CURRAMBENE CREEK
AND
MOONA MOONA CREEK

FLOOD STUDIES

VOLUME 2 – APPENDICES

November 2006

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APPENDIX A

ANALYSIS OF HISTORIC FLOODS AND HYDROLOGIC MODEL CALIBRATION

November 2006

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1. INTRODUCTION AND SCOPE

This Appendix deals with the selection of historic floods for the calibration of the RORB hydrologic model of the Currambene Creek catchment, the analysis of rainfall and runoff data for those events and the calibration process.

A stream gauging station has been in operation on Currambene Creek at The Falls located on the western side of the Princes Highway since 1969. The catchment area upstream of the gauging station is 95 km^2. A frequency analysis of flood peaks recorded at this station was undertaken and the results are discussed in **Section 2**.

A pluviometer at the RAN Air Station, HMAS Albatross (**Figure 3.1**), records the depth and temporal pattern of rainfall at that site and there are also several daily read rainfall gauges adjacent to the catchment. As discussed later, these data were used to assess the temporal and areal distribution of rainfall over the catchment for each flood event analysed.

Based on the criteria of magnitude of peak discharge and availability of data, three historic flood events were selected for analysis and model calibration. Peak flows and times to peak of these events are shown on **Table 1.1**. Two other events, which occurred in June1990 and March 1974 and which ranked 3 and 4 in the period of record, were also considered for calibration. However, no pluviographic data at HMAS Albatross were available, so the temporal pattern of rainfall could not be identified for those floods.

TABLE 1.1
HISTORIC FLOOD SELECTED FOR ANALYSIS

Date	Rank	Peak Discharge m^3/s	Time of Peak
6 February 1971	1	713	0648 hrs
11 March 1975	2	443	0437 hrs
16 October 1976	5	328	1757 hrs

Note: Flood data applies to the Currambene Creek stream gauging station at The Falls (Stn 216004).

The procedure for the calibration of the RORB model initially involved the collection and analysis of rainfall data to ascertain the temporal and areal distribution of rainfall, as well as the separation of baseflow from the recorded flows to estimate surface runoff hydrographs for each event. This step allowed the determination of rainfall losses and generation of hyetographs of rainfall excess.

The parameters of the RORB model were then adjusted to achieve correspondence between the computed and recorded hydrographs. Model parameters comprised the storage delay parameter (kc) and the non-linear routing parameter (m). This model calibration process is discussed in **Section 3**. **Appendix B** provides further information on the RORB rainfall-runoff modelling approach.

In addition to the model fitting described above, the RORB model was also “calibrated” against the frequency curve of annual flood peaks recorded at The Falls. The procedure involved the derivation of kc by matching modelled peak flows due to design hyetographs derived from Australian Rainfall and Runoff (ARR, 2001), with peak flows of the same return period obtained from the frequency analysis. The procedure is outlined in **Section 4**.

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2. FLOOD FREQUENCY ANALYSIS

2.1 Introductory Remarks

Flood frequency analysis was carried out using the annual series and partial series methods. With the annual series approach, the flood record comprised the highest instantaneous discharge in each year of the period of analysis. The advantage of using an annual series is that the selected floods are more likely to be independent of one another as only one event is chosen per year. In addition, the form of the frequency distribution of annual floods usually conforms with many theoretical distributions and hence statistical theory is readily applicable. The disadvantage is that non-floods in dry years may have an undue influence on skewness and give a fitted distribution which does not agree with the plotted flood peaks in the high range.

For the annual series analyses, the ranked flood peaks were plotted on log-normal probability paper according to equation 2.1 and the log Pearson III distribution was fitted to the plotted points.

$$PP(m) = \frac{m - 0.4}{N + 0.2} \quad \ldots\ldots\ldots\ldots \quad 2.1$$

where PP(m) is the annual exceedance probability (AEP)

m = the rank of the flood

N = the number of years of record

The partial duration series comprises all floods with peak discharge above a selected base value, regardless of the number of floods occurring in each year, provided they are independent. Investigations have found (ARR, 2001) that the best results are obtained when the number of floods included in the analysis equals the number of years of record. The advantage of the partial series is that when the base value is high, small events which are not really floods are excluded.

Partial series floods were plotted on log-linear graph paper according to the plotting formula of equation 2.1 inverted to allow estimation of frequency in terms of average recurrence interval (ARI). These points were used to draw a smooth curve or empirical probability distribution through the plotted points.

2.2 Annual Series Analysis

The ranked flood data is presented in **Table 2.1** and the results are plotted on **Figure 2.1**. **Table 2.2** shows the statistics of the distribution and the estimated peak discharges for return periods up to 1 in 200 years.

2.3 Partial Duration Series

A partial duration series was prepared using the floods from the period of record. A least squares line of best fit was applied to the plot and is shown on **Figure 2.2**. This relationship gave estimates of flood peaks which were considerably higher than the annual series estimates and is influenced by the large number of small flood events. As shown on **Figure 2.2** and **Table 2.2**, a line of best fit to the higher flood peaks gave a better agreement with the annual series results. In the case of the 100 year ARI, the peak discharge estimated from the annual series approach was 757 m^3/s, compared with the partial series estimate of 740 m^3/s.

TABLE 2.1
ANNUAL SERIES FLOOD DATA
AT THE FALLS GAUGING STATION

Rank	Year	Peak Flow m^3/s
1	1971	713
2	1975	443
3	1990	365
4	1974	338
5	1976	328
6	1989	274
7	1991	257
8	1978	242
9	1998	238
10	1977	224
11	1984	144
12	1988	123
13	1992	87
14	1999	77
15	1985	67
16	1987	64
17	1997	63
18	1973	45
19	1994	43
20	1995	38
21	2003	32
22	1979	32
23	2002	32
24	1970	31
25	1986	31
26	1972	30
27	1996	29
28	1983	28
29	1993	20
30	1981	13
31	1982	60
32	2001	5
33	2000	3
34	1980	0

TABLE 2.2
SUMMARY OF FREQUENCY ANALYSIS
AT THE FALLS

Analysis	Statistics			Peak Discharges m^3/s				
	Mean	Standard Deviation	Skew	5yr	20yr	50yr	100yr	200yr
Annual Series (1970 – 2003) Log Pearson Type III	1.75	0.683	– 0.914	216	460	638	757	868
Partial Series (1970 – 2003) Least Squares Fit. All data used	N/A	N/A	N/A	280	550	720	840	970
Partial Series (1970 – 2003) Fit to high flood flows	N/A	N/A	N/A	220	480	630	740	840

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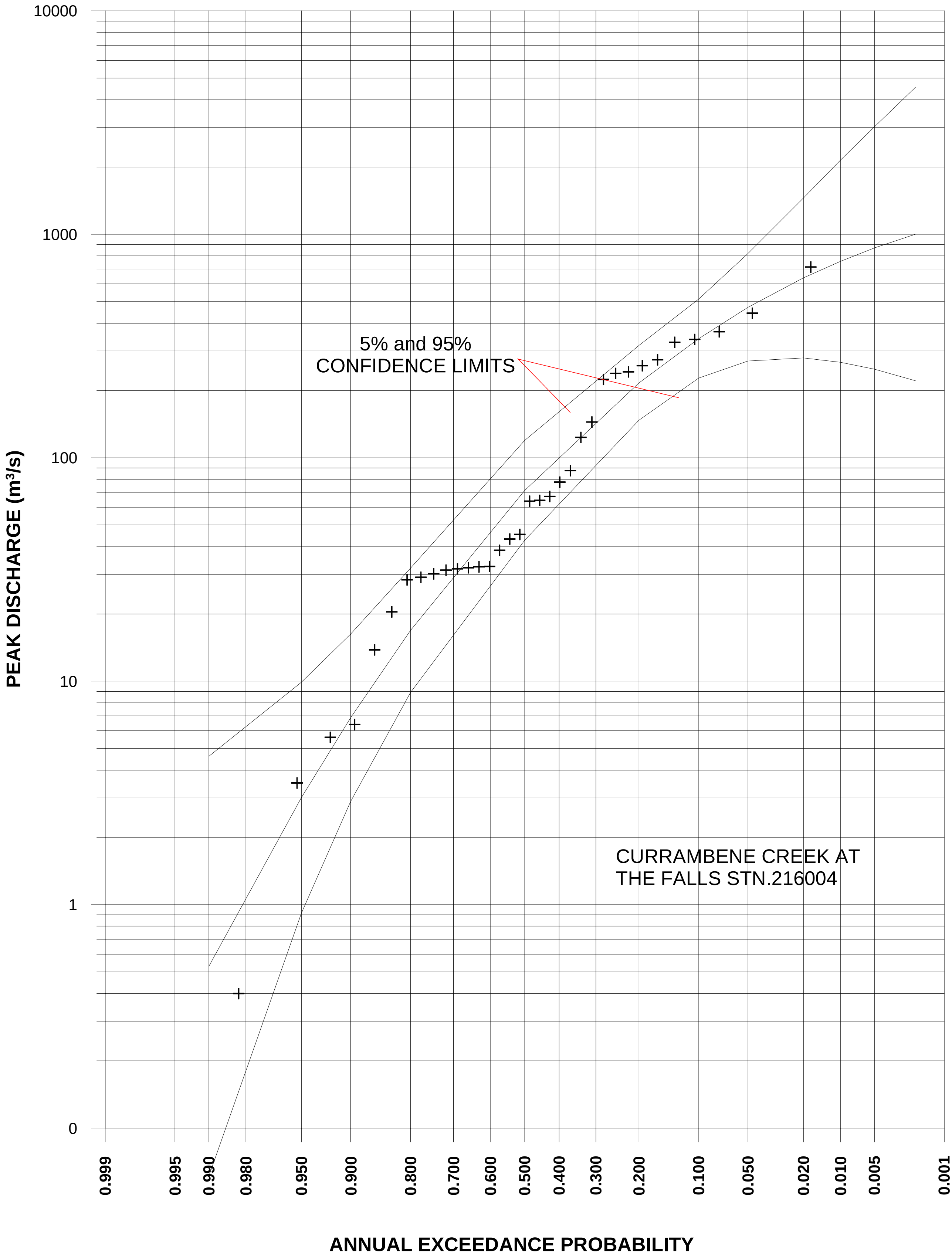
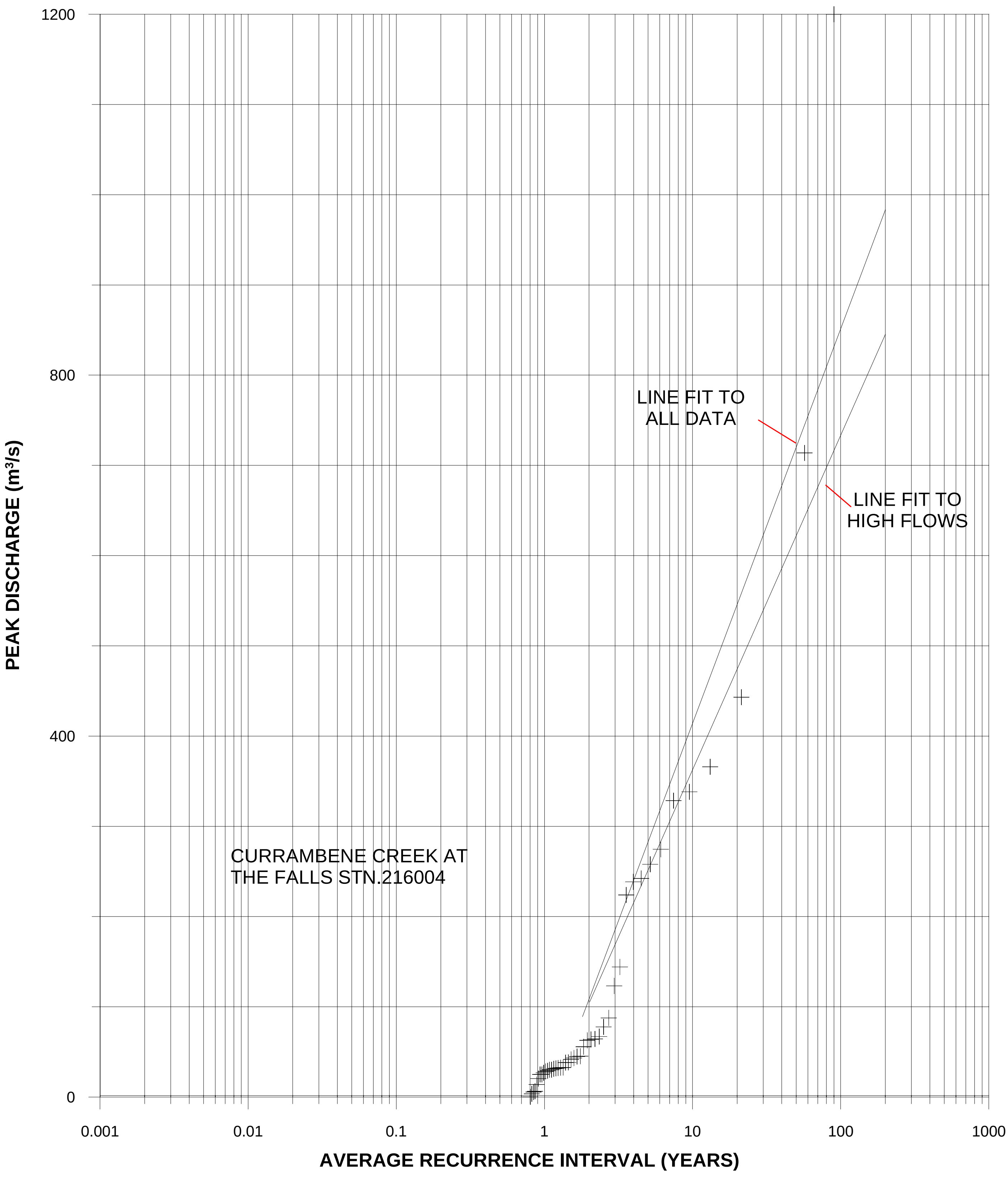


CURRAMBENE CREEK AND MOONA MOONA CREEK FLOOD STUDIES

Figure 2.1

CURRAMBENE CREEK FLOOD FREQUENCY RELATIONSHIP
ANNUAL SERIES 1970-2003



CURRAMBENE CREEK AND MOONA MOONA CREEK FLOOD STUDIES

Figure 2.2

CURRAMBENE CREEK FLOOD FREQUENCY RELATIONSHIP
PARTIAL DURATION SERIES 1970-2003

3. ANALYSIS OF HISTORIC STORMS AND MODEL CALIBRATION

3.1 March 1971 Flood

Heavy rainfall was experienced in the Shoalhaven area over the four day period from 4th to 8th February 1971. **Table 3.1** shows recorded daily falls over the period 0900 hours to 0900 hours at a number of Bureau of Meteorology (BOM) gauges. At the RAN Air Station, the heaviest daily fall was experienced over the 24 hour period ending at 0900 hours on 6th February when 179.8 mm was recorded. As reported in the previous flood study for Currambene Creek (BLA, 1983), a daily fall of 292 mm was recorded at the Nowra Bowling Club. However, this appears to be an unofficial gauge and was not used in the analysis.

It is also interesting to note that in the previous investigation (BLA, 1983), the total rainfall for the 6th February rainday as accumulated from the pluviograph chart was 240 mm, compared with 179.8 mm from the digitised records supplied by BOM for this present study. Unfortunately, the original pluviograph charts were not available for this present investigation and consequently, the reason for the difference in rainfalls could not be investigated further. It is possible that the 179.8 mm as digitised by BOM may be an underestimate of the depth of rainfall actually experienced on 6th February.

At Greenwell Point, the rainfall recorded for the 6th February was 188 mm, reducing to 135 mm at Hyams Beach on the coast to the south of Huskisson. Lesser falls were recorded inland, with 134.9 mm falling at Sassafras and 74.9 mm experienced at Yalwal. There were insufficient rainfall sites available in the vicinity to define the areal distribution of daily rainfalls over the Currambene Creek catchment by isohyetal mapping. Accordingly, the rainfall-runoff model calibration has been based on the assumption of a uniform areal distribution of rainfall over the catchment upstream of The Falls, using the RAN Air Station pluviographic record.

Currambene Creek at The Falls gauging station commenced to rise at 0900 hours on 4th February and peaked at 0648 hours on 6th February (**Figure 3.2**). The peak recorded discharge was 713 m^3/s. At the RAN Air Station, average rainfall intensities, as digitised by BOM over the most intense six to eight hours long burst, had a return period of around 1 in 20 years. However, by this time, the catchment had been subjected to considerable prior rainfall, with 82 mm being experienced at the RAN Air Station on the preceding two raindays. According to the frequency analysis, a discharge equal to the recorded peak would have a return period between 1 in 50 and 1 in 100 years. The wetness of the catchment could explain this mismatch in return periods of the recorded rainfall intensities and peak flow.

TABLE 3.1
DAILY RAINFALL DATA
FEBRUARY, 1971 FLOOD
VALUES IN mm

<table>
<tr><th>Date</th><th>Nowra Air Station
068076</th><th>Nowra Bowling Club
–</th><th>Greenwell Point
068080</th><th>Culburra
068083</th><th>Hyams Beach
068078</th><th>Jervis Bay Point Perpendicular
068034</th><th>Berkeley
068110</th><th>Jervis Bay State Forest</th><th>Nerriga
068085</th><th>Sassafras
068203</th></tr>
<tr><td>4</td><td>21</td><td>71</td><td>39</td><td>22</td><td>32</td><td>18.5</td><td>41</td><td>22</td><td>46</td><td rowspan="3">135~</td></tr>
<tr><td>5</td><td>61</td><td>98</td><td>77</td><td>46</td><td>46</td><td>30</td><td>29</td><td>66</td><td>22</td></tr>
<tr><td>6</td><td>180[#]</td><td>292[x]</td><td>188</td><td>89</td><td>119</td><td>42</td><td>79</td><td>75</td><td>9</td></tr>
<tr><td>7</td><td>55</td><td>44</td><td rowspan="2">78.7*</td><td>107</td><td>135</td><td>105</td><td>18</td><td>142</td><td>32</td><td>43</td></tr>
<tr><td>8</td><td>2</td><td>2</td><td>2</td><td>2</td><td>2</td><td>0</td><td>2</td><td>0</td><td>0</td></tr>
</table>

* Accumulated 2 day rainfall
\# Value from pluviographic chart – 240 mm (ref. BLA, 1983)
~ Accumulated 3 day total
x This depth appears on the high side and was not used in the analysis. The rain gauge is located at an unofficial site.

3.2 March 1975 Flood

Heavy rainfall was experienced over the two rain days 10th and 11th March 1975. Maximum falls were experienced in coastal areas to the south of Huskisson on 10th March with 231 mm recorded at Sussex Inlet. At RAN Air Station, a lesser fall (93 mm) was recorded. By comparison, the 206 mm fall recorded at Nowra Bowling Club on 10th March (BLA, 1983) appears on the high side and was not used in the investigation.

Rainfalls were heaviest on the Currambene Creek catchment on the 11th March rain day with recorded falls of 173 mm at RAN Air Station and 157 mm at the Nowra Bowling Club.

Currambene Creek commenced to rise at 1600 hours on 9th March and the peak of 443 m^3/s occurred at 0437 hours on 11th March (**Figure 3.3**). At the RAN Air Station, the most intense 6 to 8 hour burst of rainfall had a return period in the range 1 in 5 to 10 years ARI. According to the frequency analysis of annual flood peaks, a discharge equal to 443 m^3/s would have a return period of about 1 in 20 years. Similar to the 1971 event, the prior wetness of the catchment would appear to be responsible for the mismatch between return periods of the intense burst of rainfall and resulting flood peak.

For this flood, the rainfall-runoff model calibration was based on the assumption of a uniform areal distribution of rainfall upstream of The Falls.

TABLE 3.2
DAILY RAINFALL DATA
MARCH, 1975 FLOOD

Date	Nowra RAN Air Station 068076	Nowra Bowling Club –	Greenwell Point 068080	Culburra 068083	Sussex Inlet 068204	Jervis Bay Point Perpendicular 068034	Jervis Bay State Forest	Nerriga 068085	Yalwal 068082
8	0	0	0	0	3	11	X	0	0
9	5	3	0	1	4	5	X	1	1
10	93	206[x]	140	115	231	106	212*	10	66
11	173	157	95	79	128	117	90	56	178
12	13	21	4	2	3	2	X	2	13
13	28	32	109	94	36	46	71#	4	24

\# Accumulated 2 day total
* Accumulated 3 day total
x This depth appears on the high side and was not used in the analysis. The rain gauge is located at an unofficial site.

3.3 October 1976 Flood

Heavy rainfall was experienced over the three day period from 16th to 18th October 1976. The heaviest falls were experienced over the 24 hour period ending at 0900 hours on 17th October, when 157 mm was recorded at the RAN Air Station. Heavy falls were also recorded at Greenwell Point (127 mm) and at the Nowra Bowling Club (110 mm). The heavy falls extended inland from the coast with 135 mm recorded at Yalwal and 77 mm at Nerriga.

A focus of heavy rainfall appears to have been centred on coastal areas south of Huskisson on the previous day with 140 mm recorded at Jervis Bay Point Perpendicular and 156 mm at Sussex Inlet. Falls over the Currambene Creek catchment were lighter with 71 mm recorded at RAN Air Station.

As for the previous floods, the model calibration was based on the assumption of an areally uniform rainfall over The Falls catchment using the RAN Air Station pluviographic record.

Currambene Creek commenced to rise at 1800 hours on 15th October and reached a peak of 328 m^3/s at 1757 hours on 16th October (**Figure 3.4**). At the RAN Air Station, the most intense 6 to 8 hours burst of rainfall had a return period of around 1 in 10 years ARI, which agrees with an estimated return period of around the same magnitude for the resulting peak flow.

The apparent correspondence between return periods of rainfall and runoff for this particular flood is not consistent with the results derived for the two flood events analysed previously. It may be due to the fact that for this particular flood event, the catchment was not saturated by several days of rainfall prior to the occurrence of the rainfall burst responsible for the flood peak. Consequently, the initial loss was more characteristic of the median loss rate which produces a flood peak with a return period similar to that of the rainfall burst responsible for that peak.

