APPENDIX F

Geotechnical Investigation Report

Geotechnical Investigation Report

MR83 City of Grafton

GRAFTON BRIDGE AND APPROACHES

GEOTECHNICAL INVESTIGATION FOR ROUTE SELECTION

November 2003

Prepared by: RTA Northern Technical Services

Report No H/40856

EXECUTIVE SUMMARY

GEOTECHNICAL INVESTIGATION

A geotechnical investigation was carried out to assist in route selection. A desktop study looked at available geotechnical and soil data, the original bridge foundation plans, and the results of previous drilling. Four boreholes and seven test pits were dug to examine the broad geotechnical issues.

The boreholes were positioned on the south bank at Locations 1, 5 and 7, and on the north bank at Location 3 (existing bridge) The test pits were positioned on the approach alignments for Locations 4, 5 and 7. Detailed testing was not carried out in this stage of the work.

The foundation depth for the existing bridge is down to –20m (AHD) in the central section of the river. The boreholes indicated that the foundation depth to rock is likely to be in the range of -17 to -24m (AHD) at the other possible sites. The profile down to rock was silty clays and clays, then sands and sandy gravels, with some areas of cobbles and boulders. The likely bridge foundations would be bored piles into the rock.

The test pits indicated very variable sub-soil conditions, with a variety of clays, silts and some sand. The soil profile was quite acid in most locations investigated. As the work is mainly embankment, acid sulphate soil should not be a major problem. Steel or concrete in the foundations of drainage structures may need some protection

From a geotechnical viewpoint, the existing site (Location 3) has the least geotechnical constraints. It probably has shallower depth to the rock on the approaches, and the southern approach is above the flood plain on weathered rock, rather than alluvial soil.

The other locations were very similar in terms of subsurface conditions. Few geological constraints were identified that would have a severe impact on any of the routes. The strata of the alluvial plain is quite variable over short distances, and detailed drilling would be necessary to evaluate possible foundation problems, like the presence of substantial depth of boulders. Likewise the areas of soft ground, which would lead to greater settlement, are limited in extent and would need to be examined in a detailed investigation once the alignment is more definite.

Appendices

Appendix A — Site Plans
1) Plan of Bridge Localities
2) Borehole and Test Pit Locations
3) Copy of Geological map
4) Acid Sulphate Maps
5) Major Utility Locations

Appendix B — Borehole Logs and Photographs – Geotechnical Report No: G3510

Appendix C — Test Pit Logs and Photographs

Appendix D — Laboratory Certificates and Soil Profile Data

Appendix E — Original Bridge Section (Work as Executed Plan?)

Appendix F — Scour Investigation 2001

Appendix G — Borehole Logs and Site Plan - 1975 Investigation

1. Introduction

Background

The provision of a second crossing of the Clarence River has been proposed for many years. In its current form, the project initially considered possible routes over a wide area upstream from Maclean. A period of public consultation and initial studies narrowed the route selection corridor to the area between Susan island and Elizabeth Island.

RTA Northern Technical Services were commissioned to provide the geotechnical investigation for route selection. RTA Geotechnical and Scientific Services were also invited to contribute to the start-up workshops.

Investigation

The field investigations were aimed at providing some basic information to assist the route selection process. A Review of Environmental Factors for the geotechnical investigation was prepared, and following the decision report, an Environmental Management Plan was prepared.

Four boreholes were drilled on the riverbank, spread throughout the study area, and seven test pits were dug in the area between McLares Lane and Elizabeth Island, where possible options would involve the greatest length of approach embankment.

The drilling was completed in one and half weeks and the test pitting was carried out in one day. All field investigations were done in September 2003.

Scope

The geotechnical investigation for route selection included:

- assessment of existing geological mapping;
- assessment of the original bridge plans;
- assessment of past drilling;
- borehole drilling;
- test pit excavation;
- laboratory testing; and
- reporting.

2. Desktop Study

Topography

The topography of most of the study area consists of the flood plain of the Clarence River, a fairly flat alluvial plain only a few metres above sea level. The eastern river bank along the river to the north of the existing bridge is higher than the land back from the river, sloping away from the river at less than 1degree slope. The highway to the north of Alipou Creek is on higher ground as it skirts along the edge of the hills bordering the flood plain.
The only part of the study area that is not part of the alluvial plain is the southern approach to the existing bridge which is along a ridge of soil and rock of the Grafton Formation, forming an island in the flood plain.

The study area contains a number of artificial deep drainage channels as well as natural watercourses. The main constructed drainage channels are north of Eggins Lane (locality 7), north of McLares Lane (locality 5) and Ardent Street drain (locality 1 and 2).

