Appendix J

Geotechnical Report

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SPECIALISTS MANAGING THE EARTH

NOTE: EXTRACT - pp.1-32 only, Figure 9 only

PROJECT 50363 - POTTS HILL GEOTECHNICAL INVESTIGATION REPORT

Landcom
Level 2, 330 Church Street, Parramatta, NSW 2150

GEOTLCOV23274AA-AG
30 January 2008

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8/12 Mars Road Lane Cove West NSW 2066 Australia

30 January 2008

Landcom
Level 10, 60 Castlereagh Street
Sydney, NSW, 2000

Attention: Nicole Woodrow

Dear Nicole

RE: Project 50363 - Potts Hill Geotechnical Investigation Report

Please find enclosed our revised report for the geotechnical investigation carried out at Potts Hill.

Please do not hesitate to contact the undersigned should you have any comments or queries regarding this report.

For and on behalf of Coffey Geotechnics Pty Ltd

Duncan E. Lowe

Associate

Distribution: Original held by Coffey Geotechnics Pty Ltd

4 Paper Copies and 1 Electronic copy (PDF Format) to Landcom.

Figures

Appendices

1 INTRODUCTION

This report presents the results of a geotechnical investigation carried out by Coffey Geotechnics Pty Ltd (Coffey) on behalf of Landcom for the land in the vicinity of Potts Hill Reservoir, Potts Hill, New South Wales.

At the time of writing this report, Coffey understands that the area to the north-west, west and south west of Reservoir No. 2 (Zones 1, 2, 3 and 4 and the area adjacent to Bagdad Street) are being considered for residential development. The area to the east of Reservoir No. 2 (Zones 5, 6 and 7) is understood to be under consideration for light industrial development provisionally, new Sydney Water Corporation (SWC) Offices, a Police Station and Energy Australia Offices and an electricity substation. The location of the zones are shown in Figure 2.

2 STUDY OBJECTIVES

During the desk study investigations discussed below, a requirement for additional site specific geotechnical information was identified. The objectives of this investigation are to provide information on subsurface conditions, a geotechnical model, and discussion and recommendations on geotechnical aspects of the proposed development. This includes options for ground improvement, earthworks and site preparation, excavation conditions and support, foundation options including suitable footing types, possible retaining wall options, foundation design parameters and preliminary lot classification in accordance with AS2870-1996. Recommendations regarding pavement and floor slab design parameters and subgrade preparation are also discussed. The propose of this report is also to provide an assessment of soil salinity, sodicity, dispersion potential and soil aggressivity.

3 THE SITE

3.1 Site Description

Potts Hill Reservoir is located approximately 18km west of Sydney Central Business District in the suburb of Potts Hill, New South Wales. The area is presently occupied by Sydney Water Corporations (SWC) Potts Hill Reservoir No. 2 and associated services, facilities and surrounding grounds. This comprises a total area of approximately 126 hectares, this extent of SWC land is referred to in this report as the Facility.

In 2003, Sydney Water Corporation (SWC) commenced Environmental Site Assessment (ESA) of the Facility. Under this assessment, the Facility was divided into eleven zones based predominantly on land use and history. Consideration is being given by Landcom to develop Zones 1, 2, 3, 4, 5, 6 and 7 and the annex area adjacent to Bagdad Street. Collectively these areas of are referred to in this report as the Site.

The Site generally comprises relatively level land surrounding Reservoir No. 2. This land is elevated above the adjacent Cooper Road, Graf Avenue, Brunker Road, Rookwood Road and the Sydney-Bankstown Railway line to the north. The large level area forming the majority of the site was created by placement of excavated natural materials during reservoir construction. The margins of the fill areas form relatively steep slopes in some sections of the Site. The main features of the individual Site Zones are as follows:

3.1.1 Zone 1

Zone 1 occupies the north-western corner of the Facility comprising an irregular polygon of approximately 8.5 hectares. This zone shares boundaries with Zones 11 (east), Zone 2 (south and east), residential properties along Cooper Road (west) and the Sydney-Bankstown Railway Line and SWC water supply pipelines (to the north). The area is presently occupied by SWC's Hydrographic Services Building and industrial buildings that at the time of this investigation were undergoing demolition/partial demolition. The southern part of this zone was also used for storage of construction plant, equipment and materials and pipeline components. This zone lies typically at elevations between RL43m and RL49m. The eastern boundary of this area typically comprises an embankment that separates this zone from the adjacent zone with elevations typically above RL51m.

3.1.2 Zone 2

Zone 2 is located immediately south of Zone 1 and comprises an area of approximately 7.2 hectares. The area is bounded to the north east by SWC water supply canal, to the east by Potts Hill Reservoir No.2 and adjacent easement, to the south by an east-west trending drainage ditch that delineates Zone 2 from Zone 3 and to the west by the rear gardens and yards of the residential properties along Cooper Road. The northern boundary of Zone 2 is denoted by a distinct change in elevation. This slope has typical gradients of between 5H:1V and 7H:1V and forms the boundary with Zone 1.

The northern portion of Zone 2 is occupied by a collection of buildings and structures which are used as a SWC staff training facilities. The remainder of Zone 2 is a predominantly open area of mowed grass that lies at an elevation of between RL55m and RL56m.

To the west of the Zone 2, is a slope with typical gradients of between 4H:1V and 5H:1V, locally with gradients steeper than 2H:1V. This slope separates the elevated mowed grass area from a lower lying (approximate RL47m to RL50m) area between the toe of the slope and the residential properties along Cooper Road.

Note: Slope angles are expressed in this report as gradients in terms of Horizontal (H) to Vertical (V).

3.1.3 Zone 3

Zone 3 is located immediately to the south of Zone 2, occupying the south-western corner of the Site and comprising approximately 10.5 hectares. This zone is bounded to the east by Potts Hill Reservoir No.2 and adjacent easement, to the north by Zone 2, to the south by Brunker Road, to the west by the residential properties of Cooper Road and to the south east by a densely vegetated steeply sloping area that separates Zone 3 from the SWC properties at 36b Brunker Road (Zone 4).

The majority of Zone 3 comprises an elevated area typically between RL55m and RL56m, occupied predominantly by scrub vegetation. The relatively steep slope with mature vegetation mentioned in Zone 2 continues into Zone 3, and runs close to the western boundary with Cooper Road.

3.1.4 Zone 4

Zone 4 presently contains administration offices, workshops and car parking for SWC 36b Brunker Road compound. This zone is a roughly triangular area of approximately 4 hectares. It is bounded to the north east by Reservoir No. 2, to the south by Brunker Road and to the north west by the densely vegetated sloping area mentioned in Zone 3 above.

3.1.5 Zone 5

Zone 5 is located in the south east corner of the Site and comprises approximately 2.7 hectares. The area is bounded to the south by Brunker Road, to the east by Graff Avenue, to the west by Reservoir No. 2 and to the north by Zone 6. At the time of this investigation, the majority of Zone 5 was occupied by derelict and working SWC office facilities and associated car parking, equipment storage sheds and open storage areas. The southern and eastern edges of Zone 5 are denoted by a relatively steep slope down to Brunker Road and Graff Avenue respectively.

3.1.6 Zone 6

Zone 6 comprises an area of approximately 9.5 hectares. It is located to the east of Reservoir No.2, to the south of Zone 11 and to the north of Zone 5. The eastern boundary of Zone 6 comprises a steep slope down to a chainlink fence that denotes the site boundary along Graf Avenue.

At the time of this investigation, the majority of Zone 6 was unsealed and comprised rough grassland. Road and tracks cross cut this area and three buildings were observed in the north-western section of the zone. It is understood that these buildings were formerly used for equipment storage and anecdotal information indicates that zone was used as open storage for construction materials.

3.1.7 Zone 7

Zone 7 comprises an area of approximately 2.4 hectares and is located to the north of the Potts Park Greyhound Track, to the south of the abandoned Potts Hill Reservoir No. 1, to the west of Rookwood Road and to the east of Zone 6. At the time of this investigation, the area was occupied by abandoned storage sheds and hardstanding with several pockets of relatively mature vegetation.

3.1.8 Area Adjacent to Bagdad Street

Located approximately 50m north of the main Facility is a triangular shaped area of approximately 0.6 hectares. This area is bounded by chainlink fences that delineated the site from the SWC supply pipeline to the north, the Sydney-Bankstown Railway line to the south and Bagdad Street and residential properties to the west.

At the time of this investigation, this area of land was dissected approximately midway by a south west-north east trending drainage ditch. To the west of this ditch, was an open area of mowed grass that sloped gently to the south. To the east of this ditch was rough grass and scrub vegetation. Discarded concrete railway sleepers were observed in the east of this zone indicating that this area has possibly been used for open storage of railway construction materials.

3.2 Published Maps

Based on the Geological map of the area (Sydney Geological Series Sheet 9130 (1:100,000)), the Site is underlain by the Bringelly Shale of the Wianamatta Group (Triassic Age) comprising shale, carbonaceous claystone, laminites and fine to medium grained sandstone.

Reference to the Salinity Potential in Western Sydney Map (2002) published by the Department of Infrastructure Planning and National Resources, indicates that the site has areas of both moderate and high salinity potential.

Reference to the Sydney 1:100 000 Soil Landscape Sheet indicates that the site is located within the Blacktown Soils Landscape in which the clayey subsoils are assessed to be moderately reactive.

4 PREVIOUS WORK

In March 2006, Douglas and Partners Pty Ltd (DP) issued a report entitled "Early Economic and Risk Analysis (EERA) Potts Hill Reservoir Redevelopment, Potts Hill". The purpose of this report was to "identify the various issues that need to be addressed so that Landcom and Sydney Water can plan and budget costs associated with such studies". The information presented in the report was derived from a desk study of available DP reports and a search of records held at the Sydney Water Geotechnical Branch and Survey Branch and site visits by DP personnel.

