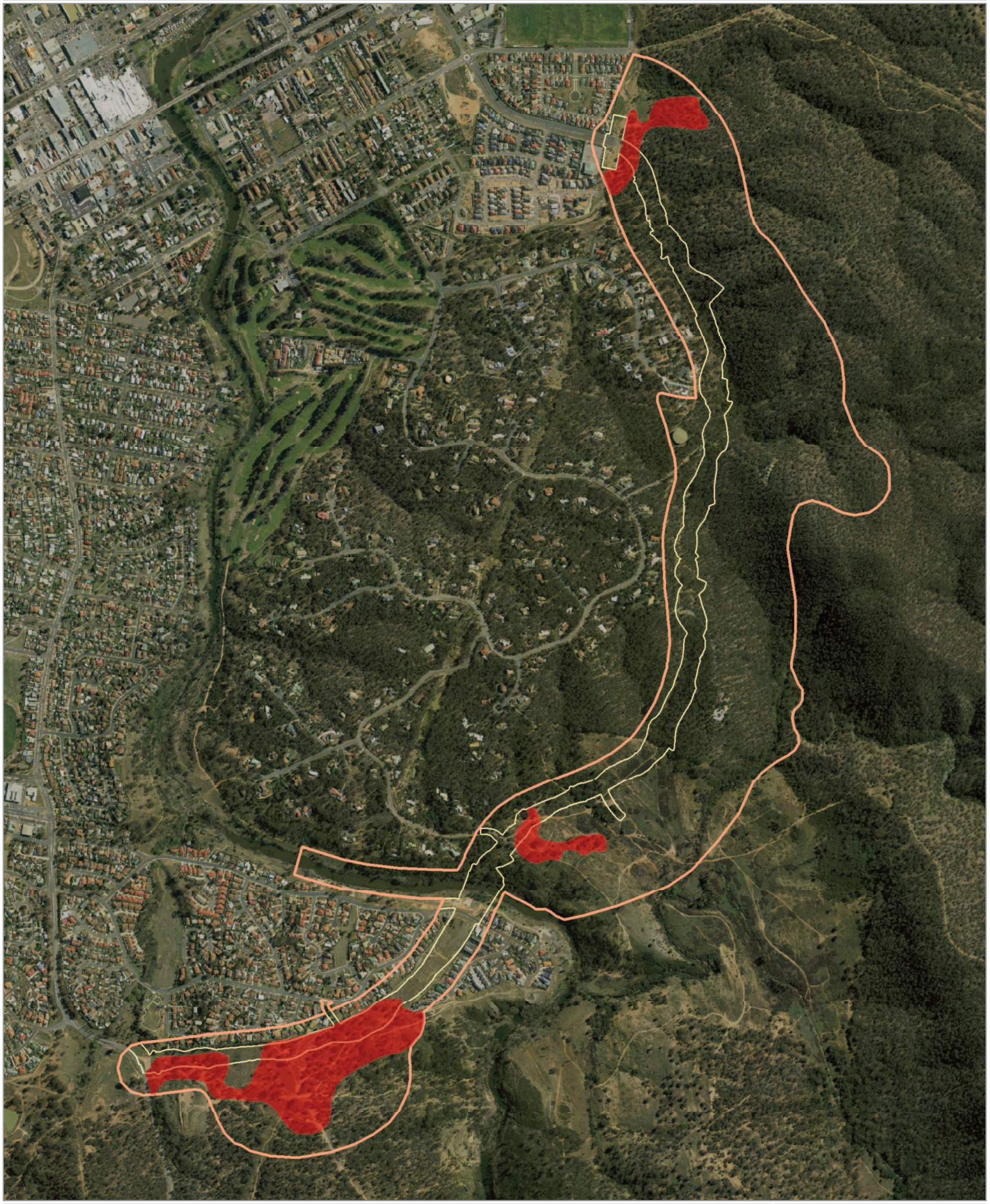
Ellerton Drive extension - Areas of more abundant Mistletoe within the study area



Study area
Subject site
Mistletoe (Amyema pendula) more abundant

Notes:
- Field data collected by nghenvironmental field staff (October 2012)
- Aerial imagery and development envelope provided by QCC

0 125 250 Meters

A3 @ 1:11000
Ref: 4733 - 8
Author: DM

ngh environmental

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Ellerton Drive extension - Subject species habitat - Diamond Firetail and Scarlet and Hooded Robin

Study area

Fauna habitat type
- Disturbed
- Dry grass forest
- Dry shrub forest
- Grassland
- Riverine
- Shrubland
- Woodland

Subject species habitat
- Diamond Firetail
- Scarlet and Hooded Robin

Notes:
- Field data collected by nghenvironmental field staff (October 2012)
- Development envelope and aerial imagery provided by QCC

0 125 250 Meters

A3 @ 1:11000
Ref: 4733 - 9-1
Author: DM

ngh environmental

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Ellerton Drive extension - Subject species habitat - Pink-tailed Worm Lizard, Golden Sun Moth and Rosenbergs Goanna

Study area

Fauna habitat type
- Disturbed
- Dry grass forest
- Dry shrub forest
- Grassland
- Riverine
- Shrubland
- Woodland

Subject species habitat
- Pink-tailed Worm Lizard and Golden Sun Moth
- Rosenbergs Goanna
- Rosenbergs Goanna (Important habitat)

Notes:
- Field data collected by nghenvironmental field staff (October 2012)
- Development envelope and aerial imagery provided by QCC

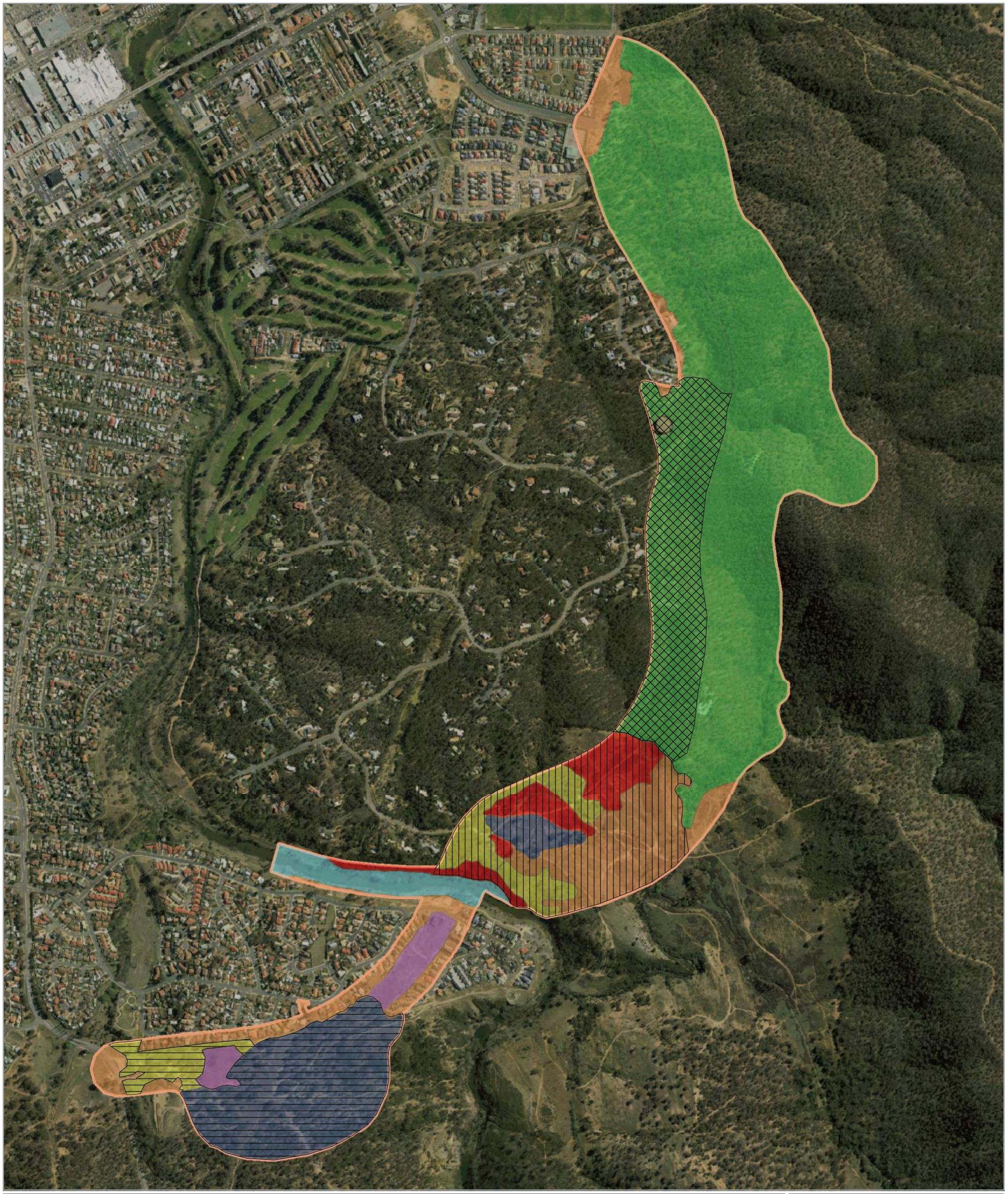
0 125 250 Meters

A3 @ 1:11000
Ref: 4733 - 9-2
Author: DM

ngh environmental

www.nghenvironmental.com.au

Ellerton Drive extension - Subject species habitat - Brown Treecreeper, Gang-gang Cockatoo and Speckled Warbler



Study area

Fauna habitat type
- Disturbed
- Dry grass forest
- Dry shrub forest
- Grassland
- Riverine
- Shrubland
- Woodland

Subject species habitat
- Brown Treecreeper
- Gang-gang Cockatoo (important habitat)
- Speckled Warbler (important habitat)

Gang-gang Cockatooe, Speckled Warbler, Koala Eastern False Pipistrelle and Eastern Bent-wing Bat could occur in all habitat types. However, habitat of specific importance for breeding has been highlighted for selected species)