Regional Geology

Reference to the 1:250 000 Geological Series Sheet (SH56-6) indicates the study area is underlain by the Grafton Formation of Jurassic-Cretaceous age. The Grafton Formation consists of sandstone, siltstone, claystone and minor coal. Quaternary alluvial stream deposits overlie the geology of the Grafton Formation across the Clarence River Floodplain. These deposits consist of clays, sands, silts and minor gravels. The Clarence River Floodplain stretches to a maximum width of approximately 7km within the study area.
The geological map was compiled in 1969, and no more recent edition has been published.

A copy of the regional geological map is attached to this report in Appendix A.

Soils

The Grafton 1:100 000 Soil Landscape Sheet and Report has not yet been published (by the Department of Land and Water Conservation (DLWC)). Some of the surrounding sheets are available, but do not provide information relevant to this investigation. Dunlop's report makes reference to a Grafton Area Study by the Soil Conservation Service, but this could not be located.
The DLWC Soil Profile Attribute Database (SPADE) was consulted. It contained three entries for the Grafton area. The first was located near Eggins Lane within the investigation area, and detailed a profile of grey clay to 1.1m depth.
The second profile was located on the levee near the river end of Queen Street, on the northern side of the river. It detailed a clay loam profile to 2.5m depth.
The third profile was located on the north-west side of Cowans Ponds, and is outside the investigation area.
The report is contained in Appendix D.

Acid Sulphate Soils

Two acid sulphate soil maps were available. The first was in Appendix D of the feasibility study and the second was the DLWC Grafton 1:25000 Acid Sulphate Soil Risk map – Edition 2 1997. These are essentially the same, but the 1:25000 sheet splits the high and low probabilities into several components. These sheets indicate high probability of occurrence on both sides of the river near Elizabeth Island, within the river on both Susan Island and Elizabeth Island, and on the south side between the highways and the riverbank over much of the rest of the area, except near the present bridge alignment.

The river bank was mostly mapped as of low probability.

The only areas of high probability at ground level or within 1m of ground level were in the corridor for locality 7 and a narrow channel in the corridor for locality 6.

A copy of the relevant section of the Acid Sulphate Soil Risk map is attached to this report in Appendix A.

Previous Investigations

The major previous investigation was done in 1975. This involved the drilling of seven boreholes on three lines. One borehole was drilled from a barge in the river on a line upstream of the present bridge. Two boreholes were drilled from a barge on a line downstream of the present bridge and four boreholes were drilled on a line further downstream on an alignment from Iolanthe Street – two holes being in the river and two boreholes on land on the southern side.

The boreholes in the river indicated mainly sand and gravels down to weathered rock, and the boreholes on the south side indicated mainly clays down to the weathered rock level. The results are discussed later in this report, but can be summarised as follows:

Levels calculated to AHD (m)	Upstream Line	Downstream Line		Iolanthe Street Line			
	Bore Hole 1	Bore Hole 2	Bore Hole 3	Bore Hole 4	Bore Hole 5	Bore Hole 6 (bank)	Bore Hole 7 (bank)
Top of Borehole	-3.32	-4.29	-12.24	-4.98	-10.8	6.26	4.13
Weathered Rock Level	-19.12	-21.64	-20.84	-20.87		-18.64	-18.47
Sandstone Rock Level	-20.41	-24.54	-22.46	-26.82	-25.2	-22.74	-22.37

The core taken from these holes was discarded several years ago and the only information available is the drillers logs. Copies of the logs and the site plan are included as Appendix G.

The report by L. Dunlop – "Geological Input for Environmental Impact Study of the Proposed Second Bridge over the Clarence River at Grafton" 1981 refers to three soil samples taken by the Scientific Officer at the time. The logs or test certificates for these could not be located. This report contains logs for two of the boreholes drilled in the 1975 investigation, and it should be noted that the location map in the report does not show the correct positions for these bores.

During the fieldwork, some landowners indicated to us that the Clarence River County Council had dug a number of investigation test pits or boreholes associated with drains and flood control structures in the study area. The County Council advised that no records were available from these investigations.

