



Douglas Partners

Geotechnics • Environment • Groundwater

Integrated Practical Solutions

INTERPRETATIVE REPORT

on

GEOTECHNICAL INVESTIGATION

(VOLUME 4)

CONCEPT DESIGN PHASE ROUTE SURVEY

DIVERSION OF SALISBURY ROAD AROUND

PROPOSED TILLEGRA DAM

Prepared for

OPUS INTERNATIONAL CONSULTANTS (NSW)

PTY LTD

On behalf of

HUNTER WATER CORPORATION

Project 39721.01

NOVEMBER 2008



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EXECUTIVE SUMMARY

This report presents the results of a geotechnical investigation for the concept design phase route survey for the diversion of Salisbury Road, as part of the proposed Tillegra Dam project.

Diversion of Salisbury Road around the proposed site of Tillegra Dam and reservoir will involve a deviation along the north-trending ridge to the north of the dam site, and will involve extensive cutting across ridgelines and spurs, and filling of incised gullies. The Tillegra dam site is situated within the Gresford Block of the Hunter-Myall Region, which is characterised by complex structural geology. Published geological data indicate that the region has undergone significant deformation in the form of extensive folding and faulting.

The purpose of the investigation was to provide advice on excavation conditions, batter slopes for cuts and fills, slope stability, and design parameters for bridge footing systems. Also included in this phase of route design was an assessment of the suitability of locally sourced gravels for use in pavement construction. In addition to the desktop study and walkover assessment of the proposed route, the investigation included drilling a total of 87 boreholes (21 of which involved diamond coring of rock) excavation of 35 test pits, field and laboratory testing, and engineering analysis. Recommendations have been made in this report for further field work involving test pits, cored boreholes, downhole camera observations and seismic refraction surveys, each of which were in progress at the time of reporting for the concept design phase investigation.

The proposed E2 alignment (16.8 km in length) requires the excavation of approximately 39 cuttings with excavation depths typically in the order of 10 m to 20 m, and fill embankments in the order of up to 20 m. The majority of cuts could be made using medium to heavy ripping, and it is likely that blasting will also be required along the route, particularly between approximate Ch 3000 and Ch 9000 and in the deepest of the excavations. A more rigorous assessment of rippability/excavatability is required during the detailed design stage of the investigations.

The road alignment will pass through areas of steep terrain, some sections of which will be subject to a higher risk of instability than others. In particular, steep slopes and colluvial soils encountered in the 'Williams Range' Soil Landscape grouping are likely to be predisposed to slope instability, and will present localised mass movement hazards. Evidence of instability

within surficial soils and colluvium was observed on slopes above several steep gullies across which the proposed route passes. Areas of instability involving shallow soil profiles are evident at a number of such locations within the southern half of the proposed route alignment. The mitigation of the risk of slope movement hazards in each of the areas of potential instability will be addressed by specific design measures during the detailed design phase.

Zones of fracturing within weathered siltstone of the Bonnington Siltstone Formation are expected throughout the proposed cuttings between approximate Ch 9200 and Ch 11600. More fractured zones within the rock strata represent a higher risk of erosion which may undercut overlying stronger layers. These zones will likely require treatment with shotcrete or similar at the time of construction. Additional work including assessments of joint orientation by downhole geophysical methods is to be undertaken during detailed design. It is also recommended that laboratory testing be undertaken to assess the potential for rapid weathering of rocks, as this may affect batter slope design and/or construction treatment.

The proposed road diversion will cross the Williams River at two locations, requiring the construction of two bridges, at approximate Ch 1100 and Ch 16220, referred to as the “lower crossing” and the “upper crossing”, respectively. It is anticipated that fill embankments up to about 2 m in height will be required for the lower crossing, and up to about 6 m in height for the upper crossing. Preliminary settlement estimates are presented in this report for the proposed bridge approaches, based on limited data from the boreholes and test pits. Suitable pile types, including driven piles and cased bored piles, approximate pile capacities, and recommendations on allowable shaft adhesion and allowable end bearing pressures are also presented in this report. Douglas Partners recommends that further assessment of possible soft soil profiles at the upper crossing site be made for the detailed design phase.

Four test pits were excavated in selected disused quarry pit locations to undertake a preliminary assessment of potential material sources for pavement construction. The findings of the preliminary work indicate that existing quarries within the reservoir area are unlikely to provide rock of a quality suitable for use as pavement gravel. If sourcing of pavement materials from near the project alignment is to be considered further, then significant additional geotechnical field and laboratory testing will be required to assess material properties in more detail.



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Project No: 39721.01

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21 November 2008

**INTERPRETATIVE REPORT ON
GEOTECHNICAL INVESTIGATION
CONCEPT DESIGN PHASE ROUTE SURVEY
DIVERSION OF SALISBURY ROAD AROUND PROPOSED TILLEGRA DAM**

1. INTRODUCTION

This report presents findings and interpretation of the results of geotechnical investigations for the concept design phase route survey for the diversion of Salisbury Road around the site of the proposed Tillegra Dam on the Williams River. The investigation was carried out by Douglas Partners Pty Ltd at the request of Opus International Consultants (NSW) Pty Ltd, acting on behalf of Hunter Water Corporation (HWC).

The purpose of the concept phase geotechnical investigation was to assess the following elements of route selection:

- Slope stability;
- Safe batter slopes;
- Depth to and excavatability of rock within proposed areas of cut;
- Suitability of excavated material for re-use in road formation works;
- Sources of potential pavement construction materials;
- Founding conditions at proposed bridge sites.

The results of the investigation have been separated into four volumes, as follows:

- Volume 1 – Data Report;
- Volume 2 – Appendices A to D (including engineering logs and results of field and laboratory testing);
- Volume 3 – Drawings 1 to 6;
- Volume 4 – Interpretative Report.

This report presents the results of geotechnical analysis and interpretation of the factual data presented in Volumes 1 to 3.

The scope of work for this investigation consisted of the following components:

- Desktop study and review of available background information, including data from the NSW Department of Commerce;
- Walkover assessment of route corridor;
- Excavation of 35 test pits;
- Drilling of 87 boreholes, including 21 boreholes with diamond coring of rock;
- Field and laboratory testing;
- Preparation of this report.

The results of the investigation are presented in the following sections of this report.

At the time the field work was undertaken, the preferred route was referred to as the Q21A4 option. Since then, some changes to the mid sections of the alignment have resulted in the Q21A4 option being superseded by the E2 option. This report covers the work undertaken for the E2 alignment option.

Field work for the detailed design phase of the project commenced immediately on completion of the concept design phase field work. This Concept Design Phase Interpretative Report is based on the results of field work and laboratory testing carried out during the concept phase of design. Recommendations contained in this report regarding further investigations have been adopted

by Opus on behalf of HWC. The field and laboratory testing and interpretation of the results of further investigations are in progress.

2. PROPOSED DEVELOPMENT

Construction of the proposed Tillegra Dam will result in the closure of Salisbury Road at the Tillegra Bridge crossing of the Williams River in the locality of Munni.

A preferred corridor for the diversion of Salisbury Road has been selected by Hunter Water Corporation (HWC) on the basis of an initial route selection study by GHD Pty Ltd. The final route selection study undertaken by Opus presented a recommendation on selection of an alignment corridor that uses the ridge forming the eastern margin of the proposed reservoir. The route selected by HWC was then subjected to adjustments and refinement by Opus during the concept design phase of the project.

The preferred road corridor leaves the existing Salisbury Road alignment approximately 1.5 km east of Tillegra Bridge (the dam site). It crosses the Williams River before rising approximately 150 m over a distance of about 5 km to the top of the ridge which forms the eastern rim of the proposed reservoir. The preferred corridor follows the top of the ridge for about 2 km before descending its western side, running over the lower slopes, before crossing Moolee Creek where it intersects Chichester Road, crossing the Williams River a second time and rejoining Salisbury Road.

At the time of preparation of the Draft Data Report for the concept phase design stage of the project, the preferred alignment was referred to as the Q21A4 option. Since then, however, a revised preferred option, referred to as the E2 option, has been developed. Significant portions of the Q21A4 option and the E2 option overlap, and some deviations along the middle portion of the route result in changes to the chainage numbering system for approximately half of the alignment. This report primarily addresses how the test data collected to date relate to the E2 alignment option, which has a total length of 16.8 km.

Preliminary road alignments have indicated numerous sections of cut and fill with vertical dimensions in excess of 10 m varying up to a maximum of 16 m, and cut volumes in the order of 3 million m³ at the time of the route selection study by GHD. The significantly refined design by Opus indicates that proposed cut volumes are now in the order of 1 million m³.

The approximate extent of the preferred route corridor is indicated on Drawing 1 in Volume 3 (E2 option alignment).

3. COMMENTS – EARTHWORKS

3.1 General

Bulk earthworks for the project will require significant cutting and filling. Based on the concept design data available for the E2 alignment, excavations in the order of up to about 15 m, and fill embankments in the order of up to about 17 m could be required along the proposed route centreline, with deeper cuts and fills in some cross-sections. Based on results obtained from a limited number of test locations, it is possible that some excavations could be achieved using medium to heavy ripping. Blasting is also likely to be required along some portions of the route and in deeper and confined excavations.

The following sections present preliminary comments regarding the proposed excavations and embankments.

3.2 Erosion and Dispersion

Dispersion is the potential for a clay soil to break down in contact with water and form a cloudy colloidal suspension. The suspension contains clay particles that are much finer than silt and thus dispersible soils greatly limit water movement through the soil, resulting in poor drainage and waterlogging. The sodicity of a soil relates to the likely dispersion of the soil on wetting.

The Emerson aggregate class (EAC) is used as a general guide to sodicity and depressiveness of a soil. However dispersion is also influenced by factors such as soil type, exchangeable cations, salinity and sodicity. Based on the results of preliminary laboratory EAC testing presented in Table 7.1 of the Data Report (Volume 1) for the Concept Design Phase of the project, the majority of the site soils tested to date are unlikely to be dispersive, with the exception of samples tested from pit locations 101A, 102A, 103A, 206B, 219C and 222A. Based on published soil landscape data discussed in Section 3.3 of the Data Report (Volume 1), these soils are characterised by high erodibility potential and high susceptibility to tunnelling or piping failure. The sandstone and mudstone (clayey silt and gravel) from Pit 230, which was pre-treated by weathering and compaction to simulate mechanical breakdown during earthworks, may also be dispersive based on Emerson aggregate class.

It is recommended that further testing be undertaken during the detailed design phase to confirm the advice presented in this preliminary assessment. In particular, the dispersive nature of soil and weathered rock profiles within the major cuttings along the preferred alignment should be assessed with regard to their suitability for reuse and requirements for treatment. Further testing may include a full soil erodibility suite (including k factor, particle size distribution and organic carbon content), in addition to further Emerson aggregate, pH, and electrical conductivity (EC) testing.

As discussed above, the majority of the soils tested for the preliminary concept phase assessment are unlikely to be dispersive based only on Emerson aggregate class numbers. However, these soils have high clay and silt fractions, with relatively high liquid limits in clays, and the majority of the soils were dry of optimum moisture content. Unless further testing proves otherwise, Douglas Partners recommends that all site clays, weathered siltstones and claystones be characterised as potentially erodible.

Design of erosion control measures in drainage lines and at culverts will need to take due consideration of the potentially dispersive nature of the site clays. It is recommended that drainage lines and areas of proposed filling to be prepared by benching or undercutting be stabilised with gypsum (approximate dosage in the range 1% to 2% by weight) prior to topsoiling or filling and re-vegetation. Proposed culvert foundation areas and approaches should be either stabilised with gypsum/lime (2% by weight), or treated by over-excavation, replacement with approved filling and recompaction for a minimum thickness of 400 mm.

Treatment of dispersive materials in cuttings will be required to reduce erosion and the potential for silting up of site drainage works. To achieve these ends, construction of defensive drains around the upslope sides of cuttings is recommended for the purpose of intercepting and diverting run-on surface flow. Treatment of dispersive materials exposed in cuttings and drain excavations might involve some form of mechanical stabilisation (such as fibre matting) followed by topsoiling and re-vegetation of all exposed batters (2H:1V or flatter) as soon as practicable. Relatively thin (for example, 50 mm thick) topsoil cover over stabilised areas will assist in root penetration through the topsoil and into the stabilised zone, reducing the potential for localised sliding of the topsoil layer.

3.3 Excavations

3.3.1 General

The proposed E2 alignment requires the excavation of approximately 39 cuttings. The maximum depth of excavation for the E2 option is approximately 15 m at about Ch 11000 on the centreline of the alignment. The maximum excavation depth at an offset of about 7 m right of the centreline at Ch 11000 (E2) is approximately 16 m.

The cross sections presented in Volume 3 (Drawing 8, Sheets 1 to 13) show a graphical representation of the conditions encountered in each of the cored boreholes and the anticipated cross section geometry at the corresponding chainage (E2 Alignment). The drawings also show a photograph of the rock core from the relevant bore to allow visual comparison.

3.3.2 Geology of Cuttings

A table presenting a summary of the mapped soil landscape groups and geological formations along the E2 alignment was presented in the Data Report. This table is summarised below, with the approximate maximum cut depth also included for the E2 alignment.

Table 3.1 – Mapped Soil Landscape Groups and Geology along Route Corridor

E2 Alignment		Maximum Depth of Excavation in Each Chainage Range (Approx, m)	Corresponding Soil Landscape Groups	Soil Origin	Corresponding Formation from Geology Maps
Approx Chainage From	Approx Chainage To				
0	580	3	Tillegra	Erosional	Flagstaff Formation
580	780	2	Salisbury Variant	Transferral/ Alluvial	Flagstaff Formation
780	1260	<1	Salisbury	Transferral	Flagstaff Formation
1260	2750	8	Dungog Variant	Transferral	Flagstaff Formation
2750	3080	15.5	Tillegra	Erosional	Flagstaff Formation
3080	3150	12	Williams Range	Colluvial	Flagstaff Formation
3150	3950	15.5	Williams Range	Colluvial	Chichester Formation
3950	5040	11	Tillegra/Williams Range	Erosional/ Colluvial	Chichester Formation
5040	6450	15.5	Tillegra/Williams Range	Erosional/ Colluvial	Flagstaff Formation
6450	9240	14.5	Tillegra	Erosional	Flagstaff Formation
9240	11470	16	Tillegra/Williams Range	Erosional/ Colluvial	Bonnington Siltstone
11470	13380	14	Tillegra/Williams Range	Erosional/ Colluvial	Chichester Formation (including Williams River Member)
13380	13930	13.5	Tillegra/Williams Range	Erosional/ Colluvial	Lostock Sandstone Member (including Verulam Oolite Member)
13930	14450	3	Tillegra	Erosional	Brownmore Sandstone Member
14450	14850	8	Tillegra/Williams Range	Erosional/ Colluvial	Brownmore Sandstone Member
14850	15040	12	Dungog	Transferral	Brownmore Sandstone Member
15040	15250	10	Williams Range	Colluvial	Brownmore Sandstone Member
15250	15380	<1	Tillegra	Erosional	Brownmore Sandstone Member
15380	15450	3	Tillegra	Erosional	Underbank Mudstone Member
15450	15680	<1	Tillegra	Erosional	Allyn River Member

Table 3.1 – Mapped Soil Landscape Groups and Geology along Route Corridor (Continued)

E2 Alignment		Maximum Depth of Excavation in Each Chainage Range (Approx, m)	Corresponding Soil Landscape Groups	Soil Origin	Corresponding Formation from Geology Maps
Approx. Chainage From	Approx. Chainage To				
15680	15950	1	Salisbury	Transferral	Allyn River Member
15950	16290	<1	Williams Range	Colluvial	Allyn River Member
16290	16845	<1	Williams Range	Colluvial	Salisbury Sandstone

3.3.3 Excavatability/Rippability

Based on the results obtained from the cored boreholes and test pits which were drilled/excavated for the proposed E2 alignment, assessments have been made regarding the potential rippability/excavatability of the materials encountered. Estimates of rock rippability/excavatability were made using a combination of company experience and reference to Figure 34 from Braybrooke (Ref 1). A reproduction of this interpretative chart is presented below.

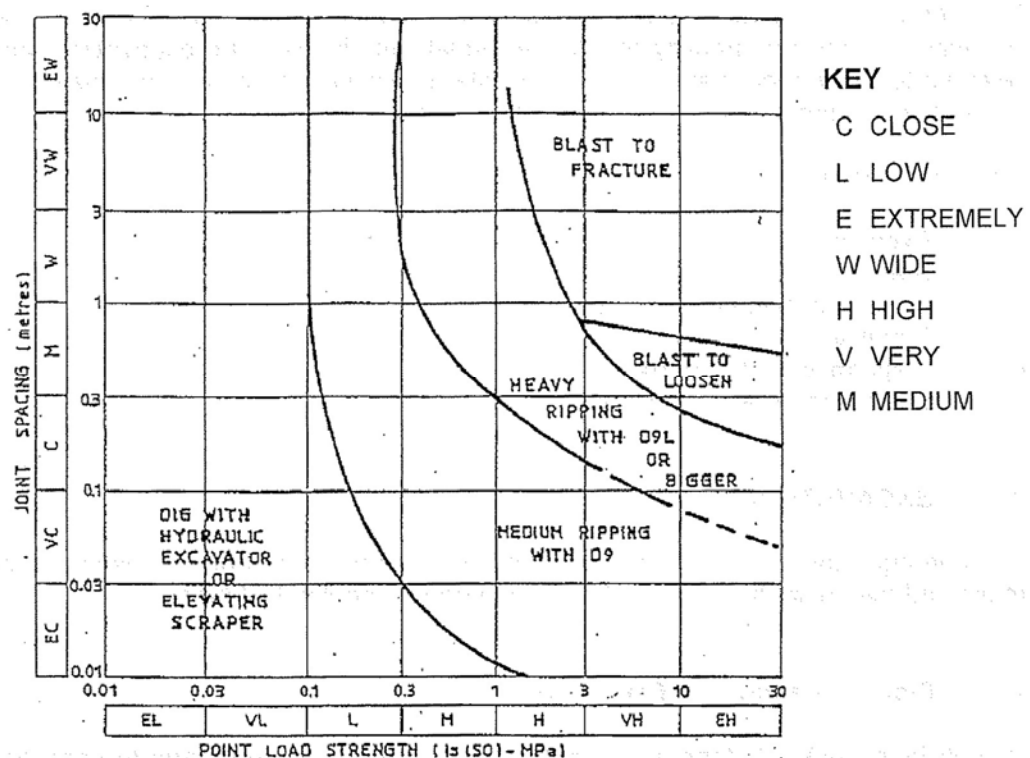


Figure 3.1 – Rippability Estimate from Strength and Joint Spacing (from Braybrooke, Ref 1)

The test pits were excavated in the general vicinity of proposed cuttings for the E2 alignment using a 20 tonne excavator. On this basis it is considered that the depth to refusal in each of these pits provides valuable information regarding the likely excavatability using traditional digging/medium ripping methods.