TABLE 3.3
DAILY RAINFALL DATA
OCTOBER, 1976 FLOOD

Date	Nowra RAN Air Station 068076	Nowra Bowling Club [X] –	Greenwell Point 068080	Culburra 068083	Hyams Beach 068078	Jervis Bay Point Perpendicular 068034	Jervis Bay State Forest	Nerriga 068085	Yalwal 068082	Sussex Inlet 068204
14	38	14	17	24	NA	48	29	20	26	40
15	1	2	5	4		12	13	0	1	20
16	71	60	81	87		140	–	57	76	156
17	157	110	127	59		8	–	77	135	5
18	27	32	21	26		7	155	14	22	4
19	12	14	25	19		27		2	15	15

[X] This depth appears on the high side and was not used in the analysis. The rain gauge is located at an unofficial site.

3.4 Calibration Strategy

The storage delay time kc is the principal parameter of the RORB model. Decreasing kc increases the peak discharge and decreases the catchment lag, while increasing kc does the opposite.

Decreasing the value of the exponent m in the catchment storage-discharge equation slightly, as well as making the consequent change in the catchment lag factor kc recommended in the RORB manual, tends to delay the start of rise and also the tail of the recession but to advance the peak. Increasing m has the opposite effect. Minor variations in m can be useful in improving the fit.

The initial loss (IL) and continuing loss (CL) are other important parameters, but in calibration, their values may be obtained objectively from the data, as in the present case. Varying initial loss can be a useful means of achieving a fit. This parameter affects the start of the hydrograph rise and in long duration storms such as those examined in this study, changing initial loss and hence the calculated continuing loss rate could cause significant changes in hydrograph peak.

However, for the calibrations discussed herein, the time of the commencement of rise of the observed hydrograph was adopted as the time at which initial loss was satisfied. The RORB model run commenced at this time and no abstraction was made from the succeeding rainfall for initial loss, ie the implicit assumption was that the catchment was saturated by the prior rainfall.

Calibration of the RORB model for each of the historic floods was carried out according to the following general plan.

- It was accepted that a single set of kc and m values, covering the entire model, would be estimated for each flood. This is in accordance with the basic structure of RORB.

- For a particular historic flood, parameters were selected or adjusted as follows:

    - RORB was run in ‘FIT’ mode with IL equal to zero, and determined CL so as to balance the depth of rainfall excess with the observed volume of runoff.
    - kc and m were varied until the best match was achieved with the observed time and magnitude of the peak, but with greater weight given to matching the peak discharge.

Figure 3.1 shows the layout of the RORB model.

Calibration results are plotted on **Figures 3.2 to 3.4** and apply for the parameters shown on **Table 3.4**. These diagrams show the actual surface runoff hydrograph as derived from the recorded flows and after separation of baseflow, the modelled surface runoff hydrograph and the hourly rainfall excess hyetograph. These results apply for an m value of 0.8.

As stated in the RORB manual, a useful procedure for fitting the model to a catchment makes use of several events covering a wide range of peak discharges. For each event, a selected range of m values is used. For each m value, kc is varied until the best fit with that m value is achieved. A graph of kc versus m for that event is then plotted. When these graphs for the several events are superimposed, they often indicate a unique pair of kc and m values that provide a good fit for all events.

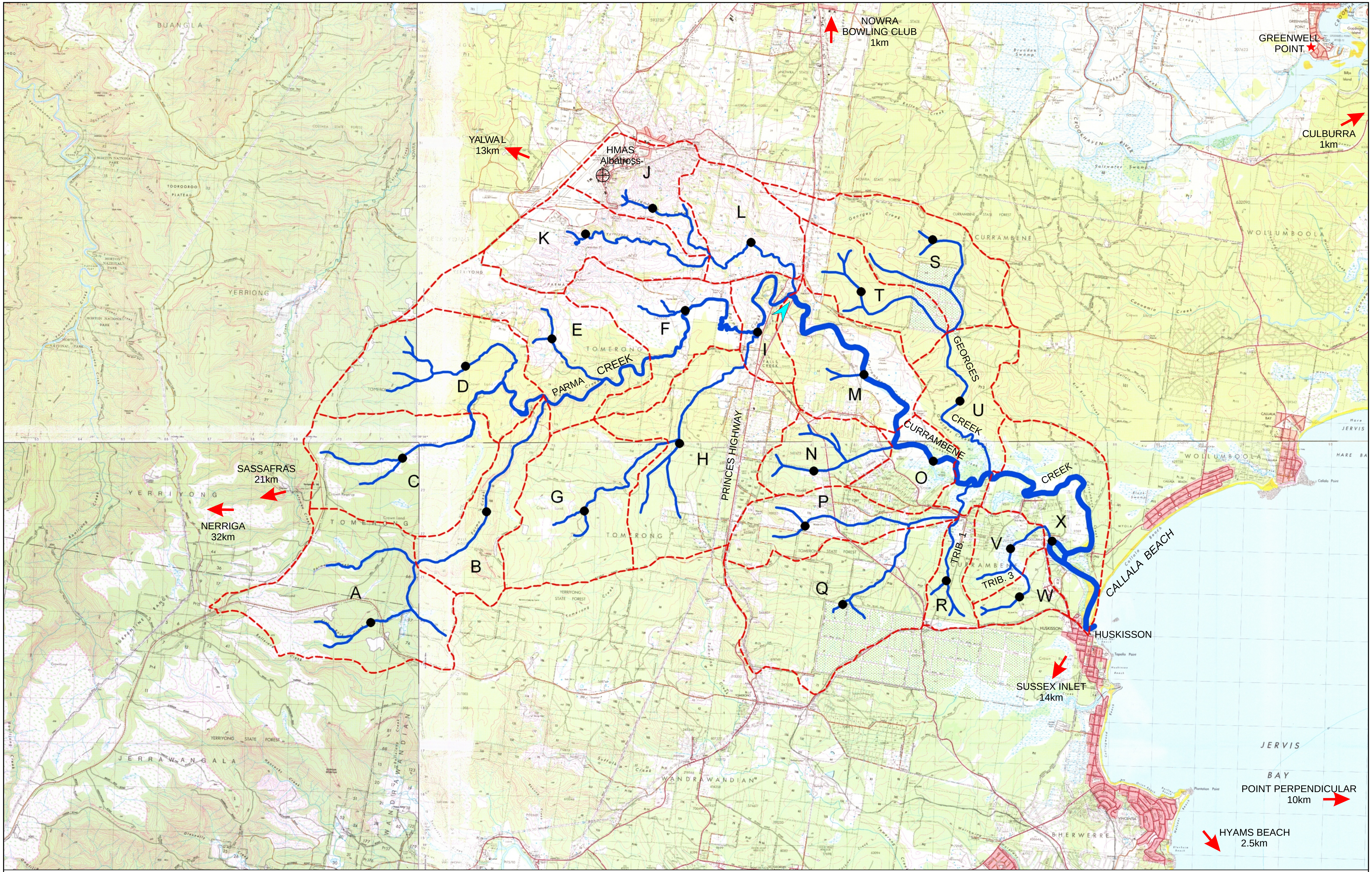
Figure 3.5 shows the interaction between kc and m for the three floods. Unfortunately the parameter interaction lines are parallel with one another and there is no evidence of an intersection, which would provide a unique pair of kc and m values giving a good fit for all events.

For each m value, the value of kc tends to reduce with increasing flood magnitude. For example, for an m value of 0.8, the kc value reduces from 12 for the October 1976 flood (rank 5) to 6.1 for the February 1971 flood (rank 1).

TABLE 3.4
RORB CALIBRATION RESULTS

Date	kc	m	IL mm	CL mm/h	Q m^3/s	tp hr
6 February 1971	6.1	0.8	0	1.9	678	– 2
11 March 1975	7.3	0.8	0	4.3	429	0
16 October 1976	12	0.8	0	3.5	332	0

Notes:
1) The values of “Q” shown on Table 3.4 are the flood peaks resulting from the transformation by the RORB model of the hyetograph of rainfall excess.
2) tp shows the difference between the modelled and observed flood peaks in hours. A negative value indicates that the modelled peak occurred before the observed peak.
3) Modelled Initial Loss (IL) is zero because the analysis commences at the time of rise of the hydrograph when initial loss is assumed satisfied. No further abstractions for further initial loss were made, only for Continuing Loss (CL).



N

0 2 4km

Scale

Currambene Creek at "The Falls" Stream gauging station. (STN 218004)

Pluviograph site

G Model sub-area and centroid

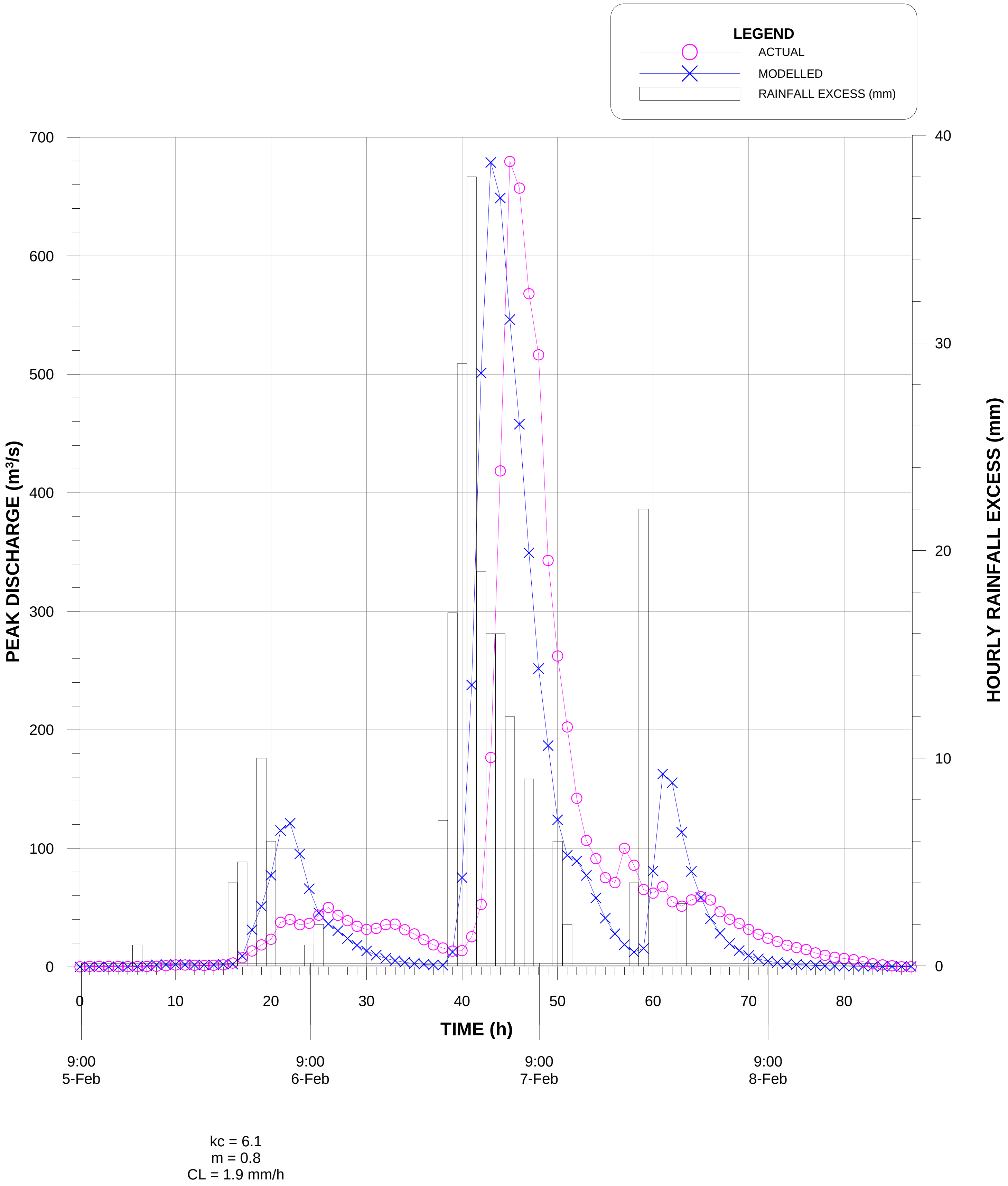
Daily Rainfall Station

YALWAL 13km Daily Rainfall Station and distance

CURRAMBENE CREEK AND MOONA MOONA CREEK FLOOD STUDIES

Figure 3.1

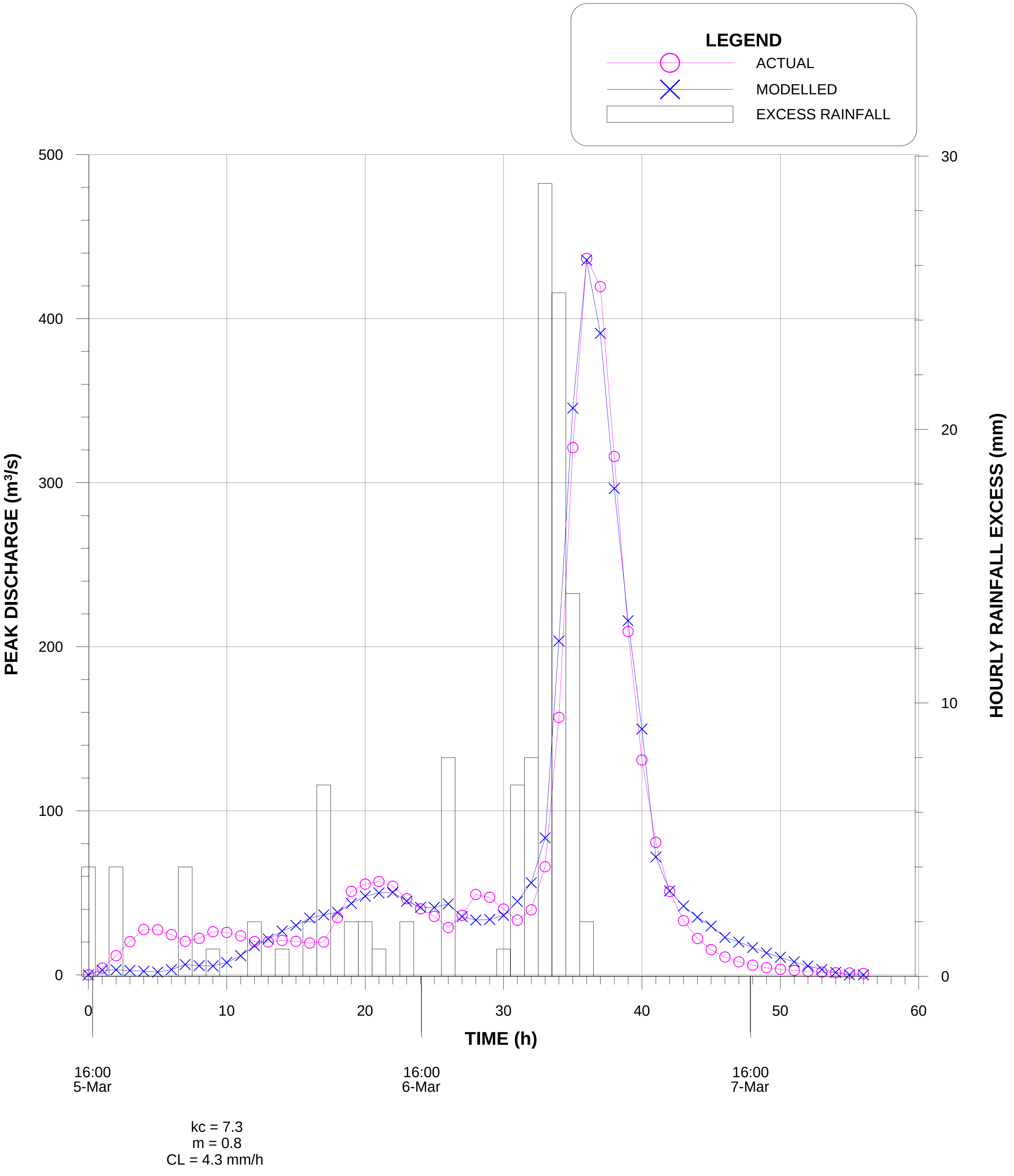
RORB MODEL LAYOUT
CURRAMBENE CREEK CATCHMENT



CURRAMBENE CREEK AND MOONA MOONA CREEK FLOOD STUDIES

Figure 3.2

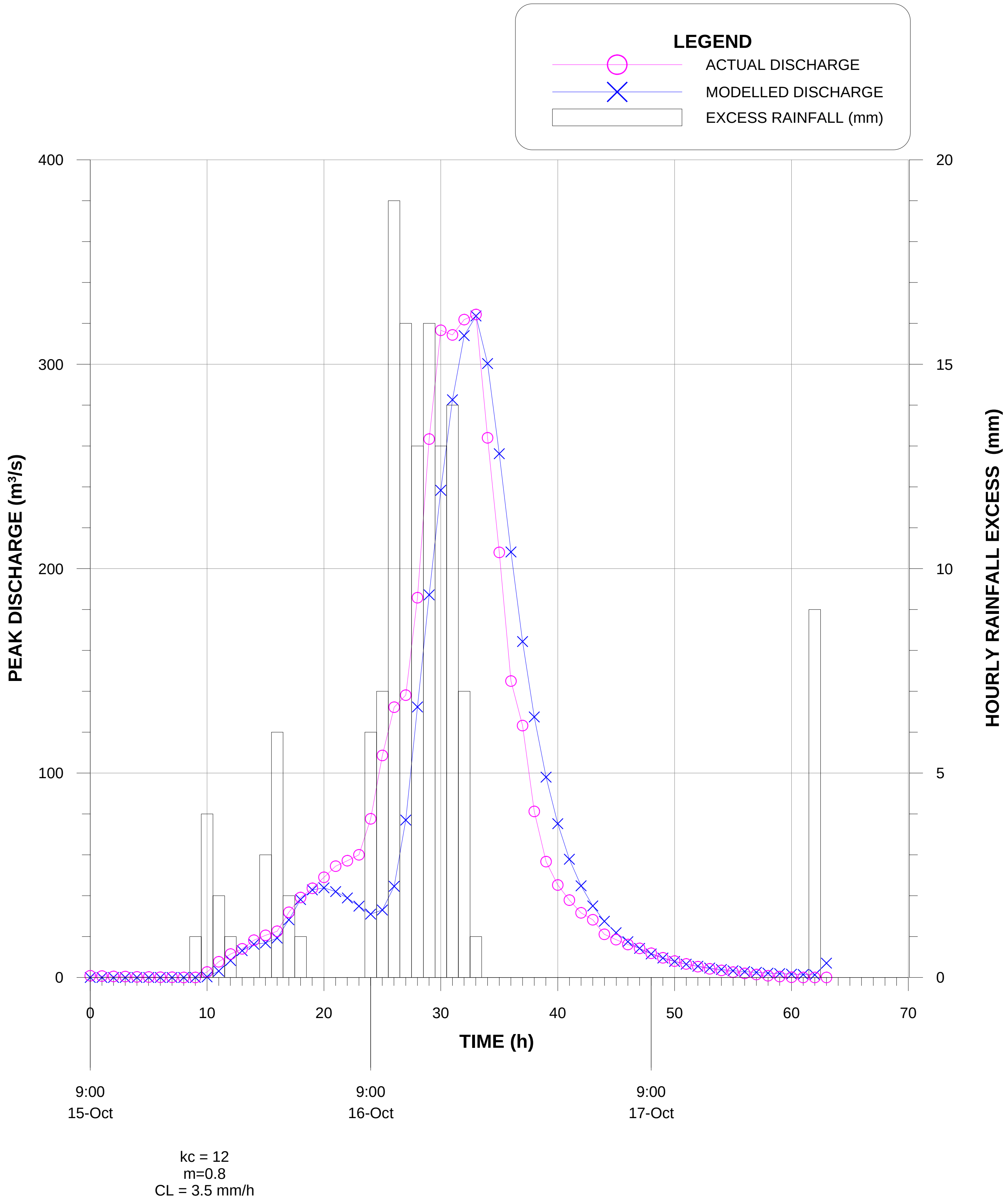
RORB MODEL CALIBRATION
CURRAMBENE CREEK AT THE FALLS
FEBRUARY 1971 FLOOD



CURRAMBENE CREEK AND MOONA MOONA CREEK FLOOD STUDIES

Figure 3.3

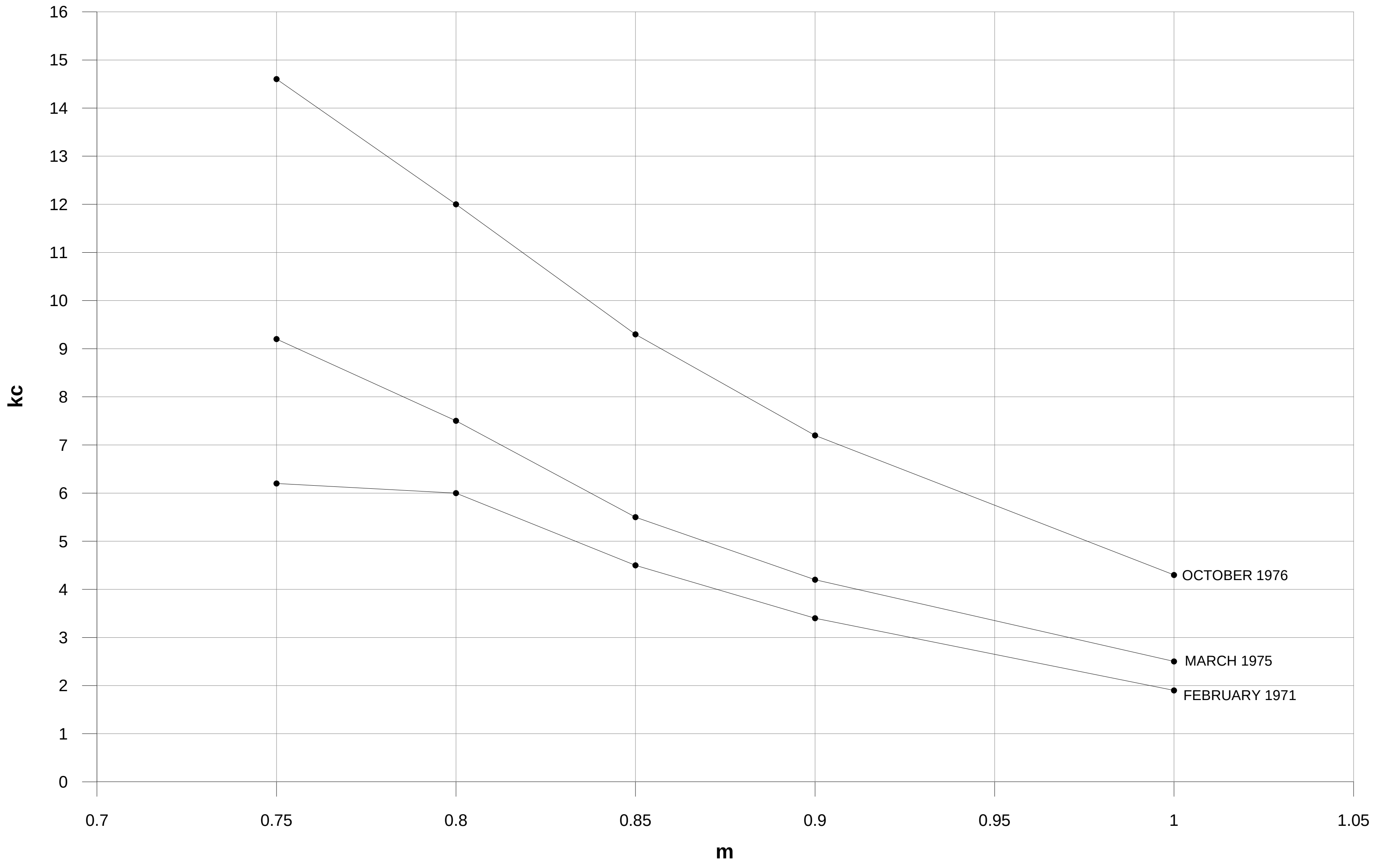
RORB MODEL CALIBRATION
CURRAMBENE CREEK AT THE FALLS
MARCH 1975 FLOOD



CURRAMBENE CREEK AND MOONA MOONA CREEK FLOOD STUDIES

Figure 3.4

RORB MODEL CALIBRATION
CURRAMBENE CREEK AT THE FALLS
OCTOBER 1976 FLOOD



CURRAMBENE CREEK AND MOONA MOONA CREEK FLOOD STUDIES

Figure 3.5

CURRAMBENE CREEK RORB MODEL PARAMETER INTERACTION DIAGRAM

4. CALIBRATING RORB MODEL AGAINST FLOOD FREQUENCY CURVE

4.1 Introduction

Flood estimation is usually based on converting a design rainfall with a given ARI to a flood by means of a runoff routing catchment model. Median rainfall loss rates are used to compute rainfall excess. This approach is based on the assumption that the ARI of the resulting flood will be the same as the ARI of the design rainfall.