Notes:
- Field data collected by nghenvironmental field staff (October 2012)
- Development envelope and aerial imagery provided by QCC

0 125 250 Meters

A3 @ 1:11000
Ref: 4733 - 9-3
Author: DM

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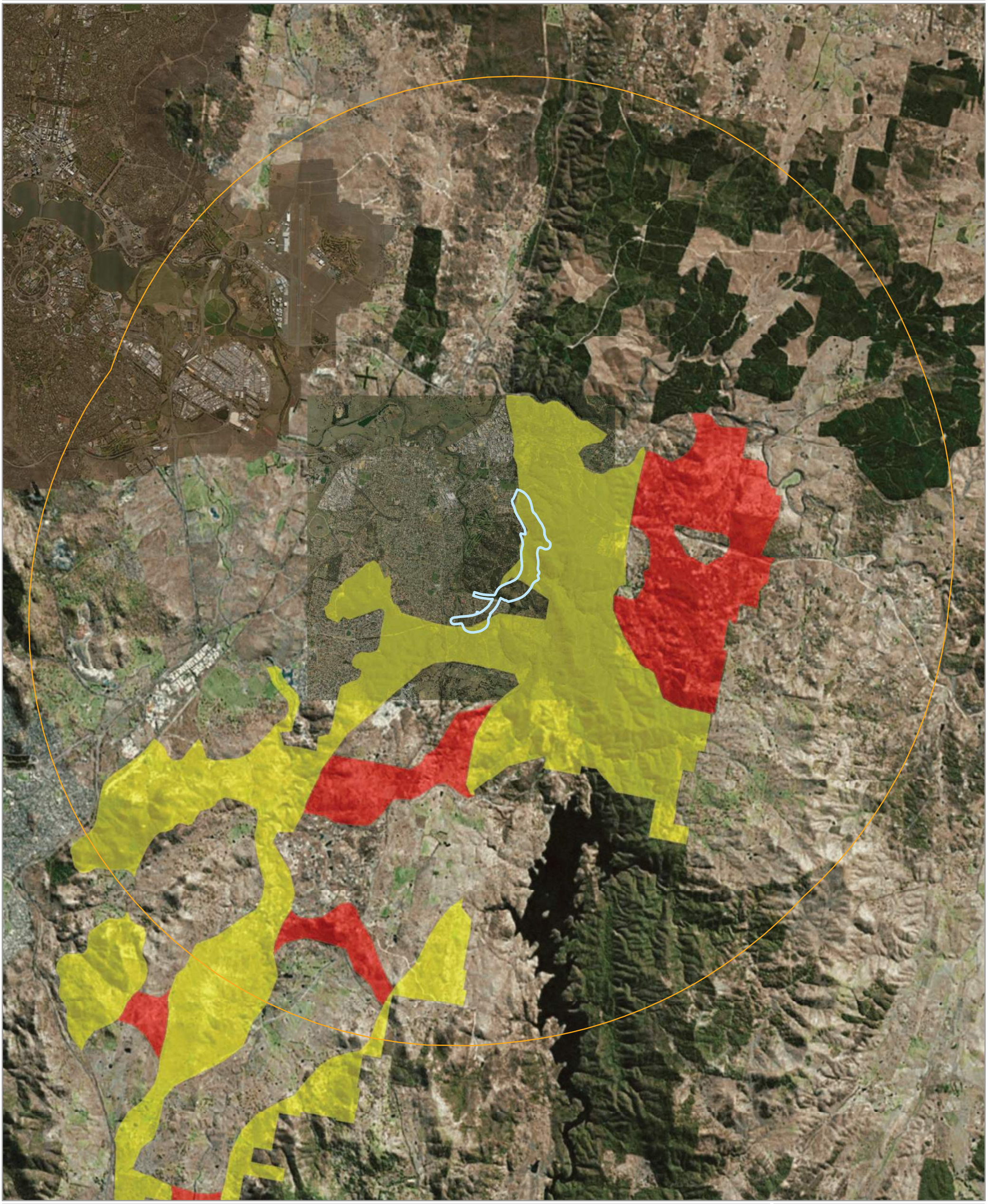
Ellerton Drive extension - Biolinks within the locality



Study area
10km Buffer
Biolinks
Local
Regional

Notes:
- Aerial imagery and study area layers provided by QCC
- Biolink layer provided by QCC
- Base map sourced from ESRI Online © 2013 Microsoft Corporation and its data suppliers

0 1 2 Kilometres

A3 @ 1:85000
Ref: 4733 - 11
Author: DM

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Ellerton Drive extension - Fauna habitat types

Study area
Subject site

Fauna habitat type
- Disturbed
- Dry grass forest
- Dry shrub forest
- Grassland
- Riverine
- Shrubland
- Woodland

Fauna habitat quality
- Good
- Moderate
- Poor

Notes:
- Field data collected by nghenvironmental field staff (October 2012)
- Development envelope and aerial imagery provided by QCC

0 125 250 Meters

A3 @ 1:11000
Ref: 4733 - 6
Author: DM

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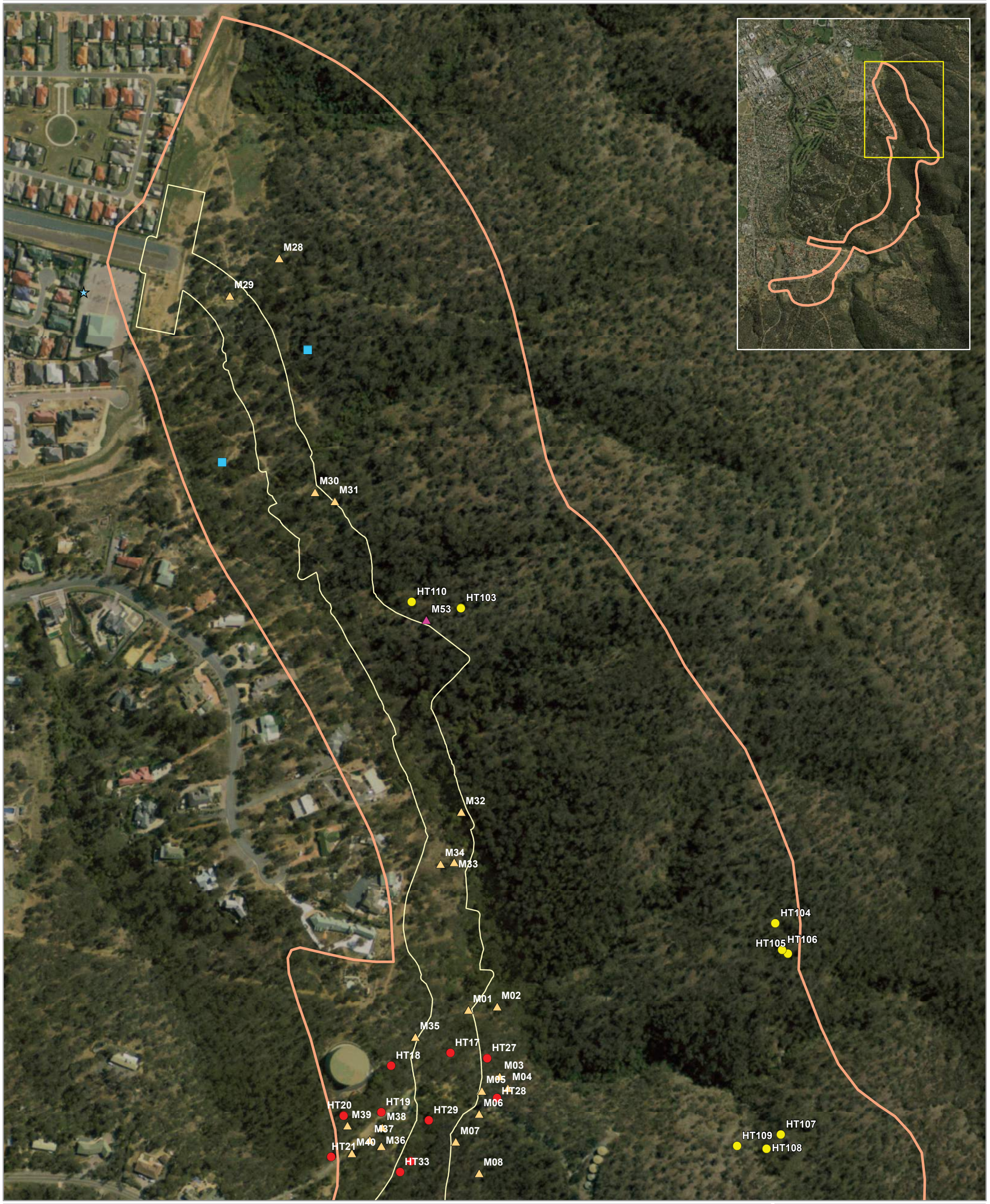
Ellerton Drive extension - Fauna survey results (north)