Table 3.2 below summarises the depth to refusal encountered in the test pits and shallow bores. The table also indicates the likely geological formation in which the test pits were excavated, based on the published geological maps and results of mapping from the site walkover.

Table 3.3 following summarises the interpreted excavation conditions for each of the cored boreholes drilled in areas of cutting for the E2 alignment. Tables 3.2 and 3.3 also indicate the appropriate corresponding E2 chainage and offset, together with the mapped geological formation in which the pits and bores were sited.

Table 3.2 - Summary of Observed Excavatability using 20 tonne Excavator for Test Pits

Test Pit Location	Approximate Chainage on E2 Alignment	Offset (E2)	Nearest Cutting on E2 Alignment (Refer Drawing 5)	Depth to Top of Rock (m)	Depth of Test Pit (m)	Reason for Terminating Test Pit	Corresponding Mapped Geology
106	1860	15 m right	3	1.2	1.8	limit of investigation	Flagstaff Formation
103D	2590	150 m right	4	0.45	2.1	limit of investigation	Flagstaff Formation
103B	2620	110 m right	4	0.8	2.0	slow progress	Flagstaff Formation
103A	2650	75 m right	4	0.5	1.3	slow progress	Chichester Formation
103C	2700	25 m left	4	1.3	2.9	slow progress	Chichester Formation
102B	3115	150 m right	6A	0.8	2.0	slow progress	Chichester Formation
102C	3115	185 m right	6A	2.0	3.0	slow progress	Chichester Formation
102A	3130	85 m right	6A	1.2	4.1	slow progress	Chichester Formation
101C	3440	120 m right	6B	NE (2.3?)	2.7	slow progress	Chichester Formation
101B	3450	80 m right	6B	2.5	3.2	slow progress	Chichester Formation
101A	3460	52 m right	6B	NE (2.0?)	2.7	slow progress	Chichester Formation
104	3830	82 m right	6C	2.5	3.1	slow progress	Chichester Formation
230	5825	0 m	11A	0.45	1.5	refusal	Flagstaff Formation
231	6295	75 m right	11B	0.45	1.4	refusal	Flagstaff Formation
223	9805	85 m right	21	0.4	1.5	refusal	Bonnington Siltstone
224	9925	70m right	21	0.45	1.5	refusal	Bonnington Siltstone
226	10315	30m right	22B	0.5	1.7	refusal	Bonnington Siltstone
227	10885	37m left	24	0.85	2.7	refusal	Bonnington Siltstone
228	10960	17m left	24	1.1	2.1	refusal	Bonnington Siltstone
229	11050	0m	24	0.35	1.7	refusal	Bonnington Siltstone
305	13460	9m left	32	0.8	4.0	virtual refusal	Williams River Member
306	13515	0 m	32	1.1	1.6	virtual refusal	Williams River Member
302	15740	22m right	39	NE (3.0?)	3.7	limit of investigation	Allyn River Member

Notes to Table 3.2: NE - Not encountered
 (3.0?) - Approximate depth to rock inferred from log of test pit

Table 3.3 – Interpreted Excavation Conditions for Cuttings Based on Profiles at Bore Locations

Bore Location	Depth Range (m)												
	421	412	414	413	415	416	417	418	420	419	411	408	407
Approx Chainage (E2 Alignment)	2700	3520	3855	5250	7190	7775	8625	9670	10980	11795	13070	14615	15060
Approx Offset from Option E2 Alignment	20 m right	23 m left	20 m left	20 m right	53 m left	15 m right	40 m right	28 m left	28 m left	14 m right	35 m left	5 m right	20 m left
Cutting Number	4	6B	6C	9	13	15	17	20	24	26B	31A	36	37B
Cross Section (Sheet of Drawing 7)	1	2	3	4	5	6	7	8	9	10	11	12	13
Geological Unit	Cef	Ceh	Ceh	Cef	Ceg	Cef	Cef	Ceg	Ceg	Ceh	Ceh	Cefb	Cefb
Hydraulic Excavator or Similar	0–2.5	0–1.5	0–1.3	0–1.1	0–1.4	0–2.5	0–1.9	0–1.9	0–1.0	0–2.0	0–2.6	0–3.0	0–2.9
Medium Ripping, e.g. using D9	> 2.5	0–1.5	1.3–1.8	1.1–1.6	1.4–4.2	0–2.5	1.9–2.6	1.9–6.0	> 1.0	0–2.0	2.6–4.0	> 3.0	0–2.9
Heavy Ripping with D9L or D10 or bigger	> 6.0	> 1.5 ⁽¹⁾	> 1.8	> 1.6 ⁽²⁾	> 4.2	> 2.5 ⁽³⁾	> 2.6 ⁽⁴⁾	> 6.0	> 1.0	> 2.0	> 4.0 ⁽⁵⁾	> 3.0	> 2.9
Blasting	–	> 1.5 ⁽¹⁾	5.5	> 1.6 ⁽²⁾	> 5.0	> 7.6 ⁽³⁾	> 4.4 ⁽⁴⁾	–	–	–	>10.5 ⁽⁵⁾	–	–

Notes to Table 3.3:

1. Blasting likely required below 1.5 m depth, although some bands may be rippable.
2. Blasting likely required below 1.6 m depth, although some bands may be rippable.
3. Strong band 5.75 m to 6 m may require blasting or rock hammer, then heavy ripping to 7.6 m and blasting below.
4. Blasting likely between about 4.4 m and 8.8 m.
5. Band from about 10.5 m to 12.4 m likely to require blasting or rock hammer.

Key to Geological Units:

- Ceh** – Chichester Formation (including the Williams River Member)
- Cef** – Flagstaff Formation (including the Lostock Sandstone, Brownmore Sandstone, Underbank Mudstone, and Allyn River members)
- Cefb** – Brownmore Sandstone
- Ceg** – Bonnington Siltstone

It is important to note that excavatability of rock is dependent not only on rock strength, but also on the presence, orientation and extent of discontinuities such as jointing and fracturing of the rock, presence of favourable and adverse bedding, presence of groundwater and other factors. For example, low strength rock with few discontinuities may be more difficult to excavate than highly fractured, high strength rock. The influence of such factors on rippability and the potential productivity benefits to be gained by the choice of other excavation methods is best assessed by the contractor/tenderer using field techniques such as costeans and ripping trials. Some of the rock encountered along the route cannot be ripped and will require blasting (for example, rock below 5.5 m in Bore 414). Where blasting will be used along portions of the alignment, consideration should also be given to the potential for disturbance of steep soil covered slopes.

It should also be noted that the execution of confined and detailed excavations in rock, such as for service trenches and drains, will likely require the use of rock hammers and/or blasting techniques.

Contractors should be responsible for selecting excavation equipment on basis of the capability of available equipment and the proposed depths of excavation, along with the geotechnical information on anticipated conditions.

If the above results from the cored boreholes are used to broadly generalise excavation conditions within different geological formations, then the following interpretations are made:

- Chichester Formation: medium to heavy ripping and blasting;
- Flagstaff Formation: heavy ripping and blasting;
- Brownmore Sandstone: medium to heavy ripping;
- Bonnington Siltstone: medium to heavy ripping and blasting.

Caution should be exercised in using these broad interpretations. They are based on as few as two boreholes in each formation, and hence are gross generalisations. The excavatability will be critically dependent on the number, orientation and spacing of joint sets within the rock mass as well as rock strength.

Contractors should be responsible for selecting appropriate excavation methods, and will likely require additional details once the route and design levels are known. No account is made in Table 3.3 for the likely variability of production rates that should be expected within each depth range.

A more rigorous assessment of rippability will be required during the detailed design phase and should include the integration of data from the drilling conducted to date with additional information from seismic survey. A program of drilling of additional cored boreholes is also being undertaken in areas of deep cuts where existing data is sparse.

3.4 Re-use of Materials from Bulk Earthworks

3.4.1 Soil

It is anticipated that, following clearing of vegetation and stripping of topsoil, the majority of soils won from excavations for proposed cuttings along the alignment will be suitable for re-use as engineered filling, providing all earthworks are carried out in line with the advice presented in Section 3.2 (stabilisation of potentially erodible materials).

It is likely that modification of soils using gypsum or an equivalent additive will be necessary in areas prone to run-on and potential erosion. Douglas Partners recommends additional testing of soils for erodibility and dispersivity be carried out for the detailed design phase investigations to further define the extent of such areas.

3.4.2 Weathered Rock

The ability to re-use weathered rock material won from bulk earthworks will be a function of the particle size achieved during excavation.

It is anticipated that some areas will require blasting in order to achieve the required excavation depths. Therefore, the blast design should take into account the required particle size. In general, particle size should not exceed 200 mm for re-use in engineered filling. The material

should be well-graded to control the compacted density and void ratio. All material to be re-used as engineered filling should be free of organics and other deleterious materials and debris. It is recommended that the maximum particle size be limited to 100 mm in the upper 2 m of filling. This is to reduce the chance of difficulties if excavations are required at a later date, for example, to install footings for barriers/posts and the like. It is anticipated that oversize rock materials would be used as rock filling at the toes of steep and/or high embankments.

Particle sizes from excavations are expected to vary with geology along the route. On the basis of the anticipated excavation methods shown in Table 3.3, likely expected rock sizes from excavations are presented in Table 3.4 below.

Table 3.4 – Estimated Rock Particle Sizes from Excavations

Rock Type	Estimated Particle Sizes From Excavations
Fractured Mudstone	80% of excavated material < 200mm in size 20% of excavated material ≥ 200mm in size
Interbeds of Shale and Sandstone	70% of excavated material < 200mm in size 30% of excavated material ≥ 200mm in size
Tuff, Meta Sandstone and Lithic Sandstone	40% of excavated material < 200mm in size 40% of excavated material 200mm-400mm in size 20% of excavated material ≥ 400mm in size

Significant variation from the estimated proportions of particle sizes shown in Table 3.4 can be expected with changes to the excavation methods and equipment used along the route.

3.4.3 Soil and Rock Acidity

No specific laboratory tests have been carried out with regard to potential acidity of soil and rock materials from proposed cuttings and other borrow areas.

The desktop study for the Concept Design Phase Data Report indicated that there is no known occurrence of acid sulphate soil (ASS) materials along the proposed road alignment, and that it

is unlikely such materials would be found at the subject site. The main reason for the apparent absence of ASS materials in the project environment is that local sediments are typical of neither coastal/marine muds nor high level marshes/acid-salt wetlands. It is recommended that sampling and screen testing of soils for the potential presence of ASS materials be carried out during the detailed design phase investigations to confirm these indications. Sampling and testing should target soils of alluvial origin in landscapes with poor drainage within the proposed route corridor. If ASS screening indicates the presence of either potential or actual ASS, then detailed ASS testing (by chromium reducible sulphur methods) should also be performed.

The results of petrographic analyses (Appendix C2, in Volume 2) indicate that rocks sampled from cored boreholes along the route have the following mineralogy:

- Illite, fine mica, and quartz as major components in shales and mudstones, with minor inclusions of calcite, carbonaceous matter and iron oxides/hydroxides;
- Siliceous and volcanic lithics, and quartz as major components in sandstones and conglomerates, with minor inclusions of feldspar, illite, altered mica, chlorite and iron oxides/hydroxides.

Rocks that are typically of sulphidic mineral origin present a risk of acid generation on exposure and oxidation. It is apparent that rocks considered typical of excavation conditions along the proposed E2 route do not tend to contain such minerals. Douglas Partners recommends that sampling and testing for the detailed design phase investigation include assessment of the potential presence of acid rocks in cuttings along the proposed route.

3.5 Embankment Filling

3.5.1 General

Achieving the specified compaction will be particularly important throughout construction of the embankment to its outer edge. The most practical way to achieve this is to construct the slope to beyond the proposed finished batter line, and then trim back to the design line following compaction. The ability to achieve compaction up to the edge of the slope batter will affect the allowable design batter slopes.

Where embankments are to be constructed against slopes of significant grade (of the order of 10% and greater), the placement of filling should be preceded by benching/keying into the existing slope. This can be achieved by excavating horizontal benches, each of which should be keyed into the slope by sufficient horizontal distance so that the height of the bench is at least that of the fill layer, that is, of the order of 300 mm.

3.5.2 Earth Filling for Embankments

The following general procedure is recommended for placement of embankment filling:

- Remove all topsoil, natural soils and existing filling that are unsuitable for placement of new filling;
- Test roll the surface in order to assess existing moisture conditions and the presence of soft zones. Moisture contents to depths within about 1 m below subgrade level should be in the range OMC -3% (dry) to OMC, where OMC is the optimum moisture content at standard compaction;
- Where unsuitable subgrade material is encountered remove by over-excavation and replace with clean approved filling. Over-excavation to depths of up to about 1 m should be anticipated in gullies;
- Where material is unsuitable based on moisture content, the surface for placement of filling at subgrade level may be prepared by tining using rippers to promote aeration followed by compaction to a dry density ratio of 98% standard (AS 1289.1.1, or equivalent);
- Compact the tined natural surface to a dry density ratio of at least 100% Standard. Where encountered, clay subgrades should be left exposed following compaction for the minimum practicable period of time prior to placement of filling, to limit the potential for occurrence of desiccation cracking;
- Approved filling should be placed in horizontal layers not exceeding 300 mm loose thickness and compacted to an appropriate dry density ratio depending on the depth of the filling as indicated in Table 3.5.

Table 3.5 – Minimum Recommended Compaction Criteria for Embankment Filling

Filling Zone	Minimum Dry Density Ratio for Clayey/Cohesive Soils *
Within 5 m of top of filling	100% Standard
More than 5 m below top of filling	98% Standard

Notes to Table 3.5:

- * Moisture content for clayey soils should be in the range as stated above.

If sandy soils are used for backfill, such as around culverts and the like, they should be compacted to a minimum density index of 80%.

It is noted that, if encountered, silty soils can be difficult to work, particularly when wet. If the soils become wet, they should be tyned and allowed to dry. Tight control of moisture will be required during compaction of these soils. Silty soils were encountered in tests undertaken near each of the two proposed river crossings.

3.5.3 Rock Filling for Embankments

Determination of the procedure for placement of rock filling was made using reference to Breitenbach, A J, "Rockfill Placement and Compaction Guidelines", 1993 (Ref 2).

Specification for compaction of rock filling is to be determined using on-site test embankments. Surveyed settlement versus roller pass curves developed in a trial fill embankment determines the optimum number of roller passes for effective compaction of the rock materials characteristic of the test area.

Test fill placement should adopt the same method and equipment that would be used in the placement of rock fill for embankment construction. The use of a vibratory roller with a minimum static weight of 10 tonne is recommended. As a general rule a maximum of 4 to 6 passes is required. More than 6 passes tends to crush and pulverise the top layer of rock without adding significant compaction to the lower part of the lift. The average settlement over 5 survey points should be used for each trial filling embankment. Survey readings are to be taken after every

second test pass. Following placement of trial filling and settlement tests, in-place bulk density testing and gradation is recommended to verify rockfill procedures and design assumptions.

The following practices should be considered in the formulation of a compaction specification for rock filling:

- Material for use in areas of rock filling should retain a minimum of 30% of sample on a 19 mm sieve and contain less than 15% fines passing the 75 μm (0.075 mm) sieve;
- Maximum particle size should not exceed two-thirds of the lift layer thickness. Occasional larger rocks are permitted if they are scraped to the side of the fill embankment or tracked over and crushed by a bulldozer;
- Wetting of the rock is required and preferable in the rock borrow area. Ideally the rock should be sufficiently wet so that no dust occurs when the haul truck or scraper dumps a load on the fill surface;
- If wetting is not accomplished in the rock borrow area, the rock should be made sufficiently wet prior to spreading the new lift or following compaction (wetting immediately prior to compaction significantly dampens the dynamic force of the compactor);
- Rock filling should be placed in horizontal layers not exceeding 800mm loose thickness and compacted using methods determined from trial embankments. (Previous cases have shown a loose layer thickness of 500mm to 800mm gives optimum compaction (Ref 2). Thicker layers are permitted but are subject to laboratory testing and results of roller pass versus settlement curves determined from trial embankments).

Geotechnical inspections and testing should be performed during construction.

4. COMMENTS – PAVEMENTS

4.1 General

Douglas Partners understands that the new Salisbury Road formation will remain a local road under the jurisdiction of Dungog Shire Council (DSC) for the purposes of operation and maintenance. In terms of design and construction of road pavements, DSC currently refers to the development specification series known as “Aus-Spec-1”, which requires compliance with the NSW Road and Traffic Authority (RTA) QA Specification 3051 for materials to be used as gravel for pavements. Douglas Partners anticipates that a similar specification would apply to design and construction of the new route for Salisbury Road. Based on the above comments, on traffic data advised by Opus, and on the results of field and laboratory testing presented in Volume 1, the design thickness for the new road pavement would be generally in the order of 100 mm of base gravel over 200 mm of sub-base. This thickness allowance is based on the following assumptions:

- Typical California Bearing Ratio (CBR) of subgrade in the range 7% to 10%;
- Design traffic loading in the order of 1×10^6 equivalent standard axle repetitions (ESAs), estimated using annual average daily traffic of 500 to 1500 vehicles (total both directions) with annual growth of 1%, 30 year design life, and up to 10% of traffic as heavy vehicles.

Low CBR values (3% and less) were found near proposed grade level at the river crossings, which would likely require subgrade improvement measures. Sections of the route will also encounter rock subgrade having substantially greater CBR strength, with a corresponding likelihood of pavement thickness reduction. Each of the above factors should be assessed and confirmed along the length of the proposed route at detailed design stage.

4.2 Sourcing of Materials for Pavement Construction

4.2.1 Potential Sources Near the Project Alignment

Four test pits were excavated in selected disused quarry pit locations to undertake a preliminary assessment of potential material sources for pavement construction. The approximate locations

of these pits are shown on Drawing 3 (Sheets 1 and 2) in Volume 2. The results of test pitting and laboratory testing are included in the Data Report and associated appendices.

Samples collected from three of the test pit sites were subjected to pre-treatment processes by compaction and artificial weathering prior to testing. The pre-treatment was undertaken to RTA Test Methods T102 (compaction weathering) and T103 (artificial weathering).

Following pre-treatment, the samples were analysed for the resulting particle size distribution, with the results compared to RTA Form 3051 grading requirements for sub-base quality gravel (DGS 40). A summary of these results is presented in Table 4.1 below. The sample from site B1 weathered completely and was not submitted for further testing.