The approach to RORB parameter estimation discussed in this section involved the derivation of design values of kc from a direct comparison of values of the same frequency obtained from the analysis of observed flood peaks at The Falls and design rainfalls derived from ARR.

The approach is analogous to the procedure adopted by Walsh et al in the derivation of initial losses for design flood estimation in NSW (Walsh et al, 1991). That investigation was based on recorded runoff at 22 gauging stations in NSW. Flood frequency curves and calibrated RORB rainfall runoff catchment models were developed for each catchment. Design storms were estimated for each catchment and applied to the RORB models. Initial loss values were varied until the modelled peak discharges from RORB corresponded with those of the flood frequency curve.

Use of the resulting initial loss values in conjunction with the design storms Walsh et al ensured that the resulting peak flows replicated the flood frequency curve. Consequently the derivation of initial loss was probabilistic in nature. In this present investigation, the initial loss values recommended by Walsh et al for design flood estimation were adopted and kc was varied, with m held constant at 0.8, until the flood frequency curve derived at The Falls (**Figure 2.1**) was replicated by the modelled flood peaks.

4.2 Steps in Deriving Catchment Storage Factor

The steps used to derive the storage factor kc for a given ARI were:

i) From the flood frequency curve of **Figure 2.1**, the peak discharge Qy for the desired ARI was noted.

ii) Using a range of storm durations of design storms from ARR, design rainfall excess hyetographs were estimated by selecting the design value of initial loss presented in Walsh et al, 1991 and an assumed continuing loss rate of 2.5 mm/h. The initial loss ranged between 40 and 55 mm depending on the ARI.

iii) The RORB model was used to transform the design rainfall excess hyetographs to runoff hydrographs. Initial runs showed that the 12 hour design storm was critical and therefore this duration was used in the subsequent analysis.

iv) The storage factor kc was varied to obtain correspondence between the calculated flood peak and the peak Qy from step (i).

4.3 Storage Factor Results

Table 4.1 shows the results achieved for a range of flood frequencies. The values of kc derived from the above approach lie in a narrow range between 10.2 and 10.8 for ARIs ranging between 10 and 100 years. For the 5 year ARI, the derived value of kc is 12.6.

TABLE 4.1
RORB MODEL PARAMETER ESTIMATION BY
CALIBRATION OF PEAK FLOWS
WITH FLOOD FREQUENCY CURVE

Average Recurrence Interval - years	RORB Model Parameters			
	IL mm	CI mm/h	kc	m
100	40	2.5	10.8	0.8
50	50	2.5	10.2	0.8
20	55	2.5	10.5	0.8
10	60	2.5	10.3	0.8
5	55	2.5	12.6	0.8

4.4 Comparison of Results with Published Data

According to the RORB manual, the value of kc is principally dependent on the catchment area, peak discharge and the parameter m. However, if m is fixed at 0.8, then kc becomes dependent only on the size of the catchment area. From previous work undertaken on the Currambene Creek catchment (BLA, 1983), a relationship between kc and catchment area A, for m equal to 0.8 was derived:

$$kc = CA^{0.5} \quad \text{…………………………..4.1}$$

where A = catchment area (km)
C = a proportionality constant

In the BLA, 1983 study, a value of 1.0 was derived for C. The form of this relationship was confirmed in the analyses described in **Section 4.3** for the catchment upstream of the gauging station, with C ranging between 1.05 and 1.1 for medium flood events larger than 10 year ARI.

Several relationships between kc and A are also given in ARR, as presented below.

For the eastern region of New South Wales, a relationship based on data from 29 catchments east of the dividing range derived by Kleemola, 1987 is:

$$kc = 1.22\,A^{0.46} \quad \text{………………………4.2}$$

A relationship (equation 4.3) was also derived from 86 catchments in Queensland. Most of the available data were for coastal catchments but values were included for streams west of the Great Dividing Range and near Mt Isa. No regional trends were evident.

$$kc = 0.88\ A^{0.53} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots 4.3$$

All of the above relationships apply for a value of m equal to 0.8.

These relationships were used to prepare estimates of kc for the Currambene Creek catchments at The Falls and at Huskisson (**Table 4.2**) for consideration in the calibration process.

TABLE 4.2
COMPARISON OF STORAGE PARAMETERS kc

Catchment	**Area (km^2)**	**Storage Parameter kc**			
		BLA, 1983 Eqn 4.1	**Kleemola, 1987 Eqn 4.2**	**Qld data Eqn 4.3**	**Present Investigation**
Currambene Creek at The Falls	95	9.9 (1)	9.9	9.8	10.2 – 10.8 (2)
Currambene Creek at Huskisson	160	12.7	15.4	13.0	13.9

Notes:

(1) This value derived from Eqn. 4.1 with C = 1.0

(2) These values apply for floods in the range 10 to 100 year ARI.

Results achieved with the calibrated RORB model at The Falls are in close agreement with the relationships between kc and A presented in ARR and the BLA, 1983 study.

When the published relationships are extrapolated to the 160 km^2 catchment upstream of Huskisson, the results presented in BLA, 1983 are close to the results presented by Kleemola, 1987 and the Qld data. The results achieved for the present investigation for flood events greater than 10 year ARI suggest value of 1.05 to 1.1 for the constant C. Adoption of a value of 1.1 would yield a kc value equal to 13.9 for Currambene Creek at Huskisson which was adopted for design flood estimation in the present study.

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5. SUMMARY

Three historic floods which occurred in the 1970s were selected for the purposes of calibrating the RORB rainfall-runoff model of the 95 km^2 catchment of Currambene Creek at The Falls stream gauging station, which is located on the western side of the crossing of the Princes Highway.

Based on an annual series analysis of flood peaks at The Falls, which has been in operation since 1969, the highest peak discharge of 713 m^3/s which occurred in February 1971, had a recurrence interval between 50 and 100 years ARI. The other floods which occurred in March 1975 and October 1978 had peaks of 443 m^3/s and 328 m^3/s and were in the range 5 and 20 years ARI.

Although there is a pluviograph located within the catchment at the RAN Air Station, which provided information on the temporal pattern of rainfall for the three floods, it was not possible to achieve a consistent set of kc and m parameters for the three floods (**Figure 3.5**), although it was possible to achieve a good correspondence between recorded and modelled discharge hydrographs for various sets of model parameters (**Figures 3.2 to 3.4**).

A second analysis was undertaken in which design storms of various frequencies were applied to the RORB model to determine the set of parameters which reproduced the recorded flood frequency relationship shown on **Figure 2.1**. The results of this analysis gave a small range of kc values (between 10.2 and 10.8) for return periods ranging between 10 and 100 years and a larger value of 12.6 for the 5 year ARI. These results were achieved with an m value of 0.8, a continuing loss of 2.5 mm/h and initial losses recommended by Walsh et al, 1991, as shown on **Table 4.1**.

The RORB model parameters shown on **Table 4.1** were used in **Appendix B** for the purposes of deriving design inflow hydrographs to the hydraulic model of Currambene Creek at The Falls.

Lateral inflow hydrographs for the sub-catchments downstream of The Falls were also required for input to the hydraulic model. They were derived from the RORB model of the catchment which extends as far as the outlet at Huskisson. A kc value of 13.9 was adopted for the “Big” RORB model of the catchment (160 km^2).

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6. REFERENCES

Walsh M A, Pilgrim D H, Cordery I (1991). ***"Initial Losses for Design Flood Estimation in New South Wales"***. International Hydrology and Water Resources Symposium. Perth 2-4 October 1991.

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APPENDIX B

DERIVATION OF DESIGN DISCHARGE HYDROGRAPHS

November 2006

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LIST OF FIGURES

1. SYNOPSIS

This appendix deals with the estimation of design peak flows on Currambene and Moona Moona Creeks. The hydrologic models developed for these catchments were used to provide inflow hydrographs for the hydraulic modelling of the two streams, the results of which are discussed in **Appendix C**.

Catchment models of the two streams were developed using the RORB rainfall-runoff routing software. Model parameters for the RORB model of Currambene Creek at The Falls gauging station were assessed from the analysis of recorded rainfall runoff data described in **Appendix A**. The Falls gauging station is located near the upstream end of the hydraulic model and design hydrographs from RORB were used as the upstream boundary condition of the hydraulic model.

A RORB model of the Currambene Creek catchment upstream of the outlet to Jervis Bay at Huskisson was also developed and used to provide discharge hydrographs for the tributary streams joining the creek between The Falls and Huskisson. This second model was denoted the “Big” RORB model.

A RORB model was also developed for the Moona Moona Creek catchment which outfalls to Jervis Bay at the northern end of Collingwood Beach. Model parameters for this model and also for the “Big” Currambene Creek model outfalling at Huskisson were developed by analogy with those derived for The Falls RORB model.

The flood storage contained in the overbank areas in the middle to lower reaches of Currambene Creek was simulated by the incorporation of a storage volume versus discharge relationship, located within the model below the junction of two major tributaries with the main arm, near Goodland Road. The storage characteristics were evaluated by using HEC-RAS in steady state mode to compute the storage volumes below water surface profiles for a number of flood discharges on Currambene Creek. In this way RORB, was able to approximate the attenuating effects of the flood storage on the floodwave.

In **Appendix C**, HEC-RAS was run in its dynamic mode and by simultaneously solving both the momentum and continuity equations of flow, was able to more accurately model the flood storage than RORB. Flood storage is actually distributed along the length of the stream rather than lumped at a particular location. A comparison of the results achieved by these two alternative modelling approaches is presented in **Appendix C**.

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2. FLOOD HYDROLOGY

2.1 Selection of Hydrologic Model

2.1.1 General

For hydrologic modelling in the flood studies, the practical choice was between the models known as RAFTS, RORB and WBNM, and any of these would have been suitable. Each of these models converts storm rainfall to discharge hydrographs using a procedure known as runoff-routing. There was little to choose technically between these models, however their usage in previous studies in the catchment, as well as the familiarity of the user with the model, were the determining factors in the selection of the RORB modelling approach.

2.1.2 Brief Review of RORB Modelling Approach

The RORB program envisages the catchment to be comprised of a series of concentrated storages which represent subcatchments defined on watershed lines, plus concentrated special storages which represent additional stream routing effects such as channel or floodplain storage.

All storage elements within the catchment are represented via the storage-discharge equation:

$$S = kQ^m \qquad (2.1)$$

where	S	=	volume of storage.
	Q	=	discharge
	k	=	a storage delay parameter.
	m	=	a measure of the catchment's non-linearity. When m is set equal to unity the catchment's routing response is linear.

The storage parameter "k" within the general storage equation is modified to reflect the catchment storage and the reach storage as follows:

$$k = kc.kr \qquad (2.2)$$

where	kc	=	an empirical coefficient applicable to the entire catchment and stream network.
	kr	=	a dimensionless ratio called the relative delay time, applicable to an individual reach storage.

RORB has been used extensively throughout Australia on a wide range of rural and urban catchments. Calibrated values for kc and m for a large number of regions have been developed and have been used to estimate flows on ungauged catchments. For the present study recorded rainfall runoff data for the Currambene Creek catchment were used to establish model parameters for that catchment (ref. **Appendix A**) and for the adjacent Moona Moona Creek catchment.

2.2 RORB Model Layouts

2.2.1 Currambene Creek

Two hydrologic models were developed for the Currambene Creek catchment. The first model terminated at The Falls stream gauging station and was used to assess inflow hydrographs at the upstream boundary of the hydraulic model. The second model terminated at the outlet to Jervis Bay at Huskisson and was used to provide lateral inflow hydrogaphs from the 65 km^2 of sub-catchments entering Currambene Creek over its tidal reach below the Princes Highway.

The model layout is shown on **Figure 2.1**. The RORB model at The Falls comprised sub-catchments A to L representing a total area of 95 km^2. The Huskisson ("Big" RORB) model comprises sub-catchments A to X, a total catchment area of 160 km^2.

To obtain accurate discharge hydrographs at a particular location, RORB requires the catchment upstream to be divided into several sub-areas. At least three sub-areas were incorporated in the model upstream of locations where an accurate assessment of the time distribution of major tributary inflows were required for hydraulic modelling. A number of the minor tributaries which join the main stream downstream of the upper limit to the hydraulic model were modelled with fewer sub-areas, as estimation of the volume of runoff was more important than the shape of the discharge hydrograph.

A special storage was introduced in the RORB model to simulate the storage in the channel and the surrounding above-tide swampy areas. The characteristics of this storage (i.e. the relationship between the volume of storage contained below water surface profiles of various discharges) were obtained from backwater analysis using HEC-RAS. This special storage was located in the model below the confluences of the main stream with Georges Creek, which is modelled as sub-catchments S to U and the tributaries to the south-west of Woollamia, modelled by sub-catchments P to R.

Incorporation of the special storage in RORB gave an approximation of the attenuating effects of floodplain storage on peak flows in the lower reaches of Currambene Creek. A more accurate representation of storage is incorporated in the dynamic model adopted for hydraulic analysis. The hydraulic model is based on a quasi two dimensional approach which solves the St. Venant equations of flow, i.e. momentum and continuity, at each model cross section. Details are presented in **Appendix C**.

2.2.2 Moona Moona Creek

The RORB model layout for Moona Moona Creek is shown on **Figure 2.2**. The total catchment area of Moona Moona Creek at the bridge over Elizabeth Drive at the northern end of Collingwood Beach is 28 km^2.

The large floodplain storage upstream of the bridge was not incorporated in RORB, but was modelled hydraulically by running HEC-RAS in dynamic mode, as discussed in **Appendix C**.

2.3 RORB Model Parameters

2.3.1 Model Parameters

There are four parameters of interest when running a RORB model:

Storage - Discharge equation exponent m

The exponent of the catchment storage-discharge equation, m, is a measure of the catchment's non-linearity with a value of unity implying a linear catchment. For this analysis, a constant m value of 0.8 was used in conformity with recommendations in the RORB manual for flood estimation on ungauged catchments.

Lag parameter kc

The parameter kc, which is the principal parameter of the RORB model, provides a measure of the storage delay time within a catchment. Decreasing kc increases the peak discharge and decreases the catchment lag, while increasing kc has the opposite effect.

The value of kc is principally dependent on the catchment area, peak discharge and the parameter m. However, if m is fixed at 0.8, then kc becomes dependent only on the size of the catchment area.

Initial Loss and Continuing Loss

The values of initial loss (IL) and continuing loss (CL), which are subtracted from the gross storm rainfalls to give rainfall excess, are other important parameters. Altering the value of these parameters will cause significant changes in the shape and the peak of the computed hydrograph.

Walsh et al (1991) gives recommendations for the adoption of initial and continuing loss values for use in the RORB model. The loss values were based on an analysis of recorded rainfall runoff data for 22 catchments in NSW, including three catchments in the Nowra area. Initial losses were found to be relatively large, in the order of 50 to 60 mm, depending on the recurrence interval of the flood.

These values of IL are considerably higher than values recommended in Table 6.2 of ARR, which range between 10 and 35 mm for catchments east of the western slopes of NSW. The result is that with the abstraction of the larger depths of rainfall adopted in this investigation, longer duration storms of 12 to 18 hours duration were found to be "critical" in the maximisation of peak discharges. The temporal patterns of rainfall contained in ARR were derived from rainfall bursts experienced in historic storms. Use of the longer rainfall durations means that a greater portion of the overall storm rainfall was used in the process of design flood estimation than would have been the case if the lesser IL values had been adopted.

The hydrologic model calibration described in **Appendix A** showed that historic storms responsible for major floods on Currambene Creek were typically of up to 24 hours duration. Consequently, the design IL values in conjunction with the longer duration critical storms are representative of historic flood conditions experienced in the study area.

Average CL rates of 2.5 mm/h were derived for the catchments east of the Great Dividing range. This value is also recommended in ARR for design flood applications and has been adopted in the present study.

2.3.2 Derivation of Storage Parameter kc

Calibration of the RORB model for Currambene Creek is discussed in detail in **Appendix A**. In the case of Currambene Creek, the model for the catchment upstream of The Falls was calibrated for three historic floods for which concurrent rainfall and runoff data were available. The calibration process yielded a set of model parameters which could be used in conjunction with design storms derived from ARR to reproduce the flood frequency curve derived from an annual series analyses of flood peaks recorded at the stream gauging station.

The catchment storage delay parameter kc was related to catchment area (A), measured in square kilometres by the following equation:

$$kc = CA^{0.5} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ (2.3)$$

where A = catchment area (km^2)
C = proportionality constant

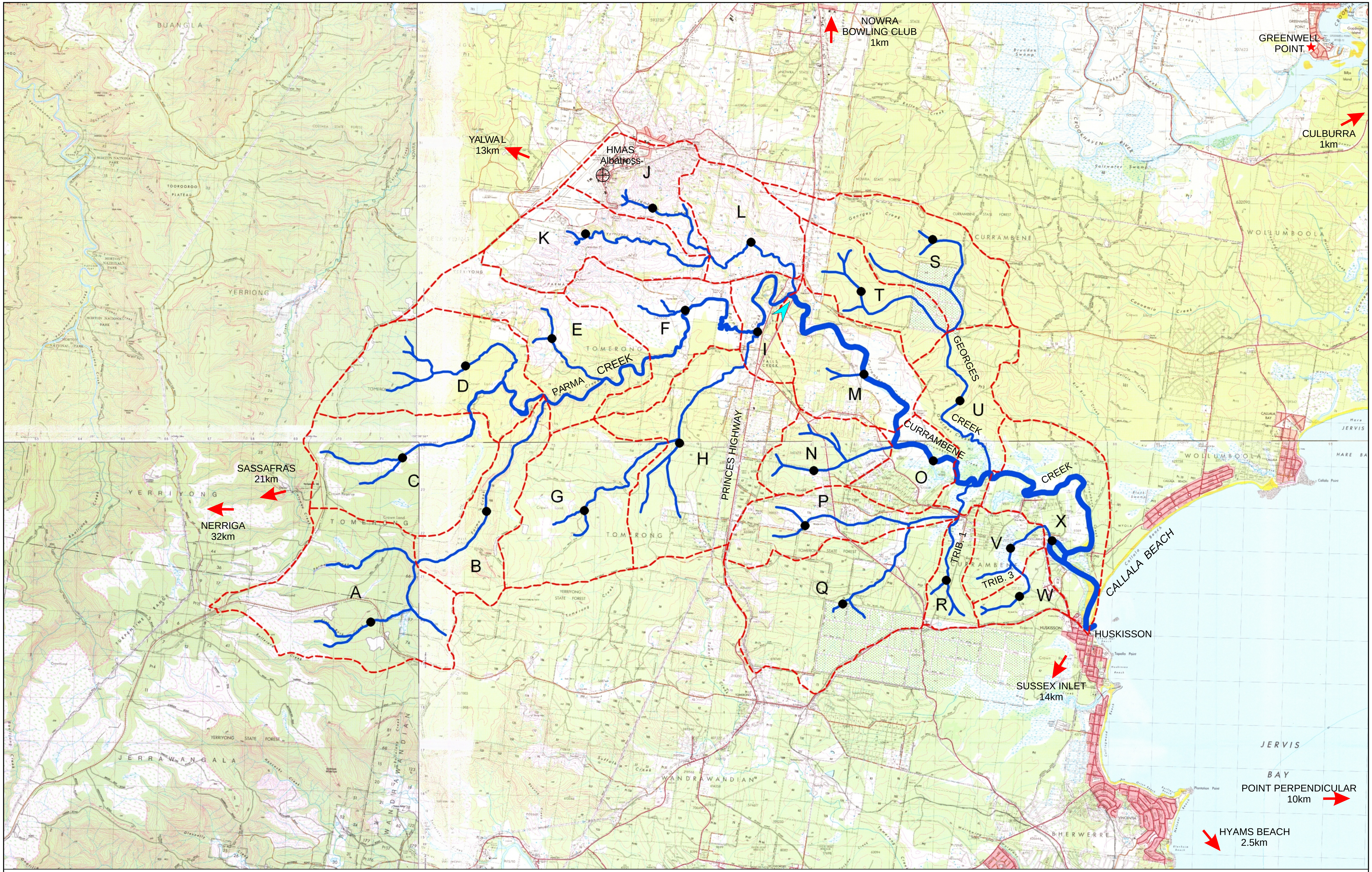
This relationship was also used to estimate kc for both the “Big” RORB and Moona Moona Creek RORB models.

