H	1V:2H
UNIT 2	1V:1H	1V:2H
UNIT 3	1V:1H	1V:2H
UNIT 4A	1V:0.5H	1V:1.5H**
UNIT 4B	1V:0.25H	1V:1.5H**

The following additional comments are made with regard to excavations and cuttings:

- It may be possible to steepen the batter slope of UNIT 4A and 4B (Weathered Rock) materials. Steepened batter slopes will require inspection by an experienced geotechnical engineer / geologist at the time of excavation to assess the need for rock bolts should any adversely orientated defects or potential wedges be identified;
- The provision of shotcreting in the upper 1.0m to 1.5m of the rock profile (UNIT 4A and 4B) may be required if highly fractured rock is exposed;
- Excavation should be carried out in a manner to limit erosion or instability. Working faces should be limited to safe heights and slopes, and surfaces should be shaped and drained to limit ponding of water;

- V-Drains should be installed at the top and cess drains at the toe of cuttings to cater for surface or any other flows;
- The recommended design batter slopes given in Table 6 are based mainly on the degree of weathering and material strength. The influence on batter slope stability due to defects such as joints and bedding should also be considered. The dip of the rock is generally in the order of 0° to 10°, however locally steeper dipping rock may be encountered which has the potential to affect the stability of cuttings. Ongoing geotechnical input should occur during excavation of the cuts to modify batter slopes as required.

TABLE 7 RECOMMENDED PARAMETERS FOR DESIGN OF RETAINING SYSTEMS

Geotechnical Unit	γ (kN/m ³)	c' (kPa)	ϕ' (o)	S_u (kPa)	E (MPa)	ν
UNIT 1 & 2	18	0	28	60	12	0.3
UNIT 3	19	7	28	150	30	0.3
UNIT 4A	22	10	30	200	40	0.3
UNIT 4B	23	20	33	300	60	0.3
Note: γ = Unit Weight c' = Effective Cohesion ϕ' = Effective Friction S_u = Undrained Shear Strength E = Young's Modulus ν = Poisson's Ratio						

The design of retention systems should allow for hydrostatic pressures resulting from a build up of water behind retaining walls. Groundwater inflows were not encountered in the geotechnical units for which permanent retention measures are expected to be required. It is therefore considered that, provided adequate surface and subsurface drainage and free draining backfill are used in conjunction with all retaining measures, no design allowance is necessary to resist hydrostatic pressures in addition to earth pressures.

The design should also allow for surcharge loading due to additional backfill or sloping backfill, or due to existing structures or roadways in the vicinity of the wall.

Rigid structures subject to lateral loads should be designed adopting an at rest coefficient of earth pressure, $K_0 = 0.5$.

6 FOUNDATIONS

6.1 Foundation Design Parameters

Proposed foundation types and founding levels are unknown at this stage. It is understood that the new buildings may have expected maximum foundation loads of approximately 2500kN to 3000kN.

Based on the subsurface conditions encountered during the field investigations and details available of the proposed development, footings are likely to comprise of pads, strip footings or end bearing piers/piles.

It is recommended that all footings be founded in the residual soils (UNIT 3), weathered rock (Unit 4A and 4B) or on engineered fill platforms. Foundations may be proportioned for the maximum allowable (serviceability) end bearing pressures outlined in Table 8.

TABLE 8 - ALLOWABLE (SERVICEABILITY) BEARING PRESSURES

GEOTECHNICAL UNIT	MAXIMUM ALLOWABLE BEARING PRESSURE (kPa)
Engineered Fill Platforms	150
UNIT 3	200
UNIT 4A	1000
UNIT 4B	3000

Allowable (serviceability) bearing pressures assume that settlements will be less than 1% of least footing width or pile diameter.

Alternatively, if loads are such that rock socket piles are required, the design parameters presented in Table 9 can be adopted for bored piles with a socket greater than 0.3m into the weathered rock.

TABLE 9 - RECOMMENDED FOUNDATION DESIGN PARAMETERS – ROCK SOCKETS

FOUNDING MATERIAL	UNIT 3 (RESIDUAL)	UNIT 4A (CLASS IV)	UNIT 4B (CLASS III)
Ultimate End Bearing Pressure ¹	-	4 MPa	20 MPa
Serviceable End Bearing Pressure	200 kPa	1MPa	3MPa
Ultimate Shaft Adhesion ²	50 kPa	250 kPa	1000 kPa
Elastic Modulus (E_{field})	-	100 kPa	500 kPa
Geotechnical Strength Reduction Factor ³ (ϕ_g)	0.5	0.5	0.5

NOTES:

1. Ultimate values occur at large settlements (>5% of minimum footing dimensions);
2. For piles to carry design loads it is assumed that the pile bases are cleared of debris and for rock socket piles the shaft is roughened category R2 or better (R2 – defined as grooves of depth 1mm to 4mm and width greater than 2mm, at spacings of 50mm to 100mm);
3. Geotechnical strengths can be obtained by applying the above geotechnical strength reduction factor to the ultimate values given.

Rock socket parameters given in Table 9 should be used with a design method that accounts for the stress distribution between the socket and base, which is related to the stiffness of the rock and geometry of the pile. Methods such as the elastic, side slip or non linear analyses described in Pell PJN 1999, "State of Practise for the Design of Socketed Piles in Rock", Proceedings, 8th Australian New Zealand Conference on Geomechanics, Hobart (Reference 3), could be used. Coffey can assist with rock socket design if required.

The socket should be cleaned and roughened by a suitable scraper such as tooth, orientated perpendicular to the auger shaft. The base of the pile should be cleaned using a suitable bucket to remove spoil, as open flight augers often cannot remove sufficient spoil to expose the majority of the pile base. If seepage occurs, piles should be dewatered prior to pouring concrete. A suitably experienced geotechnical engineer should inspect the piles.

Foundations for the structure in the vicinity of the existing service trenches should be founded below a 1V:2H projection up from the base of the service.

6.2 Site Classification

For any lightly loaded, residential type structures that may form part of future site developments, a preliminary site classification with respect to reactive soils has been carried out in accordance with the requirements of Australian Standard AS2870-1996, *"Residential Slabs and Footings"*.

On the basis of the results of the soil profiles encountered during the field investigation, laboratory testing from previous studies at the John Hunter Hospital site, and preliminary calculations, the site is classified in accordance with AS2870-1996 as follows:

- Class 'M' – Natural Soil Profile (<1.0m to weathered rock);
- Class 'H' – Natural Soil Profile (>1.0m to weathered rock);
- Class 'P' – Uncontrolled Fill Areas (such as backfilled trenches for sewer main).

The effects of changes to the soil profile by additional cutting and filling and the effects of past and future trees should be considered in selection of the design value for differential movement. Site classifications should therefore be reviewed after any site regrade works. Footings for the proposed development should be designed and constructed in accordance with the requirements of AS2870.