Prior to the construction of the existing bridge 17 boreholes were drilled. No logs are available for this investigation, only a cross section of the stratigraphy on the job plan on the file. The following depth information was measured from this cross section:

Levels calculated to AHD (m)	Bed / Ground level	Base of Borehole (Rock Depth)
Bore Hole 1	4.96	-12.57
Bore Hole 2	8.01	-9.52
Bore Hole 3	1.15	-14.86
Bore Hole 4	-1.90	-16.77
Bore Hole 5	-3.05	-22.86
Bore Hole 6	-3.81	-21.34
Bore Hole 7	-4.57	-20.20
Bore Hole 8	-6.09	-19.05
Bore Hole 9	-8.00	-19.43
Bore Hole 10	-9.91	-20.20
Bore Hole 11	-12.19	-19.81
Bore Hole 12	-12.57	-18.29
Bore Hole 13	-12.57	-17.15
Bore Hole 14	-11.81	-15.62
Bore Hole 15	-4.95	-14.86
Bore Hole 16	2.67	-6.48
Bore Hole 17	8.01	-7.62

The boreholes indicate that the bed of the river consists mainly of sands, gravels and boulders above the rock, and the banks consist mainly of clays and sands down to the rock level.

Work as Executed Plans

There are various plans on the files, and one has been taken as the likely Work as Executed Plan, although it is not formally marked as such. The plans indicated that the piers were all taken to the sandstone rock level, except for Pier 6, which appears to be founded in boulders and gravel several metres above rock level. The arched piers on the river bank at the ends of the steel skew spans appear to be founded on clay and loam materials.

The plans indicate the following foundation depths, which were either calculated from levels given or scaled off the plan, and converted to AHD:

Pier	Base of Footing (Levels calculated to AHD - m)
South Abutment	4.19
Pier 1	-8.76
Pier 2	-16.38
Pier 3	-18.29
Pier 4	-20.96
Pier 5	-19.81
Pier 6	-16.38
Pier 7	-15.24
North Abutment	4.19

The plan is attached to this report as Appendix E.

Scour

The Bridge Maintenance Engineer has advised that significant scour has occurred around some of the piers. A depth survey in 2001 indicated that up to 7m had scoured out on the downstream side, and up to 5m on the upstream side, leaving the base of the pier exposed at Piers 2 and 3 (Bascule span) and at Pier 4. Inspection by divers revealed a gap under part of a pier. It is not known whether subsequent floods have deposited material or caused additional scour. The large sizes of the existing foundations reduce risk but the scour indicates that the smaller piles of a new bridge will have to be well embedded into the rock.

The survey is attached in Appendix F (note Pier labels not consistent with above).

Materials

A detailed materials assessment has not been undertaken as part of this work, but the following sources are known and / or have been used for work in the past:

1) The only construction materials close to the site are river gravels dredged from the Clarence River. Boral at Grafton currently extract this material but the quantity that would be available is not known. This type of material would only be suitable for use in drainage layers and fill if there was not an acid sulphate problem with it.
2) McLennans Ironstone – Dundoo, located near Halfway Creek, and at Jacky's Creek, located on the Old Glen Innes Road. This material would be suitable for all pavement layers in a flexible pavement option.
3) Tuckers Ironstone. Present quality not known. Located at various sites along MR83
4) Connors Pit. Weathered sandstone suitable only for use as fill. Located at Tucabia. Current availability not known. There are also a number of other old fill quality pits around the Tucabia area
5) Weathered sandstone. Located at Woodford Island. Current availability not known.
6) Newmans Pit. Crushed sandstone, located at Tabbimobile. Suitable for use for all layers
7) Woolgoolga Quarry. Crushed argillite. Suitable for use for all layers
8) Coffs Harbour quarries. There are several quarries in the Coffs harbour area that could supply crushed argillite materials.
9) As all options will involve construction of approach embankments, fill will be needed There are a number of other old pits around Grafton that local suppliers may have access to. The availability would need to be determined closer to the work commencing
10). Possible sources of fill material may also be available from the widening of road cuttings along the Pacific Highway both north and south of Grafton, and along the Gwydir Highway west of Grafton. There are a number of cuttings that rank moderately highly in the slope risk assessment, and widening of these may be a treatment option that will produce suitable fill at reduced overall cost.

3. Field Investigation

The field investigation was limited to providing sufficient information to give a broad view of the likely conditions. The desktop study of the available geotechnical information indicated that the broad geological model of the study area would be reasonably consistent, with localised variation in alluvial strata and acid sulphate levels, that could only be defined by very close sampling. With the number of possible localities that are under consideration, the expense of detailed sampling, that would not be used in design, was not justified at this stage, particularly in the urban areas on the north side of the river. It was therefore decided that only a small number of test pits would be necessary to indicate the range of variation that could be expected.

Similarly the drilling program was limited to 4 holes spread approximately equidistant along the length of river.

A single lot of water sampling was done in the test pits and along the major drains to check for acid conditions.

A plan showing Borehole and Test Pit locations, and a list of survey location co-ordinates is presented in Appendix A.