2.4 RORB Model Parameters Adopted for Design Flood Estimation

Table 2.1 summarises the RORB model parameters adopted for generating hydrographs for the design storm events.

TABLE 2.1
RORB MODEL PARAMETERS ADOPTED
FOR DESIGN FLOOD ESTIMATION

Catchment	Area (km^2)	IL (mm)	CL (mm/hr)	kc	m
Currambene Creek at The Falls	95	40 – 60	2.5	10.2 – 12.6	0.8
Currambene Creek at Huskisson	160	40 – 60	2.5	13.9	0.8
Moona Moona Creek at Jervis Bay	28	40	2.5	5.4	0.8



N

0 2 4km

Scale

Currambene Creek at "The Falls" Stream gauging station. (STN 218004)

Pluviograph site

G ● Model sub-area and centroid

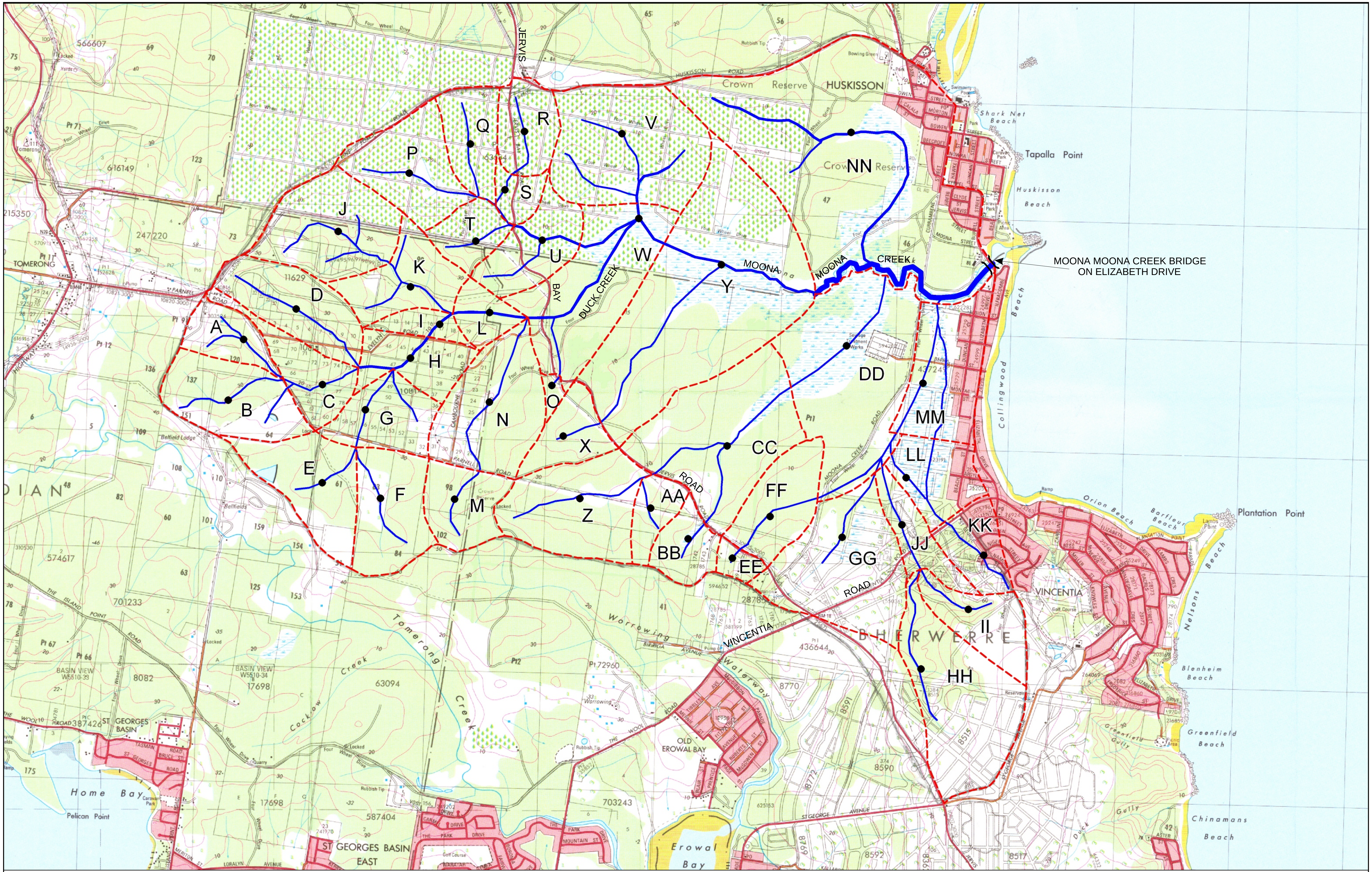
★ Daily Rainfall Station

YALWAL 13km Daily Rainfall Station and distance

CURRAMBENE CREEK AND MOONA MOONA CREEK FLOOD STUDIES

Figure 2.1

RORB MODEL LAYOUT
CURRAMBENE CREEK CATCHMENT



N

0 750 1,500m

Scale

Moona Moona Creek Bridge

G● Model sub-area and centroid

CURRAMBENE CREEK AND MOONA MOONA CREEK FLOOD STUDIES

Figure 2.2

RORB MODEL LAYOUT
MOONA MOONA CREEK CATCHMENT

3. DERIVATION OF DESIGN STORMS

3.1 Rainfall Intensity

The procedures used to obtain temporally and spatially accurate and consistent intensity-frequency-duration (IFD) design rainfall curves for the Currambene Creek and Moona Moona Creek catchments are presented in ARR. Design storms for frequencies of 5, 10, 20, 50, 100 and 200 year ARI were derived for storm durations ranging between 6 hr and 24 hrs. The procedure adopted was to generate IFD data for each catchment by using the relevant charts in ARR. These charts included design rainfall isopleths, regional skewness and geographical factors.

3.2 Areal Reduction Factors

The rainfalls derived using the processes outlined in ARR are applicable strictly to a point. In the case of a large catchment, it is not realistic to assume that the same rainfall intensity can be maintained over a large area. An areal reduction factor (ARF) is typically applied to obtain an intensity that is applicable over the entire area.

The IFD values contained in ARR were originally published by the US National Weather Service in 1980 and were derived from recorded storm data in the Chicago area. The Cooperative Research Centre for Catchment Hydrology (CRCCH) undertook a program of deriving ARF's in an Australian setting. Siriwardena and Weinmann, 1996 undertook this analysis for Victorian catchments for a range of catchments from 1 to 10,000 km^2 in area and storm durations from 18 to 120 hours. The conclusion of this investigation was that ARF's were related to rainfall frequency and that the values in ARR should be reduced by 5-8 % for storm durations in this range.

Catchlove and Ball, 2003 undertook a study on the 112 km^2 catchment of the Upper Parramatta River where 8 pluviometers were analysed. The key finding of this investigation was that for storm durations in excess of 2 hours, the best estimate of ARF for this catchment was 1.0. Application of relationships derived by ARR and CRCCH gave similar results for the Upper Parramatta River catchment, because the variations for different exceedance probabilities for a small catchment of this size are minimal. In practice, adoption of a single ARF unrelated to frequency is more appropriate.

For this present study, ARR indicates that a value of 0.95 could have been adopted for the ARF on the Currambene Creek catchment as an appropriate value for the 12 to 18 hour storm durations found to be critical on this catchment. However, a value of 1 was selected in keeping with the more recent results of Catchlove and Ball.

As the catchment area of Moona Moona Creek is relatively small (28 km^2), negligible reduction in rainfall intensity would result, thus the point values were adopted.

3.3 Temporal Patterns

Temporal patterns for various zones in Australia are presented in ARR. These patterns are used in the conversion of a design rainfall depth with a specific ARI into a design flood of the same frequency. Patterns of average variability are assumed to provide the desired conversion. The patterns may be used for ARIs up to 500 years where the design rainfall data are extrapolated to this ARI.

The derivation of temporal patterns for design storms is discussed in ARR and separate patterns are presented in ARR for ARI < 30 years and ARI > 30 years. The second pattern is intended for use for rainfalls with ARIs up to 100 years, and to 500 years in those cases where the design rainfall data are extrapolated to this ARI.

4. DESIGN HYDROGRAPHS CURRAMBENE CREEK

The RORB models were run with the parameters shown in **Table 2.1** to obtain design hydrographs for input to the hydraulic model. Peak flows at the model outlets and the corresponding critical storm durations are shown on **Table 4.1**.

At most locations, the 12 hour storm was critical for generating peak discharge in RORB, apart from the major flood events, where the 9 hour storm gave the highest peak discharges at The Falls. In the hydraulic modelling described in **Appendix C**, storm durations up to 24 hours in length were found to be critical in terms of generating peak flood levels. This is probably due to the more accurate method of modelling flood storage inherent in a dynamic hydraulic model. Discharge hydrographs for these long duration storms were derived by RORB using similar procedures to those described above.

The lumped storage downstream of the confluence with Georges Creek resulted in a considerable reduction in the peak discharge. As mentioned, in reality, the storage is actually distributed along a reach of Currambene Creek of several km in length. Its impacts on flows are more accurately assessed in the hydraulic model, the results of which are described in **Appendix C**.

Figures 4.1 and **4.2** show design hydrographs for 5 and 100 year ARI storms respectively, each of 9 hours duration. At The Falls, the flood peaks about 8 hours after the commencement of the storm. The lateral inflow hydrographs which enter between the Highway and Huskisson generally peak at 6 hours after commencement of rainfall.

TABLE 4.1
DESIGN PEAK DISCHARGES
CURRAMBENE CREEK

Location	Peak Flow (m^3/s)					
	5 yr ARI	10 yr ARI	20 yr ARI	50 yr ARI	100 yr ARI	200 yr ARI
The Falls	215 [12]	342 [12]	471 [12]	658 [9]	788 [9]	934 [9]
u/s Georges Creek Confluence	286 [12]	373 [12]	530 [12]	711 [12]	875 [12]	1017 [12]
u/s Special Floodplain Storage	330 [12]	501 [12]	719 [12]	970 [12]	1190 [12]	1385 [12]
Woollamia	240 [12]	306 [12]	437 [12]	597 [12]	683 [9]	931 [12]
Callala Beach (Huskisson)	241 [12]	307 [12]	434 [12]	600 [12]	771 [12]	938 [12]

Note : Values in square brackets represent the critical storm duration in hours which results in the peak discharge at the relevant location.

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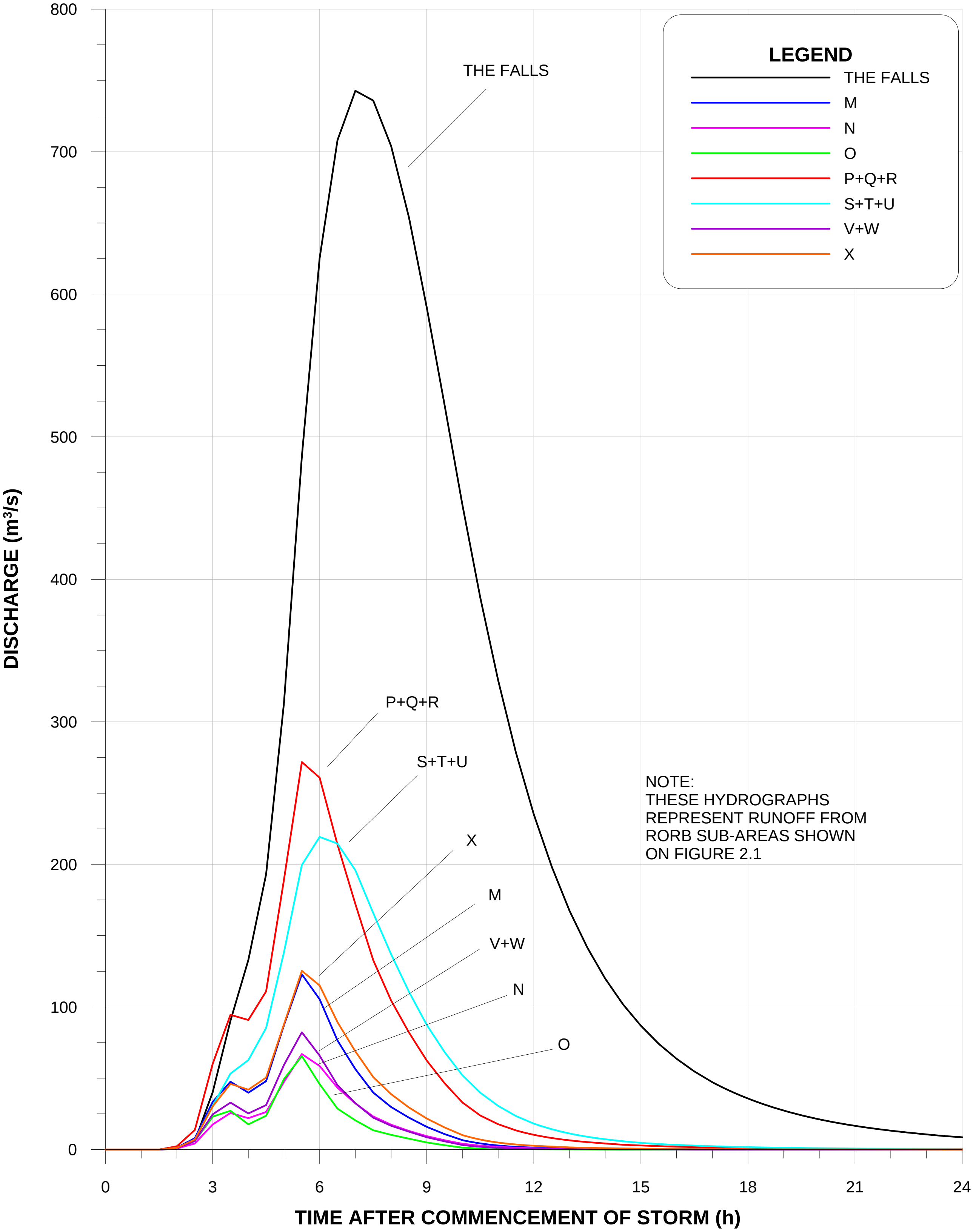
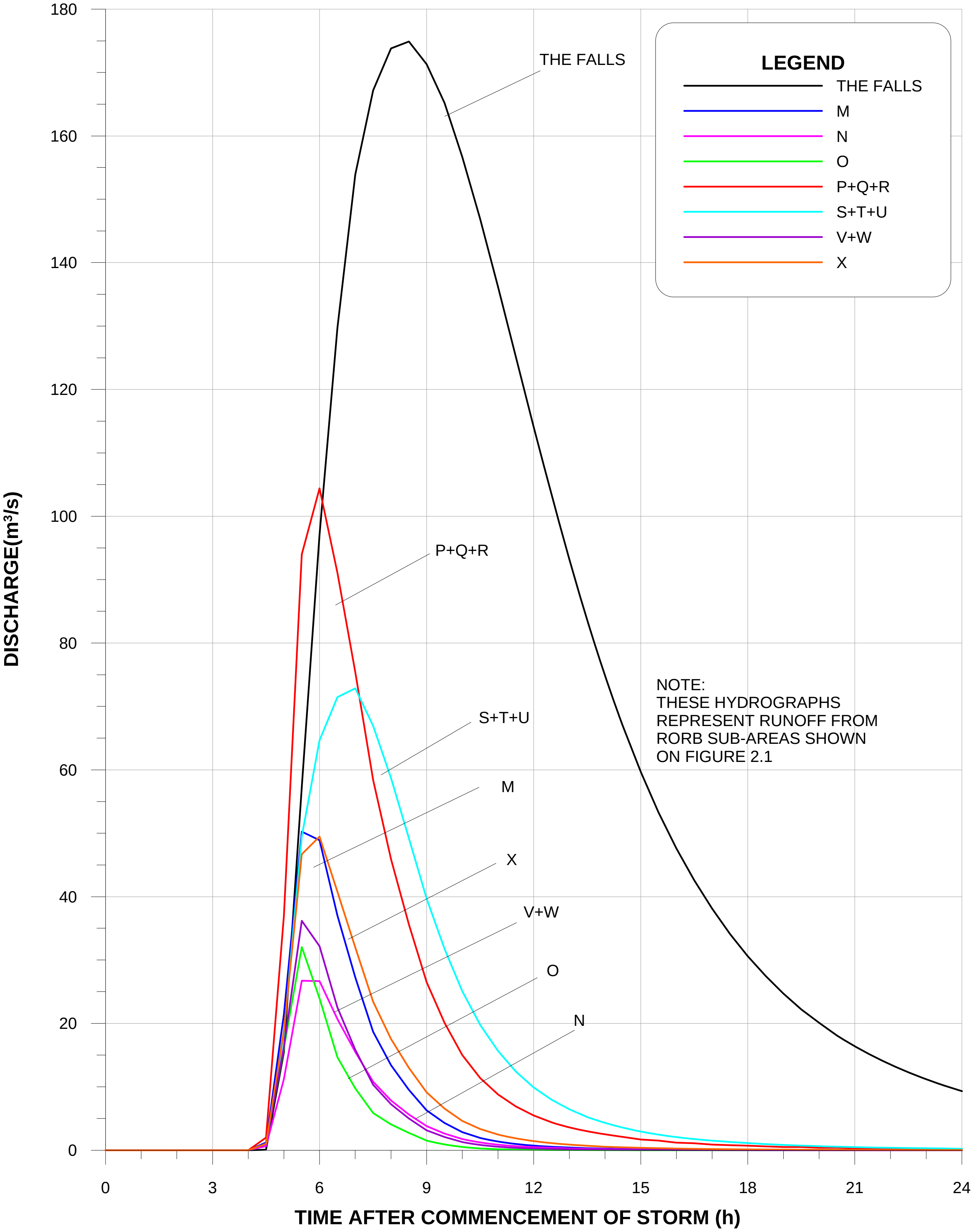


CURRAMBENE CREEK AND MOONA MOONA CREEK FLOOD STUDIES

Figure 4.1

CURRAMBENE CREEK DESIGN HYDROGRAPHS
100YR ARI, 9HR STORM



CURRAMBENE CREEK AND MOONA MOONA CREEK FLOOD STUDIES

Figure 4.2

CURRAMBENE CREEK DESIGN HYDROGRAPHS
5YR ARI, 9HR STORM

5. DESIGN HYDROGRAPHS MOONA MOONA CREEK

Peak flows on Moona Moona Creek are shown in **Table 5.1**. Downstream of the confluence with Duck Creek, the floodplain of Moona Moona Creek comprises a large overgrown storage area. The storage characteristics of this area are incorporated in the hydraulic model, which is described in **Appendix C**. The flows shown on **Table 5.1** at the bridge do not allow for the attenuating effects of the storage and are consequently on the high side of actual peak flows for the various flood frequencies.

Figures 5.1 and **5.2** show discharge hydrographs for 100 and 5 year ARI storms of 9 hour duration. The hydrograph labelled “A-W” represent flows on Moona Moona Creek downstream of the confluence with Duck Creek. Hydrographs “Z-DD” and “EE-JJ” represent contributions from the southern portion of the catchment which cross Vincentia Road and Jervis Bay Road en route to the floodplain. Hydrograph “NN” represents inputs from the northern side of the catchment.

Discharge hydrographs derived by RORB at the catchment outlet at the Elizabeth Drive bridge have not been plotted on the figures.

TABLE 5.1
DESIGN PEAK DISCHARGES
MOONA MOONA CREEK

Location	Peak Flow (m^3/s)					
	5 yr ARI	10 yr ARI	20 yr ARI	50 yr ARI	100 yr ARI	200 yr ARI
d/s Moona Moona/Duck Creek Confluence	51 [12]	64 [12]	84 [9]	108 [9]	128 [9]	149 [9]
Elizabeth Drive Bridge	117 [12]	148 [12]	185 [9]	235 [9]	278 [9]	328 [9]

Note: Figures in square brackets represent the critical storm duration in hours.