The classification presented above assumes that:

- All footings are founded in controlled fill (if applicable) or in the natural soils below all non-controlled fill, topsoil material and root zones and fill under slab panels meets the requirements of AS2870, in particular, the root zone must be removed prior to the placement of fill materials beneath slab floors;
- The performance expectations set out in AS2870 are acceptable;
- Site maintenance complies with the provisions of CSIRO Sheet BTF 18, *'Foundation Maintenance and Footing Performance: A Home Owner's Guide'*, a copy of which is attached;
- Service trenches backfilled with uncontrolled fill do not extend below a line extending out and down at 45° from the ground surface at the edge of the building;
- The constructional and architectural requirements for reactive clay sites set out in AS2870 are followed.

Where fill is to be placed to raise site levels the affected building areas will require reclassification once the depth and type of placed fill are known and the level of earthworks control has been established.

6.3 Site Seismicity

Australian Standard AS1170.4–2007, Minimum Design Loads on Structures, Part 4: Earthquake Loads, indicates an acceleration coefficient (a) of 0.11 is applicable to the site geographic location.

It is anticipated that foundations for the proposed structures will be taken to the level of rock of low strength class or better. Based on the inferred geology and anticipated excavation levels, it is assessed that these conditions will result in a site sub soil class 'B_e', in accordance with Section 4.2.2 and Table 2.1 of AS1170.4–2007.

7 PAVEMENTS

7.1 Design Traffic Loading

Design traffic loadings have been adopted in accordance with Newcastle City Council guidelines. Table 10 presents a summary of design traffic loadings adopted for new roads and car park areas.

TABLE 10 – DESIGN TRAFFIC LOADINGS

ROAD TYPE	ROAD CLASS	DESIGN TRAFFIC LOADING (ESAs)
Access Road	Major Cul-de-sac	5 X 10 ⁵
Car Park Areas	Local Access	1 X 10 ⁵

7.2 Design CBR

Based on the profiles encountered at borehole locations and one laboratory CBR test, a design subgrade CBR value of 4.5% has been adopted for proposed new road and car park pavements. This is the lowest value gained from testing and therefore represents the lowest likely value for subgrade material across the site.

7.3 Flexible Pavement Design

Flexible pavement thickness designs for proposed new roads and car park areas have been prepared with reference to Austroads - Pavement Design 2004.

The recommended material, construction specification and pavement make-ups are presented on the attached Pavement Thickness Design Summary (PTDS) sheet, Appendix C.

Subgrade materials are expected to comprise sandy and/or gravelly clay. If soft subgrades exist at the time of construction or deleterious materials are encountered at subgrade level, these materials should be over-excavated and replaced with a minimum depth of 300mm of well graded granular select material with CBR of 15% or greater.

The requirement for, and extent of subgrade replacement should be confirmed by the geotechnical authority at the time of construction.

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

7.4 Drainage

The enclosed pavement designs assume the provision of adequate surface and subsurface drainage of the pavements and adjacent areas. Subsoil drains are would be necessary upslope of all pavements across slopes and on both sides of pavements down slope.

8 SLOPE STABILITY ASSESSMENT

8.1 Background Information

The risk of slope instability has been assessed from the observed site conditions in accordance with the classification system formulated by the Australian Geomechanics Society and published in '*Australian Geomechanics News, Number 10, 1985*' (see Attachment 1: Classification of Risk of Slope Instability, for explanation of risk categories and implications for development).

This report provides an assessment of the risk of slope instability on the property and immediate surrounding area. The report also recommends some geotechnical constraints for the site development in light of the assessed risk of slope instability. The onus is on the owner, potential owner or interested party to decide whether the assessed level of risk is acceptable taking into account likely economic consequences of the risk and the recommended geotechnical constraints.

8.2 Risk of Slope Instability

The risk of slope instability for the site has been based on the site observations recorded in Section 3.0. The principal site features used in the assessment are:

- Situated in an area of gently to moderately undulating topography;
- Relatively uniform slopes;
- Natural surface slopes are typically in the order of about 10° to 15°;
- Soil depths ranging from about 0.5m to 4.0m;
- Soils comprise shallow topsoil / slopewash, overlying very stiff to hard residual clays;
- No evidence of seepage was observed;
- Minor concentration of surface waters or boggy ground was observed;
- No evidence of overall slope instability was observed at the time of the field investigations.

On the basis of these site features the site is assessed to have an overall Medium risk of slope instability in accordance with the classification system in Attachment 1.

It would be normal practice in the Newcastle area for development to proceed on a site with this risk level classification. Development should be carried out in accordance with good hillside practice (as set out in Attachment 2 and 3) and the specific geotechnical constraints defined in Section 4.2.

8.3 Recommended Geotechnical Constraints for Development

Type of Structure:

There are no particular geotechnical constraints on the type of structures provided they are founded on footings designed and constructed in accordance with the recommendation provided above.

Development should be designed to accommodate the natural slope profile.

Area for Development:

The entire site is considered feasible for development from a slope stability viewpoint. Development of the lot should be undertaken in accordance with good hillside construction practice and sound engineering principles as presented in Attachment 2.

Foundation Type:

Strip / pad footings, raft slabs, or pier and beam systems would be feasible from a slope stability viewpoint. (Raft slabs not generally suited to slopes $> 1:3$ due to slope modifications required.)

Foundations should not be founded within the fill. This will require piered foundations, removal of the fill or removal and replacement of the fill to engineering specification.

Foundations should be designed and constructed in accordance with the recommendations and advice of AS2870, *'Residential Slabs and Footings'* or the recommendations provided above.

Foundations near the crest of excavations should be taken to rock or founded behind or below a 1V:2H projection from the toe of the excavation.

Foundations near the existing sewer main should be founded behind or below a 1V:2H projection up from the base of the sewer.

Many soils and weak rocks in the area soften appreciably on exposure to air and water and care should be taken to cast foundations onto undisturbed material. Concrete should be poured within twenty four hours of excavation or else a blinding layer of concrete should be used.

Excavation:

Excavations should be supported by properly designed and constructed retaining walls or else battered at 1V:2H or flatter and protected against erosion. Excavations in competent sandstone (below the level of backhoe refusal) may be battered at 1V:1H.

Excavations exceeding 1.5m in depth should be carried out in stages of no more than 4m width, with the exposed section retained / stabilised / assessed prior to excavation of the subsequent stage. Excavations should be designed for surcharge loading from slopes, retaining walls, structures and other improvements in the vicinity of the excavation

Exposed faces are likely to undergo rapid degradation, ravelling and softening on exposure and batter protection or excavation retention measures should be implemented as soon as possible after exposure.

Should wet weather occur during construction or prior to construction of retention / batter protection measures, the exposed face should be protected by plastic sheeting extending above and below the cut. Surface water flows from upslope areas should be diverted away from the cut face.

Drainage measures should be implemented above and behind all temporary and permanent excavations to avoid concentrated water flows on the face of the cut or infiltration into the soil / rock profile behind the cut.

Filling:

The depth of unsupported fill on the site should not exceed 1.5m and should be battered at 1V:2H or flatter and protected against erosion. All fill greater than 1.5m deep should be supported by an engineer designed retaining wall.

Where fill is placed on slopes in excess of 1V:8H (7°), a prepared surface should be benched/stepped into the natural slope.

The above recommendations do not apply to controlled or engineered site regrade for which specific geotechnical engineering design and supervision is undertaken.