Table 4.1 – Laboratory Test Results from Former Quarry Sites Compared to RTA Pavement Material Specifications

Laboratory Test	Sieve Size for Grading (mm)	Laboratory Test Result			RTA Specification Form 3051 (DGS 40)
		Pit B2 Shale	Pit B3 Shale	Pit M1 Siltstone/Shale	
Grading (percent passing sieve size)	75.0	100	-	-	-
	53.0	86	100	100	100
	37.5	74	97	99	-
	26.5	65	90	95	-
	19.0	57	80	88	50-85
	13.2	45	68	80	-
	9.5	32	60	71	-
	6.7	22	52	63	30-55
	4.25	14	46	55	-
	2.36	7	36	42	25-50
	0.425	3.1	21	31	-
	0.075	1.7	11	21	-
	0.0135	0.8	7.9	19	-
A-Ratio (%)		58	46	75	35-55
B-Ratio (%)		53	55	67	35-55
C-Ratio (%)		73	68	89	35-60
Liquid Limit (%)		-	-	28	Max 23
Plastic Limit (%)		-	-	22	Max 20
Plasticity Index (%)		N/P	N/P	6	Max 12
CBR (%)		80	35	20	30 ⁽¹⁾

Notes to Table 4.1:

- ⁽¹⁾ Minimum CBR not specified in RTA 3051; general requirements are that sub-base materials have a CBR greater than 30% and base materials have a CBR greater than 80%.

N/P – Non Plastic

With reference to the above table, it can be seen that:

- Pit B2: Does not meet RTA specification for *sub-base* quality material, but could possibly be improved through sieving of oversized particles;
- Pit B3: Meets RTA specification for *sub-base* quality material based on the grading, however is marginally outside the C Ratio requirement;
- Pit M1: Does not meet RTA specification for *sub-base* quality material but may be suitable as a select subgrade material.

The extent and consistency of the above materials have not been assessed. Based on site observations and the results of laboratory testing, Douglas Partners has assessed that the materials from former quarry sites B1, B2, B3 and M1 are not suitable for use as pavement gravel for the proposed diversion of Salisbury Road.

It is anticipated that establishing a quarry as well as a material processing and screening plant would require a number of statutory approvals. If sourcing of pavement materials from near the project alignment is to be considered further, then significant additional geotechnical field and laboratory testing will be required to assess material properties in more detail. This may include core drilling in the stronger rock formations (for example, the Chichester and Flagstaff Formations) followed by laboratory testing, including testing for strength, durability and classification of particle sizes, in order to characterise the formations in terms of their potential for sourcing pavement gravels.

4.2.2 Other Potential Sources

At the time of this assessment the NSW Department of Commerce (DoC) was investigating a potential quarry near the proposed construction site for Tillegra Dam. The purpose of DoC's investigation was to characterise potential borrow materials near the dam site in terms of their suitability for use as rockfill and as aggregate for the production of concrete.

DoC provided Douglas Partners with a copy of laboratory test results of aggregate crushing value, Los Angeles abrasion, and sodium sulphate soundness on each of two samples:

- Rock core recovered from a diamond drill hole (DDH 9) at the investigation site for the potential dam quarry;
- Gravel samples from the Williams River in the vicinity of the proposed dam site.

Details of the potential quarry materials in terms of engineering properties and potential suitability for construction of road pavements were not available at the time of the concept design phase assessment. The results of aggregate testing provided by DoC indicated that each of the tested sources could produce quarry gravel of strength and durability suitable for use in construction of road pavements. The assessment of material characteristics relevant to the performance of the gravel as a pavement material would also be necessary. The test types needed for such characterisation would likely be chosen from the requirements of RTA QA Specification 3051, such as RTA T106/T107 (coarse and fine particle size distributions) and RTA T108/T109 (Atterberg limits), or equivalent testing procedures.

Douglas Partners also understands that rock materials in the former quarry at Chichester Dam are of similar petrography to the rock core sampled by DoC at the quarry site for the proposed Tillegra Dam. HWC has indicated that access can be made available to these materials for the purposes of sampling and laboratory testing of engineering properties. If the materials are found to be suitable, it is anticipated that availability for use on the road diversion project will depend on timing of establishment of the quarry for Tillegra Dam.

If road pavement materials cannot be sourced in the immediate vicinity of the project alignment, or from the proposed quarry for the Tillegra Dam construction, then it is anticipated that the nearest source will be from existing commercial quarries in the Brandy Hill and Seaham areas, north-west of Raymond Terrace.

5. COMMENTS – SLOPE STABILITY

5.1 Slope Stability

5.1.1 General

The proposed project alignment will pass through areas of steep terrain, including ridges and incised gullies. Some areas of the proposed alignment will be subject to a higher risk of instability than others, based on the results of the desktop review (air photo interpretation) and the walkover assessment. In particular, steep slopes and colluvial soils encountered in the 'Williams Range' Soil Landscape grouping are likely to be predisposed to slope instability, and present localised mass movement hazards.

The following sections present a discussion of conditions likely to be encountered along the route corridor. In addition to the comments presented below, reference should also be made to Table 3.1 in the Data Report (July 2008) which summarises mapped soil landscape groups and geology formations along the preferred route corridor, to identify areas of potentially higher risk.

Care should be exercised when undertaking earthworks in the steep areas of the site, particularly in wet weather.

5.1.2 Existing Slopes

Ch 2500 to 4000

The proposed ascent of the eastern flank crosses numerous spurs with slope angles varying in concave form between 30° near the crest and 7° near the foot of the ridge. Deeply incised gullies between these spurs have slopes of up to 35° in the vicinity of the proposed route corridor. Gully slopes flatten to angles varying between 11° and 19° east of the proposed route corridor.

Evidence of instability within surficial soils was observed on slopes above steep gullies at approximate Ch 2900, Ch 3000, and Ch 3300 on the E2 alignment option. Figures 5.1 and 5.2

below show recent movement of the slope in the vicinity of approximate Ch 2900 to Ch 3000 (photograph taken in May 2008), and at about Ch 3250, respectively (photograph taken in July 2008).



Figure 5.1 – Landslip in surface soils above approximate Ch 2950 to Ch 3000 (May 2008)



Figure 5.2 – Landslip in surface soils below approximate Ch 3250 (July 2008)

Of the potential hazards involving instability of natural slopes along the proposed alignment, it is likely that mass slope movement presents the greater risk along this portion of the route.

Ch 4000 to 9000

The proposed alignment completes ascent of the ridge at approximate Ch 4000. Areas showing signs of instability as a result of ongoing gully formation were observed on both flanks of the ridge adjacent to the proposed route corridor between approximate Ch 3100 and Ch 7800. Much of the surface instability on the western flanks (north of approximate Ch 5500) presents as development of “terraces” or small steps down the slope. The instability of soils is a result of a bedding dip and defect orientation (typically into the slope) within the underlying weathered rock, which promotes the development of a weathering surface sub-parallel to the ground surface.

Two areas of significant landslip are present on the western face of the ridge between approximate Ch 3000 and Ch 4500. These areas of landslip are not of significance in terms of proximity to the proposed route corridor.

A gradual descent along the western faces of the ridge is proposed between approximate Ch 5500 and Ch 9000, within rocks of the Flagstaff Formation. Areas of instability within colluvial soils are present between approximate Ch 5500 and Ch 7500, and development of “terraces” is also evident across the south and west-facing slopes of this portion (see Figures 5.3 and 5.4 below). Seepage of water near the contact between soil and exposures of sub-horizontally bedded weathered siltstone and sandstone was observed at approximate Ch 7400 and occurs elsewhere.

Ch 9000 to 13000

The slopes between approximate Ch 9800 and Ch 13000 are characterised by sequences of west trending spurs at slope angles varying from 7° to 12° with incised drainage features between. Tunnel erosion of surface soils was observed on the flanks of some gullies along this portion of the proposed route. The erosion appeared to be a contributor to the development of shallow instability that is part of the evolution of the dendritic drainage features in the area.



Figure 5.3 – “Terracettes” and slope movement above approximate Ch 7450 (January 2008)



Figure 5.4 – “Terracettes” above approximate Ch 7600 (January 2008)

Figure 5.5 shows erosion and slip of soils on the flank of the gully crossed by the proposed E2 alignment at approximate Ch 12150. Figure 5.6 illustrates the relationship of slope movement to the formation of a gully joining the main slope drainage feature at this location. No obvious evidence of overall slope instability was observed along this portion of the proposed route.



Figure 5.5 – Erosion and slip of surface soils below approximate Ch 12150 (July 2008)



Figure 5.6 – Slip shown in Figure 4.4 as part of ongoing gully formation

Ch 13000 to 16000

Topography between approximate Ch 13000 and Ch 14000 is dominated by two south-trending drainage features (Moolee Creek being the westernmost of these). The spurs between these watercourses are of convex shape with flanks sloping at angles of up to 18°. No obvious evidence of overall slope instability was observed along this portion of the route.

Ground slopes between approximate Ch 14000 and Ch 15400 are near uniform at angles of 3° to 6° towards north. A broad gully trending east at angles of 10° to 11° drains these slopes towards Moolee Creek. No obvious evidence of overall slope instability was observed along this portion of the proposed route.

The south-facing side of the landform between approximate Ch 14700 and Ch 15200 is a roughly 40 m high escarpment sloping at angles of about 60° to the Williams River. Field mapping of geological structure in rock exposures indicates that sediments have been distorted into an anticlinal fold oriented perpendicular to the Williams River with a slight plunge trending north in this region. The presence of this geological structure in part explains the steepness of the escarpment. The trunks of large native trees growing on the face of the escarpment were observed to be straight and vertical. No obvious evidence of overall slope instability was observed along the face of the escarpment.

5.1.3 Proposed Western Face Ascent Route

The option of an ascent across the western face of the ridge that forms the left abutment of the proposed Tillegra Dam was considered as part of an early route study by GHD, and does not form part of the proposed route corridor enclosing the Q21A4 and E2 alignments.

Based on the results of a desktop study and walkover assessment presented in Douglas Partners report 39721A-03L (25 February 2008), the implications for construction of a western face ascent route are judged to be as follows:

- A substantial length (between 600 m and 800 m) of cutting into a rock slope defined by bedding at an angle of about 40°, likely to require drill and blast methods of excavation;
- Need for pre-splitting of the rock face excavations described above, and/or support of lengths of over-break and overhanging rock defined by sub-vertical joint sets orthogonal to the predominant bedding. Support of excavations across the rock face would likely involve substantial quantities of bolting and of shotcrete application;
- Need for over-excavation of the steep rock face in benches as preparation in areas of filling across localised drainage paths;
- Filling of at least two gullies in the soil-covered slopes north of approximate Ch 2500, each of which are of significant depth and side slope angle, with the contingent need for substantial preparation work for embankment foundations.

Based on the above comments, and their likely implications for construction costs, Douglas Partners recommends that the option of a western face ascent route be discounted in favour of a corridor along the east face of the ridge.

5.1.4 Proposed Cutting and Filling

Construction of a route following the proposed E2 alignment will result in excavations to depths of up to 15 m and fill embankments to heights of up to 14 m between approximate Ch 2700 and Ch 4000 within rocks of the Chichester Formation. Excavations to depths of up to 14 m and fill embankments with total heights of up to 20 m are proposed between approximate Ch 11600 and Ch 13600, also within the Chichester Formation.

Excavations within soil and extremely weathered rock for the preparation of filling embankment footprints between approximate Ch 2700 and Ch 3500 will need to account for temporary support to depths in the order of 1 m to 1.5 m to mitigate the risk of overburden slumps into the work areas. Allowance should be made for battering of soil slopes or the construction of stabilisation or permanent retaining measures along cuttings in this portion of the route to mitigate the risk of occurrence of long term slope collapse events.

Excavations to depths of up to 15 m and fill embankments to heights of up to 17 m are proposed between approximate Ch 5000 and Ch 9200 within rocks of the Flagstaff Formation. Earthworks between approximate Ch 5000 and Ch 7500 will need to take account of substantial measures to prepare the stripped natural surface for placement of filling, including cutting of benches about 1 m wide with step heights between each bench not less than 0.3 m.

Excavations to depths of up to 16 m are proposed between approximate Ch 9200 and Ch 11600 within rocks of the Bonnington Siltstone Formation. Zones of fracturing and shearing within weathered siltstone are anticipated throughout the proposed cuttings along this portion of the route. It is expected design allowances would be made for either less steep cut batter slopes than in other portions of the route, or permanent support of batters (for example, dental shotcrete) that appear compromised by adverse shearing and/or fracturing.

The proposed cuttings within Brownmore Sandstone Member rocks (between approximate Ch 14500 and Ch 15200) will likely encounter weathered sandstones and mudstones with substantial fracturing to the proposed level of bulk excavation. It is anticipated that spalling of rock fragments from the excavated batters will occur over this portion of the route. Measures to mitigate the likelihood or consequences of spalling might involve flattening of batters, application of dental or structural shotcrete, design allowances for wider road shoulders incorporating rockfall impact beds, or combinations of these.

The setback of the proposed route corridor from the crest of the escarpment (roughly 45 m) appears to be appropriate in terms of the assessed risk of instability of existing slopes in this area.

5.2 Batter Slopes

5.2.1 Excavations

At the time of the concept design phase assessment, it was understood excavation included provision of catch drains at the base of most cutting faces.

Generalised excavation batter slopes have been interpreted for each borehole located within a proposed cutting on the basis of rock strength, weathering and fracturing data from cored boreholes along the proposed route.

Table 5.1 below summarises recommendations on design batter slopes based on the borehole data. If the results presented in Table 5.1 are used to broadly generalise possible batter slopes within the different geological formations, then the following interpretations are made:

- Chichester Formation: 2H:1V in soil, then varies in underlying rock depending on rock strength and jointing;
- Flagstaff Formation: 2H:1V in soil, then generally 0.5H:1V or 0.25H:1V rock, although some areas of 1H:1V possible;
- Brownmore Sandstone: 2H:1V in soil and 1H:1V in rock; catch fences likely required;
- Bonnington Siltstone: 2H:1V in soil; 1H:1V in upper rock and either 0.5H:1V or 0.25H:1V in underlying higher strength rock.

Caution should be exercised in using these broad interpretations. They are generalisations based in the main on adverse defect structure from as few as two boreholes in each formation. Additional work to assess defect orientations should be undertaken during detailed design. Such additional work could include cored boreholes in conjunction with down-the-hole camera viewing techniques, additional walkover assessment and additional laboratory testing of rock core.

Table 5.1 – Interpreted Batter Slopes for Cuttings based on Profiles at Bore Locations

Location	Depth Range (m)												
	421	412	414	413	415	416	417	418	420	419	411	408	407
Approximate Chainage (E2)	2700	3520	3855	5250	7190	7775	8625	9670	10980	11795	13070	14615	15060
Approximate Offset from Option E2 Alignment	20 m right	23 m left	20 m left	20 m right	53 m left	15 m right	40 m right	28 m left	28 m left	14 m right	35 m left	5 m right	20 m left
Cross Section (Sheet of Drawing 6)	1	2	3	4	5	6	7	8	9	10	11	12	13
Geological Unit	Cef	Ceh	Ceh	Cef	Ceg	Cef	Cef	Ceg	Ceg	Ceh	Ceh	Cefb	Cefb
2:1 (H:V)	0–2.5	0–2.0	0–1.8	0–1.6	0–1.5	0–2.55	0–1.6	0–2.0	0–1.0	0–2.0	0–2.6	0–3.0	0–3.0
1:1 (H:V)	> 2.5	–	–	–	1.5–5.0	2.55–4.0	–	2.0–5.9	> 1.0	> 2.0	2.6–8.0 ₍₅₎	> 3.0 _(5,6)	> 3.0 _(5,6)
0.5:1 (H:V)	–	–	> 1.8	–	–	4.0–7.5 ₍₃₎	> 1.6 ⁽⁴⁾	> 5.9	–	–	> 8.0	–	–
0.25:1 (H:V)	–	> 2.0 ⁽¹⁾	> 5.0	> 1.6 ⁽²⁾	> 5.0	> 7.5	–	–	–	–	–	–	–

Notes to Table 5.1:

- Based on profile below approx 6 m depth shown on log of Bore 412.
- Contains some more fractured zones or lower strength zones which could require additional treatment, such as shotcreting and/or bolting.
- Could possibly be steepened to 0.25H:1V but will depend on joint continuity.
- Shotcreting and possibly bolting likely required between 9 m and 10 m.
- Could possibly be steepened to 0.5H:1V but will depend on joint continuity.
- Catch fences or impact beds likely required at base of slope.

Key to Geological Units:

- Ceh** – Chichester Formation (including Williams River Member)
Cef – Flagstaff Formation (including the Lostock Sandstone, Brownmore Sandstone, Underbank Mudstone, and Allyn River members)
Cefb – Brownmore Sandstone
Ceg – Bonnington Siltstone

It should be noted that:

- All batter slopes will require assessment by a geotechnical engineer or engineering geologist during construction to confirm design parameters;
- Batter slopes might require additional treatment such as bolting, shotcreting or other support measures;
- Impact beds and/or catch fences at the toe of excavation slopes should be a design standard.

More fractured zones within the rock strata, for example the shale in Bore 417 between 9 m and 10 m depth, represent a higher risk of erosion which may undercut overlying stronger layers. These zones will likely require treatment with shotcrete or similar at the time of construction.

It is recommended that an assessment be undertaken regarding the potential for the siltstones, shales, tuffaceous sandstones and lithic sandstones to rapidly weather, as this may affect the selected slopes, and/or batter treatment during construction. This would involve laboratory testing of the durability of samples of rock core.

5.2.2 Fill Embankments

Batter slope design for fill embankments will be a function of the level of control that can be maintained in achieving the nominated compaction specifications.

As a general rule, most fill embankments should be constructed no steeper than 2:1 (H:V). However, it may be possible to construct slightly steeper embankment slopes, but only with strict control of a number of factors, such as compaction, drainage and base preparation, and only with careful engineering design and construction. In areas where topography requires steeper embankment slopes, then it is possible to increase to a slope of no steeper than 1.5:1 (H:V), but only under the following conditions:

- The method-based specification for placement of filling should be increased to a higher compaction standard for the full height of the fill embankment;

- Coarse granular fill materials should be placed at the toe to promote drainage;
- The slope batter should be stabilised by appropriate means to protect against erosion.

Based on data from the cored boreholes and test pits, it is anticipated that foundation areas prepared in line with the above comments and the guidelines in Section 3.5.1 will provide a bearing surface of suitable capacity for filled embankments with design geometries as proposed for the E2 alignment. The design of embankment filling to total heights in excess of 10 m should include camber varying between 50 mm and 75 mm (greater of these corresponds to a design maximum embankment height of about 20 m) to allow for differential settlements along the longitudinal axis of the alignment.