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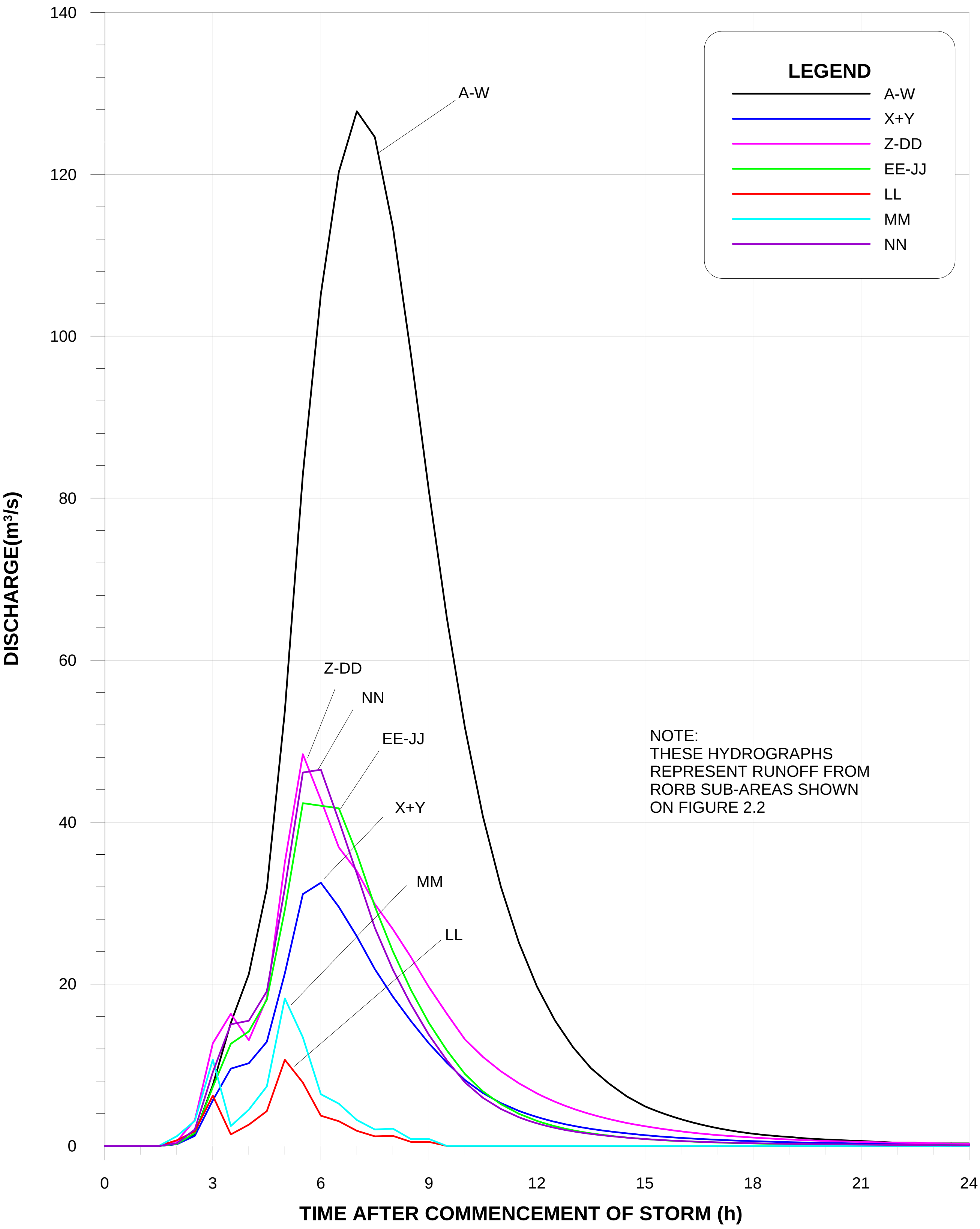
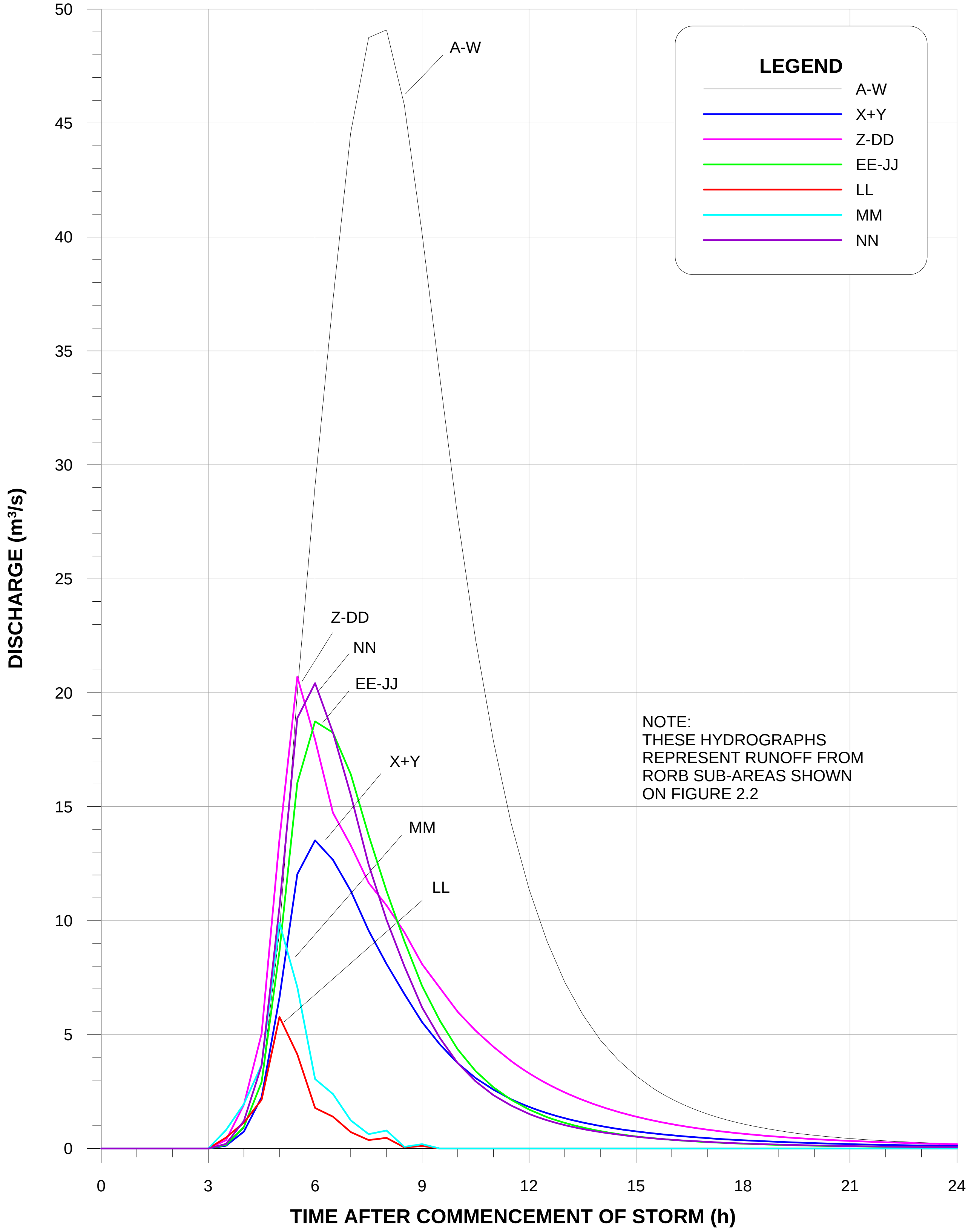


CURRAMBENE CREEK AND MOONA MOONA CREEK FLOOD STUDIES

Figure 5.1

MOONA MOONA CREEK DESIGN HYDROGRAPHS
100YR ARI, 9HR STORM



CURRAMBENE CREEK AND MOONA MOONA CREEK FLOOD STUDIES

Figure 5.2

MOONA MOONA CREEK DESIGN HYDROGRAPHS
5YR ARI, 9HR STORM

6. PROBABLE MAXIMUM FLOOD

6.1 Rainfalls

The PMF is the response of the catchment to the probable maximum precipitation (PMP) and is the largest flood event that can reasonably be expected to occur at a particular location.

PMP rainfalls were determined using the Generalised Short Duration Method (GSDM) outlined in the June 2003 Bureau of Meteorology publication (BOM, 2003).

The GSDM procedure applies for small areas and relatively short durations up to 6 hours and involves the following tasks:

- Estimation of average catchment rainfall using depth-duration-area data derived by drawing enveloping curves to the highest recorded US and Australian rainfall depths.
- Assessment of the spatial distribution of rainfall over the catchment using procedures based on US Weather Bureau experience.
- Derivation of design hyetographs using the GSDM temporal distribution which is based on pluviograph traces recorded in major Australian storms.

6.2 PMF Flows

6.2.1 RORB Model Parameters

The design flows derived for events up to the 200 year ARI on both the Currambene and Moona Moona Creek catchments were based on the assumption that the catchment behaved in a non-linear manner. Non-linear catchment behaviour occurs when the peak discharges for various storm events increase at a proportionally greater rate than the increase in rainfall intensities. Non-linear catchment response is modelled in RORB by adopting a value less than 1 for the exponent m of the catchment's storage-discharge equation (Equation 2.1).

For storms up to the 200 year ARI, a value of 0.8 was adopted for the present investigation. While there is evidence of non-linear response (i.e. a value of m not equal to unity) over the range of observed floods in most natural catchments, it is unclear whether this effect persists to the PMF. At that magnitude of flooding, the routing response depends on the relative efficiency of the drainage system and the amount of storage on the catchment.

The V-shaped valleys of the upper portion of Currambene Creek above the Princes Highway and also in the upper reaches of Moona Moona Creek catchment have comparatively small overbank areas and therefore, have a theoretical value of 0.75 - 0.8 for the exponent m of the storage versus discharge relationship used by RORB in the rainfall-runoff routing process for each sub-area of the model. This indicates that the headwaters of the creeks should continue to behave in a non-linear manner for extreme floods.

On the other hand, the lower floodplains of the study area contain a large volume of flood storage, which may have the effect of increasing the value of m for major flood events and beyond. Also, the flow resistance in extreme floods may be increased by debris, erosive processes and increased turbulence and all of these influences may promote linear behaviour.

A sensitivity analysis of the PMF was undertaken with the RORB models for both catchments run in a linear manner. The coefficient kc in the storage versus discharge relationship was first adjusted to ensure that the magnitude of peak flow at the 100 year ARI level was unchanged when used with the new value of m equal to 1.

RORB model parameters for both streams are shown on **Table 6.1** for both linear and non-linear models. The 6 hour storm duration gave the highest peak flows on Currambene Creek and the 3 hour storm on Moona Moona Creek.

TABLE 6.1
ALTERNATIVE RORB MODEL PARAMETERS FOR PROBABLE MAXIMUM FLOOD ESTIMATION

RORB Model	Non-Linear Model		Linear Model	
	Kc	m	Kc	m
Currambene Creek at The Falls	10.8	0.8	3.1	1.0
Currambene Creek at Huskisson	13.9	0.8	5	1.0
Moona Moona Creek	5.4	0.8	2.5	1.0

Note: The above Non-Linear and Linear Model parameters (kc and m) give the same estimates of 100 year ARI peak discharge at the respective catchment outlets.

6.2.2 Peak Flows

Peak flows on Currambene Creek are shown on **Table 6.2**. For the PMF, the values of initial loss and continuing loss adopted did not have a significant effect on model results due to the magnitude of the storm rainfall.

Peak flows are about 2.3 to 2.6 times the 100 year ARI discharge for the non-linear model assumption with no losses, reducing 1.7 to 1.9 times for the linear model with losses.

After consideration of the results of **Table 6.2** (as discussed in **Section 6.3**), the non linear value of m = 0.8 with no losses was adopted for design PMF estimation.

TABLE 6.2
ESTIMATES OF PROBABLE MAXIMUM FLOOD PEAK DISCHARGES
CURRAMBENE CREEK

Location	Peak Flows m^3/s			
	Non-Linear Model		Linear Model	
	With Losses	No Losses	With Losses	No Losses
The Falls	1860	1980	1530	1830
u/s Special Floodplain Storage	2530	2810	1880	2115
d/s Special Floodplain Storage	1580	1805	1350	1530
Callala Beach Huskisson	1580	1810	1340	1520

Peak flows on Moona Moona Creek are shown on **Table 6.3** at the entrance to the floodplain storage area and on its downstream side at Elizabeth Drive. The flood storage is not incorporated in the Moona Moona Creek RORB model and consequently, the flows at the bridge are higher than would actually be experienced at that location.

Peak flows are about 3.2 times the 100 year ARI discharge for the non-linear model assumption with no losses, reducing 2.4 times for the linear model with losses.

The non-linear values with no losses have been adopted for design purposes.

TABLE 6.3
ESTIMATES OF PROBABLE MAXIMUM FLOOD PEAK DISCHARGES
MOONA MOONA CREEK

Location	Peak Flows m^3/s			
	Non-Linear Model		Linear Model	
	With Losses	No Losses	With Losses	No Losses
d/s Moona Moona/Duck Creek Confluence	378	415	310	340
Elizabeth Drive Bridge	786	883	658	730

6.2.3 Discussion

Investigations on extreme flood estimation have shown that there is a trend for the ratio between PMF and 100 year ARI peak flows to reduce as the catchment area increases. On small urbanising catchments of several square kilometres area for example, the ratio is typically in the range 4 to 6 times. An investigation of PMF flooding on the Upper Nepean River gave a ratio of 2.4 for a catchment of 640 km^2 (LMCE, 1995). For the Upper Nepean Study a catchment-specific estimate of the PMP was prepared by BOM.

While the trend for the ratio between PMF and 100 year ARI flows to reduce with catchment area is evident in the present investigation, the ratio is less than would be expected on the basis of experience. The ratio for Currambene Creek, around 2.5 times, is similar to the Upper Nepean value, but the catchment is smaller, being 160 km^2 at the outlet to Jervis Bay. The catchment area of Moona Moona Creek at the outlet is much smaller at 28 km^2, but the ratio is only 3.2 times.

The PMP estimates in this present investigation were prepared using the GSDM approach set out in BOM, 2003, which applies for storm durations up to 6 hours. In the RORB analysis for Currambene Creek, there was the trend for discharges to increase for longer duration storms and it may be the case that the critical storm has not been captured by limiting the analysis to 6 hours. Storm durations longer than 6 hours may have produced greater peak discharges than shown on **Table 6.2**. However, to explore this effect further would have required commissioning the Bureau of Meteorology to prove a site specific estimate of longer duration PMP's, which was not justified.

The discharge hydrographs derived in this investigation are considered to allow a reasonable estimate of extreme flood levels to be determined in **Appendix C**.

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APPENDIX C

HYDRAULIC MODELLING OF DESIGN FLOODS

November 2006

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FIGURES

ADDENDA

1. INTRODUCTION

1.1 Synopsis

This Appendix deals with the derivation of flooding patterns on Currambene Creek and Moona Moona Creek. The hydraulic models used in the derivation of flooding were based on the unsteady flow version of the HEC-RAS software, developed by the Hydrologic Engineering Centre of the US Army Corps of Engineers and also known as UNET.

The geometric model of Currambene Creek, which extended over a distance of 15 km upstream of the outlet to Jervis Bay at Huskisson, was based on survey undertaken for the previous Flood Study by Brian Lyall and Associates (BLA, 1983), together with additional survey of the channel and off-stream storage areas undertaken in 2004. The additional survey allowed a better representation of flood storage in the floodplain than was possible in the previous investigation.

The Moona Moona Creek geometric model was based on the 2004 cross sectional survey of the creek system which extended into the flood storage areas adjacent to the tidal channel upstream of the Elizabeth Drive bridge. The hydraulic model extended about 3 km upstream of the bridge.

Discharge hydrographs generated by the RORB catchment models described in **Appendix B** were applied to the hydraulic models at the relevant locations. The downstream boundary condition comprised tidal hydrographs with appropriate allowance for storm tide effects.

There are no data available on historic flood levels on the two streams, possibly due to the absence of significant flood events in recent years. Accordingly, it was necessary to carry out analysis to test the sensitivity of results to variations in hydraulic roughness. “Best estimates” of roughness were derived by reference to the engineering literature and from experience with similar investigations. Sensitivity analysis was carried out with “best estimates” of hydraulic roughness increased by between 20 and 100 per cent in the overbank areas and up to 20 per cent in the channel.

Initially, analysis was carried out to determine the critical storm durations for 5, 20 and 100 year ARI. Model runs were carried out with discharge hydrographs resulting from a range of storms between 9 and 24 hours duration, in conjunction with the “normal” semi diurnal tidal stage hydrographs as the downstream boundary.

Analyses were also carried out to assess the impacts of storm tides on flooding in the lower reaches of the two streams. The models were run with storm tide hydrographs of 1 in 5, 20 and 100 year return periods in conjunction with minor catchment floods. The tidal hydrographs were determined using generalised data presented in *“Floodplain Management Guideline No.5 Ocean Boundary Conditions”*, with more site-specific information on local water levels and wave set ups at the outlets of the respective streams supplied by DNR.

The analyses showed that in the lower reaches of the two streams, storm tides would control design flood levels for major flood events, whereas further upstream, there was a “cross over” of the derived water surface profiles and flooding derived by storm rainfall on the catchments controlled flood levels.

The procedure of adopting an "Envelope Curve" for design purposes, based on the higher flood levels derived from the two combinations of tide and catchment flood runoff, is well established in situations where there are insufficient data to undertake a more rigorous joint probability analysis of the two flood producing mechanisms. It has been adopted in the present investigation.

On Currambene Creek, the 24 hour storm in conjunction with a storm tide gave the highest peak levels for the 5 year ARI flood. For 10 year ARI and larger floods, the influence of storm tides was restricted to the lower estuary and catchment runoff resulting from 12 to 18 hour storm durations were generally critical. For the 100 year ARI event, the 12 hour storm was critical in the upper reaches of the stream. In the middle and lower areas where floodplain storage is important, the 18 hour storm was critical due to the larger volume of runoff produced with this event; and near Huskisson, storm tides were critical.

Moona Moona Creek has a much smaller catchment area, but a large volume of storage is available for the temporary storage of floodwaters in the swampy area upstream of the bridge at Elizabeth Drive. For the 100 year ARI, the 12 hour storm was generally critical in the upper reaches, but in the storage area upstream of Elizabeth Drive, the 18 hour storm was critical.

For the Probable Maximum Flood, the 6 hour storm gave the highest flood levels. This is the longest duration for which the Generalised Short Duration Method of Probable Maximum Precipitation (BOM, 2003), adopted in this investigation for derivation of the PMF, applies.

1.2 Outline of the Appendix

Section 2 deals with the selection of the hydraulic model and includes a brief description of the solution procedure. The main reasons for the adoption of the HEC-RAS modelling system are its computational efficiency and ready availability to potential users.

Section 3 of this Appendix deals with the developing and testing of the Currambene Creek hydraulic model. It discusses the layout of the hydraulic model of the creek and several tributaries which were included in the model to take into account the attenuating effects of the flood storage in the low lying areas on the southern side of Woollamia Road. The impacts of normal semi diurnal and storm tides in the lower estuary are also discussed.

Section 4 present the results of hydraulic modelling the design flood events nominated in the Brief, which range from 5 to 200 year ARI and include the Probable Maximum Flood.

Section 5 discusses the set up and testing of the Moona Moona Creek hydraulic model.

Section 6 presents the results of hydraulic modelling of design flood events on Moona Moona Creek ranging between 5 and 200 years ARI and the PMF.

Addenda A and **B** comprise tabulations of model results, including peak flood levels and the distribution of flows and velocities across the floodplain.

2. SELECTION OF HYDRAULIC MODEL

2.1 General

A model was required which could route flows through the main streams and their tributaries, and produce time series of flows, velocities and water surface elevations at nominated locations. The model was to be capable of analysing hydraulic conditions at the culvert and bridge crossings of the streams, and capable of adjustment in a future Floodplain Risk Management Study so that it could analyse the effects of possible modifications such as levees, channel enlargement, adjustments to bridge waterways or future land use changes on the floodplain, all of which could influence flooding behaviour.

Few commercially available hydrodynamic models contain all the features required for this present study. One however, HEC-RAS, has the required capabilities and is readily available to all potential model users at nominal cost.

HEC-RAS also contains routines for detailed analysis of bridge crossings and is superior to other software in this respect.

2.2 Brief Review of HEC-RAS Modelling Approach

HEC-RAS is a one-dimensional hydraulic modelling package developed by the Hydrologic Engineering Centre of the U.S. Army Corps of Engineers and has seen widespread application in Australia in recent years.

The hydrodynamic module of HEC-RAS contains an implicit finite difference computation of unsteady flows in rivers and estuaries. The formulations can be applied to branched and looped networks and quasi two-dimensional flow simulations on floodplains.

The computational scheme is applicable to vertically homogenous flow conditions ranging from steep creek systems to tidally influenced estuaries and lagoons.

The complete non-linear St. Venant equations of open channel flow are solved numerically between user defined grid arrangements (typically, cross section locations) at specified time intervals for given boundary conditions such as inflow stage and discharge hydrographs and tidal ranges. The modeller is also able to choose other flow descriptions such as; high order, fully dynamic; diffusive wave; kinematic wave and steady state.

The most successful and accepted procedure of solving the one dimensional unsteady flow equation is the four-point implicit scheme. Under this scheme, space derivatives and function values are evaluated at an interior point of the Δx – Δt computational box.

For the reach of the river, a system of simultaneous equations results which are solved by a matrix inversion technique. The simultaneous solution is an important aspect of this scheme because it allows information from the entire reach of the stream to influence the solution at any one computational point. Consequently, the model time step can be significantly larger than with "explicit" schemes which assess flows, velocities and water levels at time step $t + \Delta t$ by using information computed at time step t and proceeding sequentially from point to point along the model, either in the upstream or downstream direction.

Another feature of HEC-RAS is the efficient solution procedure adopted to solve the linearised finite difference equations at each time step. This procedure is known as the "Skyline Solution" of matrix inversion and allows a quick and efficient solution of complex network problems.

In the case of the Currambene Creek network, a simulation of two to three days duration was effected over a period of several minutes in real time.

3. CURRAMBENE CREEK MODEL

3.1 Model Structure

The Currambene Creek hydraulic model extended over the 15 km reach from the eastern side of the Princes Highway to the outfall to Jervis Bay at Huskisson. The creek is tidal over this reach. A schematic layout of the Currambene Creek HEC-RAS model is shown on **Figure 3.1**. This figure also shows the locations of tributary inflows to the model.

The model consisted of cross sections derived from ground survey. The choice of section locations depended on the need to accurately represent features on the floodplain which influence hydraulic behaviour (eg. changes in channel and floodplain dimensions, locations of tributary inflows as well as supplying adequate flood information in existing urban areas bordering the creek. The locations and orientations of the cross sections comprising the model are shown on **Figures 4.5** to **4.9** which present model results.

For the first 4 km downstream of the Highway, the creek channel is around 40 m wide with tree lined banks at elevations ranging between RL 2 – 3 m AHD. The floodplain in this reach is between 150 to 700 m in width and comprises a grass pasture cover on the overbank areas.

A major tributary, Georges Creek, joins the northern bank of the creek about 7 km downstream of the highway opposite Goodland Road. Georges Creek was modelled as sub-catchments S, T and U of the RORB model. Georges Creek contributes flows from a catchment area of 20 km^2.

Georges Creek was not included in the hydraulic model. The storage characteristics of the floodplain at its confluence with Currambene Creek have been modelled by the cross sections of that stream. However, there is presently no detailed survey available to define flooding conditions in Georges Creek upstream of the confluence.

The unnamed tributary of 19 km^2 area draining sub-catchments P, Q and R of the RORB model joins the southern bank near Goodland Road at RS 8167 ("RS" = River Station) and has been incorporated in the HEC-RAS model as Tributary 1. Inclusion of Tributary 1 and two others, Tributaries 2 and 3, allow the floodplain storage in the off-stream areas on the southern side of Woollamia Road to be incorporated in the hydraulic model. The culverts under Woollamia Road, which convey flows from Tributary 1 into Currambene Creek, have been included in the model. The road at this location is low lying with a centreline elevation of RL 1.28 m AHD and would be overtopped by minor freshes in Currambene Creek or major storm tides.

