22 May 2014

GEOTECHNICAL INVESTIGATION REPORT

Gunnedah second road over rail bridge - Option Development

Submitted to:
Wojtek Zborowski
Project Manager
KBR
201 Kent Street
Sydney NSW 2000
Office Phone +61 02 8284 2170

Report Number. 127622030-008-Rev2

Distribution:

KBR - 1 electronic, 1 hard copy,
Golder Associates - 1 hard copy, 1 electronic

REPORT

Golder Associates

Table of Contents

FIGURES

APPENDICES

User Note: This Table of Contents section acts as a reference point for the Record of Issue, Executive Summary and Study Limitations sections as and when they might be required. Therefore, the structure of this section must not be altered in any way.

1.0 INTRODUCTION

Golder Associates Pty Ltd (Golder) has been appointed by KBR Pty Ltd (KBR) to carry out a geotechnical investigation to support design work for the development and assessment of concept options for the Gunnedah Second Road Over Rail Bridge project.

The overall project is being delivered by NSW Roads and Maritime Services (Roads and Maritime), working with Gunnedah Shire Council as a key stakeholder.

The geotechnical and environmental investigations form part of the scope of project services described in the Roads and Maritime tender document 'Scope of Work and Technical Criteria - Contract No. 12.2547.2214'.

The investigation has been carried out in accordance with Golder proposal (reference P27622089-001-L-Rev0) and has been delivered in two principal phases, namely;

1) A geotechnical and environmental desk study, comprising a review of available geotechnical and environmental information (completed and presented as Golder report reference: 127622030-002-Rev1, May 2013)

2) An intrusive geotechnical investigation, comprising fieldwork, laboratory analysis and an interpretative report (presented as this report).

2.0 PROJECT APPRECIATION

The main aim of the Gunnedah Second Road Over Rail Bridge project is to select a preferred option for a new Higher Mass Limit (HML) road vehicle crossing over a major rail line. The new crossing will upgrade the current route, which uses the level crossing located at New Street in the western part of central Gunnedah.

At the time of writing, three concept bridge alignment options are being considered, namely:

- Option A: an alignment which follows the current HML route along New Street (to the east of the Gunnedah Maize Mill) and replaces the current level crossing

- Option B: an alignment which extends from the southern end of New Street to Warrabungle Street, arcing around the western side of the Gunnedah Maize Mill close to Blackjack Creek

- Option C: a similar alignment to Option B, with an alternative connection to the Oxley Highway/New Street roundabout, incorporating a new spur off this junction.

Two bridge heights are being considered, to permit either single stack or double stack trains to pass beneath. For the purposes of this report, the generally more onerous case of double stack clearance (ie with anticipated higher bridge structure loads) has been considered.

Concept alignment options sketches, as provided by KBR, are included in Appendix A of this report.

2.1 Background review

A review of available geotechnical and environmental information has been carried out by Golder and outcomes presented in the abovementioned desk study report. These findings have not been repeated in their entirety within this report; the reader should refer to the desk study report in conjunction with reading this report.

2.2 Site setting

The location of the Roads and Maritime Study Area for concept route option development is on the western side of Gunnedah town (see Figure 1), covering an area of about 25 hectares.

The study area generally comprises:

- Open undeveloped spaces, including Blackjack Creek and areas of mature trees
- Transport corridors including: Farrar Road, Oxley Highway, View Street, South Street, New Street, Warrabungle Street and the Kamilaroi Highway, and the Hunter Valley rail corridor - managed by the Australian Rail Track Corporation (ARTC)
- Low density residential and commercial properties, notably including the Gunnedah Maize Mill, which is situated towards the centre of the study area.

2.3 Anticipated ground conditions

Published geological maps (see desk study report for references) record:

- The majority of the study area is underlain the Leard Formation, which includes flint claystone, conglomerate, sandstone and siltstones
- The north-east section of the study area is underlain by the Porcupine Formation, which includes conglomerate, sandstone and siltstone
- To the west of the study area, the Boggabri Volcanics are present, which includes andesite, dacite, rhyolite, diorite and monzonite
- The study area (and the majority of Gunnedah) is underlain at shallower depths by recent Quaternary stream alluvial deposits, which include riverine plain deposits of sandy to silty clays and minor gravels.

2.4 Key findings from Golder desk study report

Key findings from the Golder desk study report were:

- There is limited geotechnical information pertaining to the study area
- The study area is recorded as having “no known occurrence” of acid sulfate soils (ASS), though is within an area of recorded high salinity and aggressive groundwater conditions
- Blackjack Creek is an engineering constraint, with an unknown depth to bedrock beneath the channel and soils potentially susceptible to settlement. Blackjack Creek is also an ephemeral creek with intermittent flows
- The constrained route corridor may limit the use of fill batters on approach embankments, and retaining walls may be required. Significant volumes of site won fill are not envisaged from construction, and imported fill for embankment construction may be required
- There is a risk of contaminated soils within the study area, due to historical filling at nearby light industrial/commercial sites, potential hydrocarbon contamination around two fuel depots and potential hydrocarbon, heavy metals, asbestos and other contamination associated with train operations along the rail corridor.