Information relevant to the proposed Site included in the DP report comprised extracts from the following investigations:

1) Sydney Water Catchments Authorities Potts Hill Reservoir No. 2 Surveillance Report dated June 2003.

2) Ground investigation dated August 1970 into a slump failure that occurred along the southern wall of Potts Hill Reservoir No. 1. This investigation comprised drilling, sampling and insitu testing of seven boreholes, although only one borehole log (BH5) was included in the report.

3) A foundation investigation for the Yagoona Gantry Crane dated March 1972. This investigation comprised the drilling of two boreholes in the south west of the site, the location of which is shown in Figures 2, 8 and 9 and referenced in this report as YGCF1 and YGCF2.

4) A water seepage investigation along Graf Avenue and supplementary groundwater monitoring report dated December 1977 and June 1978 respectively.

5) A drilling investigation for a proposed Plant Sub-Branch, Birrong Administration Building proposed to be located in the north west of the Site, report dated June 1984. This investigation comprised the drilling of four boreholes to a maximum depth of 8.05m (BH1) the location of which is shown in Figures 2 and 3 and referenced in this report PSBBH1.

6) Investigation into a "Slope Failure South of Potts Hill Reservoir No. 2" dated February 1989. This investigation comprised surface reconnaissance and sampling and drilling one borehole to investigate a slope failure on the south eastern reservoir embankment in Zone 4 to the north east of SWC 36b Brunker Road facilities. The location of this borehole is shown in Figures 2 and 7 and referenced YEDBH21.

7) Metropolitan Board of Water Supply and Sewerage construction/design drawings for Potts Hill Reservoir No. 2, dated May 1923.

8) Historical information from a document referenced "Potts Hill Reservoir Sites CMP".

Other information referenced in the DP report, but which lie outside the proposed Site area include extracts from:

9) A foundation investigation for a Meters Workshop site located to the south of Brunker Road (dated April 1973).

10) A borehole investigation for the proposed Potts Hill Office Building that appeared to be proposed in Zone 11 although the exact location cannot accurately be determined (dated February 1981).

11) An investigation into Potts Hill Reservoir No.1 Slip on Rookwood Road near the junction with Muir Road (dated November 1988).

12) A drilling investigation for Potts Hill-Waterloo Pressure Tunnel (dated November 1993) located in the north east corner of Reservoir No. 1.

The DP report identified the following geotechnical risks at the Potts Hill site:

- The extent of existing fill in the proposed development areas.
- The salinity of the site soils.
- The reactivity, or shrink swell potential, of the site soils.
- The stability of natural slopes, existing filling embankments and cut slopes.
- The possibility of elevated groundwater levels in the areas below the reservoirs due to long term leakage.
- Foundation requirements and development control, earthwork foundations that will be required to address the above slope stability issues.
- Constraints in building over and near to pipelines and other buried utilities.

Preliminary recommendations regarding foundation and ground treatments options were also provided in DP report.

This work was subsequently reviewed by Pells Sullivan and Meynink Pty Ltd (PSM) in their report entitled “Geotechnical Early Economic Risk Analysis for Potts Hill Redevelopment” (PSM Reference: PSM1025.R1 dated 24 July 2006. PSM broadly concur with the DP report apart from the following summarised differences:

- PSM recommend Long term batter angles of 3H:1V, instead of 2:1 as advised in the DP report.
- For Zones 1 to 4, PSM recommend ground treatment by unloading and removal of 1m of fill so that the net increase in load by development is minimal. Followed by ground surface conditioning.
- For Zones 5 and 6, PSM recommend pre-loading assuming a 2m surcharge for approximately 18 months.
- Soil erosion identified as a potential risk.
- Revised qualitative risk assessment matrices.
- Requirement for additional geotechnical investigation work.

In addition to the above, environmental investigation works have also been carried out at the site by URS Australia Pty Ltd and Coffey Geosciences Pty Ltd. The results of the URS investigations are presented in a series of reports dated March 2005 entitled “Phase II Environmental Site Assessment”.

The results of the Coffey Geosciences investigations are presented in a series of reports dated July 2006 entitled “Stage II Additional Environmental Site Assessment”.

The information in the URS and Coffey Geosciences reports predominantly relates to surface and near-surface ground conditions. Where the information in these reports is considered pertinent to the proposed Landcom development, the information has been used. Exploratory holes considered pertinent are listed in Table A with the location of the exploratory holes presented in Figures 2 to 9. A copy of the corresponding exploratory hole log is enclosed in Appendix E.

TABLE A: EXPLORATORY HOLES CONSIDERED

Zone	Exploratory Hole Reference	
	URS	**Coffey Geosciences**
Zone 1	Z1A_TP01 to TP02, TP07 to TP08 Z1B_BH01 to BH04, BH06 to BH08, BH11 to BH14, BH17 to BH20, BH22 to BH28, BH30, BH34 to BH36, BH38 Z1B_TP05, TP10, TP15, TP21	Z1A_BH101 to BH103 Z1B_BH106, BH109, BH111 to BH114
	Z1C_BH21 Z1C_TP01 to TP10, TP12 to TP20, TP22 to TP23	Z1C_TP104 to TP109, TP111 to TP113, TP115 to TP139
Zone 2	Z2_TP01 to TP07, TP09 to TP27, TP30, TP31a to c, TP32 to TP35	
Zone 3	Z3_TP01 to TP14, TP16 to TP19, TP21, TP23 to TP27, TP30 to TP33, TP36 to TP38, TP43, TP51 to TP53, TP55 to TP58, TP09a Z3_BH57	Z3_TP101 to TP102, TP104 to TP118
Zone 4	Z4_TP01 to TP04 Z4_BH07 to BH15, BH18 to BH28, BH31	Z4_TP107
Zone 5	Z5_TP13, TP19, TP22 to TP24, TP26 to TP30 Z5_BH01 to BH03, BH06 to BH08, BH11a, BH12, BH14, BH16 to BH18, BH20 to BH21, BH25	Z5_TP102 to TP103, TP110 to TP111

Zone	Exploratory Hole Reference	
	URS	Coffey Geosciences
Zone 6	Z6_TP03, TP06, TP09 to TP11, TP13 to TP16, TP18 to TP19, TP42 to TP48, TP50, TP53 to TP55, TP57, TP60 to TP70 Z6_BH01 to BH02, BH07 to BH08	Z6_TP101 to TP141 Z6_BH101 to BH104, BH106 to BH111

5 GROUND INVESTIGATION FIELDWORK

Fieldwork was carried out between the 13 August and 21 September 2007 and comprised the drilling of twenty boreholes, one test pit and downhole compensated gamma density logging in thirteen boreholes. Groundwater monitoring/environmental sampling instrumentation was also installed in six boreholes.

An Engineering Geologist from Coffey was in full time attendance at the site to locate the boreholes, carry out field screening, oversee downhole testing, record test results and log samples from the exploratory holes. The positions of the exploratory holes were measured from salient site features and the grid coordinates and ground levels interpolated from the survey information provided by Landcom.

5.1 Boreholes

The boreholes were drilled using a truck mounted drilling rig. The location of the borehole positions is shown in Figure 1 and described below:

Zone	Borehole Reference	Depths
Zone 1	PHBH1 to PHBH5	5.57m and 11.58m
Zones 2 and 3	PHBH6 to PHBH11	5.4m and 17.06m
Zone 4	PHBH19 to PHBH21	5.36m and 12.84m
Zones 5, 6 and 7 (Eastern Precinct)	PHBH12, PHBH13, PHBH15 and PHBH17	8.6m and 17.5m
Bagdad Street Area	PHBH22	5.24m

Borehole reference numbers PHBH16 and PHBH18 were not used.

The boreholes were augered in soil and weak rock using a Tungsten Carbide drill bit. Standard Penetration Tests were carried out in soils and weak rock to assess strength and to obtain samples for logging. Tube samples (U_{50}) were also taken for laboratory testing and to carry out density verification assessment, salinity screening and detailed sample description.

The boreholes were advanced in rock using rotary coring techniques. A triple tube NMLC core barrel with diamond impregnated drill bit and water as the drilling flush was used to core the rock.

Where groundwater monitoring instrumentation was not installed, the boreholes were backfilled on completion with drill cuttings to ground surface.

Engineering logs describing the ground conditions encountered in the boreholes and corresponding photographs of the cored material recovered are enclosed in Appendix A to this report, together with explanatory sheets defining the terms and symbols used.

5.2 Test Pit Excavation

A test pit (PHTP23) was excavated to a depth of 1.5m in the area adjacent to Bagdad Street on the eastern side of the drainage gully. The test pit was excavated using a backhoe and hydraulic impact hammer and bulk disturbed samples collected for laboratory testing.

The engineering log describing the ground conditions encountered in the test pit is enclosed in Appendix A to this report, together with explanatory sheets defining the terms and symbols used.

The backhoe was originally mobilised to the site to construct an access track and temporary bridge over the drainage gully for drilling rig access. Heavy rainfall and waterlogged soft ground made this impracticable. As a result, a test pit was excavated in place of the scheduled borehole after the presence of rock at a relatively shallow depth was demonstrated in nearby borehole PHBH22.

5.3 Insitu Testing and Field Screening

5.3.1 Downhole Compensated Gamma Density Logging

Downhole compensated gamma density logging was carried out in the boreholes with significant thicknesses of fill material and without groundwater monitoring instruments. In total, thirteen boreholes were logged using this technique. The results of this testing are presented in Appendix B to this report.