Geological Mapping

As the entire study area away from the existing alignment is alluvium, surface geological mapping was considered to be of little value. The test pitting was too widely spaced and the strata too varied to be used to derive a detailed sub-surface geological model at this stage of the investigation.

A copy of the relevant section of the published Geological map is included in Appendix A.

Borehole Drilling

A total of four (4) boreholes were drilled for route selection. The boreholes were positioned on site by RTA staff with respect to the proposed localities, site access, and permission from property owners.

Bore Hole 1 was sited east of Elizabeth Island at locality 7. The hole was positioned 20m back from the river to reduce the risk of encountering rocks or concrete that may have been placed for bank protection, and was at an elevation of 5.77m (AHD).
The bore encountered sandy clays and silty clays to a depth of 20m (-14.2m AHD). These clays were stiff to very stiff over most of the length, with firm material between depths of 8m to 11m and 18.5m to 20m. The hole showed the presence of peaty material at depths of 8 to 9.7m. The Standard Penetration Test results ranged from 5 to 23.
Hard sandy silty gravels with cobbles were present from 20m to 27.2m, then light grey siltstone rock at a depth of 27.2 (-21.4m AHD). No core was taken from this hole to reduce the drilling time and expense, but the even penetration for 0.5m, and examination of the material remaining on the end of the bit indicate that the hole reached bedrock.

Bore Hole 2 was on the western side or the river at the end of an un-named lane north of McLares Lane, at location 5. The hole was positioned 20m back from the river to reduce the risk of encountering rocks or concrete that may have been placed for bank protection or backfill of scour (these materials being visible in the bank). It was positioned at an elevation of 5.87m (AHD).
The bore encountered clayey sand to a depth of 1.8m, clayey silt and silty clay to a depth of 12.6m, silty sand and sandy gravel to a depth of 18.25m and then coarse gravels with boulders and cobbles to a depth of 23.6m (-17.7m AHD) where bedrock was encountered. This borehole indicated the presence of layers of very soft material within the clays, particularly at depths of around 2.8m, 6.4m to 8m, and at 11 to 12.6m. The SPT in these areas was 0 to 4
The hole was cored for 5.3m to a depth of 28.9m. The core consisted of light grey sandstone, and siltstone with dark grey laminae. The rock was of medium strength, with some extremely weathered and fractured zones.

Bore Hole 3 was sited on the northern side of the river upstream of the existing bridge between the sailing club and Pier 1, locality 3. The hole was positioned 20m back from the water, and was at an elevation of 1.71m (AHD).
The bore encountered silty and sandy clays to a depth of 2.2m, sand and gravely sand to a depth of 11.9m, then sandy gravel to a depth of 20.7m (-19.0m AHD). The sandy gravel contained occasional cobbles up to 200mm in size. The Standard Penetration Test results were 2 in the upper clay, and ranged from 7 to 16 in the sands down to the gravel at 11.9m. Light grey siltstone rock was encountered at a depth of 20.7m (-19.0m AHD). No core was taken from this hole to reduce the drilling time and expense, but the even penetration for 0.65m, and examination of the material remaining on the end of the bit indicate that the hole reached bedrock.

Bore Hole 4 was sited on the southern side of the river adjacent to the Ardent Street drain, between locality 1 and 2. The hole was positioned 20m back from the main river, and was at an elevation of 1.66m (AHD).
The bore encountered silty and sandy clays to a depth of 6.2m, silty sand to a depth of 9 – 9.5m, then clean sand to a depth of 10.6m. The hole was continued in sandy gravels to a depth of 11.5m (-9.8m AHD)., where it was terminated to reduce costs.
Very soft material with Standard Penetration Test results of 0 to 1was found between depths of 0.7m and 5.2m.

An RTA Technical Officer supervised the drilling and was responsible for sampling and logging. The locations of the boreholes are indicated on the attached plan (Appendix A). The borehole logs are attached in Appendix B, together with a set of explanatory notes, which define the terms and symbols used in their preparation.

In summary, the boreholes indicate the following foundation depths:

	Ground level (m) AHD	Rock Depth (m) AHD
Bore Hole 1	5.78	-21.4
Bore Hole 2	5.87	-17.7
Bore Hole 3	1.71	-19.0
Bore Hole 4	-1.67	Not reached

Colour photographs of the recovered core are attached to the cored log in Appendix B. The Point Load Index Strength test results SPT test results and hand penetrometer test results are recorded on the cored borehole logs.

Test Pits

Seven test pits were excavated for the investigation, concentrating on lines that would have the greatest length of approach embankment. Four pits were dug spaced approximately equally along a possible line between the existing highway and locality 7 at Elizabeth Island. Two pits were dug along a possible line between the existing highway and locality 5, and one pit was dug along a possible line between the existing highway and locality 4. No test pits were dug for the approaches to locality 1 and 2 due to access difficulties.