Retaining Walls:

Retaining walls should be designed for surcharge loading from slopes, retaining walls, structures and other existing/future improvements in the vicinity of the wall. Adequate subsurface and surface drainage should be provided behind all retaining walls.

Retaining walls should be designed by an experienced engineer familiar with the site conditions.

Excavations for the construction of retaining walls results in a temporary reduction in the stability of the adjacent area particularly during wet weather until the wall is complete. This increased risk can be managed or reduced by appropriate construction planning, using temporary support, staged excavation and control of drainage.

Access/Site Clearance:

Access and site modifications should comply with the recommendations above. Driveway access is usually restricted by Council's to a grade of 1V:4H (14°) or less. Design of access must be such as to limit cut and fill to the constraints set out above.

Drainage and Sewage Disposal:

All collected stormwater run-off should be discharged / piped into an interallotment drainage system or existing watercourse in a controlled manner that limits erosion.

Septic wastes should be connected to the reticulated disposal system.

Where fill is placed across an existing watercourse, a culvert of adequate size to accommodate flood flows should be installed (subsoil drain along low point may also be required).

8.4 Summary

The proposed development is assessed to have an overall Medium risk of slope instability and it is considered that the site is appropriate for development subject to the geotechnical constraints on development detailed in Section 7.3.

9 CONSTRUCTION RISK

The extent of testing associated with this assessment is limited to discrete borehole locations and variations in ground conditions can occur between and away from such locations. If subsurface conditions encountered during construction differ from those given in this report further advice should be sought without delay.

Further advice on the uses and limitations of this report is presented in the attached document, *'Important Information about your Coffey Report'*.

For and on behalf of Coffey Geotechnics Pty Ltd



Arthur Love

Principal Geotechnical Engineer

REFERENCES

1. Pells, PJN et al (1978), 'Design Loading for Foundations on Shale and Sandstone in the Sydney Region', Aust. Geomech. Jnl. Vol 8, pp 31-39;
2. Pells et al (1998), 'Foundations on Sandstone and Shale in the Sydney Region', Aust. Geomech. Jnl. Dec. 1998;
3. Pells PJN (1999), 'State of Practise for the Design of Socketed Piles in Rock', Proceedings, 8th Australia New Zealand Conference on Geomechanics, Hobart;
4. Landslide Risk Management Concepts and Guidelines, Australian Geomechanics Society Sub-Committee on Landslide Risk Management, Australian Geomechanics, March 2000;
5. Conference on Buildings and Structures Subject to Mine Subsidence, Newcastle Division, Mine Subsidence Board, NSW, August 1988;
6. Australian Standard, AS1170.4-1993, Minimum Design Loads on Structures, Part 4: Earthquakes;
7. Australian Standard, AS2870-1996, Residential Slabs and Footings;
8. Australian Standard, AS3798-2007, Guidelines on Earthworks for Commercial and Residential Development.

Foundation Maintenance and Footing Performance: A Homeowner's Guide



BTF 18
replaces
Information
Sheet 10/91

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

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

Soil Types

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

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

Causes of Movement

Settlement due to construction

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

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

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

Erosion

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

Saturation

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

Seasonal swelling and shrinkage of soil

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

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

Shear failure

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

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

GENERAL DEFINITIONS OF SITE CLASSES

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

Tree root growth

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

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

Unevenness of Movement

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

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

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

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

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

Effects of Uneven Soil Movement on Structures

Erosion and saturation

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

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

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 external footings. The doming is accentuated and cracking reduces or disappears where it occurred because of dishing, but other cracks open up. The roof lines may become convex.

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

Movement caused by tree roots

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

Complications caused by the structure itself

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

Effects on full masonry structures

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

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

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

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

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

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

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

Effects on framed structures

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

Effects on brick veneer structures

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

Water Service and Drainage

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

Pipe leakage and trench water flows also encourage tree and shrub roots to the source of water, complicating and exacerbating the problem.

Poor roof plumbing can result in large volumes of rainwater being concentrated in a small area of soil:

- Incorrect falls in roof guttering may result in overflows, as may gutters blocked with leaves etc.

- Corroded guttering or downpipes can spill water to ground.
- Downpipes not positively connected to a proper stormwater collection system will direct a concentration of water to soil that is directly adjacent to footings, sometimes causing large-scale problems such as erosion, saturation and migration of water under the building.

Seriousness of Cracking

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

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

Prevention/Cure

Plumbing

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

Ground drainage

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

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

Protection of the building perimeter

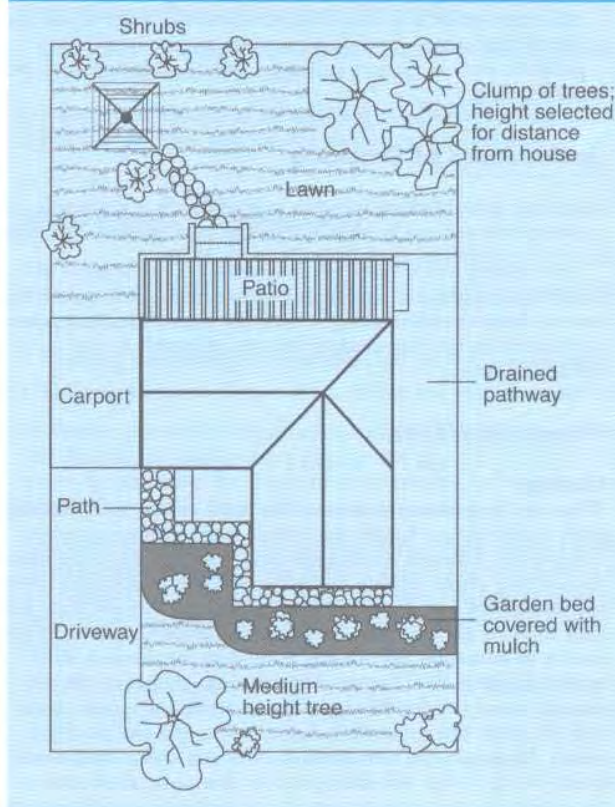
It is essential to remember that the soil that affects footings extends well beyond the actual building line. Watering of garden plants, shrubs and trees causes some of the most serious water problems.

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

CLASSIFICATION OF DAMAGE WITH REFERENCE TO WALLS

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

Gardens for a reactive site



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

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

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

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

Condensation

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

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

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

The garden

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

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

Existing trees

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

Information on trees, plants and shrubs

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

Excavation

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

Remediation

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

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

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

The information in this and other issues in the series was derived from various sources and was believed to be correct when published.

The information is advisory. It is provided in good faith and not claimed to be an exhaustive treatment of the relevant subject.

Further professional advice needs to be obtained before taking any action based on the information provided.

Distributed by

CSIRO PUBLISHING PO Box 1139, Collingwood 3066, Australia

Freecall 1800 645 051 Tel (03) 9662 7666 Fax (03) 9662 7555 www.publish.csiro.au

Email: publishing.sales@csiro.au

© CSIRO 2003. Unauthorised copying of this Building Technology file is prohibited

Attachment 1: Classification of Risk of Slope Instability

ASSESSMENT OF RISK

Natural hill slopes are formed by processes which reflect the site geology, environment and climate. These processes include downslope movement of the near surface soil and rocks, in geological time all slopes are unstable. The area of influence of these downslope movements may range from local to regional and are rarely related to property boundaries. The natural processes may be affected by human intervention in the form of construction and related activities.