As recommended above, compaction is required out to the batter slope line. In order that this be achieved, it will be necessary to construct the embankment by placing filling beyond the proposed finished batter slope line and then trimming it back, or by other approved methods of construction. Benching or keying of the filling into its foundation is also recommended, as described in Section 3.5.1.

In areas where maintenance of the batter slope is required, such as mowing, then the slope should be flattened to 3:1 (H:V) or flatter.

5.2.3 Drainage

It is understood Dungog Shire Council has indicated to the designer a preference to avoid the provision of drains behind the crest of cut batter slopes, due to potential access constraints for maintenance of such features. On this basis, Opus' concept design indicates that stormwater runoff would be allowed to sheet flow over the face of excavation batters, and that intermediate benches be tapered to similarly allow water to flow over the face. This option is not recommended, particularly in areas where the rock is fractured or highly fractured. It is also not recommended in slopes containing weaker bands that could be susceptible to erosion and hence increase the risk of undercutting beneath stronger layers of rock (for example, the shale layer in Bore 417 from 9 m to 10 m depth). Douglas Partners understands that Opus will address the matter of drainage around cut batters further during the detailed design phase.

If drainage at the crest of excavation slopes is omitted, it must be accepted that slope failures will occur, particularly in areas where jointing is adverse and/or the rock is more fractured or the rock is susceptible to weathering. Furthermore, larger catch areas may be required at the toe of excavation batters to mitigate the likelihood or consequences of rockfall hazards within cuttings. Inspection and maintenance of the roadway will need to be undertaken at an increased level after rainfall, particularly in the first few years of operation of the road. It is also possible that retrospective treatment of exposed cut slopes could be required if slope failures present ongoing concerns.

Where possible, consideration should be given to diverting water away from slopes by installing concrete lined dish drains at the crest of slopes. As a matter of risk mitigation, water should not be allowed to pond near the toe of cut or fill slopes, and/or against road pavements. Dish drains or similar should be installed to divert water out of cuttings.

5.2.4 Groundwater

Limited data are available regarding groundwater, or the flow of water through joints in rocks. At the present, groundwater levels across the site are largely unknown. It is recommended that additional data be gathered relating to the presence of groundwater in proposed excavations. Installation and monitoring of standpipe piezometers in several additional bores is recommended for the purpose of measuring groundwater levels along the route, together with compilation of rainfall data for the periods of water level monitoring.

Based on observations made during drilling and coring, it is anticipated that permanent groundwater will be present at levels below bulk excavation levels for proposed cuttings in general. Surface seepage of groundwater observed along the western flank of the ridge between approximate Ch 6000 and Ch 9000 are likely associated with out-of-slope dip of sedimentary beds, which would represent groundwater perched above the level of a permanent water table.

The preliminary interpretations presented above appear consistent with the results of recent investigations by the NSW Department of Commerce along the rim of the reservoir associated with the proposed Tilleggra Dam. In the absence of further data, it would be prudent to make

provision for installation (and maintenance) of weepholes where there is a likelihood of groundwater presence at or above the proposed formation levels within rock formations.

6. COMMENTS – PROPOSED BRIDGES

6.1 General

The proposed road diversion will cross the Williams River at two locations, requiring the construction of two bridges. The proposed bridge locations are at approximate Ch 1100 and Ch 16220 (E2 alignment). These are colloquially referred to as the “lower crossing” and the “upper crossing”, respectively.

The upper crossing will be upstream from the proposed Tillegra Dam, and hence the flows in the river at this location will be subject to the prevailing weather and without controls. The lower crossing will be downstream from the proposed dam, and hence flow will be controlled by the dam infrastructure.

Six boreholes and one test pit have been undertaken in the general vicinity of the proposed bridge for the lower river crossing. Five boreholes and two test pits have been undertaken in the general vicinity of the proposed bridge for the upper river crossing. The ground conditions at each bridge site can be generally summarised as follows:

Location: Lower River Crossing (approximate Ch 1100)
Test Pit Numbers: 108
Deep Bore Nos: 401B, 405A
Deep Cored Bore Nos: 401A, 402, 405B and 410
Geological Formation: Chichester Formation

Generalised Ground Conditions Lower Crossing:

FROM (m)	TO (m)	DESCRIPTION
0.0	0.1/0.3	TOPSOIL
0.1/0.3	0.8/3.0	SAND/SILT/CLAY MIXES: ranging from loose/firm to hard, but soft to firm in Bore 410;
0.8/3.0	4.8/7.75	GRAVEL MIXES: coarse gravel mixed with sand, silt and clay in varying proportions; generally difficult to drill through
4.8/7.75	Termination Depth	ROCK: comprising shale or siltstone of generally high to very high strength

Groundwater was encountered at depths ranging from about 3 m to 4 m during drilling and test pitting.

Location:	Upper River Crossing (approximate Ch 16220)
Test Pit Numbers:	301, 303 and 304
Deep Bore Nos:	404B
Deep Cored Bore Nos:	403, 404A, 406, 409
Geological Formation:	Allyn River Member, transitioning to Salisbury Sandstone beyond about Ch 16300

Generalised Ground Conditions Upper Crossing:

FROM (m)	TO (m)	DESCRIPTION
0.0	0.2/0.3	TOPSOIL/SANDY SILT
0.0	0.6/1.5	SILTY SAND: loose; encountered in Bores 403 and 404A
0.2/0.3	0.6/4.0	SAND/SILT/CLAY MIXES: ranging from loose/firm to hard
0.6/4.0	4.9/5.8	GRAVEL MIXES: coarse gravel mixed with sand, silt and clay in varying proportions; generally difficult to drill through
4.9/5.8	Termination Depth	ROCK: initially extremely low strength, but increasing to high strength meta siltstone

Groundwater was encountered at depths ranging from about 2 m to 4 m during drilling and test pitting.

Graphical representations of the long sections and inferred geology for each bridge site are shown on the attached Drawing 9, Sheets 1 and 2.

The following sections present recommendations regarding the proposed bridge structures.

6.2 Bridge Approach Embankments

It is anticipated that fill embankments to design maximum heights between about 2 m and 5 m will be required for the lower crossing, and up to about 7 m in height for the upper crossing. Careful consideration of bridge approaches is important to minimise the risk of differential settlement between the bridge structures, which are often founded on piles into underlying rock, and the adjoining earth fill embankment, which is founded on natural soil.

Preliminary settlement estimates have been made, based on the limited bore data, for the proposed bridge approaches. These are summarised in Table 6.1 below:

Table 6.1 – Summary of Settlement Estimates

Bridge Site	Approach	Relevant Bore Data	Proposed Embankment Height (m)	Estimated Settlement Range
Lower Bridge/Crossing	Southern	410 401A	3 m to 5 m	20 mm to 100 mm
	Northern	402 405B	Up to 2 m	10 mm to 30 mm
Upper Bridge/Crossing	Southern	403 406	Up to 7 m	30 mm to 70 mm
	Northern	404A 409	Up to 6 m	20 mm to 80 mm

It is possible that a significant proportion of the estimated settlements noted above could occur during construction. Based on the interpreted material and strength profiles of silty, sandy and gravelly sediments and the proposed embankment heights at each crossing site, it is anticipated that settlements in the vicinity of the proposed bridge abutments would trend towards the lower end of the ranges in Table 5.1. On this basis, it is possible that pre-loading of embankment construction areas, if required, could be performed over periods of the order of a few months prior to bridge construction.

It is recommended that additional work be undertaken to delineate the extent and depth of soft to firm soils encountered in bores drilled near bridge approach embankments, and the potential impact of soil profiles on construction period settlements and settlements over longer periods.

6.3 Bridge Foundations

6.3.1 General

It is considered that both bridge sites offer similar ground conditions, and hence similar options for foundation construction.

The results of the investigation indicate the presence of granular materials, including potential oversized materials such as cobbles and boulders. Figures 6.1 and 6.2 below show the river conditions and oversized material at river level at the lower crossing and upper crossing, respectively.



Figure 6.1 – River bedload and rock exposures at the Lower Crossing site

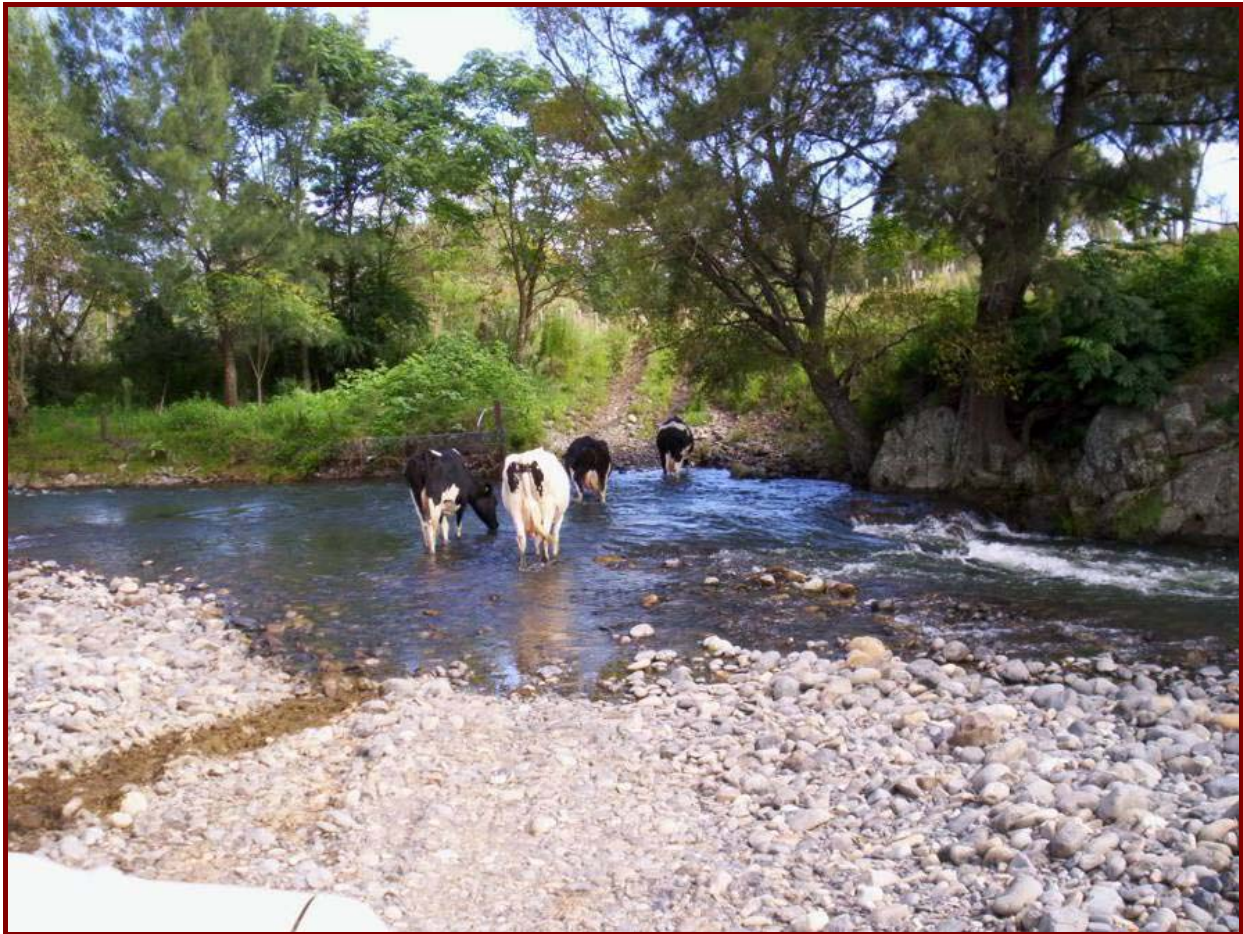


Figure 6.2 – Gravel, cobbles and boulders in the river bed at the Upper Crossing site

Six of the eight cored bores drilled at bridge sites required the use of percussion drilling/casing advance methods in order to penetrate the gravelly layers and achieve the target depths.

The presence of the oversized materials presents challenges for all types of pile installation. Traditional bored piles could refuse on oversized cobbles or boulders and hence may refuse on material that is not ideal for bridge foundations. Bored piles could also result in ground disturbance beyond the pile shaft if oversized materials prove difficult to drill. Down-hole hammer/percussion drilling techniques along with casing will likely be required if bored piles are considered.

Driven piles could similarly refuse on oversized material, however they present the opportunity to achieve a certain amount of capacity through shaft friction during the pile driving exercise, and

do not rely on end bearing alone. The monitoring of the installation process also provides the opportunity to correlate installation records with pile capacities using an appropriate dynamic theory.

It is possible that the pile installation depth may be governed by the anticipated scour depth, and hence this may also determine the appropriate pile type.

6.3.2 Suitable Pile Types

Due to the presence of saturated sand and gravel within the profiles, unsupported bored piles are not suitable. Suitable pile types, along with their potential benefits and limitations are as follows:

- **Cased Bored Piles:** Cased bored piles will likely require the use of down the hole hammer/percussion drilling techniques to penetrate the expected gravelly conditions. Casing will be required to prevent the collapse of the pile hole during drilling. Shaft adhesion should be ignored within the cased section of the pile. If the base of the pier hole cannot be checked, then the design pressures should be reduced by 20%;
- **Driven Piles:** Select driven pile types, such as steel H-sections, would generally drive with relative ease through the overlying soils, however rates of penetration could slow within the gravelly layers. There is also a risk of damage to the pile during installation due to the gravelly soils. Penetration into the underlying high strength rock is likely to be minimal. The geotechnical capacity of piles driven to refusal approaches the structural capacity of the pile, which is dependent on the pile type and the area of the section used.

Therefore, if driven piles are considered, pile selection will most likely be more a function of the structural capacity of the respective piles than the ground conditions, although the pile lengths will need to account for the depth to the stratum in which a design set is anticipated.

6.3.3 Typical Pile Capacities

Typical pile structural capacities for a range of pile types are shown in Table 6.2 below.

Table 6.2 – Approximate Typical Allowable Pile Capacities for Driven Steel H-Section Piles

Pile Type	Shape	Dimensions (mm)	Approximate Typical Allowable Structural Load ⁽¹⁾ (kN)
Driven Steel Universal Column	H-Section	200 UC 59.5	Section Depth – 210 mm Flange Width – 205 mm 1150
		250 UC 89.5	Section Depth – 260 mm Flange Depth – 256 mm 1750
		310 UC 118	Section Depth – 315 mm Flange Depth – 307 mm 2300
		310 UC 158	Section Depth – 327 mm Flange Depth – 311 mm 3100

Notes to Table 6.2:

⁽¹⁾ These values should be confirmed by piling contractors for individual pile product types

Alternatively, the parameters presented in Table 6.3 may be used as a guide for cased bored piles founded in the underlying rock.

Table 6.3 – Allowable Shaft Adhesion and End Bearing Pressures by Rock Class

Rock Class ⁽¹⁾	Allowable Shaft Adhesion for Rock Sockets (kPa)	Allowable End Bearing Pressure (MPa)
Class V	50	1.0
Class IV	150	1.5
Class III	150	3.5
Class II	350	6.0
Class I	350	12.0

Notes to Table 6.3:

⁽¹⁾ Rock class for sandstones, as defined in Pells, Douglas et al, (Ref 3) – equivalent classes for shale are one class higher (for example, Class II Sandstone ≡ Class I Shale)

Design parameters for the capacity of bridge foundations to resist lateral loading at each of the two crossing sites were presented in separate reports, 39721.03-01L (Upper Williams River, dated 23 June 2008) and 39721.03-02L (Lower Williams River, dated 15 October 2008).

Table 6.4 below summarises the approximate depth to the top of each of the above rock classes, based on the cored boreholes drilled at each bridge site.

Table 6.4 – Approximate Depth to Top of Each Rock Class at Core Bore Locations

Location	Approx. Chainage (E2)	Offset (E2)	Corresponding Bore	Approx. Depth to Top of Rock Class ⁽¹⁾ (m)				
				Class V	Class IV	Class III	Class II	Class I
Lower Bridge	900	5 m right	410	-	-	5.85	9.32	-
	1050	10 m right	401A	-	-	7.6	8.25	10.8
	1185	10 m right	402	-	-	7.75	8.4	10.2
	1235	5 m left	405B	-	5.0	-	6.85	-
Upper Bridge	16090	15 m right	406	-	-	4.9	5.2	-
	16200	5 m right	403	-	5.8	6.65	9.0 ⁽²⁾	-
	16280	10 m right	404A	4.0	-	-	5.4	8.5
	16365	10 m left	409	5.0	-	5.5 ⁽³⁾	-	-

Notes to Table 6.4:

- ⁽¹⁾ Rock classes for sandstone, as defined in Pells, Douglas et al, (Ref 3) – equivalent classes for shale are one class higher (for example, Class II Sandstone ≡ Class I Shale)
- ⁽²⁾ But contains some highly fractured to fragmented/Class III zones
- ⁽³⁾ But contains some fragmented/Class IV zones.

It is noted that the class of rock, as defined in Pells, Douglas et al (Ref 3) is not strictly relevant to driven piles, as the pile capacity is proven by the installation method and the opportunity to apply dynamic testing, such as wave equation analysis. The resulting bearing pressures achieved are likely to be somewhat higher than indicated in Ref 3, particularly if there is significant embedment into the rock strata. Notwithstanding the above, piling contractors should be responsible for selecting an appropriate piling method with reference to the ground conditions, and the pile capacities required for design.

If ground conditions prove challenging for pile construction, it may be prudent to consider pile load testing during construction.

6.3.4 Pile Durability

The chemical aggressiveness of soil or groundwater towards buried structures was not assessed as part of this investigation.

A review of the proposed design should be undertaken before construction commences to determine whether additional soil testing is required and the need for corrosion protection measures.

6.4 Seismicity

Australian Standard AS 5100.2–2004 (Ref 4) uses factors from the now superseded AS 1170.4–1993 (Ref 5) to calculate the effect of seismic loads on bridges. Factors required from the superseded AS1170.4–1993 for bridge earthquake design are:

- Site factor (S);
- Acceleration coefficient (a).

Design procedures in the new earthquake code AS 1170.4–2007 (Ref 6) replace the factors required by AS 5100.2 for bridge earthquake design. From a geotechnical perspective, the new earthquake code differs from AS 1170.4–1993 in the following ways:

- Acceleration coefficient (a) has been replaced with hazard factor (Z);
- Soil profile descriptors used for determination of site factor (S) have been replaced with five new site sub-soil classes.