3.0 SCOPE OF GEOTECHNICAL INVESTIGATION

The scope for the intrusive phase of geotechnical investigation was developed in broad accordance with Golder proposal (reference P27622089 001-L-Rev0). However, amendments to the initially proposed scope were made to reflect findings of the desk study report, project team technical workshops, ongoing concept option development, stakeholder engagement, and ground conditions encountered as the investigation proceeded.

3.1 Field investigation

Fieldwork for the investigation was carried out between 22 July and 30 July, 2013, and comprised;

- Buried service clearance searches at proposed borehole locations
- Drilling of five machine rotary excavated boreholes, with in-situ testing
- Recovery of selected soil samples for subsequent geotechnical laboratory testing
- Recovery of rock core, with on-site strength testing
- Installation of two slotted standpipe groundwater monitoring wells
- Surveying of borehole locations.

Borehole locations are shown on Figure 2. A summary of borehole drilling depths is provided in Table 1 below, and borehole logs are provided in Appendix B.

Community and other stakeholder liaisons were managed by KBR.

3.1.1 Borehole drilling

The drilling schedule upon start of fieldwork was for six boreholes - two boreholes to 8 m depth and four boreholes to 15 m depth, the latter each with 3 m of ‘NMLC-type’ rock coring (ie using a 52 mm diameter diamond impregnated coring bit).

In consultation with KBR and Roads and Maritime, the schedule was revised as a result of encountering a greater than anticipated depth of soils above bedrock within the first borehole drilled (BH02). The revised schedule provided for boreholes to be drilled to greater depths and BH06 to be omitted from the programme.

All borehole locations were checked for the absence of buried services by a Golder and ARTC approved contractor before drilling. A representative from Gunnedah Shire Council also attended the site to check borehole BH03 was clear of a recorded nearby water main.

Drilling was carried out by North Coast Drilling Pty Ltd using a truck mounted drilling rig, with solid flight augers and wash-boring within soils, and NMLC diamond coring through bedrock.

Standard Penetration Tests (SPTs) were conducted regularly within soils, typically at 1.5 m or 3.0 m intervals, and recovered soil samples were logged on site. Two ‘push-tube’ (undisturbed) samples were collected.

Steel casing was advanced to the top of rock and used to support the bore through soil when rock coring. The recovered rock core was logged, photographed and point load strength tested on site by a Golder geotechnical engineer.

3.1.2 Environmental controls

Upon completion of drilling, three boreholes were finished using cuttings as initial backfill then grouted to surface level. Two boreholes were finished by installing a standpipe piezometer and Gatic-type cover. Drilling fluids and excess cuttings were removed from site and disposed of at a licensed facility.

3.1.3 Monitoring wells

Boreholes BH02 and BH05 were installed with 50 mm diameter slotted standpipe type piezometers, to enable groundwater monitoring and sampling in the future. Details of the well construction are summarised in Table 1 and included in Appendix B.

Table 1: Borehole Drilling Summary

Borehole ID	Total Depth Drilled (m bgl)	Depth of Rock Coring (m bgl)	Monitoring Well Response Zone (m bgl)
BH01	19.45	Rock not encountered	Not Installed
BH02	49.25	Rock not encountered	9.00 - 15.00
BH03	29.10	26.10 - 29.10	Not Installed
BH04	35.50	32.50 - 35.50	Not Installed
BH05	20.95	Rock not encountered	12.00 - 15.00

Note: m bgl = metres below ground level

3.1.4 Surveying

The position of each borehole was surveyed by Stewart Surveys Pty Ltd. using 'Real Time Kinematic' (RTK)/GNSS methods with a base station and receiver, to achieve an accuracy of about +/- 0.02 m.

The positions were recorded with reference to Map Grid Australia (MGA 94) and the reduced level with reference to Australian Height Datum (AHD). Surveyed borehole coordinates are provided in Table 2 below.

Table 2: Borehole Survey Coordinates

Borehole ID	Easting	Northing	Reduce Level (m RL)
BH01	236659.84	6569610.20	267.28
BH02	236696.16	6569680.82	265.55
BH03	236616.68	6569788.28	264.75
BH04	236660.29	6569816.92	265.98
BH05	236697.76	6569834.49	265.32

3.2 Laboratory testing

Laboratory testing was carried out on selected soil samples at National Association of Testing Authorities, Australia (NATA) accredited laboratories, and comprised:

- 18 No. combined moisture content, Atterberg limit and linear shrinkage tests on disturbed samples recovered from SPTs;
- 2 No. one-dimensional consolidation (oedometer) tests on undisturbed 'push-tube' samples; and,
- 2 No. multistage triaxial tests (U75 samples) with pore water pressure (PWP) measurement on undisturbed 'push-tube' samples.

4.0 RESULTS OF GEOTECHNICAL INVESTIGATION

4.1 Laboratory testing results

Laboratory issued testing certificates and results are presented in Appendix C of this report, while a summary of test results is presented in the following sections.

4.1.1 Soil classification

Eighteen (18 No.) soil samples were analysed by Resource Laboratories Pty Ltd for Atterberg limit and moisture content testing, with results shown in Table 3 below and plotted graphically in Appendix D.

