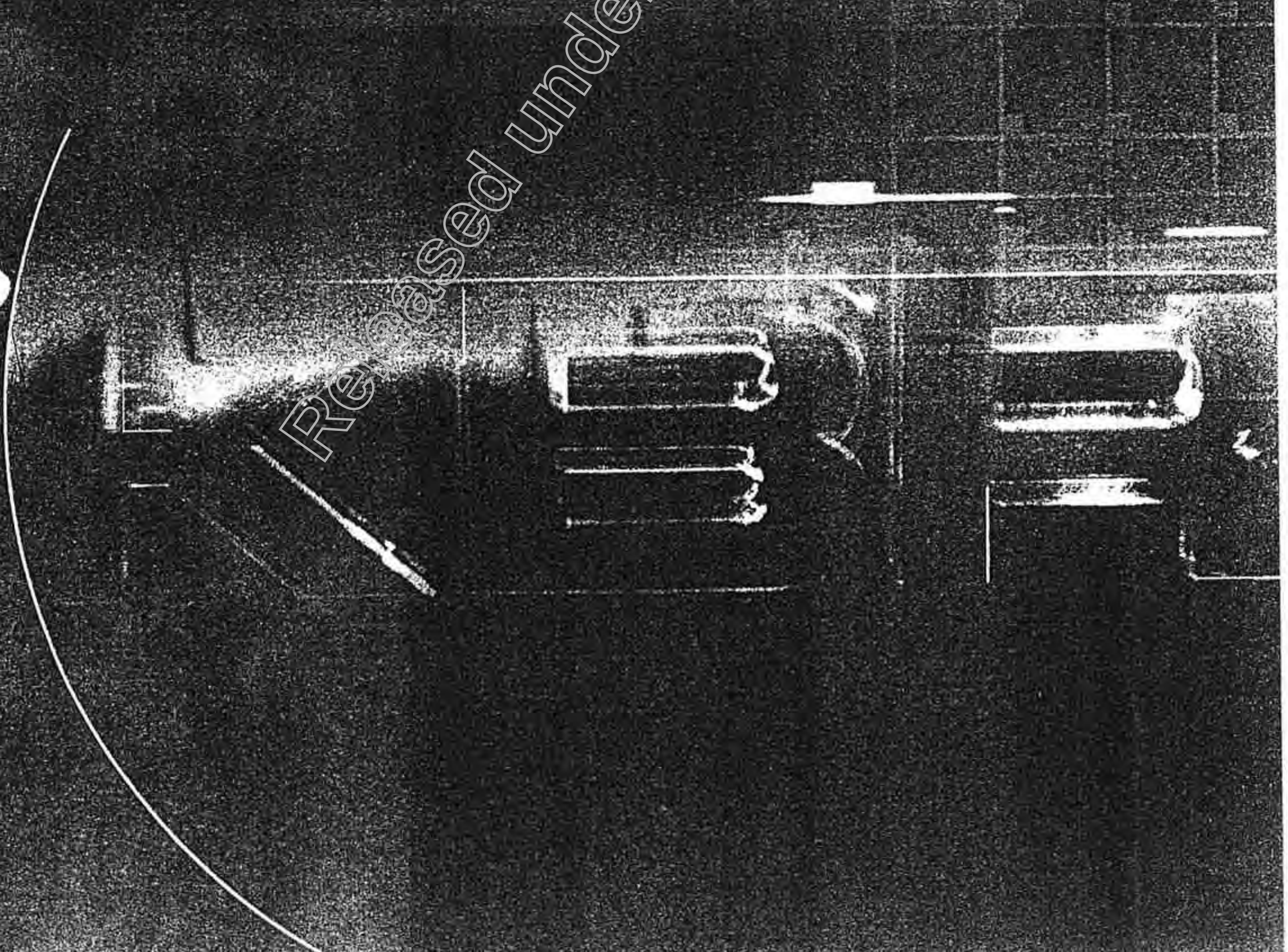
KBR

SAMFORD SUB-ARTERIAL

Local Drainage Report



SAMFORD SUB-ARTERIAL

Local Drainage Report

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5 November 2002

BEW215-W-DO-003 Rev C

Limitations Statement

The sole purpose of this report and the associated services performed by Kellogg Brown & Root Pty Ltd (KBR) is to provide drainage requirements for local catchment in accordance with the scope of services set out in the contract between KBR and Lambert & Rehbein ('the Client'). That scope of services was defined by the requests of the Client, by the time and budgetary constraints imposed by the Client, and by the availability of access to the site.

In preparing this report, KBR has relied upon and presumed accurate certain information (or absence thereof) relative to the survey and planning report provided by government officials and authorities, the Client and others identified herein. Except as otherwise stated in the report, KBR has not attempted to verify the accuracy or completeness of any such information.

This report has been prepared on behalf of and for the exclusive use of the Client, and is subject to and issued in connection with the provisions of the agreement between KBR and the Client. KBR accepts no liability or responsibility whatsoever for or in respect of any use of or reliance upon this report by any third party.

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Sheet 1 of 1

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Client Lambert & Rehbein

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This sheet records the issue and revisions of the document. If only a few revisions are made, only the new or revised pages are issued. For convenience, the nature of the revision is briefly noted under 'Remarks', but these remarks are not part of the document.

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CONTENTS

Samford Road Local Drainage Report

1 Introduction

The Main Roads Department is planning to upgrade Samford Road. As part of the final design process KBR has been commissioned, through Lambert & Rehbein, to undertake various flooding and drainage assessments of tributaries of Kedron Brook affected by the upgrade.

This report considers requirements of local drainage for minor catchments in the section between Cedar Creek and East Cedar Creek. There are five separate catchments that drain east across Samford Road. All of these catchments eventually drain towards Kedron Brook. Culverts are provided under Samford Road to drain these catchments.

During the Planning and Preliminary Design Phase of the project, Bornhorst & Ward produced a report that considered flooding aspects of the project. It was assumed, during this study, that the upgraded road would adopt the same culverts as are currently existing. However, it does not appear that the capacity of culverts was checked against required road flood immunity levels.

A more complete analysis is now required to ensure that the design can perform to the required standard. This involves assessing the capacity of the existing culverts. The objective is to provide sufficient hydraulic capacity such that there is a minimum 150 mm freeboard to the road shoulder during the ARI 50 year storm event.

2 Existing conditions

Most of the catchment area on the western side of Samford Road comprises a closed refuse tip, which has been converted into grassed playing fields. It is usual that completed landfills are capped with a low permeability layer, which tends to minimise infiltration and promote surface runoff. It is therefore expected that runoff rates from the catchment would be relatively high.

As previously mentioned, there are five discrete catchments that currently drain under Samford Road through culverts. Culvert sizes and locations are provided in Table 1.

Each culvert has been allocated a number, as shown in the table, for reference throughout this report.

Table 1 Existing drainage structures

Culvert	Approximate Samford Road Chainage (m)	Existing Drainage Details
1	5490	1/900 RCP
2	5710	1/600 RCP
3	6060	1/450 RCP
4	6140	2/1200x600 RCBC
5	6290	2/1200 RCP

It is noted that Culvert 4 is under Tramway Street and runs parallel to Samford Road.

3 Methodology

Hydrologic analysis

The first part of the local drainage analysis involves estimating the peak discharge from each catchment draining to Samford Road. The design criterion requires 150 mm freeboard to the road level at the upstream end of the culvert for the ARI 50 year event. Peak discharges have been estimated for the ARI 50 year event only.

The hydrological analysis was undertaken using the Rational Method as presented in "Australian Rainfall and Runoff" (1987). The time of concentration was calculated based on an average flow velocity over the grassed catchment of 0.55 m/s. The coefficient of runoff (C) adopted for the ARI 50 year event was 0.65 based on consideration of the losses and storm magnitude.

Hydraulic analysis

The hydraulic capacity of the existing and proposed drainage arrangements have been assessed using the Austroads Waterway Design manual. This takes into consideration flow rates, entrance losses, contraction losses and tailwater influences.

Culverts have been assessed based on a 150 mm freeboard. Details of proposed road levels and invert levels of drainage paths have been taken from the road plans provided in the planning report. The permissible headwater levels, based on these plans, are shown in Table 2.

Table 2 Allowable headwater

Culvert	Approximate Samford Road Level – M01 (mAHD)	Approximate Samford Road Natural Surface Level (mAHD)	Allowable Headwater (m)
1	55.82	54.00	1.67
2	60.27	59.00	1.12
3	58.95	57.70	1.10
4	55.11	54.00	0.96
5	54.04	52.10	1.79

Where the capacity of existing culverts (as proposed in the Preliminary Planning Study) is less than the calculated peak flow, additional waterway area is required to meet the flood criterion. In addition to the provision of additional water way area, alternative options for the drainage of excess water were also considered.

4 Description of Culverts

Culvert 1

The catchment of Culvert 1 drains a section of park land, including the closed refuse tip. The downstream portion of the flow path is a lined channel, which flows firstly under the footpath, before reaching the drain under Samford Road. The flow path reaches Samford Road north of the junction of Samford and Upper Kedron Roads. The drainage across the road is a 900 mm diameter pipe, which connects to another 900 mm pipe that directs flow under the houses on the eastern side of Samford Road. The house immediately opposite the flow path has a block fence across the flow path that would limit overland flow that may be in excess of the capacity of the pipe. Any flow in excess of the capacity of the pipe would flow over the road and then through the properties on the downstream side of the road. This water eventually reaches East Cedar Creek.

Culvert 2

Culvert 2 is located further to the north of Culvert 1 on Samford Road. It is located south of the main catchment divide and the water from this catchment also flows towards East Cedar Creek. The catchment of Culvert 2 also includes park land in the closed refuse tip. The lower part of the flow path has a short section of lined drain leading into a minor culvert under the footpath and then into the main drain across Samford Road. There is no overland flow path through the houses on the downstream side of the road.

The drainage under the road is a 600 mm diameter pipe. However the drainage immediately downstream under the houses is a 525 mm diameter pipe, so it has a smaller capacity than the pipe under the road.

Culvert 3

Culvert 3 is different from the others in this section of the road in that the flow crosses the road in the opposite direction. Water flows from a small catchment south of Claverton Street and across Samford Road from the east to the west. The catchment is mainly residential. The outflow from Culvert 3 joins the open channel that flows towards Culvert 4 under Tramway Street. The catchment is small and the drainage under the road is one 450 mm diameter pipe. Flow above the capacity of the pipe will flow over the road.

Culvert 4

Culvert 4 is a major drainage line that runs parallel to Samford Road under Tramway Street.

The catchment of Culvert 4 also consists mainly of the park land in the former refuse tip, but with some development such as the Bowls Club. The runoff from the catchment flows into an open drain that runs towards the north and parallel to Samford Road on the western side.

The drainage consists of 2/1,200x600 RCBCs. The upstream end of the culvert is at the end of the open drain. Downstream of the culvert, there is another open drain that flows towards the north past the Ferny Grove Police Station.

Culvert 5

Culvert 5 is located in the open drain that discharges from the outlet of Culvert 4. It directs water towards the east under Samford Road. The outflow from Culvert 5 flows across a small area of low lying ground before entering culverts under the railway line and taking the flow into the floodplain of Kedron Brook.

The culverts under Samford Road are 2/1,200 mm diameter pipes and the culverts under the railway line are 3/825 mm diameter pipes. The railway culverts have a smaller waterway area than the culverts under Samford Road so they will control the flow.

There is a bypass channel from the inlet of Culvert 5, which directs any flow in excess of the culvert capacity towards the north and west and into the floodplain of Cedar Creek, just upstream of the railway culverts. This is a constructed channel and seems to have been constructed to manage excess flow that does not flow through Culvert 5. In addition to the waterway area, the capacity of this culvert is also affected by the angle of the inlet and the possibility of debris. Therefore even if the capacity of Culvert 5 is inadequate there is unlikely to be any flow over the road, since the water will flow towards Cedar Creek.

5 Hydrologic analysis

The hydrologic analysis was completed using the Rational Method. A summary of the catchment properties and estimated peak discharges are presented in Table 3.

Further details of the hydrology assessment are included in Appendix A.

Table 3 **Catchment hydrology**

Culvert	Catchment Area (ha)	Time of Concentration (min)	ARI 50 year Rainfall Intensity (mm/h)	ARI 50 year Peak Discharge (m^3/s)
1	9.6	20	166	2.9
2	5.4	11	215	2.1
3	0.7	6	270	0.3
4	19.0	22	159	5.4
5	22.8	26	147	6.0

6 Hydraulic analysis (Existing)

The Preliminary Planning Study proposed that culverts identical to those in the existing road be used for the upgraded road. The capacity of the existing culverts have therefore been assessed, and the results presented in Table 4. ARI 50 year peak design discharges have been included for comparison.

Table 4 **Existing culvert capacities**

Culvert	Existing culverts	Culvert Capacity (m^3/s)	ARI 50 year Peak Discharge (m^3/s)	Existing culvert(s) suitable
1	1/900 RCP	1.70	2.9	No
2	1/600 RCP	0.58	2.1	No
3	1/450 RCP	0.34	0.3	Suitable
4	2/1200x600 RCBC	2.00	5.4	No
5	2/1200 RCP	5.20	6.0	No

Note: 1. The capacity of culverts was assessed based on the 150 mm freeboard criteria

As evident in the table, only Culvert 3 meets the design criteria. The existing arrangement at the other four culvert locations does not have sufficient capacity to pass the ARI 50 year event.

7 Drainage Options

Introduction

The culverts for these minor catchments have been analysed and options are presented for upgrading to meet the flood design standard. Details of these are as described below.

The first option was to simply increase the capacity of each culvert to convey the ARI 50 year flood. The waterway area of the culverts at these locations has been increased to meet this design criteria, with the proposed culvert arrangements summarised in Table 5. This arrangement assumes that there are no diversions and possible downstream impacts are neglected.

Table 5 Proposed culverts

Catchment	Proposed Drainage Arrangement	ARI 50 year Peak Discharge (m^3/s)	Allowable Headwater (m)	Predicted Headwater (m)
1	2/900 RCPs	2.9	1.67	1.41
2	4/600 RCPs	2.1	1.12	0.97
3	1/450 RCP	0.3	1.10	0.94
4	4/1200x600 RCBC	5.4	0.96	0.79
5	3/1200 RCPs	6.0	1.79	1.49

Following this initial assessment, each culvert location was considered in more detail to assess any alternative possibilities.

Culvert 1

Culvert 1, the catchment closest to the intersection with Upper Kedron Road, was noted as not having adequate capacity to convey the ARI 50 year flood event. The capacity of the culvert has been calculated as 1.7 m^3/s, which represents about 60% of the discharge of the ARI 50 year flood, which is 2.9 m^3/s. The culvert capacity needs to be doubled to 2/900 RCPs to convey the ARI 50 year flood from the catchment of Culvert 1.

There is a problem however with this culvert flow once it crosses the road. In the existing condition, the culvert will convey part of the ARI 50 year flood, while the excess will cross the road. On the downstream side of the road, the flow up to the capacity of the pipe will continue to flow in the pipe while the excess will flow through the downstream properties.

An alternative approach to the upgrading of this drainage is to retain the downstream piped drainage system, but divert part of the flow from the catchment upstream of Samford Road. This would involve the diversion of 1.2 m^3/s towards the east and into East Cedar Creek. This water would need to be diverted along a drainage system parallel to the road on either the northern or southern side. While both options are possible, the diversion along the northern side would seem to offer benefits, because the flow path is a shorter distance and the drainage does not need to cross the intersection of Upper Kedron Road. This option of diverting excess flow is of benefit to the properties immediately downstream of the road, since they would become protected from existing flooding problems.

As discussed below, the possibility of diverting water from Culvert 2 into this catchment is also considered a possibility. This diversion would add an additional 1.5 m^3/s to the flow into the catchment of Culvert 1. This means that there is a total of 4.4 m^3/s flowing into the culvert under the road if this water is diverted.

There are therefore two possible options for the management of drainage at Culvert 1. The first is to simply upgrade the culvert under the road and allow the excess flow above the capacity of the downstream system to flow overland. The second alternative is to upgrade the drainage under the road and then to provide for the flow parallel to the road and into East Cedar Creek.

If the first option is adopted, this will require upgrading the culvert from 1/900 RCP to 2/900 RCP, leaving the downstream conditions unaltered. This does not worsen the downstream conditions.

If the second option is adopted, the culvert under the road needs to be upgraded to 3/900 RCPs because of additional water diverted from the catchment of Culvert 2. This then needs to be in conjunction with a drain parallel to the road from the outlet of the culvert to East Cedar Creek on the northern side of the road, either as a piped system or an open channel of some type. This drain would need to have a capacity of 2.75 m^3/s. The arrangement of this system would need to be designed as part of the road drainage design. Note that this would require the construction of a surcharge pit.

Culvert 2

Culvert 2 does not have capacity to convey the ARI 50 year flood. To convey the design flood, this culvert needs to be upgraded from the existing 1/600 mm pipe to 4/600 mm pipes, a significant increase. In this case, the downstream drainage is of low capacity, similar to that of the pipe under the road. Therefore if the drainage under the road was increased, there would still be flow in excess of the downstream capacity, which would need to flow overland.

The capacity of the existing pipe is 0.6 m^3/s, 30% of the discharge of the ARI 50 year flood of 2.1 m^3/s.

As noted above in the options for Culvert 1, an option for Culvert 2 is to divert the excess flow towards the south and into the catchment of Culvert 1. This option would provide benefits to the residences downstream of the culvert by reducing the overland flow and maintaining the flow through the section to the capacity of the piped system. Diversion of this flow will require construction of a drainage channel parallel to the road towards the south. The detailed design of this would need to be part of the road design, but the flow could be conveyed by a trapezoidal channel with a base width of 0.8 m and flat side slopes of 1:6.

Culvert 3

Culvert 3 can be reconstructed with the existing capacity of 1/450 mm RCP and this will meet the ARI 50 year flood criterion.

Culvert 4

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The existing 2/1,200x600 RCBC under Tramway Street does not have sufficient capacity to convey the ARI 50 year flood. This culvert needs to upgraded to 4/1,200x600 RCBC to convey the design flood.

This is the only feasible option.

Culvert 5

The capacity of the existing 2/1,200 RCPs at this location would be 5.2 m^3/s without a downstream control, but this discharge is limited by the capacity of the railway culverts downstream. The capacity of the downstream railway culverts is only 4.1 m^3/s.

The inflow from the catchment to Culvert 5 is 6.0 m^3/s, so the culverts with their existing capacity (assuming a control from the railway culverts) can convey 70% of the total flow.

On the assumption that the channel running towards the north and into the floodplain of Cedar Creek is not altered, the excess flow above the capacity of the culvert can discharge towards Cedar creek and the existing culvert capacity is adequate. This channel must be adequate to convey a flow of 1.9 m^3/s. Based on the slopes and short distance, the overland flow channel needs to have as waterway area of 2 m^2.

This means that the existing culvert capacity does not need to be altered.

8 Discussion

While four of the culverts proposed in the Preliminary Planning Study do not meet the design criterion for the ARI 50 year event, the impact of flooding in a practical sense may be minimal. That is, since the upstream catchments are relatively small (resulting in fast response times and relatively small peak flows), the extent of overtopping and the duration of overtopping would be minimal. It is likely that neither property damage or inconvenience would be created by the floodwaters overtopping the road.

9 Conclusion

This study has identified that the culverts proposed in the Preliminary Planning Study for the Samford Road Upgrade would generally not meet the design criterion. The Preliminary Planning Study proposed that the existing culverts be matched in the new design.

Enlarged culverts with increased waterway area have been proposed to meet the design criterion of 150 mm freeboard to the design road level. In addition, alternative options have been considered.

Appendix A

HYDROLOGICAL CALCULATIONS

Summary of Rational Method Calculations

Catchment	Area (ha)	C	Time of Concentration			I (mm/hr)	Q50
			Length (m)	Av Velocity (m/s)	tc (min)		
1	9.6	0.65	650	0.55	20	166	2.9
2	5.38	0.65	375	0.55	11	215	2.1
3	0.65	0.65	210	0.55	6	270	0.3
4	18.95	0.65	710	0.55	22	159	5.4
5	22.75	0.65	850	0.55	26	147	6.0

Notes:

1. Catchment number relates to culvert number (ie. Catchment 3 drains through Culvert 3)

2. Area of Catchment 4 includes Catchment 3

3. Area of Catchment 5 includes Catchments 3 and 4

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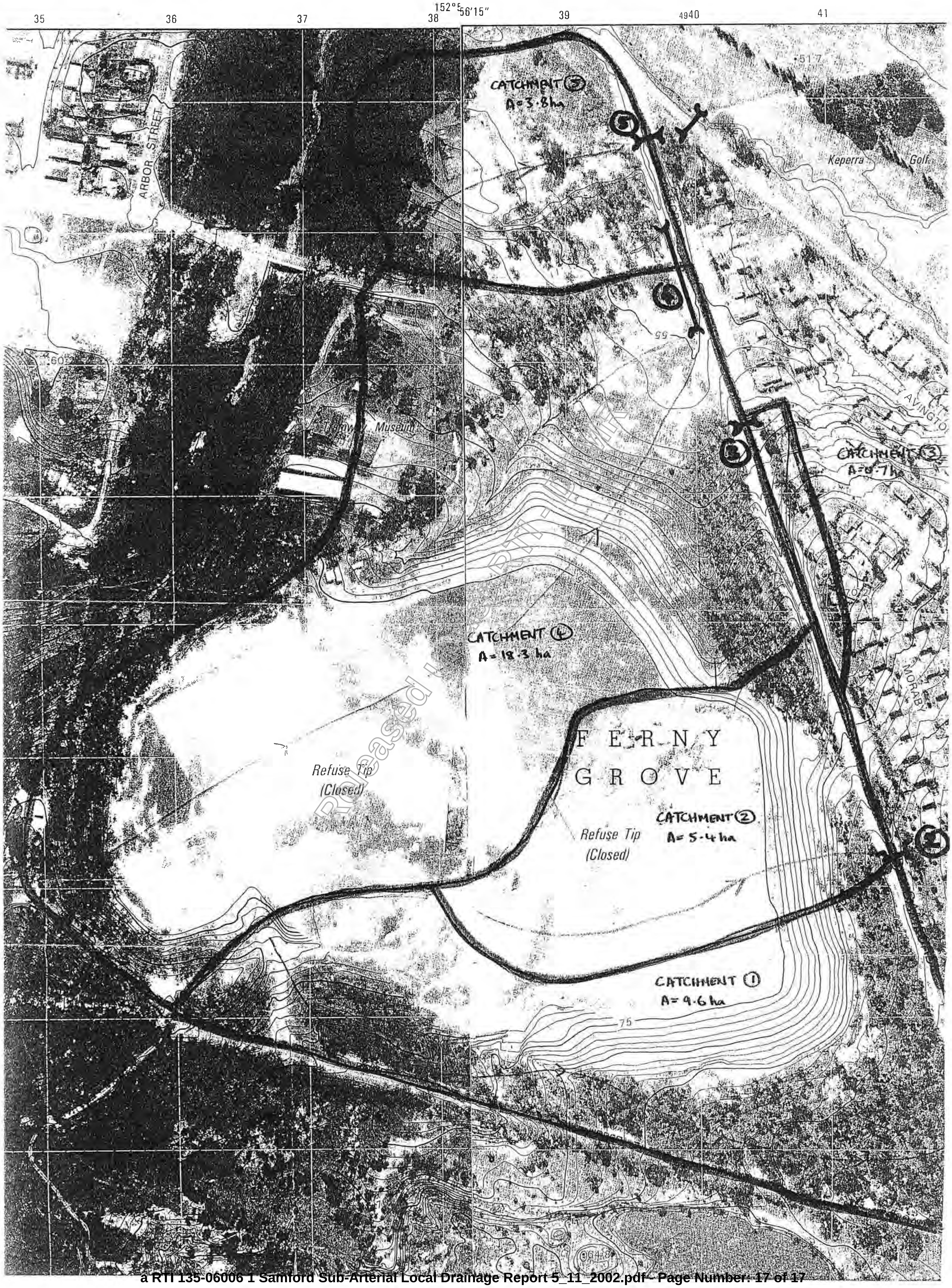

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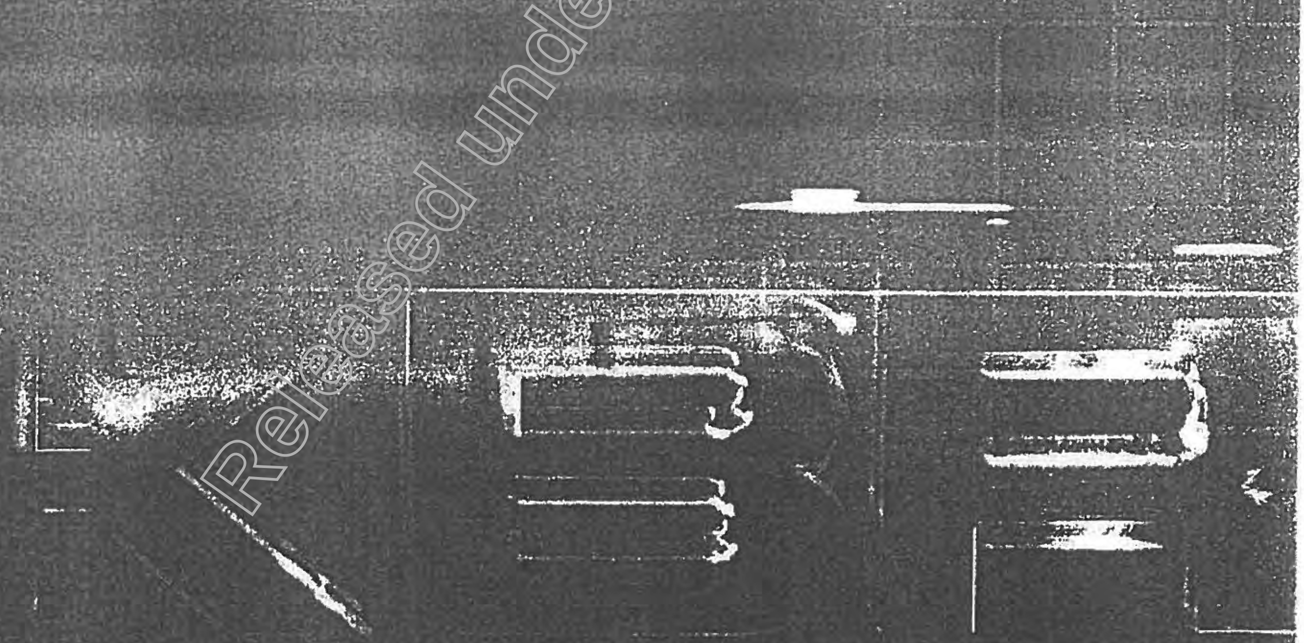
SAMFORD ROAD SUB-ARTERIAL

Supplementary Hydraulic Assessment



SAMFORD ROAD SUB-ARTERIAL

Supplementary Hydraulic Assessment

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6 November 2002

BEW215-W-DO-002 Rev B

Limitations Statement

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CONTENTS

1 Introduction

The Main Roads Department is planning to upgrade Samford Road. As part of the final design process KBR has been commissioned, through Lambert & Rehbein, to undertake a flood assessment of tributaries of Kedron Brook affected by the upgrade.

KBR completed the flood assessment and documented findings in a final report dated 7 October 2002. The assessment quantified the impact of the upgrade on flood levels and presented various alternative design options to minimise flooding.

This report is a supplement to the original flood assessment and reviews the effectiveness of three additional options for the Samford Road crossing. New options investigated are all modifications of 'Option 1', and were formulated during workshops held between Department of Main Roads, Queensland Rail (QR), Lambert & Rehbein and KBR.

For the purposes of clarity, the Samford Road Sub-Arterial Hydraulic Assessment will be referred to as the 'Main Report', and the Samford Road Sub-Arterial Supplementary Hydraulic Assessment (this report) will be referred to as the 'Supplementary Report'.

2 Background

2.1 INTRODUCTION

The section of road being investigated crosses two significant watercourses, East Cedar Creek and Cedar Creek, both of which are tributaries of Kedron Brook. A railway crossing is located just upstream of the road crossing on Cedar Creek. Currently Samford Road crosses Cedar Creek at a low level and in major storm events the floodwaters overtop the road. One of the purposes of the upgrade is therefore to raise the road above the ARI 100 year flood level.

2.2 EXISTING RAILWAY FLOOD IMMUNITY

The main report identified that the flood immunity for the railway was less than the ARI 50 year event.

Hydraulic modelling indicated that under existing conditions, the ARI 50 year event would result in a flow depth of up to 0.24 m over the railway tracks. Following the 7 October 2002 report, Queensland Rail supplied a detailed plan and long-section of the railway tracks in the section near Cedar Creek crossing. The model was consistent with the levels presented in the plan.

QR have advised that the base of the formation is used for calculation of flood immunity rather than the track level as inferred in the Main Report. This is because damage to infrastructure can occur when floodwater reaches the formation, and flow can be conveyed through the permeable formation.

The base of the formation is generally 0.43 m below the track level, and therefore under existing conditions the formation would be overtopped by up to 0.67 m in the ARI 50 year event. The flood immunity of the railway is therefore significantly less than the ARI 50 year event.

2.3 IMPROVING RAILWAY FLOOD IMMUNITY

QR have indicated that they would like to improve flood immunity of the railway. As well, it is of benefit if the upgrading of Samford Road does not have an adverse impact on the railway. This may be achieved by either raising the tracks, which would be very expensive and disruptive, or improving the hydraulic behaviour of the obstruction.

The existing head loss across the railway (ie. difference between upstream and downstream water levels) is approximately 0.8 m. Modifications to the hydraulic behaviour of the rail bridge and culverts could reduce this headloss to some extent.

Note however that there will always be some headloss across the railway because of the constriction to flow.

It would be beneficial for all stakeholders if the road design allowed for any future improvements done to the railway. For example, if QR installed additional culverts under the railway to minimise headwaters, then the road culverts would need to be of equal capacity to achieve any benefit.

This supplementary report therefore has a wider scope than the primary study, and considers other options for improving flooding at the road and railway crossings. The aim is therefore to lower flood levels at the railway, or at least retain existing levels.

3 Hydraulic modelling

3.1 INTRODUCTION

A Delft FLS model, a true 2-dimensional hydrodynamic model, was established for the original flood assessment in the area around Cedar Creek and Kedron Brook. This model has been modified to represent three new design scenarios, aiming to improve flooding at the road and railway crossing, and particularly to limit impacts to the railway.

Modelling methodology adopted for this supplementary report is consistent with that adopted in the Main Report and has therefore not been repeated.

3.2 SCENARIOS MODELLED

3.2.1 Scenario A

Scenario A involves increasing the capacity of culverts under the railway line, allowing more water to flow through the culverts, therefore decreasing the headwater level. This would be achieved by installing additional culverts under the railway line. To achieve any benefit, the capacity of culverts through the road embankment would also need to be increased. For the purposes of modelling it has been assumed that culverts through the railway and road embankments would have a similar waterway area.

Five different cases have been modelled under this scenario to assess the extent to which increasing the waterway area affects flood levels. A summary of the different cases that have been modelled is presented in Table 3.1.

Table 3.1 Culverts modelled in Scenario A

Scenarios	Railway RCBCs	Road RCBCs	Waterway area (m^2)	% increase from existing
Existing	14/1.2 x 0.9 m	11/1.5 x 0.9 m	15.12	-
A1	19/1.2 x 0.9 m	15/1.5 x 0.9 m	20.52	36%
A2	23/1.2 x 0.9 m	18/1.5 x 0.9 m	24.84	64%
A3	28/1.2 x 0.9 m	22/1.5 x 0.9 m	30.24	100%
A4	36/1.2 x 0.9 m	29/1.5 x 0.9 m	38.88	157%
A5	46/1.2 x 0.9 m	37/1.5 x 0.9 m	49.68	229%

Notes:

1. Waterway area is shown for the railway culverts. The waterway area for the road culverts has been approximately matched.

This option would be expensive and difficult to implement, particularly since the railway would need to remain operational during construction. However this option would be less expensive than the cost associated with raising the railway line

3.2.2 Scenario B

Scenario B involves combining the road and railway embankments over ~~East~~ Cedar Creek. This would be achieved by extending the existing culverts under the railway so that several long culverts penetrate the combined embankment.

This scenario would move the afflux to the upstream side of the railway and tend to divert flow away from the culverts and under the bridge to the south. This option would be significantly less expensive and disruptive to implement than Scenario A.

3.2.3 Scenario C

Scenario C involves retaining the existing railway culverts without modification and constructing the new road embankment with culverts of a similar waterway area to the railway culverts. In addition to these works, a small levee would be constructed between the railway and the road embankments to direct flow through the culverts and bridges, and also minimise the redistribution of flow between the two.

This option would be the least expensive and least disruptive to implement.

3.3 MODELLING RESULTS

3.3.1 Peak water elevations

Peak water elevations for each scenario are presented in Table 3.2. Flood levels for the existing case and for Option 1 (from the Main Report) have been included for comparison.

Table 3.2 Peak water elevation summary (m AHD)

Location	ARI	Existing	Option 1[a]	Scenario A[b]					Scenario B	Scenario C
				A1	A2	A3	A4	A5		
Samford Rd level crossing	50	54.05	54.23	54.24	54.21	54.16	54.03	54.01	54.26	54.30
	100	54.18	54.47	54.47	54.45	54.42	54.32	54.19	54.48	54.51
Railway bridge	50	53.75	53.76	53.76	53.74	53.70	53.62	53.55	53.73	53.79
	100	53.90	54.03	54.03	54.01	53.98	53.91	53.84	53.99	54.04
Railway culverts	50	53.78	54.08	54.03	53.98	53.91	53.79	53.69	53.95	54.17
	100	53.95	54.33	54.29	54.24	54.20	54.10	54.00	54.20	54.39
Road bridge	50	53.37	53.48	53.47	53.46	53.44	53.40	53.37	53.43	53.48
	100	53.59	53.75	53.74	53.73	53.72	53.68	53.65	53.71	53.74
Road culverts	50	53.22	53.26	53.26	53.27	53.25	53.24	53.22	53.27	53.34
	100	53.47	53.50	53.49	53.50	53.49	53.48	53.49	53.52	53.57

Notes:

a. Existing and Option 1 results extracted from main report

Note Low point on Rail line is 53.991 @ 20m NW of OLC towards [illegible] Grove Station

Flood level and afflux plans have not been included in this supplementary report since the differences are insignificant when compared with those presented in the Main Report.

Scenario A5, using a waterway area of 49.68 m^2 at the railway and road culverts, resulted in the greatest overall improvement in flood levels with the upgraded road. For the ARI 50 year flood there is no afflux at the railway. However, in the ARI 100 year flood there is 50 mm afflux at the railway bridge.

3.3.2 Peak water velocities

A comparison of maximum water velocity for each flood event has indicated that the peak velocities across most of the site are unchanged. Velocities through the culverts for each option range between 2 to 3 m/s in the ARI 100 year event, and it is therefore recommended that outlet protection be provided regardless of the option selected.

3.4 MODEL RELIABILITY AND ACCURACY

The results in this report rely on the performance of hydraulic modelling, with some inputs from a hydrology model. Consideration of the reliability and accuracy of the modelling is necessary since the analysis needs to consider especially small differences in flood levels where the issues of flood levels and afflux are sensitive. This is especially the case since a range of similar scenarios is compared.

The performance of any model depends on a number of factors and those that are important in this case are discussed below.

The results of this analysis consist of two components, namely the absolute water levels and the differences in levels between cases. The differences in water levels are more accurate than the absolute values, since the same assumptions are made in each case used for the calculation of the differences.

The amount and accuracy of calibration data has a critical impact on the reliability of the model results. In this case, the "calibration" has been based on a comparison with existing and accepted models, which have been previously calibrated for other locations in the Kedron Brook water course. Since the general performance of the model is quite consistent with the previously calibrated model, the results should be reasonably reliable. There is a difference in approach however since the previous work has relied on one-dimensional models, where the water level is assumed constant across a cross section, while the current analysis uses a two-dimensional approach where the water level can vary over the floodplain.

The calculation of the culvert and bridge hydraulics is especially important in this assessment and the results are analysed to compare very small differences. Culvert and bridge hydraulics are difficult to calculate to the level of accuracy required in this case, both because of the complex hydraulic behaviour as well as the potential impacts of local effects, such as debris blockage for example. The calculation of culvert and bridge hydraulics is further complicated in this project because of the close proximity of the railway and road structures and the interaction between these two structures.

It is difficult to define a level of accuracy of the absolute water levels and the differences in water levels, but an accuracy of 0.2 m for the absolute water levels and 0.05 m for the differences would seem to be reasonable. The results in this report have been presented to an apparently higher accuracy than this for comparative purposes.

Therefore while the results are presented to this high level of accuracy, the basis of the analysis must be borne in mind and the real impacts considered in this light.

3.5 DISCUSSION

As discussed in Section 2.2, under existing conditions the railway flood immunity is less than the ARI 50 year event. The construction of the upgraded road would raise flood levels at the railway for all scenarios considered in this supplementary investigation. The most effective scenario was Scenario A with a 229% increase in waterway area through the railway and road embankments, resulting in a maximum of 50 mm increase in flood levels at the railway for the ARI 100 year flood.

The practicalities and expense associated with constructing additional culverts under the railway is not considered justified. Therefore in the short term it is recommended that the road embankment be constructed to match the waterway area of the existing rail culverts, as per Option 1 in the Main Report. The level of the road should be set at the design flood immunity for the road. The flood immunity of the railway would remain below the ARI 50 year event.

If QR raise the railway above the ARI 50 year flood immunity level, this will be a relatively expensive exercise, but be the most effective at achieving flooding objectives.

4 Conclusion

The impact on flooding of three different design scenarios has been investigated in this supplementary report. These were all modifications to Option 1 from the Main Report.

It is concluded that based on all design scenarios investigated, flood levels would be increased at the railway immediately upstream of the road embankment. This would reduce the flood immunity of the railway.

There is considerable expense to upgrade the railway embankment to assist in reducing flood levels. Even with this considerable expense, there are minor benefits compared with Option 1 from the Main Report. Option 1 involves constructing a bridge of similar hydraulic capacity as the existing railway bridge (approximately 65 m in length), and installing 12 / 1.5 x 0.9 m Slab Link Box Culverts in the road embankment in the secondary flow path.

Memorandum

Our ref 1089
Your ref
Date 30/01/2018

To Shan Sivagurunathan
Copy to Chris Russell

Subject **Cedar Creek at Ferny Grove Hydrology, Hydraulics and Scour assessment**

Introduction

The TMR Hydraulics and Flooding unit has conducted a preliminary hydraulic analysis of Cedar Creek at Ferny Grove. The creek is spanned by two bridges in close succession, just upstream of its confluence with Kedron Brook. The upstream bridge is Queensland Rail (QR) bridge and the downstream one is Transport and Main Roads (TMR) bridge 36302 crossing Samford Road (U95). The bridges are approximately 40 m apart. The land located between the bridges is privately owned and is part of an adjacent commercial property. The locality plan of the subject bridges is shown in Figure 1.

The purpose of the study is to assess the cause and possible mitigation measures for a significant scour hole observed between both bridges and a substantial undermining of the upstream face of the TMR Bridge.

Hydrologic and hydraulic assessments have been completed in order to investigate potential mitigation measures. This technical memorandum details these analyses and resulting recommendations for mitigation.

Department of Transport and Main Roads
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Hydraulics and Flooding
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Figure 1 Locality plan

Terminology

A change in the use of probability terminology has been adopted in the new version of ARR. In line with these changes, TMR Hydraulics and Flooding has adopted the following changes in terminology:

- The terminology "Annual Exceedance Probability" (AEP) with results being presented as a percentage for all events of probability equal to or rarer than 50% AEP;
- The terminology "Average Recurrence Interval" (ARI) will be phased out when it is no longer necessary to refer to it.

Design rainfall intensities calculated in accordance with ARR Ed. 1987 are produced on standard ARI intervals and are a key input to the hydrological analysis in this study. As a result, the standard ARI intervals have been used by this study with converted values of AEP being presented in Table 1.

Table 1 Terminology

ARI (years)	AEP
1	63%
2	39%
5	18%
10	10%
20	5%
50	2%
100	1%
General Equation	$AEP = 1 - \exp\left(\frac{-1}{ARI}\right)$

Available data

Survey

Four sets of survey data have been made available in the area of interest. These include;

- 2002 detailed ground survey: collected to inform TMR's upgrade of Samford Road. The survey includes the Samford Road corridor in this area before the upgrade;
- 2009 Aerial Laser Survey (ALS);
- 2014 ALS; and
- 2017 ground survey: survey of the area under and between the rail and road structures to inform this investigation.

Available Previous Studies

KRB prepared a report in 2003 for TMR as part of the final design process for the section of Samford Road between Cobalt Street and Ferny Way in Ferny Grove. The models used in this previous investigation were not available, and have been conducted in a superseded software that TMR no longer has access to.

TMR obtained a report prepared by Trackstar for a rail duplication and channel works dated 2010. This is a supplementary report building on modelling that was done years earlier for a bridge replacement project. The original report and modelling were unavailable.

Brisbane City Council Kedron Brook Flood Study was originally produced in 1995 and was updated in 2014. The Cedar Creek section of the model is model using a 1D Mike 11 model. TMR was able to obtain a copy of the model. The model uses design independent storms which is no longer considered best practice. Although the one dimensional nature of the model also makes it unsuitable for this specific study, some useful information including structure details was extracted, from this model.

DTMR Interactive Mapping and Structure Drawings

DTMR interactive mapping and GIS based data layers were used for some structure details and locations.

DTMR Structure Maintenance Reports

The structure maintenance reports for the Samford Road crossing since the first one in 2006 were reviewed in order to establish the beginning of the existing undermining of this structure.

Data Review

The analysis of the various collected data sources, present a history of the changes observed in Cedar Creek and its associated crossings.

Survey data

The most recent Samford Road upgrade was completed in 2006. During these works, the available waterway area under Samford Road was significantly increased. The duplication of the rail line undertaken by the Trackstar alliance was completed around the same time.

In 2010, a hydraulic report was completed proposing earthworks under the rail bridge to provide the rail with a 1 in 20 year ARI (5% AEP) flood immunity (Refer to Section 3.3 of Keperra to Ferny Grove Rail Duplication Project Hydrologic and Hydraulic Assessment – Supplementary Report, Trackstar 2010).

The following measures were recommended in this report:

- Connection of Existing Culverts 1050/900 (TMR) and 3x825 (QR)
- Additional 900mm RCP under rail at chainage 12790m
- Waterway improvements under Cedar Creek Bridge
- Channel Improvements in between rail bridge and rail culvert (southern side of rail corridor)
- Training wall between rail and road culverts with a level of 53.8m AHD
- Debris filters on southern side of Cedar Creek overflow culverts – rail culvert at chainage 12920

It should be noted that no "As constructed" drawings of the works proposed in the 2010 report are available.

A significant undermining of the Samford Road Bridge and associated scour was first recorded at an inspection on August 2006. This was the first maintenance inspection after the road bridge was opened. This scour hole has not become significantly worse since then. However, based on various sets of survey data (ALS from 2009 and 2014 and ground survey from 2002 and 2017) it is evident that a large scour hole in the area in between the QR bridge and the Samford Road bridge developed between 2010 and 2014.