The pits were sited along public road reserves where possible, and were positioned to avoid utilities.

The test pits were excavated using an extendable backhoe, and were taken to depths ranging between 4.3 metres and 4.6 metres, which was the limit of reach of the machine. Test Pit 6, however, had to be abandoned at a depth of 2.6m due to collapse of the sides.

It was initially proposed to carry out Dynamic Cone Penetrometer (DCP) testing to assess the strength of the top of the soil profile along the approach routes. Due to the prolonged dry conditions, the upper layers were stiff to very stiff meaning that artificially high in-situ CBRs would be recorded, and so DCP testing was deferred until the detailed investigation phase, when ground conditions may be more normal.

<u>Test Pits GB1 to GB4</u>
These test pits were excavated in a line between the river and the existing highway. The plain in this area is at its highest point near the river and slopes off to a low poorly drained area near Test Pit GB4, then rises to the level of the highway. The test pit elevations were determined to be:

	Test Pit GB1	Test Pit GB2	Test Pit GB3	Test Pit GB4
Height (AHD)	5.21m	3.59m	2.99m	1.51m

The length represented is around 1250m, indicating an average slope of 0.3%.

<u>Test Pit GB1</u> was positioned towards the river bank close to where BH1 was drilled. The pit indicated very stiff brown silty clay for most of the depth (4.3m). This material had a re-compacted laboratory CBR of 2.5 when compacted at 95% OMC and a plastic index of 14. No water inflow occurred into the pit, and field testing of the material with 30% Hydrogen Peroxide had little reaction indicating low acid sulphate potential.

<u>Test Pit GB2</u> was positioned adjacent to Eggins Lane. The pit indicated medium to stiff brown and orange clayey silt and silty clay to 3.4m where there was water inflow, then wet dark grey highly plastic clay to the base of the pit at 4.35m. This lower material was quite soft. The water inflow was fairly strong and filled the pit 1m deep to a depth of –3.4m. Field testing of the material with 30% Hydrogen Peroxide had little reaction indicating low acid sulphate potential

<u>Test Pit GB3</u> was also positioned adjacent to Eggins Lane. The pit indicated very stiff to hard grey-brown and brown and orange silty clay and clay to 3.2m, then mottled sandy clay to the base of the pit at 4.10m. This lower material was softer than the overlying clay. Water inflow occurred at -2.9m and there was strong water inflow at –3.6m.
Field testing of the material with 30% Hydrogen Peroxide had little to moderate reaction indicating low acid sulphate potential.

<u>Test Pit GB4</u> was positioned in a low-lying area towards the highway. The landowner advised that during a wetter period the water table was near to the surface in this area. The pit indicated a variety of highly plastic clay materials, with strong mottling. Crystalline gypsum was present in a number of the layers, and very soft dark grey material was present between 2.0 and 2.5m. The pit was continued in stiff sandy silty clay to a depth of 4.55m. Testing of two samples from the pit indicated a plastic index of 38 and 47.
There was strong water inflow at –1.7m and –2.6m and the pit partially filled with water and the walls collapsed from the area of water inflow
Field testing of the material with 30% Hydrogen Peroxide had varied reaction in the materials, with a very strong reaction occurring in the orange mottled clay at 3.5m depth. A sample of this material was sent to a laboratory for ASS testing.

Test Pits GB5 to GB6

These test pits were excavated along an un-named lane between the river and the existing highway, in the area of locality 5. The plain in this area is at its highest point near the river and slopes off towards the highway. The test pit elevations were determined to be:

	Test Pit GB5	Test Pit GB6
Height (AHD)	4.82m	1.90m

Test Pit GB6 was dug at the edge of a channel and was slightly lower than the surrounding land surface.

Test Pit GB5 was positioned approximately 2/3 of the way towards the river. The pit indicated stiff to very stiff dark brown silty clay and orange brown silty clay to -3.7m then dark grey silty clay and clay to the base of the pit at 4.6m depth, the lowest layer having a high charcoal content. The material to 1m below ground surface had a re-compacted laboratory CBR of 3.5 when compacted at 99% OMC, and a plastic index of 16.
Some water inflow occurred near the base of the pit.
Field testing of the material with 30% Hydrogen Peroxide had little or no reaction in the brown silty clay and mild reaction with the lower dark grey clays, indicating low acid sulphate potential.