Table 3: Atterberg Limit, Moisture Content and Linear Shrinkage Test Results

BH ID	Material (laboratory description)	Depth* (m bgl)	MC (%)	LL (%)	PL (%)	PI (%)	Linear Shrink-age (%)	USCS
BH01	SILTY CLAY, trace of sand, red-brown	1.23	11.1	39	10	29	7.5	CI
BH01	SILTY CLAY with sand, trace of gravel, pale brown mottled grey/ orange-brown/ black	7.23	23.0	43	11	32	14.5	CI
BH01	SILTY SANDY CLAY, orange-brown mottled pale brown/ grey/black	14.48	20.2	37	11	26	8.5	CI
BH02	SILTY CLAY with sand, trace of gravel, brown mottled red-brown/orange-brown	1.23	18.5	40	12	28	13.5	CI
BH02	CLAY, trace of sand and gravel, mottled pale brown/orange-brown/black	5.73	22.2	52	12	40	17.0	CH
BH02	SILTY CLAY with sand and gravel, mottled pale grey/ orange-brown	10.21	17.7	48	11	37	12.0	CI
BH02	CLAY with sand and gravel, mottled orange-brown/pale grey/black	19.23	23.3	53	13	40	12.5	CH
BH03	SILTY CLAY with sand, trace of gravel, brown mottled grey	2.73	20.4	39	10	29	13.5	CI
BH03	SILTY CLAY with sand, mottled pale grey/orange-brown	8.73	22.4	38	12	26	11.0	CI
BH03	SILTY CLAY with sand, orange-brown mottled pale grey/pale brown	11.73	22.5	39	11	28	12.5	CI
BH03	CLAY, trace of sand, pale grey mottled orange-brown/red-brown/yellow	22.23	28.5	55	13	42	18.5	CH
BH04	SILTY CLAY, trace of gravel, dark brown mottled brown	1.23	15.2	36	11	25	11.0	CI
BH04	SILTY CLAY with sand, trace of gravel, mottled brown/orange-brown/grey	4.23	21.7	38	11	27	13.5	CI
BH04	SILTY CLAY with sand, mottled pale grey/orange-brown	11.73	23.1	40	12	28	13.5	CI
BH04	SILTY CLAY with sand, mottled orange-brown/ brown/pale grey	25.23	22.5	42	12	30	12.5	CI
BH05	SILTY CLAY with gravel, trace of sand, mottled dark brown/brown	1.23	16.2	40	12	28	10.0	CI
BH05	SILTY CLAY with sand, brown mottled grey/ orange-brown	5.73	20.5	33	10	23	11.0	CL
BH05	SILTY CLAY with sand and gravel, mottled orange-brown/grey	10.23	17.6	43	12	31	16.5	CI

Note: * = Midpoint of sampling depth range. MC = Moisture Content. LL = Liquid Limit. PL = Plastic Limit. PI = Plasticity Index. USCS = Unified Soil Classification System.

4.1.2 Consolidation testing

Two (2 No.) soil samples were analysed by SGS Laboratories Pty Ltd for one-dimensional consolidation testing, with results summarised in Table 4 below.

Table 4: Consolidation (Oedometer) Test Results

BH ID	Depth (m bgl)	Material (laboratory description)	Load (kPa)	Cc	Cv (m^2/yr)		Mv (kPa^{-1} x 10^{-3})	Cα (x 10^{-3})	Degree of consolidation (%)
					t50	t90			
BH01	2.50 – 2.85	Sandy Silty CLAY: brown	6-50	0.073	0.72	0.94	0.824	3.21	5.0
			50-100	0.157	0.41	0.56	0.528	3.57	7.5
			100-200	0.201	0.39	0.52	0.347	2.51	10.7
			200-400	0.199	0.52	0.61	0.178	2.78	13.9
			400-800	0.207	0.48	0.60	0.096	2.95	17.2
			800-400	0.020	-	-	0.010	-	16.9
			400-50	0.025	-	-	0.041	-	15.7
BH03	1.00 – 1.40	Sandy Silty CLAY: brown	6-50	0.092	0.71	0.92	1.1221	2.33	5.9
			50-100	0.103	0.53	0.75	0.416	0.90	7.8
			100-200	0.119	0.66	0.99	0.246	1.56	10.1
			200-400	0.120	1.44	2.27	0.127	1.53	12.4
			400-800	0.133	1.27	1.64	0.072	1.67	14.9
			800-400	0.011	-	-	0.006	-	14.7
			400-50	0.007	-	-	0.013	-	14.3

Note: Cc = Compressibility index. Cv = Coefficient of consolidation. Mv = Coefficient of volume compressibility. Cα = Creep index.

4.1.3 Multistage triaxial testing

Two (2 No.) undisturbed soil samples were subject to multistage unconsolidated undrained triaxial tests with pore water pressure measurement by SGS Laboratories Pty Ltd., with results summarised in Table 5 below.