Based on the available survey data, the potential outline of events at the subject area is:

- Prior to 2005 – Cedar Creek relatively stable
- 2005/06 – TMR upgrade Samford Road structures and QR duplicate bridge/install scour protection within corridor
- August 2006 – local scour under TMR base slab inspected – no other scour between TMR and QR reported
- 2006 to 2010 – Cedar Creek relatively stable
- 2010/11 – Trackstar report recommending channel works under bridge to provide the rail with a 1 in 20 year ARI (5% AEP) flood immunity, no 'As constructed drawings" of proposed works available
- Prior to 2021/2013 – large scour hole develops

Models available

TMR's Samford Road upgrade used a now superseded 2D DELFT-FLS hydraulic model produced by an external consultant. This model used data from Brisbane City Council's (BCC) 1D Mike 11 model, including BCC's hydrology. The hydrology for this study was conducted in URBS using a DIS storm, this technique is no longer considered best practice.

The Trackstar study produced yet another new model of the area. This one was in MIKE Flood (1D-2D) and included new hydrology based on design event modelling. This model assumed blockage of the culverts. This seems reasonable due to photographs of blockage in the culverts after a major event. Different blockage has been assumed for the proposed case to represent increased waterway area.

TMR attempted to source the models developed by Trackstar and drawings to use for this investigation, however QR have advised these models cannot be located. The BCC 1D model has been sourced but is not suitable for this investigation. Therefore new models were built for the current analysis.

Hydrologic Analysis

Hydrologic analysis was undertaken in accordance with ARR87 in order to be consistent with previous studies in the area. Peak flows were compared to those reported in the 2010 Trackstar report.

Hydrologic Model Development

The hydrologic model incorporates the Cedar Creek catchment area upstream of Samford Road, and the Kedron Brook catchment area upstream of the confluence of Cedar Creek and Kedron Brook. The Cedar Creek catchment size modelled is 1600 ha and the Kedron Brook catchment size modelled is 670 ha.

Catchment SIM was used to determine catchment delineation and characteristics. The catchment delineation and slopes are based on DSM data. The catchments determined in Catchment SIM were reviewed and considered appropriate. Data was exported from Catchment SIM and used as input into an XP-RAFTS model.

Design Rainfall

Intensity Frequency Duration (IFD) data for the catchment was calculated using the standard methods prescribed in the 1987 Edition of ARR via the BoM's Design Rainfall website. IFD data is presented in Figure 2.

Home | IFD Table | IFD Chart | Coefficients | ARI | Print IFD table | Help IFD table

Intensity-Frequency-Duration Table

Location: 27.425S 152.900E Issued: 4/9/2017

Rainfall intensity in mm/h for various durations and Average Recurrence Interval

Average Recurrence Interval

Duration	1 YEAR	2 YEARS	5 YEARS	10 YEARS	20 YEARS	50 YEARS	100 YEARS
5Mins	116	149	191	216	249	295	330
6Mins	108	140	178	202	234	276	310
10Mins	88.5	114	147	167	194	230	258
20Mins	64.9	84.3	109	125	146	174	196
30Mins	52.8	68.7	89.9	103	121	144	163
1Hr	35.5	46.4	61.1	70.3	82.6	99.1	112
2Hrs	22.8	29.8	39.3	45.3	53.2	64.0	72.6
3Hrs	17.4	22.7	29.9	34.4	40.4	48.6	55.0
6Hrs	10.8	14.1	18.6	21.3	25.0	30.0	33.9
12Hrs	6.84	8.91	11.7	13.5	15.8	18.9	21.4
24Hrs	4.40	5.74	7.60	8.76	10.3	12.4	14.1
48Hrs	2.80	3.67	4.92	5.70	6.74	8.17	9.30
72Hrs	2.07	2.72	3.67	4.26	5.04	6.12	6.98

(Raw data: 47.38, 8.99, 2.76, 97.19, 18.14, 5.9, skew=0.17, F2=4.39, F50=17.23)

Copy Table

Figure 2 IFD table

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Design Event Loss Rates

Design event loss rates were estimated based on recommendations in ARR87. These values were refined during validation to ensure a reasonable match with those reported in the 2010 Trackstar report. Assumed losses are shown in Table 2.

Table 2 Design event loss rates

AEP	Pervious areas IL	CL	Impervious areas IL	CL
63% AEP	40	2	5	0
38% AEP	40	2	5	0
18% AEP	40	2	5	0
10% AEP	30	2	5	0
5% AEP	30	2	5	0
2% AEP	20	2	5	0
1% AEP	20	2	5	0

Design Event Results and Validation

The peak discharges calculated with the latest model were compared to those reported in the 2010 Trackstar report. A good match was achieved as per the results in Table 3 and Table 4 below. Consequently, current estimates are deemed suitable for use within this study.

Table 3 Peak discharges for Cedar Creek

AEP	Current Peak Discharge (m^3/s)	Trackstar Peak Discharge (m^3/s)
63%	40	-
39%	52.5	-
18%	78.7	-
10%	129	127.2
5%	168.5	163.7
2%	235.5	233.0
1%	281	273.1

Table 4 Peak discharges for Kedron Brook

AEP	Peak Discharge (m^3/s)	Trackstar Peak Discharge (m^3/s)
63%	22.8	-
39%	29.9	-
18%	40.5	-
10%	66	56.9
5%	84	73.2
2%	118	105.7
1%	138	124.3

Hydraulic Analysis

Software Platform and Modelling Approach

The two dimensional (2D) hydrodynamic modelling package TUFLOW was chosen as it has the capability to represent the complex hydraulic conditions of the Cedar Creek and Kedron Brook flood plain. The 2016 Build AE version of TUFLOW was used for all simulations.

Two cases were modelled:

- Pre–scour condition – representing conditions before the duplication of the railway (circa 2010)
- Post-scour condition - representing current conditions of the subject area (circa 2017)

Both cases assume the same structure details but different topography in the area of interest. This is in order to replicate the effect of the channel works carried out in conjunction with the rail duplication (circa 2010). The pre-scour condition utilises a combination of 2014 and 2009 ALS. The post-scour condition uses a combination of 2014 ALS and 2017 ground survey. No blockage was assumed in this study.

The model was run for the 63%, 39%, 18%, 5%, 2% and 1% AEP events. The model set up is shown in Figure 3.

Figure 3 Model set up and structure details

Boundaries and Roughness Parameters

Three upstream and one downstream boundary were assumed. Surface roughness values are presented below in Table 5. Land use types were identified across the model extent using aerial photography.

Table 5 Assumed Manning's roughness values

Land use type	Manning's n
Open space	0.030
Roads	0.020
Rail	0.030
Channel	0.025
Dense vegetation	0.080
Urban	0.075

Structures

There are a number of structures included in the model. These include the main Cedar Creek crossing, overflow structures, and some drainage structures to the east of the intersection of the road and rail. Structure details were established from a combination of previous reporting, current council modelling, ground survey and TMR structure details. The structure locations and details are shown in Figure 3. It should be noted that not all structures have been surveyed, as such inlet and outlet levels for some structures were derived based on available terrain data.

Results and Validation

Maximum water level results were provided in the Trackstar report (2010). The reporting locations were described but not shown, so the locations have been guessed for comparison. Water level results for the 1% AEP pre-scour (and pre-channel works) condition are compared in the table below. Results are generally within 300 mm. Note that current results are slightly higher than those previously produced by the Trackstar study while head loss observed across culverts is lower, due to the assumption of no blockage.

As the purpose of this investigation is to determine the cause of a scour issue and possible mitigation measures, the focus of the study are peak velocity and bed sheer stress occurring between both bridges. The highest velocity and bed sheer stresses in the area of interest generally occur during the 18% AEP event for both cases (pre and post scour), these maximum velocity and bed sheer stress are mapped in the following figures.

Table 6 Results validation

Location	1% AEP pre scour case maximum water levels	1% AEP Trackstar existing case maximum water levels
Rail Bridge US	54.57	54.13
Rail Bridge DS	54.54	54.05
Rail Culverts US	54.73	54.57
Rail Culverts DS	54.72	54.46
Samford Road Bridge US	54.54	54.0
Samford Road Bridge DS	53.95	53.84
Near Police Station	54.75	54.89

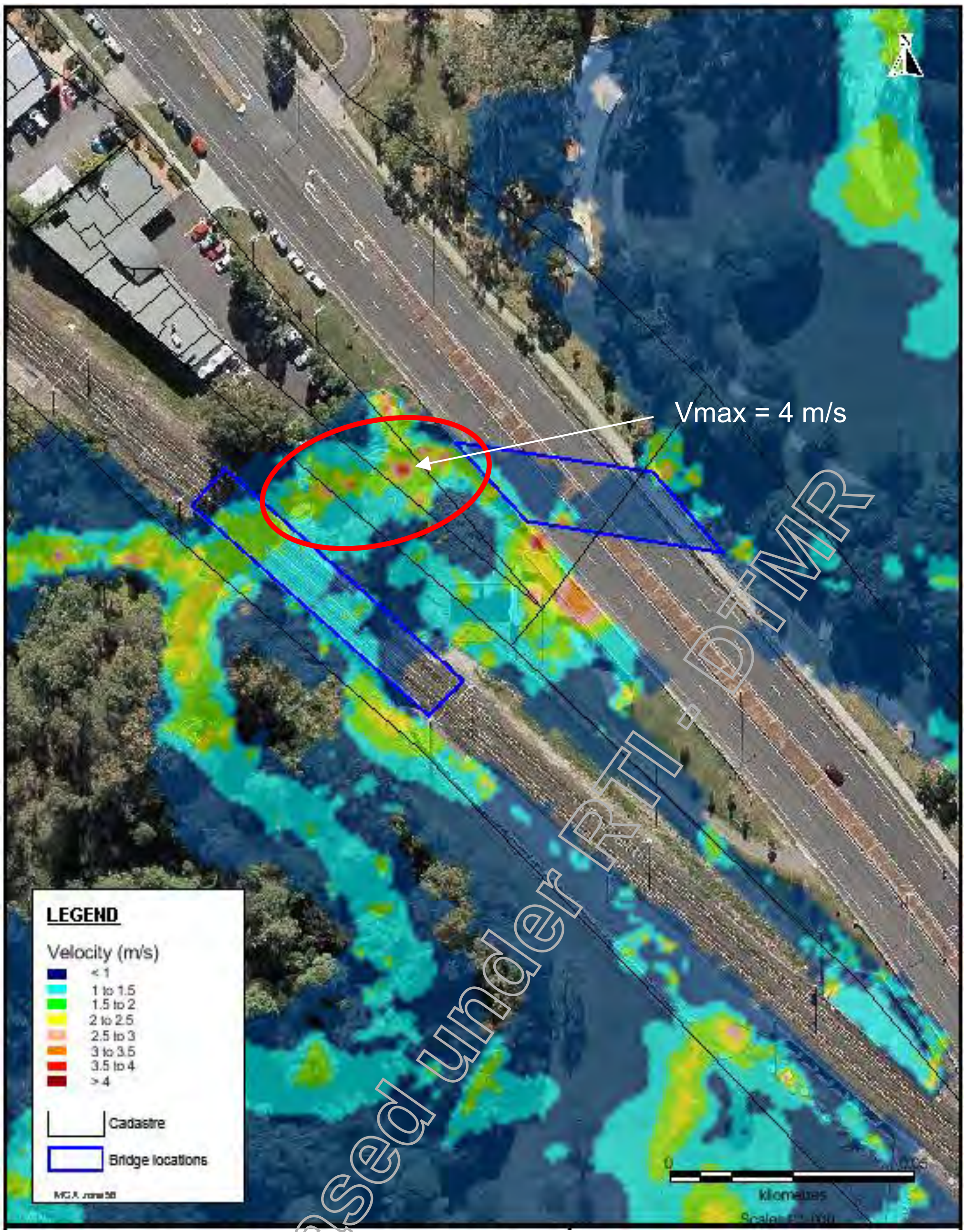

Figure 4 Pre-scour case 18% AEP maximum velocity

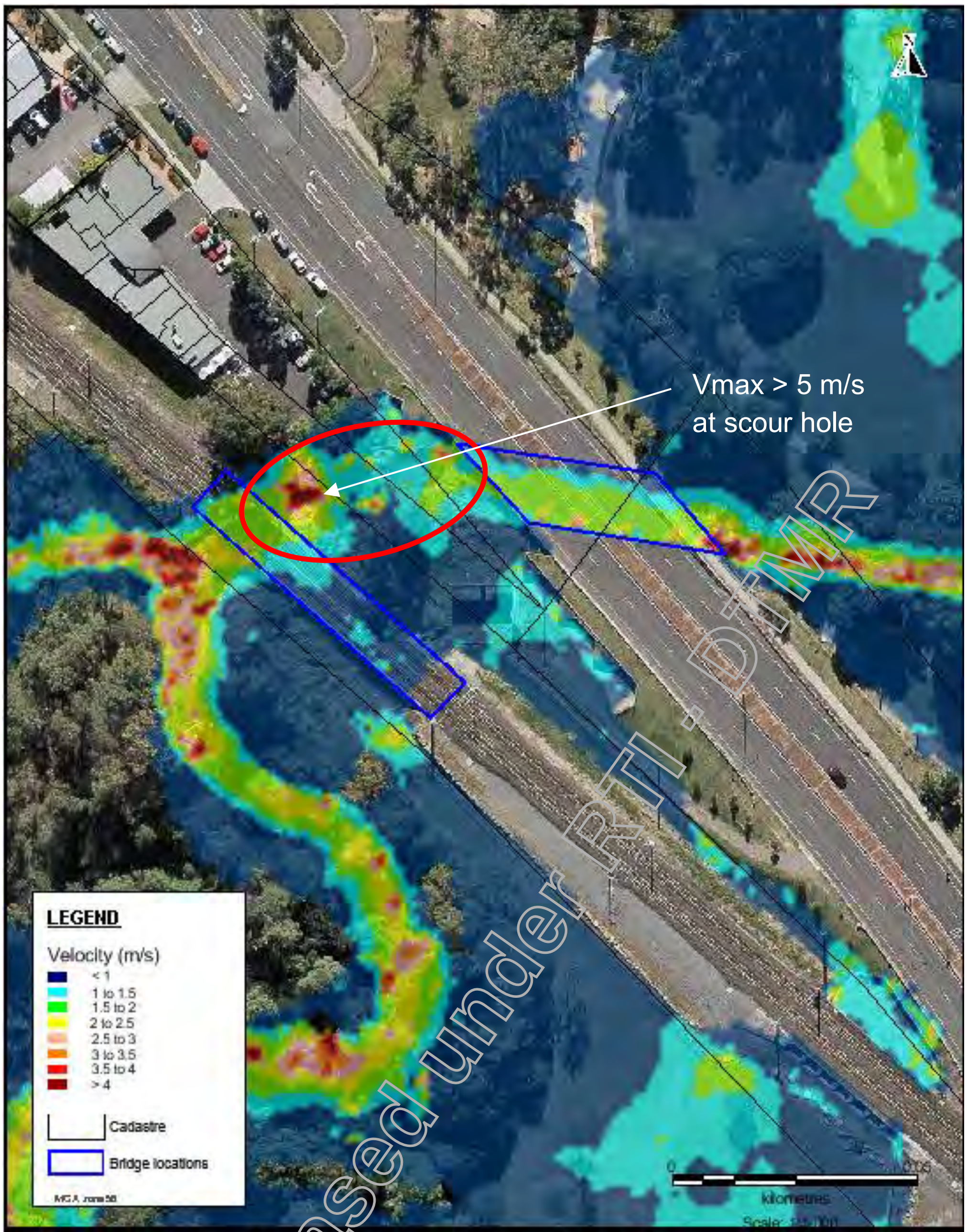

Figure 5 Post-scour case 18% AEP maximum velocity

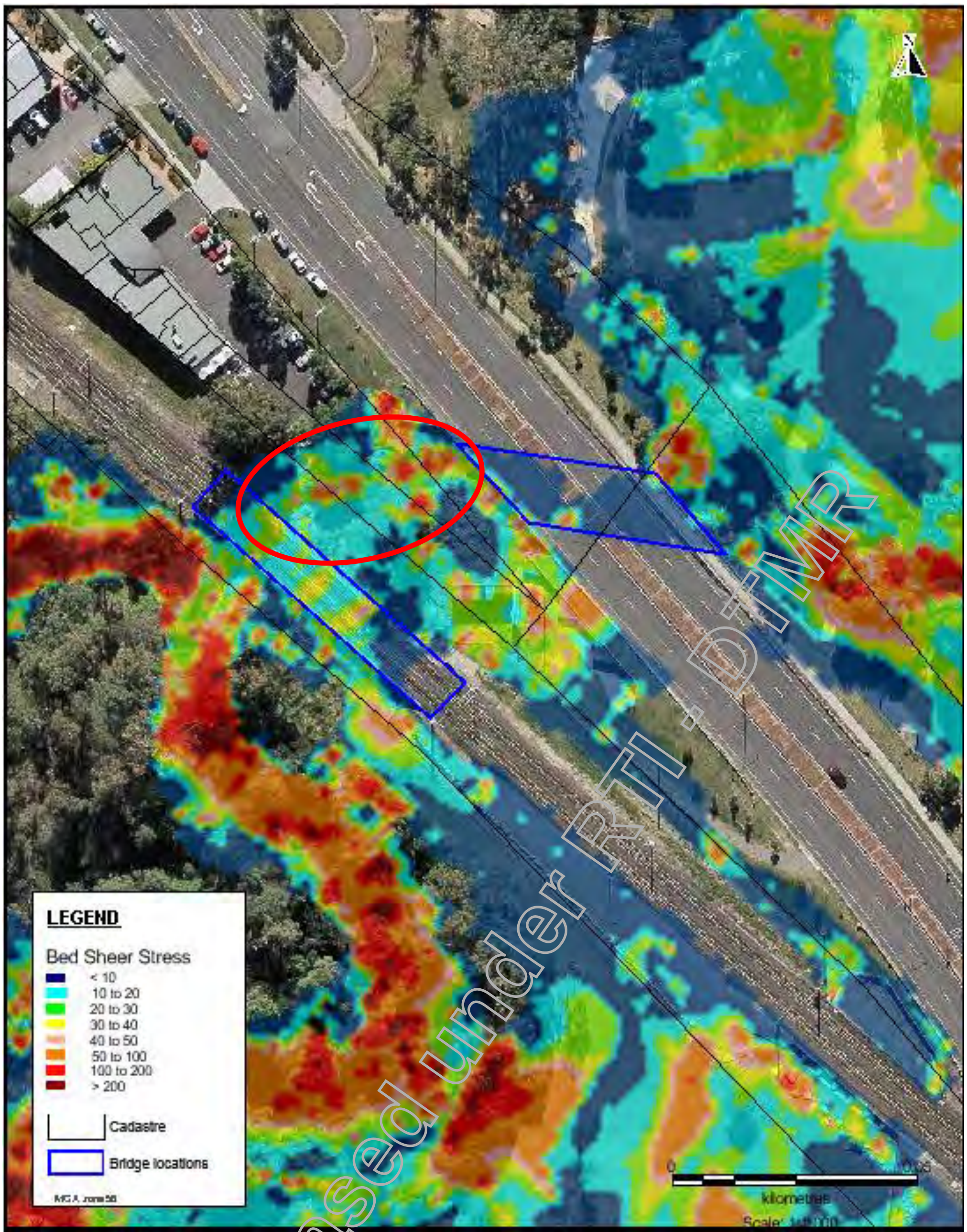

Figure 6 Pre-scour case 18% AEP maximum bed shear stress

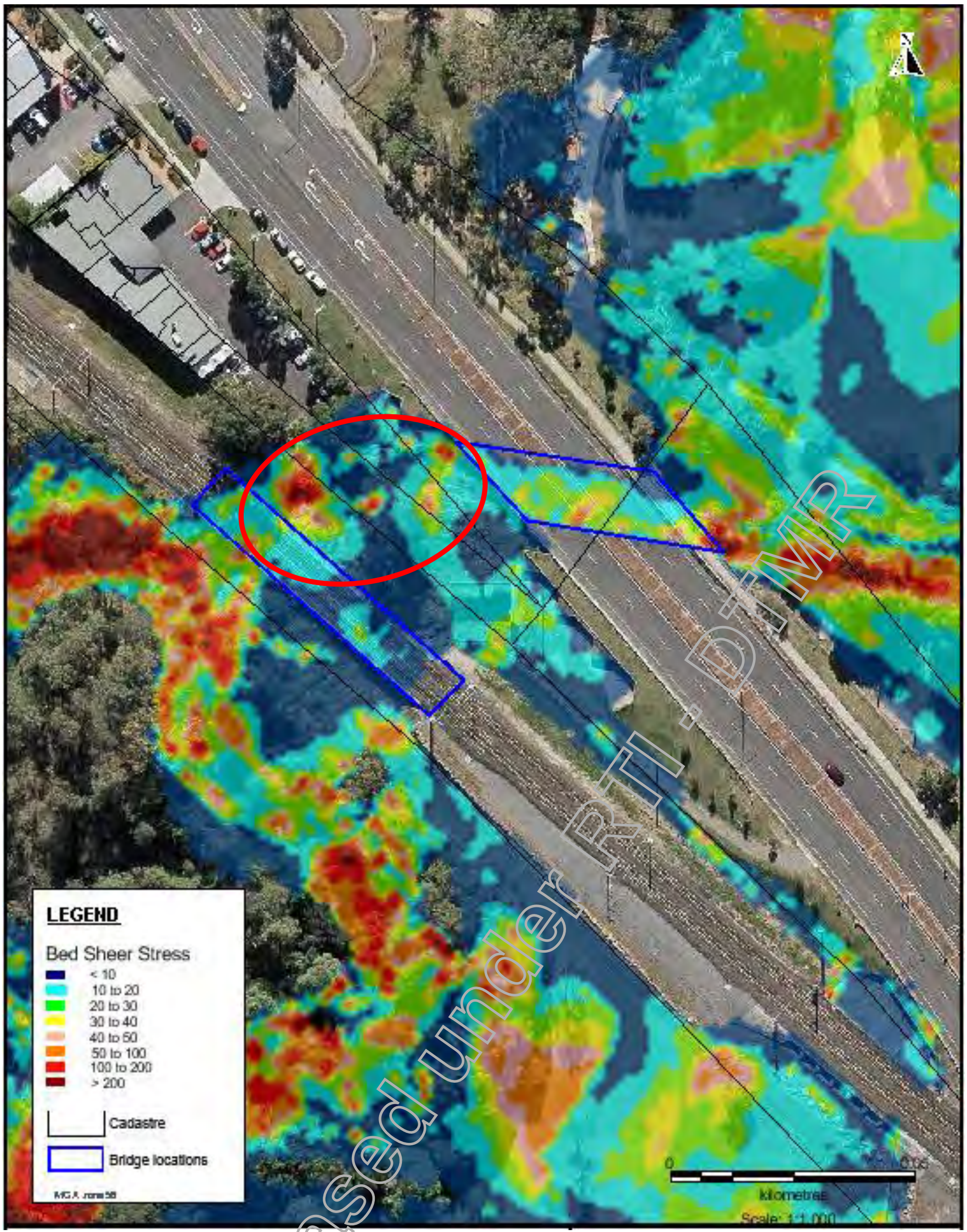

Figure 7 Post-scour case 18% AEP maximum bed shear stress

Result Analysis

Present results indicate that velocities and bed shear stress occurring between both bridges are high and have increased between pre (circa 2010) and post-scour conditions (circa 2017).

Although no “As constructed” drawings of the works proposed in the Trackstar 2010 report are available, the implementation of the works proposed in 2010 could have exacerbated the velocities and stream scour in the subject area, causing scour to progressively increase since 2010.

This is consistent with an observed large scour hole located just downstream of the rail corridor boundary. As the scour hole deepens and widens, adjacent stream banks have started to collapse inwards causing the observed undermining. Figure 8 to Figure 11 show the conditions of the scoured creek banks located between both bridges.

Figure 8 Scour hole looking upstream towards Rail Bridge

Figure 9 Existing gabions downstream of Rail Bridge

Figure 10 Collapsed banks

Figure 11 Collapsed tree due to collapsed banks

Summary and Recommendation

Results from the present hydraulic analysis indicate that velocities and bed shear stress occurring between both rail and road bridges are high and have progressively increased since 2010.

Although no “As constructed” drawings of the works proposed in the Trackstar 2010 report are available, the implementation of the works proposed in 2010 could have exacerbated the velocities and stream scour in the subject area, causing scour to progressively increase since 2010.

This is consistent with an observed large scour hole located just downstream of the rail corridor boundary. As the scour hole deepens and widens, adjacent stream banks have started to collapse inwards causing the observed undermining.

Based on the available survey data, the potential outline of events at the subject area is:

- Prior to 2005 – Cedar Creek relatively stable
- 2005/06 – TMR upgrade Samford Road structures
- August 2006 – local scour under TMR base slab inspected – no other scour between TMR and QR reported
- 2006 to 2010 – Cedar Creek relatively stable
- 2010/11 – Trackstar report recommending channel works under bridge to provide the rail with a 1 in 20 year ARI (5% AEP) flood immunity, no ‘As constructed drawings” of proposed works available
- Prior to 2012/2013 – large scour hole develops

Several options exist to protect the scoured banks between the rail and bridge to prevent further scour and undermining of the creek banks however it is envisaged that a combination of gabion walls and rock protection at the streambed might present the best option to prevent further erosion at the creek banks (see Figure 12 below for a preliminary sketch).

Geotechnical, civil and structural input is required to design the proposed mitigation measures including sizing and layout of scour protection. Dimensions shown on sketch are indicative only and are likely to change during further design stages.

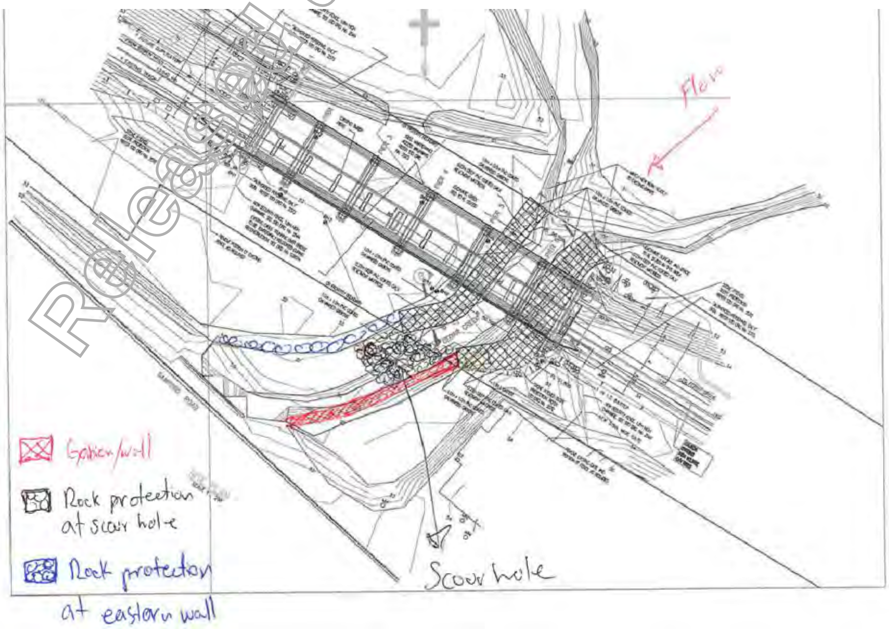

Figure 12 Preliminary sketch of proposed mitigation

Carlos Gonzalez
Principal Engineer

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Samford Road Hydraulic Assessment

January 2003 Supplementary Report No. 2

1 Introduction

The Department of Main Roads is planning to upgrade Samford Road. As part of the final design process Kellogg Brown & Root Pty Ltd (KBR) has been commissioned, through Lambert & Rehbein, to undertake a flood assessment of Cedar Creek, a tributary of Kedron Brook, which is affected by the upgrade.

Hydraulic modelling of various options for the Samford Road crossing of Cedar Creek has indicated that impacts to the neighbouring railway line will occur. The flood immunity of the railway is already approximately equal to an average recurrence interval of 50 years, which is lower than generally desired, and the proposed road upgrade would further reduce this, based on the results in the Planning Report.

The original flood assessment presented in the KBR report dated October 2002, provided an analysis of the proposed drainage works developed for the Planning Report. While the afflux produced from this analysis was similar to that provided in the Planning Report, the afflux at the railway line was considered to be a problem to the railway.

Because of this afflux problem, various options have been assessed in a previous KBR supplementary report (6 November 2002) to address the problem including:

- Increasing the capacity of the culverts;
- Increasing the span, and therefore the hydraulic capacity, of the bridge;
- Combining the road and railway embankments; and
- Constructing an internal levee to direct flow through the culverts and minimise redistribution of flows.

This report forms a second supplement to the original report (7 October 2002), and examines several additional design options.

2 Hydraulic Modelling

A DELFT-FLS model, a true two-dimensional hydrodynamic model, was established for the original flood assessment in the area around the junction of Cedar Creek and Kedron Brook, including the upstream and downstream reaches of Kedron Brook. This model has now been modified to represent three new design scenarios, aiming to improve flooding at the road and railway crossing, and particularly to limit impacts to the railway. These scenarios are described below.

Scenario E – Lowering Samford Road level

The road alignment for this scenario was provided by Lambert & Rehbein. This scenario involves lowering the design surface level of Samford Road in the vicinity of Cedar Creek. It therefore reduces the flood immunity of the road to promote weir flow over the road at a lower level and attempts to maintain the existing flood immunity of the railway. In addition to changing the level of the road, the drainage structures were also modified in this scenario.

The final road surface level would be higher than the existing road level and therefore still improve the flood immunity of the road. The level to which the road can be lowered is constrained by maximum grades and permanent features, such as the railway level crossing. The modelled scenario is at the lowest road level possible without major design modifications.

Scenario F1 – Full levee with 65 m bridge

This scenario involves the construction of a levee over the floodplain. This levee is located on the upstream side of the railway embankment and is designed to divert water towards the bridge and away from the culverts. The proposed road bridge would be 65 m span as per the design option presented in the Planning Report.

The flow patterns for the existing railway and road crossings have the main flow through the railway bridge, but there is a significant secondary flow through the railway culverts across the floodplain on the southern side of Cedar Creek. The construction of the levee is planned to divert water from the culverts to the bridge where the afflux is lower and there will be a lesser impact on flood levels produced by upgrading the road.

In large flood events the tailwater would cause backup through the culverts. The culvert dimensions are based on the Option 1 scenario, although since the flow through the culverts would be negligible, the culverts may be able to be downsized. The backwater through the culverts means that the floodplain is still inundated in major flood events.

Scenario F2 – Full levee with 75 m bridge

This scenario is a variation of Scenario F1, which involves increasing the span of the bridges by 10 m, so the total length becomes 75 m. This means that both bridges, the road and rail are increased by a similar amount. This would divert all flow through the bridge, as per Scenario F1, and would increase the capacity of the bridge to cater for these flows.

Scenario F3 – Full levee with 85 m bridge

This scenario involves increasing the span of the bridge proposed in the planning report by 20 m, so the total length is 85 m. This scenario would also divert flow through the bridge and involve the greatest increase in bridge capacity.

Scenarios F2 and F3 are variations of Scenario F1, and allow some compensation for the additional flow that is diverted through the bridge by the levee.

Scenario G1 – Partial levee over northern section

Upstream of the railway culverts there are two main flow paths over the floodplain. This option involves constructing a levee over the northern flow path and is planned to reduce flows to the culverts and direct most of the flow to the bridge, while leaving some water to flow through the culverts.

The physical and hydraulic properties of the road and rail crossings are equivalent to those presented in Option 1. The inundation areas would be similar to Option 1, however the levee would result in redistribution of flows.

Scenario G2 – Partial levee over southern section

This scenario is a variation of Scenario G1, and involves constructing a levee on the southern side of the floodplain, so is planned to provide the same type of change to the flow patterns.

3 Results

Peak water elevations for each scenario are presented in Table 1. Peak water elevations based on the existing case (as per the Main Report dated October 2002) are provided for comparison.

Table 1 Peak water elevation summary (m AHD)

Location	ARI	Existing	O1	F1	F2	F3	G1	G2
Samford Rd level crossing	50	54.05	54.23	54.01	54.01	54.01	54.28	54.23
	100	54.18	54.47	54.01	54.01	54.01	54.48	54.45
Railway bridge	50	53.75	53.76	53.96	53.91	53.86	53.80	53.81
	100	53.90	54.03	54.27	54.21	54.16	54.07	54.08
Railway culverts	50	53.78	54.08	53.37	53.21	53.28	54.08	54.07
	100	53.95	54.33	53.66	53.57	53.53	54.30	54.36
Road bridge	50	53.37	53.48	53.52	53.51	53.51	53.50	53.50
	100	53.59	53.75	53.82	53.81	53.80	53.77	53.77
Road culverts	50	53.22	53.26	53.17	53.17	53.16	53.26	53.27
	100	53.47	53.50	53.43	53.42	53.41	53.49	53.54

Note: 1. O1 refers to Option 1 results from the Original Report dated October 2002

Results for Scenario E have been omitted because preliminary modelling indicated that the option was not viable. For the option to be viable the level of the road would need to be reduced further. However, due to constraints at both ends of the road it was not possible to lower the road further. The benefits of this scenario were not as apparent as first impressions indicated, where the lower road level should allow additional flow over the embankment. This was due to the fact that the original road alignment resulted in a crest level that was actually higher than the nominal ARI 50 year flood immunity and there was some freeboard for the ARI 50 year flood in the original level. This meant that the reduction in road level did not cause a significant increase in flow over the road.

Afflux values (the difference between existing and proposed flood levels) are presented in Table 2.

Table 2 Afflux summary (m AHD)

Location	ARI	O1	F1	F2	F3	G1	G2
Samford Rd level crossing	50	0.18	-0.04	-0.04	-0.04	0.23	0.18
	100	0.29	-0.17	-0.17	-0.17	0.30	0.27
Railway bridge	50	0.01	0.21	0.16	0.11	0.05	0.06
	100	0.13	0.37	0.31	0.26	0.17	0.18
Railway culverts	50	0.30	-0.41	-0.47	-0.50	0.30	0.29
	100	0.38	-0.29	-0.38	-0.42	0.35	0.41
Road bridge	50	0.11	0.15	0.14	0.14	0.13	0.13
	100	0.16	0.23	0.21	0.21	0.18	0.18
Road culverts	50	0.04	-0.05	-0.05	-0.06	0.04	0.05
	100	0.03	-0.04	-0.05	-0.06	0.02	0.07

Note: 1. O1 refers to Option 1 results from the Original Report dated October 2002

4 Discussion

Scenarios G1 and G2 do not provide any additional benefit over Option 1. In fact, the levee causes higher velocities over the floodplain and similar flow rates pass through the culverts, causing no net benefit. The marginally increased flow through the bridge also increases flood levels at this point in excess of those for Option 1.

Scenarios F1, F2 and F3 provide the greatest benefit of all options considered. At the railway culverts all Scenario F options provide benefits with greatly reduced tail water levels. The flood levels are reduced by up to 0.5 m in the ARI 50 year event for Scenario F3.

The level of the rail over the crossing is generally at 54.00 m AHD (varies by up to +0.1 m over the section of track of interest). The flood immunity of the railway is generally taken at the base of the formation (0.5 m below the track level) and therefore the existing flood immunity of the railway is less than the ARI 50 year event. Increased flow at the railway bridge will cause flood levels to increase. For Scenario F3, in the ARI 50 year event the maximum increase at the railway line would be 110 mm, compared to 300 mm for Option 1, the originally adopted case from the Planning Report. This would have a minor adverse impact on the railway flood.

5 Conclusion

Based on the options considered in the hydraulic assessment to date (encompassing the original report, Supplementary Report No. 1 and Supplementary Report No. 2, this report), it does not appear that within the constraints of the site the existing flood immunity of the railway can be maintained.

Scenario F3 which involves construction of a levee across the floodplain upstream of the culverts and increasing the span of the two bridges by 20 m appears to provide the greatest compromise between cost and performance. The maximum afflux at the railway is 110 mm in the ARI 50 year event.

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SAMFORD SUB-ARTERIAL

Final Hydraulic Report

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11 July 2003

BEW215-W-DO-005 Rev A

Limitations Statement

The sole purpose of this report and the associated services performed by Kellogg Brown & Root Pty Ltd (KBR) is to provide a flood assessment of the proposed Samford Road Sub-Arterial Upgrade in accordance with the scope of services set out in the contract between KBR and Lambert & Rehbein ('the Client'). That scope of services was defined by the requests of the Client, by the time and budgetary constraints imposed by the Client, and by the availability of access to the site.

In preparing this report, KBR has relied upon and presumed accurate certain information (or absence thereof) relative to the study site provided by government officials and authorities, the Client and others identified herein. Except as otherwise stated in the report, KBR has not attempted to verify the accuracy or completeness of any such information.

This report has been prepared on behalf of and for the exclusive use of the Client, and is subject to and issued in connection with the provisions of the agreement between KBR and the Client. KBR accepts no liability or responsibility whatsoever for or in respect of any use of or reliance upon this report by any third party.

Revision History

Revision	Date	Comment	Signatures		
			Originated by	Checked by	Authorised by
A	11/7/03	Issue for Client Review	WDW	JOC	WDW

CONTENTS

1 Introduction

This report has been prepared for the Main Roads Department through Lambert & Rehbein to provide a flood assessment as part of the final design process for the section of Samford Road between Cobalt Street and Ferny Way in Ferny Grove. This project involves the upgrading of the section of Samford Road as Main Roads Department job number 140/U95/48. Flooding has been identified as an important issue by the Department during the planning phase of the project and a detailed analysis has therefore been carried out.

The section of road crosses two significant watercourses, which have required specific hydraulic analysis. The first of these is the relatively small stream called East Cedar Creek. The second is a more significant stream, Cedar Creek. Both are tributaries of Kedron Brook.

As well, hydraulic analysis has been carried out for the local drainage of a number of small local catchments in the project area.

This report is concerned only with the flooding aspects of the road design.

This report is a final report that consolidates the progress reports prepared during the progress of the project.

The project area is shown in Figure 1.1.

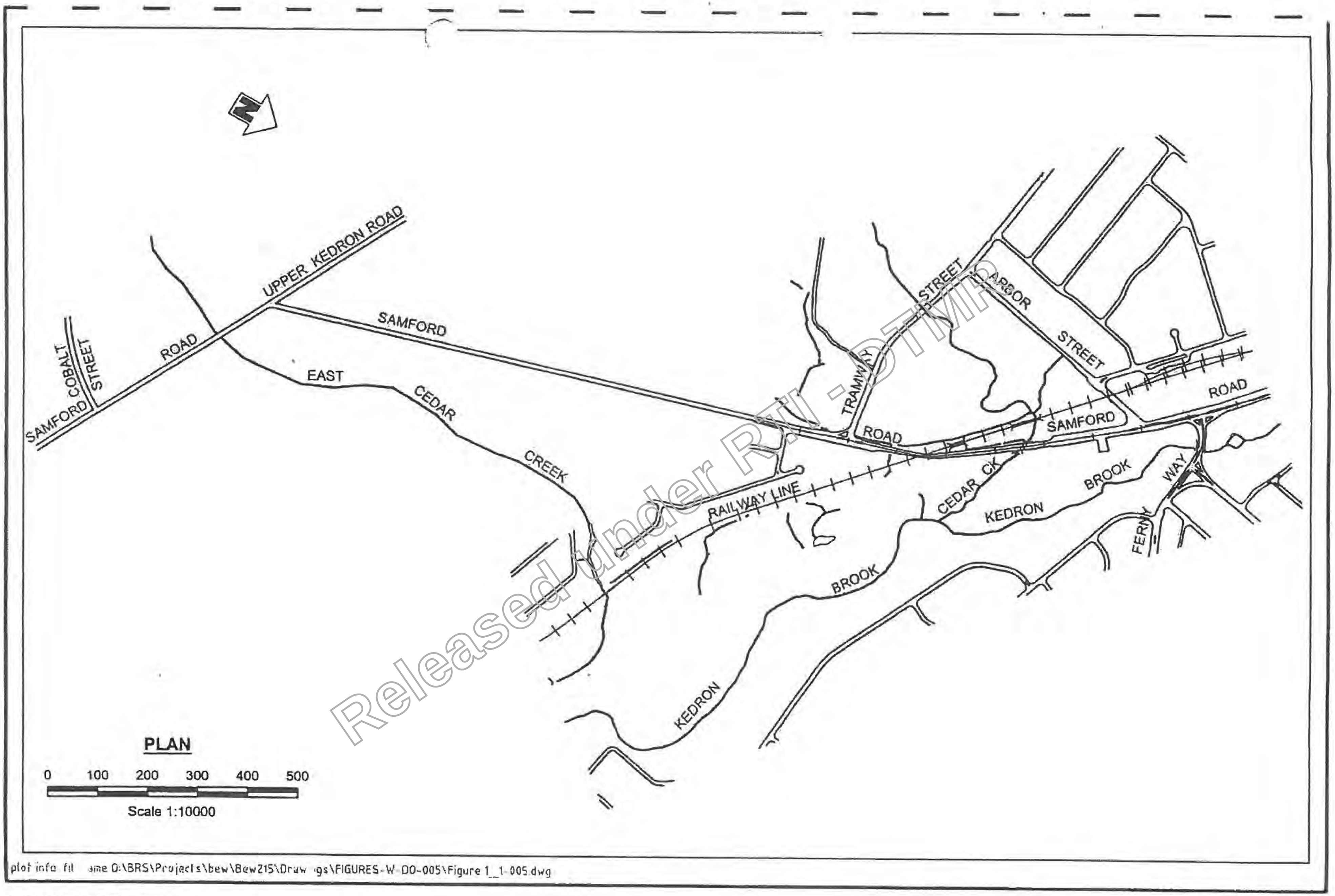

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Figure 1.1
LOCALITY PLAN

2 Background

2.1 INTRODUCTION

There are a number of previous reports available concerning flooding for this region. These have been reviewed and adapted to assist in the current project. Details of these previous reports are discussed in this section.

2.2 PLANNING PHASE REPORT

A preliminary hydraulic analysis was carried out as part of the Planning and Preliminary Design Phase of the project. Bornhorst & Ward (1998) produced this report. This preliminary analysis by Bornhorst & Ward provided sufficient information to allow a good understanding of the proposed costing of the project and to determine feasibility. A more complete analysis is now required to ensure that the design can perform to the required standard. In addition it is known that flooding is a particularly sensitive issue at this location, as it is in most urban environments.

The hydraulic investigation in the Planning Phase report used the backwater analysis model HEC-RAS. This is a steady state one-dimensional hydraulic analysis that is considered suitable for the preliminary planning phase of the project. The results indicated the suggested waterway structures required to meet the flood requirements of the project, but did not complete the analysis. Both Cedar Creek and East Cedar Creek were considered.

The Planning Phase report provided valuable background for the current detailed hydraulic analysis.

2.3 OTHER REPORTS

Information was received from the Brisbane City Council, which has carried out flood and catchment management studies for Cedar Creek and Kedron Brook. The main Brisbane City Council report was the flood study for Kedron Brook, and was carried out by Connell Wagner for the Brisbane City Council (Connell Wagner 1995), which provides the basis for floodplain management throughout Kedron Brook. This analysis uses the one-dimensional, dynamic model RUBICON for the analysis of the hydraulics. As with the hydraulic analysis for the planning report above, the study for the Brisbane City Council assumes that the water levels are flat across the whole cross section, which is a simplifying assumption. However many aspects of the work for the Brisbane City Council report are valuable for the current assessment and results have been adopted as described below.