Downhole compensated gamma density logging is carried out by lowering a probe down the borehole. The probe contains a source that emits gamma rays and two detectors in the probe measure the gamma ray scattering which allows the apparent insitu density (bulk density) of the formation to be measured. The probe possesses a calliper tool that records borehole diameter which in conjunction with the results of the short space density detector allows the results of the long spaced density sensor to be adjusted to compensate for irregularities in the borehole sidewall such as washout zones and voids. The Compensated Density Logs (CDL) are therefore the appropriate results to be considered.

To verify the results of the gamma density logging, density checks were carried out on a number of tube (U_{50}) samples. The results of this density checking are shown on the gamma density logging plots along with the results of SPT test and blow counts recorded during driving of the U_{50} sampler. Initial wet densities determined during laboratory testing (range 1.92 to 2.18 tonnes/m^3) ^{also} show good correlation with the CDL assessed densities.

The downhole gamma density logging results appear reasonable based on the SPT and U_{50} drive results, density checks on the U_{50} samples and laboratory test results. It is noted that the density checks typically indicate slightly less dense material than assessed by the CDL, however this is likely to be due to factors such as sample disturbance and moisture loss during preservation, transportation and storage of the tube samples.

5.3.2 Salinity Screening

Screening for elevated levels of salinity was carried out on selected samples using a Eutech Electrical Conductivity meter on 1:5 soil:water extract subsamples. The results are corrected for soil texture and assessed in terms ECe at the appropriate positions on the borehole logs enclosed in Appendix A.

5.4 Groundwater Monitoring/Sampling Instrumentation

In collaboration with the Environmental Consultant, six groundwater monitoring and sampling wells were installed in boreholes PHBH4, PHBH5, PHBH10, PHBH13, PHBH17 and PHBH19. The wells were installed to allow assessment of groundwater levels and to allow environmental sampling and testing of the groundwater.

The groundwater monitoring/sampling wells comprise 50mm PVC slotted screen and well casing. The slotted screen is surrounded by clean graded sand filter and the well casing sealed in the ground with bentonite and a protective surface cover constructed. Details of the installations are provided on the borehole logs in Appendix D.

6 LABORATORY TESTING

Soil samples obtained during the investigation were taken to our NATA registered laboratory. The following tests were undertaken on selected samples:

- Moisture Content.
- Atterberg Limits (Liquid Limit, Plastic Limit and Linear Shrinkage).
- Particle Size Distribution (sieve and hydrometer analysis).
- Optimum Moisture Content (Compaction Tests).
- California Bearing Ratio (CBR).
- Shrink Swell Index Tests (Soil Reactivity Tests).
- Emerson Dispersion Classification.

In addition to the above, soil samples were tested by ALS Laboratory Group Pty Ltd, a NATA accredited laboratory, for the following determinants:

- pH.
- Soluble Sulphate.
- Soluble Chloride.
- Organic Matter Content.
- Sodicity (Exchangeable Sodium Percent (ESP))

The CBR testing was carried out on samples prepared at optimum moisture content and 98% Maximum Dry Density using Standard compactive effort, after 4-days of soaking and using 4.5kg surcharge.

Point Load Strength Index tests were also undertaken on the rock core recovered from boreholes. Photographs of the recovered cores were also taken.

The results of the laboratory testing are presented in Appendix C with the Point load test results presented on the borehole logs. The core photographs are presented in Appendix A behind the borehole log to which they refer.

7 SITE CONDITIONS

7.1 Subsurface Conditions

The results of the investigation indicate that the subsurface profile at the test locations typically comprises shale fill, overlying residual soil and interstratified shale and sandstone bedrock.

The fill must be considered uncontrolled fill as there are no records confirming that the fill was compacted in accordance with an engineering specification. It is probable that the majority of the fill over the site has been derived from crushed shale and sandstone excavated during the construction of Potts Hill Reservoir No. 2 in the 1920's. The fill thicknesses are variable over the site as illustrated in Tables 1 to 9.

A geotechnical model has been developed for each of these areas and is presented in Tables 1 to 9, together with a summary of subsurface conditions at the exploratory hole locations. For a detailed description of the subsurface conditions encountered at the exploratory hole locations, refer to the Engineering Logs in Appendix A, together with Explanation Sheets describing the terms and symbols used in their preparation.

TABLE 1: ZONE 1A - SUMMARY OF SUBSURFACE CONDITIONS AT EXPLORATORY HOLE LOCATIONS AND INFERRED GEOTECHNICAL MODEL

Unit	Description	Depth to Top of Unit (m)	Thickness (m)
1A. Fill	Variable fill containing materials such as slag, ash, roots, concrete, glass, brick, asphalt, tiles and domestic refuse. • Silty Clay, sandy silty CLAY, sandy CLAY, gravelly sandy CLAY, gravelly clayey SILT, silty SAND, sandy clayey GRAVEL. • Generally soft/loose.	Ground Level	Variable generally less than 1.2m thick. Encountered up to 2.4m thick in central area of Zone 1A.
1B. Fill (Crushed Shale)	Typically: • Silty CLAY • Low to medium plasticity. • Stiff or very stiff. • Gravel size material is crushed shale. • Compensated Insitu gamma density generally greater than 2 tonnes/m^3	0.45m to 1.8m	0.2m to 1.6m Thicker in central area of Zone 1A (up to 5.5m)

Unit	Description	Depth to Top of Unit (m)	Thickness (m)
2. Residual Soil	Typically: • Silty CLAY • Medium plasticity • Stiff to hard • Compensated Insitu gamma density assessed to be greater than 2.1 tonnes/m^3 in borehole PHBH1.	1m to 1.8m Top of stratum encountered in central area between 2.4m and 5.5m.	0.6m to 1m
3A Weathered Bedrock	Encountered in borehole PHBH1 at 6.5m (RL41.53m). • Extremely to highly weathered interlaminated and interbedded Sandstone and Shale. • Very low and low strength. • Class V and Class IV Shale*. • Compensated Insitu gamma density assessed to be greater than 2.25 tonnes/m^3 in borehole PHBH1.	1.2 to 2m Deeper in central area 6.95m (PHBH1)	3m (Note 1)
3B Bedrock	Encountered in borehole PHBH1 at 9.5m (RL38.53m). • Moderately and slightly weathered interlaminated and interbedded Sandstone and Shale. • Medium and high strength. • Class III and Class II Shale*. • Insitu density assessed to be greater than 2.5 tonnes/m^3 in borehole PHBH1	9.5m Drilled for a depth of up to 11.28m (PHBH1) (RL36.62m)	---

No groundwater was observed while augering in soil or during rotary coring. However, water was used as a drilling fluid during coring which obscures groundwater observations.

Note 1: Base of Stratum Penetrated in PHBH1 only.

TABLE 2: ZONE 1B - SUMMARY OF SUBSURFACE CONDITIONS AT EXPLORATORY HOLE LOCATIONS AND INFERRED GEOTECHNICAL MODEL

Unit	Description	Depth to Top of Unit (m)	Thickness (m)
1A. Fill	Highly variable fill containing materials such as slag, ash, roots, concrete and brick. • Sandy CLAY, gravelly sandy CLAY, clayey SILT, clayey SAND, silty SAND, gravelly silty SAND. • Ranging from loose and soft to very stiff	Ground Level	Variable generally less than 1m thick Up to 4.5m thick in elevated area at eastern boundary.

Unit	Description	Depth to Top of Unit (m)	Thickness (m)
1B. Fill (Crushed Shale)	Typically: • Silty CLAY • Low to medium plasticity. • Stiff or very stiff. • Gravel size material is crushed shale. • Compensated Insitu gamma density generally greater than 2 tonnes/m^3.	Not Significantly encountered in the Majority of Zone 1B	N/A
		Eastern and South Eastern elevated areas of Zone 1B 1m to 2m	Eastern and South Eastern elevated areas between 1.5m to 2.3m thick
2. Residual Soil	Typically: • Silty CLAY/Clayey SILT • High plasticity • Stiff to very stiff.	0.7m and 2.2m Encountered in east of Zone 1B between 2.9m and 3.8m	0.3m to 1.1m thick
3A. Weathered Bedrock	Typically: • Sandstone • Extremely to highly weathered. • Very low and low strength. • Class V and Class IV Sandstone*.	Typically 1.7m to 2.45m 9.2m in east of Zone 1B (PHBH3)	0.2m to 1.2m (Note 2)
3B. Bedrock	Encountered in borehole PHBH2 and PHBH3 at 2.8m (RL46m) and 10.4m (RL42.6m) respectively. • Moderately and slightly weathered Sandstone. • Medium and high strength. • Class III and Class II Sandstone*.	2.8m to 10.4m Drilled for a depth of up to 11.58m (RL41.42) (PHBH1)	---

No groundwater was observed while augering in soil or during rotary coring. However, water was used as a drilling fluid during coring which obscures groundwater observations.

Note 2: Base of stratum penetrated in PHBH2 and PHBH3.

TABLE 3: ZONE 1C - SUMMARY OF SUBSURFACE CONDITIONS AT EXPLORATORY HOLE LOCATIONS AND INFERRED GEOTECHNICAL MODEL

Unit	Description	Depth to Top of Unit (m)	Thickness (m)
1A. Fill	Highly variable fill containing roots and materials such as slag and ash, construction waste such as concrete, brick, cement sheeting, timber, metal, asphalt, electrical wire and domestic refuse. • Silty CLAY, gravelly silty CLAY/gravelly clayey SILT, clayey GRAVEL, sandy clayey GRAVEL. • Ranging from loose and very soft to stiff.	Ground Level	0.5m to 2m Up to 5m Thick in the Central Area of Zone 1C.
1B. Fill (Crushed Shale)	Generally Absent From Zone 1C.	N/A	N/A
2. Residual Soil	Typically: • Silty CLAY/Clayey SILT • Low to medium plasticity • Firm and stiff Note: Described as very soft in URS test pits excavated in north east and central areas of Zone 1C.	2.0m to 3.7m	1.1 to 1.8m
3A. Weathered Bedrock	Typically: • Shale • Extremely to highly weathered. • Very low and low strength. • Class V and Class IV Shale*.	2m to 3m Up to 4m in Central Area of Zone 1C	2.85m to 3.5m (Note 3)
3B. Bedrock	Typically: • Shale • Moderately and slightly weathered. • Medium and high strength. • Class III and Class II Shale*	6.9m to 7.6m Drilled for a depth of up to 8.52m (RL39.1m)	(Note 4)

Localised perched water was observed in the northern central part of Zone 1C to south of existing hangar buildings. Standing groundwater monitored in groundwater installations PHBH4 at 1.75m (RL45.8m) and PHBH5 at 3.70m (RL44.9m) on the 11 September 2007.