A landslide (or landslide) is a downslope movement of a soil or rock mass as a result of shear failure at the boundaries of the moving mass. Soil creep, which is extremely slow and occurs without a well defined surface, is not included as a landslide.

It is not technically feasible to assess the stability of a particular site in absolute terms such as stable or unstable. However, the degree of risk of slope movement can be assessed by the recognition of surface features supplemented by limited information on the regional and local subsurface profile and with the benefit of experience gained in similar geological environments. The degree of risk is categorised below:

RISK OF INSTABILITY	EXPLANATION	IMPLICATIONS FOR DEVELOPMENT
VERY HIGH	Evidence of active or past landslips or rockface failure, extensive or rockface failure, extensive instability may occur.	Unsuitable for development unless major geotechnical work can satisfactorily improve the stability. Extensive geotechnical investigation necessary. Risk after development may be higher than usually accepted.
HIGH	Evidence of active soil creep or minor slips or rockface instability, significant instability may occur during and after extreme climatic conditions.	Development restrictions and/or geotechnical works required. Geotechnical investigation necessary. Risk after development may be higher than usually accepted.
MEDIUM	Evidence of possible soil creep or a steep soil covered slope, significant instability can be expected if the development does not have due regard for the site conditions.	Development restrictions may be required. Engineering practices suitable to hillside construction necessary. Geotechnical investigation may be needed. Risk after development generally no higher than usually accepted.
LOW	No evidence of instability observed, instability not expected.	Good engineering practices suitable for hillside construction required. Risk after development normally acceptable.
VERY LOW	Typically shallow soil cover with flat to gently sloping topography.	Good engineering practices should be followed.

Attachment 2: Some Guidelines for Hillside Construction

GOOD ENGINEERING PRACTICE

POOR ENGINEERING PRACTICE

ADVICE

GEOTECHNICAL ASSESSMENT	Obtain advice from a qualified, experienced geotechnical practitioner at early stage of planning and before site works.	Prepare detailed plan and start site works before geotechnical advice.
-------------------------	---	--

PLANNING

SITE PLANNING	Having obtained geotechnical advice, plan the development with the risk arising from the identified hazards and consequences in mind.	Plan development without regard for the Risk.
---------------	---	---

DESIGN AND CONSTRUCTION

HOUSE DESIGN	Use flexible structures which incorporate properly designed brickwork, timber or steel frames, timber or panel cladding. Consider use of split levels. Use decks for recreational areas where appropriate.	Floor plans which require extensive cutting and filling. Movement intolerant structures.
SITE CLEARING	Retain natural vegetation wherever practicable.	Indiscriminately clear the site.
ACCESS & DRIVEWAYS	Satisfy requirements below for cuts, fills, retaining walls and drainage. Council specifications for grades may need to be modified. Driveways and parking areas may need to be fully supported on piers.	Excavate and fill for site access before geotechnical advice.
EARTHWORKS	Retain natural contours wherever possible.	Indiscriminatory bulk earthworks.
CUTS	Minimise depth. Support with engineered retaining walls or batter to appropriate slope. Provide drainage measures and erosion control.	Large scale cuts and benching. Unsupported cuts. Ignore drainage requirements
FILLS	Minimise height. Strip vegetation and topsoil and key into natural slopes prior to filling. Use clean fill materials and compact to engineering standards. Batter to appropriate slope or support with engineered retaining wall. Provide surface drainage and appropriate subsurface drainage.	Loose or poorly compacted fill, which if it fails, may flow a considerable distance including onto property below. Block natural drainage lines. Fill over existing vegetation and topsoil. Include stumps, trees, vegetation, topsoil, boulders, building rubble etc in fill.
ROCK OUTCROPS & BOULDERS	Remove or stabilise boulders which may have unacceptable risk. Support rock faces where necessary.	Disturb or undercut detached blocks or boulders.
RETAINING WALLS	Engineer design to resist applied soil and water forces. Found on rock where practicable. Provide subsurface drainage within wall backfill and surface drainage on slope above. Construct wall as soon as possible after cut/fill operation.	Construct a structurally inadequate wall such as sandstone flagging, brick or unreinforced blockwork. Lack of subsurface drains and weepholes.
FOOTINGS	Found within rock where practicable. Use rows of piers or strip footings oriented up and down slope. Design for lateral creep pressures if necessary. Backfill footing excavations to exclude ingress of surface water.	Found on topsoil, loose fill, detached boulders or undercut cliffs.
SWIMMING POOLS	Engineer designed. Support on piers to rock where practicable. Provide with under-drainage and gravity drain outlet where practicable. Design for high soil pressures which may develop on uphill side whilst there may be little or no lateral support on downhill side.	
DRAINAGE		
SURFACE	Provide at tops of cut and fill slopes. Discharge to street drainage or natural water courses. Provide general falls to prevent blockage by siltation and incorporate silt traps. Line to minimise infiltration and make flexible where possible. Special structures to dissipate energy at changes of slope and/or direction.	Discharge at top of fills and cuts. Allow water to pond on bench areas.
SUBSURFACE	Provide filter around subsurface drain. Provide drain behind retaining walls. Use flexible pipelines with access for maintenance. Prevent inflow of surface water.	Discharge roof runoff into absorption trenches.
SEPTIC & SULLAGE	Usually requires pump-out or mains sewer systems; absorption trenches may be possible in some areas if risk is acceptable. Storage tanks should be water-tight and adequately founded.	Discharge sullage directly onto and into slopes. Use absorption trenches without consideration of landslide risk.
EROSION CONTROL & LANDSCAPING	Control erosion as this may lead to instability. Revegetate cleared area.	Failure to observe earthworks and drainage recommendations when landscaping.