The Brisbane City Council has also prepared a report on Cedar Creek (City Design 2002). The Cedar Creek report included hydraulic analysis with the backwater analysis program HEC-RAS, as well as hydrology assessment for the creek.

3 Project description

There are a number of different issues for the hydraulic design for the project. There are three main watercourses in the design area and there are slightly different concerns for each of these.

The minor crossing on East Cedar Creek crosses a highly modified channel. The preliminary design indicates that this crossing has a high level of flood immunity for the existing road alignment. The analysis for this relatively simple channel is straightforward.

The other two crossings, Cedar Creek on Samford Road and Kedron Brook on Ferny Way, are close to each other and are near the junction of Cedar Creek and Kedron Brook. There are several flood concerns with the Cedar Creek crossing. The first is the constriction of the flow in Cedar Creek due to the proposed road embankment upgrade, which affects flood levels upstream of the crossing. However, the effect of this constriction will be limited by the railway embankment located upstream and this railway provides a significant constraint on the proposed upgrading.

In addition to this effect, the raised road embankment may also affect the flow in Kedron Brook. This effect will be mainly from the loss of floodplain storage, but there may also be an impact from the constriction of the flow. The crossing of Kedron Brook by Ferny Way is insignificant for this project, since the flood immunity is not being affected and the road alignment is being changed only slightly. The impact however is investigated in the analysis.

There were a number of minor watercourses crossing the road and hydraulic assessment was also carried out for these. The analysis of these watercourses is called local drainage in this report.

4 Hydraulic assessment—Samford Road major drainage

4.1 OVERVIEW

Specific software is needed for the hydraulic analysis of this project and the selection of the most appropriate software is important for the successful completion of the project. The hydraulic model must be capable of representing the flow regime of the Kedron Brook and Cedar Creek junction as well as the Kedron Brook floodplain.

A true two-dimensional hydraulic model has been considered necessary to understand the flood behaviour. Firstly the flow patterns are difficult to define and there is significant flow on the floodplain away from the defined flow paths of rivers and creeks. Because of this complex flow pattern, a one-dimensional hydraulic model is not suitable because this type of model requires that the flow paths be set as part of the model establishment. A two-dimensional hydraulic model defines the flow paths based on the topography and is much more flexible in this representation.

In addition, another aspect to be considered in this project is the representation of floodplain storage. This is the issue of the effects of the storage of floodwater on the floodplain during the flood event. Storage of floodwater can provide a degree of natural flood mitigation in regions where there is a large volume of water that can be stored. It is necessary to represent the flood behaviour by the use of a dynamic model that simulates the flow of water into and out of the floodplain during the flood event.

The two-dimensional hydraulic modelling software DELFT-FLS is considered to be an excellent tool for the detailed analysis and has been applied for the analysis of the area of floodplain near the junction of Cedar Creek and Kedron Brook.

The approach to the analysis of the minor watercourse of East Cedar Creek that was carried out for the planning stage should be adequate for the assessment of the impact of the project on flooding. Additional work for this stream uses a one-dimensional hydraulic analysis developed from the planning phase. More complex modelling is not considered necessary for this watercourse.

4.2 CEDAR CREEK/KEDRON BROOK JUNCTION—HYDRAULIC ANALYSIS

4.2.1 Introduction

Detailed hydraulic modelling of the study site was undertaken using the DELFT-FLS package that was developed by Delft Hydraulics, The Netherlands. DELFT-FLS is a finite difference two-dimensional vertically averaged hydrodynamic model specifically designed to simulate relatively shallow flood propagation and recession across initially dry ground. This package was selected as it has the ability to simulate both sub-critical and super-critical flow regimes, and contains features which allow the representation of hydraulic structures such as culverts. These types of flows are a notorious source of problems in models that can only handle sub-critical flows. DELFT-FLS can model these flows accurately without becoming unstable, making it well suited to modelling flow patterns in the floodplain near the Samford Road project development.

DELFT-FLS requires a uniform rectangular grid to represent the bathymetry of the study site. For this study, a uniform grid size of 5 m x 5 m was adopted to ensure sufficient detail and an adequate representation of floodplain features including rivers, creeks, roads and structures for the hydraulic model.

The study site is contained within a rectangle, which is 1.5 km wide and 0.9 km long and is orientated parallel to the Samford Railway line (i.e. principal axis orientated 40.8° south of east). The grid extends sufficiently to incorporate the proposed Samford Road upgrade between Chainages 6,100 m and 7,200 m (approximately). The road to chainage 6,100 is above flood level.

With the adoption of a 5 m square cell grid size for the investigation, the study site equates to a grid consisting of 300 by 183 cells. This totals to 54,900 cells. However, to save in computational time, regions of high ground that do not become inundated in the 100-year ARI event have been removed from the model. The extent of the computational area is presented in Figure 4.1.

4.2.2 Establishment of bathymetry

The model bathymetry for the study site was generated from photogrammetry provided by Lambert & Rehbein undertaken in September 2000. The photogrammetry used in the project was generated from aerial photography at a scale of 1:1,500 with contours generated at 0.5 m. No vertical accuracy limit was provided with the survey.

In addition to the photogrammetry, the model bathymetry adjacent to the hydraulic structures under both Samford Road and the Railway have been based upon field survey cross-sections which were used in previous one-dimensional hydraulic modelling associated with the Kedron Brook Flood Study (1995). The accuracy of these field survey cross-sections was cross-referenced against both the photogrammetry survey and other Brisbane City Council structure surveys.

It should be noted that in areas where the photogrammetric survey has failed to provide sufficient level information in the waterways, the survey cross-sections were used to ensure a better representation of waterway conveyance. This additional information was used particularly in the constricted parts of the main channel.

Where necessary, 1:50,000 topographic maps were also used to assist in defining flow features on the floodplain in the absence of photogrammetric information.

The extent of photogrammetry used in the generation of the model bathymetry is presented graphically in Figure 4.2.

4.2.3 Hydraulic structures

A basic limitation of most two-dimensional models, including DELFT-FLS, is the inability to explicitly account for the interaction of flow with hydraulic structures such as bridges and culverts. The procedure for incorporating structure representation in one-dimensional models requires the replacement of the governing flow equations with largely empirically-based equations to represent flow behaviour at structure locations. However, this procedure also introduces inherent instabilities into the model as the flow is alternatively driven by fundamentally different forcing functions. Such instabilities would be amplified by the more complex two-dimensional flow condition. For this reason, DELFT-FLS relies on a continuous computational scheme based solely on the governing flow equations, with a resulting increase in model stability. This approach also allows for the implicit simulation of flow around structures, which is often of greater value but generally not possible in traditional one-dimensional models. In order to account for hydraulic structures, DELFT-FLS provides for their representation through the modification of local bed roughness and cell wall friction coefficients. Bed roughness is used to simulate conduit wall resistance while cell wall friction is used to simulate structure entry and exit losses. The representation of structures in this manner was used for structures under Samford Road and Railway, as well as along both Tramway Street and Ferny Way.

The railway bridge has needed particularly careful assessment, because of the close proximity of this existing structure, which has an effect on the hydraulics. In addition to the photogrammetry, the model bathymetry adjacent to the hydraulic structures under the railway have been based upon field survey cross-sections which were used in previous one-dimensional hydraulic modelling associated with the Kedron Brook Flood Study (Connell Wagner 1995) and Hydraulic Analysis of Major Cross Road Drainage Structures for Upgrading of Samford Sub-Arterial (Bornhorst & Ward 1998). The accuracy of these field survey cross-sections was cross-referenced against both the photogrammetry survey and other Brisbane City Council structure surveys.

Grade Levels to the railway line were provided after the report was submitted but no information on existing surface within the bridge area was provided to KBR.

This 130 m^2 for the railway bridge relates to clear waterway immediately upstream of the structure and is equivalent to that defined in the one-dimensional models. Reducing this by the area for the piers (10 off at 0.64 m width) gives a waterway area within the structure of 110 m^2, in agreement with what Queensland Rail (QR) has calculated. Whilst QR's own surveyed long-section is a more direct measure of the area, the photogrammetry combined with details in the one-dimensional models produced a similar equivalent area.

In order to account for hydraulic structures, DELFT-FLS does not reduce waterway area but instead provides for their representation through the modification of local bed roughness and cell wall friction coefficients. Bed roughness is used to simulate conduit wall resistance while cell wall friction is used to simulate structure entry and exit losses. Roughness and friction were defined to provide agreement between DELFT-FLS results and previous models, which have explicitly modelled the bridge structures.

The model would be locally sensitive to a reduction in waterway area through the bridge. However a reduction is not warranted because of the modified roughness and friction, which represents the effects.

4.2.4 DELFT-FLS boundary conditions

Unless there is a specific allowance, the DELFT-FLS model assumes no flow across the boundaries. Where this is not realistic, the hydrodynamic model requires boundary conditions, which describe the inputs into the study area. For this hydraulic analysis, four boundary conditions were applied.

These are:

1. inflow boundary situated on Kedron Brook (located approximately 100m upstream of the Ferny Way crossing);
2. inflow boundary situated on Cedar Creek (located approximately 180m upstream of Tramway Street);
3. inflow boundary situated on East Cedar Creek (located approximately 180m upstream of the rail crossing and 80m downstream of the Avington Street culvert);
4. water surface boundary located on the downstream boundary along Kedron Brook.

All inflow hydrographs used in the hydraulic modelling model were obtained from discharge hydrograph results produced in the Kedron Brook Flood Study (1995). A summary of peak flows for Kedron Brook and its two tributaries are presented in Table 4.1. The locations of these boundary conditions used in the DELFT-FLS model are illustrated in Figure 4.3.

Table 4.1 Summary of peak flows for Kedron Brook and tributaries at Samford Road

Watercourse	Peak discharge (m^3/s)	
	50-year ARI	100-year ARI
Kedron Brook	94.9	123.5
Cedar Creek	208.9	272.5
East Cedar Creek	68.3	80.4

4.2.5 Establishment of initial conditions

DELFT-FLS places restrictions on the water levels in the model at the start of a simulation (i.e the 'initial conditions'). Firstly, only a level water surface can be specified, and secondly, specified boundary condition locations cannot be 'dry'. Due to the initial conditions being unrealistic, the model must be run for a considerable length of time to remove any residual effects, before the desired scenario is modelled. This run is called a 'drawdown'. At the end of the drawdown, a 'hotstart' file is created. This is a file of water depths and velocities that is used to start the actual scenario run.

In generating 'hotstart' conditions for the DELFT-FLS model, three small base inflows (of 20 m^3/s) were applied at the model boundary edges along Kedron Brook, Cedar Creek and East Cedar Creek. These small inflows result in the study site not being dry at the onset of the flood event, but the residual water depths from the start-up condition are nevertheless very small. It should be noted that the magnitudes of these base inflows are insignificant compared to the peak discharges for the 50-year and 100-year ARI events and the volume of storage generated from the inflows are negligible. As the project involves analysing the existing situation and several proposed Samford Road bridge arrangements, differing model bathymetries were created. The only difference between the model bathymetries being the arrangement of the proposed road bridge upgrade scenario.

Due to the necessity of having different bathymetries, different hotstart simulations were required. It should be noted, that the differing bathymetries adjacent to Samford Road do result in slightly different initial starting water levels in Cedar Creek especially in scenarios where two sets of waterway openings under Samford Road are proposed. Notwithstanding, the difference in storage volumes resulting from the different levels in Cedar Creek is insignificant when taking into account the total flood volume.

4.2.6 Calibration of hydraulic model

Very limited hydraulic information was considered suitable for use in the calibration of the Samford Road DELFT-FLS model. However, field visits and photographs of the study site have been used in the selection of suitable Manning's n roughness coefficients. Roughness values were varied across the study area to account for the differing vegetation, topographic regularities, urban development and channel meanders. The final roughness values adopted for the DELFT-FLS model are listed in Table 4.2.

Table 4.2 Adopted Manning's n roughness coefficients

Region	Manning's n coefficient
Roads	0.020
Short, medium grass (i.e. Golf course fairways)	0.035
Medium dense vegetation	0.055
Waterways (Kedron Brook, Cedar Creek, East Cedar Creek)	0.070
Urban Development	0.095

The areal extent of assumed roughness values is shown in Figure 4.4.

Because there is limited actual recorded data for the study area, the suitability of these selected Manning's n values have been verified against hydraulic results produced from the two previous flood investigations undertaken in Kedron Brook and Cedar Creek (i.e. Kedron Brook Flood Study (BCC 1995) and the Planning Phase Report undertaken by Bornhorst & Ward in December 1998). Brief details of each of the studies are summarised below:

- Kedron Brook Flood Study covered the entire Kedron Brook catchment and included the Cedar Creek catchment. An URBS hydrologic model was used in the study to generate peak discharges and the one-dimensional hydrodynamic model RUBICON was used to generate flood levels.

- The Planning Phase Report was undertaken to provide sufficient hydraulic information to allow a good understanding of the proposed costing of the Samford Road upgrade project and to determine the feasibility. The extent of the study site is identical to that used in the current investigation. Hydraulic modelling produced in the planning phase predominantly used the Kedron Brook Flood Study as a basis for the hydraulic investigation. Details such as inflow hydrographs and tailwater conditions were both derived from the Kedron Brook Flood Study. The one-dimensional hydraulic model HEC-RAS was used in the investigation.

Both the models used previously are one-dimensional and require a network of cross-sections to calculate flood levels. They are best suited to modelling channellised waterways and are also appropriate for modelling hydraulic structures such as bridges and culverts. However, despite the advantages of one-dimensional modelling, there are several limitations which limit the suitability of this approach for the detailed hydraulic investigation of Samford Road. In general terms, these limitations are:

- for each cross-section, the one-dimensional hydraulic models assume a horizontal water level. This may be appropriate for straight, well-confined channels but in this case there are long cross-sections, which extend across flat floodplains, as well as junctions and other complex flow features;
- one-dimensional models are not well suited to determining flow splits such as junctions or embankments with several openings;
- HEC-RAS is a one-dimensional steady-state hydraulic model;
- one-dimensional models assume flow is perpendicular to the model cross-section and are not well suited to extremely flat terrain where flow patterns are difficult to define.

Because of these factors, the DELFT-FLS model was selected as the more appropriate model for the detailed analysis for the design. However despite the adoption of identical hydrology and similar realistic Manning's n roughness values, as well as the utilisation of more accurate survey information, the above mentioned limitations have caused problems in matching flood levels produced from the DELFT-FLS model and those produced from previous studies.

Due to the two-dimensional nature of the DELFT-FLS model and to aid in calibration, a range of peak water levels corresponding to the one-dimensional model cross-section have been extracted and are presented in Table 4.3.

Table 4.3 is a comparison of results between the DELFT-FLS model and the one-dimensional model results undertaken previously.

Table 4.3 Comparison of results between models (50-year ARI modelled flood level (m AHD)

Location	HEC-RAS	Rubicon	DELFT-FLS
U/s of Samford Rd	53.39	53.25	53.15–53.71
D/s of Samford Rd	53.38	53.05	53.13–53.57
U/s of Samford R'way	53.70	53.92	53.55–54.11
D/s of Samford R'way	53.43	53.35	53.28–53.81
U/s of Tramway Rd	55.59	55.57	55.61–55.89
D/s of Tramway Rd	55.14	55.32	55.33–55.69
U/s of Ferny Way	56.37	56.19	55.81–55.87
D/s of Ferny Way	55.89	56.10	55.32–55.62

Table 4.4 Comparison of results between models (100-year ARI modelled flood level (m AHD)

Location	HEC-RAS	Rubicon	DELFT-FLS
U/s of Samford Rd	53.70	53.44	53.43–53.85
D/s of Samford Rd	53.69	53.31	53.42–53.71
U/s of Samford R'way	54.27	54.19	53.76–54.28
D/s of Samford R'way	53.73	53.55	53.47–53.98
U/s of Tramway Rd	56.46	56.17	56.02–56.13
D/s of Tramway Rd	55.48	55.66	55.75–55.87
U/s of Ferny Way	56.63	56.50	55.98–56.13
D/s of Ferny Way	56.19	56.42	55.54–55.77

From Table 4.3, it can be seen that in the vicinity of Samford Road and Railway, model results produced by DELFT-FLS for both the 50-year and 100-year ARI events compare well with results produced previously in the other studies.

In particular, the largest difference in flood levels occur at Ferny Way, with the DELFT-FLS model predicting flood levels up to 0.56 m lower than both HEC-RAS and RUBICON in the 50-year ARI event and up to 0.65 m lower in the 100-year ARI event. Close inspection of the model set-ups in this area indicated that this water level difference is caused by the different definition of the weir cross-section along the crown of Ferny Way. In the one-dimensional models, the control level along Ferny Way results in an inundation width overtopping Ferny Way of approximately 40 m, whereas in the DELFT-FLS modelling, which is generated from photogrammetry, the model indicates that the width of flow over Ferny Way is closer to 90 m. The photogrammetry used in the hydraulic modelling for the current project is more complete than that used previously. The wider cross section results in a lower flood level.

Calibration summary

In conclusion, given the differences in survey information and limitations of one-dimensional hydraulic modelling, the peak water elevations produced by DELFT-FLS in the vicinity of the Samford Road upgrade are consistent with those produced in previous studies and are concluded to be within acceptable limits.

However, it is noted that in the vicinity of Ferny Way, peak water elevations produced by DELFT-FLS are considerably lower than those produced previously. However, the DELFT-FLS model is believed to provide a better representation of flow patterns in this area.

4.2.7 Model assumptions and limitations

The DELFT-FLS model set-up and analysis assumptions include:

- hydraulic roughness values do not change during the flood event;
- waterway size and shape does not change (i.e. there is no erosion or deposition);
- the DELFT-FLS model bathymetry has been generated to ensure that the flow paths provide the best representation of the overland flow paths;
- minor local drainage structures discharging into both Cedar Creek and Kedron Brook have been ignored in the analysis;
- the low tailwater at the downstream boundary is sufficiently far downstream of the area of interest that the effects from it are insignificant;
- that flow hydrographs used from the Kedron Brook Flood Study as input into the DELFT-FLS are correct.

4.3 EXISTING CONDITIONS

4.3.1 Introduction

Figure 4.5 contains a longitudinal section and plan view of the existing Samford Road arrangement contained within the model extent. Indicative road elevations have been presented at 20 m intervals. These road elevations were provided by Main Roads Department and are based upon the preliminary planning report.

The DELFT-FLS model contains approximately 960 m of Samford Road. Road elevations along the road contained in the model range from between an elevation of between a minimum of 52.23 m AHD at Chainage 6440 and a maximum of 62 m AHD.

Existing flood behaviour within the study area for both the 50-year and 100-year ARI events are described in the following sections in terms of peak water elevations, peak water velocities and flow patterns (i.e. unit flow). Lines of extent of inundation under existing conditions are also presented.

4.3.2 Peak water elevations

Maximum water surface elevation plots are presented for both the 50-year and 100-year ARI storm events in Figures 4.6 and 4.7 respectively. In addition, Figure 4.8 shows the extent of inundation for both these design events

Constrictions

From these figures, it is evident that there are two locations of significant flow constrictions along Kedron Brook. These are located:

- approximately 250 m downstream of Ferny Way;
- approximately 450 m downstream of the junction of Kedron Brook and Cedar Creek.

The most significant of the two constrictions is the Ferny Way constriction. At the narrowest point, the flow width in this region is approximately 20 m. Such a narrowing in flow width causes high velocities and in turn this produces a drop in peak water elevation of approximately 1.3 m in 70 m. The constriction appears to produce a small backwater effect upstream along Kedron Brook which may limit the existing culvert capacity under Ferny Way.

The second constriction downstream of the Kedron Brook and Cedar Creek junction is approximately 40 m wide and is sufficiently downstream of the structures under Samford Road and the Ferny Grove Railway to ensure that no backwater effects are experienced.

Ferny Grove immunity

Results from the hydraulic modelling have also indicated that the immunity of the Railway is just below a 50-year ARI event. Flood levels from the 50-year ARI event immediately upstream of the railway range from 53.51 m AHD (at the Railway Bridge) to 54.1 m AHD (at the level crossing). It should be noted that the photogrammetry survey provided for the project has indicated that the control level of the railway is between the range of 53.86 m AHD and 54.00 m AHD. As a result, a flow depth of up to 0.24 m is flowing over the railway line in the event. In both the previous hydraulic studies of the area, the controlling level along the railway has been assumed at 54.0 m-AHD and have both indicated that the railway has a 50-year ARI flood immunity. (i.e. peak water elevations of 53.7 m AHD for the planning phase report, and 53.92 m AHD for the Kedron Brook Flood Study).

The limitations of one-dimensional models are important in this location. As mentioned earlier, one-dimensional models can assign a sometimes 'unrealistic' horizontal water elevation at a cross-section. From the DELFT-FLS model results, the water elevations upstream of the railway vary significantly from one side of the creek to the other. The horizontal water level assumption will maximise the storage at the upstream railway cross-section and reduce flood levels.

For the 100-year ARI event, peak water elevations immediately upstream of the railway are within the range of 53.67 m AHD and 54.25 m AHD.

Therefore while this report indicates a slightly lower flood immunity for the railway line, the results are consistent with the more accurate hydraulic analysis.

Samford Road immunity

Unlike the railway, Samford Road has a flood immunity well under a 50-year ARI. Approximately 350 m of the road is inundated during this event with depths over the road ranging up to 1.1 m. Survey along the road has indicated the controlling low-point is approximately 52.23 m AHD at Chainage 6,440 m. Model results have indicated that at this location the 50-year ARI water level at the road is 53.33 m AHD.

Figure A.1 is a longitudinal profile of peak water surface elevations along Samford Road for both the 50-year and 100-year ARI events.

4.3.3 Peak water velocities

As for the peak water surface elevations, peak velocities are also presented for the two design storm events. These plots are contained on Figures 4.9 and 4.10. From these figures, it can be seen that velocities are generally less than 2 m/s across most of the study site. However, there are several locations where velocities in excess of 2 m/s occur. These areas are briefly discussed below:

1. In Cedar Creek, an area extending approximately 80 m upstream and downstream of the Tramway Street crossing. Immediately downstream of the bridge, velocities are in excess of 3.5 m/s in the 100-year ARI event due to a small constriction in Cedar Creek. The flow velocities at this point do not affect the area of concern for this project, near Samford Road.
2. At Ferny Grove Railway. There are several small areas where velocity exceeds 2 m/s due to a shallow depth overtopping the railway.
3. At Samford Road between 6,420 m and 6,440 m. This high velocity has resulted from shallow regions of sheet flow over Samford Road.
4. At Ferny Way to the east of the culvert. Again, this high velocity is caused by shallow sheet flow over the top of the road.
5. At the Kedron Brook constriction downstream of Ferny Way. As discussed in Section 4.4.1, this constriction is quite significant and velocities in this region are greater than 3 m/s.
6. At the Kedron Brook constriction located downstream of the Cedar Creek and Kedron Brook junction. Velocities in this region are in excess of 2.5 m/s.

4.3.4 Flow patterns (unit flow)

Plots of unit flow (or flow/metre width) have been provided to demonstrate the major flow patterns across the floodplain. Unit flow is calculated by multiplying the maximum depth (m) and the maximum velocity (m/s) together for each cell. Plots of unit flow provide an excellent indication of areas of maximum conveyance.

Figures 4.11 and 4.12 are plots of unit flow under existing conditions ('base case') for the 50-year and 100-year ARI events. These figures demonstrate that there are four distinct flow channels that dominate flow across the study site. These are described below:

1. Kedron Brook flow path
2. Cedar Creek flow path (through Railway bridge)
3. Cedar Creek flow path (through Railway culvert)
4. East Cedar Creek flow path.

Of these flow paths, it can be seen that flow from Cedar Creek dominates flow across the area of road included for upgrading. Cedar Creek has two defined floodplain flow channels, one of which passes under the railway bridge and then through/over the Samford Road culverts before joining Kedron Brook and the other that passes through the railway culverts and then overtops Samford Road.

A schematic representation of flow patterns under the 'base case' during large flood events is presented in Figure 4.13.

The unit flow plots also indicate that there are two main breakouts from Cedar Creek that influence the hydraulics near Samford Road and the railway.

The first of these breakouts (Location A) is a small depression that is located approximately 100 m downstream of the Tramway Street crossing. The depression runs in an easterly direction from Cedar Creek and has a control level of approximately 53.7 m AHD.

The second breakout point (Location B) along Cedar Creek is located approximately 60 m upstream of the Samford Railway. This breakout primarily occurs due to the influence of a small peninsula which directs flow away from the railway bridge and towards the railway culverts. The control level at this breakout location is approximately 52.8 m AHD.

The locations of these breakout points are also presented in Figure 4.13.

4.4 OBJECTIVES FOR ROAD UPGRADING

4.4.1 General

A longitudinal profile of the proposed upgrade of Samford Road investigated in this analysis is illustrated in Figure 4.14. The horizontal alignment has remained the same as in the existing conditions scenario ('base case'), although the road width has been increased from two lanes to four lanes. The vertical alignment (produced from control line M001) along Samford Road was provided by the Department of Main Roads with the intention of providing for a 50-year ARI flood immunity. Appendix B contains copies of longitudinal sections (Plan Nos 313868 and 313869) and drainage and alignment plans (Plan Nos 313858 and 313859) produced as part of the Planning and Preliminary Design. It should be noted that the vertical alignment along control line M001 has been maintained for the analysis.

Flooding objectives relating to the design of the waterway openings under Samford Road are as follows:

1. To provide a flood immunity of Samford Road, with a 150 mm freeboard to the road shoulder.
2. To ensure a 100-year ARI flood immunity for upstream and downstream properties as required by the Queensland Urban Drainage Manual (QUDM) and by Brisbane City Council.
3. To ensure that afflux (i.e. the increase in flood level produced by the construction of the proposed works) does not encroach on any residential property.
4. To ensure that the construction of the road upgrade will have a limited impact on the distribution of flow throughout the study site. Large affluxes are a good indicator of redistribution of flow.

4.4.2 'Control' level along Samford Road

Determination and adoption of a correct 'control' (or weir) level along Samford Road is an important component of the hydraulic analysis as it controls the total flow over the road.

The control level used in the hydraulic model has been based upon longitudinal levels as produced from the control line M001 provided by Main Roads (Refer Plan Nos 313868 and 31389 contained in Appendix B). M001 is located on the median lane edge of the proposed road cross-section. A copy of a typical cross-section along the road in the vicinity of the Cedar Creek crossing is illustrated in Figure B.1 and shows the exact location of M001.

The elevation of the control line is not the 'controlling level' for flood discharge along Samford Road. From Figure B.1, it can be seen that the preliminary planning report allows for a median strip to be constructed along Samford Road. As the median lane is higher than M001, the control level for flood overflows will be the top of the median lane.

The standard height for a median strip is 150 mm. To determine the controlling height along Samford Road, 150 mm has been added to the M001 elevation. This controlling elevation has been included in the hydraulic modelling of the upgrade options. This is the road level that must be used for the analysis of flood levels of events that overtop the road.

The second concern with the road level is the determination of the appropriate flood level for the defined design flood criterion. This is an issue that must be assessed from the information defined here. The proposed road alignment across the floodplain of Cedar Creek is shown in Figure B.1. In the area of concern for the hydraulic modelling, the road has two traffic lanes with a width of 3.3 m each and a bicycle lane with a width of 2.0 m. This total width of 8.6 m has a cross fall of 2.5%. The height of the shoulder therefore is 0.215 m below the control level. With the required freeboard of 0.15 m at the shoulder, the flood immunity criterion means that the 50-year ARI flood level should be 0.365 m below the control level.

4.4.3 Required waterway openings for Samford Road upgrade

Previous studies

The hydraulic assessment undertaken by Bornhorst & Ward as part of the planning phase was used to determine a preliminary arrangement of waterway openings suitable to meet the design guidelines stated above. A summary of the proposed structures required under the road recommended in the report are summarised below:

1. A bridge of similar hydraulic capacity to the existing railway bridge. The proposed road bridge is approximately 65 m in length and is to be located approximately between Chainages 6,522 and 6,587 m. The existing culverts under the road are to be replaced with the proposed bridge structure.

2. 12/1.5 m x 0.9 m slab link box culverts to be located downstream of the existing railway culverts.

Assessment of waterway openings

To assess the impacts of the Samford Road upgrade, the flood behaviour under existing conditions has been accepted as the 'base case' and changes to the flood regime produced by the proposed road upgrade are assessed relative to this base condition. The DELFT-FLS model of the existing conditions has been modified to incorporate the proposed road upgrade and associated waterway openings and then analysed to ascertain that the design satisfies all flooding objectives.

In addition to the proposed arrangement of waterway openings recommended in the planning phase of the project (referred to as Option 1), three additional waterway arrangements were also assessed to determine the hydraulic impacts of reducing and increasing the amount of waterway openings under the road. A brief description of each of the four proposed waterway opening options are summarised:

- *Option 1 (recommended option from planning phase)*: Waterway opening configuration as recommended in planning study. (i.e. 65 m bridge length and 12 RCBCs).
- *Option 2:* Waterway opening configuration consisting of a 65 m bridge (similar capacity to the existing railway bridge) and no allowance for culvert structures. This option was developed to assess the impact of removing the culverts in the secondary flow path.
- *Option 3:* Similar to Option 2, but with an increase in bridge length to 75 m. The increased bridge length is to accommodate for the proposed culvert openings (approximate area of culverts is 20 m^2). This option provides approximately the same water way area as Option 1 but uses only one structure.
- *Option 4:* Similar to Option 1, but with a reduced bridge size to 40 m in length. This option maintains two structures, but considers the possibility of reducing the length of bridge necessary.

Road longitudinal general arrangement plans for each of the proposed waterway configuration options are illustrated in Figures 4.14 to 4.17.

Each of the options were assessed with both the 50-year ARI and 100-year ARI events.

4.5 OPTION 1

4.5.1 General

This option is the waterway configuration recommended in the planning report.

The required waterway area associated with this option was based upon matching the existing waterway area currently existing under the Ferny Grove Railway. Under existing conditions, the total waterway area under the Samford railway is approximately 150 m^2 (130 m^2 of this is contained in the railway bridge and the remaining 20 m^2 in the railway culverts).

With the inclusion of the proposed road upgrade and recommended waterway opening configuration, the hydraulic model was re-run, with the hydraulic performance discussed as follows.

4.5.2 Peak water elevations

Maximum water surface elevation plots for Option 1 are presented for both the 50-year and 100-year ARI flood events in Figures 4.18 and 4.19 respectively.

From Figure 4.18, it can be seen that the actual 'controlling level' of Samford Road is not overtopped by the 50-year ARI event and all flow is conveyed through the two structures under the upgraded road. During the 50-year ARI flood, of the total flow of 208 m^3/s in Cedar Creek, approximately 75% (155 m^3/s) flows through the bridge with the remaining flow (53 m^3/s) passing through the culverts.

However, while the road is not overtopped, the flood level of the 50-year ARI event is above the level needed for the design criterion. There are two locations centred on chainages 6280 and 6400 where the calculated flood level is above the required design flood level of 0.365 m below the control level (allowing for the 150 mm freeboard for the road shoulder).

In addition to the plots of peak water surface elevation, Figure A.2 is a longitudinal profile of peak water elevations along Samford Road during both the 50-year ARI and 100-year ARI event. From this, it can be seen that the highest water elevation upstream of Samford Road in the 50-year ARI event is 54.23 m AHD.

While the design level of the road does not meet the flood criterion, the impact of the proposed upgrading on the 50-year ARI flood would not be affected if the road level was raised, since the road is not actually overtopped in the 50-year ARI flood event. There would however be a greater impact on the 100-year ARI flood if the road level was raised to meet the flood criterion, since there would be a smaller depth of overtopping.

4.5.3 Peak water velocities

A comparison of maximum water velocity for each flood event has indicated that the peak velocities across most of the site are unchanged. However, at Samford Road, significant reductions in velocity have occurred with the construction of the new embankment and hydraulic structures. This has occurred because in the existing conditions scenario, flow over the road is acting similar to a broad-crested weir and flow is super-critical as it passes over the road. In the developed cases, all flow is passing through the bridge and culverts and there are no longer any regions of shallow flow. Because the flow velocities are changed to a very limited amount from the existing conditions, maps of the differences in velocity have not been plotted.

The model has indicated that velocities near the road culverts are in excess of 2.5 m/s. To prevent scouring it is recommended that appropriate energy dissipating devices be implemented downstream of the culverts.

4.5.4 Afflux

The afflux (above the existing water surface levels) plots for the 50-year ARI and 100-year ARI events are presented in Figures 4.20 and 4.21 respectively.

From these figures, it can be seen that in both flood events, afflux is highest in the small region between the rail and the road (Chainage 6,420 m). It should be noted that these high affluxes do not affect flooding of any residential properties and is only restricted to the area between the road and the rail.

These high affluxes have occurred because under existing conditions once flow overtops the existing Samford Road near Chainage 6,440 m, it is unrestricted in discharging towards Kedron Brook. However, with the inclusion of the higher and wider road embankment, flow which originally crossed the existing road now banks up on the upstream side and is contained within the region between the road and the railway. The culverts and bridge structure proposed under the road have sufficient capacity in ensuring that the road is not overtopped in the 50-year ARI event although additional waterway area involving additional culverts or a longer bridge could be added to reduce this afflux.

The proposed waterway openings have produced positive affluxes ranging up to 395 mm at the railway near the existing culvert structures. In the 100-year ARI event, the highest recorded afflux at the railway is 470 mm.

Upstream of the railway, affluxes are reduced although in the 50-year ARI flood, positive affluxes of up to 150 mm occur up to about 160 m upstream of the railway. These affluxes do not affect residential properties.

However, it is noted that in the 100-year ARI event, affluxes up to 150 mm occur in a small waterway immediately downstream of Arbor Street, which is an issue because of the proximity of residential property in the area.

In conjunction with positive affluxes upstream of the proposed works, negative affluxes or reductions in peak water elevations have occurred downstream due to a redistribution of flow resulting from the waterway openings. The largest negative afflux is located downstream of the road near road Chainage 6,440 m, just downstream of the culverts. At this location, the largest negative affluxes are 220 mm and 266 mm for the 50-year and 100-year ARI events respectively.

Figures 4.20 and 4.21 also demonstrate the positive affluxes occurring downstream of the proposed bridge due to a redistribution of flow. As mentioned previously, a peak flow of 155 m^3/s passes through the bridge in the 50-year ARI event, whereas under existing conditions the peak flow through this area was less. The additional flow through the bridge has caused an increase in flood levels, that is a positive afflux.

4.6 OPTION 2

This option investigates the affects of not constructing the culverts but still maintaining a 65 m long bridge of a similar capacity to the existing railway bridge.

Hydraulic modelling of this arrangement has identified that the 65 m long bridge under the railway is not sufficient to convey the 50-year ARI event as most of Samford Road between the Chainages of 6,260 m and 6,587 m is now inundated. It should be noted that despite the new bridge conveying a peak flow of approximately 200 m^3/s in the 50-year ARI event, the backwater caused from the constriction has produced flooding of the property located in the corner of Samford Road and the railway. To demonstrate the effects of Option 2, afflux maps for both the 50-year and 100-year ARI events are provided as Figures 4.22 and 4.23 respectively.

Figure A.3 is a longitudinal profile of peak water elevations along Samford Road during both the 50-year ARI and 100-year ARI event

4.7 OPTION 3

As in Option 2, the Option 3 arrangement again involves providing a single bridge opening under Samford Road, without providing a secondary waterway by constructing culverts under the road. The secondary flow path with the culverts was recommended in the planning study and analysed in Option 1. The only difference between the waterway configurations is that in Option 3 the proposed road bridge is extended by 10 m, (from 65 m to 75 m) to allow for the additional waterway area of the proposed culverts (20 m^2).

The modelling results show that despite the additional area under the road bridge, peak water elevations in the 50-year ARI event overtop the crown level of the road and this also results in inundation of residential property. Thus Option 3 fails to provide the required 50-year ARI flood immunity. Reductions in peak water elevations with those produced in Option 2 have occurred upstream of the rail but these reductions are generally less than 20 mm. Afflux maps of the Option 3 configuration for the design events are presented in Figures 4.24 and 4.25.

Figure A.4 is a longitudinal profile of peak water elevations along Samford Road during both the 50-year ARI and 100-year ARI event

4.8 OPTION 4

The final arrangement (Option 4) investigated in this component consists of providing two sets of waterway openings. The waterway configuration proposed under Option 4 is a 40 m long bridge (25 m shorter than the original recommended option) and the recommended 12/1.5 m x 0.9 m culverts.

This option was developed primarily for two reasons:

1. To determine the impacts of reducing the capacity through the road bridge, thereby saving some of the cost of the bridge construction;
2. To make a comparison of hydraulic performance of Option 4 against Option 1.

The reduction in waterway opening from shortening the bridge length from 65 m to 40 m results in the road being overtopped in the 50-year ARI event and higher water levels upstream of the road. A comparison of peak water elevations immediately upstream of the road bridge between Option 4 and Option 1 indicates that water levels increase by up to 340 mm in the 50-year ARI event and 405 mm in the 100-year ARI event.

An effect arising from shortening the road bridge is increased affluxes in the vicinity of the Arbor Street watercourse. Despite affluxes of 222 mm and 309 mm occurring in the 50-year ARI and 100-year ARI event respectively, no residential properties are inundated. The afflux at this location therefore will not have an adverse impact.

Complete plots of afflux for Option 4 are presented in Figures 4.26 and 4.27.

Figure A.5 is a longitudinal profile of peak water elevations along Samford Road during both the 50-year ARI and 100-year ARI event.

4.9 OPTION SUMMARY

4.9.1 Peak water elevation summary

In addition to the longitudinal profiles of peak elevations contained in Appendix A, Table 4.4 is presented to enable an easier comparison of the performance of each of the four Samford Road upgrade options. This table contains a summary of peak water elevations at five key locations in the study site.

These five locations are essentially located at the centreline of all existing and proposed structure locations, as well as at the railway crossing. As a result, it should be noted that these peak levels provided at the culverts may not be the true maximum value for each of the structures (due to the water gradients produced in two-dimensional models). However, to enable a relative impact to be determined the water elevation at the same location in the model has been recorded for all options.

Table 4.5 Peak water elevation summary—all options (m AHD)

Location	ARI	Existing	Option 1	Option 2	Option 3	Option 4
Samford Road Level Crossing	50 100	54.05 54.18	54.23 54.47	54.38 54.56	54.36 54.54	54.31 54.53
At Railway Bridge	50 100	53.75 53.90	53.76 54.03	53.93 54.13	53.87 54.08	53.97 54.22
At Railway Culverts	50 100	53.78 53.95	54.08 54.33	54.36 54.54	54.33 54.52	54.18 54.41
At Road Bridge	50 100	53.37* 53.59*	53.48 53.75	53.57 53.80	53.55 53.79	53.58 53.82
At Road Culvert	50 100	53.22* 53.47*	53.26 53.50	54.28* 54.38*	54.26* 54.36*	53.34 53.58

*Note: * indicates there is no structure at this location. The water level presented in the table is at the same location as where the structure would be located.*

From Table 4.4, it is clearly evident that the waterway arrangement associated with Option 1 provides the lowest water elevations at the five key locations for both the 50-year and 100-year ARI events.

4.10 ADDITIONAL OPTIONS (AFFECTING THE RAILWAY)

4.10.1 Introduction

Following the assessment of the four initially identified options, a number of additional options were analysed to refine the analysis and to provide a better result to attempt to meet the Department's objectives.

This section therefore follows the original flood assessment and reviews the effectiveness of three additional options for the Samford Road crossing. New options investigated are all modifications of Option 1, and were formulated during workshops held between Department of Main Roads, Queensland Rail (QR), Lambert & Rehbein and KBR.

4.10.2 Background

Existing railway flood immunity

The initial hydraulic analysis identified that the flood immunity for the railway was less than the ARI 50-year event.

Hydraulic modelling indicated that under existing conditions, the ARI 50-year event would result in a flow depth of up to 0.24 m over the railway tracks. Queensland Rail supplied a detailed plan and long-section of the railway tracks in the section near Cedar Creek crossing. The model was consistent with the levels presented in the plan.

The base of the formation is used for calculation of flood immunity rather than the track level because damage to infrastructure can occur when floodwater reaches the formation, and flow can be conveyed through the permeable formation.

The base of the formation is generally 0.43 m below the track level, and therefore under existing conditions the formation would be overtopped by up to 0.67 m in the ARI 50-year event. The flood immunity of the railway is therefore significantly less than the ARI 50-year event.

Improving railway flood immunity

QR have indicated that they would like to improve flood immunity of the railway. As well, it is of benefit if the upgrading of Samford Road does not have an adverse impact on the railway. This may be achieved by either raising the tracks, which would be very expensive and disruptive, or improving the hydraulic behaviour of the obstruction.

The existing head loss across the railway (i.e. difference between upstream and downstream water levels) is approximately 0.8 m. Modifications to the hydraulic behaviour of the rail bridge and culverts could reduce this headloss to some extent, but there will always be some head loss across the railway because of the constriction to flow.

It would be beneficial for all stakeholders if the road design allowed for any future improvements done to the railway. For example, if QR installed additional culverts under the railway to minimise headwaters, then the road culverts would need to be of equal capacity to achieve any benefit.

4.10.3 Hydraulic modelling

Scenarios modelled

Scenario A

Scenario A involves increasing the capacity of culverts under the railway line, allowing more water to flow through the culverts, therefore decreasing the headwater level. This would be achieved by installing additional culverts under the railway line. To achieve any benefit, the capacity of culverts through the road embankment would also need to be increased. For the purposes of modelling it has been assumed that culverts through the railway and road embankments would have a similar waterway area.

Five different cases have been modelled under this scenario to assess the extent to which increasing the waterway area affects flood levels. A summary of the different cases that have been modelled is presented in Table 4.6.

Table 4.6 Culverts modelled in Scenario A

Scenarios	Railway RCBCs	Road RCBCs	Waterway area (m^2)	% increase from existing
Existing	14/1.2 x 0.9 m	11/1.5 x 0.9 m	15.12	—
A1	19/1.2 x 0.9 m	15/1.5 x 0.9 m	20.52	36%
A2	23/1.2 x 0.9 m	18/1.5 x 0.9 m	24.84	64%
A3	28/1.2 x 0.9 m	22/1.5 x 0.9 m	30.24	100%
A4	36/1.2 x 0.9 m	29/1.5 x 0.9 m	38.88	157%
A5	46/1.2 x 0.9 m	37/1.5 x 0.9 m	49.68	229%

Notes:

1. Waterway area is shown for the railway culverts. The waterway area for the road culverts has been approximately matched.

This option would be expensive and difficult to implement, particularly since the railway would need to remain operational during construction. However this option would be less expensive than the cost associated with raising the railway line

Scenario B

Scenario B involves combining the road and railway embankments over East Cedar Creek. This would be achieved by extending the existing culverts under the railway so that several long culverts penetrate the combined embankment.

This scenario would move the afflux to the upstream side of the railway and tend to divert flow away from the culverts and under the bridge to the south. This option would be significantly less expensive and disruptive to implement than Scenario A.

Scenario C

Scenario C involves retaining the existing railway culverts without modification and constructing the new road embankment with culverts of a similar waterway area to the railway culverts. In addition to these works, a small levee would be constructed between the railway and the road embankments to direct flow through the culverts and bridges, and also minimise the redistribution of flow between the two.

This option would be the least expensive and least disruptive to implement.

Modelling results

Peak water elevations

Peak water elevations for each scenario are presented in Table 4.7. Flood levels for the existing case and for Option 1 have been included for comparison.