According to AS 1170.4–2007, a Site Sub-soil Class of C_e is considered to be appropriate if the depth of soil and/or weathered rock with UCS less than 1 MPa exceeds 3 m below the ground surface. Factors required for bridge earthquake design are presented in Table 6.5 below.

Table 6.5 – Factors for Bridge Earthquake Design

Bridge Site	AS 1170.4–1993 (superseded)		AS 1170.4–2007	
	Acceleration Coefficient (a)	Site Factor (S)	Hazard Factor (Z)	Site Sub-Soil Class
Lower Bridge/Crossing	0.9	0.67	0.9	C_e
Upper Bridge/Crossing	0.9	0.67	0.9	C_e

7. COMMENTS – FURTHER INVESTIGATION

Douglas Partners recommends that further drilling investigations be carried out at a number of cuttings along the proposed route not yet subject to subsurface investigation. Seismic survey would also provide additional information on excavatability in selected cuttings, provided it can be correlated with borehole data and site survey data.

Various suites of laboratory testing have also been recommended for assessment of the following:

- Durability of rock to be exposed in cut batters;
- Durability of rock material won for embankment formations in rockfill;
- Potential acid generation due to exposure and oxidation of soil and rock;
- Corrosion potential of soil and groundwater towards buried concrete structures at the proposed bridge sites.

Furthermore, delineation of potentially soft to firm soils is recommended along the western approach embankment to the proposed upper river crossing, so that potential settlements can be assessed.

It may be possible to refine batter slope design with additional cored boreholes and down-hole camera assessment of joint orientation. Assessment of the weathering potential of the various rock types may assist with selection of batter slopes and/or construction treatment. Drilling of additional boreholes as part of geotechnical investigations for the detailed design phase should also allow for the installation of piezometers to monitor groundwater levels.

Douglas Partners is proceeding with additional field sampling and laboratory testing to assess likely subgrade conditions for pavement construction, including in the vicinity of cut to fill transitions where profile variations from filling through natural surface soils to weathered rock will be encountered along the alignment and across the road formation. It is expected that pavement thickness design for the proposed new road will be completed as part of the detail design phase investigations on the basis of all the data collected at that time.

If the sourcing of pavement materials from the area around the proposed road alignment is to be considered further, then additional detailed material source investigation and laboratory testing will also be required. It is anticipated this will be a subject of enquiry dealt with in parallel to investigation of the route corridor.

A substantial amount of data has been collected from additional investigation work carried out to date in line with the above recommendations. The results and interpretations of this work will be reported in the detailed design phase of the investigations.

It should be noted that all investigation work is subject to the clemency of the prevailing weather conditions. Investigations by Douglas Partners in the localities around the project area have been hampered by sustained periods of rainfall that make access to the project area unsafe, and have resulted in delays to the work program. The impact of these constraints on all field work should be borne in mind during ongoing investigations, and should also be a consideration for the purposes of construction planning.

8. LIMITATIONS

Conditions on site different to those identified during this assessment may exist. Therefore Douglas Partners cannot provide unqualified warranties nor does Douglas Partners assume any liability for site conditions not recorded in the data available for this study.

This report and associated documentation and the information herein have been prepared solely for the use of Opus International Consultants (NSW) Pty Ltd and Hunter Water Corporation. Any reliance assumed by other parties on this report shall be at such party's own risk. Any ensuing liability resulting from use of the report by other parties cannot be transferred to Douglas Partners.

DOUGLAS PARTNERS PTY LTD

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6. Australian Standard AS 1170.4–2007 “Earthquake actions in Australia” Standards Australia.

NOTES RELATING TO THIS REPORT

Introduction

These notes have been provided to amplify the geotechnical report in regard to classification methods, specialist field procedures and certain matters relating to the Discussion and Comments section. Not all, of course, are necessarily relevant to all reports.

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

Description and Classification Methods

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

Soil types are described according to the predominating particle size, qualified by the grading of other particles present (eg. sandy clay) on the following bases:

Soil Classification	Particle Size
Clay	less than 0.002 mm
Silt	0.002 to 0.06 mm
Sand	0.06 to 2.00 mm
Gravel	2.00 to 60.00 mm

Cohesive soils are classified on the basis of strength either by laboratory testing or engineering examination. The strength terms are defined as follows.

Classification	Undrained Shear Strength kPa
Very soft	less than 12
Soft	12—25
Firm	25—50
Stiff	50—100
Very stiff	100—200
Hard	Greater than 200

Non-cohesive soils are classified on the basis of relative density, generally from the results of standard penetration tests (SPT) or Dutch cone penetrometer tests (CPT) as below:

Relative Density	SPT "N" Value (blows/300 mm)	CPT Cone Value (q_c — MPa)
Very loose	less than 5	less than 2
Loose	5—10	2—5
Medium dense	10—30	5—15
Dense	30—50	15—25
Very dense	greater than 50	greater than 25

Rock types are classified by their geological names. Where relevant, further information regarding rock classification is given on the following sheet.

Sampling

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

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

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

Details of the type and method of sampling are given in the report.

Drilling Methods.

The following is a brief summary of drilling methods currently adopted by the Company and some comments on their use and application.

Test Pits — these are excavated with a backhoe or a tracked excavator, allowing close examination of the in-situ soils if it is safe to descent into the pit. The depth of penetration is limited to about 3 m for a backhoe and up to 6 m for an excavator. A potential disadvantage is the disturbance caused by the excavation.

Large Diameter Auger (eg. Pengo) — the hole is advanced by a rotating plate or short spiral auger, generally 300 mm or larger in diameter. The cuttings are returned to the surface at intervals (generally of not more than 0.5 m) and are disturbed but usually unchanged in moisture content. Identification of soil strata is generally much more reliable than with continuous spiral flight augers, and is usually supplemented by occasional undisturbed tube sampling.

Continuous Sample Drilling — the hole is advanced by pushing a 100 mm diameter socket into the ground and withdrawing it at intervals to extrude the sample. This is the most reliable method of drilling in soils, since moisture content is unchanged and soil structure, strength, etc. is only marginally affected.

Continuous Spiral Flight Augers — the hole is advanced using 90—115 mm diameter continuous spiral flight augers which are withdrawn at intervals to allow sampling or in-situ testing. This is a relatively economical means of drilling in clays and in sands above the water

table. Samples are returned to the surface, or may be collected after withdrawal of the auger flights, but they are very disturbed and may be contaminated. Information from the drilling (as distinct from specific sampling by SPTs or undisturbed samples) is of relatively lower reliability, due to remoulding, contamination or softening of samples by ground water.

Non-core Rotary Drilling — the hole is advanced by a rotary bit, with water being pumped down the drill rods and returned up the annulus, carrying the drill cuttings. Only major changes in stratification can be determined from the cuttings, together with some information from 'feel' and rate of penetration.

Rotary Mud Drilling — similar to rotary drilling, but using drilling mud as a circulating fluid. The mud tends to mask the cuttings and reliable identification is again only possible from separate intact sampling (eg. from SPT).

Continuous Core Drilling — a continuous core sample is obtained using a diamond-tipped core barrel, usually 50 mm internal diameter. Provided full core recovery is achieved (which is not always possible in very weak rocks and granular soils), this technique provides a very reliable (but relatively expensive) method of investigation.

Standard Penetration Tests

Standard penetration tests (abbreviated as SPT) are used mainly in non-cohesive soils, but occasionally also in cohesive soils as a means of determining density or strength and also of obtaining a relatively undisturbed sample. The test procedure is described in Australian Standard 1289, "Methods of Testing Soils for Engineering Purposes" — Test 6.3.1.

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

The test results are reported in the following form.

- In the case where full penetration is obtained with successive blow counts for each 150 mm of say 4, 6 and 7
as 4, 6, 7
N = 13
- In the case where the test is discontinued short of full penetration, say after 15 blows for the first 150 mm and 30 blows for the next 40 mm
as 15, 30/40 mm.

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

Occasionally, the test method is used to obtain samples in 50 mm diameter thin walled sample tubes in clays. In such circumstances, the test results are shown on the borelogs in brackets.

Cone Penetrometer Testing and Interpretation

Cone penetrometer testing (sometimes referred to as Dutch cone — abbreviated as CPT) described in this report has been carried out using an electrical friction cone penetrometer. The test is described in Australian Standard 1289, Test 6.4.1.

In the tests, a 35 mm diameter rod with a cone-tipped end is pushed continuously into the soil, the reaction being provided by a specially designed truck or rig which is fitted with an hydraulic ram system. Measurements are made of the end bearing resistance on the cone and the friction resistance on a separate 130 mm long sleeve, immediately behind the cone. Transducers in the tip of the assembly are connected by electrical wires passing through the centre of the push rods to an amplifier and recorder unit mounted on the control truck.

As penetration occurs (at a rate of approximately 20 mm per second) the information is plotted on a computer screen and at the end of the test is stored on the computer for later plotting of the results.

The information provided on the plotted results comprises: —

- Cone resistance — the actual end bearing force divided by the cross sectional area of the cone — expressed in MPa.
- Sleeve friction — the frictional force on the sleeve divided by the surface area — expressed in kPa.
- Friction ratio — the ratio of sleeve friction to cone resistance, expressed in percent.

There are two scales available for measurement of cone resistance. The lower scale (0—5 MPa) is used in very soft soils where increased sensitivity is required and is shown in the graphs as a dotted line. The main scale (0—50 MPa) is less sensitive and is shown as a full line.

The ratios of the sleeve friction to cone resistance will vary with the type of soil encountered, with higher relative friction in clays than in sands. Friction ratios of 1%—2% are commonly encountered in sands and very soft clays rising to 4%—10% in stiff clays.

In sands, the relationship between cone resistance and SPT value is commonly in the range:—

$$q_c \text{ (MPa)} = (0.4 \text{ to } 0.6) N \text{ (blows per 300 mm)}$$

In clays, the relationship between undrained shear strength and cone resistance is commonly in the range:—

$$q_c = (12 \text{ to } 18) c_u$$

Interpretation of CPT values can also be made to allow estimation of modulus or compressibility values to allow calculation of foundation settlements.

Inferred stratification as shown on the attached reports is assessed from the cone and friction traces and from experience and information from nearby boreholes, etc. This information is presented for general guidance, but must be regarded as being to some extent interpretive. The test method provides a continuous profile of engineering properties, and where precise information on soil classification is required, direct drilling and sampling may be preferable.

Hand Penetrometers

Hand penetrometer tests are carried out by driving a rod into the ground with a falling weight hammer and measuring the blows for successive 150 mm increments of penetration. Normally, there is a depth limitation of 1.2 m but this may be extended in certain conditions by the use of extension rods.

Two relatively similar tests are used.

- Perth sand penetrometer — a 16 mm diameter flat-ended rod is driven with a 9 kg hammer, dropping 600 mm (AS 1289, Test 6.3.3). This test was developed for testing the density of sands (originating in Perth) and is mainly used in granular soils and filling.
- Cone penetrometer (sometimes known as the Scala Penetrometer) — a 16 mm rod with a 20 mm diameter cone end is driven with a 9 kg hammer dropping 510 mm (AS 1289, Test 6.3.2). The test was developed initially for pavement subgrade investigations, and published correlations of the test results with California bearing ratio have been published by various Road Authorities.

Laboratory Testing

Laboratory testing is carried out in accordance with Australian Standard 1289 "Methods of Testing Soil for Engineering Purposes". Details of the test procedure used are given on the individual report forms.

Bore Logs

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

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

Ground Water

Where ground water levels are measured in boreholes, there are several potential problems;

- In low permeability soils, ground water although present, may enter the hole slowly or perhaps not at all during the time it is left open.
- A localised perched water table may lead to an erroneous indication of the true water table.
- Water table levels will vary from time to time with seasons or recent weather changes. They may not be

the same at the time of construction as are indicated in the report.

- The use of water or mud as a drilling fluid will mask any ground water inflow. Water has to be blown out of the hole and drilling mud must first be washed out of the hole if water observations are to be made.

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

Engineering Reports

Engineering reports are prepared by qualified personnel and are based on the information obtained and on current engineering standards of interpretation and analysis. Where the report has been prepared for a specific design proposal (eg. a three storey building), the information and interpretation may not be relevant if the design proposal is changed (eg. to a twenty storey building). If this happens, the Company will be pleased to review the report and the sufficiency of the investigation work.

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

- unexpected variations in ground conditions — the potential for this will depend partly on bore spacing and sampling frequency
- changes in policy or interpretation of policy by statutory authorities
- the actions of contractors responding to commercial pressures.

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

Site Anomalies

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

Reproduction of Information for Contractual Purposes

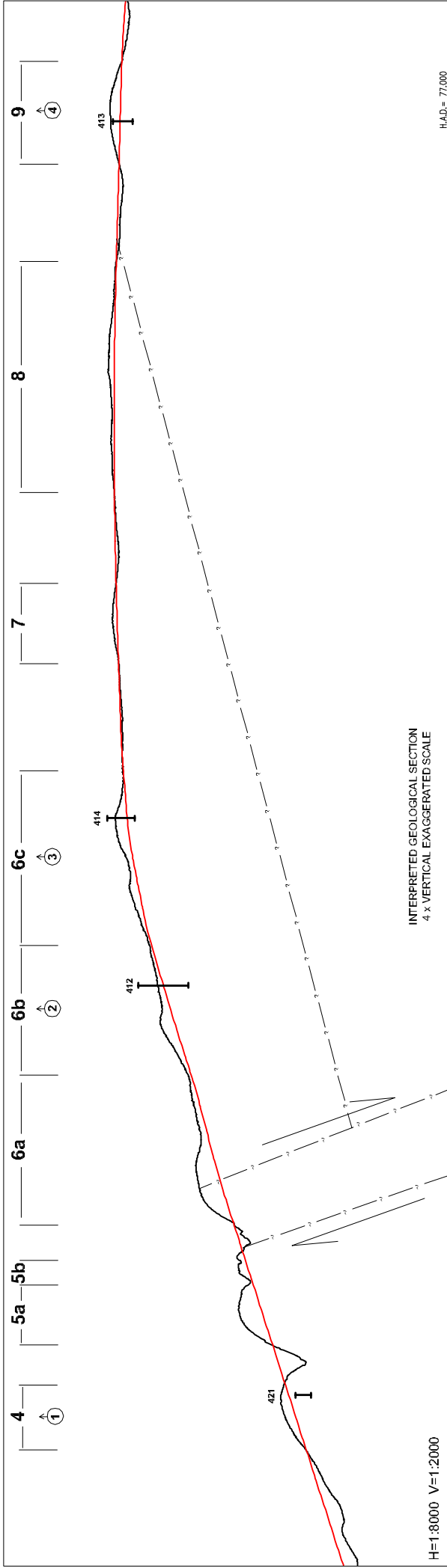
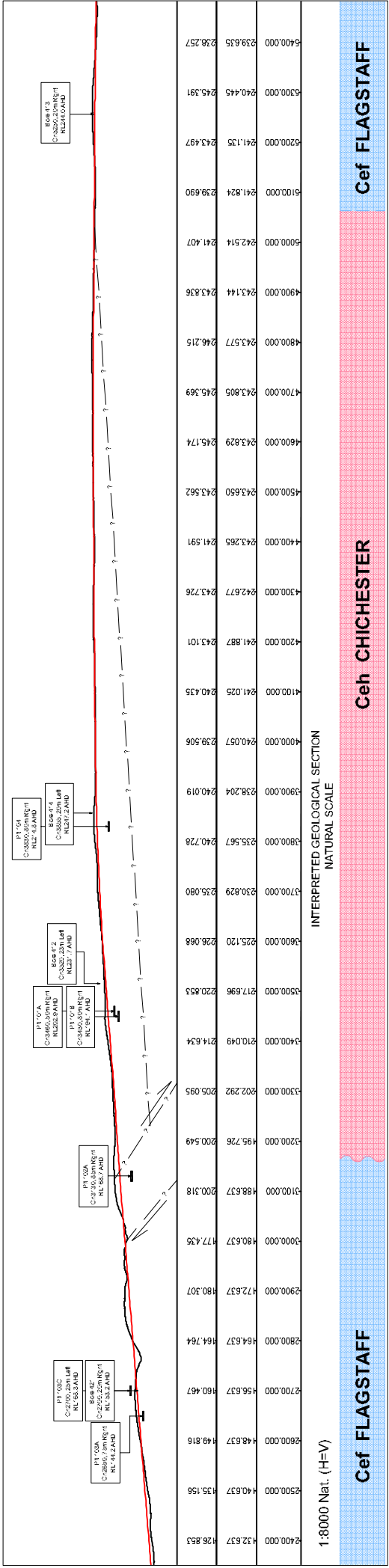
Attention is drawn to the document "Guidelines for the Provision of Geotechnical Information in Tender Documents", published by the Institution of Engineers, Australia. Where information obtained from this investigation is provided for tendering purposes, it is recommended that all information, including the written report and discussion, be made available. In circumstances where the discussion or comments section

is not relevant to the contractual situation, it may be appropriate to prepare a specially edited document. The Company would be pleased to assist in this regard and/or to make additional report copies available for contract purposes at a nominal charge.