Study area
Subject site

Threatened fauna

2012 survey
- Gang-gang Cockatoo
- Scarlet Robin

2013 survey
- Gang-gang Cockatoo
- Speckled Warbler

Habitat features

2012 survey
- Hollow-bearing tree
- Termite mound

2013 survey
- Hollow-bearing tree
- Termite mound

0 50 100 Meters

A3 @ 1:4000
Ref: 4733 - 7
Author: DM

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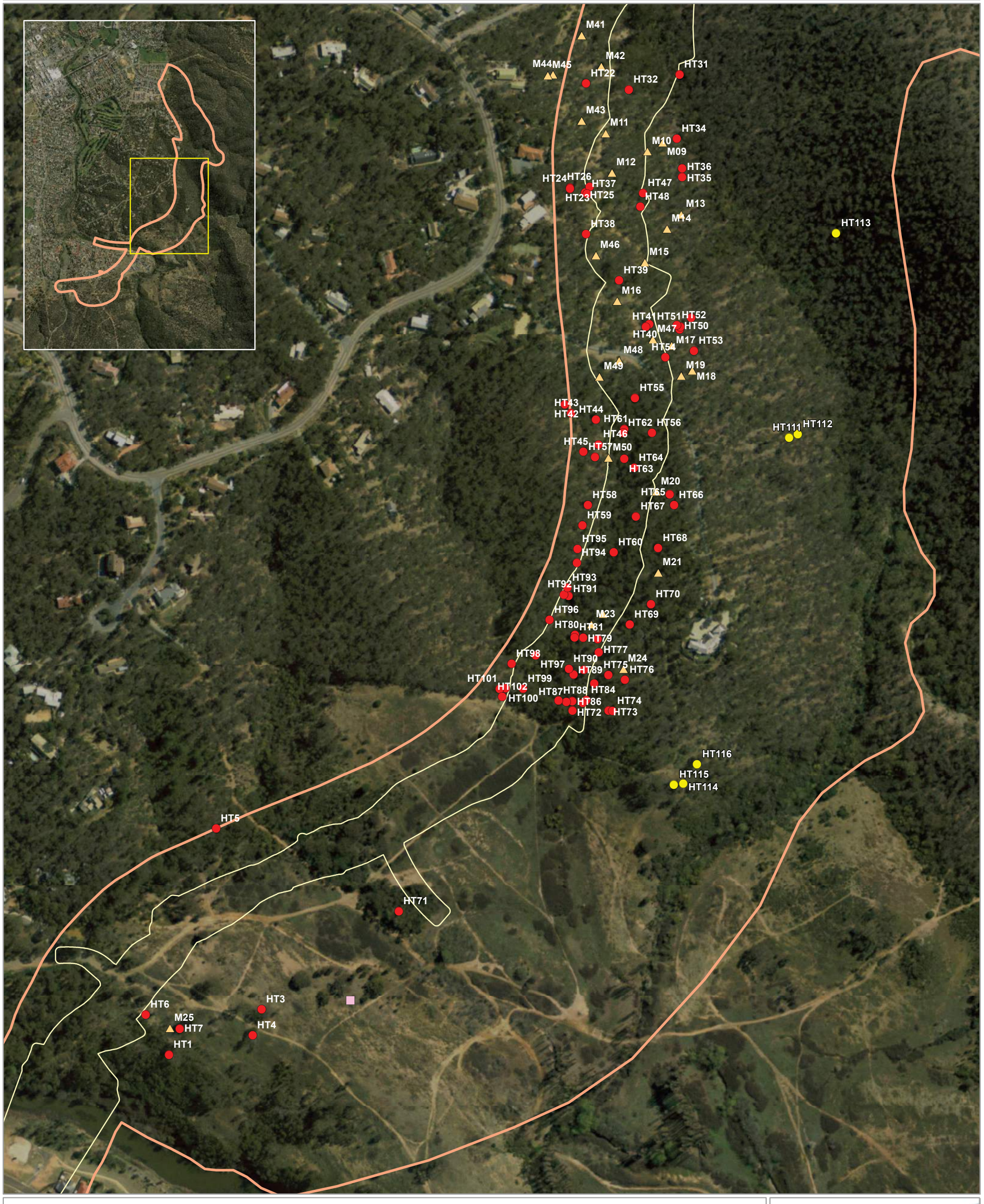
Ellerton Drive extension - Fauna survey results (central)



Study area
Subject site

Threatened fauna
2012 survey
Gang-gang Cockatoo
Scarlet Robin
2013 survey
Gang-gang Cockatoo
Speckled Warbler

Habitat features
2012 survey
Hollow-bearing tree
Termite mound
2013 survey
Hollow-bearing tree
Termite mound

0 50 100 Meters

A3 @ 1:4000
Ref: 4733 - 7
Author: DM

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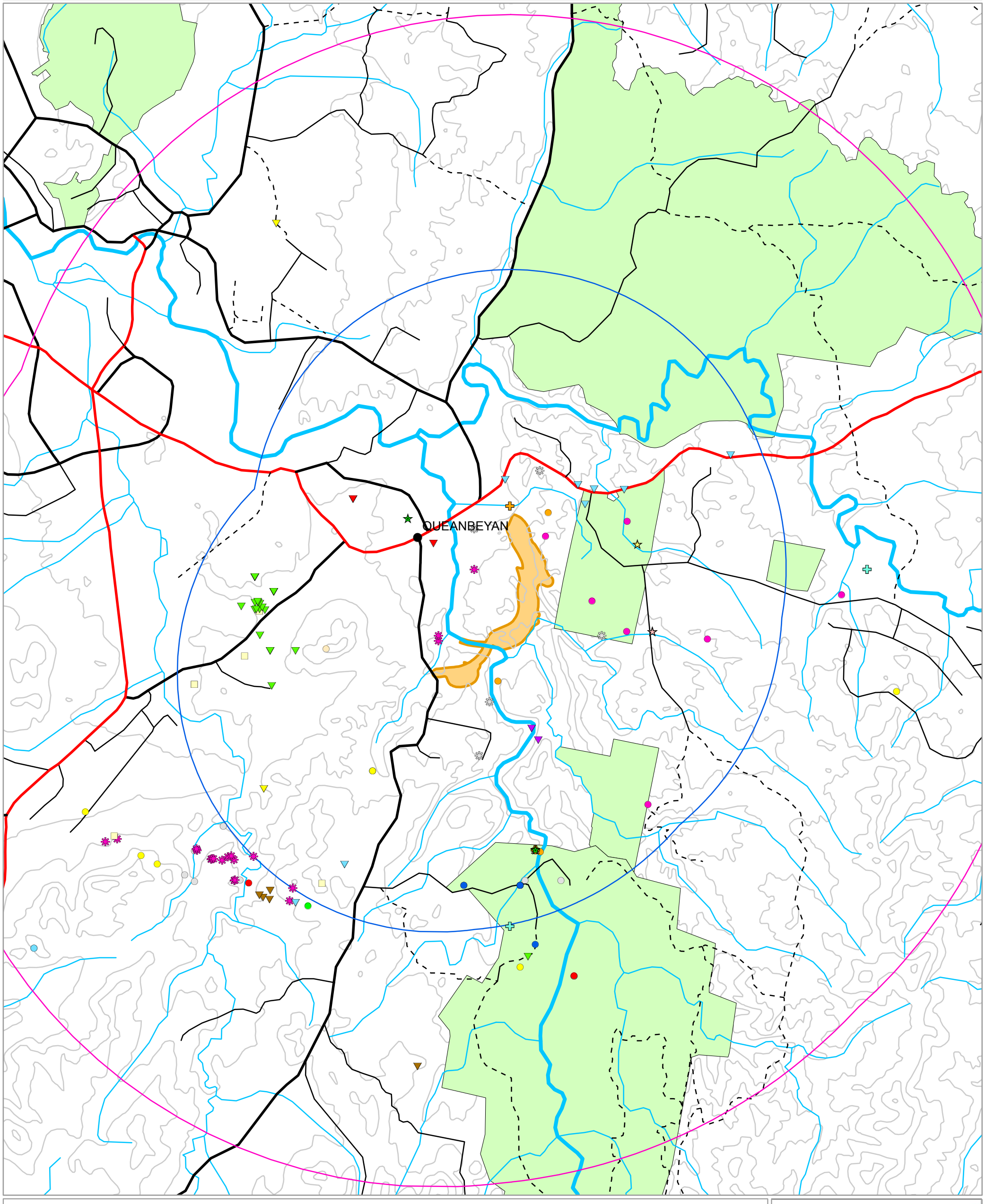
Ellerton Drive extension - Threatened species records within 10km of the study area



Study area
5km Buffer
10km Buffer
Reserves
Contours 50m

Water courses
Major watercourse
Minor watercourse

Roads
Principal Road
Secondary Road
Minor Road
Track

Threatened flora
Caladenia tessellata
Leucochrysum albicans var. tricolor
Pomaderris pallida
Rutidosis leptorrhynchoides
Swainsona recta
Swainsona sericea

Threatened fauna records
Golden Sun Moth
Green and Golden Bell Frog
Southern Bell Frog
Eastern Bentwing-bat
Eastern False Pipistrelle
Southern Myotis
Koala
Spotted-tailed Quoll
Pink-tailed Legless Lizard
Rosenberg's Goanna
Brown Treecreeper (eastern subspecies)
Diamond Firetail
Flame Robin
Gang-gang Cockatoo
Hooded Robin (south-eastern form)
Little Eagle
Scarlet Robin
Speckled Warbler
Spotted Harrier
Varied Sittella
White-fronted Chat

Notes:
- Threatened Species locations sourced from Bionet - NSW Wildlife Atlas 2012
- Topographic data Geoscience Australia © 2006

0 1 2 Kilometres

A3 @ 1:70000
Ref: 4733 - 10
Author: DM

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Appendix F – Geotechnical Report

Opus International Consultants

Ellerton Drive Extension

Geotechnical Investigation Report

28 July 2014

Results emerge
when local knowledge
intersects with
global expertise

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Ellerton Drive Extension

Prepared for
Opus International Consultants

Prepared by
Coffey Geotechnics Pty Ltd
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t: +61 2 6260 7288 f: +61 2 6260 7211
ABN: 93 056 929 483

28 July 2014

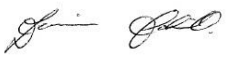
Document Authorisation

Our ref: GEOTFYSH09703AA-AC Rev 2

Coffey Geotechnics Pty Ltd (Coffey) is pleased to present the results of our geotechnical investigation for the proposed Ellerton Drive Extension.

If you have any questions related to the report, or we can be of any further assistance, please contact Bernice Cahill on (02) 6260 7288.

For and on behalf of Coffey



Bernice Cahill
Geotechnical Engineer

Quality information

Revision history

Revision	Description	Date	Author	Reviewer	Signatory
Draft	Draft Report	27 June 2014	Bernice Cahill	Peter Waddell	Bernice Cahill
Revision 1	Draft Report	17 July 2014	Bernice Cahill	Peter Waddell	Bernice Cahill
Revision 2 - Final	Final Report	28 July 2014	Bernice Cahill	Peter Waddell	Bernice Cahill

Distribution

Report Status	No. of copies	Format	Distributed to	Date
Revision 2	1	PDF	Opus International Consultants	28 July 2014

Executive Summary

The Ellerton Drive Extension Project (EDE) will provide an alternative route around Queanbeyan CBD, in addition to providing improved access over the Queanbeyan River during major flood events. The project comprises approximately 4.6 km of dual carriageway extending from the existing Ellerton Drive to the north to Edwin Land Parkway to the southwest and the construction of a bridge to cross the Queanbeyan River.