Table 4.7 **Peak water elevation summary (m AHD)**

Location	ARI	Existing	Option 1[a]	Scenario A[b]					Scenario B	Scenario C
				A1	A2	A3	A4	A5		
Samford Rd level crossing	50	54.05	54.23	54.24	54.21	54.16	54.03	54.01	54.26	54.30
	100	54.18	54.47	54.47	54.45	54.42	54.32	54.19	54.48	54.51
Railway bridge	50	53.75	53.76	53.76	53.74	53.70	53.62	53.55	53.73	53.79
	100	53.90	54.03	54.03	54.01	53.98	53.91	53.84	53.99	54.04
Railway culverts	50	53.78	54.08	54.03	53.98	53.91	53.79	53.69	53.95	54.17
	100	53.95	54.33	54.29	54.24	54.20	54.10	54.00	54.20	54.39
Road bridge	50	53.37	53.48	53.47	53.46	53.44	53.40	53.37	53.43	53.48
	100	53.59	53.75	53.74	53.73	53.72	53.68	53.65	53.71	53.74
Road culverts	50	53.22	53.26	53.26	53.27	53.25	53.24	53.22	53.27	53.34
	100	53.47	53.50	53.49	53.50	53.49	53.48	53.49	53.52	53.57

Notes:

a. Existing and Option 1 results extracted from main report

Flood level and afflux plans have not been included for these cases since the differences are insignificant when compared with those presented in the Main Report.

Scenario A5, using a waterway area of 49.68 m^2 at the railway and road culverts, resulted in the greatest overall improvement in flood levels with the upgraded road. For the ARI 50-year flood there is no afflux at the railway. However, in the ARI 100-year flood there is 50 mm afflux at the railway bridge.

Peak water velocities

A comparison of maximum water velocity for each flood event has indicated that the peak velocities across most of the site are unchanged. Velocities through the culverts for each option range between 2–3 m/s in the ARI 100-year event, and it is therefore recommended that outlet protection should be provided regardless of the option selected.

4.10.4 Discussion

As discussed above, under existing conditions the railway flood immunity is less than the ARI 50-year event. The construction of the upgraded road would raise flood levels at the railway for all scenarios considered in this supplementary investigation. The most effective scenario was Scenario A with a 229% increase in waterway area through the railway and road embankments (Scenario A5), resulting in a maximum of 50 mm increase in flood levels at the railway for the ARI 100-year flood.

The practicalities and expense associated with constructing additional culverts under the railway would be difficult to justify. Therefore this additional analysis indicated that the road embankment be constructed to match the waterway area of the existing rail culverts, as per Option 1 above. The level of the road should be set at the design flood immunity for the road. The flood immunity of the railway would remain below the ARI 50-year event.

If QR raise the railway above the ARI 50-year flood immunity level, this will be a relatively expensive exercise, but be the most effective at achieving flooding objectives.

4.10.5 Conclusion

Three additional scenarios were analysed, all modifications to Option 1.

It is concluded that based on all design scenarios investigated, flood levels would be increased at the railway immediately upstream of the road embankment. This would reduce the flood immunity of the railway.

There is considerable expense to upgrade the railway embankment to assist in reducing flood levels. Even with this considerable expense, there are minor benefits compared with Option 1 from the Main Report. Option 1 involves constructing a bridge of similar hydraulic capacity as the existing railway bridge (approximately 65 m in length), and installing 12/1.5 x 0.9 m Slab Link Box Culverts in the road embankment in the secondary flow path.

4.11 RAILWAY IMPACT OPTIONS

4.11.1 Introduction

Additional scenarios were then assessed to limit impacts on the railway, since these impacts were identified as a particular concern.

4.11.2 Hydraulic modelling

The additional scenarios are described below.

Scenario E—Lowering Samford Road level

The road alignment for this scenario was provided by Lambert & Rehbein. This scenario involves lowering the design surface level of Samford Road in the vicinity of Cedar Creek. It therefore reduces the flood immunity of the road to promote weir flow over the road at a lower level and attempts to maintain the existing flood immunity of the railway. In addition to changing the level of the road, the drainage structures were also modified in this scenario.

The final road surface level would be higher than the existing road level and therefore still improve the flood immunity of the road, though at a lower level than required. The level to which the road can be lowered is constrained by maximum grades and permanent features, such as the railway level crossing. The modelled scenario is at the lowest road level possible without major design modifications.

Scenario F1—Full levee with 65 m bridge

This scenario involves the construction of a levee over the floodplain. This levee is located on the upstream side of the railway embankment and is designed to divert water towards the bridge and away from the culverts. The proposed road bridge would be a 65 m span as per the design option presented in the Planning Report.

The flow patterns for the existing railway and road crossings have the main flow through the railway bridge, but there is a significant secondary flow through the railway culverts across the floodplain on the southern side of Cedar Creek. The construction of the levee is planned to divert water from the culverts to the bridge where the afflux is lower and there will be a lesser impact on flood levels produced by upgrading the road.

In large flood events the tailwater would cause backup through the culverts. The culvert dimensions are based on the Option 1 scenario, although since the flow through the culverts would be negligible, the culverts may be able to be reduced. The backwater through the culverts means that the floodplain is still inundated in major flood events.

Scenario F2—Full levee with 75 m bridge

This scenario is a variation of Scenario F1, which involves increasing the span of the bridges by 10 m, so the total length becomes 75 m. This means that both bridges, the road and rail are increased by a similar amount. This would divert all flow through the bridge, as per Scenario F1, and would increase the capacity of the bridge to cater for these flows.

Scenario F3—Full levee with 85 m bridge

This scenario involves increasing the span of the bridges proposed in the planning report by 20 m, so the total length is 85 m. This scenario would also divert flow through the bridge and involve the greatest increase in bridge capacity.

Scenarios F2 and F3 are variations of Scenario F1, and allow some compensation for the additional flow that is diverted through the bridge by the levee.

Scenario G1—Partial levee over northern section

Upstream of the railway culverts there are two main flow paths over the floodplain from the creek. This option involves constructing a levee over the northern flow path and is planned to reduce flows to the culverts and direct most of the flow to the bridge, while leaving some water to flow through the culverts.

The physical and hydraulic properties of the road and rail crossings are equivalent to those presented in Option 1. The inundation areas would be similar to Option 1, however the levee would result in redistribution of flows.

Scenario G2—Partial levee over southern section

This scenario is a variation of Scenario G1, and involves constructing a levee on the southern side of the floodplain, so is planned to provide the same type of change to the flow patterns.

4.11.3 Results

Peak water elevations for each scenario are presented in Table 4.8. Peak water elevations based on the existing case and Option 1 are provided for comparison.

Table 4.8 Peak water elevation summary (m AHD)

Location	ARI	Existing	O1	F1	F2	F3	G1	G2
Samford Rd level crossing	50	54.05	54.23	54.01	54.01	54.01	54.28	54.23
	100	54.18	54.47	54.01	54.01	54.01	54.48	54.45
Railway bridge	50	53.75	53.76	53.96	53.91	53.86	53.80	53.81
	100	53.90	54.03	54.27	54.21	54.16	54.07	54.08
Railway culverts	50	53.78	54.08	53.37	53.31	53.28	54.08	54.07
	100	53.95	54.33	53.66	53.57	53.53	54.30	54.36
Road bridge	50	53.37	53.48	53.52	53.51	53.51	53.50	53.50
	100	53.59	53.75	53.82	53.81	53.80	53.77	53.77
Road culverts	50	53.22	53.26	53.17	53.17	53.16	53.26	53.27
	100	53.47	53.50	53.43	53.42	53.41	53.49	53.54

Note: 1. O1 refers to Option 1 results from the Original Report dated October 2002

Results for Scenario E have been omitted because preliminary modelling indicated that the option was not viable. For the option to be viable the level of the road would need to be reduced further. However, due to constraints at both ends of the road it was not possible to lower the road further. The benefits of this scenario were not as apparent as first impressions indicated, where the lower road level should allow additional flow over the embankment. This was due to the fact that the original road alignment resulted in a crest level that was actually higher than the nominal ARI 50-year flood immunity and there was some freeboard for the ARI 50-year flood in the original level. This meant that the reduction in road level did not cause a significant increase in flow over the road.

Afflux values (the difference between existing and proposed flood levels) are presented in Table 4.9

Table 4.9Afflux summary (m AHD)

Location	ARI	O1	F1	F2	F3	G1	G2
Samford Rd level crossing	50	0.18	-0.04	-0.04	-0.04	0.23	0.18
	100	0.29	-0.17	-0.17	-0.17	0.30	0.27
Railway bridge	50	0.01	0.21	0.16	0.11	0.05	0.06
	100	0.13	0.37	0.31	0.26	0.17	0.18
Railway culverts	50	0.30	-0.41	-0.47	-0.50	0.30	0.29
	100	0.38	-0.29	-0.38	-0.42	0.35	0.41
Road bridge	50	0.11	0.15	0.14	0.14	0.13	0.13
	100	0.16	0.23	0.21	0.21	0.18	0.18
Road culverts	50	0.04	-0.05	-0.05	-0.06	0.04	0.05
	100	0.03	-0.04	-0.05	-0.06	0.02	0.07

Note: 1. O1 refers to Option 1 results from the Original Report dated October 2002

4.11.4 Discussion

Scenarios G1 and G2 do not provide any additional benefit over Option 1. In fact, the levee causes higher velocities over the floodplain and similar flow rates pass through the culverts, causing no net benefit. The marginally increased flow through the bridge also increases flood levels at this point in excess of those for Option 1.

Scenarios F1, F2 and F3 provide benefits. At the railway culverts all Scenario F options provide benefits with greatly reduced tail water levels. The flood levels are reduced by up to 0.5 m in the ARI 50-year event for Scenario F3.

The level of the rail over the crossing is generally at 54.00 m AHD (varies by up to +0.1 m over the section of track of interest). The flood immunity of the railway is generally taken at the base of the formation (0.5 m below the track level) and therefore the existing flood immunity of the railway is less than the ARI 50-year event. Increased flow at the railway bridge will cause flood levels to increase. For Scenario F3, in the ARI 50-year event the maximum increase at the railway line would be 110 mm, compared to 300 mm for Option 1, the originally adopted case from the Planning Report. This would have a minor adverse impact on the railway flood.

The impacts of Scenario F1 (the most feasible of the options involving the levee) on the property near Arbor Street have been assessed in detail.

General observations include:

- ARI 50 and 100-year flood levels are currently below the Arbor Street road level. The ARI 100-year flood level for the existing condition is just on the road level;
- the development option including the levee would increase the ARI 50 and ARI 100-year flood levels to above the Arbor Street road level;
- the option will increase the ARI 50-year flood level by 0.24 m, and cause a maximum inundation at the road of up to 0.11 m over a length of 6 m;
- the option will increase the ARI 100-year flood level by 0.41 m, and cause a maximum inundation at the road of up to 0.41 m over a length of 43 m.

Detailed ground survey of the adjacent lots is not available, but ground survey was carried out for the residences with the lowest floor levels. The lowest of the three floor levels and three carport levels was surveyed as 54.639 m AHD. This is higher than the ARI 100-year flood level of just over 54.4 m AHD for Option F1.

4.11.5 Conclusion

Based on analysis of all of the options considered in the hydraulic assessment, it does not appear that within the constraints of the site the existing flood immunity of the railway can be maintained.

Scenario F3 which involves construction of a levee across the floodplain upstream of the culverts and increasing the span of the two bridges by 20 m appears to provide the greatest compromise between cost and performance. The maximum afflux at the railway is 110 mm in the ARI 50-year event.

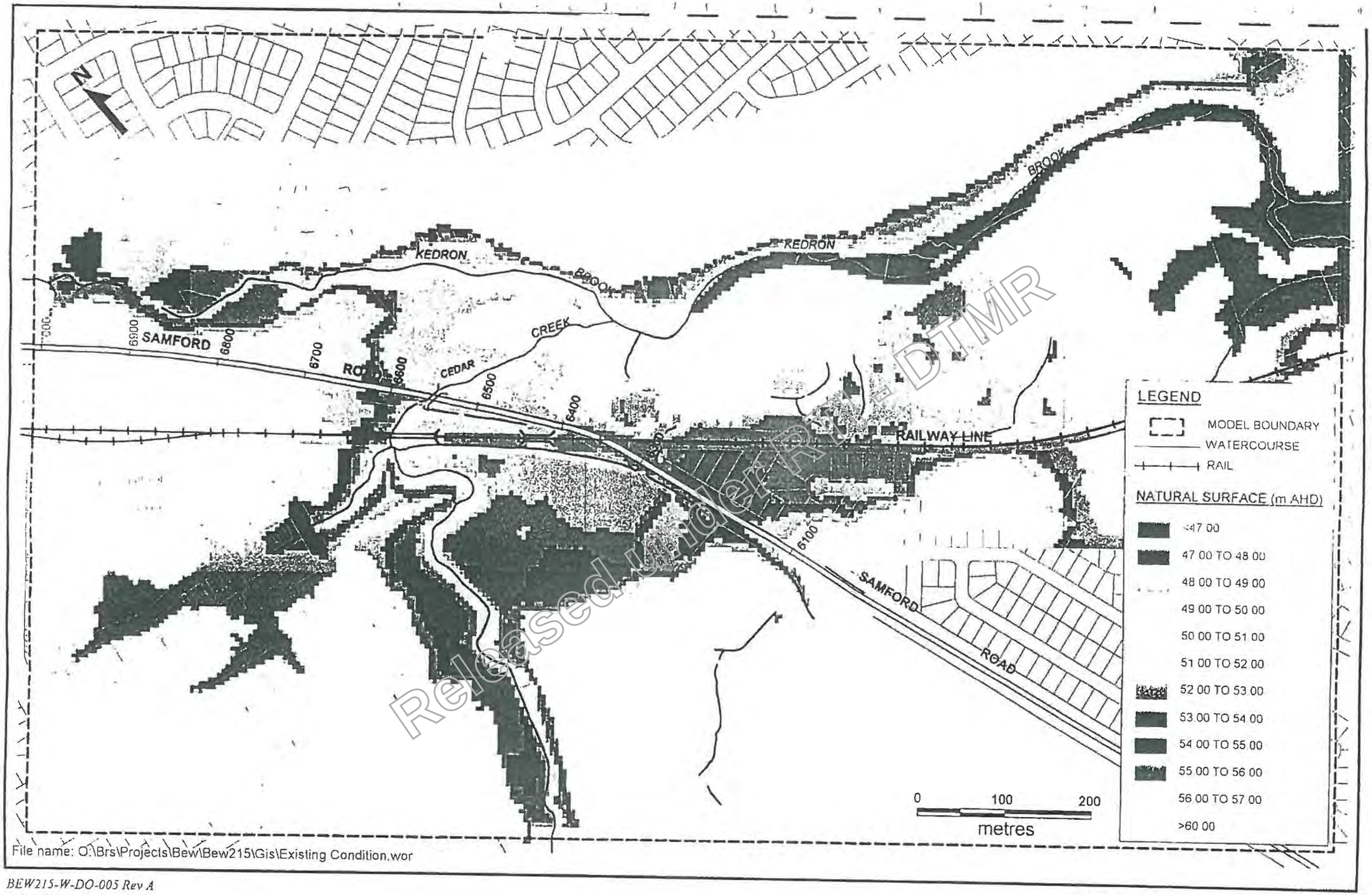

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Figure 4.1
DELFT-FLS MODEL BATHYMETRY
EXISTING CONDITIONS

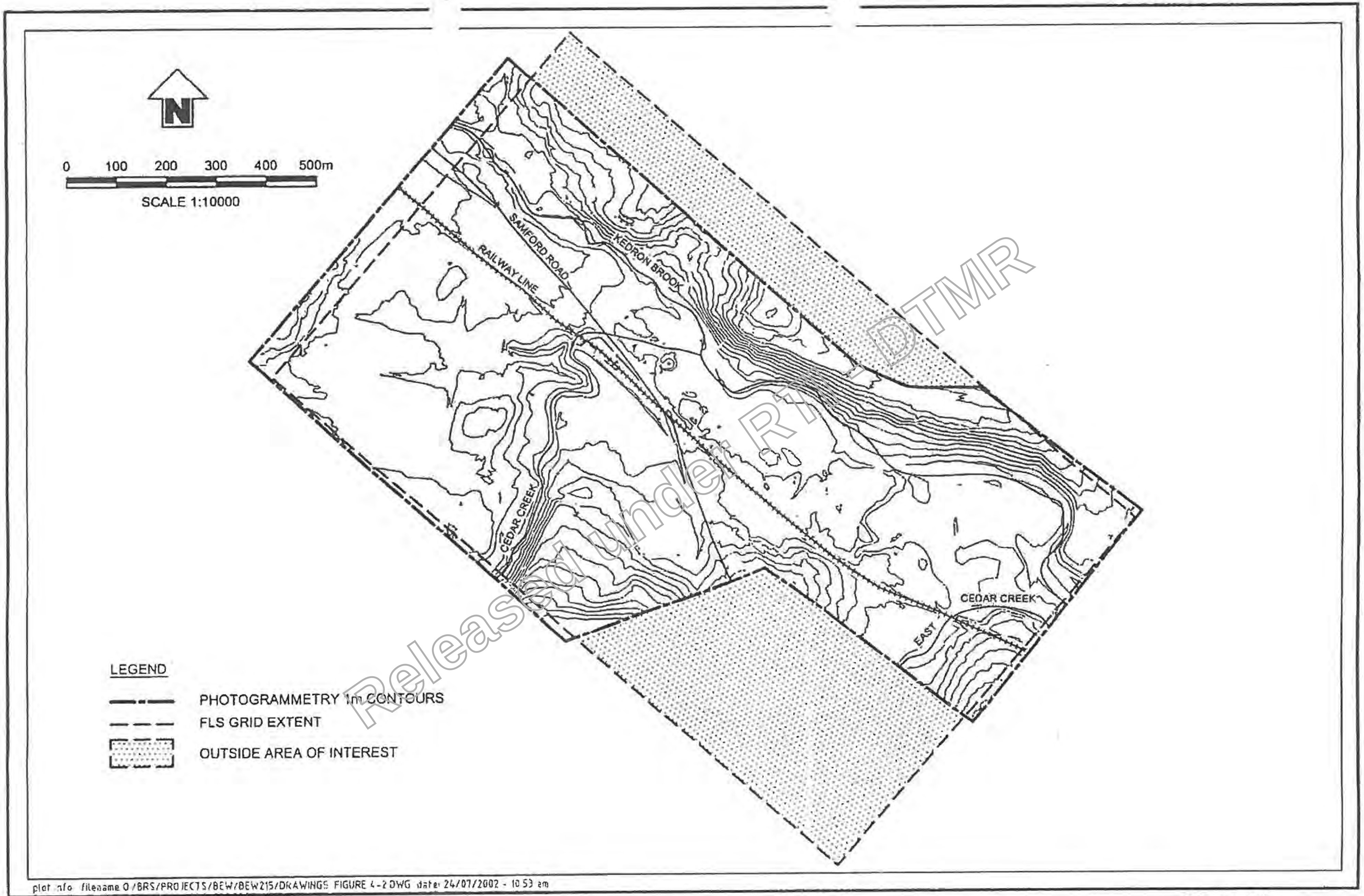

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Figure 4.2
SURVEY SOURCES

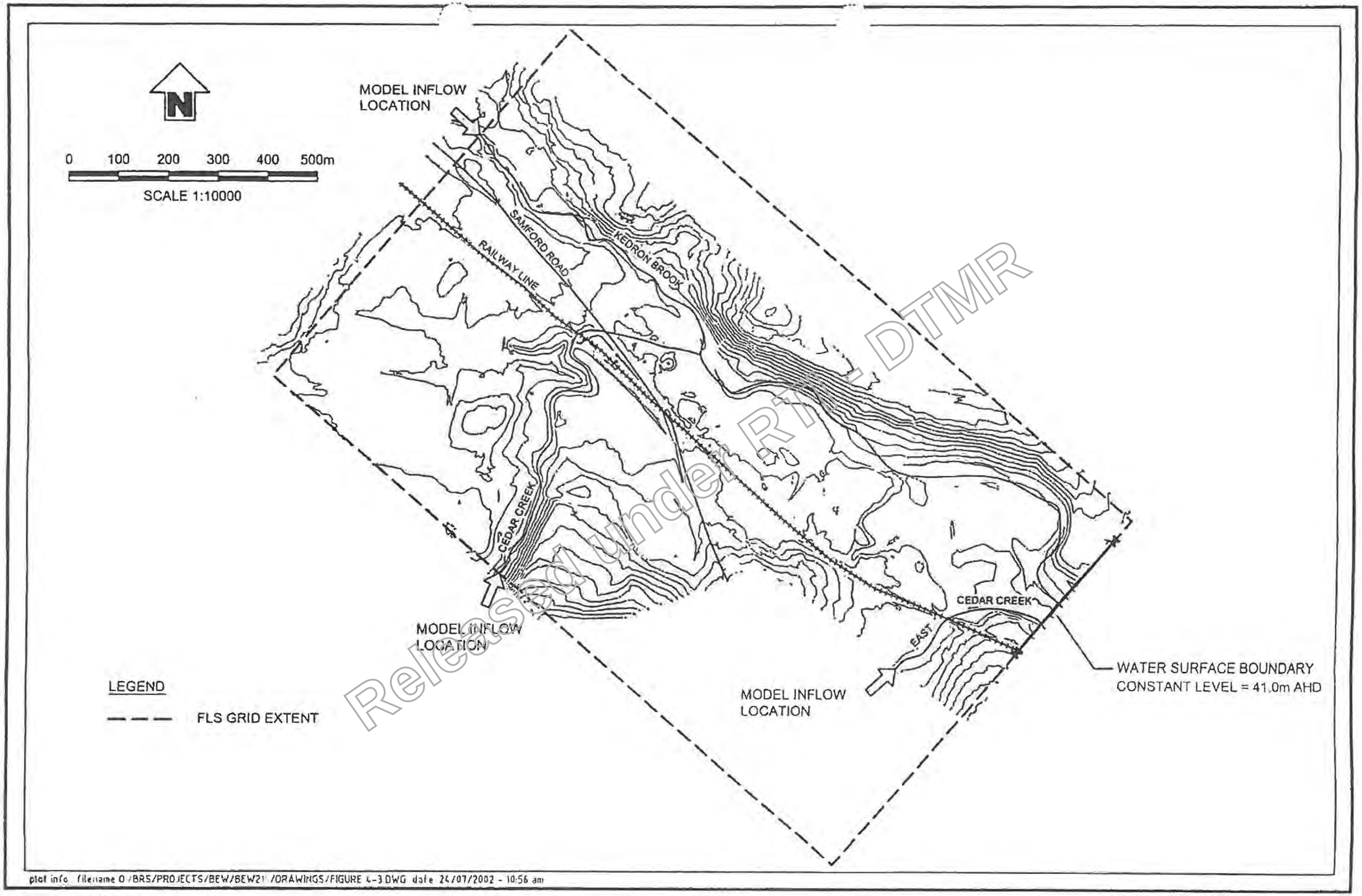

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Figure 4.3
DELFT - FLS MODEL
BOUNDARY CONDITIONS

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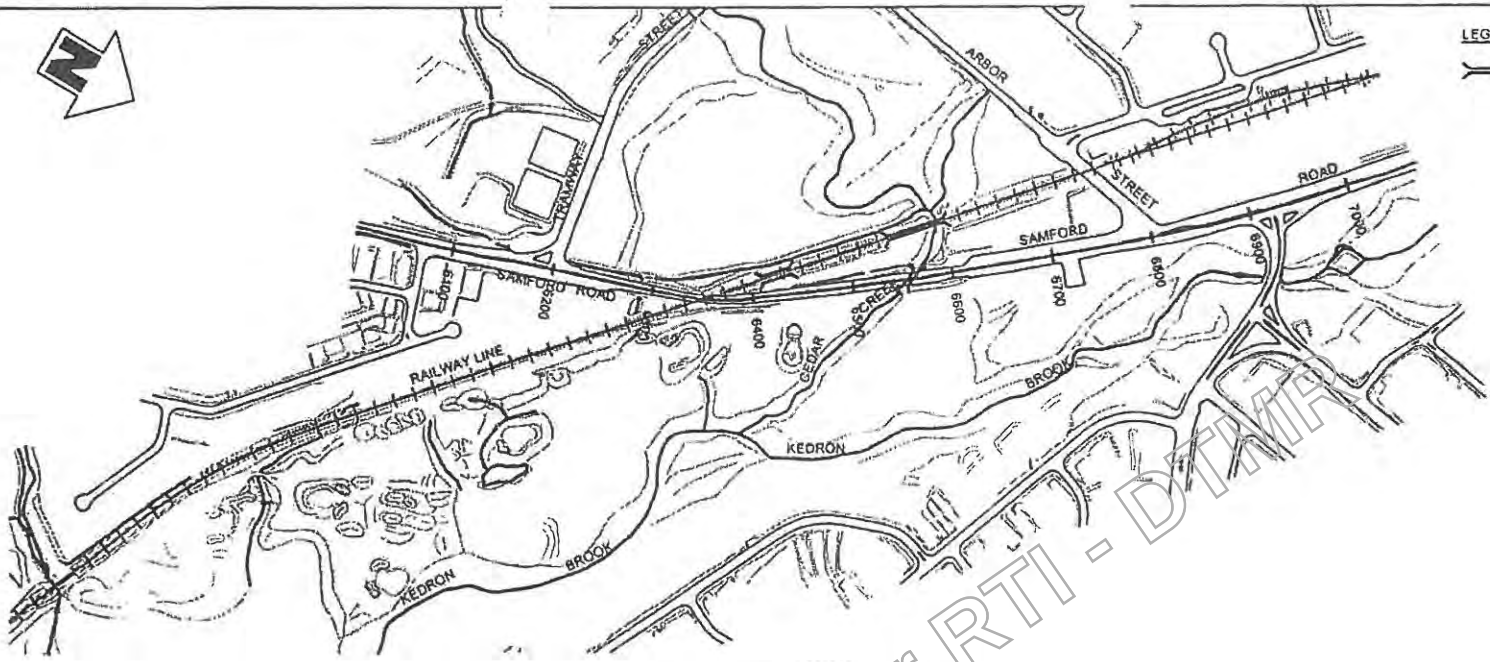
Figure 4.4
DELFT-FLS MODEL BATHYMETRY ROUGHNESS VALUES



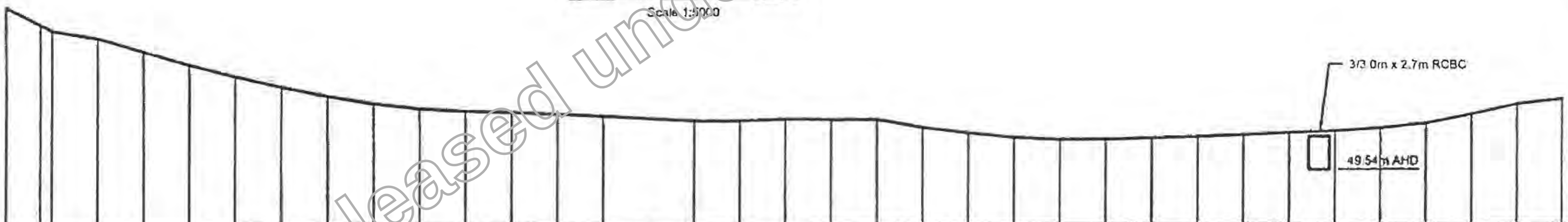
LEGEND

STRUCTURE

PLAN

0 50 100 150 200 250 m

Scale 1:5000



DATUM 45.00m AHD

CONTROL LINE CHAINAGE	5980	5995	6000	6020	6040	6060	6080	6100	6120	6140	6160	6180	6200	6220	6240	6260	6280	6300	6320
EXISTING SAMFORD ROAD LEVEL (m AHD)			61.970	61.263	60.103	58.939	57.884	56.934	56.150	55.500	55.056	54.803	54.701	54.536	54.348		53.932	53.931	54.050

CONTROL LINE CHAINAGE	6340	6360	6380	6400	6420	6440	6460	6480	6500	6520	6540	6560	6580	6600	6620	6640	6655	6660
EXISTING SAMFORD ROAD LEVEL (m AHD)	54.002	53.968	53.329	52.797	52.405	52.229	52.248	52.329	52.441	52.593	52.781	52.931	53.170	53.589	54.334	55.247		55.693

ROAD LONGITUDINAL SECTION

0 20 40 60 80 100 m

Horizontal Scale 1:2000

0 4 8 12 16 20 m

Vertical Scale 1:400

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Figure 4.5
EXISTING CONDITIONS
LONGITUDINAL PROFILE AND
GENERAL ARRANGEMENT

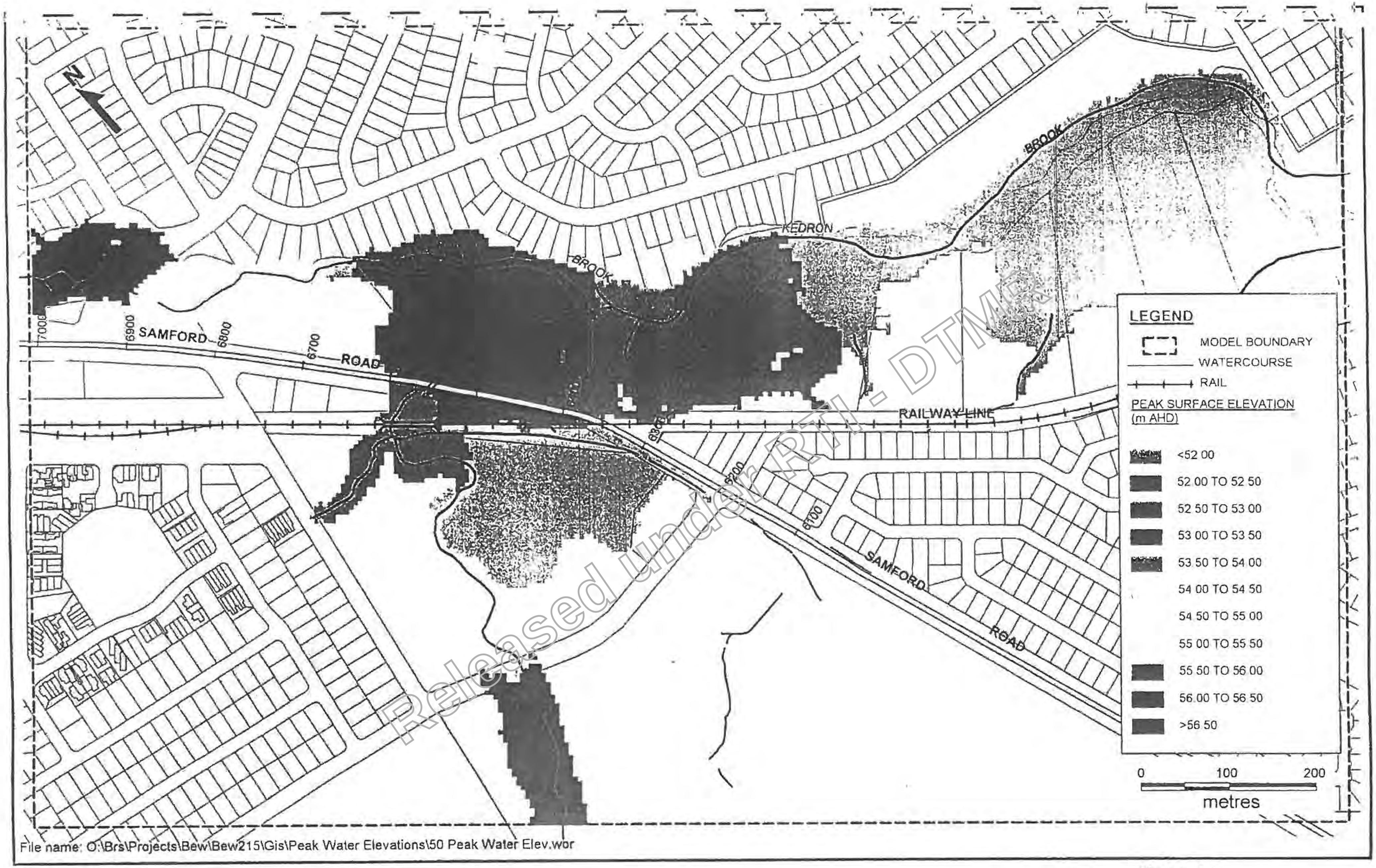

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Figure 4.6
50 YEAR ARI FLOW EVENT
PEAK WATER ELEVATIONS
EXISTING CONDITIONS

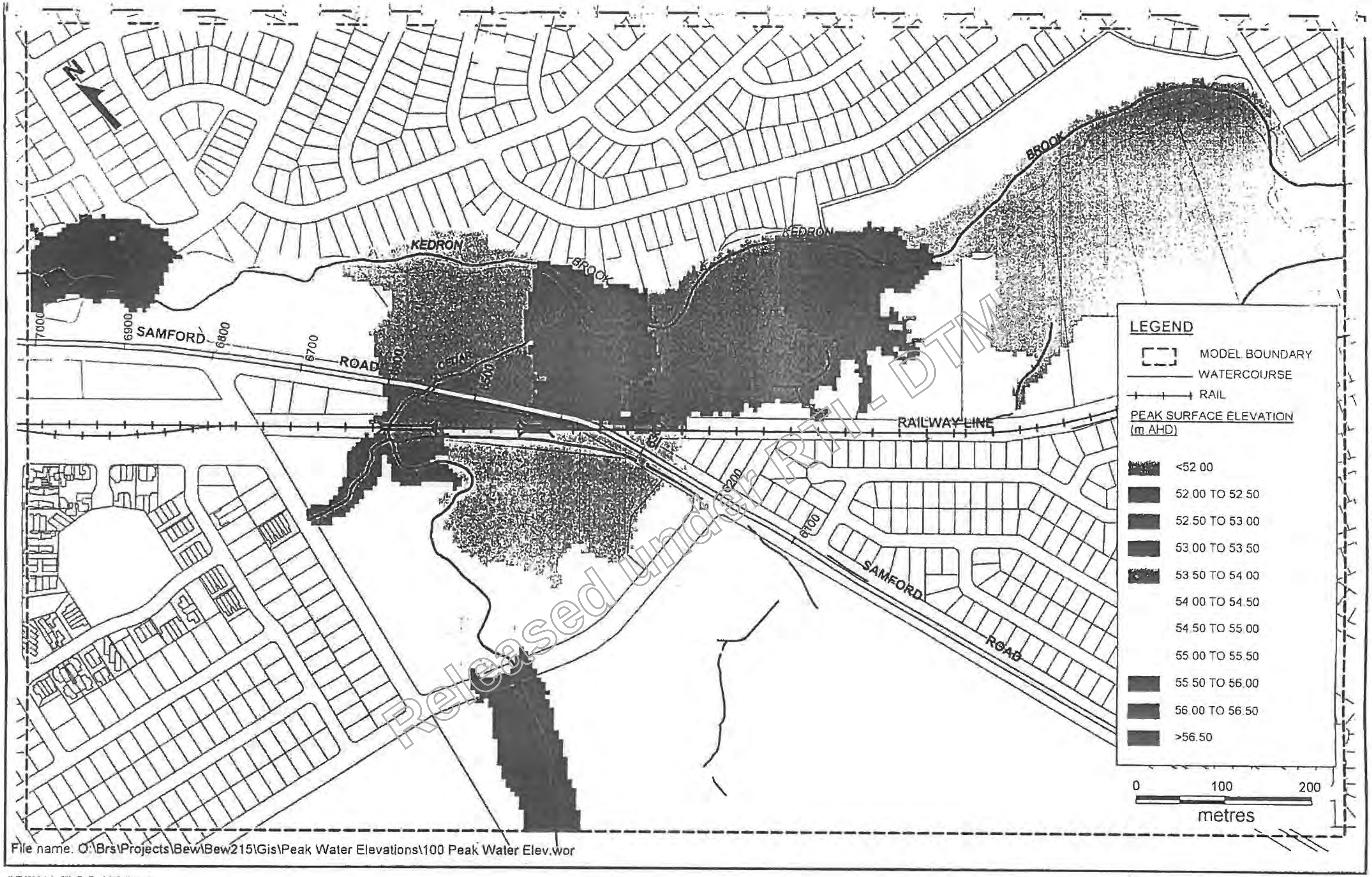

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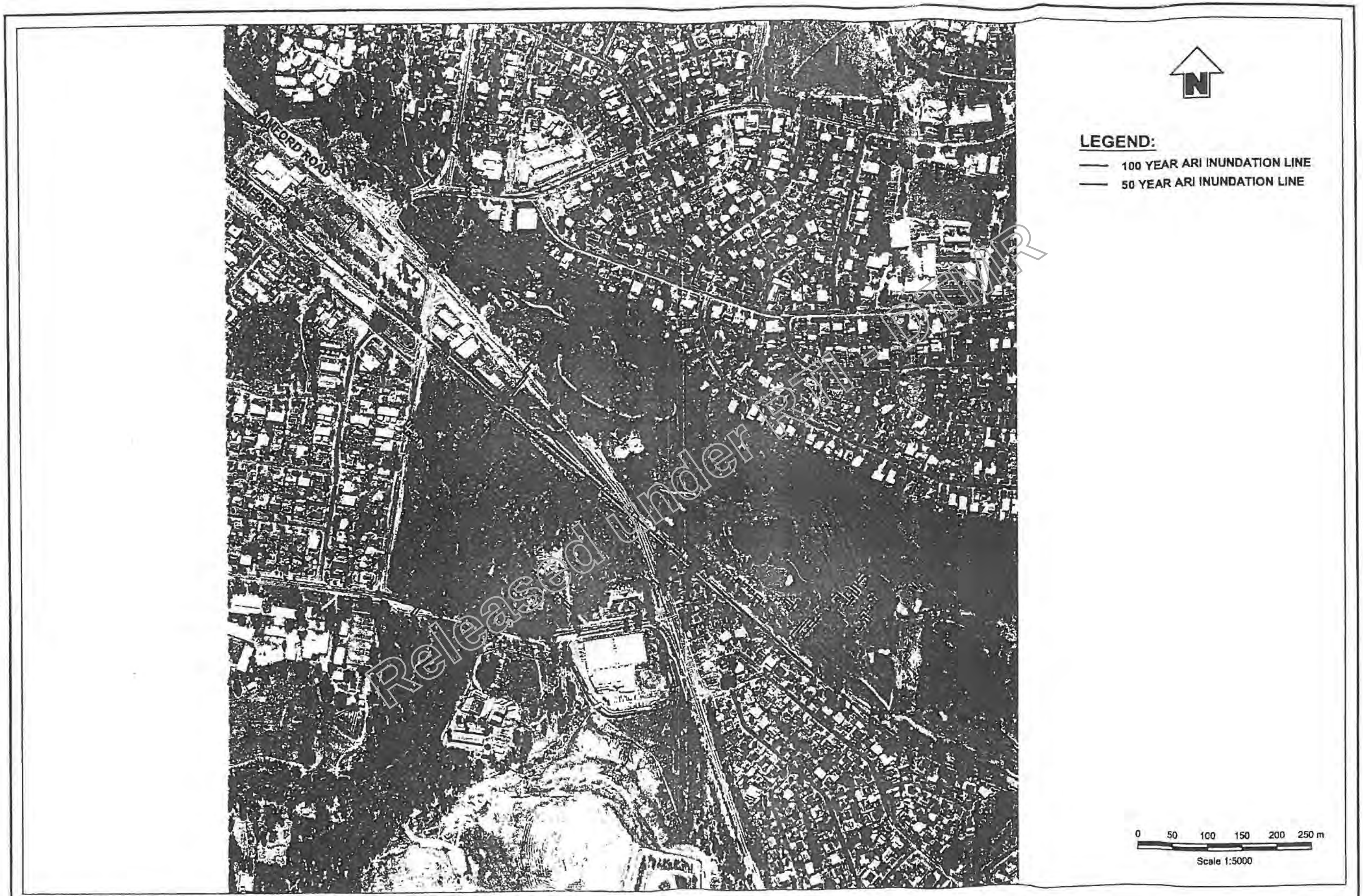
Figure 4.7
100 YEAR ARI FLOW EVENT
PEAK WATER ELEVATIONS
EXISTING CONDITIONS



plot info. filename O:\BRS\Projects\bew\Bew215\Drawings\FIGURES-W-DO-005\Figure 4.8 inundation-005.dwg date: Jul 11, 2003 - 2:42pm

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Figure 4.8
INUNDATION LINES
EXISTING CONDITIONS

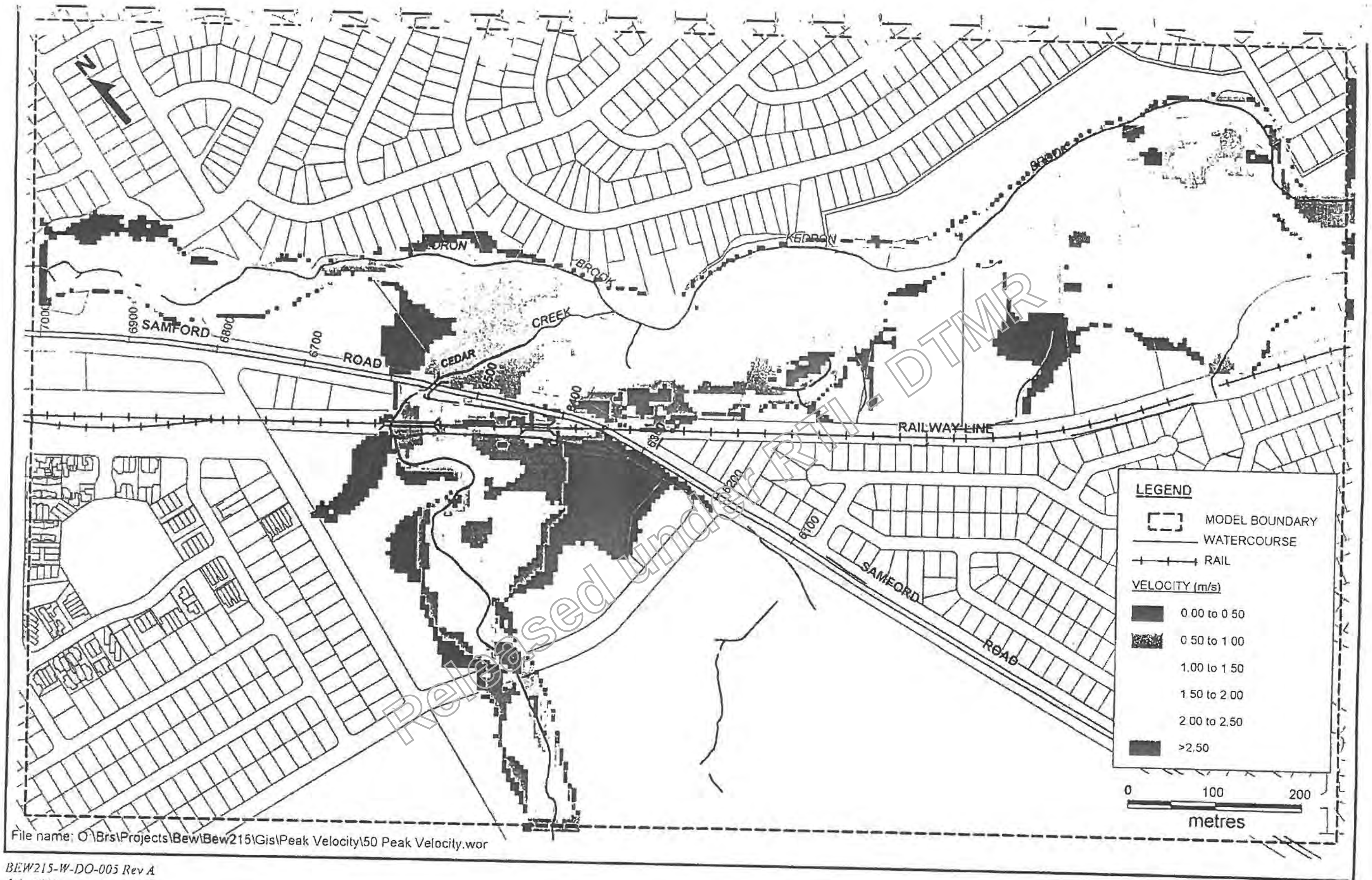

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Figure 4.9
50 YEAR ARI FLOW EVENT
PEAK VELOCITIES
EXISTING CONDITION