Table 5: Multi-Stage Triaxial Test Results

BH ID	Depth (m bgl)	Material (laboratory Description)	Confining Pressures (kPa)	MC (%)	Friction Angle ø (°)	Cohesion c (k Pa)
BH01	2.50 – 2.85	Sandy Silty CLAY: brown	600 750 950	24.6	29.6	6.9
BH03	1.00 – 1.40	Sandy Silty CLAY: brown	600 750 950	15.5	33.7	3.6

4.1.4 Rock strength testing

Seven (7 No.) samples of rock core retrieved from boreholes BH03 and BH04, were selected for point load index testing. A total of twelve (12 No.) individual tests were performed, with both diametral and axial orientated performed on each sample, with the exception of two samples where testing on both axes was not feasible.

The point load test results are summarised in Table 6 below.

Table 6: Point Load Test Results

BH ID	Sample Depth (m bgl)	Rock Type	Sample Test Orientation	Point Load Strength Index Is(50) (MPa)	Equivalent UCS (MPa)	Strength Description
BH03	26.95	Rhyolitic Tuff	Diametral	0.49	12	Medium
	27.95	Rhyolitic Tuf	Diametral	5.50	137	Very High
			Axial	5.04	126	Very High
	28.85	Rhyolitic Tuff	Diametral	6.43	161	Very High
			Axial	4.09	102	Very High
BH04	32.95	Ashflow Tuff	Diametral	2.05	51	High
			Axial	2.25	56	High
	33.95	Ashflow Tuff	Diametral	0.78	19	Medium
			Axial	1.56	39	High
	34.95	Ashflow Tuff	Axial	1.94	48	High
	35.45	Ashflow Tuff	Diametral	2.13	53	High
			Axial	2.63	66	High

Note: * = Point Load Strength Index (Is(50)) to UCS conversion factor of 25 applied, after Hoek, 2007.

4.2 Field testing

4.2.1 Standard penetration test (SPT)

Standard Penetration Tests were carried out within all boreholes at depth intervals of about 1.5 m at shallow depths (<16 m), and at 3.0 m intervals at greater depths (>16 m). SPT results are presented on the borehole logs, in Appendix B, and have also been summarised as a graphical plot in Appendix D.

SPT results show a general trend of increasing density/soil stiffness with depth to about 10 m depth below ground level, beneath which a results tend to be within a more constant range.

At depths less than 4 m below ground level, the SPT results also showed less dense/less stiff soils located to the north of the rail line than to the south. This is detailed in section 6.1 of this report.

Fifteen SPTs, out of a total of 75 carried out, reached effective refusal (ie blow counts exceeding 50 without a full 300 mm test depth penetration having been achieved). Examination of the soil recovered from the SPT tip from these tests, and subsequent boring, indicated these were generally carried out within gravel bands.

4.3 Groundwater observations

The method of drilling using mud below the upper few metres precluded detailed observation of groundwater. However, groundwater was noted at shallow depth (1.0 m bgl) within borehole BH03 during drilling.

Groundwater levels were subsequently measured in the two standpipes. BH02 was measured 5 days after completion of drilling, while BH05 was measured one day after completion of borehole drilling. Groundwater observations are presented in Table 7 below.

Table 7: Groundwater Observations

Borehole ID	Groundwater Level Observed During Drilling (m bgl)	Groundwater Level Measured in Piezometer (m bgl)
BH01	Groundwater not observed	Standpipe not installed
BH02	Groundwater not observed	1.82 m at 4:30pm, 29/7/2013
BH03	1.00 m	Standpipe not installed
BH04	Groundwater not observed	Standpipe not installed
BH05	Groundwater not observed	2.22 m at 7:00am, 30/7/2013

5.0 SUBSURFACE CONDITIONS

Subsurface conditions encountered in boreholes comprised localised fill and topsoil over a thick sequence of undifferentiated alluvial and residual soils. Some of the boreholes encountered weathered volcanic rock.

Fill and topsoil was encountered to shallow depths (ie <2.0 m). A greater thickness of fill (up to 2 m) was encountered north of the rail line in boreholes BH04 and BH05.

Underlying the fill/topsoil consisted of a sequence of inferred alluvial and residual soils, comprising; clay, silty clay, sandy silt, and clayey sand. Some intermittent subordinate gravelly clay and sandy gravel lenses were observed, up to 3.4 m in thickness. It was difficult to distinguish between alluvial and residual soils, due to their similar characteristics.

Cohesive soils demonstrated a general increase in stiffness with depth, generally being stiff to about 5 m depth, then becoming very stiff to hard. However, north of the railway line, in boreholes BH04 and BH05 at depths less than 4 m, cohesive soils were soft to firm.

Boreholes BH01 and BH05 were terminated at 20 m bgl before encountering bedrock. Borehole BH02, located proximal to the proposed central span of the Option A alignment, was terminated at 49.25 m bgl. This borehole was terminated within the sequence of alluvial and residual soils, having not encountered bedrock.

Bedrock, comprising two distinguishable types of volcanic tuff material, was encountered within boreholes BH03 and BH04 at 26.10 and 32.50 m bgl, respectively. These boreholes are at the south and north ends of the proposed central span of the Option B and C alignments, at the railway crossing. The rock strength was typically high to very high.