DRAWINGS AND SITE VISITS DURING CONSTRUCTION

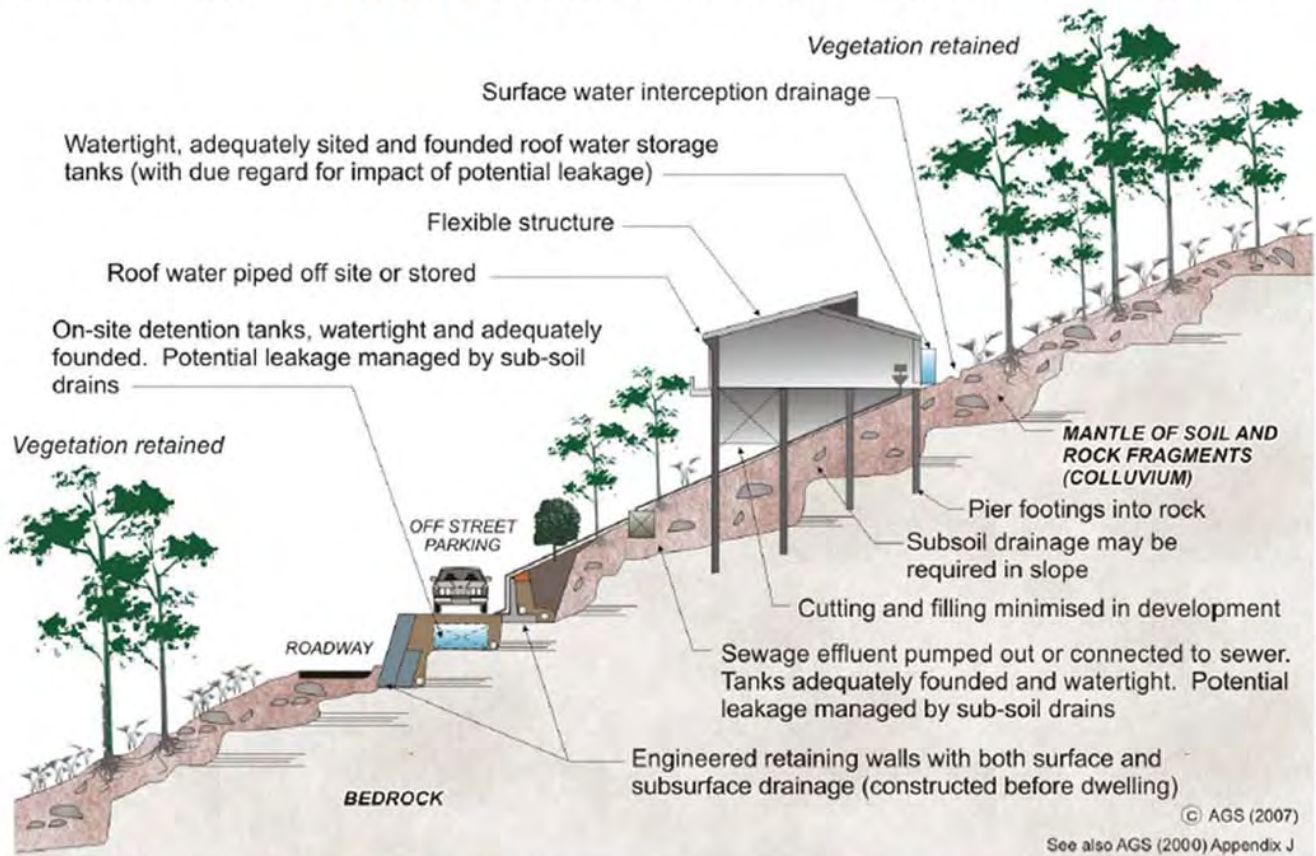
DRAWINGS	Building Application drawings should be viewed by geotechnical consultant	
SITE VISITS	Site Visits by consultant may be appropriate during construction/	

INSPECTION AND MAINTENANCE BY OWNER

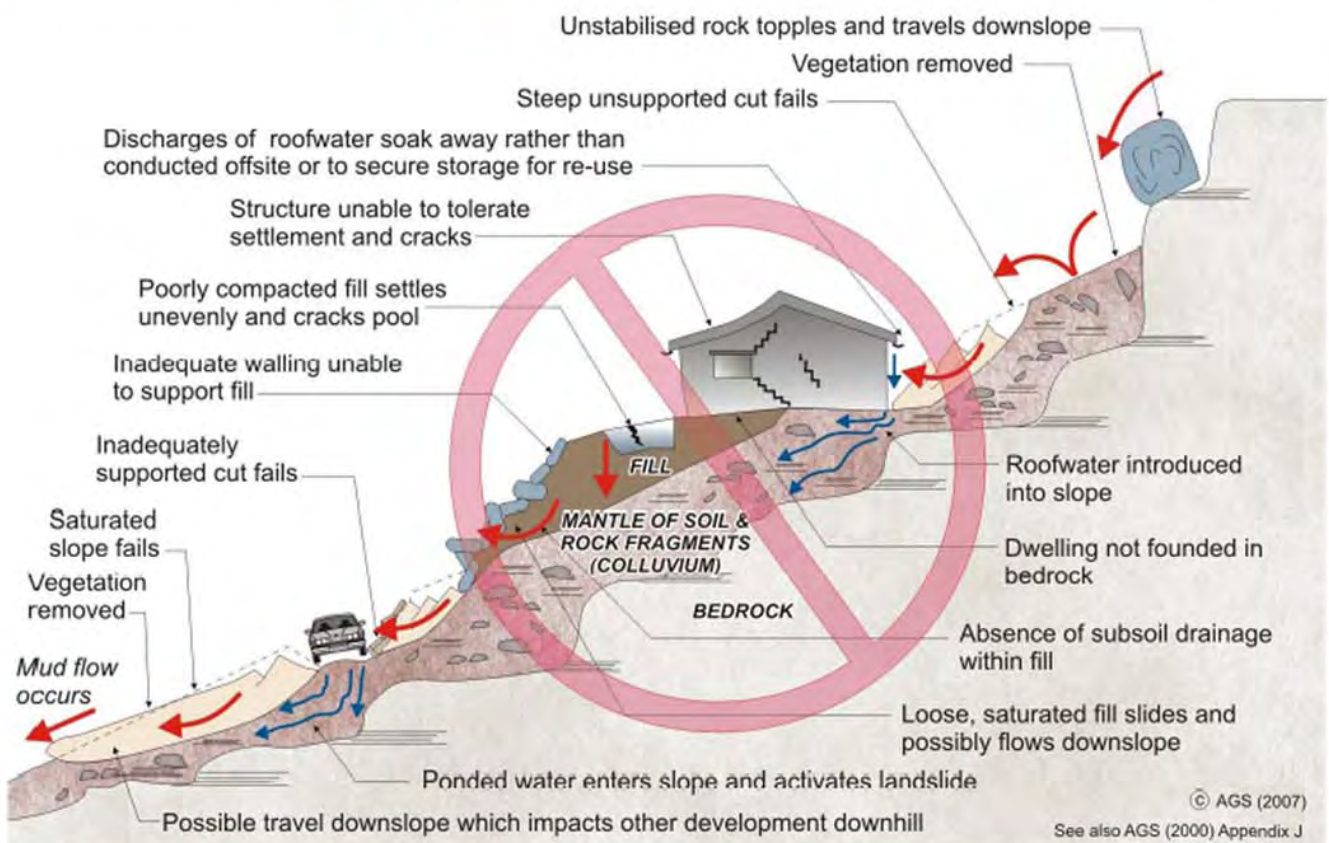
OWNER'S RESPONSIBILITY	Clean drainage systems; repair broken joints in drains and leaks in supply pipes. Where structural distress is evident see advice. If seepage observed, determine causes or seek advice on consequences.	
------------------------	--	--

Attachment 3: Illustrations of Good and Poor Hillside Practise

EXAMPLES OF **GOOD** HILLSIDE CONSTRUCTION PRACTICE



EXAMPLES OF **POOR** HILLSIDE CONSTRUCTION PRACTICE



Important information about your **Coffey** Report

As a client of Coffey you should know that site subsurface conditions cause more construction problems than any other factor. These notes have been prepared by Coffey to help you interpret and understand the limitations of your report.