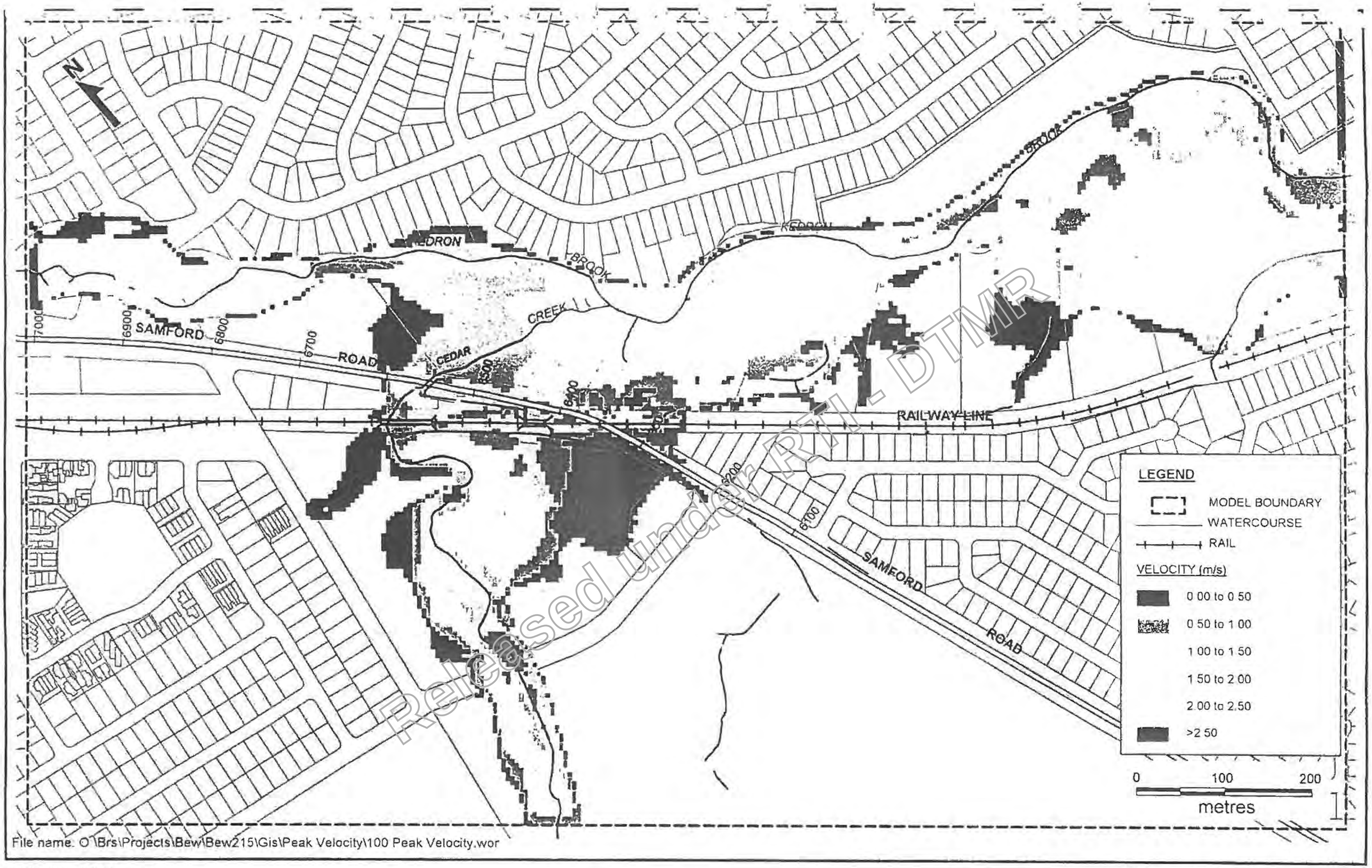

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Figure 4.10
100 YEAR ARI FLOW EVENT
PEAK VELOCITIES
EXISTING CONDITION

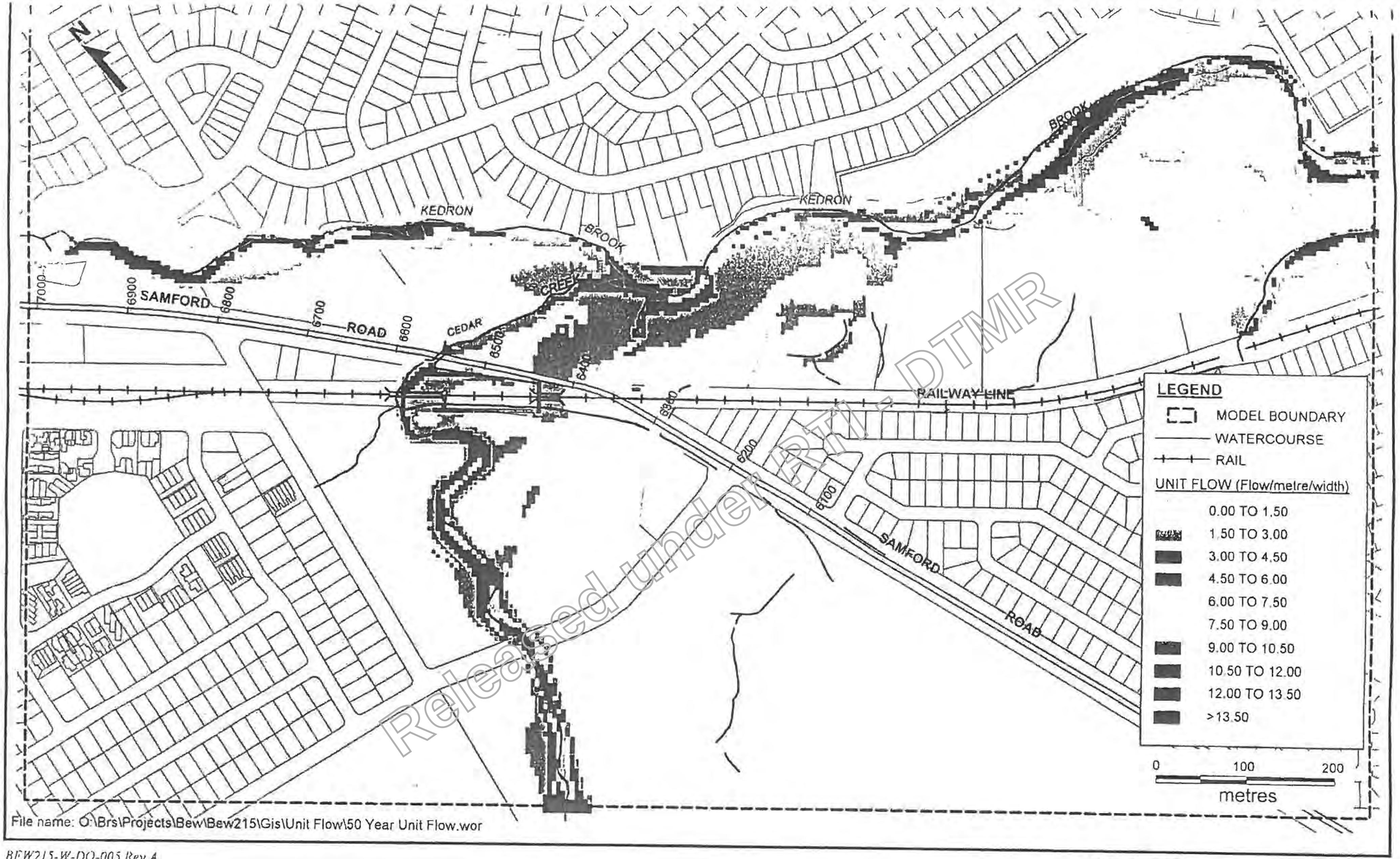

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Figure 4.11
50 YEAR ARI FLOW EVENT
UNIT FLOW (FLOW/METRE/WIDTH)
EXISTING CONDITION

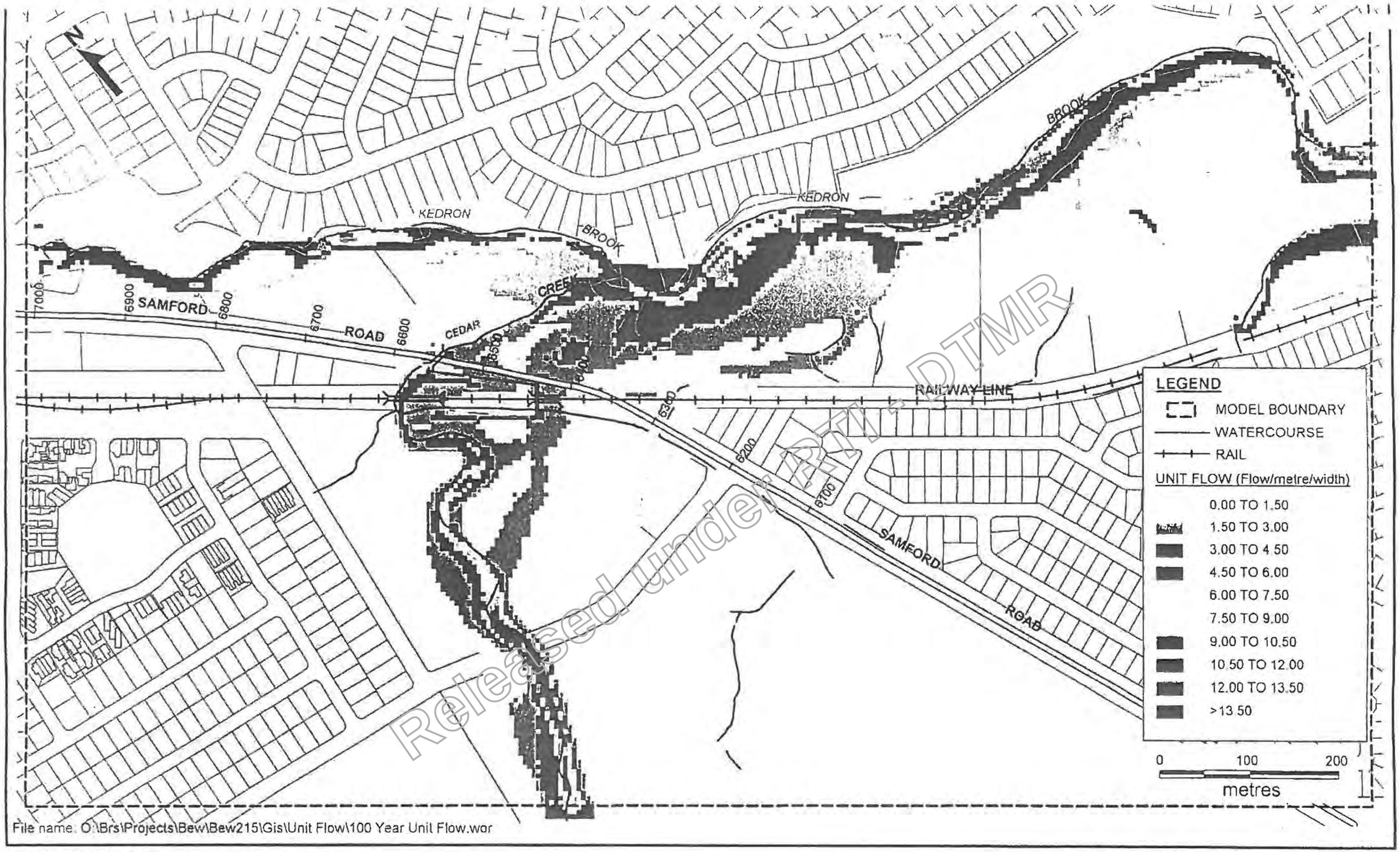

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Figure 4.12
100 YEAR ARI FLOW EVENT
UNIT FLOW (FLOW/METRE/WIDTH)
EXISTING CONDITION

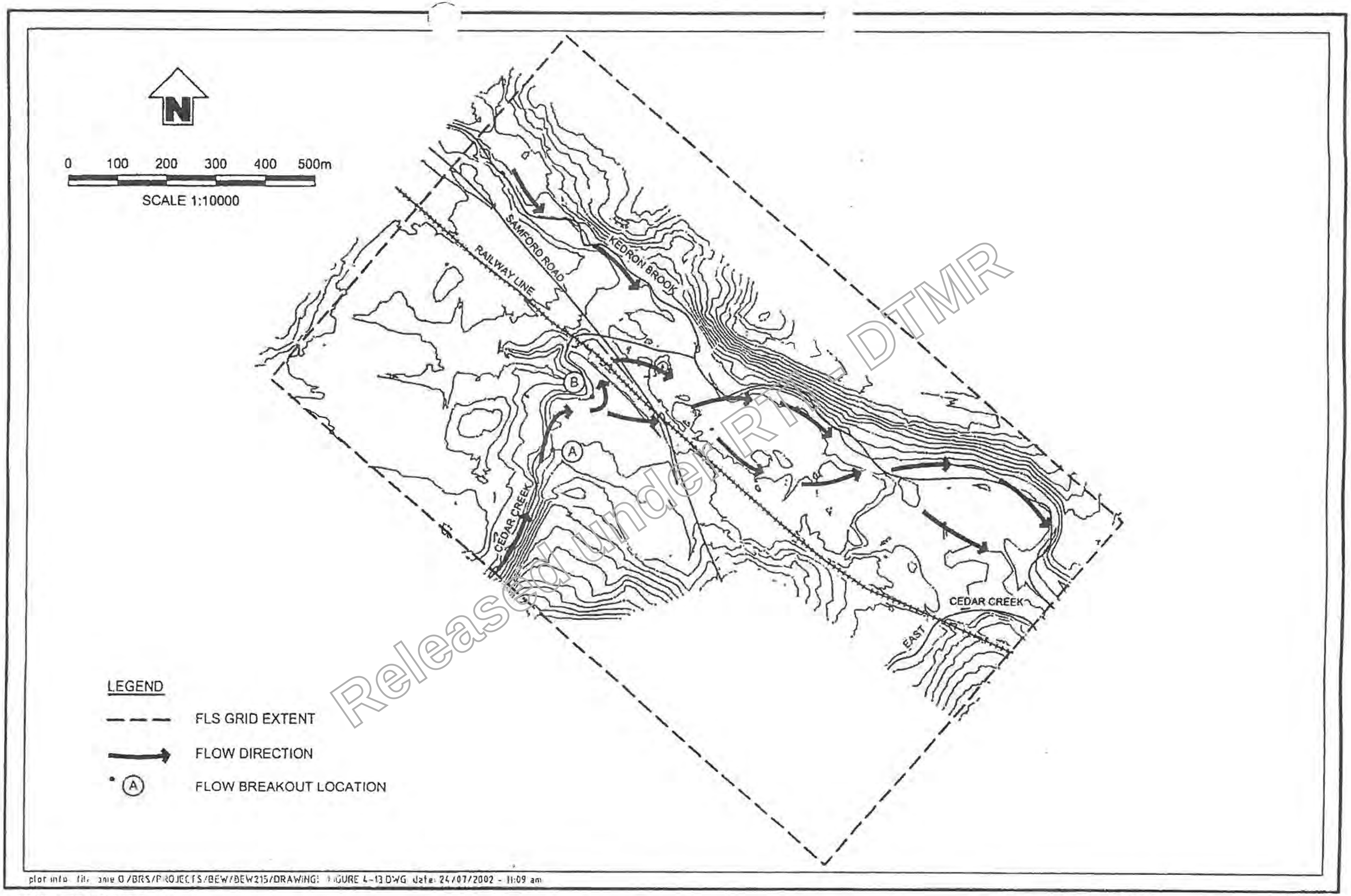

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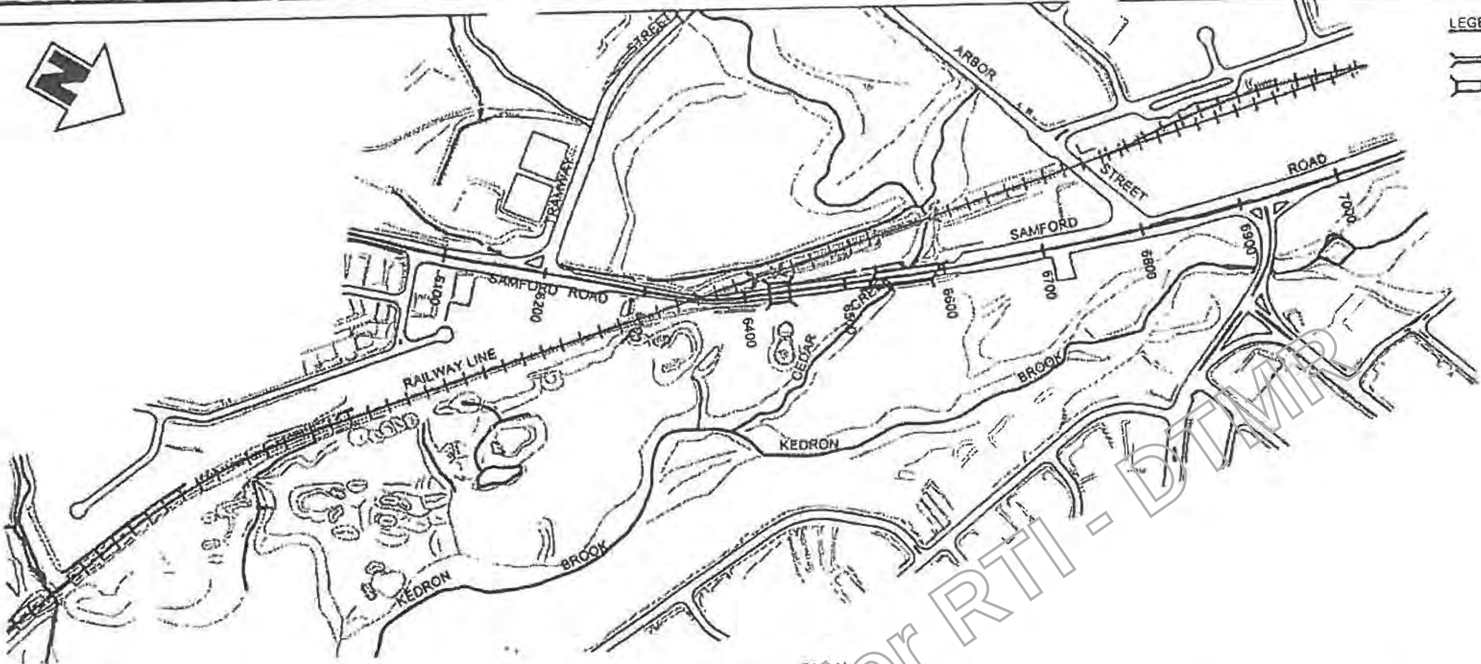
Figure 4.13
SCHEMATIC OF FLOW PATTERNS
EXISTING CONDITIONS

LEGEND

EXISTING STRUCTURE

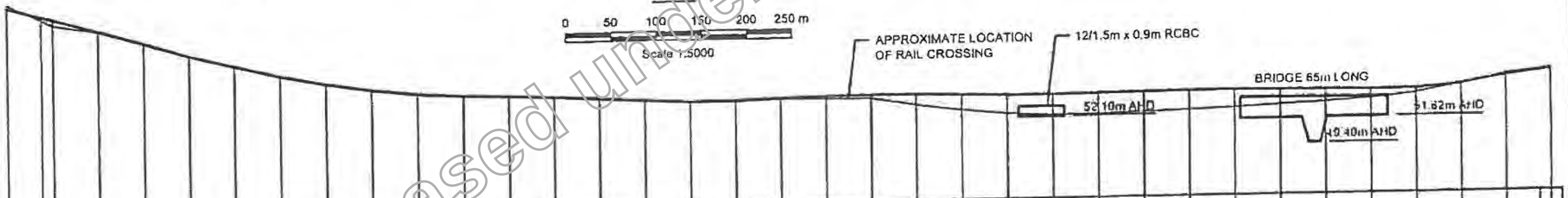
PROPOSED STRUCTURE



PLAN

0 50 100 150 200 250 m

Scale 1:5000



DATUM 45.00m AHD

CONTROL LINE CHAINAGE	5980	5995	6000	6020	6040	6060	6080	6100	6120	6140	6160	6180	6200	6220	6240	6260
EXISTING SAMFORD ROAD LEVEL (m AHD)			61.97	61.26	60.10	58.94	57.88	56.94	56.15	[illegible]	55.08	54.80	54.70	54.54	54.35	54.10
CONTROL LINE M001 FINISHED LEVEL (m AHD)			62.51	61.36	60.21	58.99	57.95	56.58	56.19	55.58	55.03	54.91	54.78	54.63	54.42	54.21

CONTROL LINE CHAINAGE	6280	6300	6320	6340	6360	6380	6400	6420	6440	6460	6480	6500
EXISTING SAMFORD ROAD LEVEL (m AHD)	53.93	53.93	54.05	54.00	53.97	53.33	52.80	52.41	52.23	52.25	52.33	52.44
CONTROL LINE M001 FINISHED LEVEL (m AHD)	54.10	54.11	54.17	54.25	54.25	54.23	54.14	54.04	54.00	54.02	54.07	54.12

CONTROL LINE CHAINAGE	6520	6540	6560	6580	6600	6620	6640	6655	6660
EXISTING SAMFORD ROAD LEVEL (m AHD)	52.59	52.78	52.93	53.17	53.59	54.33	55.25		53.69
CONTROL LINE M001 FINISHED LEVEL (m AHD)	54.13	54.08	54.02	53.98	54.01	54.37	55.07		55.48

ROAD LONGITUDINAL SECTION

0 20 40 60 80 100 m

Horizontal Scale 1:2000

0 4 8 12 16 20 m

Vertical Scale 1:400

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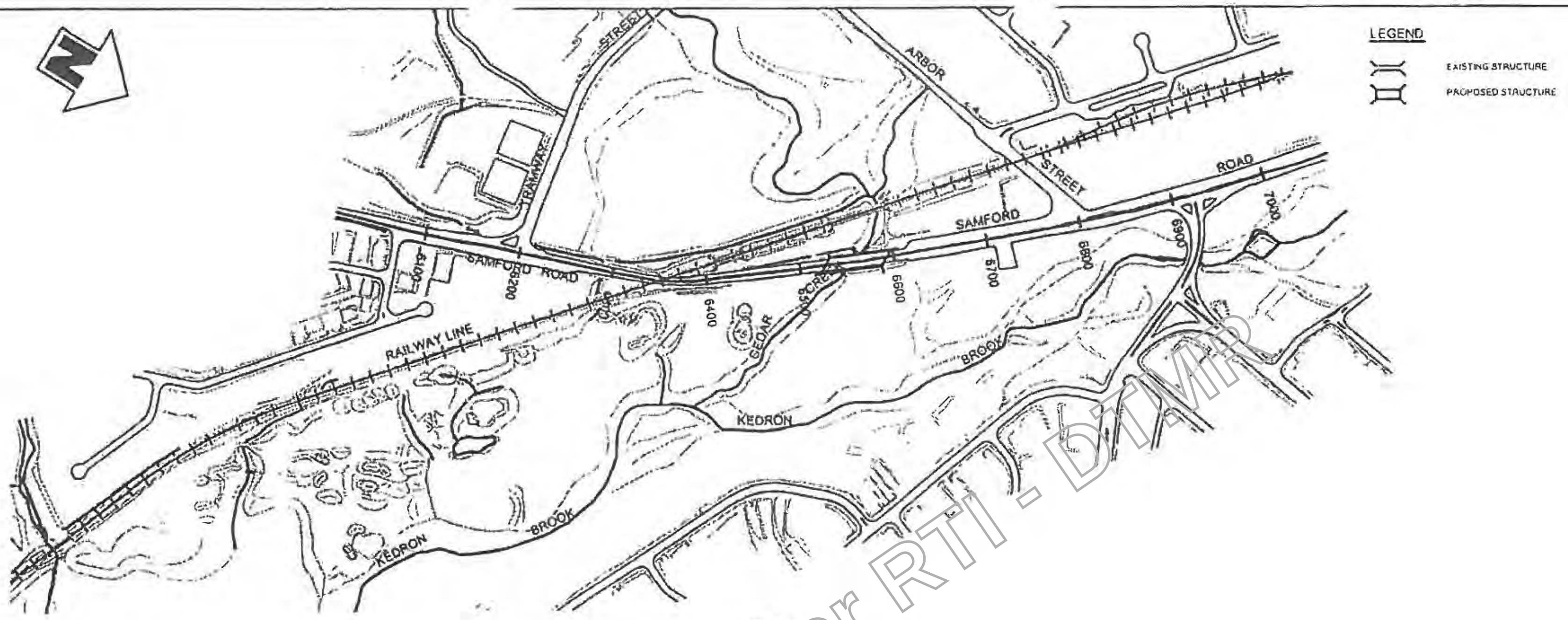
Figure 4.14

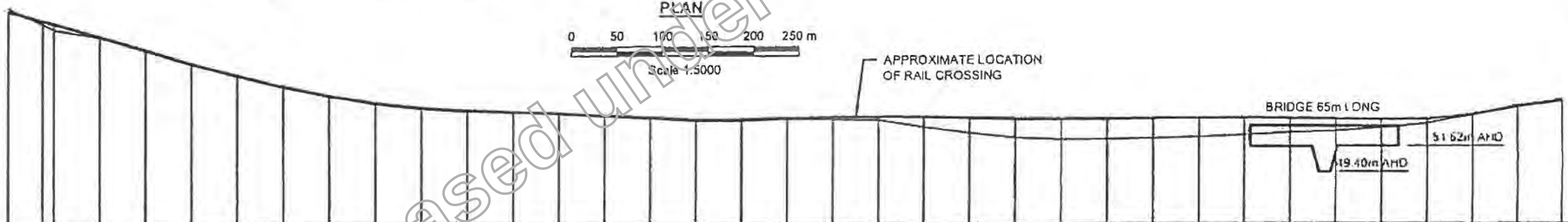
OPTION 1

LONGITUDINAL PROFILE AND GENERAL ARRANGEMENT



PLAN

0 50 100 150 200 250 m
Scale 1:5000



DATUM 45.00m AHD

CONTROL LINE CHAINAGE	EXISTING SAMFORD ROAD LEVEL (m AHD)	CONTROL LINE M001 FINISHED LEVEL (m AHD)
5980		
5995		
6000	61.97	62.51
6020	61.26	61.36
6040	60.10	60.21
6060	58.94	58.99
6080	57.88	57.95
6100	56.94	56.98
6120	56.15	56.19
6140	55.50	55.58
6160	55.05	55.03
6180	54.80	54.91
6200	54.70	54.78
6220	54.54	54.63
6240	54.35	54.42
6260	54.10	54.21
6280	53.93	54.10
6300	53.93	54.11
6320	54.05	54.17
6340	54.00	54.25
6360	53.97	54.25
6380	53.33	54.23
6400	52.80	54.14
6420	52.41	54.04
6440	52.23	54.00
6460	52.25	54.02
6480	52.33	54.07
6500	52.44	54.12
6520	52.59	54.13
6540	52.78	54.08
6560	52.93	54.02
6580	53.17	53.96
6600	53.59	54.01
6620	54.33	54.37
6640	55.25	55.07
6655		
6660	55.69	55.46

ROAD LONGITUDINAL SECTION

0 20 40 60 80 100 m
Horizontal Scale 1:2000

0 4 8 12 16 20 m
Vertical Scale 1:400

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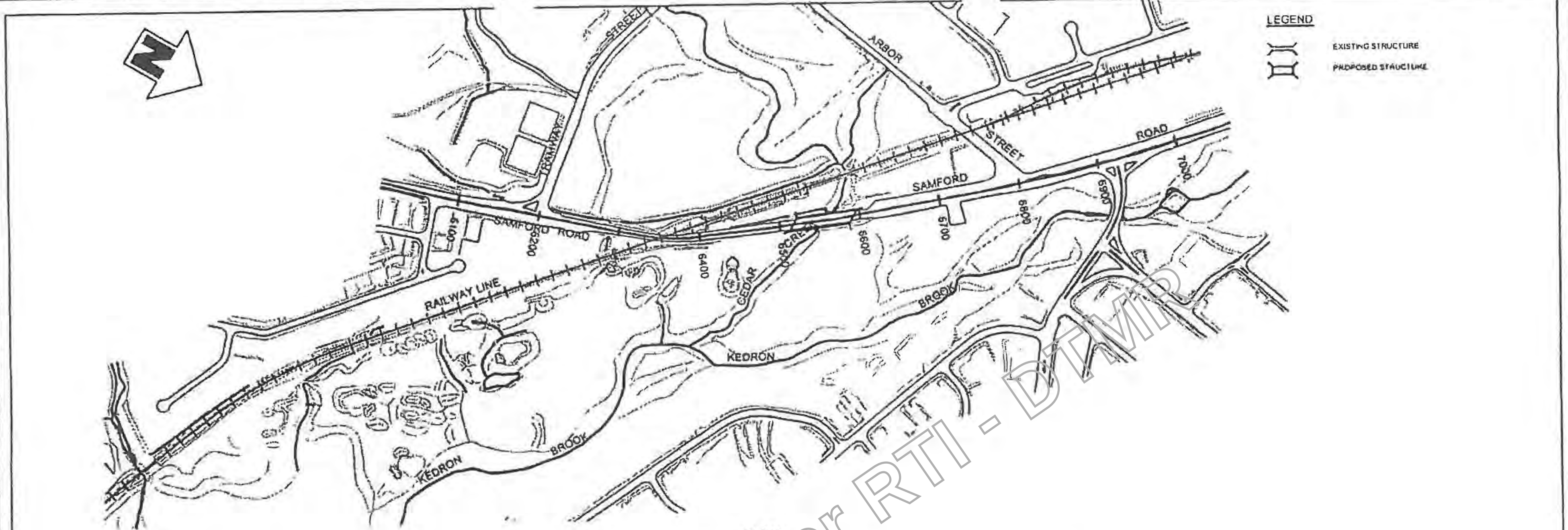
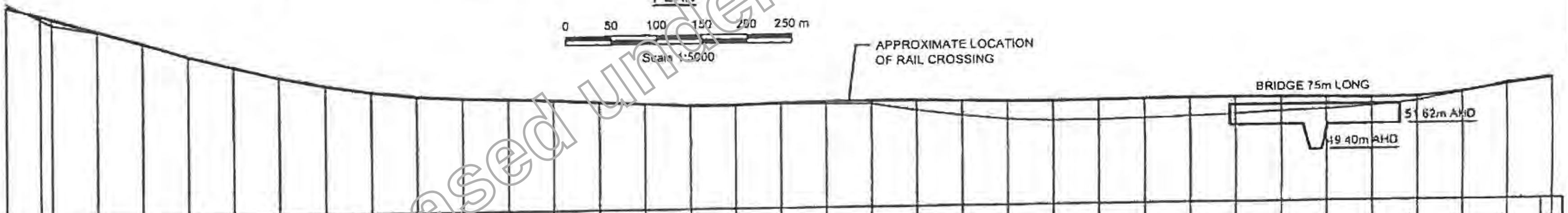
Figure 4.15
OPTION 2
LONGITUDINAL PROFILE AND GENERAL ARRANGEMENT



PLAN

0 50 100 150 200 250 m
Scale 1:5000



DATUM 45.00m AHD

CONTROL LINE CHAINAGE	EXISTING SAMFORD ROAD LEVEL (m AHD)	CONTROL LINE M001 FINISHED LEVEL (m AHD)
5980		
5995		
6000	61.97	62.51
6020	61.26	61.36
6040	60.10	60.21
6060	58.94	58.99
6080	57.88	57.95
6100	56.94	56.95
6120	56.15	56.19
6140	55.50	55.58
6160	55.06	55.03
6180	54.80	54.91
6200	54.70	54.78
6220	54.54	54.63
6240	54.35	54.42
6260	54.10	54.21
6280	53.93	54.10
6300	53.93	54.11
6320	54.05	54.17
6340	54.00	54.25
6360	53.97	54.25
6380	53.33	54.23
6400	52.80	54.14
6420	52.41	54.04
6440	52.23	54.00
6460	52.25	54.02
6480	52.33	54.07
6500	52.44	54.12
6520	52.59	54.13
6540	52.78	54.08
6560	52.93	54.02
6580	53.17	53.96
6600	53.59	54.01
6620	54.33	54.37
6640	55.25	55.07
6655		
6660	55.69	55.31

ROAD LONGITUDINAL SECTION

0 20 40 60 80 100 m
Horizontal Scale 1:2000

0 4 8 12 16 20 m
Vertical Scale 1:400

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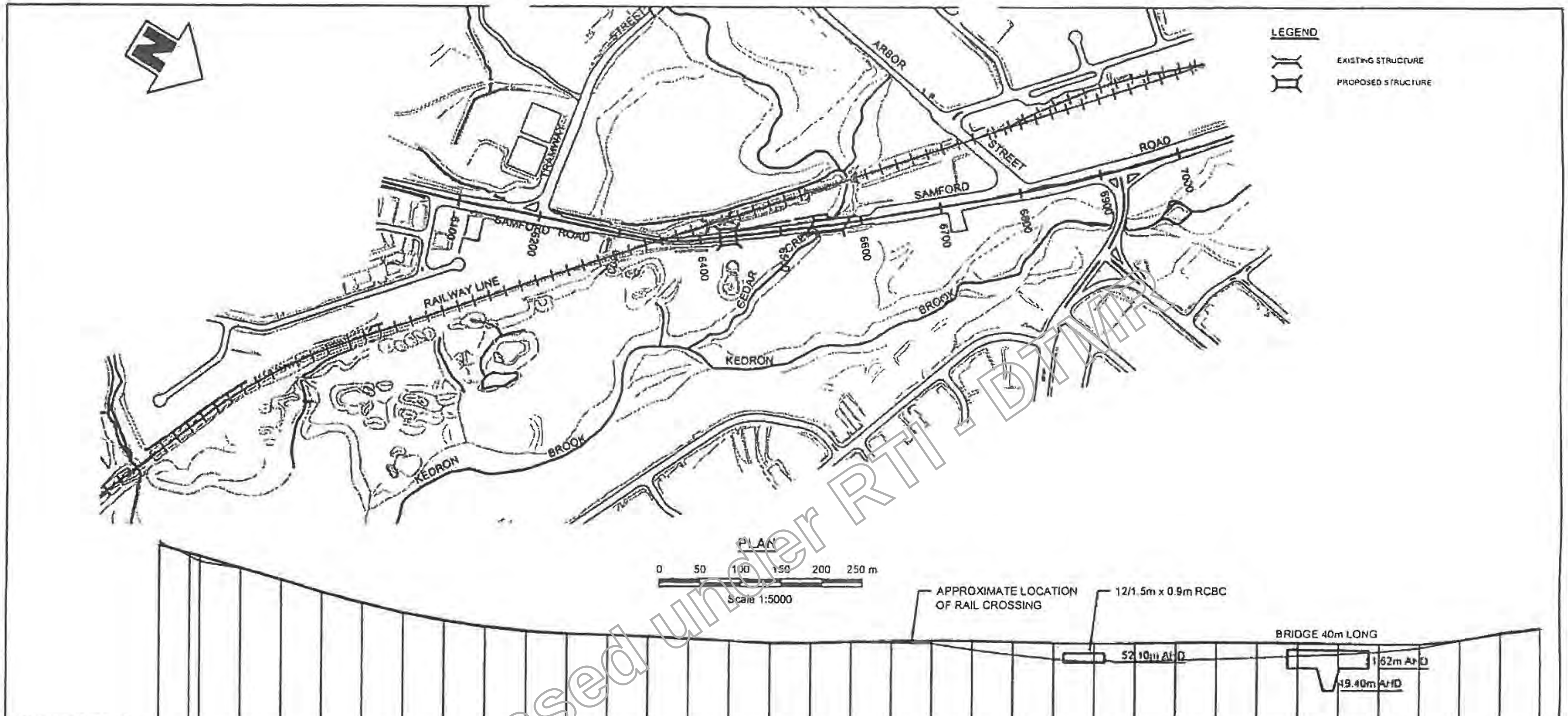
Figure 4.16
OPTION 3
LONGITUDINAL PROFILE AND
GENERAL ARRANGEMENT



DATUM 45.00m AHD

EXISTING SAMFORD ROAD LEVEL (m AHD)			61.97	61.26	60.10	58.94	57.88	56.94	56.15	55.50	55.06	54.80
CONTROL LINE M001 FINISHED LEVEL (m AHD)			62.51	61.36	60.21	58.99	57.95	56.98	56.19	55.58	55.03	54.91
CONTROL LINE CHAINAGE	5980	5995	6000	6020	6040	6060	6080	6100	6120	6140	6160	6180

EXISTING SAMFORD ROAD LEVEL (m AHD)	54.70	54.54	54.35	54.10	53.93	53.93	54.05	54.00	53.97	53.33	52.80	52.41
CONTROL LINE M001 FINISHED LEVEL (m AHD)	54.76	54.63	54.42	54.21	54.10	54.11	54.17	54.25	54.25	54.23	54.14	54.04
CONTROL LINE CHAINAGE	6200	6220	6240	6260	6280	6300	6320	6340	6360	6380	6400	6420

EXISTING SAMFORD ROAD LEVEL (m AHD)	52.23	52.25	52.33	52.44	52.59	52.78	52.93	53.17	53.59	54.33	55.25		55.66
CONTROL LINE M001 FINISHED LEVEL (m AHD)	54.00	54.02	54.07	54.12	54.13	54.08	54.02	53.96	54.01	54.37	55.07		55.46
CONTROL LINE CHAINAGE	6440	6460	6480	6500	6520	6540	6560	6580	6600	6620	6640	6655	6660

ROAD LONGITUDINAL SECTION

0 20 40 60 80 100 m
Horizontal Scale 1:2000

0 4 8 12 16 20 m
Vertical Scale 1:400

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Figure 4.17
OPTION 4
LONGITUDINAL PROFILE AND GENERAL ARRANGEMENT

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Figure 4.18
50 YEAR ARI EVENT
WATER SURFACE ELEVATIONS
OPTION 1

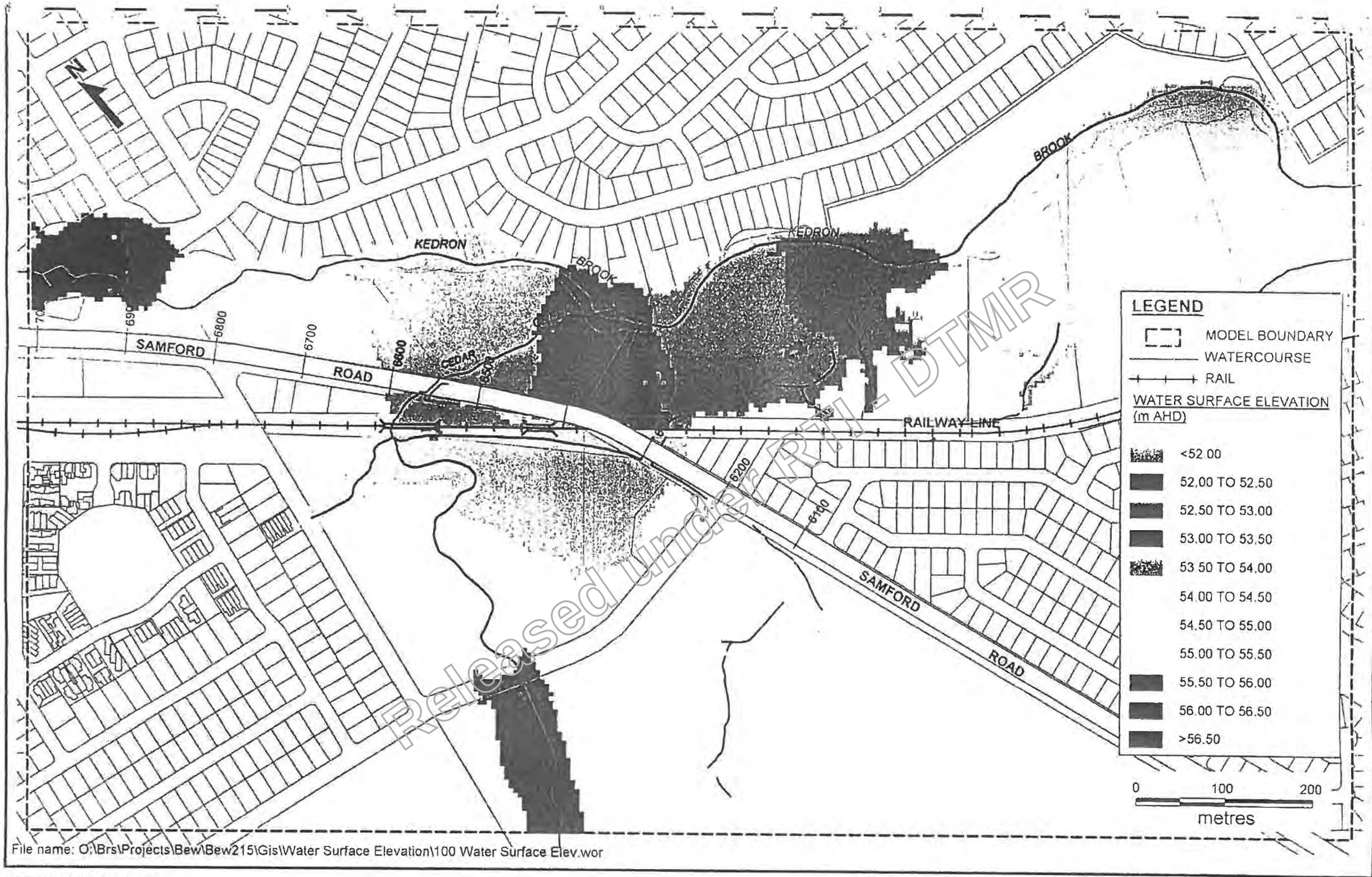

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Figure 4.19
100 YEAR ARI EVENT
WATER SURFACE ELEVATIONS
OPTION 1