Fieldwork for the geotechnical investigation was carried out between 28 May and 6 June 2014 and comprised the investigation of the proposed route alignment, the Queanbeyan River Bridge Crossing and a pavement investigation of an existing portion of Ellerton Drive.

The investigation comprised drilling 17 boreholes to a target depth of approximately 1m below the proposed final cut level ranging from 3 m to 9.3 m and the excavation of 65 test pits to a target depth of 3 m or prior refusal on bedrock along the proposed route alignment. The Queanbeyan River Bridge Crossing investigation comprised the drilling of six at proposed bridge abutments and pier locations to depths ranging from 7.6 m to 15.8 m. To assess the existing pavement formation at Ellerton Drive, four augered boreholes were drilled to a target depth of 1.5 m.

On completion of fieldwork, soil and rock samples were taken to our NATA accredited laboratory for testing.

For specific detail on encountered subsurface conditions reference should be made to the attached Engineering Borehole Logs included in our report (Coffey Reference GEOTFYSH09703AA-AC, dated 1 July 2014). In summary the results of our investigation identified the following main geotechnical units within the investigation area; topsoil, fill, colluvium, residual soil, alluvium, and bedrock of the Pittman Formation, Barracks Creek Formation, and Colinton Volcanics.

Trafficability

The fine grained soils (clays) at the site are likely to become disturbed and soften when exposed and subjected to wetting. We recommend that where fine grained soils are exposed a general working platform of at least 0.3 m thickness of good quality crushed rock or recycled concrete be placed. A geotextile separator between the subgrade and working platform should be considered in areas subject to heavy traffic to reduce the risk of softened subgrade mixing with the working platform.

Excavatability

The distribution and/or presence of each unit will vary somewhat within each cut. Therefore, we recommend that the excavation assessment provided be referred to in conjunction with the Engineering Borehole and Test Pit Logs

Excavation within topsoil, fill, colluvium, alluvium and residual soils should be achievable with conventional plant such as dozers, scrapers and excavators. Excavation within extremely to highly weathered bedrock will likely to require ripping by dozer supplemented by hydraulic rock breakers. Ripping with dozer or rock hammering with large excavator may be very difficult in moderately weathered bedrock depending on in-situ defect spacing and will require very large plant. Ripping may be possible in slightly weathered to fresh bedrock but productivity may be low and the product blocky. Blasting is likely to be required in very high strength rock. Blasting is likely to be required for economic extraction and shaping of batters

Material Reuse

Site won material from topsoil, fill, colluvium, alluvium, residual soil and extremely to highly weathered bedrock should be suitable for re-use as engineered fill from a geotechnical perspective, provided unsuitable materials such as organics, waste or oversized particles are not present or can be removed.

It is expected that excavated moderately to freshly weathered bedrock will require crushing and screening to meet the nominated engineered fill criteria. Contractors should make their own assessment of processing required and the suitability of specific processing plant, based on the

engineering borehole and test pit logs and core photographs presented in our report and make their own assessment of excavation conditions and the suitability and production rates of specific plant.

Further geotechnical assessment, sampling and testing may be required during construction to assess the suitability of particular soils for re-use.

Cut Batter Design

Unprotected permanent cut and engineered fill batter slopes should be constructed not steeper than 2H: 1V for stability provided that drainage measures are implemented to intercept and divert water runoff from the crests of batters. Generally for batters higher than 7m, a 4.5m wide bench should be included with a 2:1 batter above the bench. It should be noted that should the batters be vegetated, it may require placement of geofabric to prevent erosion of placed topsoil while vegetation matures as erosion at this batter slope may be an issue, particularly after periods of heavy rainfall.

If more granular materials are encountered, having a low proportion of clayey fines, these materials are more likely to be subject to erosion by water flows. Protection against erosion by the establishment of vegetation and/or the use of jute mesh or other suitable covering is recommended.

The recommended batters assume no surcharge loads will be placed within a horizontal distance from the crest of the batter equal to the height of the batter.

Design CBR

The subsurface conditions along the proposed alignment of Ellerton Drive Extension are varied but generally comprise a topsoil layer, overlying colluvium, overlying variably weathered bedrock. Design CBR for various units encountered along the proposed alignment ranged from 1.5% to 10%.

Queanbeyan River Bridge Crossing

The foundation conditions at the proposed bridge crossing differ to the north and south of the Queanbeyan River. To the north of the river, sand and clay colluvium overlies moderately weathered to fresh shale bedrock. To the south of the Queanbeyan River fill and sand, gravel and clay alluvium overlies variably weathered limestone. It is expected that footings will comprise piled footings to rock.

Where piles are designed to extend into moderately to slightly weathered bedrock (or better), a large piling rig with appropriate rock drilling augers or buckets will be required.

Due to the nature of the site (being adjacent to an existing creek) groundwater inflows are expected to occur into pile excavations. However we have not assessed likely inflow rates at this stage. Open bored piles are unlikely to be practicable particularly on the southern piers and abutment. Continuous flight auger piles or cased bored piles should be practicable.
For additional details pertaining to bridge design parameters, reference should be made to Section 5.6.2 in our report (Coffey reference GEOTFYSH09703AA-AC, dated 1 July 2014).

Limitations

The executive summary provided should be read in conjunction with our geotechnical investigation report (Coffey Reference GEOTFYSH09703AA-AC, dated 1 July 2014) attached.

Subsurface conditions can be complex, vary over relatively short distances and over time. The inferred geotechnical model and recommendations in this report are based on limited subsurface investigations at discrete locations. The engineering logs describe subsurface conditions only at the investigation locations.

Additional investigations may be required to support detailed design due to factors such as scope limitations and changes to the nature of the project. During construction a geotechnical engineer should verify that conditions exposed are consistent with design assumptions.

The attached document entitled “Important Information about Your Coffey Report” forms an integral part of this report and presents additional information about the uses and limitations of the report.

Table of contents

Tables

Figures

Appendices

1. Introduction

The Ellerton Drive Extension Project (EDE) will provide an alternative route around Queanbeyan CBD, in addition to providing improved access over the Queanbeyan River during major flood events. The project comprises approximately 4.6 km of dual carriageway extending from the existing Ellerton Drive to the north to Edwin Land Parkway to the southwest and the construction of a bridge to cross the Queanbeyan River.

This report presents the results of a geotechnical investigation carried out by Coffey Geotechnics Pty Ltd (Coffey) to assist Opus Consulting Engineers (Opus) with preliminary design for the proposed EDE and provide detailed geotechnical information to allow engineering design and estimate of cost / quantities related to the following:

- Excavation conditions;
- Potential re-use of site won materials;
- Site preparation;
- Bulk Earthworks; and
- Pavements.

2. Desktop Review

2.1. Site Description and Observations

The proposed route alignment is shown in Figure 1. The majority of the proposed extension route is within the Cuumbuen Nature Reserve to the North of the Queanbeyan River and comprises rolling hills. The nature reserve is generally densely vegetated with some minor cleared areas for walking and bike trails.

To the south of the Queanbeyan River, the proposed route extension flanks Karabar residential developments and Barracks Flat Park. The area comprises undulating hills and rocky outcrops.

At the time of our investigation, some fill stockpiles containing variable material such as soil, rock and construction debris were observed within the corridor – most notably on the Reservoir access road east of Severne Street and to the immediate south of Barracks Flat Drive at the proposed location of the Queanbeyan River Bridge Crossing.

2.2. Local Geology

The Canberra Geological Sheet (1:100,000 scale) an extract of which is shown below, indicates that the site locality is underlain by early Silurian members of the Colinton Volcanics, Pittman and Cappanana Formations.