Site Inspection

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

Copyright © 1998 Douglas Partners Pty Ltd



H=1:8000 V=1:2000

426.853	432.637	448.637	449.816	456.637	464.457	472.637	480.307	477.436	488.637	500.000	480.637	510.000	488.637	500.518	520.000	495.726	520.549	530.000	502.292	530.095	540.000	510.049	540.634	550.000	517.696	550.853	560.000	525.120	560.068	570.000	530.829	575.080	580.000	535.567	580.728	590.000	538.204	590.019	600.000	540.057	600.506	610.000	541.025	610.435	620.000	541.887	620.101	630.000	542.677	630.726	640.000	543.265	641.591	650.000	543.650	643.562	660.000	543.829	645.174	670.000	543.805	645.369	680.000	543.577	646.215	690.000	543.144	643.836	700.000	542.514	641.407	710.000	541.824	639.690	720.000	541.135	643.497	730.000	540.445	645.391	740.000	539.635	640.000	750.000	538.257	640.000
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CUTTINGS

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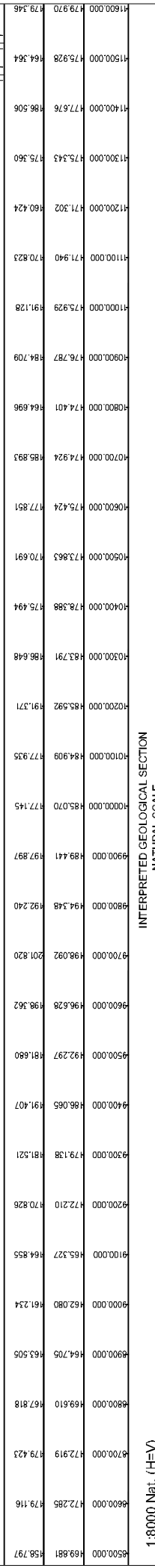
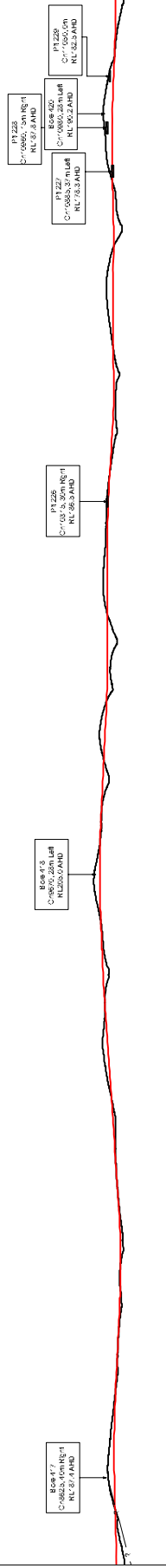
APPARENT DIRECTION AND DIP OF BEDDING

INTERPRETED GEOLOGICAL BOUNDARY

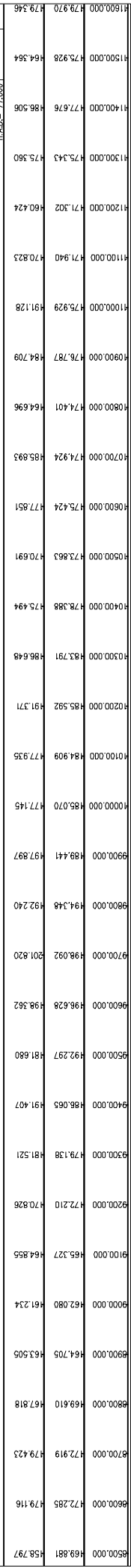
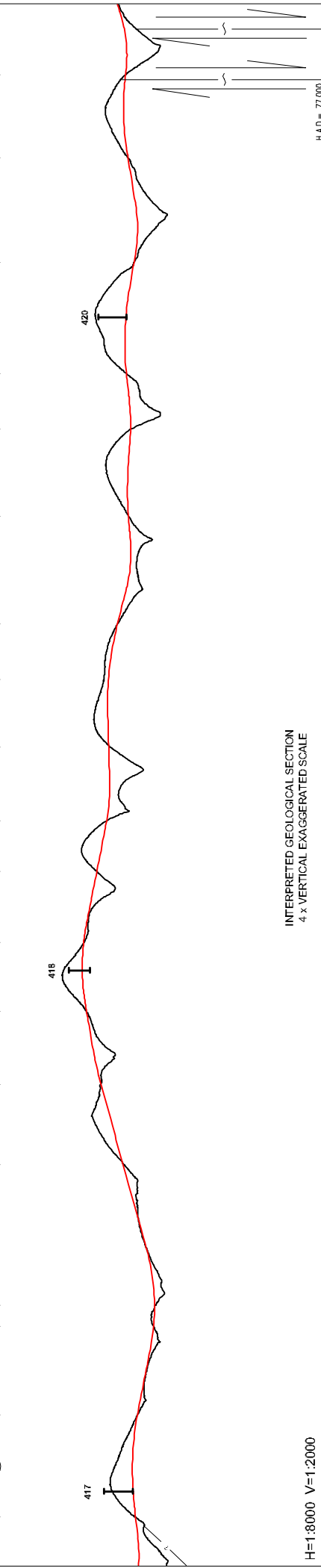
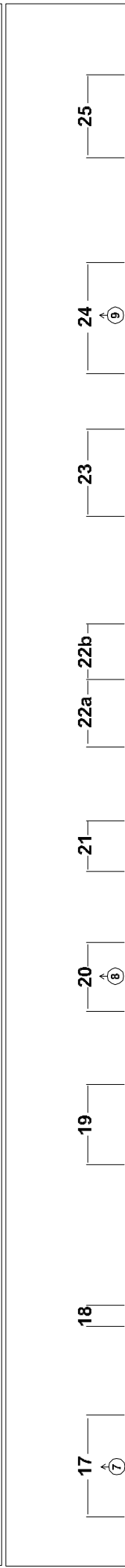
INTERPRETED BLOCK MOVEMENT ZONE

STRATIGRAPHIC DISCONFORMITY


DRAWING ADAPTED FROM E2 ALIGNMENT LONG SECTIONS PROVIDED BY OPUS INTERNATIONAL CONSULTANTS (NSW) PTY LTD, DATED 5/5/08





Ceq BONNINGTON





LEGEND


 CUTTINGS


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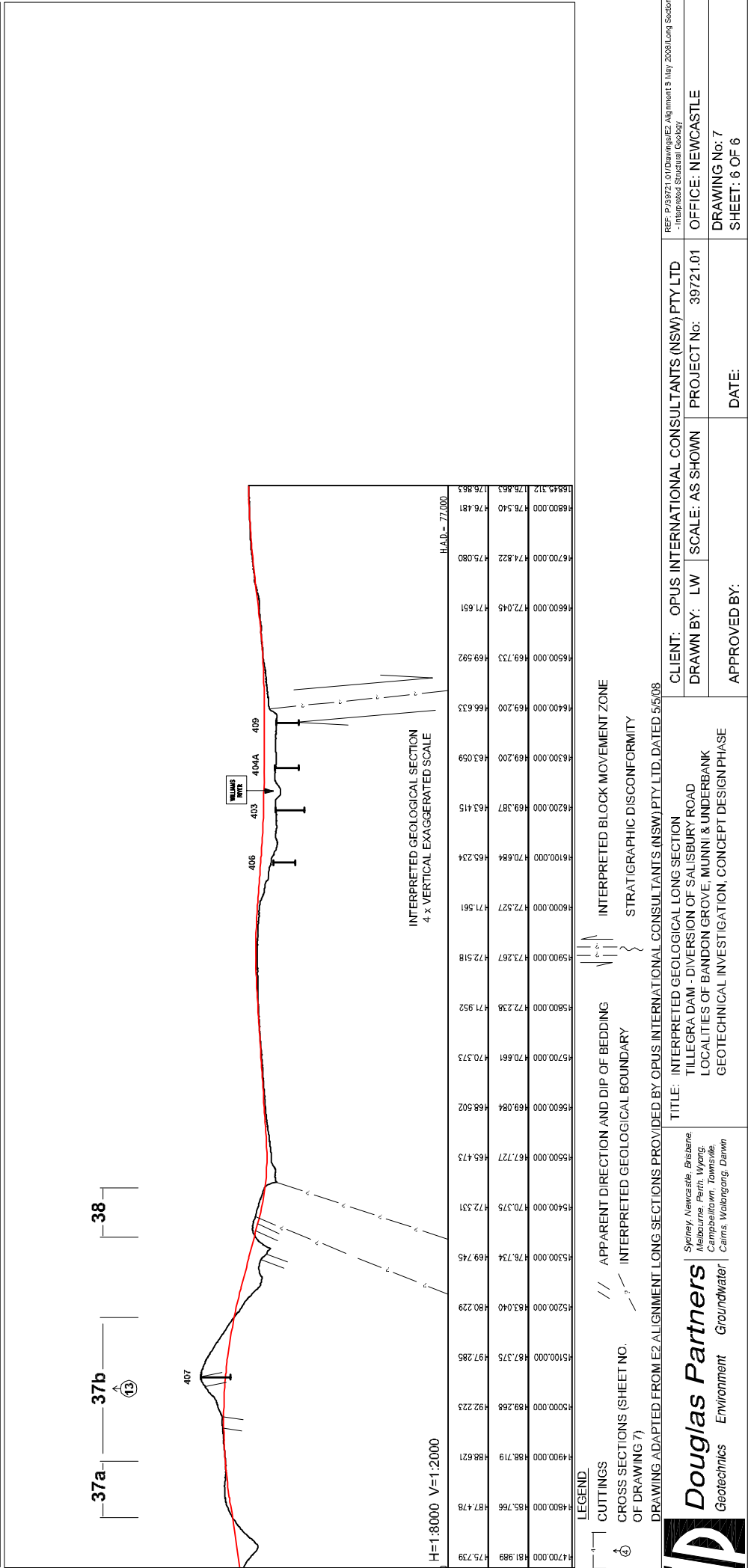
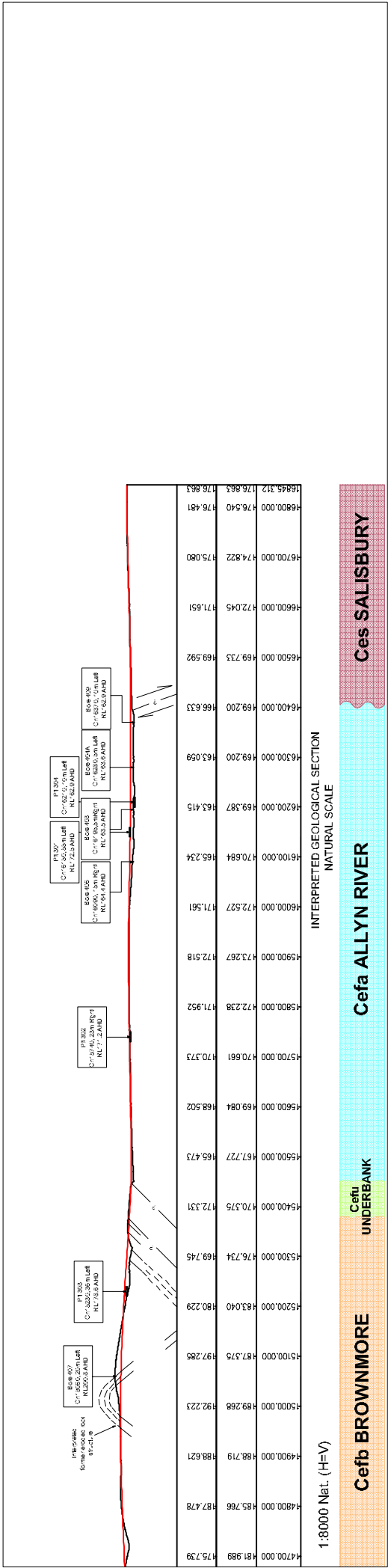
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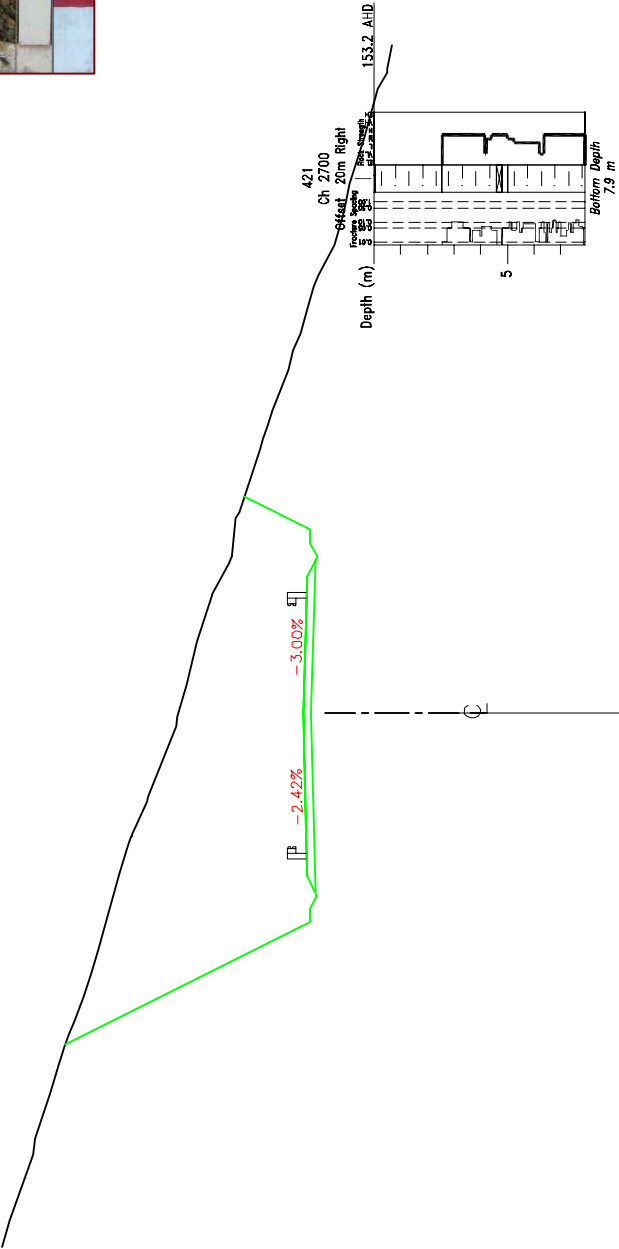
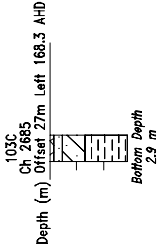
 INTERPRETED GEOLOGICAL BOUNDARY

 INTERPRETED BLOCK MOVEMENT ZONE

 STRATIGRAPHIC DISCONFORMITY

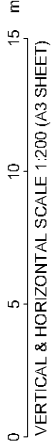
 Douglas Partners Geotechnics Environment Groundwater	Sydney, Newcastle, Brisbane, Melbourne, Perth, Wuyang, Campbelltown, Townsville, Cairns, Wollongong, Darwin	TITLE: INTERPRETED GEOLOGICAL LONG SECTION TILLEGRA DAM - DIVERSION OF SALISBURY ROAD LOCALITIES OF BANDON GROVE, MUNNI & UNDERBANK GEOTECHNICAL INVESTIGATION, CONCEPT DESIGN PHASE		CLIENT: OPUS INTERNATIONAL CONSULTANTS (NSW) PTY LTD DRAWN BY: LW SCALE: AS SHOWN PROJECT No: 39721.01	REF: P19721.01(Drawing)E2 Alignment 1 May, 2008(1) Long Section - Interpreted Structural Geology
				OFFICE: NEWCASTLE	
			APPROVED BY:	DRAWING No: 7 SHEET: 4 OF 6	





DESIGN LEVEL	167.081	166.832	165.948	165.872	165.284	164.776	164.582	164.737	163.631	163.552	163.188	163.173	155.564	155.564	155.314	155.689	162.253	161.670	155.752	155.837	160.207	159.811	159.245	159.245	158.514	158.620	155.559	158.223	157.444	157.174	156.996	156.394	156.194	155.928	155.628	155.426	154.665	154.317	154.093	153.536	153.031																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																														
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- LEGEND
- Filling
 - Siltstone
 - Core Loss

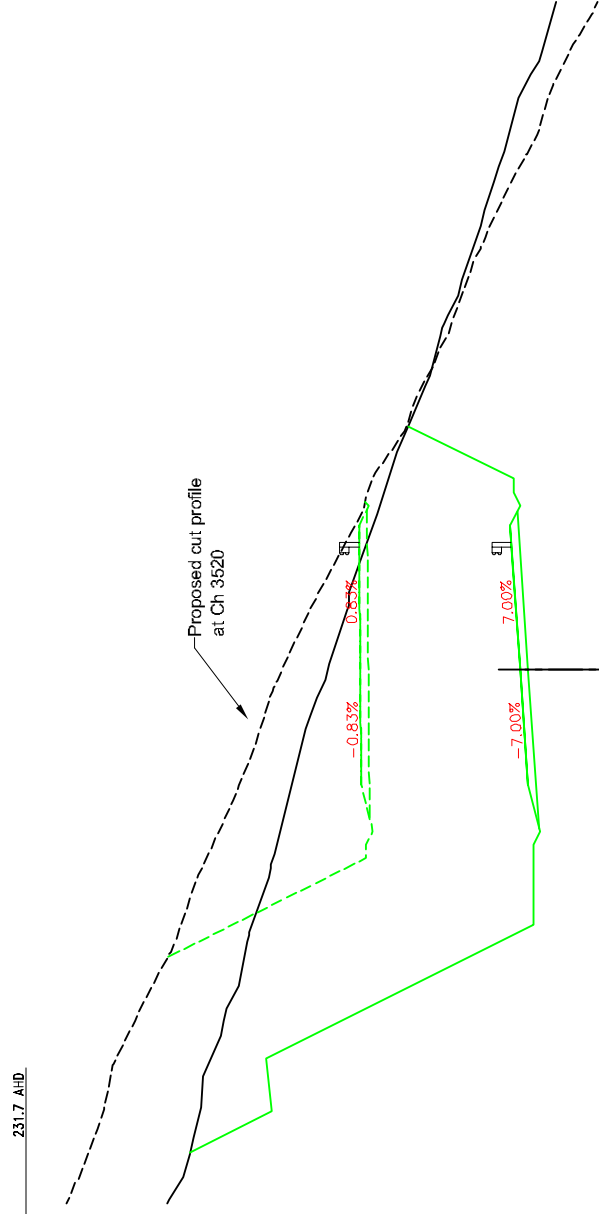
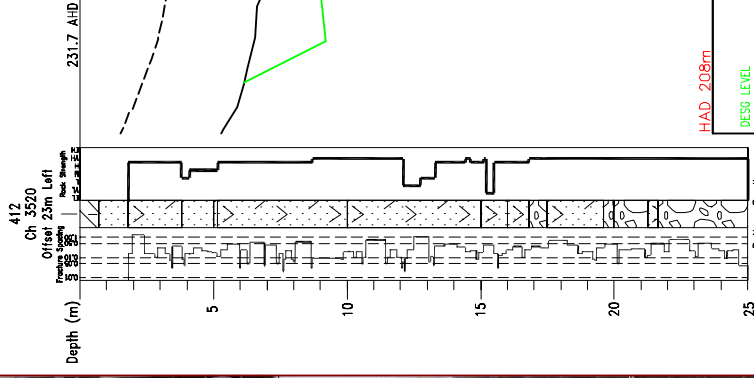


DRAWING ADAPTED FROM CROSS-SECTIONS PROVIDED BY OPUS INTERNATIONAL CONSULTANTS (NSW) PTY LTD, DATED 13/5/08







DUNGOG 39721.01 BH 412 4 MAR 2008

START CORING AT 1.8m

[illegible]

ITEM 1	214.85																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																				
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LEGEND

-  Silty Clay
 Sandstone fine grained
 Tuff
 Conglomerate

DRAWING ADAPTED FROM CROSS-SECTIONS PROVIDED BY OPUS INTERNATIONAL CONSULTANTS (NSW) PTY LTD, DATED 13/5/08



Douglas Partners

Geotechnics Environment Groundwater

*Sydney, Newcastle, Brisbane,
Melbourne, Perth, Wyoing,
Campbelltown, Townsville,
Cairns, Wollongong, Darwin*