Your report is based on project specific criteria

Your report has been developed on the basis of your unique project specific requirements as understood by Coffey and applies only to the site investigated. Project criteria typically include the general nature of the project; its size and configuration; the location of any structures on the site; other site improvements; the presence of underground utilities; and the additional risk imposed by scope-of-service limitations imposed by the client. Your report should not be used if there are any changes to the project without first asking Coffey to assess how factors that changed subsequent to the date of the report affect the report's recommendations. Coffey cannot accept responsibility for problems that may occur due to changed factors if they are not consulted.

Subsurface conditions can change

Subsurface conditions are created by natural processes and the activity of man. For example, water levels can vary with time, fill may be placed on a site and pollutants may migrate with time. Because a report is based on conditions which existed at the time of subsurface exploration, decisions should not be based on a report whose adequacy may have been affected by time. Consult Coffey to be advised how time may have impacted on the project.

Interpretation of factual data

Site assessment identifies actual subsurface conditions only at those points where samples are taken and when they are taken. Data derived from literature and external data source review, sampling and subsequent laboratory testing are interpreted by geologists, engineers or scientists to provide an opinion about overall site conditions, their likely impact on the proposed development and recommended actions. Actual conditions may differ from those inferred to exist, because no professional, no matter how qualified, can reveal what is hidden by

earth, rock and time. The actual interface between materials may be far more gradual or abrupt than assumed based on the facts obtained. Nothing can be done to change the actual site conditions which exist, but steps can be taken to reduce the impact of unexpected conditions. For this reason, owners should retain the services of Coffey through the development stage, to identify variances, conduct additional tests if required, and recommend solutions to problems encountered on site.

Your report will only give preliminary recommendations

Your report is based on the assumption that the site conditions as revealed through selective point sampling are indicative of actual conditions throughout an area. This assumption cannot be substantiated until project implementation has commenced and therefore your report recommendations can only be regarded as preliminary. Only Coffey, who prepared the report, is fully familiar with the background information needed to assess whether or not the report's recommendations are valid and whether or not changes should be considered as the project develops. If another party undertakes the implementation of the recommendations of this report there is a risk that the report will be misinterpreted and Coffey cannot be held responsible for such misinterpretation.

Your report is prepared for specific purposes and persons

To avoid misuse of the information contained in your report it is recommended that you confer with Coffey before passing your report on to another party who may not be familiar with the background and the purpose of the report. Your report should not be applied to any project other than that originally specified at the time the report was issued.

Important information about your **Coffey** Report

Interpretation by other design professionals

Costly problems can occur when other design professionals develop their plans based on misinterpretations of a report. To help avoid misinterpretations, retain Coffey to work with other project design professionals who are affected by the report. Have Coffey explain the report implications to design professionals affected by them and then review plans and specifications produced to see how they incorporate the report findings.

Data should not be separated from the report*

The report as a whole presents the findings of the site assessment and the report should not be copied in part or altered in any way.

Logs, figures, drawings, etc. are customarily included in our reports and are developed by scientists, engineers or geologists based on their interpretation of field logs (assembled by field personnel) and laboratory evaluation of field samples. These logs etc. should not under any circumstances be redrawn for inclusion in other documents or separated from the report in any way.

Geoenvironmental concerns are not at issue

Your report is not likely to relate any findings, conclusions, or recommendations about the potential for hazardous materials existing at the site unless specifically required to do so by the client. Specialist equipment, techniques, and personnel are used to perform a geoenvironmental assessment. Contamination can create major health, safety and environmental risks. If you have no information about the potential for your site to be contaminated or create an environmental hazard, you are advised to contact Coffey for information relating to geoenvironmental issues.

Rely on Coffey for additional assistance

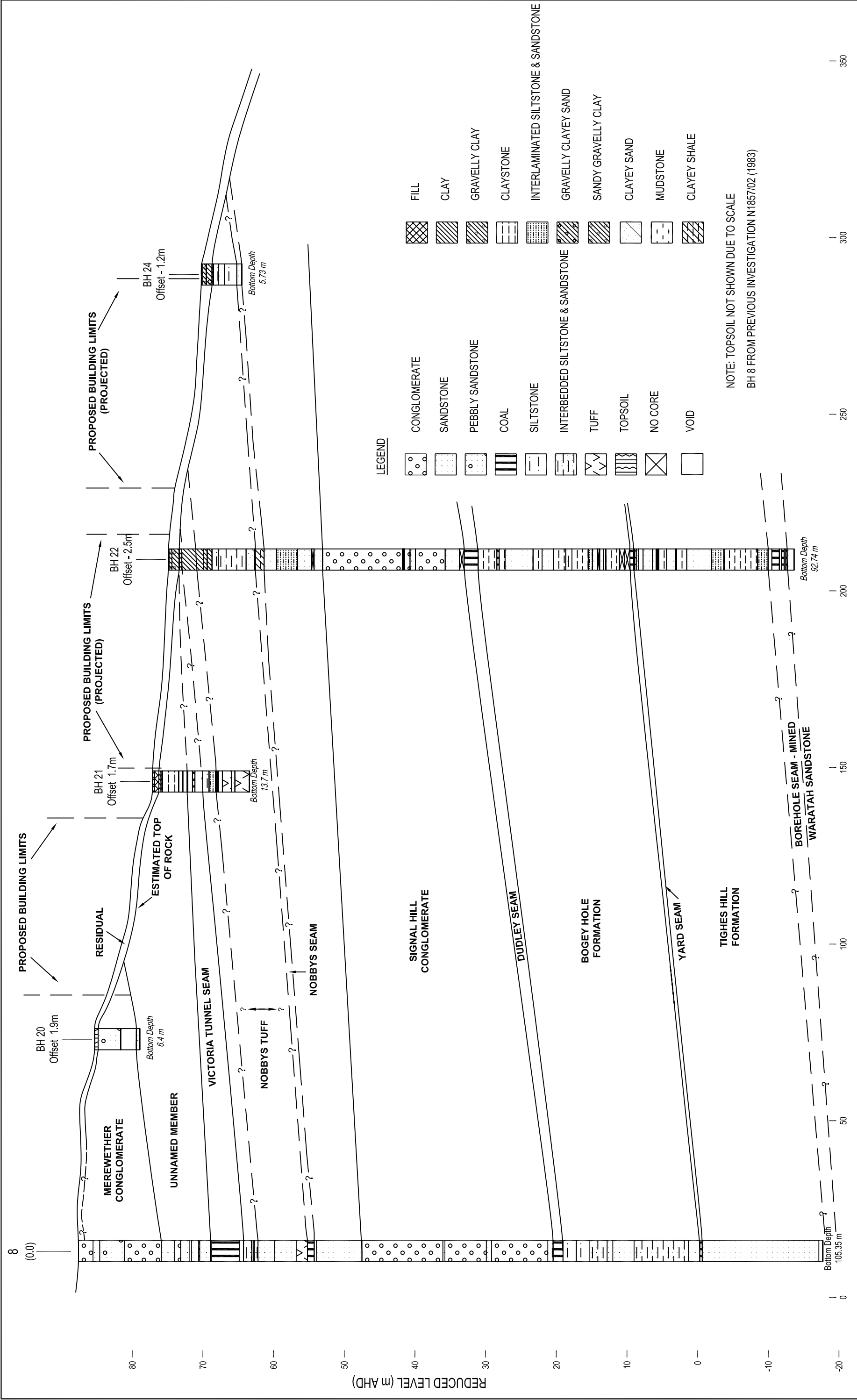
Coffey is familiar with a variety of techniques and approaches that can be used to help reduce risks for all parties to a project, from design to construction. It is common that not all approaches will be necessarily dealt with in your site assessment report due to concepts proposed at that time. As the project progresses through design towards construction, speak with Coffey to develop alternative approaches to problems that may be of genuine benefit both in time and cost.