Test Pit GB6 was positioned approximately 1/4 of the way towards the river, at the edge of a small creek. The pit indicated sandy silty clay to 1.1m, then sandy clay, clayey sand and sand to 2.6m. These materials were increasingly saturated, with strong water inflow occurring at 2.4m, which resulted in collapse of the pit walls. The sand had a narrow range of particle size and no clayey fines.
Field testing of the material with 30% Hydrogen Peroxide had little or no reaction with the materials in this pit

Test Pit GB7 was positioned along McClares Lane, approximately half way between the river and the highway, close to the possible alignment for the approaches to locality 4, at an elevation of 2.62m. The pit indicated very stiff brown silty clay to 0.85m, then dark grey silty clay to –2.6m. The material at 1m below ground surface had a re-compacted laboratory CBR of 3.5 when compacted at 101% OMC. Below –2.6m was dark grey to black highly plastic clay to the base of the pit at 4.6m. Below 3.7m, the clay had orange mottling and contained fine carbonaceous matter
Water inflow occurred at 3.1m and 4m. The lower material smelled very strongly of sulphur and had a strong reaction with 30% Hydrogen Peroxide, indicating potential acid sulphate soil. Subsequent testing confirmed acid conditions in this material.

The locations of the test pits are indicated on the attached plan (Appendix A). The test pit logs and colour photographs are attached in Appendix C, together with a set of explanatory notes, which define the terms and symbols used in their preparation. Hand Penetrometer test results are summarised on the test pit logs.
Selected subsurface materials were sampled from the test pits and transported to the RTA Grafton laboratory for testing. The test certificates are attached in Appendix D.

3.4 Laboratory Testing

3.4.1 Geotechnical / Pavement

The job is not likely to involve cut to fill operations, but will be predominately embankment construction to the bridge abutments. Thus the need for soil testing at this stage is limited, other than to investigate potential subgrade CBR and to classify materials. The soil testing was therefore limited to 3 samples to look at likely subgrade CBR (T111, T117A), five samples for plasticity (T108, T109) and one sample for grading (T106, T107). A range of samples from the test pits were tested for moisture content (T120)

In addition to the samples tested, a range of representative samples of the subsurface profile was recovered from the test pits in case further testing is required, and these are stored at the RTA Grafton Laboratory.

The investigation indicated that the subgrade CBR was 2.5 to 3.5. The design subgrade CBR is likely to be 2.5, but as the embankment depth is more than 1m, the pavement design CBR will be that of the fill that is used.

The laboratory test certificates are presented in Appendix F.

3.4.2 Moisture Regime

Twenty-six moisture content samples were taken from the test pits, and these indicated moisture contents in the range of 17.5% to 57.7%.
Compaction tests were done on the samples tested for CBR, with the following results:

Sample No	Field M/C (%)	Optimum M/C (%)	Ratio FMC / OMC (%)
1A	19.2	19.4	99
5A	24.6	20.4	121
7A	25.8	26.8	96

It is expected that most of the area will have materials that are at high moisture content, probably at >90% of optimum moisture content. The moisture contents could be considerably higher after prolonged rain or a flood.

3.4.3 Soil Acidity and Acid Sulphate Potential

Reference to the 1:25 000 Acid Sulphate Soil Risk Map for Grafton indicates a high probability of acid sulphate soils at the surface in the area of Test Pit GB4, high probability of acid sulphate soils within the top 1m to 3m of the soil profile in the area of Test Pits GB2 and GB3, and high probability of acid sulphate soils below 3m of the soil profile in the area of Test Pits GB6 and GB7.

During the test pitting, field testing was carried out for acid sulphate potential, using 30% Hydrogen Peroxide. Tests were done on most soil types, but only samples from Test Pits 4 and 7 showed extensive reaction.

Thirteen soil samples were tested for pH back at the laboratory. Seven samples representing the full soil profile, were tested from Pit GB3, four samples were tested from Pit GB4, and one sample each from Pits GB6 and GB7. The results ranged between pH 3.3 and pH 5, indicating that much of the soil profile contained quite acidic material. Two samples of black clay from Boreholes 2 and 4 were also tested but the recorded pH was >5.5.

Two samples were sent to the Environmental Analysis Laboratory at Lismore for Acid Sulphate testing (analysis by the POCAS method (ie Peroxide Oxidation – Combined Acidity and Sulphate) and 'Chromium Reducible Sulphur' technique). The results indicated that the sample from Test Pit GB4 did not show actual or potential ASS, but the sample from Pit GB7 showed actual ASS, having a pH of 4.2, and indicating a neutralising requirement of 7.1kg of lime per cubic metre.

Selected water samples were also taken from groundwater that flowed into the excavations, and from the drains north of Eggins Lane and at Ardent Street, at low tide. These samples did not show significant acidity – all had pH >6.