Goodland Road defines the commencement of the estuarine section of the creek, with a consequent widening of the floodplain and development of low lying off-stream storage areas on both sides. Below this point, the channel widens to 100 to 200 m and the extent of inundation in the event of major flooding could reach in excess of 1 km. The low lying areas remote from the main channel are covered with dense stands of mangroves and other vegetation which would offer considerable resistance to the passage of floodwaters.

Two other tributaries which join the southern bank of Currambene Creek have also been included in the model. Tributary 2 crosses Woollamia Road and joins the creek at model RS 6085, just upstream of Willowford Road. The culverts beneath Woollamia Road, which convey Tributary 2 flows to the creek, have been included in the model. The road centreline at the crossing is about RL 2.4 m AHD, between the 10 and 20 year ARI flood level. Tributary 2 does not have a large catchment area, but has been included so that the large flood storage on the southern side of Woollamia Road in this vicinity could be modelled.

Tributary 3 joins Currambene Creek on the downstream side of Edendale Street. This tributary has a catchment area of 4.5 km^2 comprising sub-catchments V and W of the RORB model. Tributary 3 crosses Woollamia Road and Edendale Street before joining the creek at model RS 2328. Both of the road crossings are incorporated in the HEC-RAS model. At Woollamia Road the elevation of the road centreline is RL 1.43 m AHD and is inundated by floods greater than 5 year ARI. The level of Edendale Street at the stream crossing is RL 1.45 m AHD.

Downstream of Willowford Road and Streamside Street, which are located on high ground on the southern side of the stream, Currambene Creek turns and flows in a generally southerly direction for about 3 km to outfall to Jervis Bay at Huskisson.

On its passage to Huskisson, the creek passes the townships of Wollamia on the western bank and Myola on the eastern side. The creek outlet is located at the southern extremity of Callala Beach, with Currambene Creek flowing along the rear of the frontal dune of the beach over the final kilometre of its length. The existing outlet is about 100 m wide and its invert level is currently at RL –4 m AHD.

The outlet is sheltered by a reef formation which extends into Jervis Bay. The low energy environment behind the reef has encouraged the development of the Callala Beach barrier spit. North of the reef, erosion of the beach berm and frontal dune has occurred in the past and a stabilisation (re-grassing) programme has been undertaken. Should the creek break through the beach to form a new outlet, however, such events as storm tides and wave action may have a more pronounced effect upstream of the creek's mouth than under present day conditions.

3.2 Currambene Creek Survey Data

A survey of the creek and overbank areas comprising 13 cross sections up to 2 km long was carried out for the 1983 Flood Study. For the present investigation an additional 12 within-bank cross sections of the channel were surveyed and the survey was extended into the overbank areas on the southern side of Woollamia Road.

A comparison of the two surveys indicated a variation of up to1 m in the invert levels in areas along Currambene Creek had taken place over the past 20 years. In the upper reaches of the creek, deposition has occurred probably as a result of sediment build up in the absence of significant recent flood flows. In the more downstream areas where tidal currents are the more significant drivers of sediment, there are alternating areas of deposition and scour of the channel. The creek is tidal over the modelled extent and does not have a continuous bed gradient in the downstream direction.

Several recently surveyed sections of the overbanks indicated little variation has taken place in natural surface levels of the above-tide areas over the years. The data base used to construct the hydraulic model of Currambene Creek comprised a mix of information based on the two surveys.

The hydraulic modelling approach used in this study adopted a “rigid boundary” model, with no account taken of any variation in cross sectional geometry which may occur over the duration of the flood. Implementation of a dynamic channel modelling approach would have required considerable historic flood and morphological data to have been used with confidence as the basis for determining design flood levels in this study. Such data are not usually available in routine catchment studies and were not available for the two streams which are the subject of this present investigation.

The geometric model of the creek and its floodplain was set up to give a realistic picture of flooding patterns under present day conditions.

Runs of the hydraulic model were carried out for Currambene Creek and Moona Moona Creek to test the sensitivity of results to scour at the respective entrances. These results are discussed later in **Sections 4.6** and **6.5** for the respective streams. Apart from those analyses, the absence of data precluded further investigation of the response of channel morphology to flooding.

3.3 Model Parameters

3.3.1 General

The main physical parameter for HEC-RAS is hydraulic roughness. There are other parameters, such as contraction and expansion head loss coefficients. These coefficients are of a hydraulic nature, but do not greatly affect computed flood levels in relatively slow moving streams such as Currambene and Moona Moona Creeks.

There are no historic flood level data available to assist with calibration of the model. Accordingly, roughness was estimated from site inspection, past experience and values contained in the engineering literature (Arcement and Schneider, 1984; Cowan, 1956; Barnes, 1967).

Hydraulic roughness is a rather subjective parameter and in the absence of calibrating flood data, it is necessary to test the sensitivity of the results to variations in design values.

In the upper reaches of the creek from the Princes Highway to a point about 5 km downstream, near the intersection of Knoll Parade and Woollamia Road, (i.e. from River Station 15206 to 10170), the creek channel ranges between 30 to 50 m in width and is lined with a row of trees on each side rising to pasture covered overbanks. The top width of flow during major floods would range between 100 m near the upstream end of this reach and 1,000 m at RS 10170. The depth of flow over the mainly grassed floodplain would be around 2 to 4 m. The floodplain is generally hydraulically smoother than the swampy, tree covered areas further downstream. Conversely the channel is hydraulically rougher than in the wider, estuarine areas.

The majority of the flow in the middle to lower reaches of Currambene Creek downstream of RS 10170 is conveyed in the vicinity of the channel, which is well defined with a smooth, sandy invert and where the hydraulic roughness may be assessed with reasonable accuracy from the literature. The off-stream areas, which are often heavily vegetated and where estimates of hydraulic roughness are much more uncertain, are important for the temporary storage of floodwaters and convey their proportion of the total flow at low velocities less than 0.5 m/s.

3.3.2 Roughness Values for Stream Channel

Although several factors affect the selection of an "n" value for the channel, the most important factors are the type and size of the materials that compose the bed and banks of the channel as well as its shape. Cowan, 1956 developed a procedure for estimating the effects of these factors.

In this procedure, the value of n may be computed by the following equation:

$$n = (nb + n1 + n2 + n3 + n4)\, m \quad \ldots\ldots\ldots\ 3.1$$

where	nb	=	a base value of n for a straight, uniform, smooth channel in natural materials
	n1	=	a value added to correct for the effects of surface irregularities
	n2	=	a value for variations in shape and size of the channel cross section
	n3	=	a value for obstructions to flow
	n4	=	a value for vegetation and flow conditions
	m	=	a correction factor for meandering of the channel

3.3.3 Roughness Values for Floodplain

It is usually necessary to determine roughness values for channels and floodplains separately. The fabric of a floodplain can be quite different from that of a channel. The physical shape of a floodplain is different and the vegetation covering a floodplain is typically different from that found in a channel.

Cowan's procedure was altered by Arcement and Schneider, 1984 to assess n values for a floodplain, using equation 3.1, where:

nb	=	a base value of n for the floodplain's natural bare soil surface, with no vegetation cover
n1	=	a value to correct for the effects of surface irregularities on the floodplain
n2	=	a value for variations in shape and size of the floodplain cross section
n3	=	a value for obstructions on the floodplain
n4	=	a value for vegetation on the floodplain
m	=	a correction factor for the sinuosity of the floodplain

Arcement and Schneider, 1984 also developed a "vegetation density" method for assessing n values for a heavily wooded floodplain. This method related roughness to the total frontal area of vegetation blocking the flow, the depth of flow and the drag coefficient of vegetation in the direction of flow. Application of this method requires a quantitative assessment of the diameters of trees and their density in terms of numbers per square metre of floodplain area.

Application of the vegetation density method is difficult in inaccessible areas such as the tidal zones of Currambene Creek bordering the main channel and was therefore not used in the present investigation.

Arcement and Schneider, 1984 also present photographs of densely vegetated floodplains for which roughness coefficients have been verified from historic flood data. These photographs were used together with application of equation 3.1 for estimating floodplain roughness.

Table 3.1 summarises the "best estimate" values of hydraulic roughness adopted for the investigation.

TABLE 3.1
"BEST ESTIMATE" OF HYDRAULIC ROUGHNESS VALUES
CURRAMBENE CREEK

	Channel	Floodplain
Princes Highway to Knoll Parade	0.055	0.065
Knoll Parade to Goodland Road	0.055	0.12
Goodland Road to Edendale Street	0.045	0.12
Edendale Street to Jervis Bay	0.030	0.12

3.4 Upstream Boundary Conditions

Discharge hydrographs derived from RORB provided the boundary conditions at the upstream end of the model. Lateral inflow hydrographs were added at various locations shown on **Figure 3.1** to account for runoff from the sub-catchments. In all, a total of 8 hydrographs were applied to the Currambene Creek hydraulic model for each flood event.

3.5 Downstream Boundary Conditions

Flooding in the lower portion of Currambene Creek (as well as Moona Moona Creek) may be influenced by both elevated ocean levels and catchment runoff. Elevated ocean levels are caused by storms which generate strong onshore winds, large waves and have low atmospheric pressure. These factors cause the ocean level at the shoreline or in entrances to estuaries or lakes to be elevated above normal tidal levels. The main components of this increased water level include wind setup, wave setup and inverse barometric setup. This abnormal elevation normally is characterised by a relatively rapid increase to a peak followed by a subsequent decline over periods up to 3 days. This elevation is superimposed upon the normal tidal variation.

The probability of the peak elevated ocean level occurring at similar times to the peak of the catchment runoff depends on the response time of the catchment, and the duration of the elevated ocean levels, assuming that both are caused by the same storm. The assumption of a common storm is not always valid or if so, not necessarily representative of the most severe conditions.

It would be expected that the lower estuary of Currambene Creek would fill relatively rapidly with the catchment runoff. This relatively short response time of the estuary due to rainfall, may increase the possibility for the coincidence of filling due to rainfall runoff and elevated ocean levels in cases of a common storm causing major inputs at both boundaries of the hydraulic models.

However, a rigorous joint probability analysis of the two flood producing mechanisms is not practicable, as there are negligible flood level data available in the lower reaches of the Currambene Creek estuary. Therefore, although stream flow data (at The Falls gauge) are available for the past 30 years and wind and tide data are available for the Jervis Bay area, it is not possible to accurately separate the impacts of these forcing influences, either separately or jointly, as there are no historic data on the response within the estuary.

Adoption of coincident 100 year ARI catchment flooding with 1 in 100 year storm tides would lead to a conservatively high estimate of 100 year ARI flood levels in the lower estuary. Accordingly, a pragmatic approach is often used whereby the "Envelope" of flood levels derived for a 100 year ARI catchment flood in conjunction with a lesser storm tide, or vice versa, is often adopted for defining 100 year ARI flood levels.

To assess the reach of the creek where flooding may be influenced by a backwater due to storm tides in Jervis Bay, the following procedure was adopted. This approach is supported by the *"Floodplain Management Guideline No. 5 Ocean Boundary Conditions*" supplied by DIPNR.

(1) Hydraulic modelling was carried out with "Normal" semi diurnal tidal hydrographs for a range of catchment floods. Initially, the 5 and 100 year ARI catchment floods were modelled for a range of storm durations.

(2) Modelling was then carried out with storm tidal hydrographs of 1 in 20 and 1 in 100 year return periods, in conjunction with a minor catchment flood of 5 year ARI. Storm tides were assessed using generalised procedures described in *Guideline No. 5*, in conjunction with a site specific assessment of design peak tailwater levels at the entrances of the two creeks prepared for DIPNR and entitled *"Estimates of Tail Water Levels in Currambene Creek and Moona Moona Creek".*

3.6 Hydraulic Model Testing

3.6.1 Catchment Flooding in Association with Normal Tides

Peak flood levels were derived for 5 and 100 year ARI events and condition (1) above, i.e. catchment runoff in conjunction with Normal Semi-Diurnal Tides. Several cases of hydraulic roughness were analysed: the "best estimate" values of **Table 3.1**; with roughness values in both the channel and floodplain values increased by 20 per cent *(Sensitivity Run 1)*; and also with roughness values in the floodplain only increased by 20 per cent *(Sensitivity Run 2).*

Additional sensitivity runs were carried out for the 100 year ARI. In *Sensitivity Run 3*, the hydraulic roughness of the floodplain downstream of Knoll Parade was increased by 50 percent above its best estimate from 0.12 to 0.18. In *Sensitivity Run 4*, the hydraulic roughness of the channel was reduced to 0.025 in the middle to lower reaches of Currambene Creek below Goodland Road, whilst maintaining the increased floodplain roughness as per Run 3. Finally, *Sensitivity Run 5* involved a doubling of the best estimate roughness of the floodplain in the heavily vegetated areas downstream of Knoll Parade to 0.25, with the channel roughness maintained at best estimate values.

Table 3.2 summarises the hydraulic roughness values used for the various sensitivity studies.

TABLE 3.2
SUMMARY OF SENSITIVITY ANALYSES FOR CURRAMBENE CREEK

Sensitivity Run	Roughness Condition
1	Channel and Floodplain Best Estimate Roughness increased by 20% over extent of model.
2	Channel remains at Best Estimate Roughness (**Table 3.1**) and Floodplain Roughness increased by 20% over extent of model.
3	Channel remains at Best Estimate Roughness over extent of model. Floodplain Roughness increased to 0.18 downstream of RS 10170 (Knoll Parade).
4	Floodplain Roughness increased downstream of RL 10170 (Knoll Parade) to 0.18. Channel Roughness reduced to 0.025 downstream of RS 8317 (u/s Goodland Road).
5	Channel remains at Best Estimate Roughness over extent of model. Floodplain Roughness increased to 0.25 downstream of RS 10170 (Knoll Parade).

Model runs were undertaken for the full range of storm durations from 9 to 24 hours. In the case of the 5 year ARI event the 24 hour storm duration was critical along the extent of the modelled reach and the results for that duration are shown in **Table 3.3**. For the 100 year ARI flood, the 12 hour storm was critical in the middle reaches and the 18 hour storm produced marginally higher peak water levels at the upstream and downstream ends. The results for the 12 hour storm are shown on **Table 3.4**.

Uncertainties associated with numerical hydraulic modelling are such that computed flood levels are usually rounded off to the nearest 100 mm. However, for comparison purposes, the results have been presented to two decimal places (i.e. to the nearest 10 mm) to highlight differences in the model results for the various cases analysed.

From **Table 3.4**, the peak flood levels are not particularly sensitive to variations in hydraulic roughness, except at the upper reaches of the creek and for the case where the roughness of both the channel and floodplain are increased *(Sensitivity Run 1)*. At the upstream end of the creek at RS 15206, the increases in peak flood levels are 0.31 m and 0.48 m for the 5 and 100 year ARI floods respectively. Further downstream at RS 7515, which is located near Goodland Road, the increase in peak flood levels amounts to 0.09 and 0.15 m for the two floods respectively.

Increasing hydraulic roughness of the floodplain increases flood levels in the middle reaches of the creek and results in a transfer of flow to the channel which becomes relatively more hydraulically efficient. There is also the tendency for the middle reaches of the creek to store flow on the floodplain, resulting in minor reductions in peak flood levels and flows in the lower reaches.

The above effects were evident to increasing degrees in the cases of *Sensitivity Runs 2, 3 and 5*. In the case of *Run 3* for example, there was an increase of up to 0.19 m in peak 100 year ARI flood levels in the vicinity of Goodland Road and coincident flows in the channel increased from 200 m^3/s to 240 m^3/s out of a total discharge conveyed by Currambene Creek of 760 m^3/s. Channel velocity increased from 0.7 to 0.8 m/s along with a minor reduction in overbank velocity from 0.17 m/s to 0.13 m/s (the above values apply at RS 7052, downstream of Goodland Road).

3.6.2 Discussion of Sensitivity Analysis

It is generally accepted that hydraulic roughness in comparatively wide estuarine channels such as exist in the middle to lower reaches of Currambene Creek may be estimated with a reasonable degree of accuracy. Estimation of appropriate values of hydraulic roughness in floodplains is more uncertain even though calibrated values are presented in the engineering literature.

The sensitivity studies presented in the previous section cover the range of values which could reasonably be adopted by practitioners in the absence of calibrating data. The results have demonstrated that variations in floodplain roughness within quite large limits do not result in large variations in peak flood levels. Similarly, the distributions of flows and velocities across the waterway areas of the various cross sections comprising the hydraulic model do not vary greatly with changes in roughness.

The stream is quite flat in terms of bed gradient, flow velocities are comparatively low and there is a large volume of flood storage attenuating the floodwave and damping out variations in flood levels resulting from variations in estimates of roughness. Consequently, although the lack of site specific historic flood data on Currambene Creek is unfortunate, it is considered that a reasonable level of confidence could be placed in the design flood levels derived in later sections of this Appendix.

TABLE 3.3
HEC-RAS TEST RUNS ON CURRAMBENE CREEK
5 YEAR ARI 24 HOUR STORM IN CONJUNCTION WITH NORMAL TIDES

River	Reach	River Station	Best Estimate Maning's 'n'	Sensitivity Run 1 Manning's 'n' 20% Increase Channel and Floodplain		Sensitivity Run 2 Manning's 'n' 20% Increase Floodplain Only	
			RL m AHD	RL m AHD	Δm	RL m AHD	Δm
Currambene	Curr 1	15206	3.82	4.13	0.31	3.86	0.04
Currambene	Curr 1	14633	3.67	3.95	0.28	3.70	0.03
Currambene	Curr 1	13934	3.40	3.65	0.25	3.45	0.05
Currambene	Curr 1	13372	3.11	3.31	0.20	3.17	0.06
Currambene	Curr 1	12390	2.83	2.99	0.16	2.90	0.07
Currambene	Curr 1	11308	2.75	2.89	0.14	2.80	0.05
Currambene	Curr 1	10170	2.66	2.81	0.15	2.70	0.04
				Knoll Parade			
Currambene	Curr 1	9196	2.57	2.70	0.13	2.58	0.01
Currambene	Curr 1	8317	2.28	2.39	0.11	2.30	0.02
Currambene	Curr 1	8167	2.12	2.22	0.10	2.15	0.03
Currambene	Curr 2	8117	2.12	2.22	0.10	2.15	0.03
Currambene	Curr 2	7517	2.03	2.12	0.09	2.07	0.04
				Goodland Road			
Currambene	Curr 2	7052	1.98	2.06	0.08	2.02	0.04
Currambene	Curr 2	6085	1.84	1.91	0.07	1.86	0.02
Currambene	Curr 3	6035	1.84	1.91	0.07	1.86	0.02
Currambene	Curr 3	5935	1.82	1.89	0.07	1.84	0.02
Currambene	Curr 3	4535	1.60	1.67	0.07	1.61	0.01
Currambene	Curr 3	3418	1.31	1.36	0.05	1.30	– 0.01
Currambene	Curr 3	2866	1.16	1.19	0.03	1.14	– 0.02
Currambene	Curr 3	2328	1.06	1.07	0.01	1.04	– 0.02
				Edendale Street			
Currambene	Curr 4	2288	1.06	1.07	0.01	1.04	– 0.02
Currambene	Curr 4	1094	0.94	0.94	0.00	0.93	– 0.01
Currambene	Curr 4	100	0.87	0.87	0.00	0.87	0.00
Currambene	Curr 4	0	0.87	0.87	0.00	0.87	0.00
Currambene	Curr 4	– 100	0.87	0.87	0.00	0.87	0.00
				Jervis Bay			
TRIB 1	T1	490	2.47	2.48	0.01	2.16	– 0.31
TRIB 1	T1	325	2.45	2.45	0.00	2.16	– 0.29
TRIB 1	T1	310	2.44	2.44	0.00	2.16	– 0.28
TRIB 1	T1	295		Wollamia Road Crossing			
TRIB 1	T1	280	2.12	2.23	0.11	2.16	0.04
TRIB 1	T1	0	2.12	2.22	0.10	2.15	0.03
TRIB 1	T1	– 400	2.12	2.22	0.10	2.15	0.03
TRIB 2	T2	852	1.90	1.96	0.06	1.92	0.02
TRIB 2	T2	832	1.90	1.96	0.06	1.92	0.02
TRIB 2	T2	812	1.90	1.96	0.06	1.92	0.02
TRIB 2	T2	806		Woollamia Road Crossing			
TRIB 2	T2	800	1.85	1.92	0.07	1.87	0.02
TRIB 2	T2	20	1.84	1.91	0.07	1.86	0.02

Δ(m) = the increase in peak flood levels compared with values derived from Best Estimate Manning's n.

TABLE 3.3
HEC-RAS TEST RUNS ON CURRAMBENE CREEK
5 YEAR ARI 24 HOUR STORM IN CONJUNCTION WITH NORMAL TIDES
(Con't)

River	Reach	River Station	Best Estimate Maning's 'n'	Sensitivity Run 1 Manning's 'n' 20% Increase Channel and Floodplain		Sensitivity Run 2 Manning's 'n' 20% Increase Floodplain Only	
			RL m AHD	RL m AHD	∆m	RL m AHD	∆m
TRIB 3	T3	1712	1.94	2.03	0.09	1.95	0.01
TRIB 3	T3	1212	1.94	2.03	0.09	1.95	0.01
TRIB 3	T3	997	1.79	1.86	0.07	1.79	0.00
TRIB 3	T3	982	1.77	1.83	0.06	1.77	0.00
TRIB 3	T3	972		Woollamia Road Crossing			
TRIB 3	T3	962	1.75	1.83	0.08	1.74	– 0.01
TRIB 3	T3	712	1.59	1.62	0.03	1.56	– 0.03
TRIB 3	T3	706		Edendale Street Crossing			
TRIB 3	T3	700	1.53	1.59	0.06	1.52	– 0.01
TRIB 3	T3	20	1.06	1.07	0.01	1.04	– 0.02

∆(m) = the increase in peak flood levels compared with values derived from Best Estimate Manning's n.