Responsibility

Reporting relies on interpretation of factual information based on judgement and opinion and has a level of uncertainty attached to it, which is far less exact than the design disciplines. This has often resulted in claims being lodged against consultants, which are unfounded. To help prevent this problem, a number of clauses have been developed for use in contracts, reports and other documents. Responsibility clauses do not transfer appropriate liabilities from Coffey to other parties but are included to identify where Coffey's responsibilities begin and end. Their use is intended to help all parties involved to recognise their individual responsibilities. Read all documents from Coffey closely and do not hesitate to ask any questions you may have.

* For further information on this aspect reference should be made to "Guidelines for the Provision of Geotechnical information in Construction Contracts" published by the Institution of Engineers Australia, National headquarters, Canberra, 1987.

Figures



revision				client:			
	description	drawn	approved	date	APP CORPORATION PTY LTD		
					project:		
					PROPOSED HMRI BUILDING JOHN HUNTER HOSPITAL, NEW LAMBTON GEOTECHNICAL ASSESSMENT		
					title:		
					SECTION A-A		
					project no:	GEOTWARA20576AB	figure no: FIGURE AC2



coffey
geotechnics
SPECIALISTS MANAGING
THE EARTH

drawn

approved

date

scale

original size

ALR

ABL

13-05-2009

H - 1:1000
V - 1:500

A3

Horizontal Scale (metres)

Vertical Scale (metres)

Appendix A

Results of Field Investigations

Soil Description Explanation Sheet (1 of 2)

DEFINITION:

In engineering terms soil includes every type of uncemented or partially cemented inorganic or organic material found in the ground. In practice, if the material can be remoulded or disintegrated by hand in its field condition or in water it is described as a soil. Other materials are described using rock description terms.

CLASSIFICATION SYMBOL & SOIL NAME

Soils are described in accordance with the Unified Soil Classification (UCS) as shown in the table on Sheet 2.

PARTICLE SIZE DESCRIPTIVE TERMS

NAME	SUBDIVISION	SIZE
Boulders		>200 mm
Cobbles		63 mm to 200 mm
Gravel	coarse	20 mm to 63 mm
	medium	6 mm to 20 mm
	fine	2.36 mm to 6 mm
Sand	coarse	600 μ m to 2.36 mm
	medium	200 μ m to 600 μ m
	fine	75 μ m to 200 μ m

MOISTURE CONDITION

Dry Looks and feels dry. Cohesive and cemented soils are hard, friable or powdery. Uncemented granular soils run freely through hands.

Moist Soil feels cool and darkened in colour. Cohesive soils can be moulded. Granular soils tend to cohere.

Wet As for moist but with free water forming on hands when handled.

CONSISTENCY OF COHESIVE SOILS

TERM	UNDRAINED STRENGTH S_u (kPa)	FIELD GUIDE
Very Soft	<12	A finger can be pushed well into the soil with little effort.
Soft	12 - 25	A finger can be pushed into the soil to about 25mm depth.
Firm	25 - 50	The soil can be indented about 5mm with the thumb, but not penetrated.
Stiff	50 - 100	The surface of the soil can be indented with the thumb, but not penetrated.
Very Stiff	100 - 200	The surface of the soil can be marked, but not indented with thumb pressure.
Hard	>200	The surface of the soil can be marked only with the thumbnail.
Friable	–	Crumbles or powders when scraped by thumbnail.

DENSITY OF GRANULAR SOILS

TERM	DENSITY INDEX (%)
Very loose	Less than 15
Loose	15 - 35
Medium Dense	35 - 65
Dense	65 - 85
Very Dense	Greater than 85

MINOR COMPONENTS

TERM	ASSESSMENT GUIDE	PROPORTION OF MINOR COMPONENT IN:
Trace of	Presence just detectable by feel or eye, but soil properties little or no different to general properties of primary component.	Coarse grained soils: <5% Fine grained soils: <15%
With some	Presence easily detected by feel or eye, soil properties little different to general properties of primary component.	Coarse grained soils: 5 - 12% Fine grained soils: 15 - 30%

SOIL STRUCTURE

ZONING	CEMENTING
Layers Continuous across exposure or sample.	Weakly cemented Easily broken up by hand in air or water.
Lenses Discontinuous layers of lenticular shape.	Moderately cemented Effort is required to break up the soil by hand in air or water.
Pockets Irregular inclusions of different material.	

GEOLOGICAL ORIGIN

WEATHERED IN PLACE SOILS

Extremely weathered material Structure and fabric of parent rock visible.

Residual soil Structure and fabric of parent rock not visible.

TRANSPORTED SOILS

Aeolian soil Deposited by wind.

Alluvial soil Deposited by streams and rivers.

Colluvial soil Deposited on slopes (transported downslope by gravity).

Fill Man made deposit. Fill may be significantly more variable between tested locations than naturally occurring soils.

Lacustrine soil Deposited by lakes.

Marine soil Deposited in ocean basins, bays, beaches and estuaries.

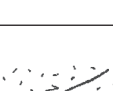
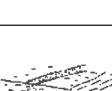
Soil Description Explanation Sheet (2 of 2)

SOIL CLASSIFICATION INCLUDING IDENTIFICATION AND DESCRIPTION

FIELD IDENTIFICATION PROCEDURES (Excluding particles larger than 60 mm and basing fractions on estimated mass)					USC	PRIMARY NAME
COARSE GRAINED SOILS More than 50% of materials less than 63 mm is larger than 0.075 mm	GRAVELS More than half of coarse fraction is larger than 2.0 mm	CLEAN GRAVELS (Little or no fines)	Wide range in grain size and substantial amounts of all intermediate particle sizes.		GW	GRAVEL
			Predominantly one size or a range of sizes with more intermediate sizes missing.		GP	GRAVEL
		GRAVELS WITH FINES (Appreciable amount of fines)	Non-plastic fines (for identification procedures see ML below)		GM	SILTY GRAVEL
			Plastic fines (for identification procedures see CL below)		GC	CLAYEY GRAVEL
	SANDS More than half of coarse fraction is smaller than 2.0 mm	CLEAN SANDS (Little or no fines)	Wide range in grain sizes and substantial amounts of all intermediate sizes		SW	SAND
			Predominantly one size or a range of sizes with some intermediate sizes missing.		SP	SAND
		SANDS WITH FINES (Appreciable amount of fines)	Non-plastic fines (for identification procedures see ML below).		SM	SILTY SAND
			Plastic fines (for identification procedures see CL below).		SC	CLAYEY SAND
FINE GRAINED SOILS More than 50% of material less than 63 mm is smaller than 0.075 mm	SILTS & CLAYS Liquid limit less than 50	IDENTIFICATION PROCEDURES ON FRACTIONS <0.2 mm.				
		DRY STRENGTH	DILATANCY	TOUGHNESS		
		None to Low	Quick to slow	None	ML	SILT
		Medium to High	None	Medium	CL	CLAY
	SILTS & CLAYS Liquid limit greater than 50	Low to medium	Slow to very slow	Low	OL	ORGANIC SILT
		Low to medium	Slow to very slow	Low to medium	MH	SILT
		High	None	High	CH	CLAY
		Medium to High	None	Low to medium	OH	ORGANIC CLAY
HIGHLY ORGANIC SOILS	Readily identified by colour, odour, spongy feel and frequently by fibrous texture.			Pt	PEAT	
• Low plasticity – Liquid Limit W _L less than 35%. • Medium plasticity – W _L between 35% and 50%.						