Note 3: Base of stratum penetrated in PHBH4 and PHBH5.

Note 4: Encountered in PHBH4 and PHBH5

TABLE 4: ZONES 2 AND 3 - SUMMARY OF SUBSURFACE CONDITIONS AT EXPLORATORY HOLE LOCATIONS AND INFERRED GEOTECHNICAL MODEL.

Unit	Description	Depth to Top of Unit (m)	Thickness (m)
1A. Fill	Typically not encountered in Zones 2 and 3. Local areas of fill containing construction waste such as asphalt, scrap metal, glass fragments encountered to depths greater than 2m in test pits Z3_TP07, Z3_TP08, Z3_TP116 , Z3_TP117 and Z3_TP118 located in the vicinity of the access track in the south west of Zone 3.	Typically Not Encountered	Typically Not Encountered
1B. Fill (Crushed Shale)	Typically: • Silty CLAY with gravel size fragments of crushed shale. • Low to medium plasticity. • Stiff and very stiff. • Compensated Insitu gamma density of generally greater than 2 tonnes/m^3.	Ground Level to less than 0.5m	5.2m to 8.9m
2. Residual Soil	Typically: • Silty CLAY. • Medium and high plasticity. • Stiff and very stiff. • Compensated Insitu gamma density generally greater than 2 tonnes/m^3	6m to 9m	1m to 5.75m
3A. Weathered Bedrock	Typically: • Shale interstratified with Sandstone • Extremely to highly weathered. • Very low and low strength. • Class V and Class IV Shale/Sandstone*. • Compensated Insitu gamma density generally greater than 2.2 tonnes/m^3.	7.5m to 12m	0.6m to 2.4m
3B. Bedrock	Typically: • Shale interstratified with Sandstone • Moderately and slightly weathered. • Medium and high strength. • Class III and Class II Shale/Sandstone* • Compensated Insitu gamma density generally greater than 2.5 tonnes/m^3	11m to 14m Drilled for a depth of up to 17.06m (RL38.58m)	---

Groundwater observed during drilling at 5.2m (RL50.8m) in PHBH6 and 5.5m (RL48.04m) in PHBH10. This is considered to be localised perched water as groundwater monitoring well installed in PHBH10 was found to be dry on the 11 September 2007.

TABLE 5: ZONE 4 - SUMMARY OF SUBSURFACE CONDITIONS AT EXPLORATORY HOLE LOCATIONS AND INFERRED GEOTECHNICAL MODEL

Unit	Description	Depth to Top of Unit (m)	Thickness (m)
1A. Fill	Road Pavement material • Clayey GRAVEL Local area near PHBH19 observed to contain up to 4.8m of fill with brick, discarded rubbish, glass and plastic.	Ground Level	Typically less than 0.6m
1B. Fill (Crushed Sandstone/ Shale)	Typically: • Clayey SAND/Sandy CLAY with gravel size fragments of crushed sandstone and shale. • Clay is low plasticity. • Stiff and very stiff/Medium dense • Compensated Insitu gamma density generally greater than 2 tonnes/m^3.	<0.6m	1m to 3.9m
2. Residual Soil	Typically: • Silty CLAY/Clayey SAND • Clay is medium and high plasticity. • Very stiff and hard/Very dense. • Compensated Insitu gamma density generally greater than 2 tonnes/m^3	5.3m (Near Reservoir No.2) to 1.5m (Near Brinker Road)	0.7m to 2.85m
3A. Weathered Bedrock	Typically: • Interstratified Sandstone and Shale • Extremely to highly weathered. • Very low and low strength. • Class V and Class IV Shale/Sandstone*. • Compensated Insitu gamma density generally greater than 2.3 tonnes/m^3.	2.4m to 6.75m	0.9m to 2.6m
3B. Bedrock	Typically: • Interstratified Sandstone and Shale • Moderately and slightly weathered. • Medium and high strength. • Class III and Class II Shale/Sandstone* • Compensated Insitu gamma density generally greater than 2.4 tonnes/m^3	3.3m to 10.3m Drilled for a depth of up to 12.84m (Elevation: RL51.2m)	---

Groundwater was not observed during the drilling works. Level monitored in groundwater monitoring well PHBH19 on the 11 September 2007 was 5.3m (RL58.8m).

TABLE 6: ZONES 5 SUMMARY OF SUBSURFACE CONDITIONS AT EXPLORATORY HOLE LOCATIONS AND INFERRED GEOTECHNICAL MODEL

Unit	Description	Depth to Top of Unit (m)	Thickness (m)
1A. Fill	Variable fill containing materials such as slag and ash, roots, plastic, asphalt, scrap metal, drainage pipe fragment and timber. • Sandy CLAY/Silty CLAY. • Firm.	Ground Level	Typically less than 1m. (locally up to 2m)
1B. Fill (Crushed Shale)	Typically: • Silty CLAY with gravel size fragments of crushed sandstone and shale. • Low, medium and high plasticity. • Stiff and very stiff. • Compensated insitu gamma density generally greater than 2 tonnes/m^3. Note: Less dense material (between 1.75 and 1.9 tonnes/m^3) assessed at base of this stratum.	<1m (Locally to 2m)	2.1m to 5.5m
2. Residual Soil	Typically: • Silty CLAY • Medium and high plasticity. • Very stiff and hard. • Compensated Insitu gamma density generally greater than 2 tonnes/m^3	3.2m to 6m (Locally to 7.7m)	0.5m to 1.3m
3A. Weathered Bedrock	Typically: • Interstratified Shale and Sandstone • Extremely to highly weathered. • Very low and low strength. • Class V and Class IV Shale/Sandstone*. • Compensated Insitu gamma density generally greater than 2.3 tonnes/m^3 in PHBH17.	5m to 6.6m	2.5m
3B. Bedrock	Typically: • Interstratified Sandstone and Shale • Moderately and slightly weathered. • Low to Medium strength. • Class III and Class II Shale/Sandstone*	9.10m Drilled for a depth of up to 10.18m (RL44.92m)	---

Groundwater observed at 1.1m in Test Pit Z5_TP110. Standing water level monitored in PHBH13 on the 11 September 2007 was 4.2m (RL51.8m).

TABLE 7: ZONES 6 – SUMMARY OF SUBSURFACE CONDITIONS AT EXPLORATORY HOLE LOCATIONS AND INFERRED GEOTECHNICAL MODEL

Unit	Description	Depth to Top of Unit (m)	Thickness (m)
1A. Fill	Fill containing materials such as asphalt, glass, brick, scrap metal, timber, drainage pipe fragment and roots. • Silty CLAY, Sandy CLAY, Gravelly SILT, silty SAND, Gravelly SAND, sandy GRAVEL, Clayey sandy GRAVEL, gravelly sandy CLAY, Silty CLAY. • Firm.	Ground Level	Typically Less than 0.5m
1B. Fill (Crushed Shale)	Typically: • Silty CLAY with gravel size fragments of crushed sandstone and shale. • Low, medium and high plasticity. • Stiff and very stiff. • Compensated insitu gamma density generally greater than 2 tonnes/m^3. Note: Less dense material (between 1.75 and 1.9 tonnes/m^3) assessed at base of this stratum.	0.2 to 0.5m	2.1m to 5.5m
2. Residual Soil	Typically: • Silty CLAY • Medium and high plasticity. • Very stiff and hard. • Compensated Insitu gamma density generally greater than 2 tonnes/m^3	3m to 6m	2m to 6m
3A. Weathered Bedrock	Typically: • Interstratified Shale and Sandstone • Extremely to highly weathered. • Very low and low strength. • Class V and Class IV Shale/Sandstone*. • Compensated Insitu gamma density generally greater than 2.3 tonnes/m^3 in PHBH17.	9.15m to 11.4m	2.6m to 6.25m
3B. Bedrock	Typically: • Interstratified Sandstone and Shale • Moderately and slightly weathered. • Low to Medium strength. • Class III and Class II Shale/Sandstone*	14m to 15.4m Drilled for a depth of up to 17.50m (RL38.50m)	---

Groundwater observed during drilling at 1.1m in Test Pit Z5_TP110, 3.3m (RL52.7m) in PHBH13 and 4.9m (RL50.6m) in PHBH12. Standing water level monitored in PHBH13 on the 11 September 2007 was 4.2m (RL51.8m).