TITLE: CROSS-SECTION AT CUT 6B

**TILLEGRA DAM - DIVERSION OF SALISBURY ROAD
LOCALITIES OF BANDON GROVE, MUNNI & UNDERBANK
GEOTECHNICAL INVESTIGATION, CONCEPT DESIGN PHASE**

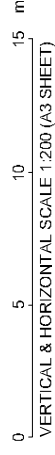
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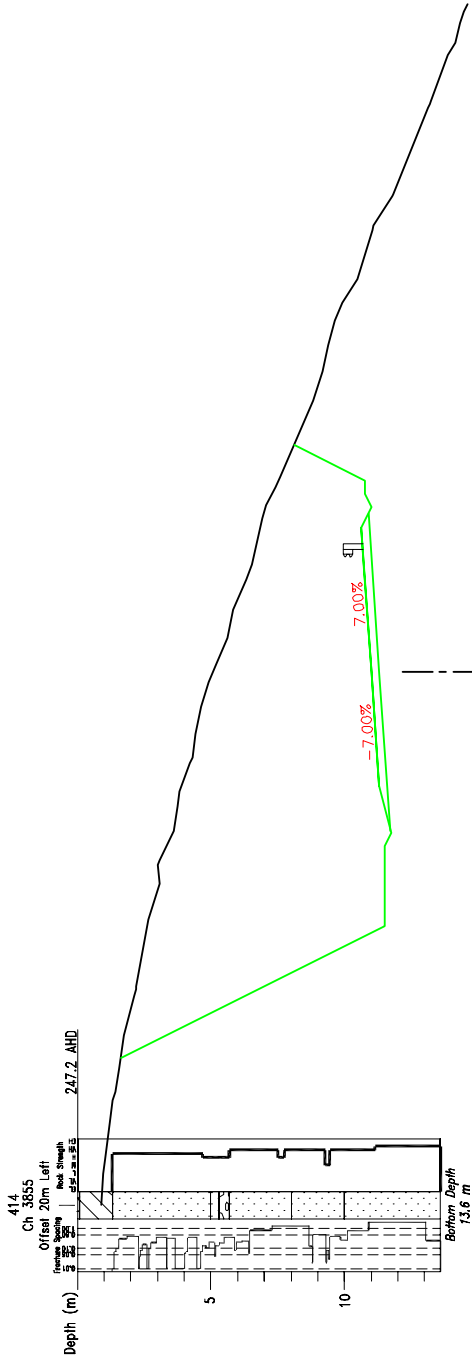
DRAWN BY: LW	SCALE: 1:200
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APPROVED BY:

OFFICE: NEWCASTLE

DRAWING No:	8
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DESIGN LEVEL	247.321	247.227	246.989	246.888	246.775	246.624	246.469	246.072	245.985	245.533	236.702	236.702	-9.550	-9.050	245.128	245.176	244.576	244.375	243.994	243.886	243.775	243.560	243.298	242.579	242.358	241.886	241.665	237.457	237.569	241.149	237.194	240.784	6.650	6.150	7.150	8.475	240.094				
EXIST LEVEL	247.321	247.227	246.989	246.888	246.775	246.624	246.469	246.072	245.985	245.533	236.702	236.702	-9.550	-9.050	245.128	245.176	244.576	244.375	243.994	243.886	243.775	243.560	243.298	242.579	242.358	241.886	241.665	237.457	237.569	241.149	237.194	240.784	6.650	6.150	7.150	8.475	240.094				
DESIGN OFFSET																																									

Ch 3850.000

LEGEND

- ☒ Topsoil
- ☒ Clay
- ☒ Sandstone fine grained
- ☒ Conglomerate

DRAWING ADAPTED FROM CROSS-SECTIONS PROVIDED BY OPUS INTERNATIONAL CONSULTANTS (NSW) PTY LTD, DATED 13/5/08

0 5 10 15 m
VERTICAL & HORIZONTAL SCALE 1:200 (A3 SHEET)



Douglas Partners
Geotechnics Environment Groundwater

Sydney, Newcastle, Brisbane,
Melbourne, Perth, Wyoing,
Campbelltown, Townsville,
Calms, Wollongong, Darwin

TITLE: CROSS-SECTION AT CUT 6C
TILLEGRA DAM - DIVERSION OF SALISBURY ROAD,
LOCALITIES OF BANDON GROVE, MUNNI & UNDERBANK
GEOTECHNICAL INVESTIGATION, CONCEPT DESIGN PHASE

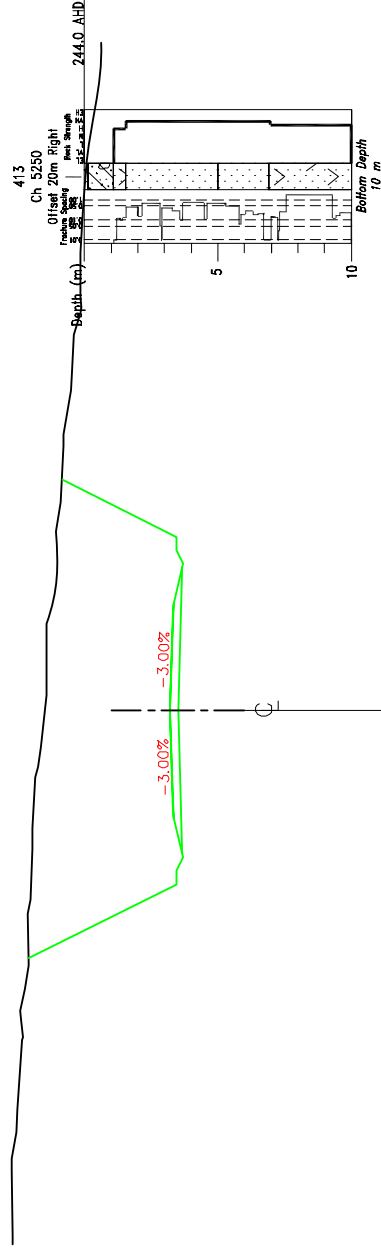
CLIENT: OPUS INTERNATIONAL CONSULTANTS (NSW) PTY LTD

DRAWN BY: LW SCALE: 1:200 PROJECT No: 39721.01

DATE: OFFICE: NEWCASTLE

DRAWING No: 8

SHEET 3 OF 13



DESIGN OFFSET	EXIST LEVEL	DESIGN LEVEL
246.681	246.681	
246.737	246.737	
246.531	246.531	
246.506	246.506	
246.318	246.318	
246.401	246.401	
246.224	246.224	
246.100	246.100	
-9.275	246.094	246.096
246.114	246.114	
-6.500	246.027	240.545
-6.000		240.545
-5.500		240.295
-4.000	245.935	240.685
-3.500	245.750	240.670
-0.000	245.857	240.790
	245.750	
	245.429	
3.500	245.411	240.685
4.000	245.026	240.295
5.500	245.023	240.545
6.000		240.545
8.645	244.877	244.835
	244.773	
	244.793	
	244.487	
	244.453	
	244.396	
	244.140	
	244.118	
	244.155	
	244.160	
	243.939	
	243.898	
	243.376	

HAD 230m

5250.000

LEGEND

- Topsoil
Gravel Sandy
Clay
Tuff
Sandstone fin

ped

DRAWING ADAPTED FROM CROSS-SECTIONS PROVIDED BY OPUS INTERNATIONAL CONSULTANTS (NSW) PTY LTD, DATED 13/5/08

CLIENT: OPUS INTERNATIONAL CONSULTANTS (NSW) PTY LTD

DRAWN BY: LW	SCALE: 1:200	PROJECT No: 39721.01
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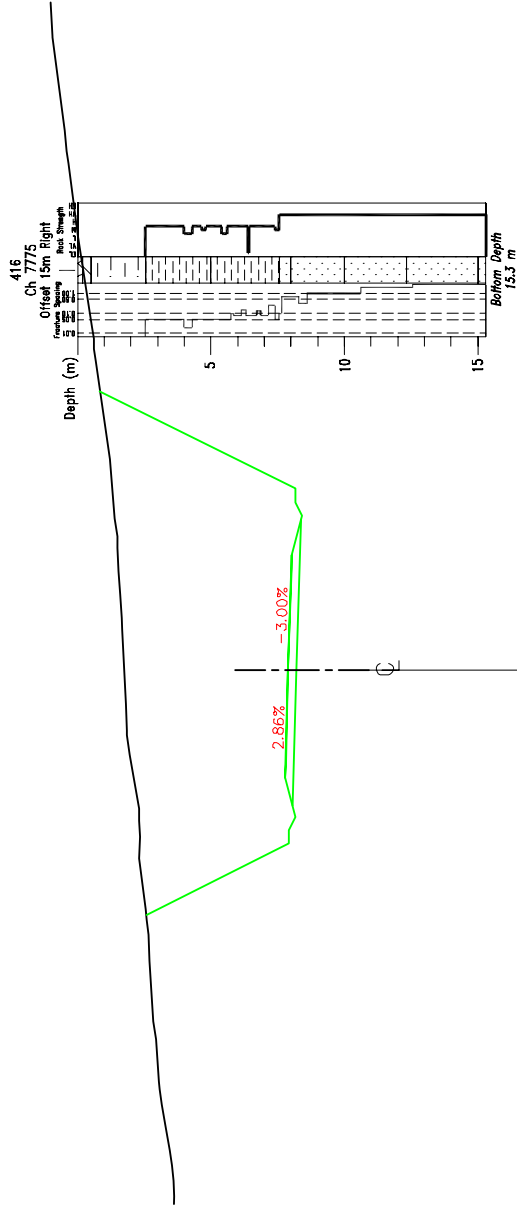
OFFICE: NEWCASTLE
DRAWING No: 8
SHEET 4 OF 13

APPROVED BY:

DATE: SHEET 4 OF 13

SHEET 4 OF 13

0 5 10 15 m
VERTICAL & HORIZONTAL SCALE 1:200 (A3 SHEET)



HAD 170m	
DESIGN LEVEL	188.066
EXIST LEVEL	189.910
DESIGN OFFSET	188.066

7770.000

LEGEND

- Topsoil
- Silty Clay
- Siltstone
- Shale
- Sandstone fine grained

0 5 10 15 m
VERTICAL & HORIZONTAL SCALE 1:200 (A3 SHEET)

DRAWING ADAPTED FROM CROSS-SECTIONS PROVIDED BY OPUS INTERNATIONAL CONSULTANTS (NSW) PTY LTD, DATED 13/5/08



Douglas Partners
Geotechnics Environment Groundwater

Sydney, Newcastle, Brisbane,
Melbourne, Perth, Wyoing,
Campbelltown, Townsville,
Calms, Wollongong, Darwin

TITLE: CROSS-SECTION AT CUT 15
TILLEGRA DAM - DIVERSION OF SAUSBURY ROAD,
LOCALITIES OF BANDON GROVE, MUNNIE & UNDERBANK
GEOTECHNICAL INVESTIGATION, CONCEPT DESIGN PHASE

CLIENT: OPUS INTERNATIONAL CONSULTANTS (NSW) PTY LTD

DRAWN BY: LW SCALE: 1:200 PROJECT No: 39721.01

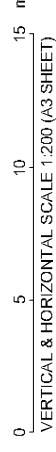
APPROVED BY: DATE: OFFICE: NEWCASTLE
DRAWING No: 8
SHEET 6 OF 13



HAD 165m

	Topsoil
	Silty Clay
	Siltstone
	Sandstone
	Mudstone

Sandstone fine grained
Mudstone



Douglas Partners
Geotechnics Environment Groundwater

*Sydney, Newcastle, Brisbane,
Melbourne, Perth, Wyong,
Campbelltown, Townsville,
Cairns, Wollongong, Darwin*

TITLE: CROSS-SECTION AT CUT 17
TILLEGRA DAM - DIVERSION OF SALISBURY ROAD
LOCALITIES OF BANDON GROVE, MUNNI & UNDERBANK
GEOTECHNICAL INVESTIGATION, CONCEPT DESIGN PHASE

CLIENT: OPUS INTERNATIONAL CONSULTANTS (NSW) PTY LTD

DRAWN BY: LW	SCALE: 1:200
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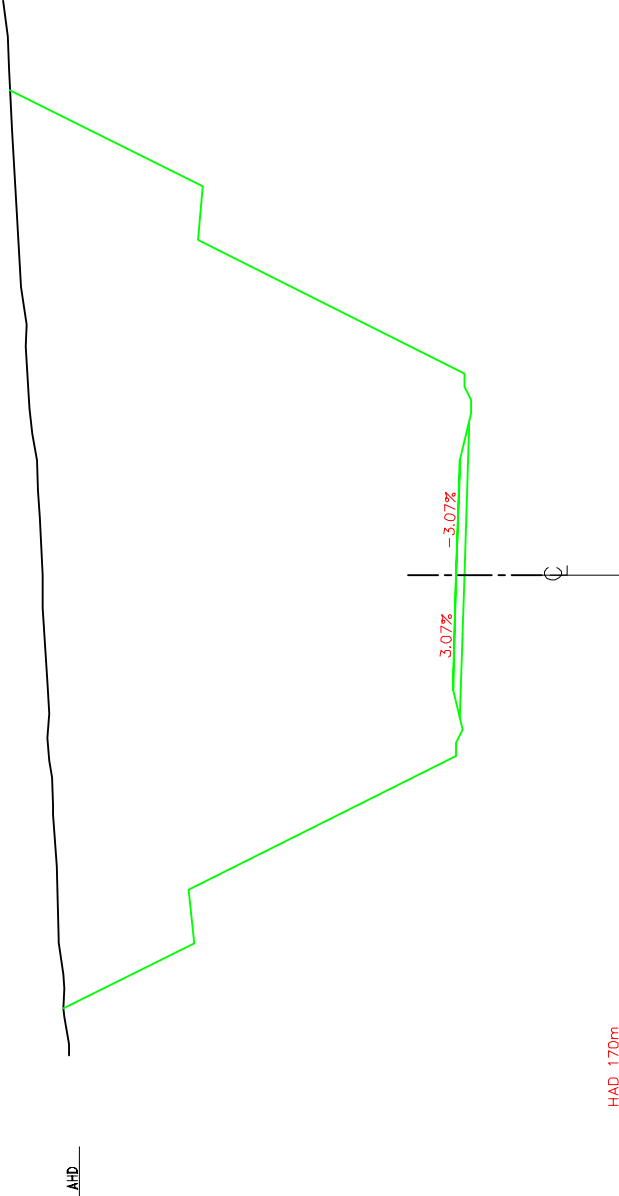
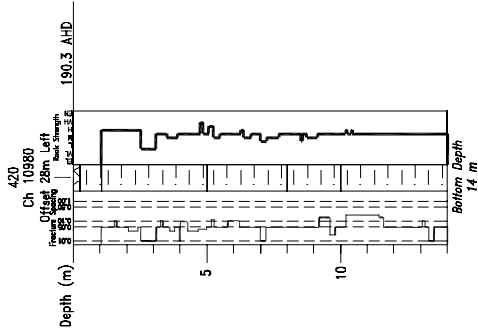
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PROJECT

OFFICE: NEWCASTLE

DRAWING No: 8

SHEET: 7 OF 13

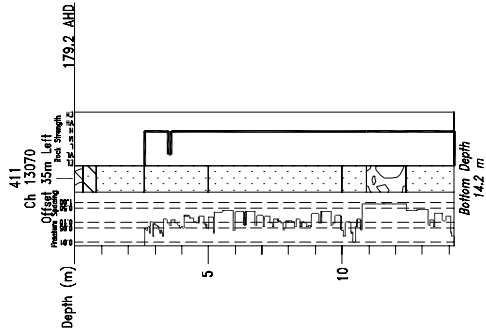


DESIGN LEVEL	190.840	190.703	190.852	190.898	190.868	190.894	191.082	191.093	191.158	191.282	191.286	191.335	191.448	191.448	191.495	191.427	191.517	191.517	191.680	191.661	191.766	191.864	191.897	192.056	192.171	192.197	192.550	192.766	192.809	192.882	192.964
EXIST LEVEL	190.840	190.703	190.852	190.898	190.868	190.894	191.082	191.093	191.158	191.282	191.286	191.335	191.448	191.448	191.495	191.427	191.517	191.517	191.680	191.661	191.766	191.864	191.897	192.056	192.171	192.197	192.550	192.766	192.809	192.882	192.964
DESIGN OFFSET		-16.247					-13.800	-11.800																							

LEGEND
Topsoil
Siltstone

VERTICAL & HORIZONTAL SCALE 1:200 (A3 SHEET)

DRAWING ADAPTED FROM CROSS-SECTIONS PROVIDED BY OPUS INTERNATIONAL CONSULTANTS (NSW) PTY LTD, DATED 13/5/08



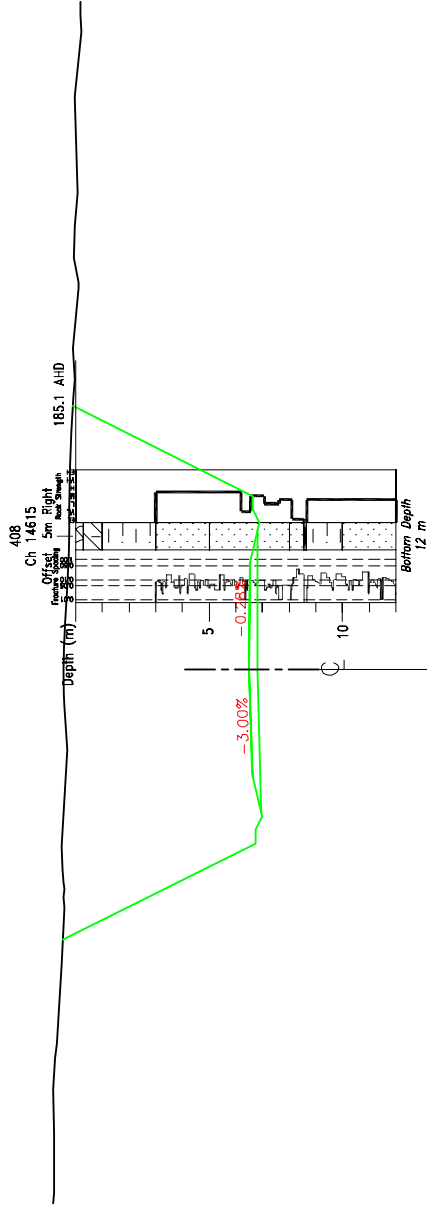
- LEGEND
- Topsoli
 - Sandy Clay
 - Sandstone fine grained
 - Conglomerate