• Low plasticity – Liquid Limit W_L less than 35%. • Medium plasticity – W_L between 35% and 50%.

COMMON DEFECTS IN SOIL

TERM	DEFINITION	DIAGRAM	TERM	DEFINITION	DIAGRAM
PARTING	A surface or crack across which the soil has little or no tensile strength. Parallel or sub parallel to layering (eg bedding). May be open or closed.		SOFTENED ZONE	A zone in clayey soil, usually adjacent to a defect in which the soil has a higher moisture content than elsewhere.	
JOINT	A surface or crack across which the soil has little or no tensile strength but which is not parallel or sub parallel to layering. May be open or closed. The term 'fissure' may be used for irregular joints <0.2 m in length.		TUBE	Tubular cavity. May occur singly or as one of a large number of separate or inter-connected tubes. Walls often coated with clay or strengthened by denser packing of grains. May contain organic matter	
SHEARED ZONE	Zone in clayey soil with roughly parallel near planar, curved or undulating boundaries containing closely spaced, smooth or slickensided, curved intersecting joints which divide the mass into lenticular or wedge shaped blocks.		TUBE CAST	Roughly cylindrical elongated body of soil different from the soil mass in which it occurs. In some cases the soil which makes up the tube cast is cemented.	
SHEARED SURFACE	A near planar curved or undulating, smooth, polished or slickensided surface in clayey soil. The polished or slickensided surface indicates that movement (in many cases very little) has occurred along the defect.		INFILLED SEAM	Sheet or wall like body of soil substance or mass with roughly planar to irregular near parallel boundaries which cuts through a soil mass. Formed by infilling of open joints.	

Rock Description Explanation Sheet (1 of 2)

The descriptive terms used by Coffey are given below. They are broadly consistent with Australian Standard AS1726-1993.

DEFINITIONS: Rock substance, defect and mass are defined as follows:

Rock Substance In engineering terms rock substance is any naturally occurring aggregate of minerals and organic material which cannot be disintegrated or remoulded by hand in air or water. Other material is described using soil descriptive terms. Effectively homogenous material, may be isotropic or anisotropic.

Defect Discontinuity or break in the continuity of a substance or substances.

Mass Any body of material which is not effectively homogeneous. It can consist of two or more substances without defects, or one or more substances with one or more defects.

SUBSTANCE DESCRIPTIVE TERMS:

ROCK NAME Simple rock names are used rather than precise geological classification.

PARTICLE SIZE Grain size terms for sandstone are:
Coarse grained Mainly 0.6mm to 2mm
Medium grained Mainly 0.2mm to 0.6mm
Fine grained Mainly 0.06mm (just visible) to 0.2mm

FABRIC Terms for layering of penetrative fabric (eg. bedding, cleavage etc.) are:

Massive No layering or penetrative fabric.

Indistinct Layering or fabric just visible. Little effect on properties.

Distinct Layering or fabric is easily visible. Rock breaks more easily parallel to layering of fabric.

CLASSIFICATION OF WEATHERING PRODUCTS

Term	Abbreviation	Definition
Residual Soil	RS	Soil derived from the weathering of rock; the mass structure and substance fabric are no longer evident; there is a large change in volume but the soil has not been significantly transported.
Extremely Weathered Material	XW	Material is weathered to such an extent that it has soil properties, ie, it either disintegrates or can be remoulded in water. Original rock fabric still visible.
Highly Weathered Rock	HW	Rock strength is changed by weathering. The whole of the rock substance is discoloured, usually by iron staining or bleaching to the extent that the colour of the original rock is not recognisable. Some minerals are decomposed to clay minerals. Porosity may be increased by leaching or may be decreased due to the deposition of minerals in pores.
Moderately Weathered Rock	MW	The whole of the rock substance is discoloured, usually by iron staining or bleaching, to the extent that the colour of the fresh rock is no longer recognisable.
Slightly Weathered Rock	SW	Rock substance affected by weathering to the extent that partial staining or partial discolouration of the rock substance (usually by limonite) has taken place. The colour and texture of the fresh rock is recognisable; strength properties are essentially those of the fresh rock substance.
Fresh Rock	FR	Rock substance unaffected by weathering.

Notes on Weathering:

- AS1726 suggests the term "Distinctly Weathered" (DW) to cover the range of substance weathering conditions between XW and SW. For projects where it is not practical to delineate between HW and MW or it is judged that there is no advantage in making such a distinction. DW may be used with the definition given in AS1726.
- Where physical and chemical changes were caused by hot gasses and liquids associated with igneous rocks, the term "altered" may be substituted for "weathering" to give the abbreviations XA, HA, MA, SA and DA.

ROCK SUBSTANCE STRENGTH TERMS

Term	Abbreviation	Point Load Index, I_{s50} (MPa)	Field Guide
Very Low	VL	Less than 0.1	Material crumbles under firm blows with sharp end of pick; can be peeled with a knife; pieces up to 30mm thick can be broken by finger pressure.
Low	L	0.1 to 0.3	Easily scored with a knife; indentations 1mm to 3mm show with firm bows of a pick point; has a dull sound under hammer. Pieces of core 150mm long by 50mm diameter may be broken by hand. Sharp edges of core may be friable and break during handling.
Medium	M	0.3 to 1.0	Readily scored with a knife; a piece of core 150mm long by 50mm diameter can be broken by hand with difficulty.
High	H	1 to 3	A piece of core 150mm long by 50mm can not be broken by hand but can be broken by a pick with a single firm blow; rock rings under hammer.
Very High	VH	3 to 10	Hand specimen breaks after more than one blow of a pick; rock rings under hammer.
Extremely High	EH	More than 10	Specimen requires many blows with geological pick to break; rock rings under hammer.

Notes on Rock Substance Strength:

- In anisotropic rocks the field guide to strength applies to the strength perpendicular to the anisotropy. High strength anisotropic rocks may break readily parallel to the planar anisotropy.
- The term "extremely low" is not used as a rock substance strength term. While the term is used in AS1726-1993, the field guide therein makes it clear that materials in that strength range are soils in engineering terms.
- The unconfined compressive strength for isotropic rocks (and anisotropic rocks which fall across the planar anisotropy) is typically 10 to 25 times the point load index (I_{s50}). The ratio may vary for different rock types. Lower strength rocks often have lower ratios than higher strength rocks.