5.1 Geotechnical units

The subsurface conditions encountered have been characterised into geotechnical units and are summarised in Table 8 below.

Table 8: Inferred Geotechnical Units

Unit ID	Unit Name	Depth Range (m bgl)	Maximum Thickness (m)	Typical Material Description
1	Topsoil/Fill	0.00 – 2.00	2.00	FILL: sandy Gravel / silty Clay / gravelly Clay of variable consistency
2A	Alluvium (clay, silt and clayey-sand)	0.35 – 48.00	48.00*	Medium to high plasticity gravelly CLAY, brown, with varying composition of sand and sub-rounded gravel. Typically stiff to very stiff, but soft to firm in places.
2B	Alluvium (Gravel)	4.80 - 8.20 6.80 - 7.30 48.00 - 48.95	3.40	Clayey sandy GRAVEL, orange brown, with gravel being sub-rounded to sub-angular and fine to coarse. Typically dense to very dense.
3	Residual Soil	24.50 – 25.15 31.00 – 32.50	1.50	Medium to high plasticity silty CLAY to sandy clay, orange brown mottled grey, with bands of sub-angular gravel. Typically very stiff to hard.
4A	Bedrock (Rhyolitic Tuff)	26.10+	3.00+	Rhyolitic welded Tuff - fine to medium grained, yellow brown and pale red, quartz rich, with some coarse rounded phenocrysts, and some sub-horizontal to sub-vertical clay seams 3 to 30mm thick.
4B	Bedrock (Ashflow Tuff)	32.50+	3.00+	Ashflow Tuff - fine to medium grained, flow banded, pale grey with brown, rhyolitic, banded at 0-20°, with some dark grey staining and clay seams.

* excludes thickness of subordinate alluvial gravel layer within sequence.

5.2 Geotechnical cross-section

An inferred geotechnical cross section has been prepared, along the alignment of proposed Concept Options B and C, and is included as Figure 3.

5.3 Infrastructure services

Golder is aware of several recorded services located along the proposed route alignments. These include, but may not be limited to:

- Electricity (Essential Energy) - buried and overhead services;
- Telecommunications (Telstra);
- Water (shown on Gunnedah Shire Council proposed construction drawings provided by KBR):
  - A new 'Rising Main'; and,
  - 'Water Main'.

In addition to the recorded buried services, an artificially aligned and open drainage channel is present in the southern half of the study area, running roughly southeast to northwest. From a visual observation it appears about 2 m in depth and 4 m in width and is concrete lined.

The channel is marked as 'Ashfords Water Course' on the Gunnedah Shire Council water main construction drawings, and is the western part of a drainage channel which runs about parallel to and to the south of the rail line, around the south side of central Gunnedah.

6.0 GROUND ENGINEERING DISCUSSION

6.1 Approach embankments/bridge abutments

The concept options propose various combinations of structural bridge spans and earth approach embankments with piled abutments up to about 4.0 m in height. Embankments will need to be designed to meet the requirements of Roads and Maritime Specification R44.

For bridge approach embankments/abutments, Reinforced Soil Wall (RSW) structures have been proposed to limit the embankment footprint. Detailed design of RSW structures will need to be in accordance with Roads and Maritime Specification R57.

It is not anticipated significant volumes of fill will be generated during construction along the route corridor. Engineered fill material will therefore need to be imported to the site for embankment and RSW construction.

For Option A, the approach embankments/abutments would be founded on ground presently beneath existing pavement. No direct investigation of the pavement sub-formation or soils beneath it has been carried out as part of this study, although borehole BH02 was drilled next to the centre of this alignment.

Option A would need to consider incorporation of the existing culvert, or construction of a new culvert (Ashfords Water Course) at its southern end.

For Options B and C, the near surface soils to the north and south of the rail line are different, with thicker fill and lower strength soils to the north of the rail line compared to those along the alignment to the south. Further discussion of geotechnical aspects of Options B and C is provided below.

North of the Rail Line – Options B & C

To the north of the rail line, between the rail line and the southern end of the caravan park on Warrabungle Street, the soils comprise up to 2 m of variable fill over soft to firm clay to about 4 m depth. The fill and soft to firm clay in their current condition are unsuitable as a foundation subgrade for the proposed embankments. This is because there is the potential for significant total and differential settlements under the embankment loads which could adversely impact the operational performance of the future road pavement. Further analyses will be required to quantify the likely magnitude of settlements.

At this stage we anticipate the fill and soft to firm soils will need to be improved so they can support the embankment loads and so long-term settlements are maintained within operational limits. Ground improvement techniques could be considered include preloading, soil-mixing, vibro-stone columns or concrete modulus columns. These techniques would need to improve the soils to about 4 m depth.

Similarly, the existing soils north of the rail line are unsuitable for supporting RSWs on shallow footings. The ground improvement could also be aimed at improving the foundation conditions so the RSWs can be supported on shallow footings. Alternatively, short piled footings extending into the stiff and very stiff clay at greater than 4 m depth may be needed depending on the magnitude of the RSW loads and the settlement performance criteria for the walls.