TABLE 8: ZONE 7 - SUMMARY OF SUBSURFACE CONDITIONS AT EXPLORATORY HOLE LOCATION AND INFERRED GEOTECHNICAL MODEL

Unit	Description	Depth to Top of Unit (m)	Thickness (m)
1A. Fill	Encountered in PHBH15 as: • Silty CLAY. • Firm. • Contains black sand/ash.	Ground Level	0.15m
1B. Fill (Crushed Shale)	Not encountered in lower lying areas of Zone 7. Elevated areas near Reservoir No. 1 appear to have up to 3m.	N/E Approximately 0.5m near to Reservoir No. 1	N/A Approximately 3m near to Reservoir No. 1
2. Residual Soil	Encountered in PHBH15 as: • Silty CLAY. • High plasticity. • Firm and stiff.	0.15m	1.6m
3A. Weathered Bedrock	Encountered in PHBH15 as: • Shale. • Extremely and highly weathered. • Very low and low strength. • Class V and Class IV Shale/Sandstone*. • Compensated Insitu gamma density generally greater than 2.2 tonnes/m^3 in PHBH15.	1.75m	3.5m
3B. Bedrock	Typically: • Interstratified Sandstone and Shale • Moderately and slightly weathered. • Medium to high strength. • Class III and Class II Shale/Sandstone* • Compensated insitu gamma density generally greater than 2.6 tonnes/m3 in PHBH15.	5.35m Drilled for a depth of 8.6m in PHBH15. (RL39.2m)	---

Groundwater was not observed in PHBH15 during augering in soil and weak rock. The use of water as the drilling flush precludes the assessment of groundwater levels in this borehole.

TABLE 9: BAGDAD STREET AREA - SUMMARY OF SUBSURFACE CONDITIONS AT EXPLORATORY HOLE LOCATIONS AND INFERRED GEOTECHNICAL MODEL

Unit	Description	Depth to Top of Unit (m)	Thickness (m)
1A. Fill/Topsoil	Typically: • Clayey SILT. • Low and medium plasticity. • Soft. • With roots.	<0.3m	Up to 0.3m.
1B. Fill (Crushed Shale)	Not encountered in PHBH22 or PHTP23.	N/E	N/A
2. Residual Soil	Typically: • Silty CLAY • High plasticity. • Stiff and very stiff.	0.2m to 0.3m	0.8m to 1.4m
3A. Weathered Bedrock	Typically: • Interstratified Shale and Sandstone • Extremely to highly weathered. • Very low and low strength. • Class V and Class IV Shale/Sandstone*.	1.1m to 1.6m	1.4m (Note 8)
3B. Bedrock	Typically: • Interstratified Sandstone and Shale • Moderately and slightly weathered. • Low to Medium strength. • Class III and Class II Shale/Sandstone*	3m Drilled for a depth of up to 5.24m (RL33.56m) (Note 8)	---

Groundwater was not observed in PHBH22 and PHTP23. The use of water as the drilling flush in PHBH22 precludes the assessment of groundwater levels in this borehole deeper than 2.45m.

Note 8: Penetrated in PHBH22 only.

*Rock class assessed in accordance with Pells et all (1998) “Foundations on Sandstone and Shale in the Sydney Region” Aust. Geomech. Jnl. Dec 1998.

7.2 Laboratory Testing Results

The results of the laboratory testing are presented in Appendix B and summarised in Table 10.

TABLE 10: SUMMARY OF LABORATORY TESTING RESULTS

Unit	Sample Location(s)	Field Moisture Content (%)	Liquid Limit (%)	Plastic Limit (%)	Plasticity Index (%)	Clay (%)	Silt (%)	Sand (%)	Gravel (%)	CBR Value (%) (Note 9)	CBR Swell (%) (Note 9)	Initial Dry Density (t/m^3)	Initial Wet Density (t/m^3)	Bulk Density at MDD/OMC (t/m^3)	MDD (t/m^3)	OMC (%)	Shrink-Swell Index (I_{ss} %)	Emerson Dispersion Class	pH value	Soluble Sulphate (%)	Chloride Content (%)	Exchangeable Sodium (%)	Organic Matter Content (%)
1B. Fill (Crushed Shale)	**Zone 2** PHBH6 (0.1-1.45)	14.6	37	19	18	28	39	28	5	2.5	4	1.76	2.02	2.09	1.77	18.1	1.6	C5					
	Zone 2 PHBH7 (0.5-1.95)	12.9	30	16	14	36	46	14	4	2	2.5	1.93	2.18	2.10	1.86	13	1.6	C2					
	Zone 3 PHBH10 (0.1-1.4)	13.8 CBR								2.5	3.5			2.12	1.88	13							
	Zone 3 PHBH11 (1.4-2) & (2-2.4)	10.2	34	15	19							1.74	1.92				0.9		8.1	0.02	0.001	0.2	1.3
	Zone 5 PHBH17 (0.5-1.75)	11.6	34	18	16	18	25	23	34	2.5	3	1.76	1.96	2.16	1.89	14.5	1.3	C2	8.4	0.2	0.19	3	0.8
	Zone 6 PHBH12 (0.5-0.95)		31	17	14	15	30	48	7									C5					

Unit	Sample Location(s)	Field Moisture Content (%)	Liquid Limit (%)	Plastic Limit (%)	Plasticity Index (%)	Clay (%)	Silt (%)	Sand (%)	Gravel (%)	CBR Value (%) (Note 9)	CBR Swell (%) (Note 9)	Initial Dry Density (t/m^3)	Initial Wet Density (t/m^3)	Bulk Density at MDD/OMC (t/m^3)	MDD (t/m^3)	OMC (%)	Shrink-Swell Index (I_{ss} %)	Emerson Dispersion Class	pH value	Soluble Sulphate (%)	Chloride Content (%)	Exchangeable Sodium (%)	Organic Matter Content (%)
2. Residual Soil	**Zone 1B** PHBH2 (0.5-2.25)	13.4	51	16	35	42	36	18	4	2.5	2.5	1.95	2.21					C2	7.8	0.3	0.4	4.6	0.7
	Zone 1B PHBH20 (1-1.45)	13.4	41	17	24	31	32	21	16	2.5	2.5	1.84	1.97	2.00	1.65	21.5	1.7	C2					
	Zone 7 PHBH15 (0.5-1.5)	20 CBR								2.5	4			2.03	1.71	18.5							
	Bagdad St Area PHBH22 and PHTP23 (0.3-1.45)	18.9								3.5	2.5	1.77	2.10	1.94	1.6	21	4.2	C2					

Field moisture Content, Initial Dry Density and Initial wet Density taken from Shrink-Swell Test results unless stated otherwise.
MDD: Maximum Dry Density (t/m^3)
OMC: Optimum Moisture Content (%)

Note 9: CBR samples compacted to 98%Standard Maximum Dry Density and Standard Optimum Moisture Content. CBR Surcharge of 4.5kg applied. Test carried out after 4 days soaking.

8 DISCUSSIONS AND RECOMMENDATIONS

8.1 Earthworks

8.1.1 Fill Treatment

With the exception of the Bagdad Street Area, areas in the south west corner of Zone 3 and areas of lower elevation in the western areas of Zones 2 and 3, the geotechnical investigation indicates that the site is underlain by fill. As there are no records confirming that the fill has been compacted in accordance with an engineering specification the fill should be classified as uncontrolled.

Topsoil and root affected material should be stripped for possible re-use as landscaping fill, subject to horticultural, contamination and environmental assessment.

Unit 1A Fill contains material such as soft/loose or oversize material, roots, concrete, glass, brick, asphalt, tiles, domestic refuse and other putrescible material. Such material is unsuitable for re-use as engineered fill or for in-place ground improvement. Use of this material may be limited and there may be a need to sort and dispose of unsuitable material. Where used for landscaping, this material should be subject to geotechnical, horticultural, contamination and environmental assessment/remediation.

Unit 1B Fill (Crushed Shale/Sandstone) is unlikely to contain significant amounts of unsuitable material and is generally stiff to very stiff. The results of a limited number of laboratory tests indicates that the insitu material has a density of between -7% and +4% of Standard Maximum Dry Density and the samples tested are about 3% dry of Optimum Moisture Content. The results of the Downhole Compensated Gamma Density Logging indicate that the Unit 1B material is relatively dense with typical insitu densities of more than 2 tonnes/m^3. However, density variations of between +32% and -18% were recorded by the gamma density logging and the assessment of the location in plan and elevation of lower density zones would be difficult without extensive additional investigation. A zone of typically less dense material (ranging between 1.77 and 2.22 tonnes/m^3) was encountered in a number of boreholes near the interface between the Unit 1B material and Unit 2 Residual Soil. Hence, although the Unit 1B material is assessed to be reasonably dense for fill placed without known engineering control, it cannot be considered to be “Controlled Fill” and may not provide uniform support to pavements, floor slabs or footings unless it is stripped and recompacted, replaced or treated in-place.

Subject to the results of site specific trials, in-place ground treatment of Unit 1B Fill may be possible using high energy impact compaction such as an impact roller. The aim of the treatment would be to improve the density of poorly compacted layers and pockets and form a raft with relatively uniform properties. The depth to which improvement can be achieved will depend on site conditions. We recommend that a compacted layer of at least 98% Standard Maximum Dry Density Ratio to a depth not less than 1.5m should be the objective of an in-place treatment program. Fill within pavement subgrade level should be compacted to a minimum of 100% Standard Maximum Dry Density Ratio.

Such an approach would result in some risk remaining of variable settlement of the uncontrolled fill remaining beneath the treated zone. This risk should be considered in the design of services, pavements and footings. The risk of settlement could be further reduced (but not totally negated) by unloading (removing) at least 1m of the fill material prior to ground treatment. Careful consideration should be given to avoiding non-uniform loading of the fill during site preparation (e.g. location of stockpiles in areas where structures susceptible to differential settlement are not to be constructed).

Such care is required to reduce the risk of differential settlement over and between loaded areas and areas not loaded.

To classify sites as other than ‘P’, based on the system in AS2870-1996, the fill should be placed under engineering control.   Total excavation and recompaction of the fill may not be economical where there is a significant thickness of fill. Partial treatment of the fill depth may allow foundations equivalent to the standard footing designs of AS2870-1996 to be adopted. The site would however remain classified ‘P’ and engineering designs would be required.