TABLE 3.4
HEC-RAS TEST RUNS ON CURRAMBENE CREEK
100 YEAR ARI 12 HOUR STORMS IN CONJUNCTION WITH NORMAL TIDES

River	Reach	River Station	Best Estimate Manning's 'n	Sensitivity Run 1 Manning's 'n' 20% Increase Channel and Floodplain		Sensitivity Run 2 Manning's 'n' 20% Increase Channel and Floodplain		Sensitivity Run 3 Manning's 'n' of Floodplain Increased to 0.18 d/s Knoll Pde		Sensitivity Run 4 Manning's 'n' of Floodplain Increased to 0.18 d/s Knoll Pde and Channel reduced to 0.025 d/s Goodland Road		Sensitivity Run 5 Manning's 'n' of Floodplain Increased to 0.25 d/s Knoll Pde	
			RL m AHD	RL m AHD	Δm	RL m AHD	Δm	RL m AHD	Δm	RL m AHD	Δm	RL m AHD	Δm
Currambene	Curr1	15206	6.58	7.06	0.48	6.68	0.10	6.59	0.01	6.57	– 0.01	6.60	0.02
Currambene	Curr1	14633	6.07	6.51	0.44	6.18	0.11	6.09	0.02	6.06	– 0.01	6.10	0.03
Currambene	Curr1	13934	5.48	5.80	0.32	5.59	0.11	5.50	0.02	5.45	– 0.03	5.52	0.04
Currambene	Curr1	13372	4.99	5.26	0.27	5.11	0.12	5.05	0.06	4.93	– 0.06	5.10	0.11
Currambene	Curr1	12390	4.56	4.80	0.24	4.69	0.13	4.68	0.12	4.48	– 0.08	4.78	0.22
Currambene	Curr1	11308	4.43	4.64	0.21	4.53	0.10	4.56	0.13	4.33	– 0.10	4.68	0.25
Currambene	Curr1	10170	4.32	4.53	0.21	4.40	0.08	4.48	0.16	4.21	– 0.11	4.61	0.29
Currambene	Curr1			Knoll Parade									
Currambene	Curr1	9196	4.16	4.35	0.19	4.21	0.05	4.28	0.12	3.89	– 0.27	4.37	0.21
Currambene	Curr1	8317	3.79	3.96	0.17	3.84	0.05	3.91	0.12	3.50	– 0.29	4.00	0.21
Currambene	Curr1	8167	3.58	3.76	0.18	3.65	0.07	3.74	0.16	3.36	– 0.22	3.86	0.28
Currambene	Curr2	8117	3.58	3.76	0.18	3.65	0.07	3.74	0.16	3.36	– 0.22	3.86	0.28
Currambene	Curr2	7517	3.37	3.52	0.15	3.45	0.08	3.56	0.19	3.27	– 0.10	3.71	0.34
Currambene	Curr2			Goodland Road									
Currambene	Curr2	7052	3.27	3.41	0.14	3.36	0.09	3.46	0.19	3.21	– 0.06	3.61	0.34
Currambene	Curr2	6085	3.09	3.22	0.13	3.16	0.07	3.23	0.14	2.97	– 0.12	3.33	0.24
Currambene	Curr3	6035	3.09	3.22	0.13	3.16	0.07	3.23	0.14	2.97	– 0.12	3.33	0.24
Currambene	Curr3	5935	3.07	3.20	0.13	3.13	0.06	3.20	0.13	2.94	– 0.13	3.29	0.22
Currambene	Curr3	4535	2.84	2.96	0.12	2.87	0.03	2.91	0.07	2.62	– 0.22	2.92	0.08
Currambene	Curr3	3418	2.44	2.56	0.12	2.44	0.00	2.42	– 0.02	2.26	– 0.18	2.34	– 0.10
Currambene	Curr3	2866	2.20	2.30	0.10	2.17	– 0.03	2.14	– 0.06	2.07	– 0.13	2.03	– 0.17
Currambene	Curr3	2328	2.00	2.10	0.10	1.96	– 0.04	1.91	– 0.09	1.85	– 0.15	1.84	– 0.16
Currambene	Curr3			Edendale Street									
Currambene	Curr4	2288	2.00	2.10	0.10	1.96	– 0.04	1.91	– 0.09	1.85	– 0.15	1.84	– 0.16
Currambene	Curr4	1094	1.62	1.70	0.08	1.57	– 0.05	1.51	– 0.11	1.48	– 0.14	1.43	– 0.19
Currambene	Curr4	100	0.92	0.95	0.03	0.91	– 0.01	0.90	– 0.02	0.88	– 0.04	0.90	– 0.02
Currambene	Curr4	0	0.87	0.87	0.00	0.87	0.00	0.87	0.00	0.87	0.00	0.87	0.00
Currambene	Curr4	– 100	0.87	0.87	0.00	0.87	0.00	0.87	0.00	0.87	0.00	0.87	0.00
Currambene	Curr4			Jervis Bay									
TRIB1	T1	490	3.60	3.77	0.17	3.66	0.06	3.78	0.18	3.38	– 0.22	3.86	0.26
TRIB1	T1	325	3.59	3.76	0.17	3.66	0.07	3.76	0.17	3.37	– 0.22	3.86	0.27
TRIB1	T1	310	3.59	3.76	0.17	3.66	0.07	3.76	0.17	3.37	– 0.22	3.86	0.27
TRIB1	T1	295		Woollamia Road Crossing									
TRIB1	T1	280	3.59	3.76	0.17	3.66	0.07	3.76	0.17	3.37	– 0.22	3.86	0.27
TRIB1	T1	0	3.59	3.76	0.17	3.65	0.06	3.74	0.15	3.37	– 0.22	3.86	0.27
TRIB1	T1	– 400	3.58	3.76	0.18	3.65	0.07	3.74	0.16	3.38	– 0.20	3.86	0.28
TRIB2	T2	852	3.10	3.22	0.12	3.16	0.06	3.24	0.14	2.97	– 0.13	3.33	0.23
TRIB2	T2	832	3.10	3.22	0.12	3.16	0.06	3.24	0.14	2.97	– 0.13	3.33	0.23
TRIB2	T2	812	3.10	3.22	0.12	3.16	0.06	3.24	0.14	2.97	– 0.13	3.33	0.23
TRIB2	T2	806		Woollamia Road Crossing									
TRIB2	T2	800	3.09	3.22	0.13	3.16	0.07	3.24	0.15	2.97	– 0.12	3.33	0.24
TRIB2	T2	20	3.09	3.22	0.13	3.16	0.07	3.23	0.14	2.97	– 0.12	3.33	0.24
TRIB3	T3	1712	2.59	2.69	0.10	2.61	0.02	2.63	0.04	2.63	0.04	2.66	0.07
TRIB3	T3	1212	2.59	2.69	0.10	2.61	0.02	2.63	0.04	2.63	0.04	2.66	0.07
TRIB3	T3	997	2.23	2.31	0.08	2.25	0.02	2.26	0.03	2.24	0.01	2.28	0.05
TRIB3	T3	982	2.18	2.25	0.07	2.20	0.02	2.21	0.03	2.18	0.00	2.23	0.05
TRIB3	T3	972		Woollamia Road Crossing									
TRIB3	T3	962	2.18	2.25	0.07	2.20	0.02	2.21	0.03	2.18	0.00	2.23	0.05
TRIB3	T3	712	2.11	2.19	0.08	2.09	– 0.02	2.08	– 0.03	2.06	– 0.05	2.07	– 0.04
TRIB3	T3	706		Edendale Street Crossing									
TRIB3	T3	700	2.10	2.19	0.09	2.09	– 0.01	2.08	– 0.02	2.06	– 0.04	2.07	– 0.03
TRIB3	T3	20	2.00	2.10	0.10	1.96	– 0.04	1.91	– 0.09	1.85	– 0.15	1.84	– 0.16

Note: Δ (m) = The increase in peak flood levels compared with values derived from Best Estimate Manning's n.

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3.6.3 Storm Tides in Association with Minor Catchment Floods

This section deals with Envelope condition (2) of **Section 3.5**, i.e. major Storm Tide events in conjunction with a minor 5 year ARI catchment flood.

According to the generalised design tidal hydrographs shown in *Guideline No. 5*, the 100 year ARI storm tide rises to a peak of RL 2.6 m AHD over a period of 16 hours and then attenuates to the normal semi-diurnal tidal pattern over the following 24 hours. The storm tide hydrograph is superimposed on the semi diurnal tide and therefore has two peaks of RL 1.0 m and RL 2.6 m AHD.

The localised estimates of peak Storm Tide levels contained in *"Estimates of Tail Water Levels in Currambene Creek and Moona Moona Creek"* combine water levels in Jervis Bay and local wave set ups at the entrances to the two creeks. Estimated tailwater levels from that analysis are shown on **Table 3.5**.

TABLE 3.5
PEAK TAILWATER LEVELS
m AHD

Location	Average Recurrence Interval Years	
	20	100
Currambene Creek	1.705	1.885
Moona Moona Creek	1.905	2.085

The above peaks are slightly lower than the peak Storm Tides presented in *Guideline No. 5* and have been used in the present investigation, with interpolation and extrapolation as required, for the other recurrence intervals for which design flood estimates are required.

The results of using these Storm Tide hydrographs as the downstream boundary and running the HEC-RAS model for 100, 20 and 5 year ARI storms between 12 and 24 hours durations are presented in **Figures 3.2 to 3.4**. Also shown on these diagrams for comparison purposes are water surface profiles for 100 year ARI catchment floods in conjunction with Normal Tides (i.e. Envelope condition (1) of **Section 3.5**).

The storm tidal backwater extends upstream to RS 2288 and controls 100 year ARI design flood levels from the outlet to the village of Myola on the left or eastern bank of Currambene Creek. Upstream of this location, catchment flooding controls design flood levels.

For the 20 year ARI, storm tides control design flood levels to RS 2866, several hundred metres upstream of Edendale Street.

For the 5 year ARI, the water surface profile resulting from the 5 year ARI local catchment flood in conjunction with a small Storm Tide tailwater is marginally above the water surface profile resulting from the same catchment flood and a normal semi diurnal tidal hydrograph as the tailwater.

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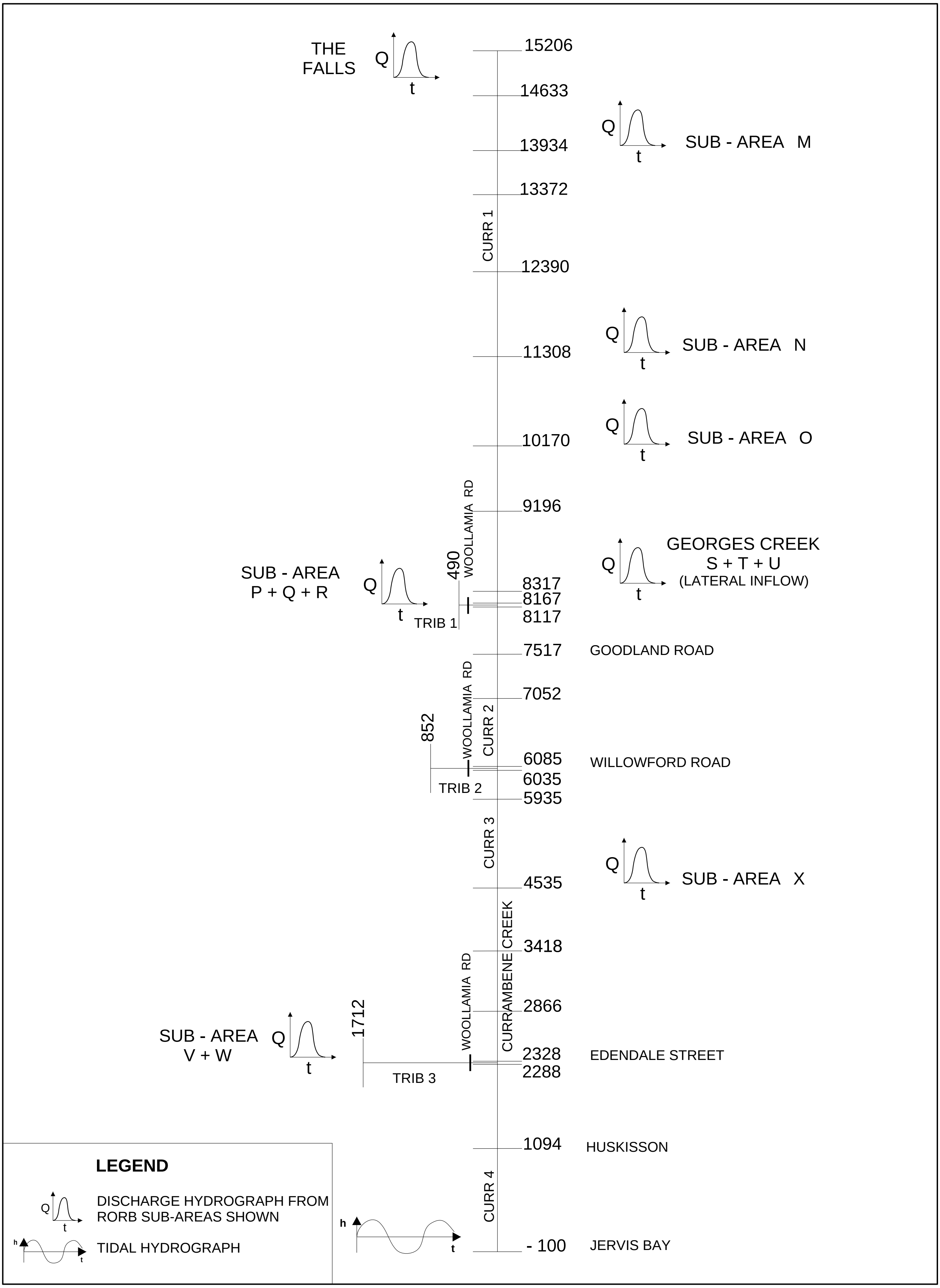
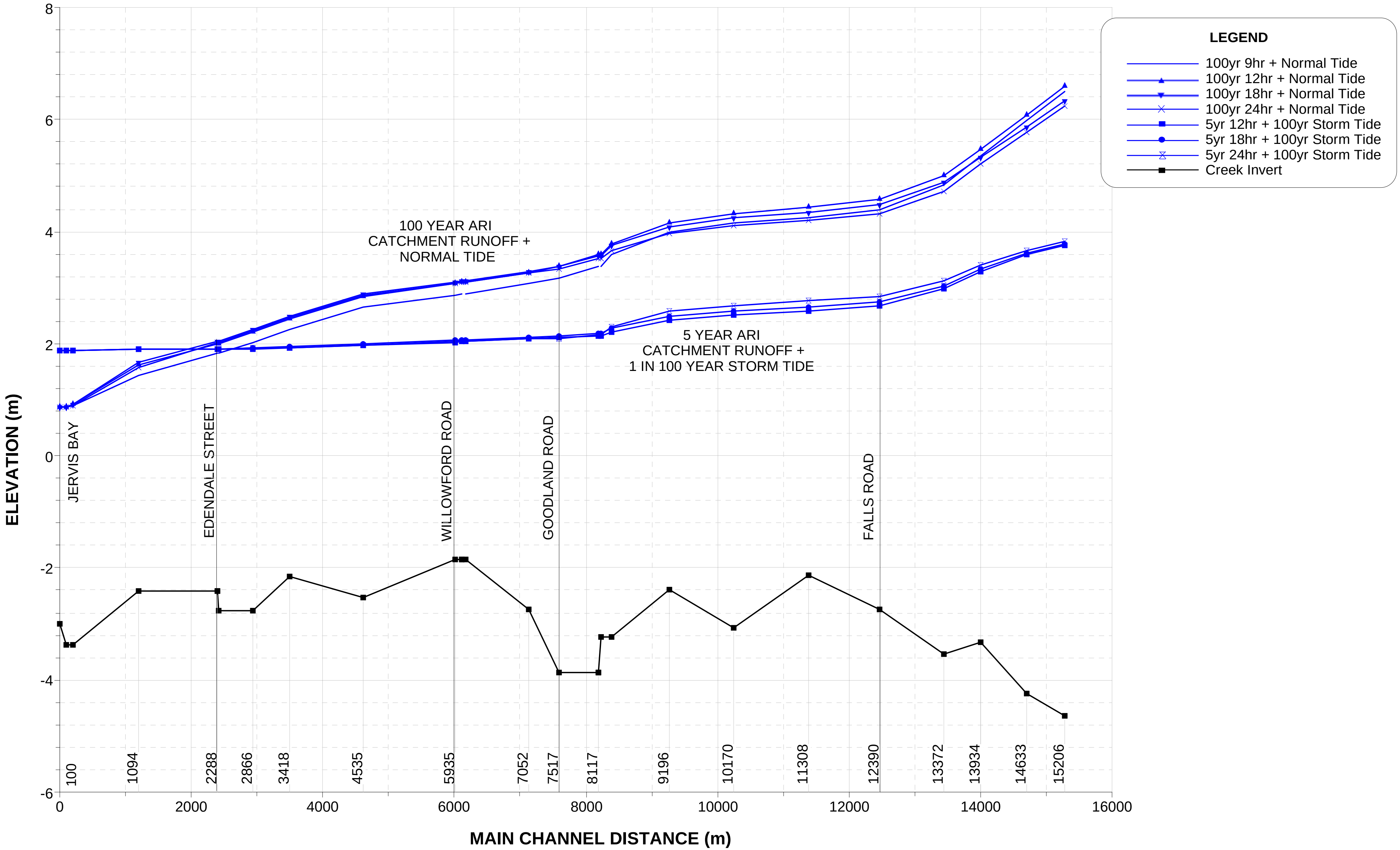


CURRAMBENE CREEK AND MOONA MOONA CREEK FLOOD STUDIES

Figure 3.1

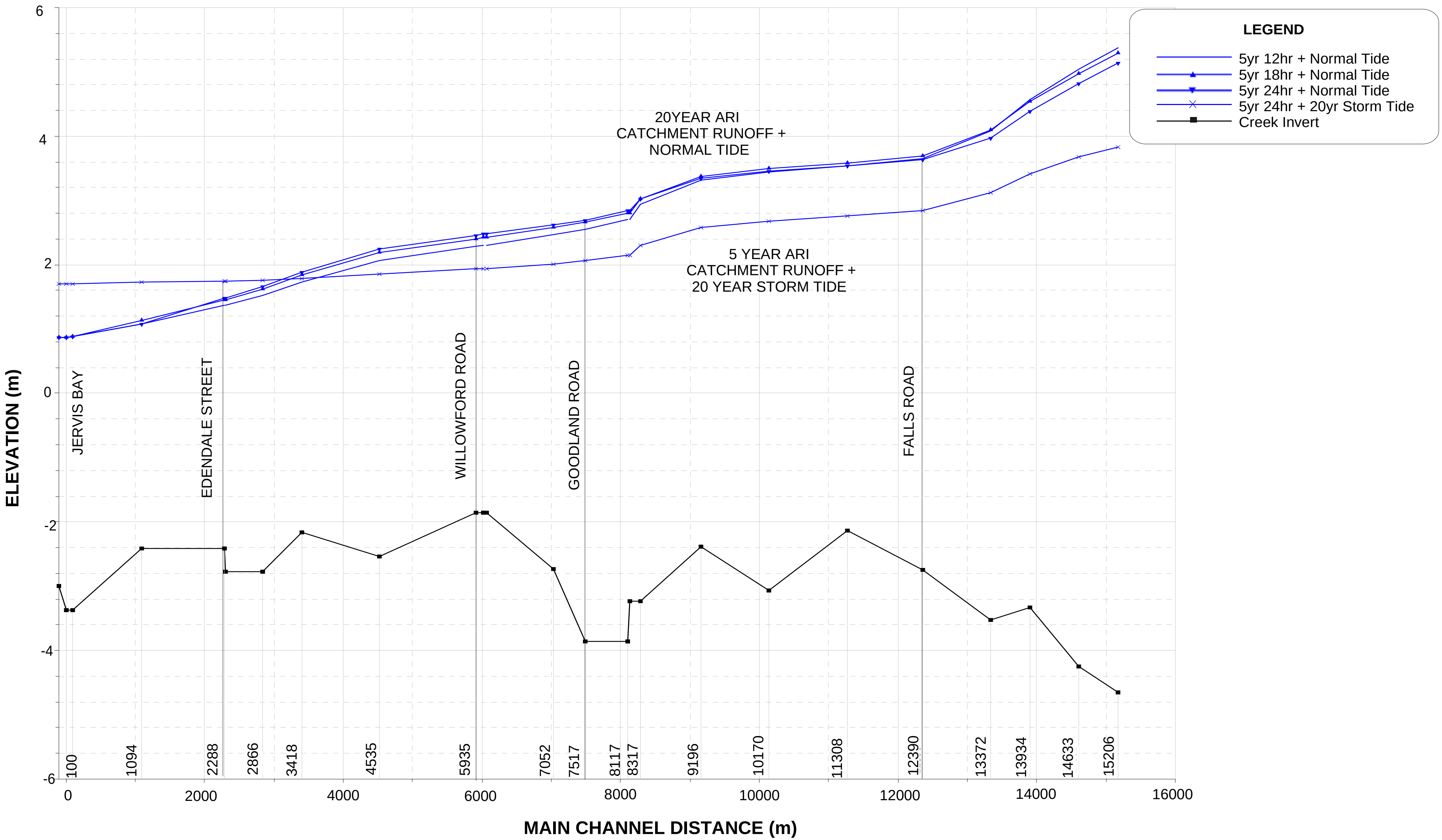
CURRAMBENE CREEK
HEC - RAS SCHEMATIC LAYOUT