HAD 160m

DESIGN LEVEL	177.518	177.426	177.347	177.246	177.241	177.066	170.851	171.226	171.338	171.583	171.89	176.137	175.893	175.604	171.828	171.963	171.548	171.428	171.738	171.302	174.832	174.601	174.575	174.594	174.814	174.950	174.837	174.904	175.291	175.622	176.272	176.690	177.027
EXIST LEVEL	177.518	177.426	177.347	177.246	177.241	177.066	170.851	171.226	171.338	171.583	171.89	176.137	175.893	175.604	171.828	171.963	171.548	171.428	171.738	171.302	174.832	174.601	174.575	174.594	174.814	174.950	174.837	174.904	175.291	175.622	176.272	176.690	177.027
DESIGN OFFSET	-10.058	-6.850	-6.350	-6.350	-6.850	-5.850	-5.100	-5.100	-3.500	0.000	0.000	0.000	0.000	0.000	3.500	4.000	5.500	5.500	6.000	6.300	7.982												

13070.000

DRAWING ADAPTED FROM CROSS-SECTIONS PROVIDED BY OPUS INTERNATIONAL CONSULTANTS (NSW) PTY LTD, DATED 13/5/08



DESIGN LEVEL	185.921	185.890	185.928	185.866	185.815	185.640	185.528	185.516	185.584	185.634	178.345	178.095	178.470	178.485	185.414	185.523	178.590	178.580	178.454	178.204	178.454	185.292	185.277	185.243	185.129	185.165	184.998	185.118	184.895	184.913
EXIST LEVEL	185.921	185.890	185.928	185.866	185.815	185.640	185.528	185.516	185.584	185.634	178.345	178.095	178.470	178.485	185.414	185.523	178.590	185.420	185.386	185.356	185.346	185.277	185.243	185.129	185.165	184.998	185.118	184.895	184.913	
DESIGN OFFSET	-10.123	-10.123	-10.123	-10.123	-10.123	-10.123	-10.123	-10.123	-10.123	-10.123	-10.123	-10.123	-10.123	-10.123	-10.123	-10.123	-10.123	-10.123	-10.123	-10.123	-10.123	-10.123	-10.123	-10.123	-10.123	-10.123	-10.123	-10.123	-10.123	

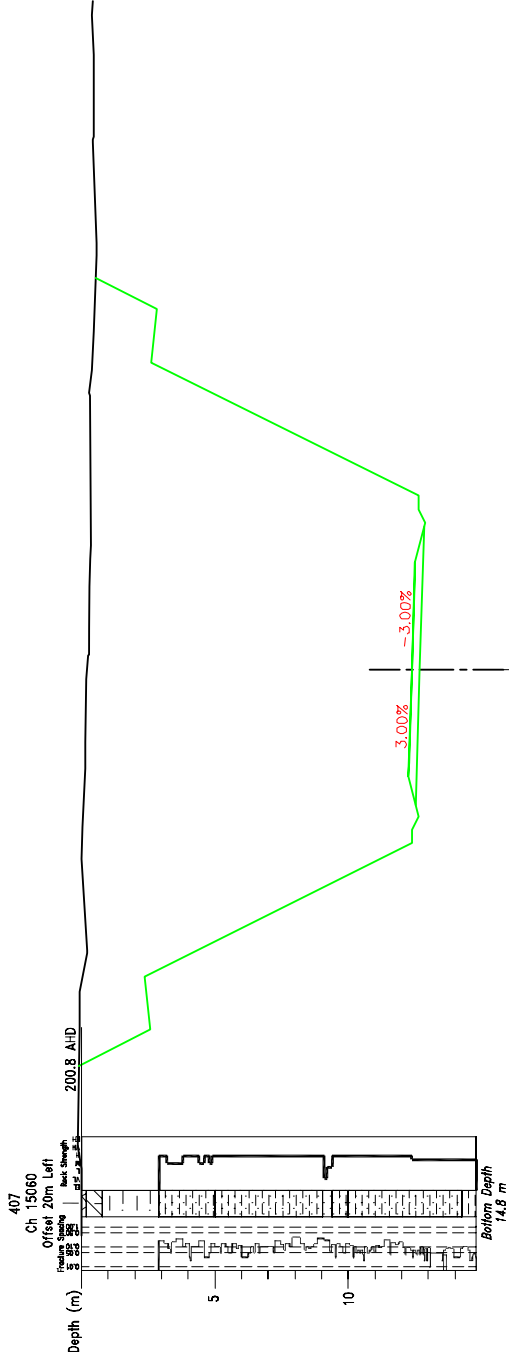
14610.000

- LEGEND:
- Topsoil
 - Silty Clay
 - Siltstone
 - Sandstone fine grained
 - Mudstone

Vertical & Horizontal Scale 1:200 (A3 Sheet)

0 5 10 15 m

DRAWING ADAPTED FROM CROSS-SECTIONS PROVIDED BY OPUS INTERNATIONAL CONSULTANTS (NSW) PTY LTD, DATED 13/5/08



HAD 180m		15060.000	
DESC LEVEL	200.946	200.801	200.946
EXIST LEVEL	200.946	200.801	200.946
DESC OFFSET	200.946	200.801	200.946
200.402	200.324	200.350	200.368
200.270	200.270	200.232	200.248
198.181	198.181	200.321	200.321
198.181	198.181	200.410	200.410
198.181	198.181	200.486	200.486
198.181	198.181	200.516	200.516
198.181	198.181	200.447	200.447
198.306	198.306	200.479	200.479
198.321	198.321	200.471	200.471
198.426	198.426	200.526	200.526
198.426	198.426	200.609	200.609
198.426	198.426	200.533	200.533
198.426	198.426	200.655	200.655
198.426	198.426	200.657	200.657
198.426	198.426	200.757	200.757
198.426	198.426	200.805	200.805
198.426	198.426	200.863	200.863
198.426	198.426	200.892	200.892
198.426	198.426	200.884	200.884
198.426	198.426	200.903	200.903
198.426	198.426	200.841	200.841
198.426	198.426	200.850	200.850
198.426	198.426	200.800	200.800
198.426	198.426	200.750	200.750
198.426	198.426	200.700	200.700
198.426	198.426	200.650	200.650
198.426	198.426	200.600	200.600
198.426	198.426	200.550	200.550
198.426	198.426	200.500	200.500
198.426	198.426	200.450	200.450
198.426	198.426	200.400	200.400
198.426	198.426	200.350	200.350
198.426	198.426	200.300	200.300
198.426	198.426	200.250	200.250
198.426	198.426	200.200	200.200
198.426	198.426	200.150	200.150
198.426	198.426	200.100	200.100
198.426	198.426	200.050	200.050
198.426	198.426	200.000	200.000

- LEGEND
- ☒ Topsoil
 - ☒ Silty Clay
 - ☒ Siltstone
 - ☒ Sandstone fine grained
 - ☒ Mudstone

0 5 10 15 m
VERTICAL & HORIZONTAL SCALE 1:200 (A3 SHEET)

DRAWING ADAPTED FROM CROSS-SECTIONS PROVIDED BY OPUS INTERNATIONAL CONSULTANTS (NSW) PTY LTD, DATED 13/5/08



Douglas Partners
Geotechnics Environment Groundwater

Sydney, Newcastle, Brisbane,
Melbourne, Perth, Wyoing,
Campbelltown, Townsville,
Cairns, Wollongong, Darwin

TITLE: CROSS-SECTION AT CUT 37B
TILLEGRA DAM - DIVERSION OF SALESBURY ROAD
LOCALITIES OF BANDON GROVE, MUNNIE & UNDERBANK
GEOTECHNICAL INVESTIGATION, CONCEPT DESIGN PHASE

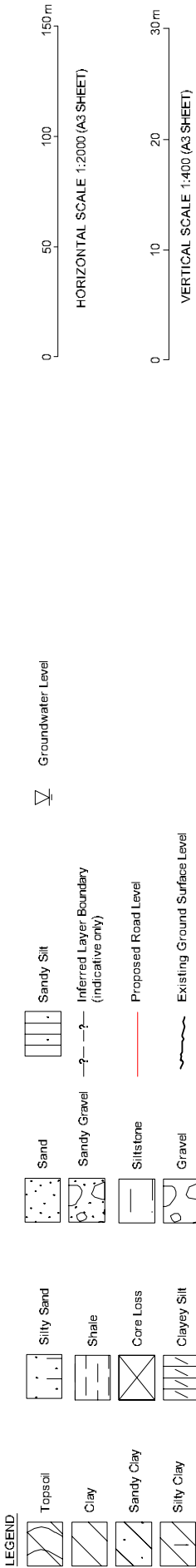
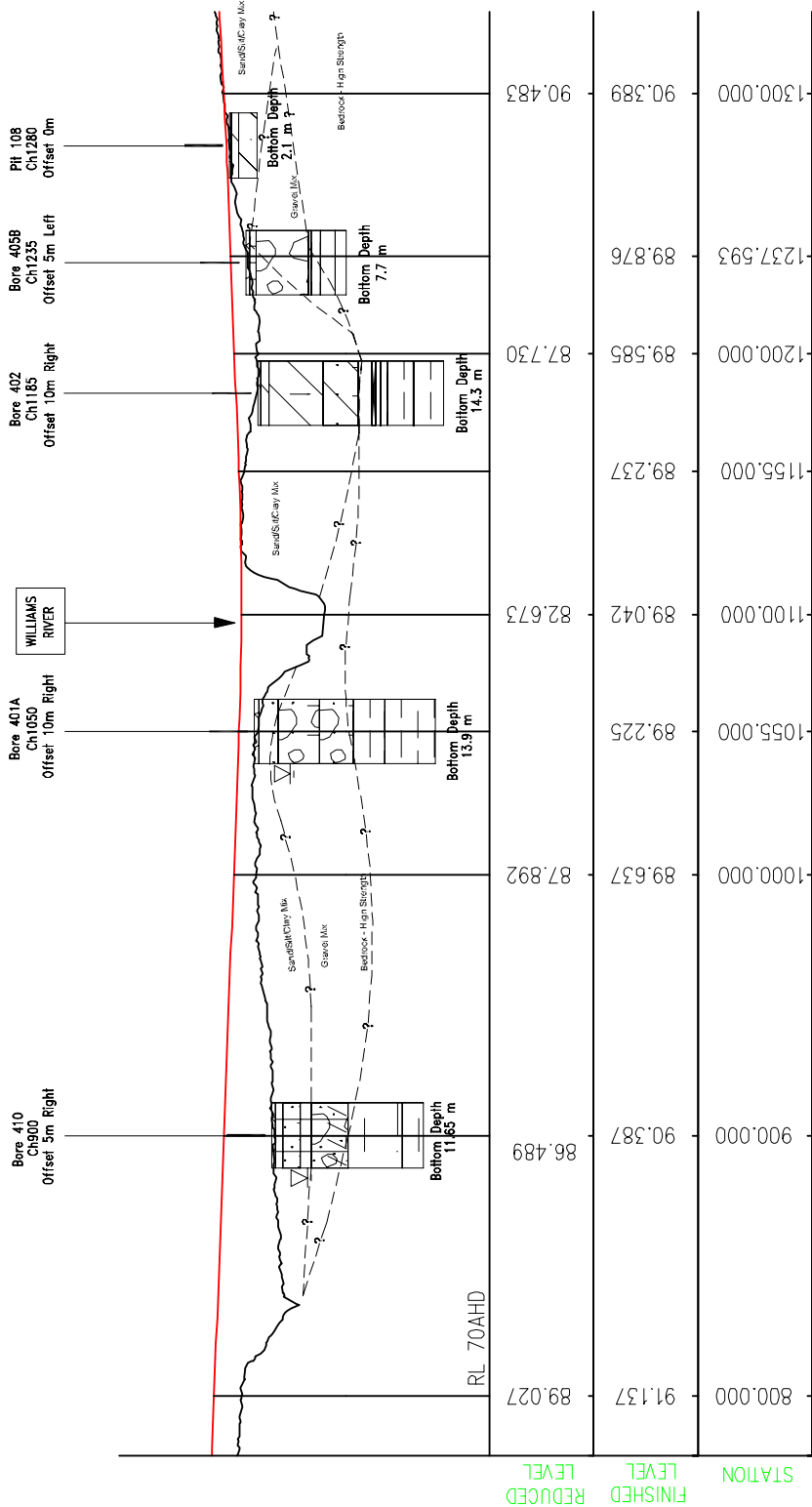
CLIENT: OPUS INTERNATIONAL CONSULTANTS (NSW) PTY LTD

DRAWN BY: LW SCALE: 1:200 PROJECT No: 39721.01

APPROVED BY: DATE: OFFICE: NEWCASTLE

DRAWING No: 8 SHEET: 13 OF 13

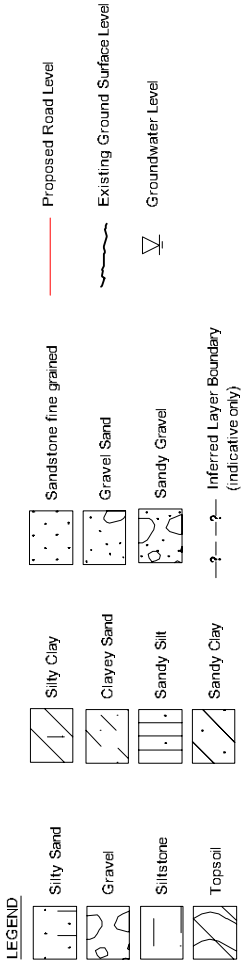
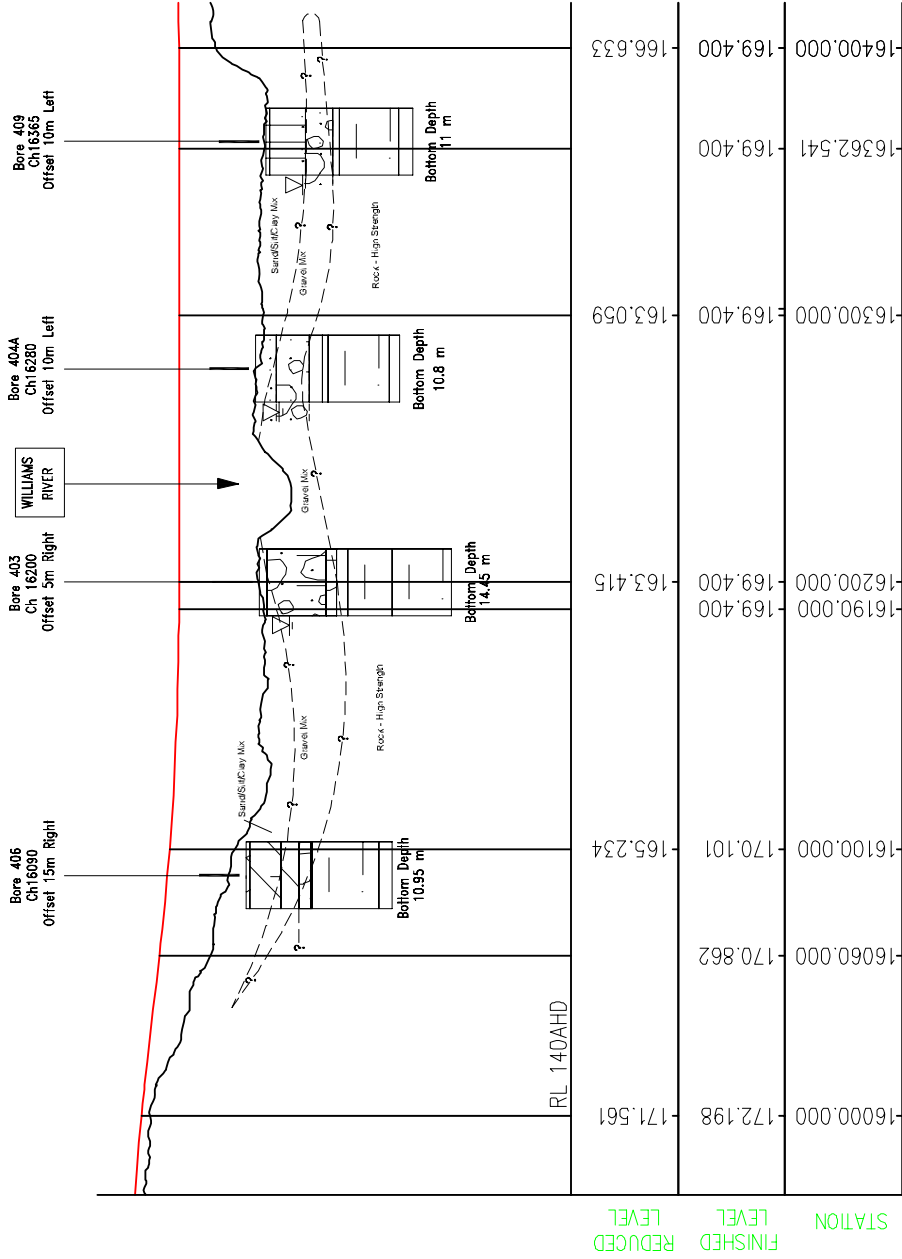
LOWER WILLIAMS RIVER CROSSING



DRAWING ADAPTED FROM E2 ALIGNMENT LONG SECTIONS PROVIDED BY OPUS INTERNATIONAL CONSULTANTS (NSW) PTY LTD, DATED 5/5/08

	CLIENT: OPUS INTERNATIONAL CONSULTANTS (NSW) PTY LTD			OFFICE: NEWCASTLE	
	TITLE: INTERPRETED GEOLOGICAL LONG SECTION PROPOSED BRIDGE OVER LOWER WILLIAMS RIVER TILLEGRA DAM - DIVERSION OF SALISBURY ROAD LOCALITIES OF BANDON GROVE, MUNNI & UNDERBANK			DRAWING No: 9	
	DRAWN BY: LW SCALE: AS SHOWN PROJECT No: 39721.01			SHEET: 1 OF 2	

UPPER WILLIAMS RIVER CROSSING



DRAWING ADAPTED FROM E2 ALIGNMENT LONG SECTIONS PROVIDED BY OPUS INTERNATIONAL CONSULTANTS (NSW) PTY LTD, DATED 5/5/08

 Douglas Partners Geotechnics Environment Groundwater	TITLE: INTERPRETED GEOLOGICAL LONG SECTION PROPOSED BRIDGE OVER LOWER WILLIAMS RIVER TILLEGRA DAM - DIVERSION OF SALISBURY ROAD LOCALITIES OF BANDON GROVE, MUNNI & UNDERBANK			CLIENT: OPUS INTERNATIONAL CONSULTANTS (NSW) PTY LTD		
				DRAWN BY: LW	SCALE: AS SHOWN	PROJECT No: 39721.01
				OFFICE: NEWCASTLE DRAWING No: 9 SHEET: 2 OF 2		