The suitability of impact rolling to form the required compacted later should be assessed by site trials and stringent Construction Quality Assurance should be implemented during the ground treatments works. This may include but should not be limited to, continual impact response assessment and continuous induced settlement monitoring. Pockets of soft material, unsuitable material or material not adequately responding to ground treatment should be removed and replaced. Verification testing such as Cone Penetrometer Tests (DCP/CPT), density testing, insitu CBR testing, Falling Weight Deflectometer and geophysical tests should be considered to demonstrate uniform compaction.

It is recommended that preliminary ground improvement trials are carried out as soon as possible to assess suitability of this method of ground improvements as requirements for other types of ground improvement will have significant cost implications to the project.

It should be noted that the Unit 1B Fill material in the north of Zone 5 and south of Zone 6 does not appear to be as consistent as other areas of the site and hence this area may require more work to achieve acceptable ground conditions.

Unit 1B material or imported fill such as crushed shale or sandstone, may be used in areas where placement of fill is required to achieve project design grade levels. The fill should be placed under a Level 1 observation and testing regime as defined in AS3798 2007 “Guidelines on Earthworks for Commercial or Residential Developments”.

On site verification of fill materials for re-use as structural fill or suitability for in-place ground treatment should be assessed by a suitably qualified geotechnical engineer following site grade preparation and prior to commencement of ground improvement.

8.1.2 Excavation Conditions

A hydraulic excavator and bucket or bulldozer blade should be adequate for excavation of Unit 1A, Unit 1B material, Residual Soil (Unit 2) and highly weathered shale (Unit 3A). Below the upper weathered rock, higher strength rock is likely to be encountered. The excavations in higher strength rock may require considerably more effort, such as the use of impact hammers or ripping.

It should be noted that trafficability of Unit 1A and 1B material and Unit 2 Residual Soil is likely to be difficult during and immediately following wet weather.

Contractors should be required to examine the engineering logs and core photographs to make their own assessment of excavation plant and production rates.

The recommendations in this report on the re-use of existing fill is from a geotechnical perspective and does not consider heritage, horticultural, environmental or contamination constraints.

8.2 Temporary and Permanent Cut Batter Slopes

Recommendations for temporary and permanent unsupported cuts are presented in Table 11, below.

TABLE 11: RECOMMENDATIONS FOR TEMPORARY AND PERMANENT CUTS

Material	Temporary Cuts	Permanent Cuts
Unit 1A – Fill	1.5H:1V	3H:1V
Unit 1B – Crushed Shale/ Sandstone	1.5H:1V	3H:1V
Unit 2 – Residual Soil	1.5H:1V	2H:1V
Unit 3A - Weathered Rock	1H:1V	1.5H:1V

Surcharge loads should be kept well clear of the crest of temporary cuts.

8.3 Existing Fill Slopes

8.3.1 Slope No. 1: Western Slope

The crest of Slope No. 1 passes from the south west corner of the site running parallel with Cooper Road and is set back between 10m and 50m (typically about 25m) from the site boundary with the properties along Cooper Road (refer to Figure 1). Slope No. 1 generally decreases in height towards Zone 1 and curves around (passing to the east of) the existing structures in Zones 1B and 1C. Slope No. 1 is between 9m and 12m in height where it runs parallel with Cooper Street and between 2m and 5m in the north of the site near to Zone 1A and 1B. The slope typically possesses a gradient of between 3.5H:1V and 5.5H:1V, with local gradients up to 2.5H:1V.

At the time of this investigation, the slope was vegetated with mature trees and dense undergrowth. Treatment works such as localised re-grading where slope gradients are steeper than 3H:1V and treatment of areas subject to erosion using geotextile/geogrids or gabion baskets/mattresses or similar is required. Assuming a slope gradient no steeper than 3H:1V, an easement of at least 1m is recommended at the crest of the slope in which no surcharge load should be applied.

If a slope of steeper gradient is required or surcharge placed within the easement, an engineered retaining solution will be necessary such as crib walls, gabion walls, soil nail walls, modular block retaining walls, reinforced earth walls, mass concrete walls or piled retaining structures. Detailed design of retaining structures will be required following finalisation of the development scheme. Adequate drainage measures should be provided to all slopes and all retaining structures on the site.

8.3.2 Slope No.2: Southern Slope

Slope No. 2 forms the north western and northern boundary of Zone 4 and runs parallel with Reservoir No.2 (refer to Figure 1). Slope No. 2 is typically between 6m and 8m in height with an increasing gradient from west to east from approximately 6H:1V in the north-west to 1.2H:1V in the south eastern corner of Zone 4.

Localised failure of this slope occurred in 1989 in the vicinity of borehole YEDBH21 (Figures 2 and 7), it should be noted that the slope has a gradient steeper than 2H:1V in this area. Borehole PHBH21 drilled near the crest of the slope to the west of the 36b Brunker Road facility encountered fill containing domestic refuse to over 2m depth. The unsuitable material should be removed and this slope re-graded to no steeper than 3H:1V. Assuming a slope gradient no steeper than 3H:1V, an easement of at least 1m is recommended to the rear of the slope in which no surcharge load should be applied. If surcharge loads encroach to within 1m of the slope, a specific slope stability analysis should be carried out.

If a slope of steeper gradient is required or surcharge to be placed within the easements, an engineered slope solution and/or retaining structure will be necessary. This may include slope stabilisation by shotcrete and soil nails or construction of modular block retaining walls (Rocla Masbloc or similar), mass concrete retaining walls or a pile wall solution. Detailed design of retaining structures will be required following finalisation of the development scheme. Adequate drainage measures should be provided to all slopes and all retaining structures.

Assessment of adverse impacts of the proposed development on Reservoir No. 2 and associated facilities should be carried out and the proposed development approved by Sydney Water Corporation.

8.3.3 Slope No. 3: Eastern Slope

Slope No. 3 forms the southern boundary of Zone 5 along Brunker Road and eastern boundary of Zones 5 and 6 along Graf Avenue (refer to Figure 1). Along the southern and eastern boundaries of Zone 5, the slope is between 4m and 6m in height with a typical gradient of about 2H:1V. Along the eastern boundary of Zone 6, parallel to Graf Avenue, the slope height increases to over 9.5m with a typically gradient of 2H:1V.

This slope has a history of instability and water seepage problems and during the geotechnical fieldwork, the slope was observed to show signs of distress. Tension cracks and localised slump failures were observed. Water seepage from the face of the slope was also visible following a period of heavy rainfall. The level area between the toe of the slope and the boundary fence line along Graf Avenue was observed to be waterlogged and boggy following the wet weather. It was also observed during the fieldwork that drainage installation works and slope maintainance works had recently been carried out on the slope in the area to the west of Potts Park Greyhound Track.

It is recommended that the slope is re-graded to a gradient no steeper than 4.5H:1V. Assuming a slope gradient no steeper than 4.5H:1V, an easement of at least 1m is recommended to the rear of the slope in which no surcharge load should be applied.

If a slope of steeper gradient than 4.5H:1V or surcharge loading closer than 1m to the slope crest is proposed or if the site is required to be developed up to the boundary along Graf Avenue, an engineered retaining wall solution will be necessary. Permanent retaining wall solutions could include reinforced earth walls, modular block retaining walls (Rocla Masbloc (or similar)) or mass concrete retaining walls. The soft material at the toe of the slope will require excavating to Unit 3A or 3B to allow construction of foundations for retaining wall.

It should be noted that Unit 1B material may not be suitable for use as Select Fill for construction of a Reinforced Earth Retaining Wall. Should Reinforced Earth Walls be the preferred option, allowances should be made for the import of material such as crushed sandstone until the suitability of the Unit 1B material is assessed to be unsuitable for use as Select Fill. The specific requirements of the Select Fill should be confirmed with the Reinforced Earth Wall designer/supplier.

Intermediate options between regarding the slope and construction of large permanent retaining walls could include, terracing of the slope with associated retention by criblock walls, gabion walls, concrete retaining walls or soil nail walls. Detailed design of retaining structures will be required following finalisation of the development scheme. Adequate drainage measures should be provided to all slopes and all retaining structures on the site.

8.4 Pavement Subgrade

The Unit 1A material is likely to be unsuitable as a pavement subgrade. The Unit 1B Fill has a relatively low CBR values of between 2% and 2.5% and a swell after four days soaking of between 2.5% and 4% indicating a highly expansive subgrade material.

The Unit 2 Residual Soil has relatively low CBR values, typically 2.5% (3.5% for the Bagdad Street Area) and a swell after four days soaking of between 2.5% and 4% indicating a highly expansive subgrade material.

Both the Unit 1B Fill and Unit 2 Residual Soil is considered a poor bearing stratum for pavements without modification. Some options for subgrade improvement or replacement are as follows.

Lime Modification

Subgrade improvement could be by lime modification to a minimum depth of 300mm. The addition of 4% hydrated lime (percentage dry weight of soil) by specialist pulverising, mixing and recompacting to a at least 100% Standard Maximum Dry Density, should raise the insitu CBR value of the subgrade and a design value of 3% could be adopted.

Coffey’s previous experience of materials similar to Crushed Shale and Residual Soil indicate that CBR values greater than this may be achievable with the addition of greater proportion of lime typically up to about 8%. The addition of 8% hydrated lime (percentage dry weight of soil) by specialist pulverising, mixing and recompacting to at least 100% Standard Maximum Dry Density, should raise the insitu CBR value of the subgrade and a design value of 5% could be adopted

The effectiveness of the lime modification is dependant on many factors such as construction method, construction plant used, the degree of original soil pulverisation, the original moisture content of the soil, the type and properties of the lime and the mineralogy of the clay in the soil. The effectiveness of lime stabilisation and the optimum percentage and necessary depth of modification should be assessed by laboratory testing and field trials.