Further geotechnical investigation would also be needed to provide greater certainty about the extent of the fill and soft to firm soils, which is not reliably defined from the current limited investigation.

South of the Rail Line – Options B & C

Shallow ground conditions within 4 m depth are generally more favourable to the south of the rail line. Embankment subgrade preparation is expected to be routine and involve stripping of topsoil and localised removal of fill to the top of the underlying stiff clay. The subgrade would then need to be proof-rolled to check for the presence of soft spots before tining, moisture conditioning and re-compaction before commencing embankment construction.

Similarly, the shallow stiff clays are expected to be a suitable foundation layer for shallow footings to support the RSWs. Footings for RSWs founded on the stiff clays may be designed for an allowable bearing capacity of 100 kPa.

Other Construction Considerations – Options B & C

Both north and south of the rail line, Options B and C run close to Blackjack Creek. Consideration will need to be given to the impact the creek may have on the stability of embankments and RSWs, particularly during flood events. Embankments and RSWs which are likely to be inundated will need to be designed and constructed to mitigate flood impacts, which could cause adverse erosion, rapid drawdown effects or could undermine RSWs and the toe of embankments.

The road will also need to be designed so it does not adversely impact the stability and erodibility of the creek banks.

6.2 Bridge foundations

The presence of low strength soils and uncontrolled fill near the surface, and the expected high magnitude of bridge loadings are likely to preclude the use of shallow footings for the bridge structures. The bridges are likely to be founded on piles, of which there are several alternatives types and configurations which are practically feasible, including piles relying on end-bearing on rock, or floating piles in soil.

Pile solutions which extend through the sequence of alluvial and residual soils to found on rock offer the most robust solution. At the railway crossing, for Options B and C, rock is known to be at about 24 to 32 m depth. However, elsewhere along the alignment the depth to rock is not known, and in the limited number of boreholes drilled, was in excess of 20 m deep and greater than 49 m deep in BH02 along Option A alignment.

Driven or bored piles extending to rock for Options B and C are feasible, each with their benefits and limitations (see Table 9 below).

A variety of driven pile types are available, most commonly 350 mm or 450 mm wide square pre-cast concrete piles, or steel tube piles. Multiple pre-cast concrete piles, tied together at the surface with a pile cap, would be required to carry the anticipated loads. The ultimate axial capacity of individual pre-cast concrete piles is governed by their structural capacity, which is about 1,400 to 1,700 kN for the piles sizes discussed here. Detailed analyses would be required to design the pile layout to accommodate both the vertical and lateral loads.

Steel tube piles, of say 1 m diameter, could also be driven to rock. These are likely to be able to carry larger axial loads than individual pre-cast piles, and hence fewer piles would be needed. To accommodate lateral loads the steel tubes would most likely need to be mucked out in say the upper 10 m and in-filled with reinforced concrete. This depth could vary due to other factors including durability and scour.

Bored piles could be drilled at relatively large diameters, say 1.8 m diameter, with the objective of reducing the number of piles. It may also be possible to tie these piles directly into the headstock, negating the need for a pile cap. A potential limitation of bored piles at this site is the presence of shallow groundwater, which could yield relatively high inflows in the more gravelly layers. For this reason, we would expect bored piles would need to be drilled using heavy mud (bentonite) to stabilise the hole, or temporary steel casing. The rock encountered in the boreholes is of high to very high strength and suitable rock drilling equipment would be needed if the piles are to be socketed into rock.

Some of the limitations of bored piles could be overcome using continuous flight auger piles (CFA). There have been improvements to the capability of CFA piling methods in recent years, and the Roads and Maritime has revised its position to permit the use of CFA piles on a limited basis. Roads and Maritime Bridge Technical Direction BTD2011/02 outlines the application and limitations of CFA piles, and Roads and Maritime Specification B63 covers the construction of CFA piles.

Floating piles have the benefit of reduced pile length, and may be practical and economically feasible where rock is relatively deep, and may be the only practical solution for much of the Option A alignment. A floating pile solution could use driven or bored piles, although the presence of gravel layers increases the risk of adverse load-displacement outcomes for bored piles. Bored piles could be installed with enlarged bases to increase basal resistance. A detailed pile analysis would be required to investigate alternative floating pile concepts.

There are many other pile types and configurations which could be considered. Piling contractors would be best qualified to propose a solution which balances performance objectives with practical feasibility and economy.

Regardless, once a preferred design is adopted we recommend further ground investigation at bridge pier and abutment locations to confirm ground conditions along the alignment and reduce uncertainty for contractors pricing the work. These additional investigations should be scoped in accordance with the requirements of a project specific Roads and Maritime scope of work and technical criteria.