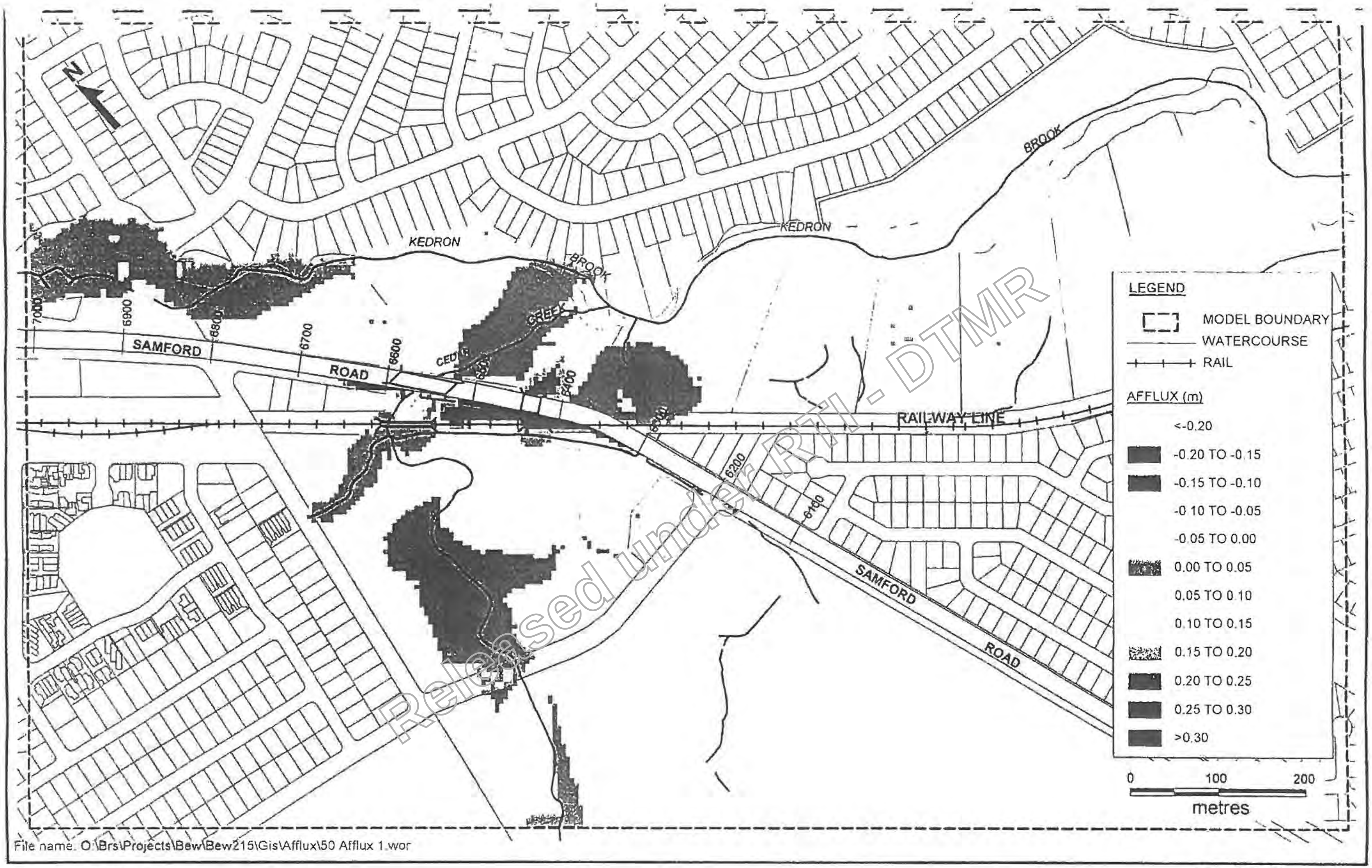

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Figure 4.20
50 YEAR ARI EVENT AFFLUX OPTION 1

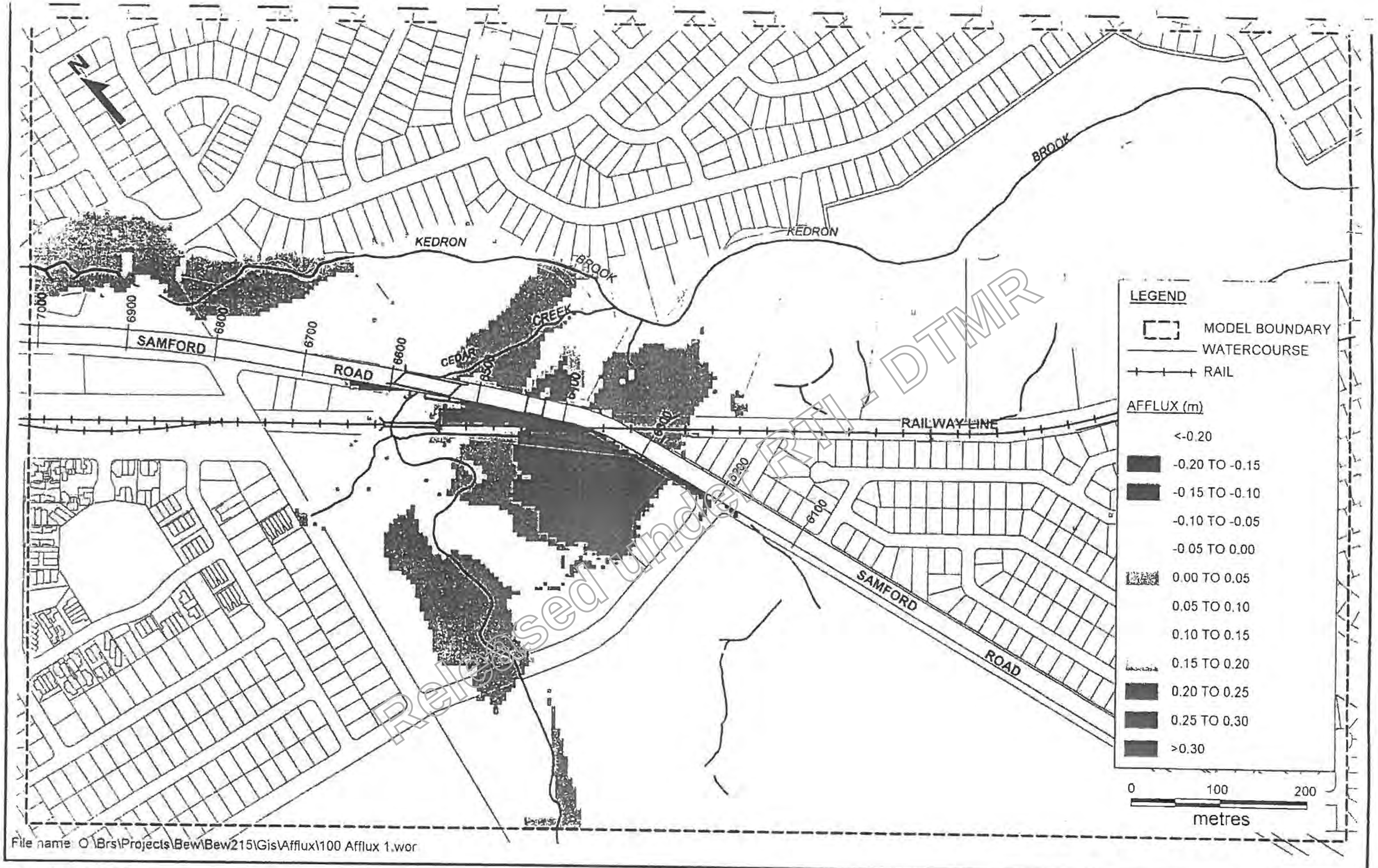

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Figure 4.21
100 YEAR ARI EVENT
AFFLUX
OPTION 1

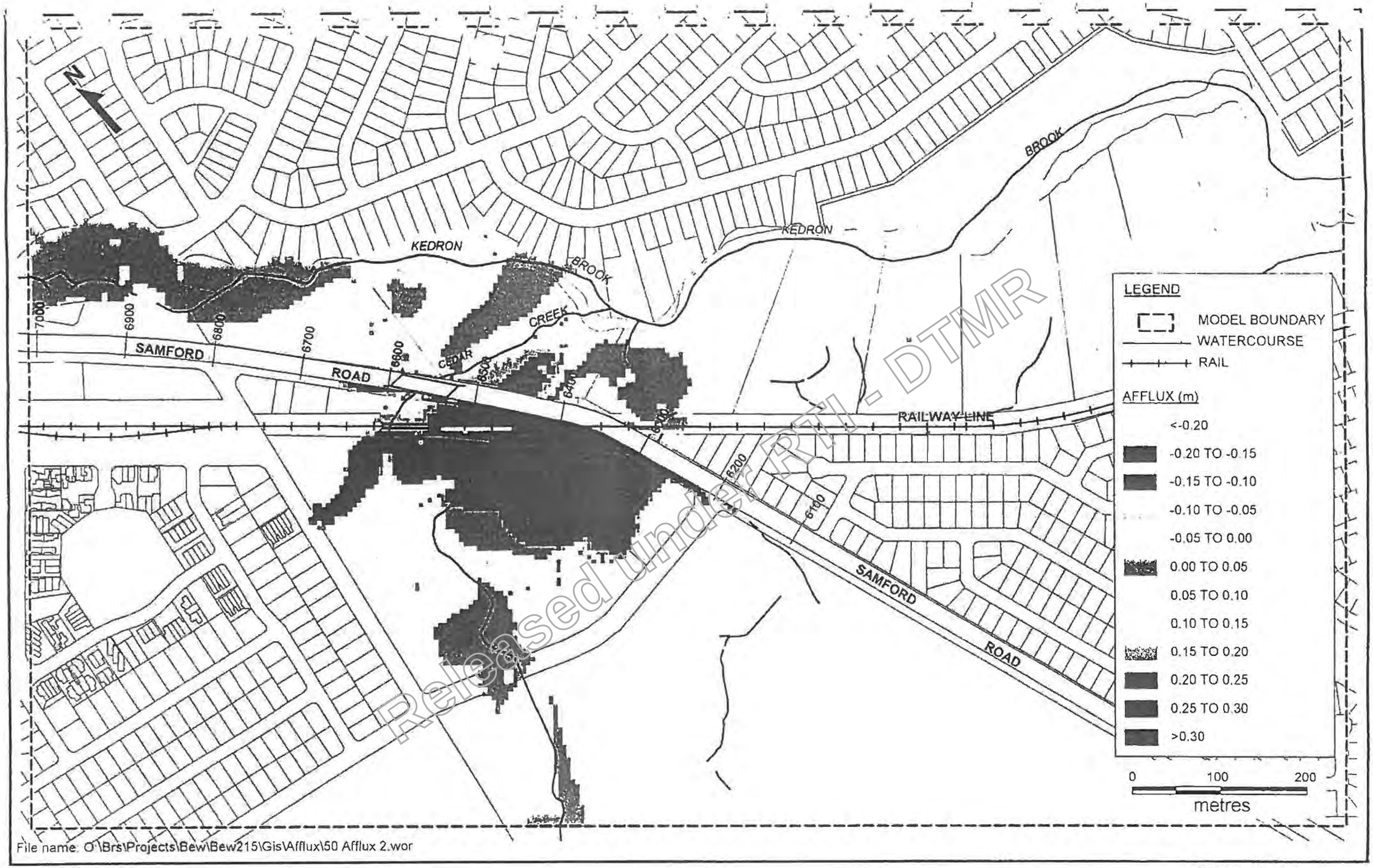

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Figure 4.22
50 YEAR ARI EVENT
AFFLUX
OPTION 2

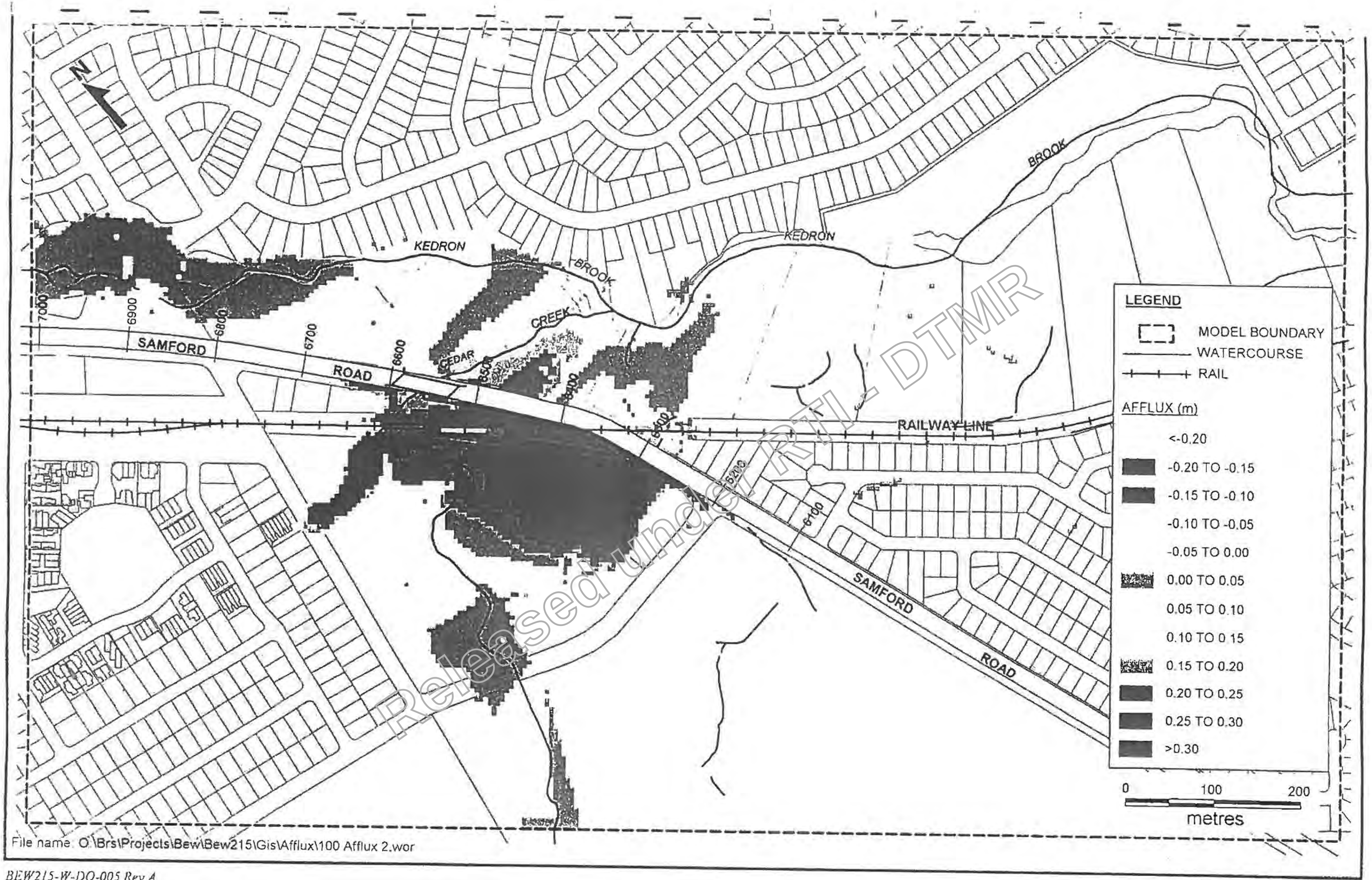

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Figure 4.23
100 YEAR ARI EVENT
AFFLUX
OPTION 2

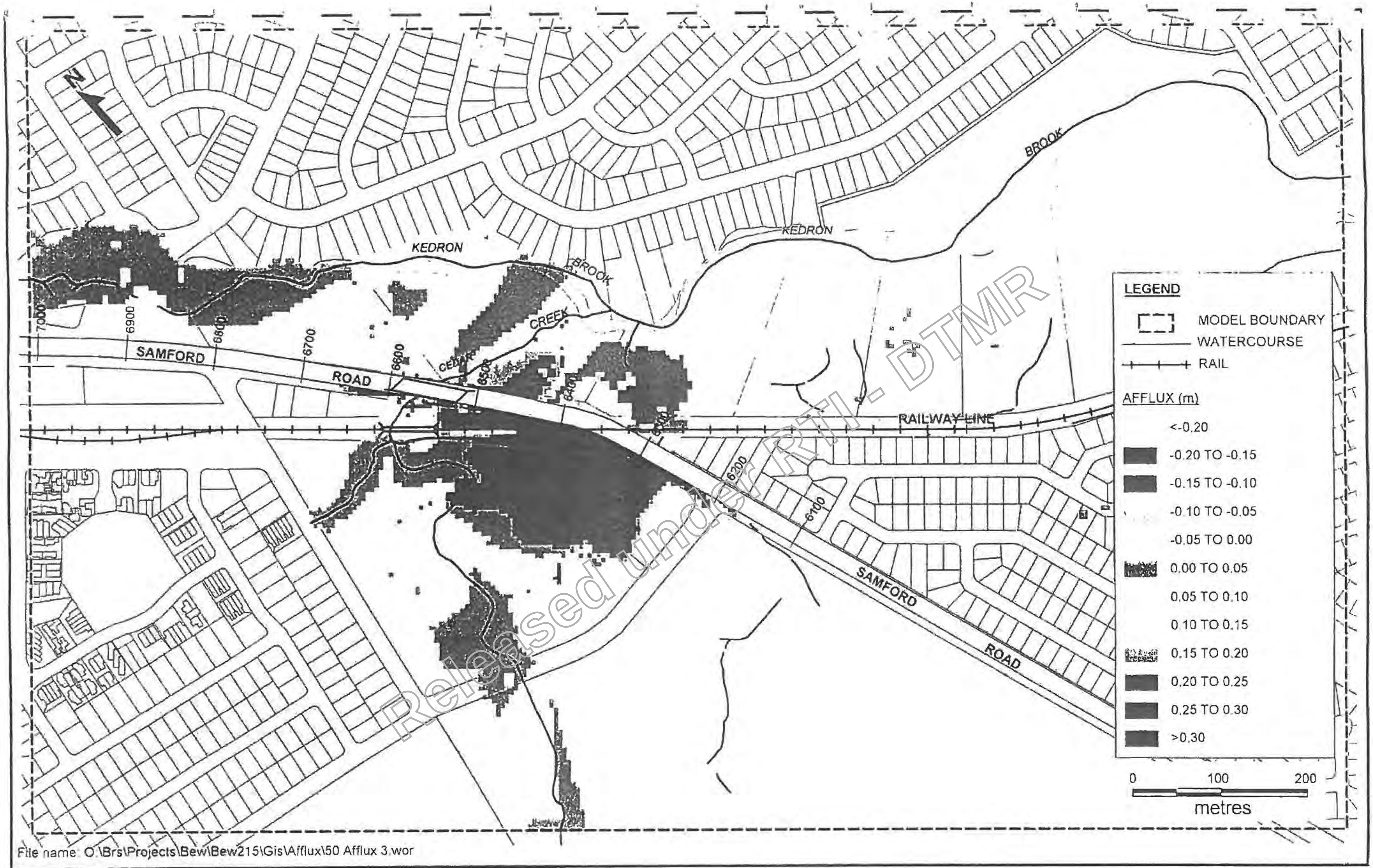

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Figure 4.24
50 YEAR ARI EVENT
AFFLUX
OPTION 3

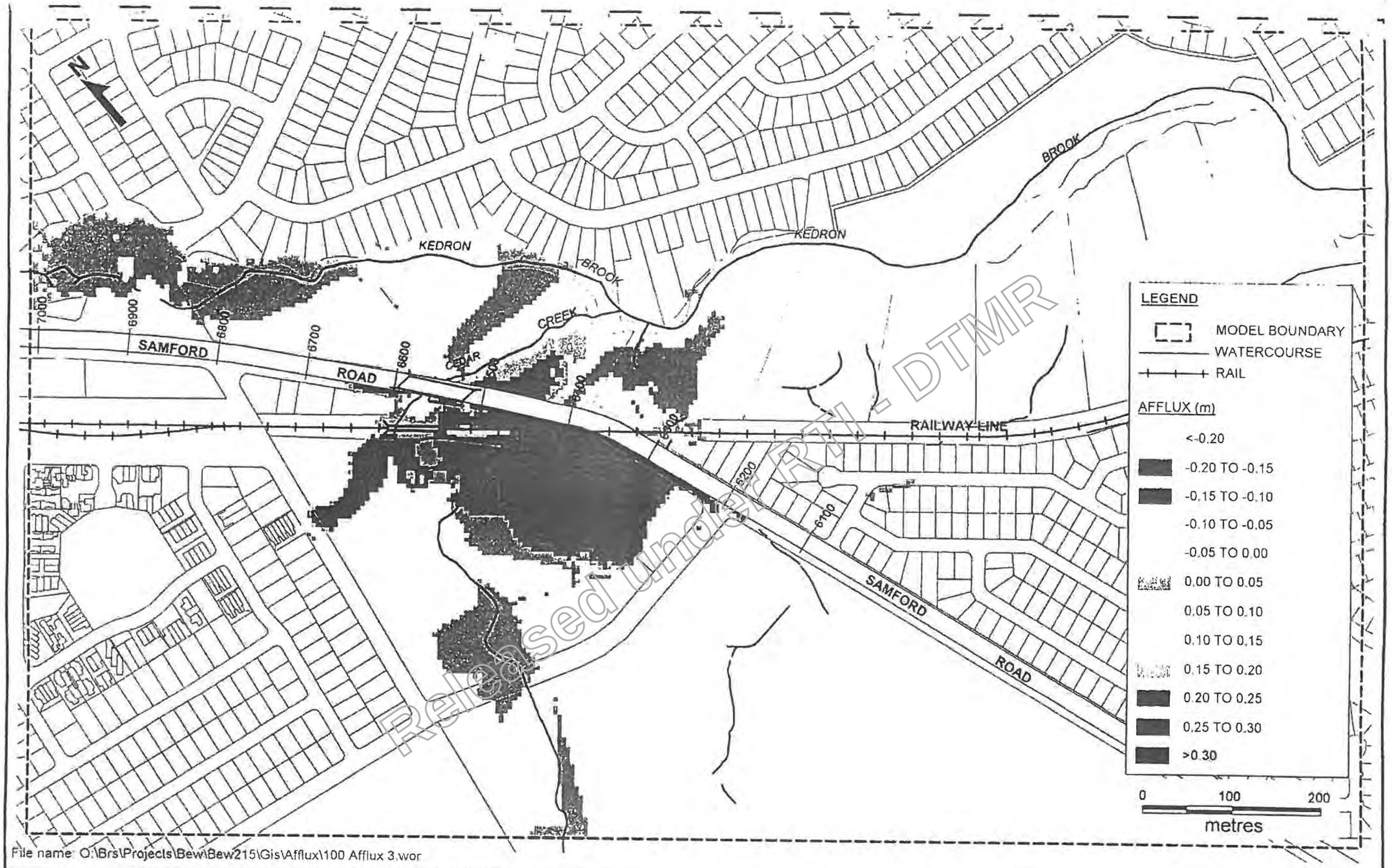

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Figure 4.25
100 YEAR ARI EVENT
AFFLUX
OPTION 3

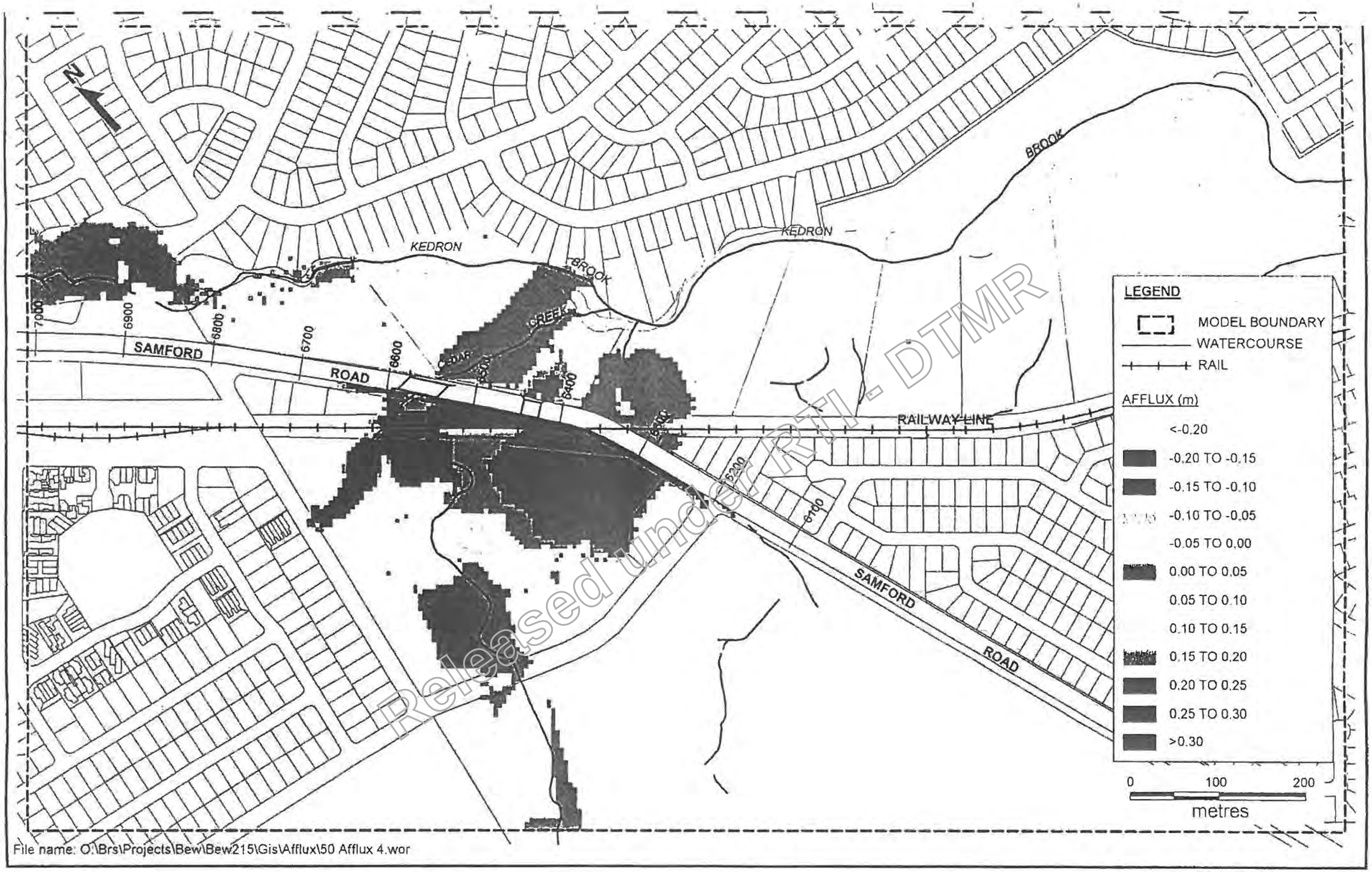

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Figure 4.26
50 YEAR ARI EVENT
AFFLUX
OPTION 4

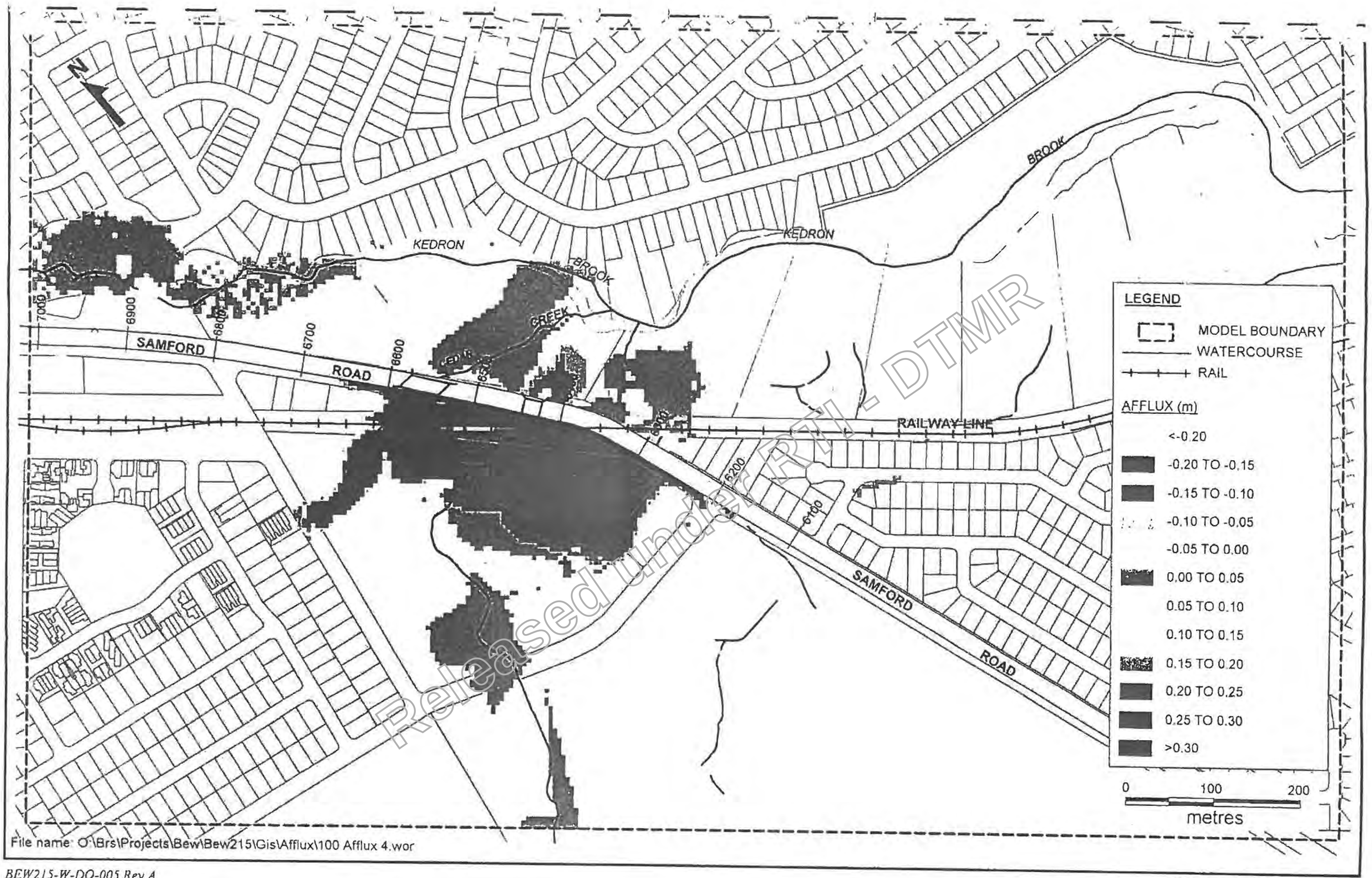

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Figure 4.27
100 YEAR ARI EVENT
AFFLUX
OPTION 4

5 Model reliability and accuracy

The results in this report rely on the performance of hydraulic modelling, with some inputs from a hydrology model. Consideration of the reliability and accuracy of the modelling is necessary since the analysis needs to consider especially small differences in flood levels where the issues of flood levels and afflux are sensitive. This is especially the case since a range of similar scenarios is compared.

The performance of any model depends on a number of factors and those that are important in this case are discussed below.

The results of this analysis consist of two components, namely the absolute water levels and the differences in levels between cases. The differences in water levels are more accurate than the absolute values, since the same assumptions are made in each case used for the calculation of the differences.

The amount and accuracy of calibration data has a critical impact on the reliability of the model results. In this case, the 'calibration' has been based on a comparison with existing and accepted models, which have been previously calibrated for other locations in the Kedron Brook watercourse. Since the general performance of the model is quite consistent with the previously calibrated model, the results should be reasonably reliable. There is a difference in approach however since the previous work has relied on one-dimensional models, where the water level is assumed constant across a cross section, while the current analysis uses a two-dimensional approach where the water level can vary over the floodplain. This is a particular issue in this region where flow patterns are complex and show clear two-dimensional flow properties.

The calculation of the culvert and bridge hydraulics is especially important in this assessment and the results are analysed to compare very small differences. Culvert and bridge hydraulics are difficult to calculate to the level of accuracy required in this case, both because of the complex hydraulic behaviour as well as the potential impacts of local effects, such as debris blockage for example. The calculation of culvert and bridge hydraulics is further complicated in this project because of the close proximity of the railway and road structures and the interaction between these two structures.

It is difficult to define a level of accuracy of the absolute water levels and the differences in water levels, but an accuracy of 0.2 m for the absolute water levels and 0.05 m for the differences would seem to be reasonable. The results in this report have been presented to an apparently higher accuracy than this for comparative purposes.

Therefore while the results are presented to this high level of accuracy, the basis of the analysis must be borne in mind and the real impacts considered in this light.

6 Ferny Way—hydraulic analysis

As part of the Samford Road upgrade, some minor works are also proposed along Ferny Way. These works involve the widening of the existing road from two to four lanes. With the widening of the road, an extension of the existing culvert cells (by approximately 20 m) is also required. It should be noted that the existing culvert configuration and the vertical alignment along Ferny Way remain unchanged.

As part of this hydraulic study, the effects of the increasing the length of the culvert at Ferny Way has been investigated using the DELFT-FLS model used in the hydraulic assessment along Samford Road. As the culvert dimensions and vertical elevation of the road remains the same, the only afflux generated at the crossing is due to the additional friction loss generated from a longer culvert length.

Hydraulic modelling of Ferny Way has indicated that the maximum afflux for both the 50 year and 100-year ARI events produced from lengthening the culverts are 30 mm and 27 mm respectively. These small increases in peak water elevation have a minor impact on the extent of inundation upstream of Ferny Way.

7 East Cedar Creek—hydraulic analysis

Similarly to the works along Ferny Way, the proposed works at the Samford Road crossing of East Cedar Creek involves widening of the existing road from two to four lanes. At present, the waterway configuration at East Cedar Creek consists of a single 9.15 m span. Under the proposed works associated with the Samford Road upgrade, the bridge size is to remain the same but is widened from 11-25 m to accommodate the additional two lanes.

The primary objective of the hydraulic investigation at East Cedar Creek is to determine the hydraulic impacts from widening the bridge. This analysis has not used the DELFT-FLS model used in the Cedar Creek analysis but has been undertaken by using the one-dimensional hydraulic model HEC-RAS. Previous HEC-RAS models for East Cedar Creek developed in the preliminary planning phase in the region were used as the basis of the current investigation. These models were reviewed and have been amended to provide an assessment of the currently adopted designs. The relatively simple condition of the channel in this reach means that the assumptions of the one-dimensional modelling apply.

A review of the hydraulic performance of the existing East Cedar Creek bridge has indicated that the bridge appears small for the amount of flow that occurs as it is producing backwater effects upstream in both the 50 year and 100-year ARI events. In should also be noted that peak velocities through the bridge are in excess of 3.5 m/s. This high velocity produces head losses across the structure of approximately 1 m for both design events. While this flow velocity would not normally be recommended for design of a bridge because of the possibility of scour damage, the history of this bridge indicates that the flow velocity would be acceptable.

The flood profiles through the reach of East Cedar Creek are plotted in Figures 7.1 and 7.2 which show this significant water level drop. HEC-RAS modelling of the proposed bridge arrangement indicates affluxes immediately upstream of the bridge of 110 mm and 120 mm for the 50 year and 100-year ARI events respectively. Notwithstanding this increase in upstream flood levels, however, does not cause 'breakout' of the constructed channel upstream of the bridge.

Figure 7.3 shows the cross section immediately upstream of Samford Road, which shows the ARI 100-year flood still contained within the channel. This figure shows that the flood level is still contained within the main channel of East Cedar Creek and therefore would not affect neighbouring properties.

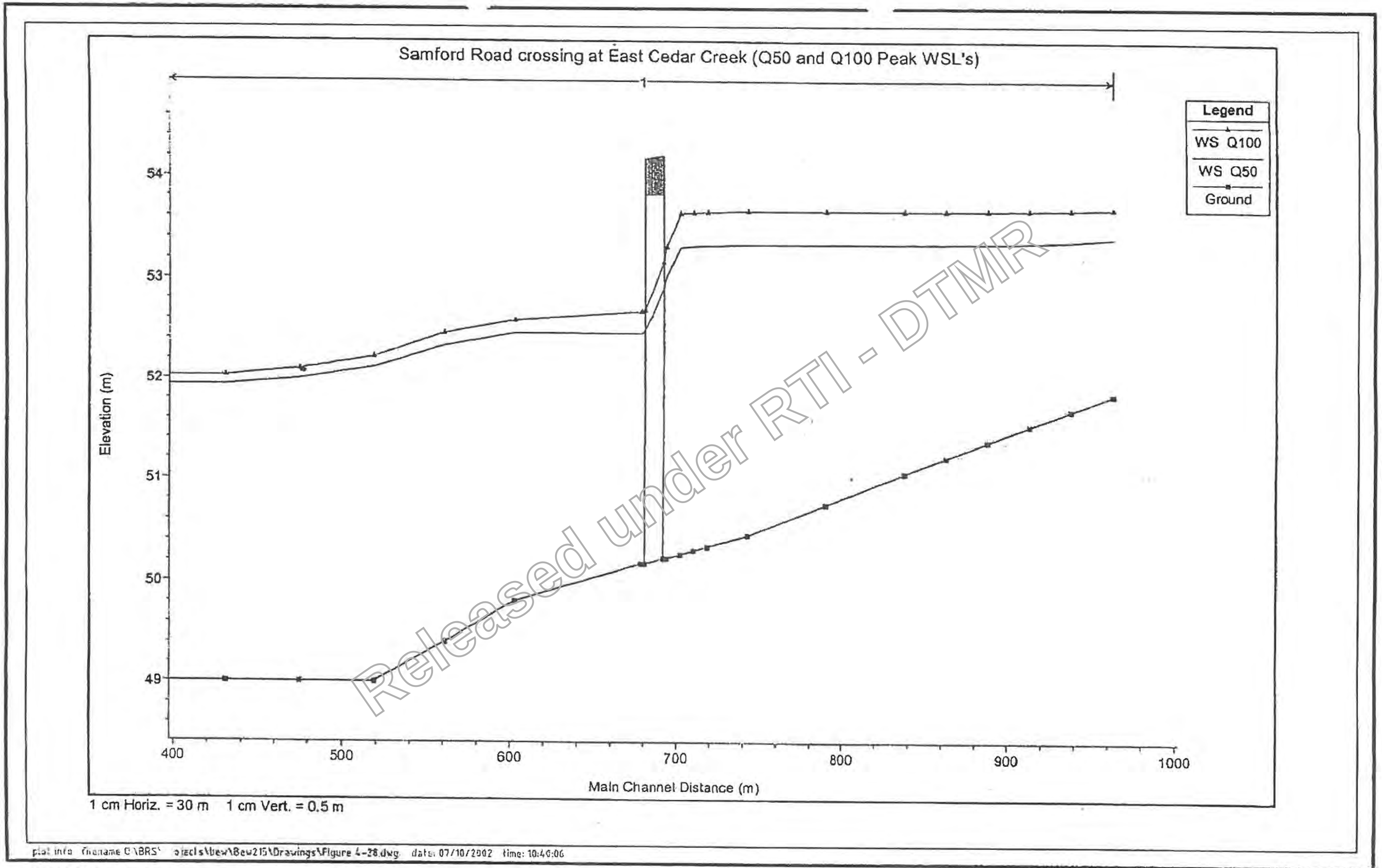

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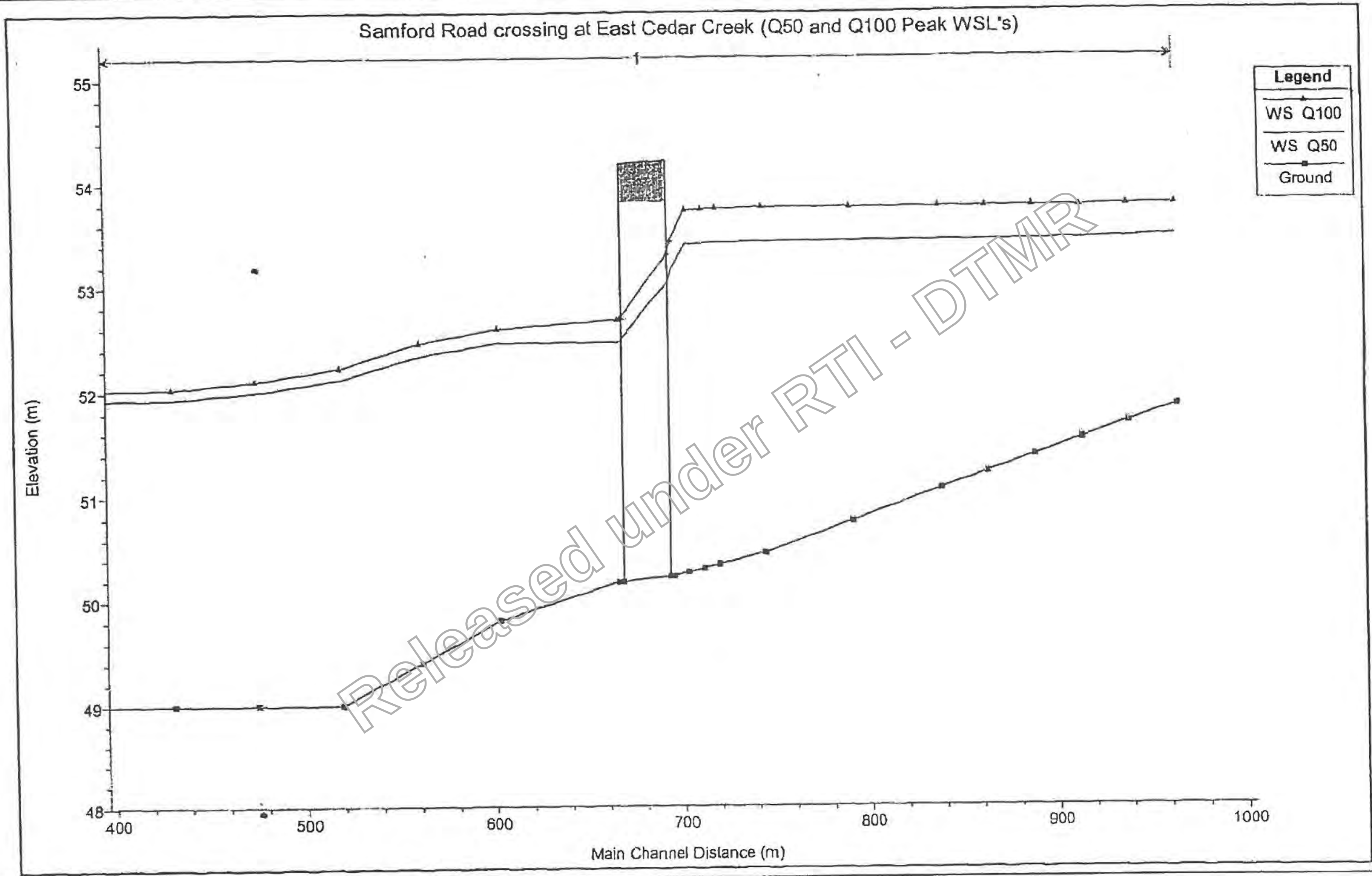
Figure 7.1
EAST CEDAR CREEK
50YR AND 100YR ARI PEAK WSL's
EXISTING CONDITIONS



plot info filename O:\BRS\Projects\bew\Bew215\Drawings\Figure 4 29.dwg date: 07/10/2002 time: 10:42:06

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Figure 7.2
EAST CEDAR CREEK
50YR AND 100YR ARI PEAK WSL's
PROPOSED CONDITIONS

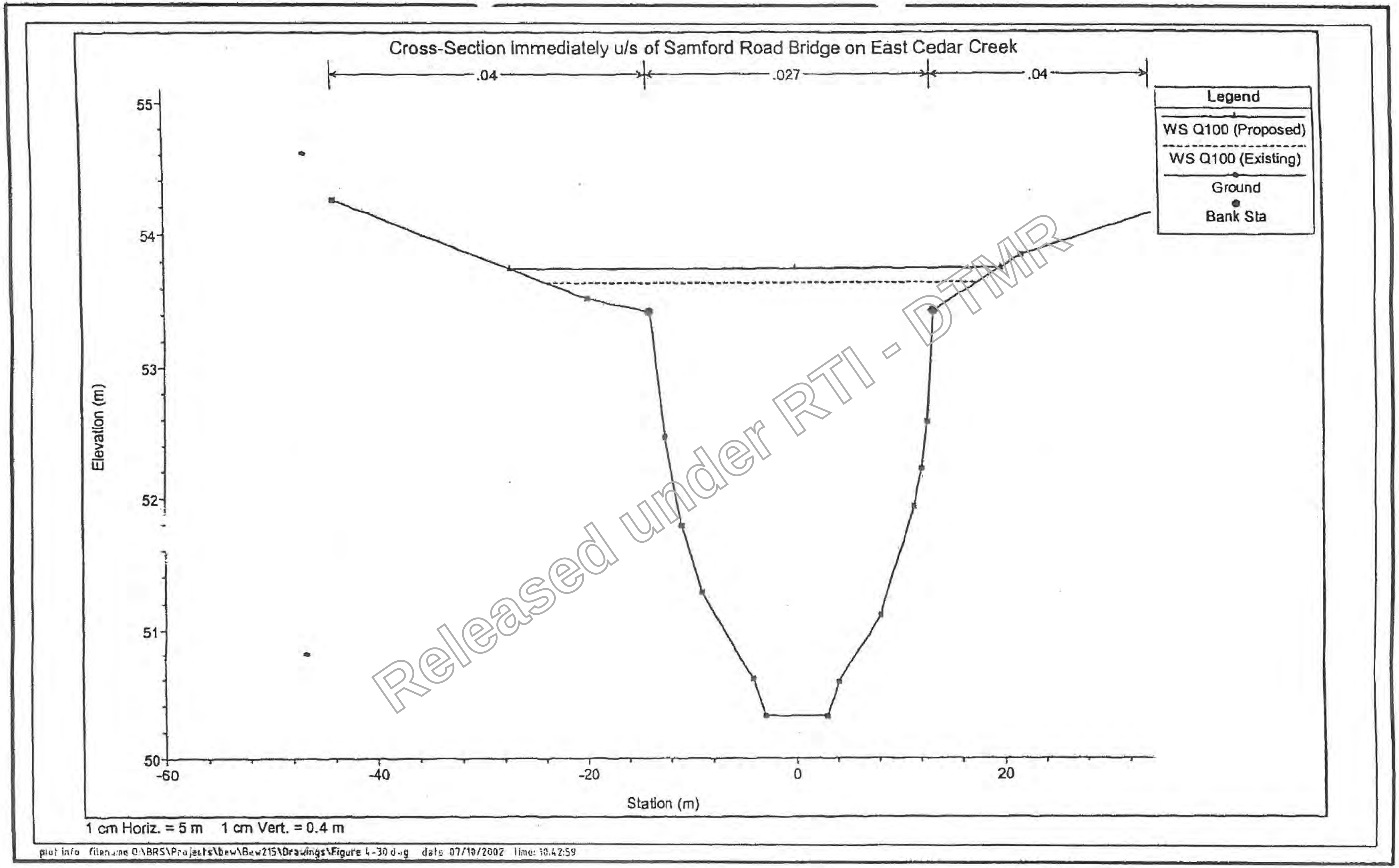

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Figure 7.3
EAST CEDAR CREEK
100YR ARI PEAK WSL COMPARISON AT CROSS SECTION IMMEDIATELY U/S OF SAMFORD RD

8 Local drainage—Samford Road

8.1 INTRODUCTION

This section considers requirements of local drainage for minor catchments in the section between Cedar Creek and East Cedar Creek. There are five separate catchments that drain east across Samford Road. All of these catchments eventually drain towards Kedron Brook. Culverts are provided under Samford Road to drain these catchments.