Subgrade Replacement

Subgrade replacement could be carried out by placing well graded, durable, non-expansive granular material compacted to at least 100% Standard Maximum Dry Density Ratio at a moisture content within ±2% of Standard Optimum Moisture Content. The fill should be placed in maximum 200mm compacted layer thicknesses.

A well graded crushed sandstone of 60mm maximum size and compacted to the aforementioned density ratio and moisture content should provide a remoulded CBR of at least 20%. Assuming a remoulded CBR of 20%, a layer at least 0.3m thick should raise the insitu CBR value of the subgrade to at least 3%. Pavements should however be designed on the basis of the CBR value of the actual replacement material and the effectiveness of the subgrade replacement should be checked by laboratory testing and field trials.

All pavements should be provided with long term surface and subsurface drainage to protect the subgrade from moisture ingress.

8.5 Foundations

8.5.1 Residential Development – Zone 1 to Zone 4 and Bagdad Street Area

For the design of residential structures and structures with areas and loads consistent with residential structures, classifications of individual lots should be made with reference to AS2870-1996 “Residential Slabs and Footings”.

With the exception of the Bagdad Street Area and localised areas of site, the site is underlain by significant thicknesses of uncontrolled fill materials. In accordance with AS2870-1996, the classification of these lots should be ‘P’. If ground treatment and verification is carried out to form a compacted soil raft over uncontrolled fill, engineer designed raft slabs with similar stiffness to that of the standard designs in AS2870-1996 may be able to be adopted.

The shrink swell index test results for the Unit 1B material indicates a moderate shrink swell potential. Raft slabs may be able to be adopted with stiffness equivalent to the standard designs for ‘H’ lot classification. It should be noted that this is a preliminary assessment without consultation with a structural engineer. More detailed design and lot specific assessment should be carried out when design levels are known and after earthworks are complete.

The field descriptions and index testing indicate that the Unit 2 Residual Soil is typically medium to high plasticity and tests on the Unit 2 Residual Soil in the Bagdad Street Area indicates a high shrink swell potential. In site areas where the residential footings will bear directly onto Unit 2 Residual Soil, such as is likely in the Bagdad Street Area or areas where it may be cost effective to remove the Unit 1 Fill materials, the lot classification may be ‘H’ or ‘E’. This classification is preliminary and based on a limited number of tests.

Further lot specific assessment should be carried out once the earthworks are completed. AS2870 recommends that this is preferably undertaken towards the end of subdivision and road construction, but before house construction starts and after the lots have been pegged.

8.5.2 Light Industrial Development (The Eastern Precinct)

8.5.2.1 Pile Foundations

Due to the thickness of Unit 1B Fill in Zones 5 and 6 and the elevated areas of Zone 7, structures with loads that are not consistent with residential structures and/or structures that are sensitive or may contain plant and equipment sensitive to movements will require footings taken into the Unit 3A or 3B bedrock. Where the depth to rock exceeds 1.5m it may be necessary to adopt pile foundations unless footings excavations are shored or battered.

Open bored piles or continuous flight auger piles could be adopted. We would expect that with appropriate capacity piling rigs, piles should be able to penetrate to Unit 3A Class IV Shale or Unit 3B Class III Shale. An experienced geotechnical engineer should observe boring of the piles in order to assess the rock levels and to confirm that the rock is suitable for the adopted design parameters.

Allowable design parameters for bored piles are provided in Table 12. The use of the recommended allowable bearing pressures would be expected to result in pile settlement of less than 1% of the pile diameter.

TABLE 12: FOUNDATION DESIGN PARAMETERS

Geotechnical Unit	Allowable Bearing Pressures (kPa)	Allowable Shaft Adhesion for Piles (kPa) [(a)]
Unit 3A – Class V Shale	700	50
Unit 3A – Class IV Shale	1000	75
Unit 3B – Class III Shale or better	3000	300

Notes:

(a) Shaft adhesion should only be assumed where piles have a minimum embedment of at least 3 pile diameters into the nominated stratum and a rough socket (at least grooves of depth 1mm to 4mm and width greater than 5mm spacing of 50mm to 200mm). The socket should be cleaned and roughened by a suitable scraper such as a tooth, orientated perpendicular to the auger shaft. Shaft adhesion should be ignored for pad or strip footings.

Open bored piles may require temporary liners through the Fill and Residual Soil (Units 1 and 2) or if groundwater seepage occurs. Piles should be cleaned, dewatered and concreted without delay to prevent softening of the pile base.

For uplift capacity, the shaft adhesion values given in Table 12 should be multiplied by 0.6. In addition to shaft adhesion, the uplift capacity should be checked for a cone pullout failure mode assuming a cone angle of 70° considering the submerged weight of the soil or rock and adopting a factor of safety of 1.0 against pullout.

Isolated boulders were encountered during the investigation in PHBH12 and five other exploratory holes carried out during previous investigations in Zones 5 and 6. It is recommended that a contingency is allowed for mitigation measures such as localised excavation of boulder and replacement/recompacting of fill, predrilling pile locations where boulder are encountered or relocation/addition of pile positions and redesign of pile cap. The presence of boulders in the fill could impact on the suitability of driven piles.

8.5.2.2 Pad and Strip Footings

Pad and strip footing should be founded in the Unit 3A Weathered Bedrock. Where the depth to Unit 3A Weathered Bedrock is generally less than 1.5m, or where footings excavations can be practicably shored or battered to expose this unit (such as in the lower lying areas of Zone 7), pad and strip footings may be feasible.

The Weathered Bedrock Unit 3A is typically Class V Shale, where the rock is extremely weathered and Class IV Shale where the rock is less weathered and contains fewer clay seams. An allowable bearing pressure of 700 kPa can be adopted for the footings with a minimum embedment of 0.3m into Class V Shale. An allowable bearing pressure of 1000kPa could be adopted if the footing excavations are taken through the extremely weathered shale into Class IV Shale.

It should be noted that the Unit 3A rock may soften in footing excavations. The footing should be dewatered, cleaned and concreted within 12 hours of excavation or a blinding layer of concrete should be placed to protect the base. An experienced geotechnical engineer should visually inspect the footing excavations prior to blinding to confirm that the founding material is suitable for the adopted design parameters.

8.5.2.3 Slab On-Ground Construction

Slab On-ground construction may occur on Unit 2 Residual Soil and following successful ground treatment and verification of Unit 1B Fill.

The potential for uplift pressures and ground movements acting on the ground floor slab of the building due to shrinkage and swelling of the treated Unit 1B Fill and Unit 2 Residual Soil should be considered. The effects of the ground treatment work on the shrink and swell characteristics of the subgrade should be assessed on a structure by structure basis following ground treatment.

The laboratory test results indicate that the Unit 2 Residual Soil has a high shrink swell potential. If, in addition, significant shrink swell potential remains for Unit 1B following ground treatment, then moisture conditioning through tyning and a sub-base of good quality non-expansive crushed rock should be placed beneath ground slabs.

8.6 Preliminary Soil Salinity and Sodicity Assessment

Soil salinity represents the amount of salt in the soil. An excess amount of salt is harmful for plant growth, and it affects the durability of materials such as concrete, steel and bitumen. Saline reactive soils are often more prone to shrink-swell movements if the salt can be leached out, for example by garden watering, which will exacerbate the swell movements in particular.

Sodicity indicates the level of exchangeable sodium in soil and sodic soils are characterised by slow rates of water infiltration (from rain or irrigation), poor water and nutrient transport within the soil, restricted vegetation growth and severe surface crusting. When wet, sodic soils are boggy and soft. If slightly sodic material is exposed or brought close to the surface by the development, it may prevent or retard the establishment of vegetation and where excess water enters the site, the slightly sodic material may prevented or retard water from moving vertically through the soil profile. This may result in soil erosion issues and/or problematic drainage conditions.

The results of the field salinity screening and laboratory testing indicate that the Unit 1B Fill and Unit 2 Residual Soil is typically Non-saline and Non-sodic. Slightly saline conditions were assessed in the near surface Unit 1A material in boreholes PHBH1 (<1m) (Zone 1A) and PHBH13 (<0.5m) (Zone 6).

Based on the results of the field screening and laboratory testing, salinity is not presently considered an issue likely to significantly impact on the Potts Hill development. Development of the site can however change the water balance and flow of surface and groundwater and cumulative impacts can lead to increased soil and groundwater salinity and sodicity.

Good general practices for development in Western Sydney should be adopted. Good practice includes but is not limited to:

- Reduce exposure and disturbance of the soil e.g. minimising cutting and filling.
- Reduce the infiltration of stormwater and provide good surface and sub-surface drainage. Establish adequate drainage measures in poorly drained areas.
- Reduce water input and maintain natural water balance.
- Reduce the infiltration of stormwater and manage excess water on the site.
- Retain existing vegetation and planting of suitable vegetation in areas susceptible to erosion.
- Cover, vegetate or use gypsum or lime in areas of exposed soil and stockpiles to reduce the risk of soil dispersion and erosion.
- Provision of properly installed high impact damp proof membranes under slabs and foundations, typically underlain by at least 50mm of sand to allow free drainage.

The size of the proposed Potts Hill development is such that a Level Three Salinity Management Response is likely to be required. The conditions of development may stipulate the requirement for a salinity management plan. The readers attention is drawn to the following publication “Western Sydney Salinity Code of Practice” (January 2004, Western Sydney Regional Organisation of Councils (WSROC)).

8.7 Preliminary Assessment of Soil Dispersion Potential

Dispersibility (erodibility of soil material) is a measure of soil dispersion when immersed in water. The characteristics of dispersive soils are high erosion in rainfall events. The results of the laboratory testing indicate that the Unit 1B Fill and Unit 2 Residual Soil samples tested, typically have an Emerson Classification of Class 2, indicating a moderately to slightly dispersive soil.