The following table provides summary comment on some of the pile types, respective to the ground conditions encountered, which may be considered:

Table 9: Pile Types - Summary Comments

Pile Type	Comment
Bored Piles	■ Bored piles, either acting as floating piles or end-bearing, may be feasible ■ Fewer piles in a 'pile-to-pier' arrangement may be possible compared to other solutions using a pile cap ■ Disadvantages are they will need temporary or permanent casing, or bentonite slurry, to support the bore against collapse and 'necking', due to sandy deposits and groundwater inflows. Larger diameter piles may help mitigate groundwater inflow issues. End bearing on disturbed gravels may be unreliable ■ Disposal of soil cuttings from boring would be required ■ Boring to create a rock-socket in the underlying volcanic tuff may be difficult due to the high strength of the rock.
Driven Piles	■ Various driven pile types are considered feasible, as either floating or end-bearing piles. Concrete square section piles are readily available and their installation well understood ■ Steel tube piles may be considered, although their use is likely to be limited to an end bearing solution, due to the uncertainty in demonstrating skin friction capacity ■ Driven piles minimise generation of cuttings, may be installed relatively quickly, and can be fabricated off-site which may reduce construction plant and working areas required. ■ Dense/very stiff-hard layers, with high gravel contents, may prevent piles being driven to target depths as required for lateral capacity (considering scour). Pre-boring of piles may mitigate this issue, but would add to the time and cost of construction ■ The environmental impact from noise and vibrations will need to be evaluated, although the former can be reduced with the use of a shroud ■ Deeper piles would need to be spliced, which for concrete piles especially creates a potential weak point and may limit lateral capacity. Steel pile sections are typically welded together.

Continuous Flight Auger (CFA) piles	■ CFA piles may be favoured due to their relative speed of installation and lower noise and vibration effects. They may be end-bearing, floating, or a combination of both ■ Acceptance may be an issue, based on historic concerns over quality control and structural integrity ■ A principal challenge to the feasible use of CFA piles, if relying on a significant end-bearing component, is the depth to bedrock. Whilst CFA rigs can typically install to 40 m depth or greater, Roads and Maritime require the use of single continuous auger for pile construction, which is uncommon beyond about 20 m length. The depth to bedrock should be further investigated and assessed ■ CFA piles are typically suited to the ground conditions encountered on the site, although further site investigation should be carried out to assess the likelihood of granular layers being present, which could potentially cause 'draw-in' (the granular layers encountered in the site investigation generally had high cohesive contents which would reduce or prevent this effect) ■ CFA piles may struggle to penetrate the relatively hard volcanic bedrock, and pile designed may need to consider a much reduced proportion of the end-bearing pile capacity component. The slope of the bedrock should also be further investigated and assessed, as end-bearing CFA piles are limited to founding on a bedrock gradient of 1 vertical to 4 horizontal ■ Disposal of spoil from the pile construction process will be required.

6.2.1 Soil and groundwater aggressivity

The study area is within an area of recorded high salinity and aggressive groundwater conditions (see Golder desk study report). Development and sampling of groundwater from the two installed piezometer standpipes, for subsequent groundwater aggressivity testing, is advised.

The results from such analysis should be used to assess whether aggressive ground conditions are likely and provide an indication if more durable steel or concrete foundation materials are required.

6.3 Earthworks

6.3.1 Excavatability

Excavations should be readily achievable using conventional tracked excavator plant.

Groundwater ingress is likely at shallow depths (groundwater observed at about 1.0 m depth in BH03), and excavations may require temporary dewatering, eg with sump pumps. The anticipated groundwater level should be reassessed from further groundwater monitoring.

6.3.2 Batter slopes

The stability of temporary batter slopes and excavations may be variable, due to the potential variation in soil composition and groundwater ingress. Shallow angle slopes no steeper than 1.5 H : 1 V should be adopted for temporary excavations, or trench support (eg trench boxes) be utilised. Significant permanent cut slopes are not considered within the current concept options.

Embankment batters may be designed to be no steeper than 2 H : 1 V for concept design purposes. These would need to be assessed further during detailed design by undertaking limit equilibrium stability analyses for a range of load cases including static, earthquake and rapid drawdown.

6.3.3 Re-use of excavated soil

Soil from shallow excavations is likely to be variable in composition and organic content (ie topsoil), and hence may not be suitable for use as engineered fill.

If considered for re-use, cohesive soils may require moisture conditioning, either drying out or wetting up to be within acceptable limits, as confirmed from compaction testing. Naturally occurring granular soils are likely to be suitable for re-use as engineered fill without treatment. Use of silt rich material should be avoided.

Spoil from excavation, including topsoil, should be suitable as filling for landscaping purposes.

6.4 Contamination

Further to the assessment made within the Golder desk study report, no additional indicators of contamination issues were identified during the intrusive investigation.

A detailed contamination assessment should be required as part of detailed design assessment work to characterise existing fill materials and facilitate classification of spoil for potential off site disposal.

6.5 Concept option alignments - geotechnical appraisal

The following discussion provides comments on the key geotechnical issues affecting the respective design concepts. Options B and C have been addressed together, as the alignments are similar.

6.5.1 Option A – New Street Alignment

Key geotechnical issues for Option A include;

- Potential settlement of earth approach embankments and abutment RSWs. Some ground improvement may be required.
- Increased loading of the culvert at the southern end of New Street. Detailed assessment of this structure would need to be carried out.
- Significant depth to bedrock (BH02 drilled to 49.25 m without encountering bedrock), which may preclude the economic use of end-bearing piles founded on rock.
- Piling work close to residential properties, with associated noise and vibration issues.
- Constrained working areas, especially for construction of approach embankment RSW at northern end of the alignment.
- Loading of potentially compressible soils within the eastern spur of Blackjack Creek at the southern end of the alignment, as part of the roundabout reconfiguration. Ground improvement may be required and scour protection should be considered.