During the Planning and Preliminary Design Phase of the project, Bornhorst & Ward produced a report that considered flooding aspects of the project. It was assumed, during this study, that the upgraded road would adopt the same culverts as are currently existing. However, it does not appear that the capacity of culverts was checked against required road flood immunity levels.

A more complete analysis is now required to ensure that the design can perform to the required standard. This involves assessing the capacity of the existing culverts. The objective is to provide sufficient hydraulic capacity such that there is a minimum 150 mm freeboard to the road shoulder during the ARI 50-year storm event.

8.2 EXISTING CONDITIONS

Most of the catchment area on the western side of Samford Road comprises a closed refuse tip, which has been converted into grassed playing fields. It is usual that completed landfills are capped with a low permeability layer, which tends to minimise infiltration and promote surface runoff. It is therefore expected that runoff rates from the catchment would be relatively high.

As previously mentioned, there are five discrete catchments that currently drain under Samford Road through culverts. Culvert sizes and locations are provided in Table 8.1. Each culvert has been allocated a number, as shown in the table, for reference throughout this report.

Table 8.1 Existing drainage structures

Culvert	Approximate Samford Road chainage (m)	Existing drainage details
1	5490	1/900 RCP
2	5710	1/600 RCP
3	6060	1/450 RCP
4	6140	2/1200x600 RCBC
5	6290	2/1200 RCP

It is noted that Culvert 4 is under Tramway Street and runs parallel to Samford Road.

8.3 METHODOLOGY

8.3.1 Hydrologic analysis

The first part of the local drainage analysis involves estimating the peak discharge from each catchment draining to Samford Road. The design criterion requires 150 mm freeboard to the road level at the upstream end of the culvert for the ARI 50-year event. Peak discharges have been estimated for the ARI 50-year event only.

The hydrological analysis was undertaken using the Rational Method as presented in 'Australian Rainfall and Runoff' (Institution of Engineers Australia, 1987). The time of concentration was calculated based on an average flow velocity over the grassed catchment of 0.55 m/s. The coefficient of runoff (C) adopted for the ARI 50-year event was 0.65 based on consideration of the losses and storm magnitude.

8.3.2 Hydraulic analysis

The hydraulic capacity of the existing and proposed drainage arrangements have been assessed using the Austroads Waterway Design manual (AUSTROADS 1994). This takes into consideration flow rates, entrance losses, contraction losses and tailwater influences.

Culverts have been assessed based on a 150 mm freeboard. Details of proposed road levels and invert levels of drainage paths have been taken from the road plans provided in the planning report. The permissible headwater levels, based on these plans, are shown in Table 8.2.

Table 8.2 Allowable headwater

Culvert	Approximate Samford Road level—M01 (m AHD)	Approximate Samford Road natural surface level (m AHD)	Allowable headwater (m)
1	55.82	54.00	1.67
2	60.27	59.00	1.12
3	58.95	57.70	1.10
4	55.11	54.00	0.96
5	54.04	52.10	1.79

Where the capacity of existing culverts (as proposed in the Preliminary Planning Study) is less than the calculated peak flow, additional waterway area is required to meet the flood criterion. In addition to the provision of additional water way area, alternative options for the drainage of excess water were also considered.

8.4 DESCRIPTION OF CULVERTS

8.4.1 Culvert 1

The catchment of Culvert 1 drains a section of park land, including the closed refuse tip. The downstream portion of the flow path is a lined channel, which flows firstly under the footpath, before reaching the drain under Samford Road. The flow path reaches Samford Road north of the junction of Samford and Upper Kedron Roads. The drainage across the road is a 900 mm diameter pipe, which connects to another 900 mm pipe that directs flow under the houses on the eastern side of Samford Road. The house immediately opposite the flow path has a block fence across the flow path that would limit overland flow that may be in excess of the capacity of the pipe. Any flow in excess of the capacity of the pipe would flow over the road and then through the properties on the downstream side of the road. This water eventually reaches East Cedar Creek.

8.4.2 Culvert 2

Culvert 2 is located further to the north of Culvert 1 on Samford Road. It is located south of the main catchment divide and the water from this catchment also flows towards East Cedar Creek. The catchment of Culvert 2 also includes park land in the closed refuse tip. The lower part of the flow path has a short section of lined drain leading into a minor culvert under the footpath and then into the main drain across Samford Road. There is no overland flow path through the houses on the downstream side of the road.

The drainage under the road is a 600 mm diameter pipe. However the drainage immediately downstream under the houses is a 525 mm diameter pipe, so it has a smaller capacity than the pipe under the road.

8.4.3 Culvert 3

Culvert 3 is different from the others in this section of the road in that the flow crosses the road in the opposite direction. Water flows from a small catchment south of Claverton Street and across Samford Road from the east to the west. The catchment is mainly residential. The outflow from Culvert 3 joins the open channel that flows towards Culvert 4 under Tramway Street. The catchment is small and the drainage under the road is one 450 mm diameter pipe. Flow above the capacity of the pipe will flow over the road.

8.4.4 Culvert 4

Culvert 4 is part of a major drainage line that runs parallel to Samford Road under Tramway Street.

The catchment of Culvert 4 also consists mainly of the park land in the former refuse tip, but with some development such as the Bowls Club. The runoff from the catchment flows into an open drain that runs towards the north and parallel to Samford Road on the western side.

The drainage consists of 2/1,200 x 600 RCBCs. The upstream end of the culvert is at the end of the open drain. Downstream of the culvert, there is another open drain that flows towards the north past the Ferny Grove Police Station.

8.4.5 Culvert 5

Culvert 5 is located in the open drain that discharges from the outlet of Culvert 4. It directs water towards the east under Samford Road. The outflow from Culvert 5 flows across a small area of low lying ground before entering culverts under the railway line and taking the flow into the floodplain of Kedron Brook.

The culverts under Samford Road are 2/1,200 mm diameter pipes and the culverts under the railway line are 3/825 mm diameter pipes. The railway culverts have a smaller waterway area than the culverts under Samford Road so they will control the flow.

There is a bypass channel from the inlet of Culvert 5, which directs any flow in excess of the culvert capacity towards the north and west and into the floodplain of Cedar Creek, just upstream of the railway culverts. This is a constructed channel and seems to have been constructed to manage excess flow that does not flow through Culvert 5. In addition to the waterway area, the capacity of this culvert is also affected by the angle of the inlet and the possibility of debris. Therefore even if the capacity of Culvert 5 is inadequate there is unlikely to be any flow over the road, since the water will flow towards Cedar Creek.

8.5 HYDROLOGIC ANALYSIS

The hydrologic analysis was completed using the Rational Method. A summary of the catchment properties and estimated peak discharges are presented in Table 8.3.

Further details of the hydrology assessment are included in Appendix C.

Table 8.3 Catchment hydrology

Culvert	Catchment Area (ha)	Time of Concentration (min)	ARI 50 year Rainfall Intensity (mm/h)	ARI 50 year Peak Discharge (m^3/s)
1	9.6	20	166	2.9
2	5.4	11	215	2.1
3	0.7	6	270	0.3
4	19.0	22	159	5.4
5	22.8	26	147	6.0

8.6 HYDRAULIC ANALYSIS (EXISTING)

The Preliminary Planning Study proposed that culverts identical to those in the existing road be used for the upgraded road. The capacity of the existing culverts have therefore been assessed, and the results presented in Table 8.4. ARI 50-year peak design discharges have been included for comparison.

Further details of the hydraulic assessment are provided in Appendix D.

Table 8.4 Existing culvert capacities

Culvert	Existing culverts	Culvert Capacity (m^3/s)	ARI 50-year peak discharge (m^3/s)	Existing culvert(s) suitable
1	1/900 RCP	1.70	2.9	No
2	1/600 RCP	0.58	2.1	No
3	1/450 RCP	0.34	0.3	Suitable
4	2/1200x600 RCBC	2.00	5.4	No
5	2/1200 RCP	5.20	6.0	No

Note: 1. The capacity of culverts was assessed based on the 150 mm freeboard criteria

As evident in the table, only Culvert 3 meets the design criteria. The existing arrangement at the other four culvert locations does not have sufficient capacity to pass the ARI 50-year event.

8.7 DRAINAGE OPTIONS

8.7.1 Introduction

The culverts for these minor catchments have been analysed and options are presented for upgrading to meet the flood design standard. Details of these are as described below.

The first option was to simply increase the capacity of each culvert to convey the ARI 50-year flood. The waterway area of the culverts at these locations has been increased to meet this design criteria, with the proposed culvert arrangements summarised in Table 8.5. This arrangement assumes that there are no diversions and possible downstream impacts are neglected.

Table 8.5 Proposed culverts

Catchment	Proposed drainage arrangement	ARI 50-year peak discharge (m^3/s)	Allowable headwater (m)	Predicted headwater (m)
1	2/900 RCPs	2.9	1.67	1.41
2	4/600 RCPs	2.1	1.12	0.97
3	1/450 RCP	0.3	1.10	0.94
4	4/1200x600 RCBC	5.4	0.96	0.79
5	3/1200 RCPs	6.0	1.79	1.49

Following this initial assessment, each culvert location was considered in more detail to assess any alternative possibilities.

8.7.2 Culvert 1

Culvert 1, the catchment closest to the intersection with Upper Kedron Road, was noted as not having adequate capacity to convey the ARI 50-year flood event. The capacity of the culvert has been calculated as 1.7 m^3/s, which represents about 60% of the discharge of the ARI 50-year flood, which is 2.9 m^3/s. The culvert capacity needs to be doubled to 2/900 RCPs to convey the ARI 50-year flood from the catchment of Culvert 1.

There is a problem however with this culvert flow once it crosses the road. In the existing condition, the culvert will convey part of the ARI 50-year flood, while the excess will cross the road. On the downstream side of the road, the flow up to the capacity of the pipe will continue to flow in the pipe while the excess will flow through the downstream properties.

An alternative approach to the upgrading of this drainage is to retain the downstream piped drainage system, but divert part of the flow from the catchment upstream of Samford Road. This would involve the diversion of 1.2 m^3/s towards the east and into East Cedar Creek. This water would need to be diverted along a drainage system parallel to the road on either the northern or southern side. While both options are possible, the diversion along the northern side would seem to offer benefits, because the flow path is a shorter distance and the drainage does not need to cross the intersection of Upper Kedron Road. This option of diverting excess flow is of benefit to the properties immediately downstream of the road, since they would become protected from existing flooding problems.

As discussed below, the possibility of diverting water from Culvert 2 into this catchment is also considered a possibility. This diversion would add an additional 1.5 m^3/s to the flow into the catchment of Culvert 1. This means that there is a total of 4.4 m^3/s flowing into the culvert under the road if this water is diverted.

There are therefore two possible options for the management of drainage at Culvert 1. The first is to simply upgrade the culvert under the road and allow the excess flow above the capacity of the downstream system to flow overland. The second alternative is to upgrade the drainage under the road and then to provide for the flow parallel to the road and into East Cedar Creek.

If the first option is adopted, this will require upgrading the culvert from 1/900 RCP to 2/900 RCP, leaving the downstream conditions unaltered. This does not worsen the downstream conditions.

If the second option is adopted, the culvert under the road needs to be upgraded to 3/900 RCPs because of additional water diverted from the catchment of Culvert 2. This then needs to be in conjunction with a drain parallel to the road from the outlet of the culvert to East Cedar Creek on the northern side of the road, either as a piped system or an open channel of some type. This drain would need to have a capacity of 2.75 m^3/s. The arrangement of this system would need to be designed as part of the road drainage design. Note that this would require the construction of a surcharge pit.

8.7.3 Culvert 2

Culvert 2 does not have capacity to convey the ARI 50-year flood. To convey the design flood, this culvert needs to be upgraded from the existing 1/600 mm pipe to 4/600 mm pipes, a significant increase. In this case, the downstream drainage is of low capacity, similar to that of the pipe under the road. Therefore if the drainage under the road was increased, there would still be flow in excess of the downstream capacity, which would need to flow overland.

The capacity of the existing pipe is 0.6 m^3/s, 30% of the discharge of the ARI 50-year flood of 2.1 m^3/s.

As noted above in the options for Culvert 1, an option for Culvert 2 is to divert the excess flow towards the south and into the catchment of Culvert 1. This option would provide benefits to the residences downstream of the culvert by reducing the overland flow and maintaining the flow through the section to the capacity of the piped system. Diversion of this flow will require construction of a drainage channel parallel to the road towards the south. The detailed design of this would need to be part of the road design, but the flow could be conveyed by a trapezoidal channel with a base width of 0.8 m and flat side slopes of 1:6.

8.7.4 Culvert 3

Culvert 3 can be reconstructed with the existing capacity of 1/450 mm RCP and this will meet the ARI 50-year flood criterion.

8.7.5 Culvert 4

The existing 2/1,200 X 600 RCBC under Tramway Street does not have sufficient capacity to convey the ARI 50-year flood. This culvert needs to upgraded to 4/1,200 x 600 RCBC to convey the design flood.

This is the only feasible option.

8.7.6 Culvert 5

The capacity of the existing 2/1,200 RCPs at this location would be 5.2 m^3/s without a downstream control, but this discharge is limited by the capacity of the railway culverts downstream. The capacity of the downstream railway culverts is only 4.1 m^3/s.

The inflow from the catchment to Culvert 5 is 6.0 m^3/s, so the culverts with their existing capacity (assuming a control from the railway culverts) can convey 70% of the total flow.

On the assumption that the channel running towards the north and into the floodplain of Cedar Creek is not altered, the excess flow above the capacity of the culvert can discharge towards Cedar Creek and the existing culvert capacity is adequate. This channel must be adequate to convey a flow of 1.9 m^3/s. Based on the slopes and short distance, the overland flow channel needs to have as waterway area of 2 m^2.

This means that the existing culvert capacity does not need to be altered.

8.8 DISCUSSION

While four of the culverts proposed in the Preliminary Planning Study do not meet the design criterion for the ARI 50-year event, the impact of flooding in a practical sense may be minimal. That is, since the upstream catchments are relatively small (resulting in fast response times and relatively small peak flows), the extent of overtopping and the duration of overtopping would be minimal. It is likely that neither property damage or inconvenience would be created by the floodwaters overtopping the road.

8.9 CONCLUSION

This study has identified that the culverts proposed in the Preliminary Planning Study for the Samford Road Upgrade would generally not meet the design criterion. The Preliminary Planning Study proposed that the existing culverts be matched in the new design.

Enlarged culverts with increased waterway area have been proposed to meet the design criterion of 150 mm freeboard to the design road level. In addition, alternative options have been considered.

9 Conclusion

This report provides a flood assessment of a proposed upgrade along a section of Samford Road between Cobalt Street and Ferny Way. The section of road crosses two significant watercourses, East Cedar Creek and Cedar Creek, both of which are tributaries of Kedron Brook.

Preliminary hydraulic analysis for the road upgrade was first undertaken by Bornhorst & Ward in 1998 as part of the Planning and Preliminary Design Phase of the project. This preliminary analysis recommended a waterway opening arrangement that met all the flood requirements of the road. The objective of this report gave a good understanding of the proposed costing of the project and to determine its feasibility.

The project now requires a more detailed hydraulic analysis to be undertaken to ensure that the proposed road upgrade arrangement could still maintain flooding requirements. To do this, the two-dimensional hydraulic model DELFT-FLS was selected as an appropriate tool to complete the detailed analysis.

In addition to simulating the proposed waterway openings recommended in the preliminary report (referred to as Option 1), a number of other options of waterway openings were simulated to determine the most cost-effective arrangement that meets all flooding requirements.

From the analysis, it was determined that Option 1 (the preliminary arrangement of waterway openings proposed in the preliminary study) was the option that came closest to meeting all the design guidelines. While the freeboard criterion was not met, the impacts of the proposed upgrading as analysed in this option would be the same as for a road with a higher control level, since the road was not totally overtopped. The disadvantage of this option was the impact on the railway line, immediately upstream of the road.

Of the other options, Option F1 provides benefits to the afflux at the railway line by diverting flow away from the railway culverts and towards the bridge, but does have the effect of increasing water levels in the western side of Cedar Creek and into the minor tributary that crosses Arbor Street and flows into the creek. The afflux in this region would cause a minor area of inundation in Arbor Street but the increased water level does not reach the floor level of the lowest residence in the location.

The impacts of the proposed upgrading has been analysed for the floods with average recurrence intervals of 50 and 100 years. The largest affluxes are in the region between Samford Road and the railway line, with smaller affluxes further upstream. The afflux at the railway line does affect the flood immunity of the railway.

In addition to the investigation at Cedar Creek, this report also provides a hydraulic assessment of the proposed crossings at Ferny Way and at East Cedar Creek. For both locations, it was determined that lengthening of the existing structures to accommodate the widening of Samford Road was hydraulically adequate.

The local drainage for a number of minor watercourses was also analysed and culvert sizing prepared.

10 References

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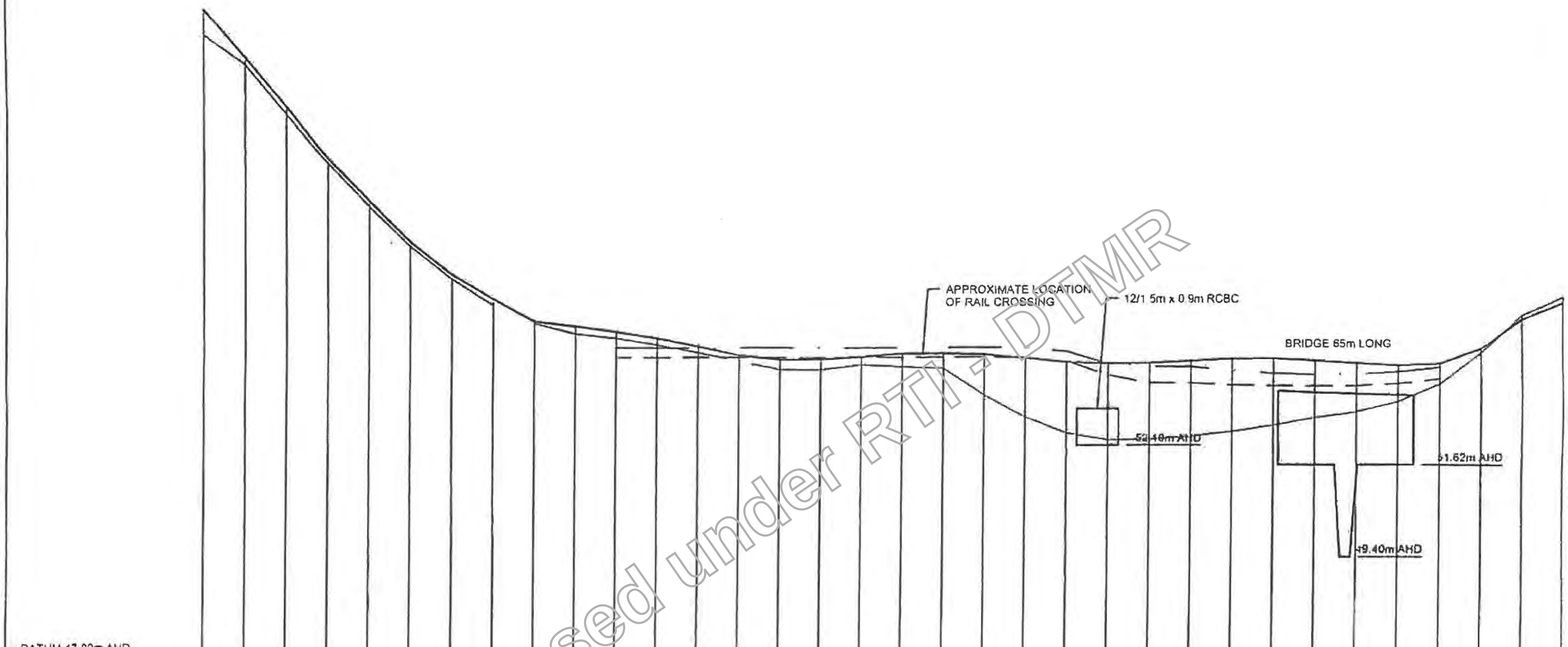
Appendix A

LONGITUDINAL PROFILES



DATUM 47.00m AHD

CONTROL LINE CHAINAGE	100 YR ARI FLOOD LEVEL	50 YR ARI FLOOD LEVEL	EXISTING SAMFORD ROAD LEVEL (m AHD)	CONTROL LINE M001 FINISHED LEVEL (m AHD)
5980				
5995				
6000			61.97	62.51
6020			61.26	61.36
6040			60.10	60.21
6060			58.94	58.99
6080			57.88	57.95
6100			56.94	56.98
6120			56.15	56.19
6140			55.50	55.58
6160			55.08	55.03
6180			54.80	54.91
6200	54.46	54.23	54.70	54.78
6220	54.46	54.23	54.54	54.63
6240	54.46	54.23	54.35	54.42
6260	54.46	54.23	54.10	54.21
6280	54.46	54.23	53.93	54.10
6300	54.46	54.23	53.93	54.11
6320	54.46	54.23	54.05	54.17
6340	54.46	54.23	54.00	54.25
6360	54.47	54.23	53.97	54.25
6380	54.47	54.23	53.33	54.23
6400	54.46	54.23	52.80	54.14
6420	54.37	54.13	52.41	54.04
6440	54.07	53.80	52.23	54.00
6460	54.01	53.61	52.25	54.02
6480	54.00	53.61	52.33	54.07
6500	53.90	53.56	52.44	54.12
6520	53.85	53.54	52.59	54.13
6540	53.82	53.54	52.78	54.08
6560	53.83	53.55	52.93	54.02
6580	53.88	53.60	53.17	53.96
6600	53.99	53.71	53.59	54.01
6620			54.33	54.37
6640			55.25	55.07
6655				
6660			55.69	55.46

0 20 40 60 80 100 m

Horizontal Scale 1:2000

0 1 2 3 4 5 m

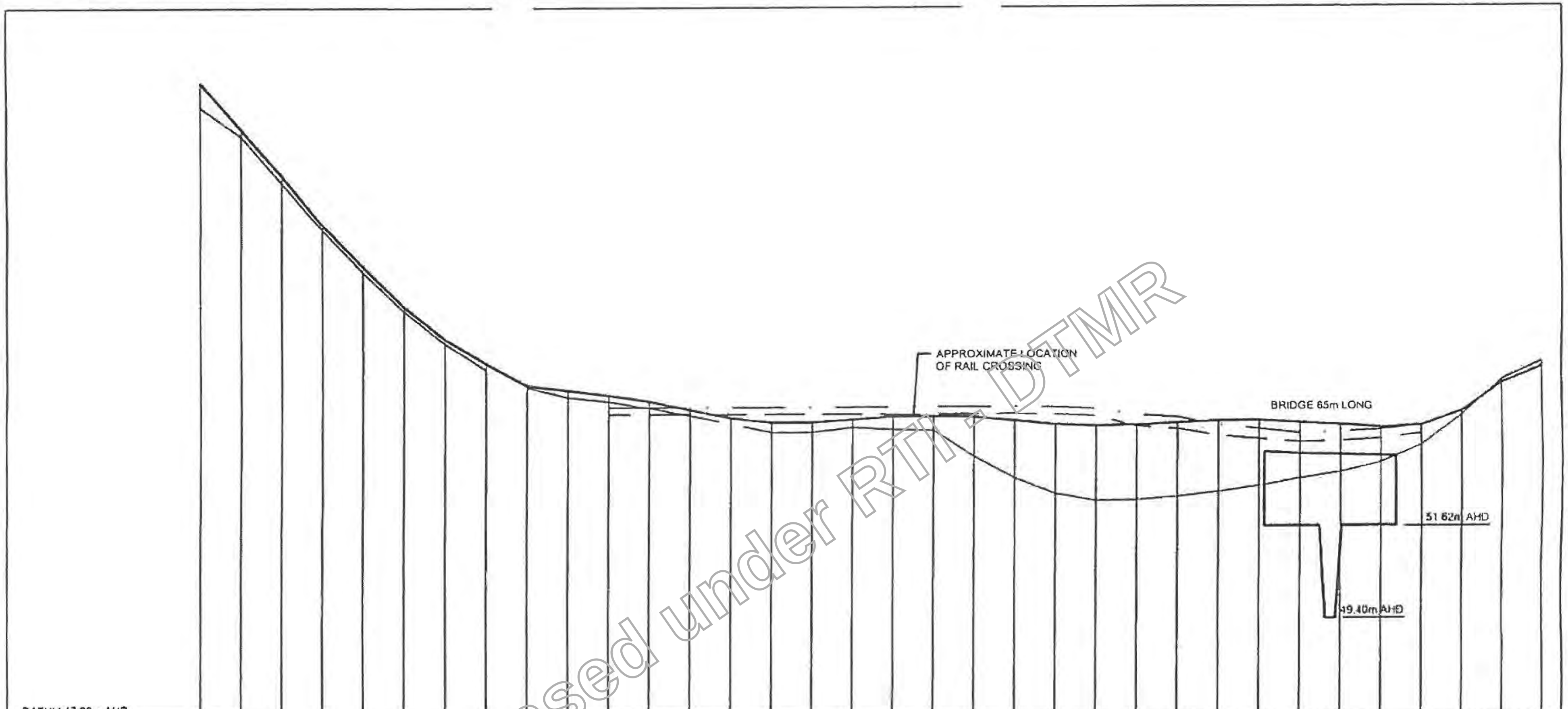
Vertical Scale 1:100

ROAD LONGITUDINAL SECTION

plot info filename O:\BRS\Projects\bew\Bew215\Drawings\FIGURES-W-DO-005\Figure A_2-005.dwg

BEW215-W-DO-005 Rev A
July 2003

Figure A.2
OPTION 1
LONGITUDINAL PROFILE AND GENERAL ARRANGEMENT



DATUM 47.00m AHD

CONTROL LINE CHAINAGE	5980	5995	6000	6020	6040	6060	6080	6100	6120	6140	6160	6180	6200	6220	6240	6260	6280	6300
100 YR ARI FLOOD LEVEL													54.55	54.56	54.56	54.56	54.56	54.56
50 YR ARI FLOOD LEVEL													54.38	54.38	54.38	54.38	54.38	54.38
EXISTING SAMFORD ROAD LEVEL (m AHD)			61.97	61.26	60.10	58.94	57.88	56.94	56.15	55.50	55.06	54.80	54.70	54.54	54.35	54.10	53.93	53.93
CONTROL LINE M001 FINISHED LEVEL (m AHD)			62.51	61.36	60.21	58.99	57.95	56.98	56.19	55.58	55.03	54.91	54.78	54.63	54.42	54.21	54.10	54.11

CONTROL LINE CHAINAGE	6320	6340	6360	6380	6400	6420	6440	6460	6480	6500	6520	6540	6560	6580	6600	6620	6640	6655	6660
100 YR ARI FLOOD LEVEL	54.56	54.56	54.57	54.56	54.56	54.54	54.51	54.35	54.29	54.10	53.99	53.88	53.92	53.97	54.09				
50 YR ARI FLOOD LEVEL	54.38	54.38	54.38	54.38	54.38	54.37	54.33	54.06	53.98	53.82	53.74	53.66	53.69	53.75	53.88				
EXISTING SAMFORD ROAD LEVEL (m AHD)	54.05	54.00	53.97	53.33	52.80	52.41	52.23	52.25	52.33	52.44	52.59	52.78	52.93	53.17	53.59	54.33	55.25		55.84
CONTROL LINE M001 FINISHED LEVEL (m AHD)	54.17	54.25	54.25	54.23	54.14	54.04	54.00	54.02	54.07	54.12	54.13	54.08	54.02	53.96	54.01	54.37	55.07		55.48

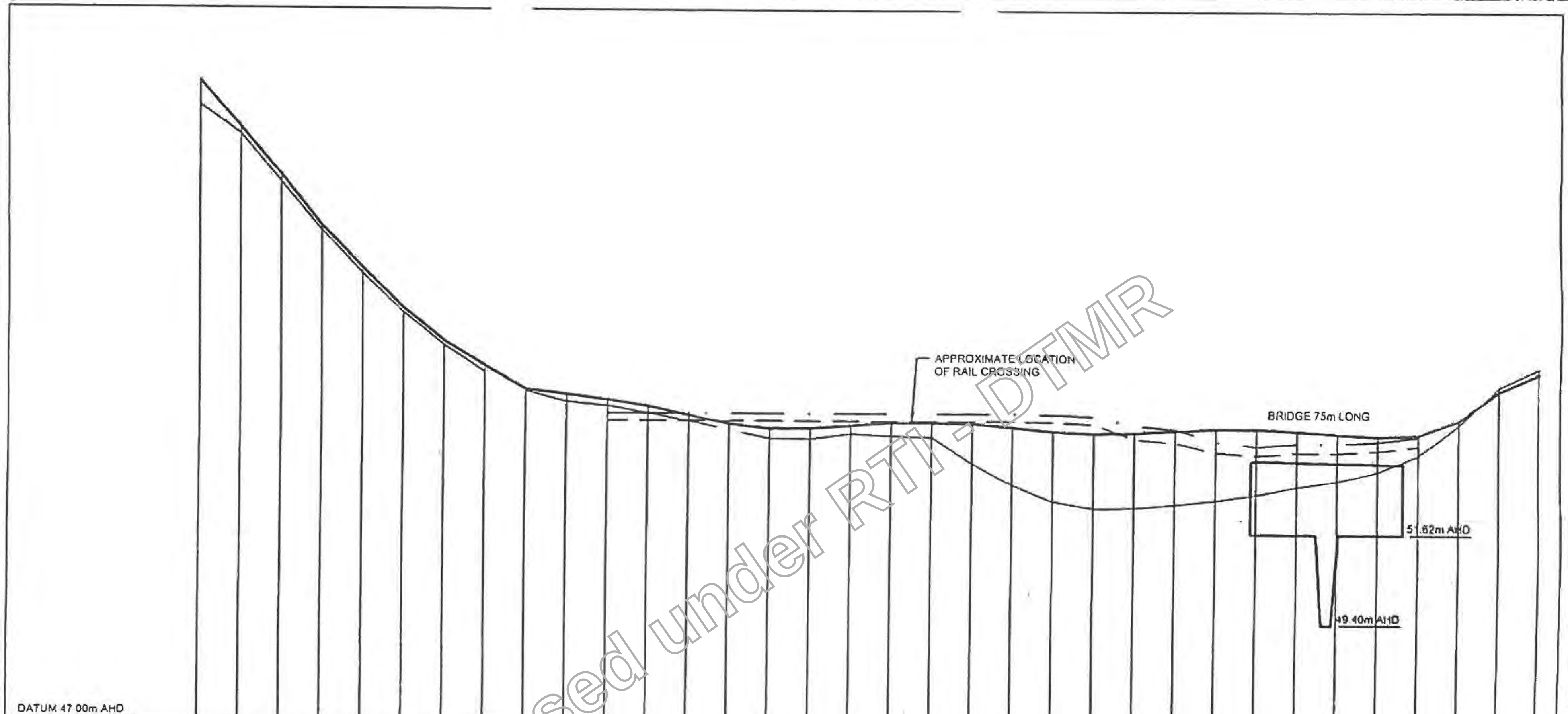
0 20 40 60 80 100 m — Horizontal Scale 1:2000

0 1 2 3 4 5 m — Vertical Scale 1:100

ROAD LONGITUDINAL SECTION

BEW213-H-DO-005 Rev A
July 2003

Figure A.3
OPTION 2
LONGITUDINAL PROFILE AND
GENERAL ARRANGEMENT



DATUM 47 00m AHD

CONTROL LINE CHAINAGE	5980	5995	6000	6020	6040	6060	6080	6100	6120	6140	6160	6180	6200
100 YR ARI FLOOD LEVEL													54.54
50 YR ARI FLOOD LEVEL													54.36
EXISTING SAMFORD ROAD LEVEL (m AHD)			61.97	61.26	60.10	58.94	57.88	56.94	56.15	55.50	55.06	54.80	54.70
CONTROL LINE M001 FINISHED LEVEL (m AHD)			62.51	61.36	60.21	58.99	57.95	56.98	56.19	55.58	55.03	54.91	54.78

CONTROL LINE CHAINAGE	6220	6240	6260	6280	6300	6320	6340	6360	6380	6400	6420	6440	6460
100 YR ARI FLOOD LEVEL	54.54	54.54	54.54	54.54	54.54	54.54	54.54	54.55	54.55	54.54	54.52	54.48	54.27
50 YR ARI FLOOD LEVEL	54.36	54.36	54.36	54.36	54.36	54.36	54.36	54.36	54.36	54.36	54.34	54.29	53.95
EXISTING SAMFORD ROAD LEVEL (m AHD)	54.54	54.35	54.10	53.93	53.93	54.05	54.00	53.97	53.33	52.80	52.41	52.23	52.25
CONTROL LINE M001 FINISHED LEVEL (m AHD)	54.63	54.42	54.21	54.10	54.11	54.17	54.25	54.25	54.23	54.14	54.04	54.00	54.02

CONTROL LINE CHAINAGE	6480	6500	6520	6540	6560	6580	6600	6620	6640	6655	6660
100 YR ARI FLOOD LEVEL	54.16	53.90	53.78	53.87	53.87	53.92	54.04				
50 YR ARI FLOOD LEVEL	53.87	53.63	53.55	53.63	53.63	53.69	53.82				
EXISTING SAMFORD ROAD LEVEL (m AHD)	52.33	52.44	52.59	52.78	52.93	53.17	53.59	54.33	55.25		55.69
CONTROL LINE M001 FINISHED LEVEL (m AHD)	54.07	54.17	54.13	54.08	54.02	53.96	54.01	54.37	55.07		55.48

0 20 40 60 80 100 m
Horizontal Scale 1:2000

0 1 2 3 4 5 m
Vertical Scale 1 100

ROAD LONGITUDINAL SECTION

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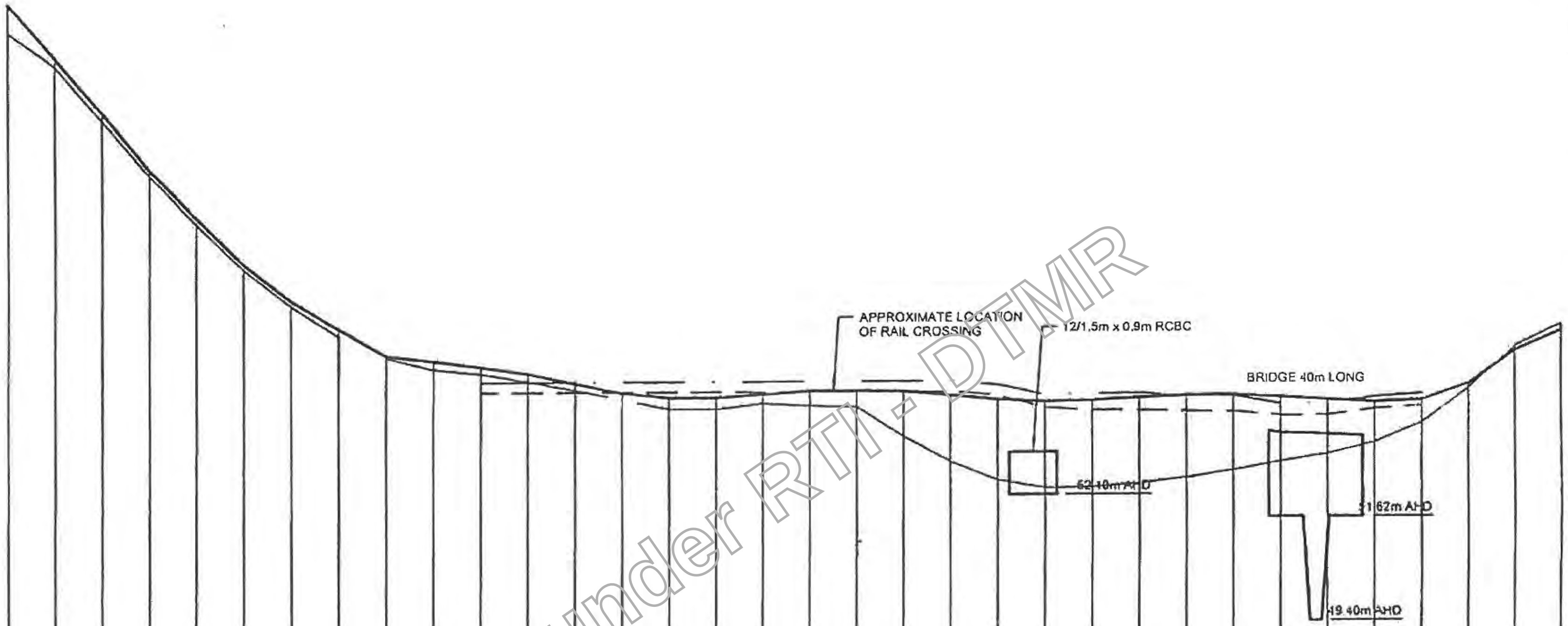
July 2003

Figure A.4
OPTION 3
LONGITUDINAL PROFILE AND GENERAL ARRANGEMENT



DATUM 47.00m AHD

100 YR ARI FLOOD LEVEL													
50 YR ARI FLOOD LEVEL													
EXISTING SAMFORD ROAD LEVEL (m AHD)			61.97	61.26	60.10	58.94	57.88	56.94	56.15	55.50	55.06	54.80	54.70
CONTROL LINE M001 FINISHED LEVEL (m AHD)			62.51	61.36	60.21	58.99	57.95	56.98	56.19	55.58	55.03	54.91	54.78
CONTROL LINE CHAINAGE	5980	5995	6000	6020	6040	6060	6080	6100	6120	6140	6160	6180	6200

100 YR ARI FLOOD LEVEL	54.53	54.53	54.53	54.53	54.53	54.53	54.53	54.53	54.53	54.52	54.45	54.22
50 YR ARI FLOOD LEVEL	54.31	54.31	54.31	54.31	54.31	54.31	54.31	54.31	54.31	54.31	54.22	53.93
EXISTING SAMFORD ROAD LEVEL (m AHD)	54.54	54.35	54.10	53.93	53.93	54.05	54.00	53.97	53.33	52.80	52.41	52.23
CONTROL LINE M001 FINISHED LEVEL (m AHD)	54.63	54.42	54.21	54.10	54.11	54.17	54.25	54.25	54.23	54.14	54.04	54.00
CONTROL LINE CHAINAGE	6220	6240	6260	6280	6300	6320	6340	6360	6380	6400	6420	6440

100 YR ARI FLOOD LEVEL	54.24	54.25	54.20	54.17	54.00	54.03	54.15	54.22				
50 YR ARI FLOOD LEVEL	53.88	53.88	53.85	53.84	53.76	53.76	53.89	53.96				
EXISTING SAMFORD ROAD LEVEL (m AHD)	52.25	52.33	52.44	52.59	52.78	52.93	53.17	53.59	54.33	55.25		55.69
CONTROL LINE M001 FINISHED LEVEL (m AHD)	54.02	54.07	54.12	54.13	54.08	54.02	53.96	54.01	54.37	55.07		55.30
CONTROL LINE CHAINAGE	6460	6480	6500	6520	6540	6560	6580	6600	6620	6640	6655	6660