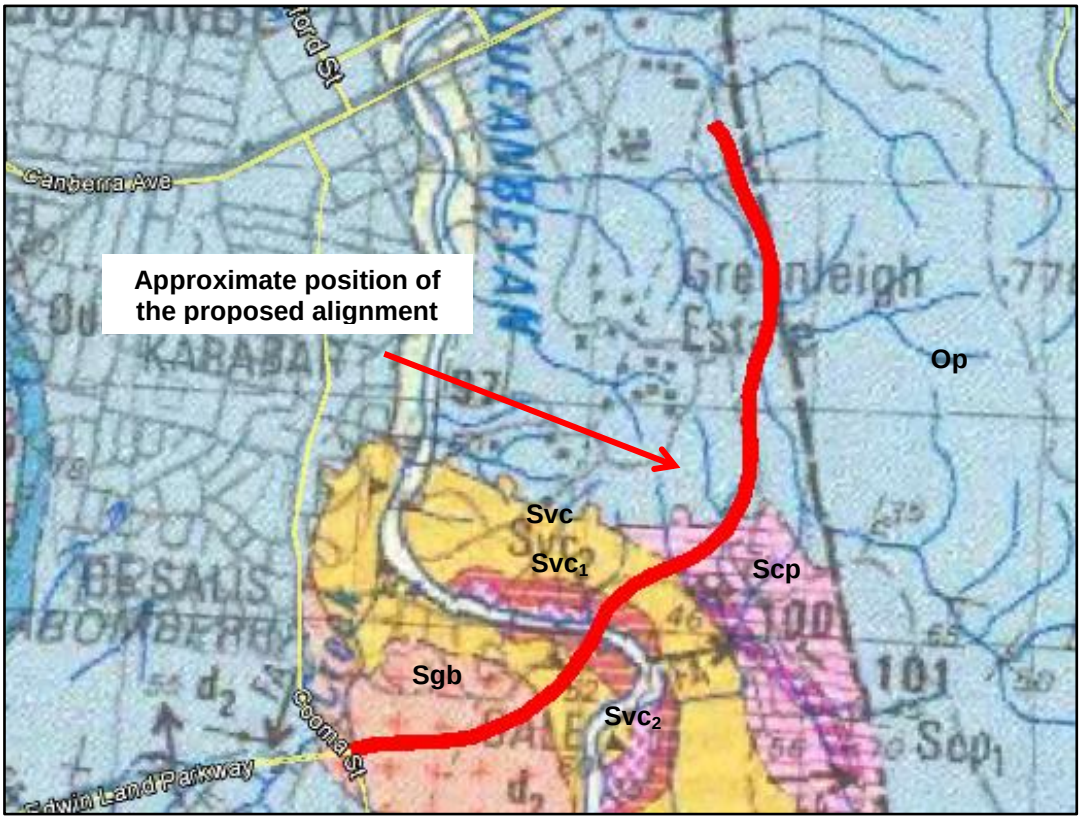


Figure 1: Extract of 1:100,000 Geological Map

Table 1: Geological Unit Description

Legend	Era	Period	Lithology
Op	Palaeozoic	Ordovician	Pittman Formation – Interbedded sandstone, siltstone and shale and minor black shale, chert and impure calcareous sandstone.
Scp	Palaeozoic	Silurian	Cappanana Formation – Shale, siltstone minor quartzite and tuff.
Svc	Palaeozoic	Silurian	Colinton Volcanics – Dark green dacitic ignimbrite and minor volcaniclastic sediments.
Svc_1	Palaeozoic	Silurian	Colinton Volcanics - Tuffaceous Shale.
Svc_2	Palaeozoic	Silurian	Colinton Volcanics – Limestone and dolomitic limestone.
Sgb	Palaeozoic	Devonian	Barracks Creek Adamellite and Leucogranite

3. Fieldwork

Fieldwork for the geotechnical investigation was carried out between 28 May and 6 June 2014 and comprised the investigation of the proposed route alignment, the Queanbeyan River Bridge Crossing and a pavement investigation of an existing portion of Ellerton Drive.

Proposed Route Alignment

The proposed route alignment extends 4.6 km from Ellerton Drive to Edwin Land Parkway. The investigation comprised drilling 17 boreholes (BH01 to BH17) to a target depth of approximately 1m below the proposed final cut level ranging from 3 m to 9.3 m and the excavation of 65 test pits to a target depth of 3 m or prior refusal on bedrock.

Queanbeyan River Bridge Investigation

The Queanbeyan River Bridge Crossing investigation comprised the drilling of six boreholes (B-BH01 to B-BH06) at proposed bridge abutments and pier locations to depths ranging from 7.6 m to 15.8 m.

Existing Ellerton Drive Pavement Investigation

To assess the existing pavement formation at Ellerton Drive, four augered boreholes (A-BH01 to A-BH04) were drilled to a target depth of 1.5 m.

3.1.1. Borehole Drilling

The boreholes were drilled using a Commachio 205 utility mounted drilling, an Edson 3000 truck mounted rig and a track mounted rig to access bridge abutment and pier locations. The boreholes were initially advanced using solid flight augers and a Tungsten Carbide (TC) drill bit. Following TC refusal, all boreholes were continued using NMLC and PQ diamond rock coring techniques to the nominated termination depths.

Groundwater inflows and soil moisture content observed during auger drilling were recorded on the engineering borehole logs.

On completion of drilling and logging, boreholes will be backfilled with cuttings. Where boreholes were positioned on the existing portion of Ellerton Drive the boreholes were backfilled with cuttings, tamped in place and a concrete plug was placed to the ground surface.

Borehole drilling was observed by a Coffey Geotechnical Engineer who was present throughout the drilling operations to undertake sampling and testing, record test results and log materials encountered. The Engineering Logs of the boreholes are attached in Appendix A, together with Coffey soil and rock explanation sheets which describe the terms and symbols used in log preparation.

3.1.2. Test Pitting

Test pitting was carried out using 6 and 20 tonne tracked excavators. Test pits were advanced to the nominated target depth of 3 m or prior refusal on bedrock. On completion, the test pits were backfilled with the excavated spoil and tamped at surface using the excavator bucket. The surface of the reinstated test pits was left slightly mounded above the general ground surface.

Dynamic Cone Penetrometer (DCP) testing were be carried out adjacent to each test pit location to a depth of 2m or prior refusal in order to provide an indication of soil strength. Such testing was not carried out where bedrock was observed to be at or near surface.

3.2. Laboratory Testing

On completion of fieldwork, soil and rock samples were taken to our NATA accredited laboratory, where the following tests were carried out:

- 22 ten day soaked California Bearing Ratios (CBR) (20 along the proposed route alignment and 2 from the existing portion of Ellerton Drive).
- 9 Atterberg Limits and Linear Shrinkages.
- 4 Particle Size Distributions (PSDs).
- 5 Emerson Class Numbers.
- 21 Natural Moisture Contents.
- 2 Shrink Swell Indexes.
- 4 Uniaxial Compressive Strengths (UCS) on rock cores.
- Point Load Strength Indexes were undertaken at approximately 1m intervals on rock cores.

Rock cores were colour photographed before testing. The point load strength index test results are included on the borehole logs presented in Appendix A. Laboratory test results of soil and rock UCS test results are presented in Appendix B.

Coffey had originally allowed for 5 shrink swell index tests as indicated in our proposal (Coffey Reference GEOTFYSH09703AA-Rev 2, dated 11 November 2013). However, due to the nature of the soils encountered, sample recovery from thin walled push tubes was limited. Additional Atterberg Limit and Linear Shrinkage testing was carried out in place of the shrink swell tests to provide an indication of shrink swell potential.

4. Results of Investigation

4.1. Subsurface Conditions - Proposed Ellerton Drive Extension

For specific detail on encountered subsurface conditions reference should be made to the attached Engineering Borehole Logs (Appendix A) and Engineering Test Pit Logs (Appendix B). In summary the results of our investigation identified the following main geotechnical units within the investigation area:

- **Topsoil** (**Unit 1**): CLAY, SILT and silty/clayey SAND, brown to dark brown, low plasticity clay, fine grained sand (up to 0.3 m thickness).
- **Fill (Unit 2)**: Clayey SAND, BOULDERS, brown to red-brown, coarse grained sand.

- **Colluvium (Unit 3):** Sandy CLAY, Gravelly CLAY, CLAY and Silty GRAVEL, pale brown to orange, low plasticity clay, fine grained sand.
- **Residual Soil (Unit 4):** Silty CLAY, Clayey SAND and SAND; pale orange, coarse grained sand, medium plasticity clay.
- **Bedrock (Unit 6):** Pittman Formation logged as interbedded SILTSTONE, SANDSTONE and SHALE.
- **Bedrock (Unit 7):** Barracks Creek Formation, logged as ADAMELLITE.

Groundwater was not observed during auger drilling in the boreholes or in the test pit excavations. The subsequent use of water flush for core drilling prevented further observations within bedrock during drilling.

The following sections provide an overview of the subsurface conditions encountered within each of the proposed cut and fill areas. Interpreted geotechnical sections for each area are presented as Drawing 021 to Drawing 027.

The depths and layer thicknesses of the geological units provided are based on subsurface conditions at the borehole and test pit locations and may not represent all areas of the site.

Table 2 and 3 overleaf provide a summary of subsurface conditions observed within boreholes and test pits located within the general areas of cut and fill along the proposed alignment.

Table 2: Summary of Subsurface Conditions with Areas of Proposed Cut

Cut ID	Approximate Start Chainage	Approximate End Chainage	Relevant Borehole/ Test Pit Logs	Unit 1	Unit 2	Unit 3a	Unit 3b	Unit 4	Units 6a & 7a	Units 6b & 7b	Unit 6c
				Topsoil	Fill	Colluvium (F to VST & L to MD)	Colluvium (H to FB & D to VD)	Residual Soil	Extremely to Highly Weathered Bedrock	Highly to Moderately Weathered Bedrock	Slightly Weathered to Fresh Bedrock
				Unit Thickness (m)							
1	200	440	TP01 to TP07, BH01	-	-	0.1 – 0.2	0.3 - > 3.1	2.0	> 0.2	-	-
2	500	610	BH02, BH03	-	-	-	0.8 – 3.8	-	> 0.2	> 0.2	>1.6
3	690	750	TP10, TP11	0.2	-	0.2 – 0.5	1.0 - > 1.9	-	-	> 0.3	
4	950	1075	BH04, BH05, TP15	-	-	0.2 - 0.3	-	-	> 0.4	> 2.4	> 1.9
5	1120	1240	BH06, BH07	-	0.7	0.7	-	-	> 0.3	> 6.2	> 0.8
6	1480	1560	BH08	-	-	-	0.4	-	-	0.6	> 5.0
7	1570	1640	BH09, TP26	-	-	-	-	-	> 0.7	> 0.4	> 5.0
8	1660	1800	BH10, BH11	-	-	0.1 - 0.25	-	-	0.55 – 0.6	> 8.42	> 0.6
9	2490	2770	TP40 to TP45, BH12 to BH14	0.2	-	0.1 - 0.5	0.3 – 1.9	0.7	> 0.5	> 5.36	> 0.9
10	3175	3530	BH15, BH16	0.2	-	0.4 - 0.6	-	-	1.8	0.4	> 3.2
11	3640	3770	BH17	0.2	-	0.6	-	-	-	2.0	> 0.2