The laboratory test certificates are presented in Appendix D.

4. Results of Investigation

Geotechnical Model for Study Area

On a broad scale the geotechnical conditions in the study area are fairly consistent. Most of the area is in alluvial flats containing clay, silt, sand and gravel layers. The underlying rock is the Grafton formation, consisting of sandstone siltstone, claystone and minor coal. The depth to rock varies, with outcrop of the rock at the surface at South Grafton, and at depths up to -25m under the alluvium. The rock is mostly of medium strength, with the core that was recovered indicating that the strength is fairly constant with increasing depth. The rock is fresh to slightly weathered, with some thin, more weathered zones.

The foundation depth under the river averages –20m AHD, varying mostly between -17.5m to –23m. The drill holes indicate that the depth to rock along the river in the study area is not likely to vary much.

On a smaller scale, the alluvial deposits across the plain appear to vary considerably over relatively short distances. Some test pits indicated deep silty clay, and other pits indicated sand and clayey sand. Layers of carbonaceous material were found, and also gypsum crystals at one site. The boreholes showed silty clay, sand and gravels above the rock, with no consistent profile being present. Larger gravels, cobbles and boulders were found in some boreholes.

Four holes that are relatively close together, near the northern pier, are:
Bore hole 3 (original bridge), Boreholes 1A and 2 (1975 drilling) and Borehole 3 (present investigation). These show the rock to be at a level of –14.86, -19.12, -21.64, and –20.7 respectively. Boulders and cobbles are referred to as the main material in the original bridge plan, and in the 1975 Borehole 1A, yet Borehole 3, in the present investigation, found mainly sand, with occasional cobbles, down to the rock.

The soil profile in the areas that were test pitted was fairly acid, yet the groundwater tested as near neutral pH. The soil profiles and test results indicate that acid sulphate material may be present in small patches throughout the area, rather than as broad zones. All potential localities could be expected to have acid sulphate material in the profile.

4.2 Groundwater

The test pits indicated that the groundwater levels were close to AHD 0 to 1m over most of the area. Inflow into some of the test pits occurred at more than one level along thin sandy layers. The groundwater level is what would be expected, due to the presence of drains at river level. A landowner advised that during wet periods the groundwater level is close to the surface near Pit GB4 (1.5m AHD).

5. Engineering Considerations

Design Characteristics

The main differences in the possible localities are the length of the structure and the length of the fill embankment. Foundation conditions and depths for all bridge sites are indicated to be similar and likewise the embankment conditions should be similar in a broad sense for all locations other than the present bridge alignment. This alignment is superior on the southern side, as it is mostly on weathered rock material rather than alluvium

Bridge Footings

The results of the present drilling program, and past investigations, together with the old bridge plans, indicate that the bridge will need to be founded on rock at approximately AHD –20m. The Bridge Investigation Report recommends either driven or bored piles. The presence of boulder or cobble sized gravels may be sufficient to support a bridge, but the site conditions are not very consistent in terms of material above the rock. At some sites, the material above the rock is mainly sand, which would be more susceptible to scour.

The survey of the existing bridge indicates that up to 7m of scour has occurred at the piers, in some cases being to the base of the footing. This indicates that a new bridge should be founded on piles socketed into the rock, and thus bored piles are the best option. It is likely that they will have to be cased for the full depth.

Acid sulphate soils are possibly present, although the initial drilling and test pit did not indicate a problem immediately adjacent to the river, but allowance should be made for the protection of the upper portion of the piles.

This investigation did not find any geotechnical issues in the foundations that would influence route selection for locations 1,2 and 4 to 7. The increasing length of bridge away from the present site is the main factor. The existing bridge location is superior in terms of shallower foundation levels on the approaches.

The foundation type and depths cannot be firmly established until after the detailed drilling for the bridge.

Fill Embankments and Compressible Foundations

The test pits indicated that most of the alluvial material to 4m depth was stiff clay, even with moisture contents around OMC. There has been a prolonged drought so conditions at the time of construction may be significantly wetter. Little soft compressible material was found in the test pits, but bore holes 2, 3 and 4 found very soft material as follows:

Bore Hole	Depth (m)	SPT N-value	H.P Strength (kpa)
BH2	6.4 – 8.0	0	<25
	11.0 – 12.6	2	<25
BH3	0.8 – 3.0	2	90
BH4	0.7 – 5.2	0 - 1	0 - 10

The result from BH4 may be influenced by the site of the borehole immediately adjacent to Ardent Street Drain, which may have resulted in a greater degree of saturation than would be the case at either Location 1 or Location 2 on either side of Ardent Street.