0 20 40 60 80 100 m
Horizontal Scale 1:2000

0 1 2 3 4 5 m
Vertical Scale 1:100

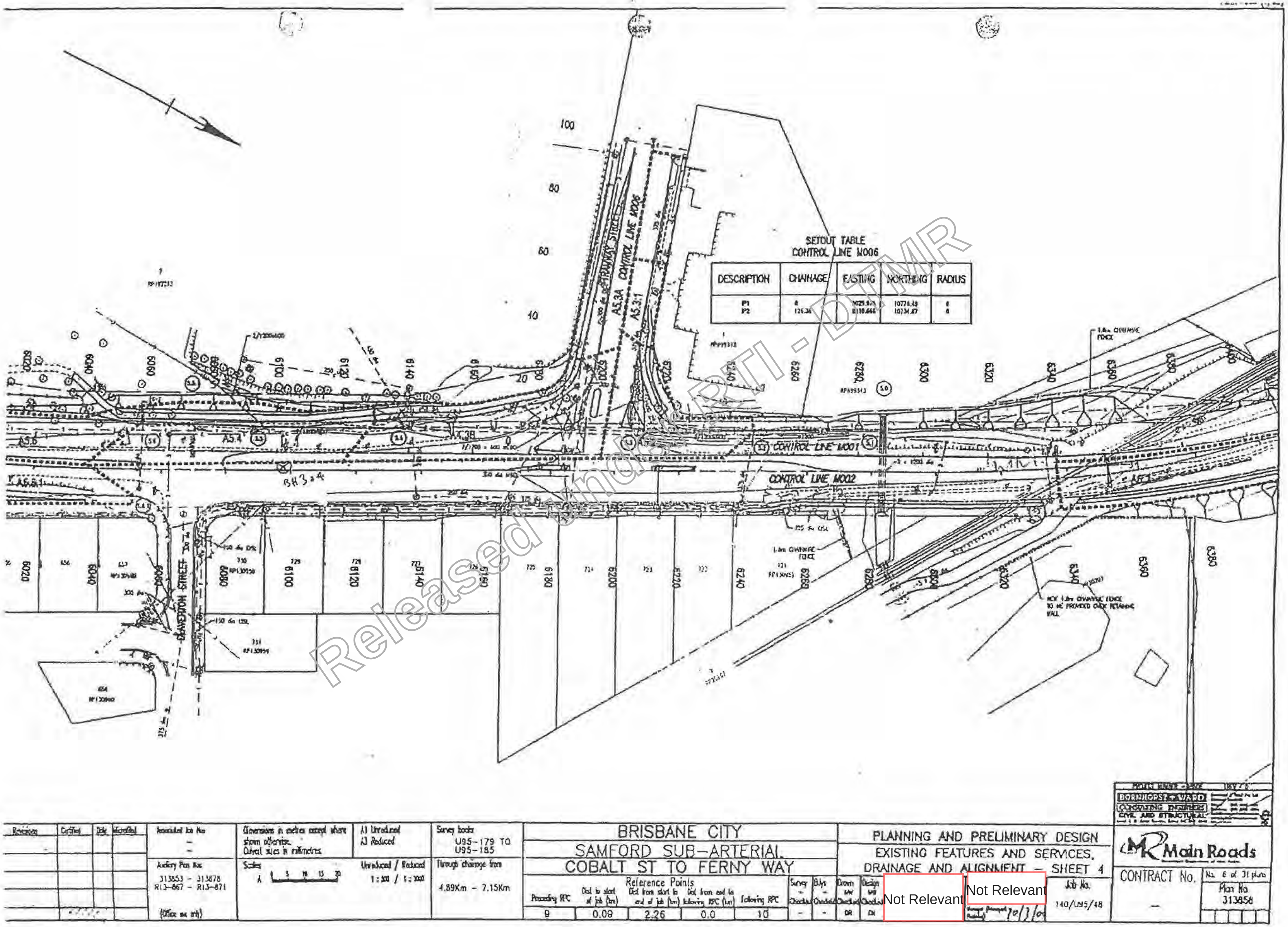
ROAD LONGITUDINAL SECTION

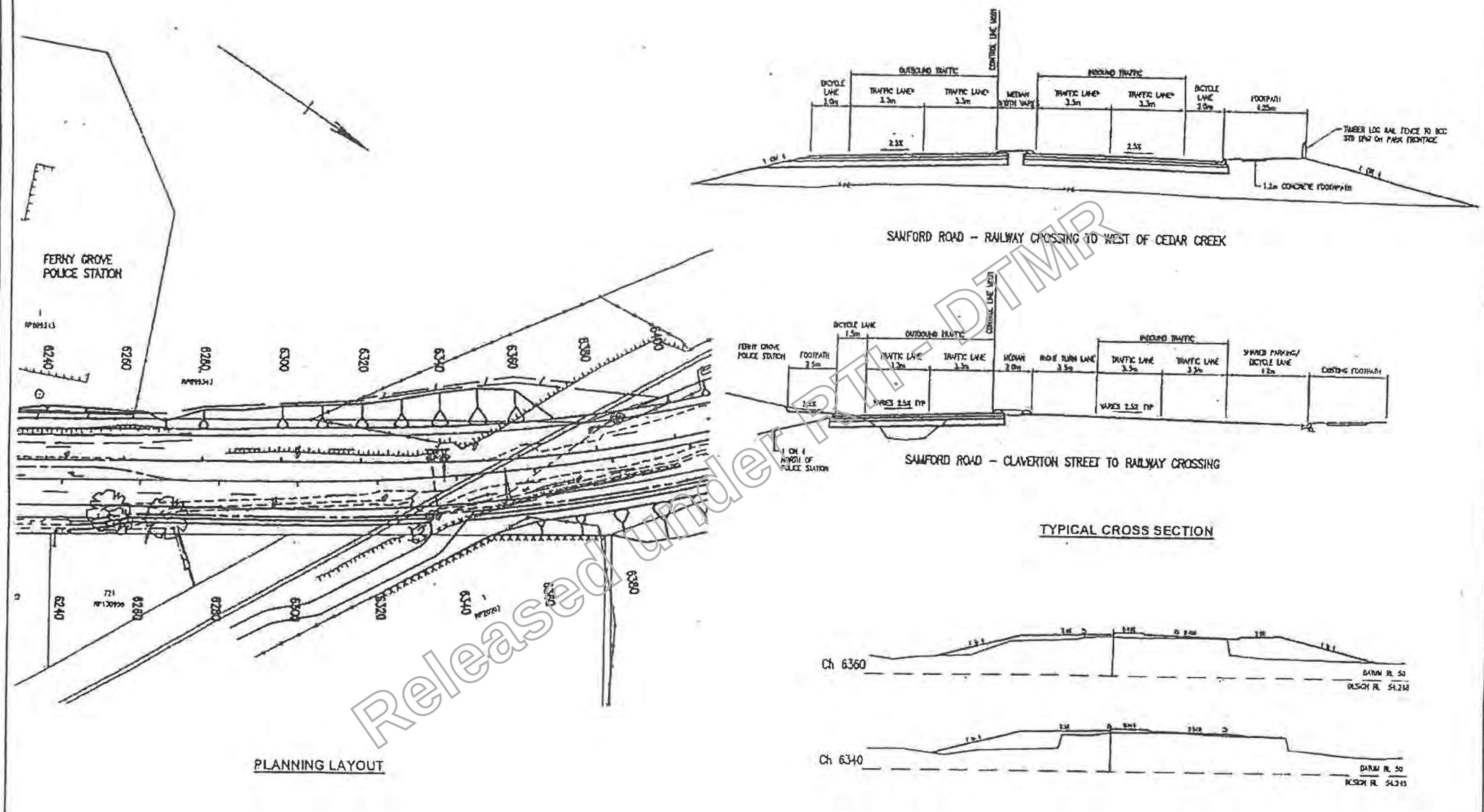
BEW215-W-DO-005 Rev A
July 2003

Figure A.5
OPTION 4
LONGITUDINAL PROFILE AND GENERAL ARRANGEMENT

Appendix B

FIGURES FROM PRELIMINARY PLANNING REPORT

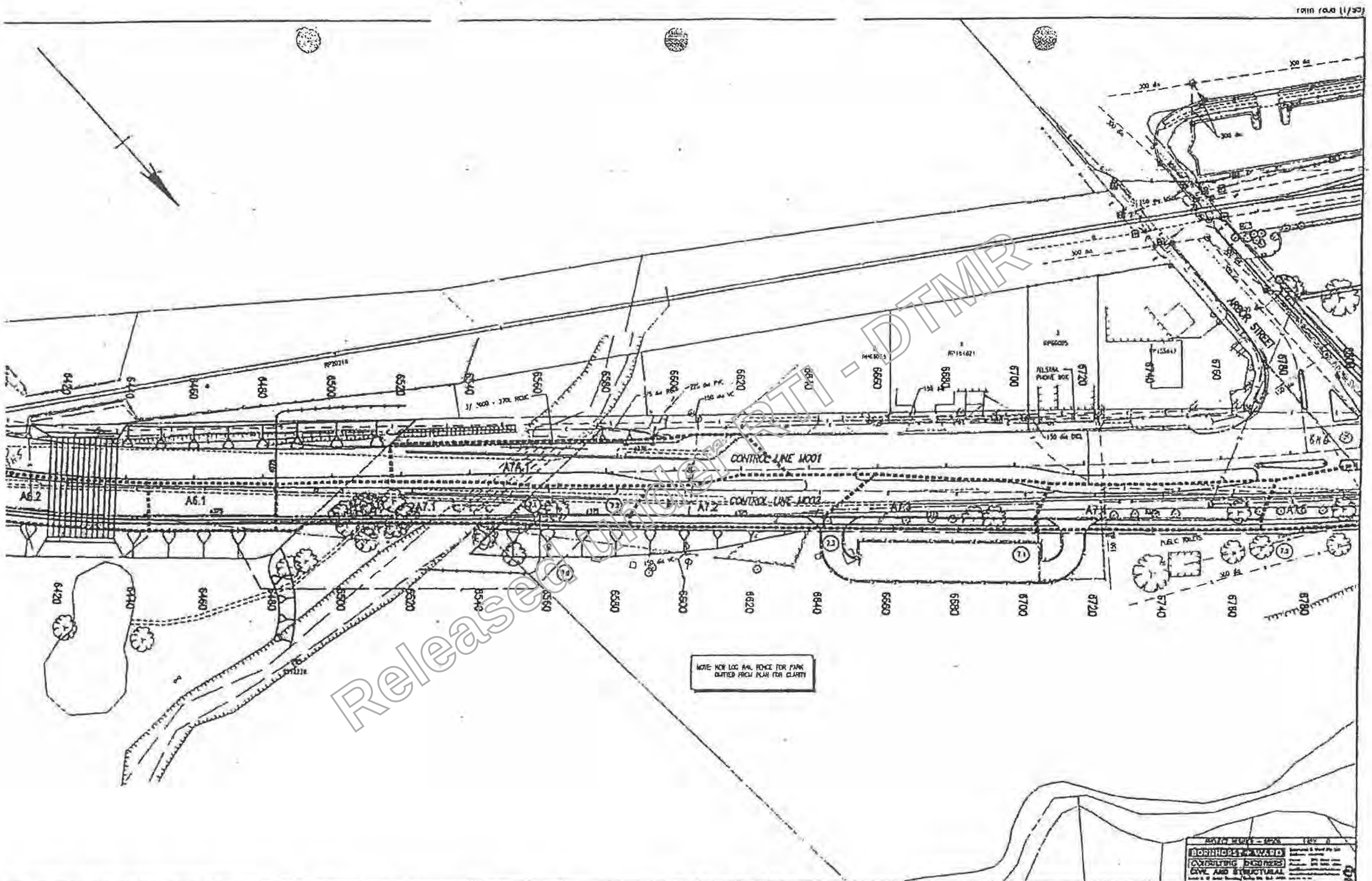

SETOUT TABLE
CONTROL LINE M006
DESCRIPTION
CHAINAGE
EASTING
NORTHING
RADIUS
CONTROL LINE M001
CONTROL LINE M002
CONTROL LINE M006
CLAVERTON STREET
A5.6
A5.4
A5.3A
A5.3.1
BH3+4
BRISBANE CITY
SAMFORD SUB-ARTERIAL
COBALT ST TO FERNY WAY
PLANNING AND PRELIMINARY DESIGN
EXISTING FEATURES AND SERVICES.
DRAINAGE AND ALIGNMENT – SHEET 4
Main Roads
CONTRACT No.
Dimensions in metres except where shown otherwise. Culvert sizes in millimetres
A1 Unreduced
A3 Reduced
Survey books
U95-179 TO
U95-185
Through chainage from
4.89Km – 7.15Km
Ancillary Plan Nos
313853 – 313878
R13-867 – R13-871
Scales
Unreduced / Reduced
1:500 / 1:1000
Reference Points
Preceding RPC
Following RPC
9
0.09
2.26
0.0
10
Job No.
140/U95/48
Plan No.
313858
Not Relevant
Not Relevant



SOURCE: MAIN ROADS - BRISBANE CITY SAMFORD SUB-ARTERIAL
COBALT ST TO FERNY WAY
PLAN No's 313854, 313864 & 313874

BEW215-W-DO 005 Rev A
July 2003

Figure B.1
PLANNING AND PRELIMINARY DESIGN OF SAMFORD ROAD NEAR RAILWAY CROSSING



Dimensions in metres except where shown otherwise. Culvert sizes in millimetres	A1 Unreduced A3 Reduced	Survey books U95–179 TO U95–185	
Ancillary Plan Nos. 313853 – 313878 R13–867 – R13–871	Scales	Unreduced / Reduced 1:500 / 1:1000	Through chainage from 4.89Km – 7.15Km

BRISBANE CITY
SAMFORD SUB–ARTERIAL
COBALT ST TO FERNY WAY

Preceding RPC	Dist to start of job (km)	Dist from start to end of job (km)	Dist from end to following RPC (km)	Following RPC
9	0.09	2.26	0.0	10

PLANNING AND PRELIMINARY DESIGN
EXISTING FEATURES AND SERVICES, DRAINAGE AND ALIGNMENT – SHEET 5

Not Relevant Not Relevan

Job No. 140/U95/48

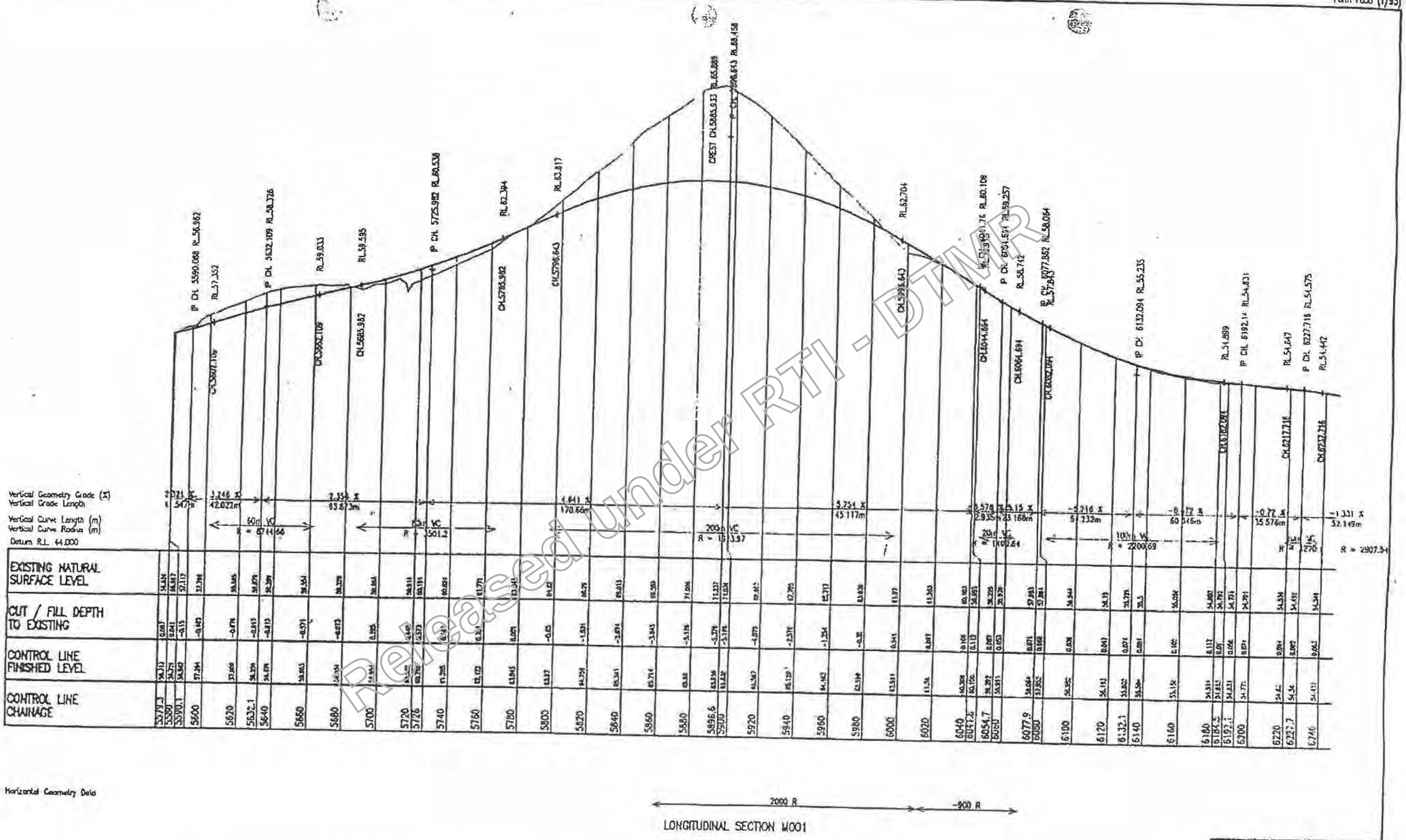
Main Roads

CONTRACT No. –

No. 7 of 31 plans

Plan No. 313859

Form F808 (1/95)



Revisions	Certified	Date	Verified	Associated Job Nos	Dimensions in metres except where shown otherwise. Culvert sizes in millimetres.	A1 Unreduced A3 Reduced	Survey books U95-179 TO U95-185
				Auxiliary Plan Nos 313853 - 313878 R13-867 - R13-471	Scales HORZ VERT	Unreduced / Reduced 1:1000 / 1:2000 1:100 / 1:200	Through chainage from 4.80Km - 7.15Km

BRISBANE CITY
SAMFORD SUB-ARTERIAL
COBALT ST TO FERNY WAY

Preceding RPC	Dist to start of job (km)	Dist from start to end of job (km)	Dist from end to following RPC (km)	Following RPC
9	0.09	2.26	0.0	10

PLANNING AND PRELIMINARY DESIGN
CONTROL LINE M001
LONGITUDINAL SECTION SHEET 2

Not Relevant | Not Relevant

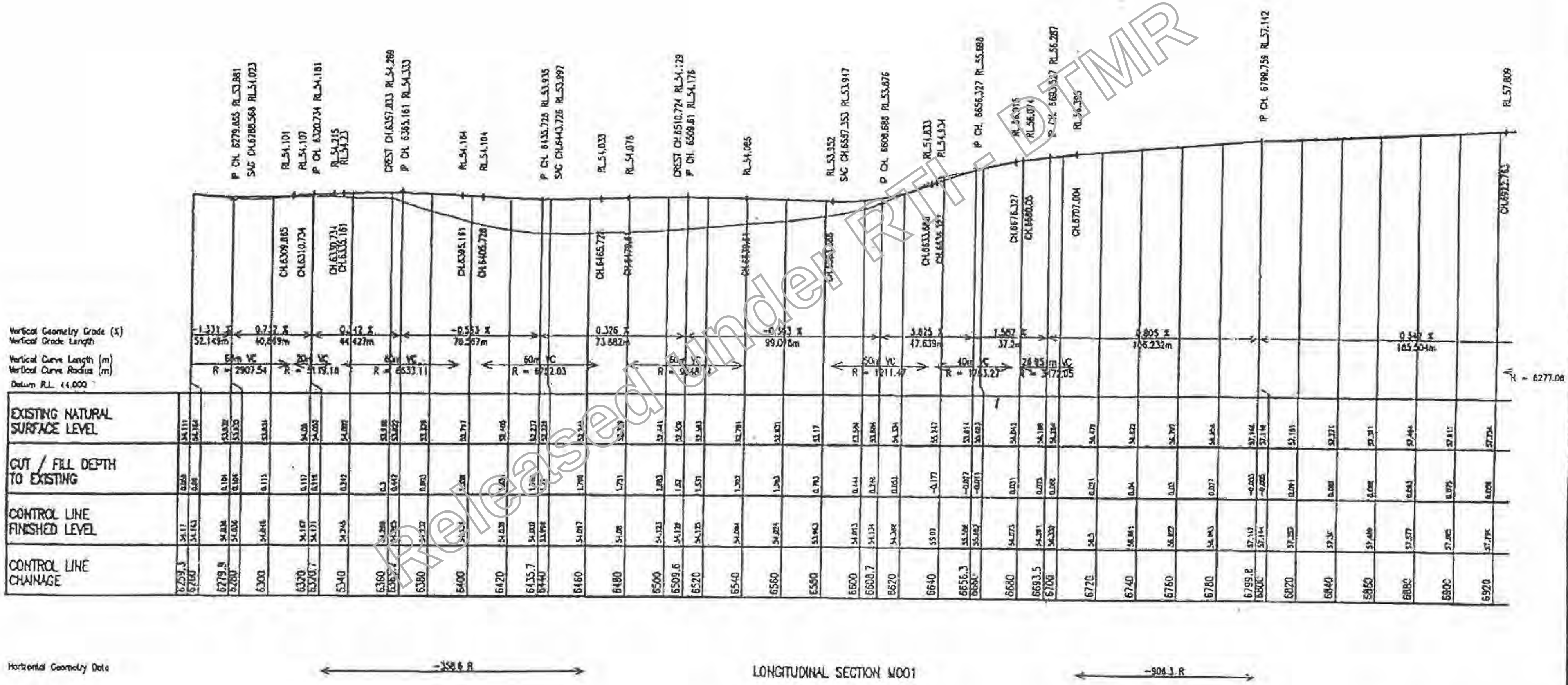
Job No. 110/U95/48

Main Roads

CONTRACT No.

No 16 of 31 plans
Plan No. 313868

Form F808 (1/85)



Revisions	Certified	Date	Microfilmed

Associated Job Nos: –
Ancillary Plan Nos: 313853 – 313876, R13-667 – R13-871
Dimensions in metres except where shown otherwise. Culvert sizes in millimetres.
A1 Unreduced / A3 Reduced
Scales: HORZ 1:500 / 1:1000; VERT 1:100 / 1:200
Survey books: U95-178 TO U95-185
Through chainage from 4.89Km – 7.15Km

BRISBANE CITY
SAMFORD SUB-ARTERIAL
COBALT ST TO FERNY WAY

Preceding RPC	Dist to start of job (km)	Dist from start to end of job (km)	Dist from end to following RPC (km)	Following RPC
9	0.09	7.25	0.0	10

PLANNING AND PRELIMINARY DESIGN
CONTROL LINE M001
LONGITUDINAL SECTION – SHEET 3

Not Relevant
Not Relevant

Job No. 140/U95/48

BORNHORST + WARD — CONSULTING ENGINEERS, CIVIL AND STRUCTURAL

Main Roads

CONTRACT No. —
No. 17 of 31 plans
Plan No. 313869

Appendix C

LOCAL DRAINAGE— HYDROLOGICAL CALCULATIONS

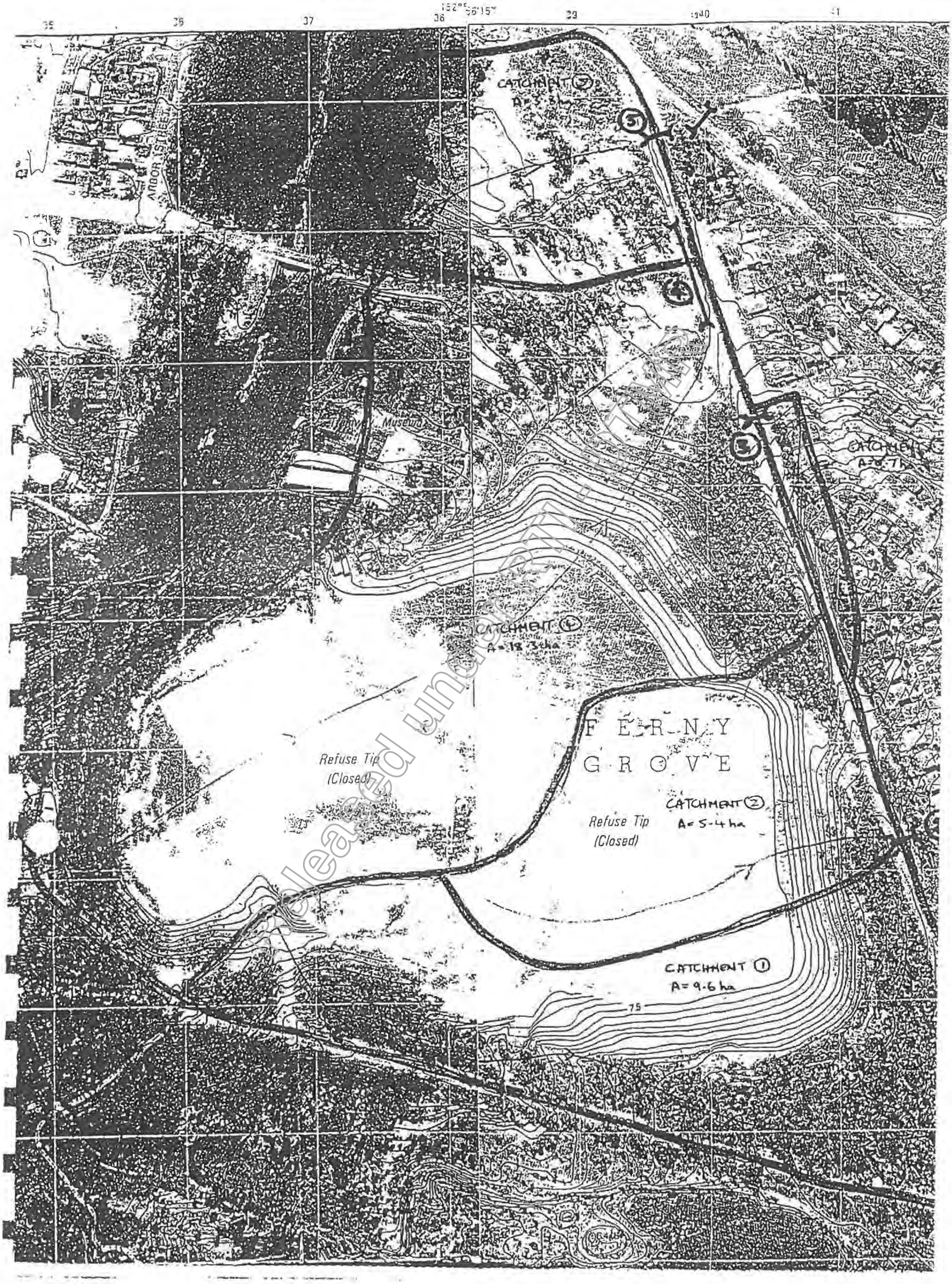
Summary of Rational Method Calculations

Catchment	Area (ha)	C	Time of Concentration			I (mm/hr)	Q50
			Length (m)	Av Velocity (m/s)	tc (min)		
1	9.6	0.65	650	0.55	20	166	2.9
2	5.38	0.65	375	0.55	11	215	2.1
3	0.65	0.65	210	0.55	6	270	0.3
4	18.95	0.65	710	0.55	22	159	5.4
5	22.75	0.65	850	0.55	26	147	6.0

Notes:

1. *Catchment number relates to culvert number (ie. Catchment 3 drains through Culvert 3)*
2. *Area of Catchment 4 includes Catchment 3*
3. *Area of Catchment 5 includes Catchments 3 and 4*

EDITION 1



CATCHMENT ④
A = 18.3 ha
F E R N Y
G R O V E
Refuse Tip
(Closed)
Refuse Tip
(Closed)
CATCHMENT ②
A = 5.4 ha
CATCHMENT ①
A = 9.6 ha
75
Museum

Appendix D

LOCAL DRAINAGE—
HYDRAULIC CALCULATIONS

Summary of Culvert Modelling Input

Existing Case

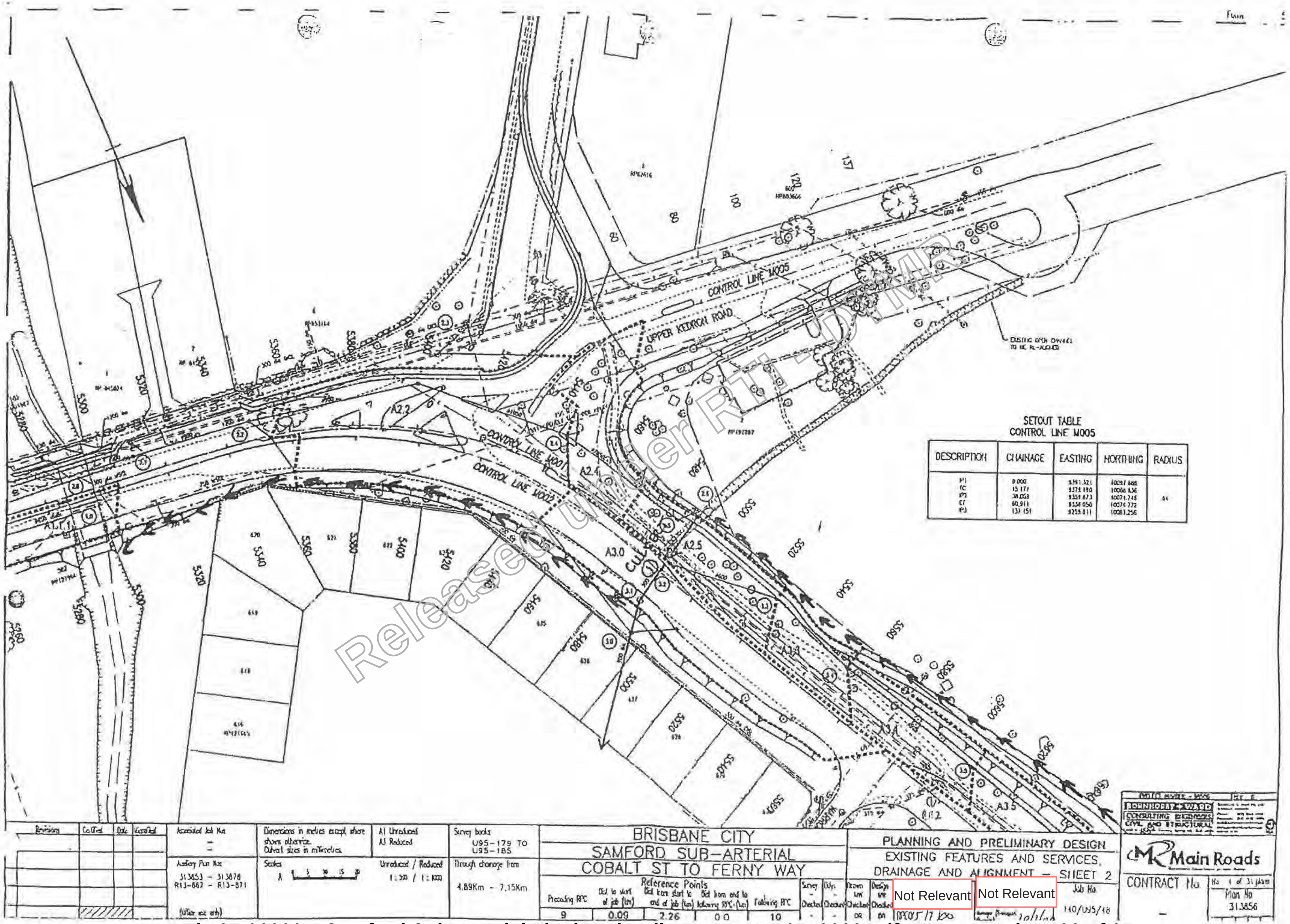
Culvert	Arrangement	ke	C	Length	Slope	n	IL	Road Level	Permissible HWL	TWL	Control	Qmax	Q50 local	Q50 total	Excess Q	Capacity	Manage excess
1	1/900 RCP	0.5	0.8	35	0.001	0.012	54.00	55.82	55.67	54.10	Outlet	1.65	2.88	4.41	2.76	Limiting	Upgrade culvert
2	1/600 RCP	0.5	0.8	25	0.001	0.012	59.00	60.27	60.12	59.10	Outlet	0.58	2.09	2.09	1.51	Limiting	Bypass channel
3	1/450 RCP	0.5	0.8	25	0.001	0.012	57.70	58.95	58.80	57.80	Outlet	0.34	0.32	0.32	-	Suitable	-
4	2/1200x600 RCBC	0.5	0.8	85	0.001	0.012	54.00	55.11	54.96	54.60	Outlet	2.00	5.44	5.44	3.44	Limiting	Upgrade culvert
5 - Road	2/1200 RCP	0.5	0.8	25	0.001	0.012	52.10	54.04	53.89	53.30	Outlet	5.20	6.04	4.10	0.84	Not limiting	Bypass channel
5 - Rail	3/825 RCP	0.5	0.8	12	0.001	0.012	52.00	53.94	53.79	53.20	Outlet	4.10	6.04	4.10	1.94	Limiting	-

Upgraded Culverts

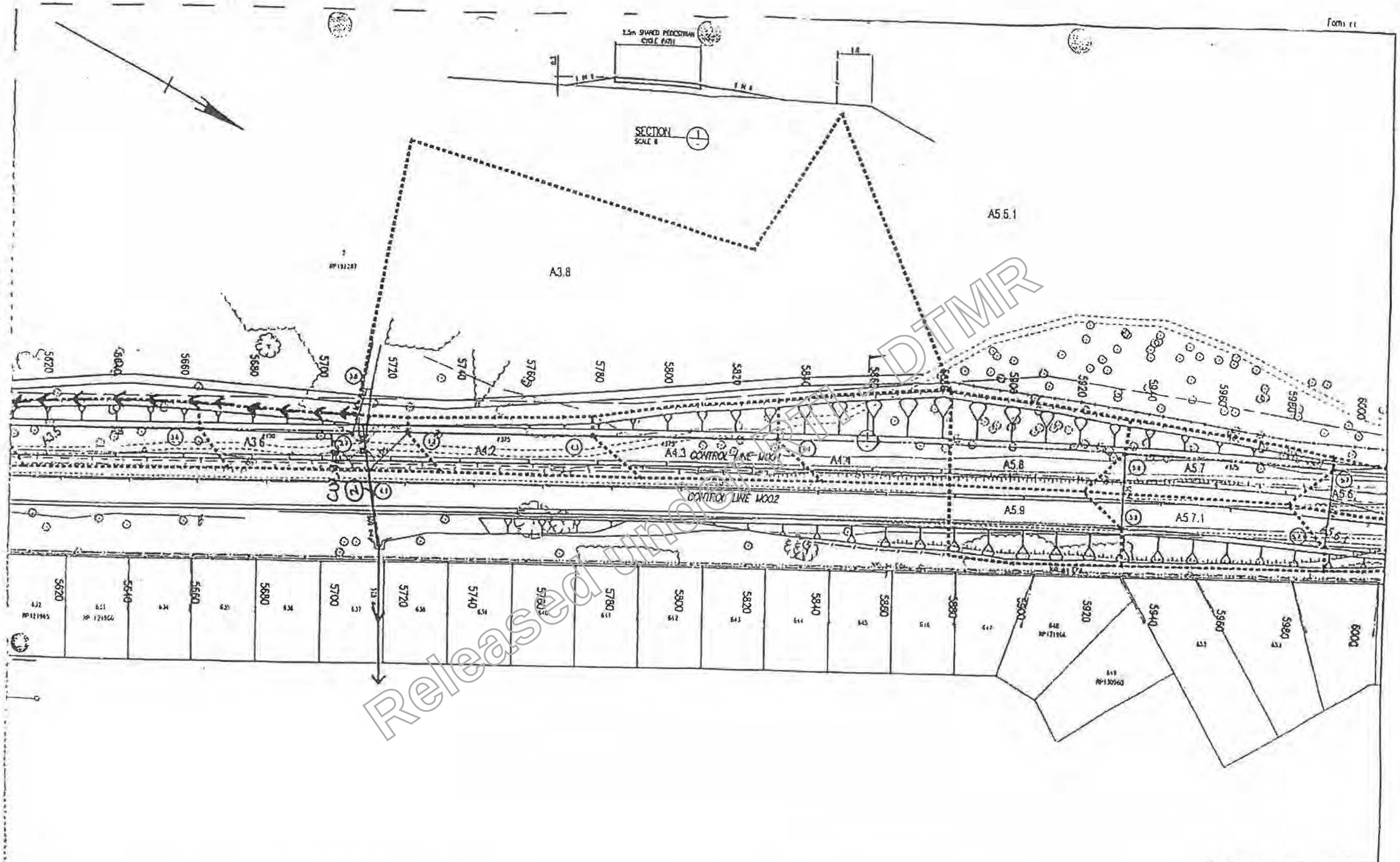
Culvert	Arrangement	ke	C	Length	Slope	n	IL	Road Level	Permissible HWL	TWL	Control	Q50 total	HWL
1	3/900 RCPs	0.5	0.8	35	0.001	0.012	54.00	55.82	55.67	54.10	Outlet	4.41	55.46
4	4/1200x600 RCBC	0.5	0.8	85	0.001	0.012	54.00	55.11	54.96	54.60	Outlet	5.44	54.79

Bypass Channels

From Culvert	Qbypass	Slope	n	Base	Batter	Depth	V
2	1.51	2.3%	0.035	0.8	1 in 6	0.35	1.49
5	1.90	1.3%	0.035	0.7	1 in 6	0.45	1.28



CONTROL LINE M005
UPPER KEDRON ROAD
CONTROL LINE M001
CONTROL LINE M002
EXISTING OPEN DRAIN TO BE RE-ALIGNED
SETOUT TABLE
CONTROL LINE M005
DESCRIPTION
CHAINAGE
EASTING
NORTHING
RADIUS
BRISBANE CITY
SAMFORD SUB-ARTERIAL
COBALT ST TO FERNY WAY
PLANNING AND PRELIMINARY DESIGN
EXISTING FEATURES AND SERVICES,
DRAINAGE AND ALIGNMENT – SHEET 2
Main Roads
CONTRACT No
Survey books
U95–179 TO
U95–185
Through chainage from
4.89Km – 7.15Km
Reference Points
Not Relevant
Not Relevant



Revisions	Certified	Date	Verified

Associated Job Nos –

Auxiliary Plan Nos 313853 – 313878 R13-867 – R13-871

(Office use only)

Dimensions in metres except where shown otherwise. Culvert sizes in millimetres

Scales

AI Unreduced AI Reduced

Unreduced / Reduced 1:500 / 1:1000

Survey books U95-179 TO U95-185

Through chainage from 4.89Km – 7.15Km

BRISBANE CITY

SAMFORD SUB-ARTERIAL

COBALT ST TO FERNY WAY

Reference Points

PLANNING AND PRELIMINARY DESIGN

EXISTING FEATURES AND SERVICES, DRAINAGE AND ALIGNMENT – SHEET 3

Not Relevant Not Relevant

Job No

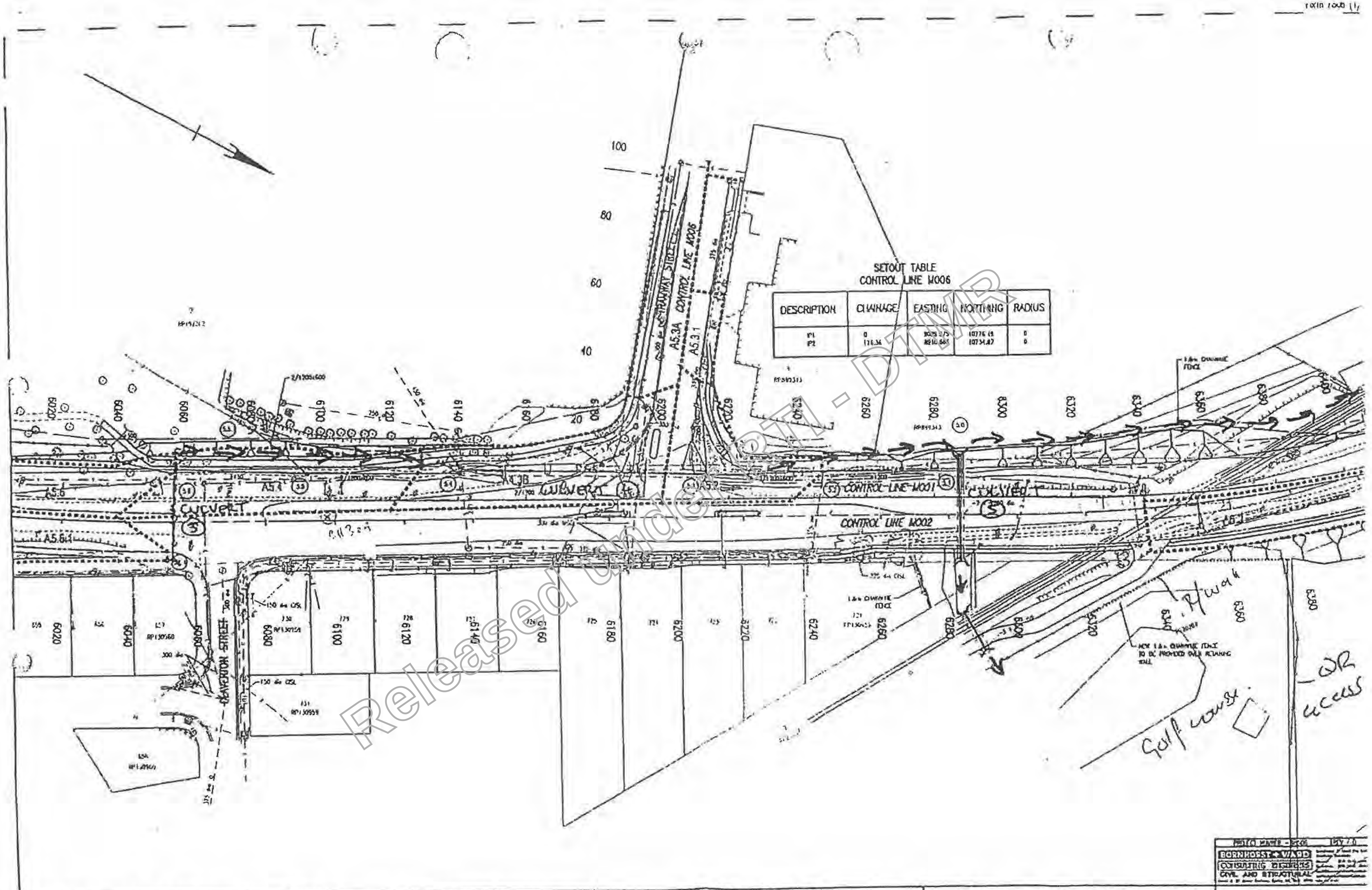
BORNHORST + WARD CONSULTING ENGINEERS CIVIL AND STRUCTURAL

Main Roads

CONTRACT No.

No. 5 of 31 plans

Plan No. 313857



Revisions	Certified	Date	Verified	Associated Job Nos	Dimensions in metres except where shown otherwise. Culvert sizes in millimetres.	A1 Unreduced A3 Reduced	Survey books U95-179 TO U95-185
				Auxiliary Plan Nos 313853 - 313878 R13-867 - R13-871	Scales	Unreduced / Reduced 1:500 / 1:1000	Through chainage from 4.89Km - 7.15Km

BRISBANE CITY

SAMFORD SUB-ARTERIAL

COBALT ST TO FERNY WAY

Reference Points

Preceding RPC	Dist to start of job (km)	Dist from start to end of job (km)	Dist from end to following RPC (km)	Following RPC
9	0.09	2.26	0.0	10

PLANNING AND PRELIMINARY DESIGN

EXISTING FEATURES AND SERVICES, DRAINAGE AND ALIGNMENT — SHEET 4

Survey Checked | Bdys Checked | Drawn Checked | Design Checked | Not Relevant | Not Relevant | Job No 110/U95/48

Main Roads

CONTRACT No —

Plan No. 313858