Table 3: Summary of Subsurface Conditions with Areas of Propose Fill

Fill ID	Approximate Start Chainage	Approximate End Chainage	Relevant Borehole/ Test Pit Logs	Unit 1	Unit 2	Unit 3a	Unit 3b	Unit 4	Units 6a & 7a	Units 6b & 7b	Unit 6c
				Topsoil	Fill	Colluvium (F to VST & L to MD)	Colluvium (H to FB & D to VD)	Residual Soil	Extremely to Highly Weathered Bedrock	Highly to Moderately Weathered Bedrock	Slightly Weathered to Fresh Bedrock
				Unit Thickness (m)							
1	440	500	TP08	-	-	0.7	0.3	-	>0.2	-	-
2	610	690	TP09	-	-	0.2	-	-	>0.5	-	-
3	750	950	TP12 to TP14	0.3	-	0.1 - 0.3	0 - >3.4	-	>1.5	-	-
4	1075	1120	TP15 & TP16	-	-	0.2 - 0.4	0.5	-	>0.1	-	-
5	1240	1480	TP20 to TP23	0.2	-	0.1 – 0.3	0.7 - > 3.0	-	>0.2	-	-
6	1560	1570	TP27	-	-	0.1	-	-	> 0.5	-	-
7	1640	1660	TP27	-	-	0.1	-	-	> 0.5	-	-
8	1800	2490	TP28 To TP39	0.2 - 0.3	-	0.2 – 0.5	0.1 - > 2.7	-	> 0.1	-	-
9	2770	3175	TP50 To TP52	-	-	0.1 – 0.6	-	-	> 0.2	-	-
10	3530	3640	TP55 & TP56	0.2	-	-	-	0.85	> 0.2	-	-
11	3770	4150	TP58 To TP65	0.1 - 0.2	0.2 - 0.5	0.2 – 0.6	0.4 - 1.0		> 0.2	-	-

4.1.1. Subsurface Conditions – Queanbeyan River Crossing

For specific detail on encountered subsurface conditions reference should be made to the attached Engineering Borehole Logs (Appendix A) and Engineering Test Pit Logs (Appendix B). In summary the results of our investigation identified the following main geotechnical units within the Queanbeyan River Bridge Crossing investigation area:

- **Topsoil (Unit 1):** CLAY, SILT and silty/clayey SAND, brown to dark brown, low plasticity clay, fine grained sand.
- **Fill (Unit 2):** Clayey SAND, BOULDERS, brown to red-brown, coarse grained sand.
- **Colluvium (Unit 3):** Sandy CLAY, Gravelly CLAY, CLAY and Silty GRAVEL, pale brown to orange, low plasticity clay, fine grained sand.
- **Alluvium (Unit 5):** comprising Sandy CLAY/Clayey SAND, low plasticity, fine grained sand.
- **Bedrock (Unit 8);** Colinton Volcanics Formation, logged as interbedded SHALE.
- **Bedrock (Unit 9)** Colinton Volcanics Formation, logged as LIMESTONE.

Groundwater inflow was observed during auger drilling at B-BH03 at a depth of 1.3 m. The subsequent use of water flush for core drilling prevented further observations within bedrock during drilling.

The following sections provide an overview of the subsurface conditions encountered within each of the areas. Interpreted geotechnical sections for the bridge site are presented as Drawing 025.

The depths and layer thicknesses of the geological units provided are based on subsurface conditions at the borehole and test pit locations and may not represent all areas of the site.

Table 4 below provides a summary of subsurface conditions observed within boreholes B-BH01 to B-BH06 and test pits TP48 to TP49.

Table 4: Summary of Subsurface Conditions (Queanbeyan River Crossing)

Unit	Geotechnical Unit	Material Description	Depth to Top of Unit (m)	Unit Thickness (m)
1	Topsoil	Silty SAND, fine to medium grained sand	0.0	0.1 – 0.2
2	Fill	Clayey SAND, BOULDERS, coarse grained sand, medium to large boulders, medium plasticity clay	0.0 – 0.1	2.5 – Unproven
3a	Colluvium	Sandy CLAY, Clayey SAND, low plasticity, fine grained, firm to very stiff or loose to medium dense	0.8	1.9
3b		Sandy/Gravelly/Silty CLAY,CLAY, low plasticity, fine grained sand, fine grained sub-angular gravel, hard	0.0	0.8
5a	Alluvium	Silty SAND/GRAVEL, fine to coarse grained, loose to medium dense	0	2.8
5b		Silty/Sandy CLAY, high plasticity, stiff to very stiff consistency	0.0 – 2.5	1.6 - 6.4

Unit	Geotechnical Unit	Material Description	Depth to Top of Unit (m)	Unit Thickness (m)
8a	Shale Bedrock – Colinton Volcanics	Shale: Extremely to Highly Weathered Typically very low to medium strength	0.0	0.45
8b		Shale: Moderately Weathered Typically medium and high strength	0.45 – 2.7	0.45 - 1.2
8c		Shale: Slightly Weathered to Fresh Typically high and very high strength	1.2 – 2.8	Unproven
9a	Limestone Bedrock - Colinton Volcanics	Limestone: Extremely to Highly Weathered Typically very low to medium strength	2.5 - 2.8	0.1 – 1.8
9b		Limestone: Moderately Weathered Typically medium and high strength	3.25 – 4.5	Unproven
9c		Limestone: Slightly Weathered to Fresh Typically high and very high strength	3.18 - 7.7	Unproven

4.1.2. Subsurface Conditions – Existing Ellerton Drive

The subsurface conditions observed within boreholes A-BH01 to A-BH04 within the existing portion of Ellerton Drive are presented below in Table 5.

Table 5: Summary of Subsurface Conditions (Existing Ellerton Drive)

Geotechnical Unit	Material Description	Depth to Top of Unit (m)	Unit Thickness (m)
2 Road Base	Gravelly SAND, fine to coarse grained, grey fine to medium grained sub-angular gravel	0.03	0.07 – 0.3
2 Fill	Gravelly/Clayey SAND, Gravelly CLAY, fine to coarse grained, orange to orange brown, fine to medium grained, sub-angular gravel, medium plasticity clay	0.1 - 0.33	0.23-0.27
3 Colluvium	Sandy CLAY, Clayey SAND, low plasticity, fine grained, firm to very stiff or loose to medium dense	0.33 – 0.6	To limit of investigation
5 Residual Soil	Silty CLAY/Clayey SAND, low plasticity, fine to coarse grained, very stiff or dense	0.33	0.27
6a Bedrock	Interbedded Siltstone/ Sandstone and Shale Extremely to Highly Weathered Typically very low to medium strength	1.3	To limit of investigation

4.2. Laboratory Test Results

The results of laboratory testing are summarised in Table 6 to Table 8. The moisture content test results ranged from 8.6% to 29.3%. For further detail reference should be made to the laboratory test certificates in Appendix C.

Table 6: Summary Soil Classification Test Results

Sample ID	Sample Interval (m)	Liquid Limit (%)	Plastic Limit (%)	Plasticity Index (%)	Linear Shrinkage (%)	Gravel (%)	Sand (%)	Fines (%)	Emerson Class Number	Iss (%)
TP02	0.7-0.9	33	14	19	10	-	-	-	5	-
TP12	0.8-0.9	19	14	5	-	-	-	-	-	-
TP13	1.0-1.5	19	12	7	4.0	-	-	-	-	-
TP23	0.5-0.6	24	15	9	4.5	-	-	-	5	-
TP39	1.0-1.1	25	13	12	-	-	-	-	-	-
TP39	1.1-1.5	24	15	9	4.5	-	-	-	-	-
TP40	0.4-0.6	-	-	-	-	14	39	47	-	-
TP41	0.0-0.5	38	19	19	8.5	-	-	-	-	-
TP47	1.0-1.3	-	-	-	-	47	13	30	-	-
TP56	0.2-1.0	-	-	-	-	-	-	-	5	-
TP57	0.4-0.62	-	-	-	-	-	-	-	-	1.2
TP64	0.2-0.6	28	13	15	5.5	-	-	-	-	-
TP65	2.5-3.0	-	-	-	-	24	31	45	5	-
B-BH03	0.6-0.9	31	18	13	-	-	-	-	4	-
B-BH03	1.0-1.45	22	13	9	-	-	-	-	-	0
B-BH03	2.5-2.95	-	-	-	-	33	34	33	-	-

Table 7: Summary of CBR Test Results

Sample ID	Material Description	Sample Interval (m)	Maximum Dry Density (t/m^3)	Optimum Moisture Content (%)	CBR (%)
TP02	Sandy CLAY	0.7-0.9	1.76	14.6	5
TP03	Sandy CLAY	0.0-0.5	1.86	12.6	1.5
TP08	Sandy CLAY	0.5-0.6	1.75	17.5	2.5
TP13	Gravelly CLAY	1.0-1.5	1.96	11.3	11
TP15	SANDSTONE	0.2-0.6	1.95	12.8	6
TP20	Gravelly CLAY	0.6-0.7	1.9	13.2	2.5
TP22	SANDSTONE	0.8-0.9	2.0	9.9	11

Sample ID	Material Description	Sample Interval (m)	Maximum Dry Density (t/m^3)	Optimum Moisture Content (%)	CBR (%)
TP23	Silty CLAY	0.5-0.6	1.92	11.5	13
TP27	SANDSTONE	0.5-0.6	1.98	12.2	9
TP35	SHALE	0.4-0.5	1.75	18.2	9
TP39	Gravelly CLAY	1.1-1.5	1.98	10.1	15
TP41	Silty CLAY	0.0-0.5	1.74	16.8	5
TP42	SHALE	0.5-1.0	1.88	13.2	7
TP44	SHALE	0.5-0.6	2.0	11.4	15
TP48	Silty CLAY	2.5-2.8	1.38	28.4	1.5
TP52	Clayey SAND	0.3-0.6	1.92	12.5	13
TP53	SHALE	0.5-1.0	1.92	12.1	9
TP59	ADAMELLITE	0.8-1.2	1.95	11.9	8
TP60	Clayey SAND	0.5-1.0	1.94	12.4	9
TP64	Silty CLAY	1.0-1.5	1.80	15.8	6
	Existing Ellerton Drive				
A-BH02	CLAY	0.9-1.2	1.72	17.9	3.5
A-BH03	Clayey SAND	1.2-1.4	1.82	15.6	4.5

Table 8: Summary of Unconfined Compressive Strength Test Results

Borehole ID	Sample Interval (m)	Material Description	Uniaxial Compressive Strength (MPa)
BH05	3.25 – 3.57	Sandstone	4.33
BH14	2.4 – 2.62	Siltstone	9.94
BH15	3.33 – 3.57	Sandstone	9.64
BH17	1.8 – 2.0	Adamellite	6*

*Sample was unsuitable for UCS testing; this value is inferred value from Is50 point load strength index testing.