6.5.2 Option B and Option C – West of Gunnedah Maize Mill

- Option B only: increased loading of the culvert at the southern end of New Street. Detailed assessment of this structure would need to be carried out
- Route alignments conflict with several major buried services easements, as well as the ‘Ashfords Water Course’ drainage channel. Service relocation or protection will be required. Localised realignment of the drainage channel may be needed
- Significant depth to bedrock (bedrock not proven away from central span over rail line, where bedrock was encountered circa 30 m below ground level). May preclude the economic use of end-bearing piles founded on rock. Further ground investigation should be carried out to support detailed design
- Piling work close to residential properties at northern end of alignment, with associated noise and vibration issues
- Potential settlement of earth approach embankments and abutment RSWs. Some ground improvement may be required, particularly north of the rail line, which could impact on the construction programme, particularly if preloading is used.

- Construction close to Blackjack Creek;
  - Loading of potentially compressible soils. Ground improvement may be required and scour protection should be considered
  - Creek embankment stability
  - Stability and founding level of RSWs
  - Construction access - working from creek bed.

7.0 CONCLUSION & RECOMMENDATIONS

The following provides a summary of key findings and recommendations for further work.

- Five boreholes were drilled within the study area to depths ranging from 20 m to 49.25 m bgl. Three boreholes were backfilled and grouted, and two were finished with groundwater wells (BH2 and BH5). Selected soil and rock samples were subject to geotechnical testing
- Subsurface conditions encountered comprised localised fill and topsoil, over a thick sequence of undifferentiated alluvial and residual cohesive soils with subordinate gravel layers, in turn over volcanic bedrock
  - Alluvial soils were typically stiff to very stiff, but soft to firm in places. Residual soils were typically very stiff to hard
  - Bedrock, comprising variably weathered tuff, was encountered in two boreholes at 26.10 and 32.50 m bgl, and was typically high to very high in strength.
- Key geotechnical issues for the proposed three concept options, namely Option A (New Street alignment) and Option B and Option C (alignments to the west of Gunnedah Maize Mill), include:
  - Low strength soils and uncontrolled fill near the surface, notably at the northern abutment of Option B and Option C (Refined), and the expected high magnitude of bridge loadings are likely to preclude the use of shallow footings for the bridge structures
  - Unproven depth to bedrock for the majority of the study area, which may be in excess of 50 m depth bgl. Bedrock was only encountered proximal to the rail line along the Option B and Option C alignments. The depth to bedrock may preclude the economic use of end bearing piles founded on bedrock, and floating piles may be more economical
  - Reinforced soil walls could be considered to reduce the footprint of the bridge approach earthworks
  - Ground improvement may be required beneath embankments and/or reinforced soil walls, particularly north of the rail line on Option B and C alignments
  - Excavations are likely to be possible using conventional tracked plant. Some groundwater ingress may be encountered from depths below 1 to 2 m bgl, and temporary dewatering may be required. It is anticipated only a limited amount of site-won fill will be available for re-use as engineered fill, noting also the strict material requirements for RSW fill (Roads and Maritime R57 specification)
  - Detailed design will need to consider implications of construction next to Blackjack Creek, where issues include creek bank instability, scour and flooding.
- To develop the design further, supplemental investigation work should be carried out as follows:
  - To better understand the distribution and extent of poor subgrade soils along the alignment, so ground improvement can be designed, where required. Excavator test pits are recommended for this purpose, along with Cone Penetration Testing
  - To refine pile solutions for the bridge structures. This will require additional boreholes extending to rock followed by value engineering of alternatives, and detailed pile design analyses. Subsequently, non-conforming foundation systems could be considered from experienced piling contractors with appropriate transfer of performance risk
  - To confirm groundwater levels and aggressivity. This will require groundwater level monitoring and sampling for laboratory analysis.

Report Signature Page

GOLDER ASSOCIATES PTY LTD

Chris Roll
Senior Geotechnical Engineer

Phil Davies
Principal Geotechnical Engineer

CLR-NI/PRED/clr-ni

A.B.N. 64 006 107 857

j:\geo\2012\127622030_kbr_bridge si_gunnedah\correspondence out\127622030-r-008-rev2-geotechnical investigation report.docx

FIGURES

APPENDIX A

KBR Option Alignment Sketches

APPENDIX B

Borehole Logs

APPENDIX C

Laboratory Test Results

APPENDIX D

Plasticity Chart and Standard Penetration Test Plot

Africa	+ 27 11 254 4800
Asia	+ 86 21 6258 5522
Australasia	+ 61 3 8862 3500
Europe	+ 356 21 42 30 20
North America	+ 1 800 275 3281
South America	+ 56 2 2616 2000

solutions@golder.com
www.golder.com

Golder Associates Pty Ltd
124 Pacific Highway
St. Leonards, New South Wales 2065
Australia
T: +61 2 9478 3900

Golder Associates