Dispersive soils have a high probability of forming pipe or tunnel type erosion if water is able to make its way through the surface and move laterally through the profile, removing material by water erosion. Dispersive soils can quickly erode and develop rills and gullies and undermine the soil support of structures.

Soil loss from surface erosion of dispersive soils can be high in sloping areas and material stockpiles. Application of ameliorants such as gypsum may be useful for treating stockpiles, particularly when low volumes of stable material are available for capping.

Good drainage is necessary to prevent erosion along concrete-soil interfaces such as to the rear of retaining walls and other buried concrete structures in contact with the soil.

Adequate compaction of material and good moisture control is necessary as is the construction of good surface and subsurface drainage to reduce the potential for soil erosion.

8.8 Preliminary Assessment of Soil Aggressivity

Table 10 summarises the results of the laboratory testing. The results of the soil aggressivity parameters determined indicate that the tested Unit 1B Fill and Unit 2 Residual Soil are assessed as Non-Aggressive to buried concrete and steel as determined with reference to Australian Standard AS 2159-1995 Piling –Design and Installation. It is advised that further testing be carried out following completion of earthworks and prior to commencement of construction to verify this statement.

8.9 Groundwater and Drainage Control

The need for good drainage both stormwater, surface and subsurface is an important consideration for the Potts Hill Development. Well designed and constructed temporary and permanent drainage is essential to maintain the stability of constructed slopes and retaining walls, reduce shrink/swell movement and potential for adverse effects on foundations and pavements, reduce soil erosion and mitigate against increased site sodicity and salinity. It is also necessary to decrease the potential for ground movement of the existing fill by softening of the Crushed Shale on wetting.

It was concluded in the 1978 investigation into seepage along Graf Avenue that “the [groundwater levels monitored] correlation was rather imprecise, but nethertheless sufficient to conclude that the stored water in Potts Hill Reservoir No. 2 does appear to be related to the seepage in Graf Avenue” (Report 6038499 GT/JC dated 31 May 1978). A number of exploratory holes in the north of Zone 5 and south of Zone 6 encountered wet soil conditions towards the base of the Unit 1B material. It is therefore possible that leakage of Reservoir No.2 may be occurring and affecting the ground in this area.

It is recommended that trial excavations are carried out adjacent to the Reservoir to investigate possible reservoir leakage. If the potential for widespread leakage is assessed, it may be necessary to construct a vertical cut off drain along the reservoir/site perimeter to remove reservoir leakage water.

8.10 Development Impact on Existing Facilities

The proposed development has the potential to impact on important exiting Sydney Water facilities that will remain during and after development of the site. Establishment of “No Development Easements” may be necessary adjacent to sensitive structures and facilities. The potential for destabilisation of reservoir walls, tunnels, telemetry systems and other associated structures should be investigated and potential impacts of the proposed development assessed and approved by Sydney Water Corporation. Flooding risk at the Potts Hill Development should also be addressed.

Development along the northern boundary of Zone 1A and in the Bagdad Street Area is also likely to require the involvement/approval of Railcorp.

Pre-condition dilapidation surveys of potentially affected buildings and structures along Copper Road is also advised.

9 LIMITATIONS

The discussions and recommendations provided assume no restriction from Sydney Water or other involved parties. An assessment of impacts of the proposed development on Sydney Water and Railcorp facilities and surrounding properties should be carried out.

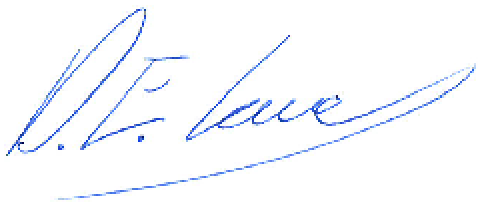
Environmental assessment is beyond our commissioned scope of works, the recommendations assume that materials are fit for purpose in terms of contamination and environmental considerations. Contamination, environmental, heritage and ecological issues may override the geotechnical considerations and these should be fully assessed.

Coffey have reviewed the existing exploratory hole data and the information has been used where the information appears sensible and credible. Coffey cannot however be responsible for the accuracy, quality and content of work carried out by others.

The findings within this report are the result of discrete/specific investigations methodologies used in accordance with normal practices and standards. Subsurface conditions can change over relatively short distances and the subsurface conditions revealed at the test locations may not be representative of subsurface conditions across the site. We recommend that a geotechnical engineer be engaged during construction to confirm the subsurface conditions are consistent with design assumptions.

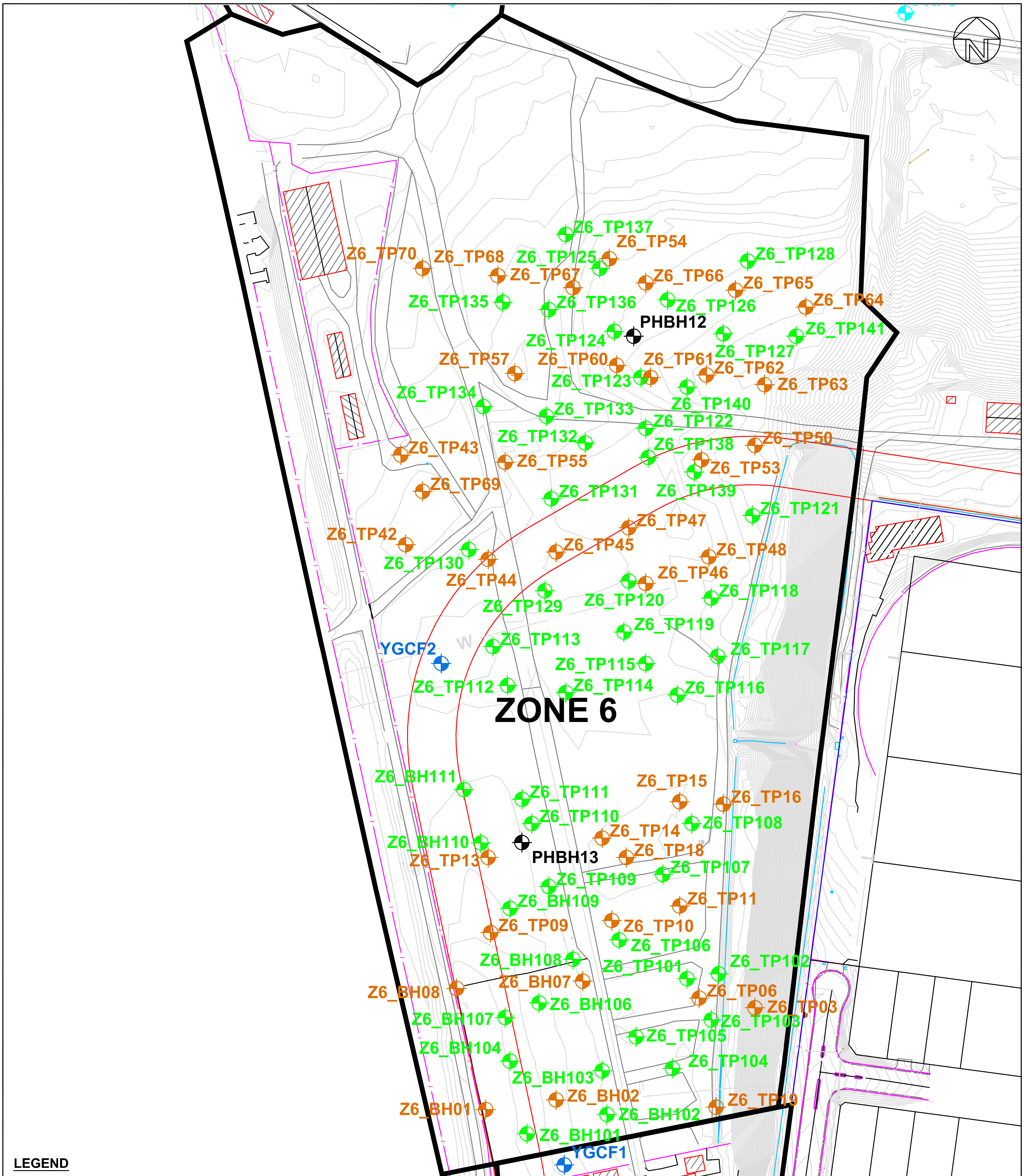
The reader's attention is drawn to the attached document entitled 'Important Information about your Coffey Report', which presents additional information on the uses and limitations of this report.

For and on behalf of Coffey Geotechnics Pty Ltd



DUNCAN LOWE

Associate



LEGEND

YGCF SERIES EXPLORATORY HOLES - EXTRACTED FROM YAGOONA GANTRY CRANE FOUNDATION INVESTIGATION (1971)

PHBH CURRENT BOREHOLE LOCATIONS

EXTRACTED FROM URS AUSTRALIA PTY ENVIRONMENTAL ASSESSMENT EXPLORATORY HOLES (2004)

EXTRACTED FROM COFFEY GEOSCIENCES STAGE II ADDITIONAL ENVIRONMENTAL INVESTIGATION (2006)

* GROUNDWATER MONITORING/ENVIRONMENTAL WELL INSTALLED

ZONE BOUNDARIES

drawn	MP/LT
approved	DL
date	1/02/08
scale	1:1500
original size	A3

coffey geotechnics
SPECIALISTS MANAGING THE EARTH

client:	LANDCOM
project:	POTTS HILL GEOTECHNICAL CONSULTANCY POTTS HILL, NSW
title:	LOCATION OF SELECTED EXPLORATORY HOLES FOR ZONE 6 FROM PREVIOUS INVESTIGATIONS
project no:	GEOTLCOV23274AA
figure no:	FIGURE 9