5. Discussion and Recommendations

5.1. Earthworks

5.1.1. Trafficability of Soils

The fine grained soils (clays) at the site are likely to become disturbed and soften when exposed and subjected to wetting. We recommend that where fine grained soils are exposed a general working platform of at least 0.3 m thickness of good quality crushed rock or recycled concrete be placed. A geotextile separator between the subgrade and working platform should be considered in areas subject to heavy traffic to reduce the risk of softened subgrade mixing with the working platform.

Where heavy construction plant such as piling rigs are required to travel on site a specific working platform design should be undertaken and the thickness of working platform may need to be increased from that recommended above.

5.1.2. Excavation Conditions

Materials encountered during excavation will be governed by the final proposed excavation depths. We understand that a final cut level range of up to approximately 8 m is proposed. However, final excavation depths may vary following consideration of the findings of this investigation.

It is expected that excavations within each area will extend through the soil profile and terminate within bedrock. Excavation conditions within each cut are expected to vary based on the thickness of the soil profile, variations in geological stratum, degree/extent of weathering and discontinuity spacing within the bedrock. Reference should be made to Tables 2 and 3 and the interpreted geotechnical sections (Drawing 021 to Drawing 027), which show the general distribution of the geotechnical units at the various cut and fill locations.

Excavation contractors should be given an opportunity to review the engineering borehole and test pit logs and core photographs in Appendix A and Appendix B to make their own assessment of excavation conditions and the suitability and production rates of specific plant.

The distribution and/or presence of each unit will vary somewhat within each cut. Therefore, we recommend that the excavation assessment provided in Table 9 be referred to in conjunction with the Engineering Borehole and Test Pit Logs (attached as Appendix A and Appendix B).

Table 9: Excavatability Assessment

Unit ID	Geological Unit	Material Description	Summary
1 to 5	Topsoil, Fill, Colluvium, Residual Soil and Alluvium	Silty SAND, clayey SAND, gravelly SAND, fine to medium grained sand	Conventional plant such as dozers, scrapers and excavators
6a, 7a, 8a, 9a	Bedrock	Extremely to Highly Weathered, typically very low and low strength	Likely to require ripping by dozer supplemented by hydraulic rock breakers.
6b, 7b, 8b, 9b	Bedrock	Moderately Weathered, typically medium and high strength	Ripping with dozer or rock hammering with large excavator may be very difficult depending on in-situ defect spacing and will require very large plant.

Unit ID	Geological Unit	Material Description	Summary
6c, 8c, 9c	Bedrock	Slightly Weathered to Fresh, typically high and very high strength	Ripping may be possible in high strength rock but productivity may be low and the product blocky. Blasting is likely to be required in very high strength rock. Blasting is likely to be required for economic extraction and shaping of batters.

5.1.3. Suitability of On-Site Materials for use as Engineered Fill

Site won material from Units 2, 3, 4, 5a, 6, 7a, 8a should be suitable for re-use as engineered fill from a geotechnical perspective, provided unsuitable materials such as organics, waste or oversized particles are not present or can be removed.

It is expected that excavated moderately to freshly weathered bedrock (Units 6b, 6c, 7b, 8b, 8c, 9b and 9c) will require crushing and screening to meet the nominated engineered fill criteria. Contractors should make their own assessment of processing required and the suitability of specific processing plant, based on the engineering borehole and test pit logs and core photographs in Appendix A and Appendix B to make their own assessment of excavation conditions and the suitability and production rates of specific plant.

Further geotechnical assessment, sampling and testing may be required during construction to assess the suitability of particular soils for re-use.

5.1.4. Site Preparation and Compaction Criteria

Stiff clays or dense sands at subgrade level should not require extensive treatment provided they are not disturbed by traffic or water ingress. Where natural soils are exposed and no filling is required, pavement subgrade and foundation preparation should consist of bulk excavation to subgrade or foundation level followed by geotechnical assessment of the exposed stratum. If assessed to be suitable, the foundation should be protected from softening as soon as possible. Pavement subgrades should be graded to drain effectively and should be cleaned of any softened material prior to placement of pavement materials.

General guidelines for earthworks are as follows:

- Strip all topsoil and unsuitable material such as softened or heaving soils, if present;
- Box out pavements to proposed subgrade level if this is deeper than the stripped level;
- Engineered Fill should be placed uniform layer thicknesses suitable for capacity of the available plant.
- Fill should be compacted to the required level within the specified moisture content range.
- Testing should be undertaken at the frequency required under the relevant road construction specification.

5.1.5. Cut Batter Design

Excavations within areas of cut will require the forming of permanent batters during excavation, construction and subsequent longer term operation.

The major criteria influencing cut batter design is cut slope stability (per batter and overall slope) together with the potential for erosion of exposed materials. The geotechnical model for the project, as presented in the geotechnical long sections and cross sections, suggests many of the cuts will contain colluvium, extremely weathered rock (soil strength) and highly weathered to moderately weathered interbedded siltstone, sandstone and shale, which is distinctly bedded and highly fractured.

The Emerson Class Number dispersion tests testing performed on these materials indicates the majority of the soils are Class 5 indicating a low potential for dispersion within the upper soil profile. Unprotected permanent cut and engineered fill batter slopes should be constructed not steeper than 2H: 1V for stability provided that drainage measures are implemented to intercept and divert water runoff from the crests of batters. If more granular materials are encountered, having a low proportion of clayey fines, these materials are more likely to be subject to erosion by water flows. For batters in excess of 7m, benching should be included with a 2:1 batter above the bench. Protection against erosion by the establishment of vegetation and/or the use of jute mesh or other suitable covering is recommended.

The recommended batters assume no surcharge loads will be placed within a horizontal distance from the crest of the batter equal to the height of the batter.

Notwithstanding the above recommended batter slopes, there may be unfavourable conditions within the exposed materials which may require treatment, such as laying back and/or shotcreting. Unsupported batters should be assessed by a suitably qualified geotechnical engineer or engineering geologist at regular intervals during excavation and immediately after rainfall events to assess requirements for stabilisation measures or maintenance.

5.2. Proposed Ellerton Drive Extension

The subsurface conditions along the proposed alignment of Ellerton Drive Extension are varied but generally comprise a topsoil layer, overlying colluvium, overlying variably weathered bedrock.

Table 10 below provides recommended design CBR for various units encountered along the proposed alignment.

Table 10: Recommended Pavement Design CBR for Various Units

Unit	Geotechnical Unit	Material Description	Design CBR (%)
3a	Colluvium	Sandy CLAY, Clayey SAND	1.5
3b		Sandy/Gravelly/Silty CLAY,CLAY, Clayey/Silty Sand	5
4	Residual Soil	Silty CLAY/Clayey SAND	5
5	Alluvium	Silty/Sandy CLAY	1.5
6a, 7a	Bedrock	Extremely to Highly Weathered Typically very low to low strength	7.5
6b, 7b		Moderately Weathered Typically medium to high strength	10

We recommend that provision be allowed for observation of the prepared subgrade by an experienced geotechnical engineer to confirm the design value presented above is consistent with the exposed conditions

Where fill embankments are constructed from imported fill materials, the design subgrade CBR value at subgrade level will depend on the nature of the imported materials. The pavement designer should take account of the presence of potentially lower strength materials if on-site materials are present within a depth of 1m below the pavement subgrade level.

5.3. Existing Ellerton Drive

The subsurface conditions along the existing portion of Ellerton Drive comprise the existing road formation underlain by Units 3a (colluvium) and 4 (residual soil). The CBR test result on Units 3a and 4 yielded a soaked CBR of 4.5% and 3.5% respectively. We recommend a design CBR of 3% for pavement design purposes.

5.4. Lonergan Drive Connection

The subsurface conditions along the proposed Lonergan Drive connections comprise the existing road formation underlain by Units 3a, 3b and 6a. The CBR test result on Units 3a, 3b and 6a yielded a soaked CBR of 1.5%, 5%, 7% and 15% respectively with the low value of 1.5% obtained on a sample of Unit 3a comprising sandy clay of firm consistency. We recommend stripping the low CBR Unit 3a sandy clay to expose underlying hard clay or weathered bedrock and adoption of a design CBR of 5% for pavement design purposes.

5.5. Barracks Flat Drive Connection

The subsurface conditions along the proposed Lonergan Drive connections comprise the existing road formation underlain by units 1, 2, 6b and 7a. The CBR test result on Units 6b and 7a yielded a soaked CBR of 9% and 8% respectively. We recommend stripping Units 1 and 2 (topsoil and fill) to expose underlying weathered bedrock and adopt a design CBR of 8% for pavement design purposes.

5.6. Queanbeyan River Bridge Crossing

5.6.1. Foundation Options

The foundation conditions at the proposed bridge crossing differ to the north and south of the Queanbeyan River. To the north of the river, sand and clay colluvium overlies moderately weathered to fresh shale bedrock. To the south of the Queanbeyan River fill and sand, gravel and clay alluvium overlies variably weathered limestone. It is expected that footings will comprise piled footings to rock.

Where piles are designed to extend into moderately to slightly weathered bedrock (or better), a large piling rig with appropriate rock drilling augers or buckets will be required.

Due to the nature of the site (being adjacent to an existing creek) groundwater inflows are expected to occur into pile excavations. However we have not assessed likely inflow rates at this stage. Open bored piles are unlikely to be practicable particularly on the southern piers and abutment. Continuous flight auger piles or cased bored piles should be practicable. For cased piles provision should be made for suitable cleaning buckets, dewatering equipment and concrete tremies. Should groundwater inflows be encountered within bored pile holes, allowance should be made for concreting bored piles using tremie placement methods.