At the present time no need for a bridging layer is indicated. However if construction commences in a wetter period, then provision of a bridging layer at the base of the embankments may be required.

Acid Sulphate soils

There are no cuttings proposed in the work, and therefore acid sulphate soils in the present profile should not be a problem except for the protection of the bridge piles as previously mentioned, plus any culvert bases. The worst area is the alignment for location 7, where there may be acidic material at the surface in the low area near Test Pit GB4.

All embankment material will need to be imported, so there should be little risk of exposure of potentially acid soils.

Embankment Batters and Settlement

The approach embankments are proposed to be 3 to 8m in height. The batter cannot be determined until the type of material to be used in the fill is specified, but at this stage a maximum 2:1 slope should be satisfactory, or 3:1 if poor quality fill is used.

It is not expected that the settlement of fill embankments will pose a major construction constraint, as most is likely to occur within the construction time. The immediate abutment fills have some potential to settle, and it would be advisable to construct the approach fill prior to the bridge foundations.

Large settlements are not expected at any of the proposed locations, but it is recommended that some settlement testing be undertaken in the detailed geotechnical investigation

Pavement Design

Due to the likely height of the embankment, the pavement design thickness will depend on the quality of the imported fill. All types of pavement may therefore be options. There are no geotechnical factors that will influence pavement design on any of the possible alignments, other than the existing pavement structure if the new alignment at Location 3 is part of the present roadway.

Embankment Stability

There is not likely to be any risk of embankment instability during construction. The embankment may, however, need to be rock faced to prevent erosion during periods of flooding. This will need to be determined from the hydrological modelling.

Utilities

Prior to the fieldwork of test pitting and drilling, utility enquiries were made. The following utilities were located and the information is included in this report to assist planning

a) Fibre optic telecommunications cable

i) Optus. This cable follows Eggins Lane and Meona Lane and then is a submarine crossing of the river

ii) Nextgen. This cable follows McLares Lane, then the drain to the river and then follows the river bank south to cross at the existing bridge

iii) Telstra The main Telstra cable crosses on the existing bridge

There are other local telephone cables on the northern side of all localities and on the south side of Localities 1,2 and 3 and 7. At locality 7 the Telstra cable follows Eggins lane to the last house, then goes north approximately 60m back from the river

b) Electricity

The submarine electric cable crossings go from the end of Fitzroy St above the sailing club to near the South Grafton Club, and from Abbott Street to Duke Street

c) Water

The water main northwards is located within the study area on the western side of the existing Pacific highway. At location 7 a local water supply line follows Eggins lane and also goes to the north. No other water pipe locations were obtained

The utility information is presented on a map in Appendix A

6. Comparison of Routes

No geotechnical issues were identified that would adversely influence any of the possible routes in a major manner. The bridge construction would be similar at all locations, varying mainly in length of structure required. Similarly the embankment construction would be similar at all alternate locations (other than the existing site), except for the length of the work.

The following specific issues should be noted:
Localities 1 and 2. Possibility of more settlement of the southern abutment than for the abutments at the other sites

Locality 3 This locality is the best in terms of stability of the southern approach, and because some of the foundation rock is at shallower depth

Localities 4 to 6 No geotechnical issues were identified that would affect the choice between any of these alignments

Locality 7 The approaches to this site are the longest and cross the main area of shallow acid sulphate soil.

In summary from a geotechnical viewpoint, the existing corridor /locality provides the best crossing point of the river. Of the other localities, no major geotechnical constraints were identified within them.

7. Further Investigation

This investigation was very limited in extent, and only sought to get an initial indication of the geological conditions in the study area. The work was planned so that major issues were identified, and also to give an idea of possible foundation depths along the river.

Following selection of the preferred route, detailed investigation of the bridge site and the approaches will be necessary.

8. References and Related Documents

(1) Brunker, R. L., Chestnut, W. s., 1969 Grafton 1:250 000 Geological Map. New South Wales Geological Survey, Sydney

(2) DLWC, 1:25 000 Acid Sulphate Soil Risk Map – Grafton. December 1997.

(3) RTA, Additional Crossing of the Clarence River. Feasibility Study Report. February 2003.

(4) Dunlop L., Geological Input for Environmental Impact Study of the Proposed Second Bridge Over The Clarence River at Grafton M&R L Report No. 949

(5) DLWC – SPADE Soil Profile Database

(6) NSWGR Grafton to South Grafton. Bridge Over Clarence River (Road and Rail) Plan (undated)

(7) Drillers Logs. Investigation in 1975. RTA file 173.152.

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Reviewed

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