5.6.2. Foundation Design Parameters

Recommendations for ultimate end bearing pressure and ultimate shaft adhesion values for piles are presented in Table 11 below:

Table 11: Foundation Design Parameters for Piled Foundations

Unit	Bulk Unit Weight (kN/m^3)	Drilled Shafts in Rock [1]				
		Ultimate Skin Friction (kPa)	Ultimate End Bearing Capacity (MPa)	Ultimate Lateral Yield Pressure (MPa)	Young's Modulus for Elastic Response (MPa)	Young's Modulus for Lateral Response (MPa)
2, 3, 5	20	50	-	0.25	20	14
8a, 9a	21	120	3	1.5	100	70
8b	23	500	12	6	780	550
8c	24	1,000	60	30	2,800	2,000
9b	22	500	12	6	900	675

Unit	Bulk Unit Weight (kN/m^3)	Drilled Shafts in Rock [1]				
		Ultimate Skin Friction (kPa)	Ultimate End Bearing Capacity (MPa)	Ultimate Lateral Yield Pressure (MPa)	Young's Modulus for Elastic Response (MPa)	Young's Modulus for Lateral Response (MPa)
9c	24	1,500	40	20	2,400	1,800

To adopt the recommended end bearing pressures footings should have a minimum embedment of 0.5 m into the relevant bearing stratum. Shaft adhesion should only be adopted where piles have a minimum socket of 2 pile diameters into the relevant stratum.

For limit state design a geotechnical reduction factor (Φg) is to be applied to the ultimate geotechnical pile capacity assessed using the ultimate shaft resistance and end bearing values shown in Table 11 to derive the design ultimate geotechnical pile capacity.

In accordance with AS2159-2009, Φg is dependent on assignment of an Average Risk Rating (ARR) which takes into account various geotechnical uncertainties, redundancy of the foundation system, construction supervision, and the quantity and type of pile testing. The assessment of Φg therefore depends on the structural design of the foundation system as well as the design and construction method, and testing (if any) to be employed by the designer and piling contractor.

The selected value of Φg should be reviewed by Coffey. Where testing is undertaken, it may be possible to adopt a Φg value that leads to a more economical design.

The use of limit state design also requires that serviceability performance of the foundation system be assessed, including pile group interaction effects. Such assessment should be carried out by experienced geotechnical professional using well-established and soundly based methods. The elastic modulus values given in Table 11 may be adopted for such assessment, but it should be recognised that the accuracy of settlement prediction is a function of construction methodology as well as the assessed values of material stiffness, both of which can involve considerable uncertainty. Therefore, the accuracy of settlement predictions may be no better than ± 50%. Where foundation settlement is critical to the performance of the structure, serviceability pile load testing should be carried out to confirm the design assumptions and/or assess prediction accuracy.

The maximum depth of investigation penetrated of the order of 15m. If pile loads result in pile lengths that exceed the depth of investigation then the pile designs should be reviewed by a geotechnical engineer and additional boreholes may be required to verify that the assumed rock quality is available below the proposed pile toe level.

5.6.3. Foundation Construction and Verification

The above recommended design parameters are based on rock strength, defect and defect spacing as observed in the logs. For the better quality rock (Units 7c and 8c) rock, the design values assume that defects such as clay seams are relatively minor. These Units will be relatively difficult to penetrate and therefore an assumption of long sockets may prove to be problematic to construct.

The recommended shaft adhesion values assume that the sides of bored pile holes are rough. Previous experience by Coffey indicates that provided the drilling contractor uses an auger and not a bucket to drill the bored piles the above parameters may be adopted and a reduction is not needed. Should a bucket auger be used to drill the piles the sides of the hole are more likely to be smeared (i.e. not rough) and a reduction in the side adhesion value is likely to be required unless the pile hole

is grooved with a specific-purpose grooving tooth. The bases of bored pile holes should be cleaned with a cleaning bucket.

Limestone (identified as Unit 9) is prone to karst solution features and voids, additional cored boreholes should be drilled at the remaining pier and abutment locations to verify that rock of suitable quality is present and particularly that there are no voids within a distance of at least 5 pile diameters or 5 m, whichever is greater, below target pile toe levels.

The recommended values outlined above assume that bearing surfaces are clean and free from spoil and other soft and loose material. We recommend footing excavations be assessed by a Geotechnical Engineer or Engineering Geologist to confirm subsurface conditions are consistent with the findings of this investigation prior to placement of reinforcement or pouring of concrete.

6. Limitations

Subsurface conditions can be complex, vary over relatively short distances and over time. The inferred geotechnical model and recommendations in this report are based on limited subsurface investigations at discrete locations. The engineering logs describe subsurface conditions only at the investigation locations.

Additional investigations may be required to support detailed design due to factors such as scope limitations and changes to the nature of the project. During construction a geotechnical engineer should verify that conditions exposed are consistent with design assumptions.

The attached document entitled “Important Information about Your Coffey Report” forms an integral part of this report and presents additional information about the uses and limitations of the report.

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

OPUS INTERNATIONAL CONSULTANTS

ELLERTON DRIVE EXTENSION

QUEANBEYAN, NSW

revision	no.	description	drawn	approved	date

drawn	BC / LH
approved	BC
date	28/07/14
scale	N.T.S.
original size	A3

coffey

client:	OPUS INTERNATIONAL CONSULTANTS		
project:	ELLERTON DRIVE EXTENSION ELLERTON DRIVE, QUEANBEYAN NSW		
title:	COVER SHEET		
project no: GEOTFYSH09703AA	drawing no: 001	rev: A	

PLOT DATE: 28/07/2014 10:32:35 AM DWG FILE: F:\GEOTECHNICS\1.PROJECTS\OTHER OFFICE JOBS\CANBERRA\GEOTFYSH09703AA\CAD\GEOTFYSH09703AA.DWG

LEGEND FOR LONG SECTION

GEOTECHNICAL KEY

- FILL
- TOPSOIL
- SHALE
- SILTY SAND
- SANDY CLAY
- GRAVEL
- SANDY GRAVEL
- LIMESTONE
- CLAYEY GRAVEL
- SILTY CLAY
- SILTSTONE
- SANDSTONE
- CLAYEY SAND
- QUARTZITE
- SILTY GRAVEL
- SANDY SILT
- GRANITE
- GRAVELLY SILTY CLAY
- SHALE
- GRAVELLY CLAY
- CLAY
- SILTY GRAVELLY SAND
- NO CORE

SOIL CONSISTENCY/RELATIVE DENSITY

VS	VERY SOFT
S	SOFT
F	FIRM
ST	STIFF
VST	VERY STIFF
H	HARD
Fb	FRIABLE
VL	VERY LOOSE
L	LOOSE
MD	MEDIUM DENSE
D	DENSE
VD	VERY DENSE
N=30	STANDARD PENETRATION TEST RESULTS

ROCK STRENGTH

EL	EXTREMELY LOW
VL	VERY LOW
L	LOW
M	MEDIUM
H	HIGH
VH	VERY HIGH
EH	EXTREMELY HIGH

WEATHERING GRADE

EW	EXTREMELY WEATHERED
HW	HIGHLY WEATHERED
MW	MODERATELY WEATHERED
SW	SLIGHTLY WEATHERED
FR	FRESH

GEOTECHNICAL UNIT

- UNIT 1, TOPSOIL
- UNIT 1, FILL
- UNIT 3a, COLLUVIUM (F-Vst OR l TO MD)
- UNIT 3b, COLLUVIUM (H-Fb or D TO VD)
- UNIT 4, RESIDUAL SOIL
- UNIT 5, ALLUVIUM
- UNIT 6a AND UNIT 8a, SILTSTONE / SANDSTONE / SHALE (XW-HW)
- UNIT 6b AND UNIT 8b, SILTSTONE / SANDSTONE / SHALE (HW-MW)
- UNIT 6c AND UNIT 8c, SILTSTONE / SANDSTONE / SHALE (SW-FR)
- UNIT 7a, ADAMELLITE (XW-HW)
- UNIT 7b, ADAMELLLITE (MW-FR)
- UNIT 9a, LIMESTONE (XW-HW)
- UNIT 9b, LIMESTONE (HW-MW)
- UNIT 9c, LIMESTONE (SW-FR)

EXISTING GROUND LEVEL

PROPOSED DESIGN LEVEL

LEGEND FOR SITE PLANS

- BOREHOLE
- TEST PIT

DRAWING INDEX

DRAWING No.	TITLE
001	COVER SHEET
002	DRAWING INDEX AND LEGEND
003	INVESTIGATION LOCATION PLAN
011	INVESTIGATION LOCATION PLAN - SHEET 1 OF 6
012	INVESTIGATION LOCATION PLAN - SHEET 2 OF 6
013	INVESTIGATION LOCATION PLAN - SHEET 3 OF 6
014	INVESTIGATION LOCATION PLAN - SHEET 4 OF 6
015	INVESTIGATION LOCATION PLAN - SHEET 5 OF 6
016	INVESTIGATION LOCATION PLAN - SHEET 6 OF 6
017	INVESTIGATION LOCATION PLAN - QUEANBEYAN RIVER CROSSING
021	GEOTECHNICAL LONG SECTION - SHEET 1 OF 7
022	GEOTECHNICAL LONG SECTION - SHEET 2 OF 7
023	GEOTECHNICAL LONG SECTION - SHEET 3 OF 7
024	GEOTECHNICAL LONG SECTION - SHEET 4 OF 7
025	GEOTECHNICAL LONG SECTION - SHEET 5 OF 7
026	GEOTECHNICAL LONG SECTION - SHEET 6 OF 7
027	GEOTECHNICAL LONG SECTION - SHEET 7 OF 7

PLOT DATE: 28/07/2014 10:32:36 AM DWG FILE: F:\GEOTECHNICS\1.PROJECTS\OTHER OFFICE JOBS\CANBERRA\GEOTFYSH09703AA\CAD\GEOTFYSH09703AA.DWG

revision

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coffey

client: OPUS INTERNATIONAL CONSULTANTS

project: ELLERTON DRIVE EXTENSION
ELLERTON DRIVE, QUEANBEYAN NSW

title: DRAWING INDEX AND LEGEND

project no: GEOTFYSH09703AA | drawing no: 002 | rev: A