CURRAMBENE CREEK AND MOONA MOONA CREEK FLOOD STUDIES

Figure 3.2

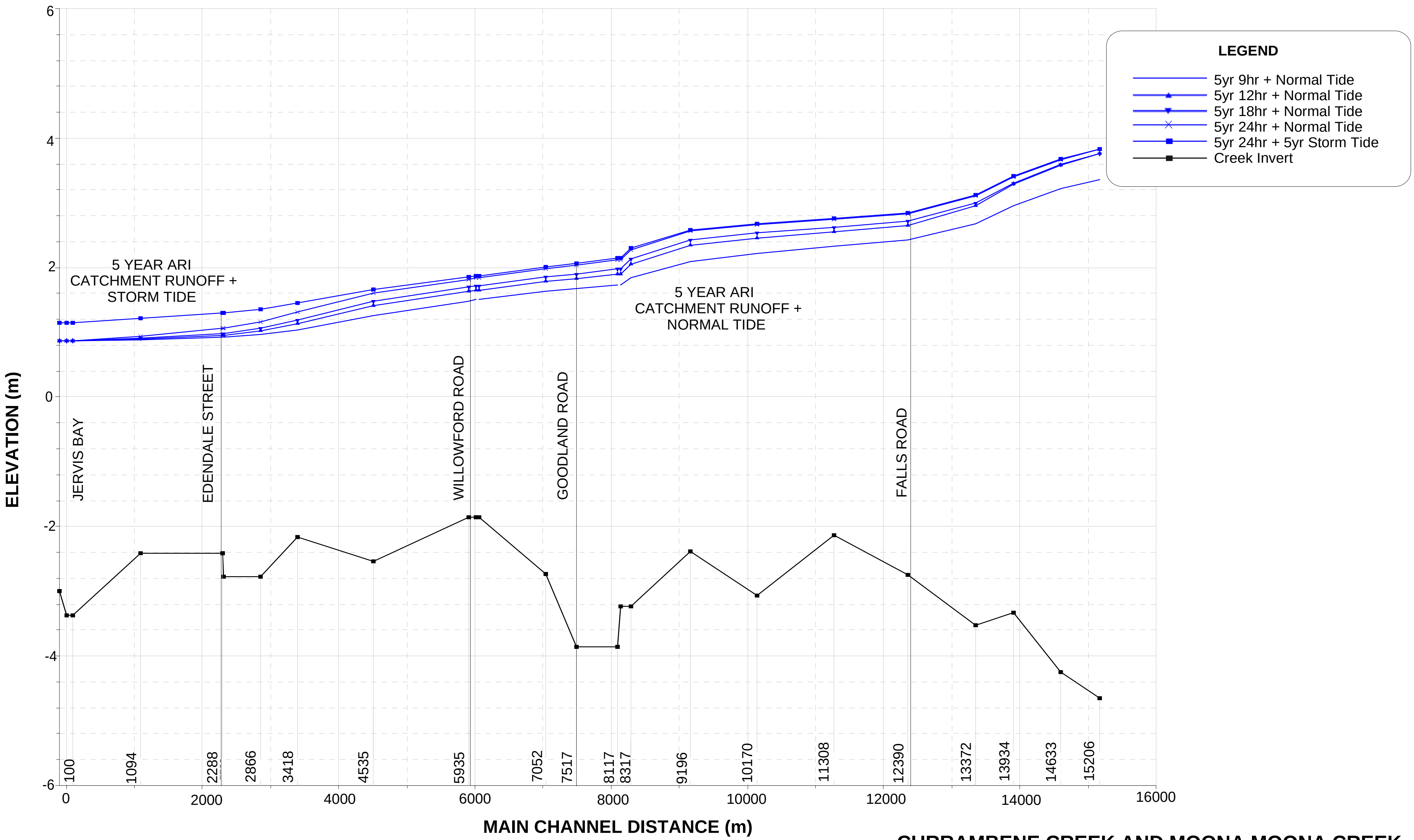
CURRAMBENE CREEK
100 YEAR ARI FLOOD ENVELOPE



CURRAMBENE CREEK AND MOONA MOONA CREEK FLOOD STUDIES

Figure 3.3

CURRAMBENE CREEK
20 YEAR ARI FLOOD ENVELOPE



CURRAMBENE CREEK AND MOONA MOONA CREEK FLOOD STUDIES

Figure 3.4

CURRAMBENE CREEK
5 YEAR ARI FLOOD ENVELOPE

4. CURRAMBENE CREEK – RESULTS OF MODELLING DESIGN FLOODS

4.1 General

This chapter describes the results of modelling the design flood events of 5 to 200 year ARI and the Probable Maximum Flood. The procedure involved running storms for each frequency ranging between 9 hours and 24 hours durations, both with a "Normal Semi-Diurnal Tide" and "Storm Tides" as the downstream boundary condition and selecting the flood envelope (i.e. the highest flood level at each model cross section) as the design peak.

The results of this procedure have been presented in this Appendix as follows:

Figures 4.1 to 4.4 shows the flood envelopes for each flood along the 15 km length of Currambene creek and on the three tributaries modelled.

Figure 4.5 is a plan of the Currambene Creek floodplain showing the indicative extents of inundation and peak flood levels for the 10 and 100 year ARI floods and the PMF.

Figures 4.6 and 4.7 relate to the 10 and 100 year ARI floods. These figures show the subdivision of the inundated areas into floodway and flood storage zones and also include flood contour and flow velocity information at typical locations.

Figures 4.8 and 4.9 present provisional flood hazard information for the 10 and 100 year ARI floods, based on flow velocity and depth data obtained from the hydraulic analyses.

Addendum A presents tabulated data on flood levels and distribution of flow and velocities in the waterway sections. The flow and velocity data shown in this Addendum are coincident with the occurrence of the peak flood levels at the respective cross sections. In the upper and downstream reaches of Currambene Creek, the occurrence of peak discharge is coincident with peak flood level. In the middle reaches where flood storage attenuates the flood wave, the peak flow occurs up to 2 hours prior to the occurrence of the peak flood level. This is the manifestation of the "loop rating curve" effect which often occurs in wide floodplains and low gradient streams.

By inspection of the data presented in **Addendum A** for the three tributary streams, and also the water surface profiles, it will be noted that peak flood levels are essentially backwater levels projected up the tributary streams from their junctions with Currambene Creek.

Tributaries 1 and 3 convey flows from sub-catchments of considerable catchment area, however, their peak flows occur up to 6 hours prior to the arrival of the peak on Currambene Creek. In the 100 year ARI flood, the peak discharge from the Tributary 1 sub-catchments amounts to 220 m^3/s and occurs when the Currambene Creek water level at the junction with this stream is 400 mm below its subsequent peak. On Tributary 3, the corresponding peak discharge is 58 m^3/s, occurring when the water level in Currambene Creek at the junction is 250 mm below its subsequent peak.

Although the tributaries contribute flood storage and attenuate the floodwave on Currambene Creek, they also function as floodways for the conveyance of runoff from their respective sub-catchments in the early stages of the design floods.

The extents of inundation shown on **Figure 4.5** are necessarily indicative only. On Currambene Creek and the three tributaries they are based on flood levels derived at the surveyed cross sections, as well as limited survey along Woollamia Road in the vicinity of the tributary crossings and surveyed road levels along Woodland Road, Willowford Road, Streamside Street and Edendale Street. A line of levels was also surveyed along Myola Road on the eastern bank. In addition, the GIS data obtained from CMA in Bathurst contained limited contour information along a portion of the area on the western side of Woollamia Road, which allowed indicative mapping of the extent of inundation in the three tributary areas.

Whilst the flood level and velocity data derived from the analyses are accurate at the sections comprising the model, the flood extent diagrams should not be used to determine the flood affectation in individual allotments.

4.2 Discussion of Results

Over the first 4 km from the upstream end of the model to the intersection of Woollamia Road and Falls Road, Currambene Creek floodplain is confined to an extent of 700 m. Over this reach the creek, although tidal, is characteristic of an upland stream, with tree lined banks leading to pasture covered floodplain.

Most of the flow is conveyed in the vicinity of the channel although the left (northern) floodplain conveys a progressively higher proportion of flow with increasing flood magnitude. For the 100 year ARI, flow velocities in the main channel over this region are in the range 2.1 to 0.6 m^3/s reducing in the downstream direction and flow velocities on the floodplain are generally around 0.5 m/s.

Downstream of the Falls Road intersection, the extent of the floodplain widens to over 1100 m prior to a local narrowing to 700 m at RS 9196 resulting from a ridge of high ground extending northwards from Woollamia Road.

A short distance downstream of this location, the major tributary, conveying contributions to flow from RORB sub-areas P, Q and R, enters Currambene Creek from the southern side just upstream of Goodland Road. The off stream storage in this tributary is modelled by Tributary 1. On the northern side, Georges Creek joins Currambene Creek. The extent of inundation on Georges Creek upstream of the junction is uncertain due to lack of survey data.

Together, Georges Creek and Tributary 1 contribute about 400 m^3/s of peak flow at the 100 year ARI. However, the large volume of flood storage contained in the main stream and off stream storage areas on both sides of the creek largely offsets the increase in peak flows in Currambene Creek. The extent of inundation continues about 700 m into the low lying areas on the southern side of Woollamia Road, which would be overtopped by about 2.3 m at the peak of the 100 year ARI and by 0.9 m at the 5 year ARI (**Table 4.1**).

At Goodland Road (RS 7517), the floodplain is confined to a width of around 700 m by the promontory of high ground on which the road is located. Downstream of this location, the two tributaries modelled as Tributary 2 and Tributary 3 join the south bank of Currambene Creek. At Tributary 3, which conveys contributions from RORB sub-areas V and W, the inundation extends about 1.2 km south of Woollamia Road. The width of inundation on the main arm of Currambene Creek extends across the floodplain a distance of about 1.5 km to the north-east of Woollamia Road.

In the event of a 100 year ARI, the crossings of Woollamia Road at Tributary 2 would be inundated by about 0.9 m and at Tributary 3 by 1.3 m. Woollamia Road would be on the point of being inundated at the Tributary 3 crossing in the event of the 5 year ARI flood. The threshold frequency for overtopping the crossing at Tributary 2 is about the 10 year ARI.

In the middle to lower reaches of Currambene Creek, flow velocities gradually reduce as the extent of inundation and the attenuating effects of the flood storage increase. Channel velocities are generally less than 1 m/s and overbank velocities around 0.2 m/s at the 100 year ARI. Between the outlet and Myola, where design flood levels are controlled by the Storm Tide scenario, and reach RL 1.9 m at the 100 year ARI, flow velocities are less than 0.2 m/s in the channel and negligible in the overbanks.

TABLE 4.1
LOCATIONS OF OVERTOPPING OF
WOOLLAMIA ROAD

Location	Depth of Flow over Road – m		Threshold Frequency at which flow commences to overtop road ARI - years
	5 Year ARI	100 Year ARI	
Tributary 1 West of Goodland Road Intersection	0.9	2.3	< 5
Tributary 2 East of Goodland Road Intersection	–	0.9	10 (approx)
Tributary 3 North of Edendale Street Intersection	–	1.3	5

4.3 Floodwave Features

The flood wave takes about 6 to 7 hours to travel the 15 km modelled reach of Currambene Creek, equivalent to an average celerity of 0.4 to 0.5 m/s.

The attenuating effects of the flood storage in the overbank areas offset the increase in flows arising from the contributions from the major tributaries. **Table 4.2** shows peak flows along Currambene Creek for the 100 year ARI 12 hour storm, which is critical for flood levels over the upper and middle reaches. This table also shows peak flows generated by the RORB catchment model, which as mentioned in **Appendix B,** contains a lumped storage volume versus discharge relationship to simulate the effects of the floodplain storage.

The peak flows as derived from HEC-RAS which models storage at each cross section, do not vary greatly along the length of the model. In RORB, the lumped storage is located downstream of the entry of Georges Creek and Tributary 1 (near RS 5935 of the HEC-RAS model). Consequently, there is a large reduction in flow between RS 5935 and RS 2826 in the RORB results, which is not reproduced in HEC-RAS. However, further downstream, peak flows derived from the two modelling approaches are similar.

TABLE 4.2
PEAK FLOWS ON CURRAMBENE CREEK
100 YEAR ARI, 12 HOUR STORM
Values in m^3/s

Location	HEC-RAS Model with Distributed Flood Storage	RORB Hydrologic Model with Lumped Flood Storage
RS 15206 (d/s Princes Highway)	753	753
RS 9196 u/s Georges Creek and Tributary 1)	712	875
RS 5935 (Willowford Road)	713	1190
RS 2866 Edendale Street/Myola	670	766
RS 1094	679	680
Outlet at Callala Beach	655	770

Similar comparative results were achieved in the case of the Probable Maximum Flood as shown on **Table 4.3**.

TABLE 4.3
PEAK FLOWS ON CURRAMBENE CREEK
PROBABLE MAXIMUM FLOOD
Values in m^3/s

Location	HEC-RAS Model with Distributed Flood Storage	RORB Hydrologic Model with Lumped Flood Storage
RS 15206 (d/s Princes Highway)	1970	1970
RS 9196 u/s Georges Creek and Tributary 1)	1920	2190
RS 5935 (Willowford Road)	2010	2810
RS 2866 Edendale Street/Myola	1905	1805
Outlet at Callala Beach	1853	1810

4.4 Hydraulic Categorisation of the Floodplain

For the purposes of the Floodplain Development Manual, 2005, there are three hydraulic categories of flood prone land:

- Floodways;
- Flood storage; and
- Flood fringe

Floodways are those areas where a significant volume of water flows during floods and are often aligned with obvious natural channels. They are areas that, even if partially blocked, would cause a significant increase in flood levels and/or a significant redistribution of flow, which may in turn adversely affect other areas. They are often, but not necessarily, areas with deeper flow or areas where higher velocities occur.

Flood Storage areas are those parts of the floodplain that are important for the temporary storage of floodwaters during the passage of a flood. If the capacity of a flood storage area is substantially reduced by, for example, the construction of levees or by landfill, flood levels in nearby areas may rise and the peak discharge downstream may be increased. Substantial reduction of the capacity of a flood storage area can also cause a significant redistribution of flood flows.

Flood Fringe is the remaining area of land affected by flooding, after floodway and flood storage areas have been defined. Development in flood fringe areas would not have any significant effect on the pattern of flood flows and/or flood levels.

In determining appropriate hydraulic categories, it is important that the cumulative impact of progressive development be evaluated, particularly with respect to floodway and flood storage areas. Whilst the impact of individual developments may be small, the cumulative effect of the ultimate development of the area can be significant and may result in unacceptable increases in flood levels and flood velocities elsewhere in the floodplain.

In practice, development of flood liable areas bordering a stream usually proceeds from the shallower flood fringe areas towards the channel. The FDM, 2005 provides guidelines on determining the boundary between the floodway and flood storage zones using the hydraulic model and what may be termed “encroachments” into the floodplain. In this approach, conceptual vertical boundary lines are progressively moved into the floodplain from both sides thereby constricting the flow to the degree where peak flood levels and peak flows are increased anywhere within the extent of the model by a specific amount. The FDM, 2005 suggests a limiting increase of 0.1 m in peak flood levels and 10% in peak downstream discharges.

The portions of the floodplain on the landward side of the encroachment lines giving the above limiting increases in flood peak represent that part of the floodplain which may be removed both in terms of conveyance capacity and flood storage without causing excessive adverse impacts on flood behaviour. The locations of the encroachment lines on each side of the stream represent the boundary between the floodway and the flood storage zone.

The hydraulic model was used to estimate the floodway/flood storage boundary for both the 100 year ARI and 10 year ARI floods.

Initially the model was run with the whole of the floodplain external to the channel filled. As expected this simulation resulted in large increases in flood levels along the length of the creek. For the 100 year ARI flood, peak flood levels would be raised by up to 4.9 m, with the largest afflux being experienced in the upper reaches of the modelled reach near RS 12390. For the 10 year ARI the afflux would be smaller, reaching about 3.6 m at the same location.

Simulations were then carried out with the encroachment limits progressively widened. In the case of the 100 year ARI flood, the encroachment limits on Currambene Creek were positioned at the extents of flooding reached by the 5 and 10 year ARI floods. **Table 4.4** shows the maximum increase in peak flood levels and percentage increases in peak downstream flows.

TABLE 4.4
RESULTS OF ENCROACHMENT ANALYSIS
ON CURRAMBENE CREEK

Encroachment Limit on Currambene Creek	Maximum Increase in Peak Flood Level m	Maximum Increase in Downstream Discharge %
5 year ARI Flood Extent	0.36	7
10 year ARI Flood Extent	0.13	6

For the purpose of floodway delineation on Currambene Creek, the 10 year ARI flood extent was adopted as the width of the floodway for the 100 year ARI on the basis of the results presented in **Table 4.4**.

As discussed in **Section 4.1**, the tributaries convey significant flow from the respective sub-catchments prior to the occurrence of the flood peak on Currambene Creek and therefore do not solely represent flood storage areas. A sensitivity study with the encroachment limits on the tributaries fixed to allow a 50 m width to convey the sub-catchment flows resulted in significant increases in peak flood levels at their upstream limits compared with unobstructed conditions. For example, on the headwaters of Tributaries 1 and 3, the peak levels would be increased by 0.3 m and 2.9 m respectively.

In the future *Floodplain Risk Management Study* for Currambene Creek, permissible development in the storage areas will be examined in the context of formulating a local Flood Policy DCP for the area. At that time, the effects on flooding patterns of encroachment into the floodplain (including the Tributary areas) will be examined in more detail, along with hydraulic modelling of Georges Creek.

For the purposes of the present study, the 10 year ARI flood extent has been adopted as the width of 100 year ARI floodway within the three tributary areas also.

A similar procedure led to the adoption of the 5 year ARI flood extent being adopted as the floodway for the 10 year ARI flood. These hydraulic categorisations of the floodplain of Currambene Creek are shown on **Figures 4.6** and **4.7**.

4.5 Definition of Provisional Flood Hazard

Flood hazard categories may be assigned to flood affected areas in accordance with the procedures outlined in the Floodplain Development Manual, 2005.

Flood prone areas may be provisionally categorised into *Low Hazard* and *High Hazard* areas depending on the depth of inundation and flow velocity. Flood depths as high as 0.8 m in the absence of any significant flow velocity represent Low Hazard conditions. Similarly, areas of flow velocities up to 2.0 m/s but with minimal flood depth also represent *Low Hazard* conditions.

Following a review of the modelled distribution of flows and velocities at the various model cross sections a depth of 1 m was adopted in the present investigation as the boundary between *Low* and *High Hazard* zones. Provisional flood hazard diagrams for the 10 and 100 year ARI floods are shown on **Figures 4.8** and **4.9**.

As noted in the Floodplain Development Manual, 2005, other considerations such as rate of rise of floodwaters and access to high ground for evacuation from the floodplain should also be taken into consideration before a final determination of Flood Hazard can be made. These factors are normally taken into account in the *Floodplain Risk Management Study* for the catchment, which is the next stage in the flood management process for the area.

4.6 Impacts of Entrance Scour on Flood Levels

4.6.1 Design Flood Events

The hydraulic analyses described in this study have been carried out assuming that the dimensions of the channel are maintained over the duration of the simulation i.e. on the assumption of a “rigid boundary” for all cross sections comprising the model.

For sand bed channels such as Currambene Creek and Moona Moona Creek, it is likely that considerable movement of the bed may take place over the duration of the flood event, with scour occurring on the rising limb of the flood hydrograph and deposition and filling of the scour holes on the recession limb, as flow velocities and associated tractive forces reduce.

Further opening of the entrance to Currambene Creek could occur, with the actual scour depending on the tailwater level in Jervis Bay. Some hydraulic modelling software packages are capable of modelling the erosion process, which is dynamic in nature. However, the results achieved are heavily dependent on the assumptions made and model parameters adopted, as is the case with all models of natural systems. With the long duration storms which were found to maximise flows on the two streams, it is likely that the openings would have been scoured in the early stages of the flood so that by the time the peak arrived, the erosion process would have been largely completed.

This would especially have been the case for an intermittently opened entrance such as exists on Moona Moona Creek outlet downstream of Elizabeth Drive.

To some extent, the uncertainties regarding the likely scour at the two openings are academic, as the Storm Tide scenario was found to govern design flood levels in the lower reaches of the two streams.

Scouring of the Currambene Creek entrance would not have an impact on the design flood profiles. For example, In the case of the 100 year ARI, the design flood scenario comprises a 100 year storm tide in association with a minor 5 year ARI catchment flood. Hydraulic modelling showed that at the flood peak, the velocity at the entrance would be only 0.2 m/s, with a waterway area of 527 m^2 and width of opening 207 m. Because of the mild velocity, the flood gradient would be quite low and would not be significantly affected by scour.

4.6.2 Probable Maximum Flood

As discussed previously, the scenario adopted for the PMF comprised a 1 in 100 year Storm Tide in association with a local catchment runoff derived from the PMP storm of 6 hours duration.

At the outlet of Currambene Creek at Jervis Bay, the flow velocity associated with the peak of the flood was 3.6 m/s and the resulting friction slope was 0.2%, resulting in an increase in upstream water surface level to RL 3.52 m at RS 1094. The elevation of the spit on the northern side of the creek at the end of Callala Beach is no lower than RL 4 m and consequently, it would not be overtopped. However, it is likely that even with the damping effect of the high tailwater level, the high flow velocity may promote scour of the outlet.

To estimate the effects on upstream flood levels, a simulation was carried out assuming that the outlet would open to a trapezoidal section, with the width of flow at storm tide level increased from 200 m to about 400 m, with the invert maintained at about RL −4 m. The water surface profile resulting from this assumption may be compared with the rigid boundary assumption on **Figure 4.10**.

The reduction in flow velocities resulting from scour reduces the peak flood levels upstream as far as RS 5935, near Willowford Road.

At RS 1094, the reduction in peak water level is 1.16 m, from RL 3.52 m to RL 2.36 m. Further upstream at Edendale Street, the assumed entrance scour reduces peak flood levels by 0.23 m.

As noted in **Appendix B**, there were reservations that limitations in the GSDM method may have resulted in estimates of PMF peak discharges which were on the low side. The assumption that a “rigid boundary” is maintained for the outlet cross section would tend to offset the effects on the PMF water surface profile of a low estimate of peak flow.

It is considered that in order to provide a safe estimate of flood levels for the extreme flood event, the higher flood levels resulting from the “rigid boundary” ie No Scour assumption on **Figure 4.10** should be adopted.

4.7 Comparison of Results with Previous (1983) Flood Study

Peak flood levels computed in the present study are considerably lower than in the 1983 Flood Study, except at the outlet where storm tides control flooding. **Table 4.5** shows a comparison of peak 100 year ARI levels at representative locations. The main reasons for the reduction in peak flood levels are:

(1) **The hydraulic modelling approach adopted.** The 1983 study adopted a steady state modelling approach based on the HEC-2 modelling system. Peak flows derived from a hydrologic model based on a RORB rainfall runoff model were applied to the HEC-2 model. The impact of flood storage on flood flows was simulated by incorporating a lumped conceptual storage in the middle reaches of the RORB model near the confluence of Currambene and Georges Creeks. In the 2006 study, an unsteady flow hydraulic model was used which incorporated three tributary streams which join Currambene Creek in its middle reaches. Accordingly, the attenuating effects of the flood storage in the drainage system were allowed for in a true hydraulic sense. Typically, steady state modelling approaches which solve a simplified version of the momentum equation of flow without regard to storage, tend to give flood levels which are higher than is the case with dynamic modelling. The 1983 study also carried out analyses of Currambene Creek using the unsteady flow modelling package USTFLO, but without the inclusion of the tributary streams.

The USTFLO results, which incorporated the routing effects of the flood storage on the main arm of the creek were about 0.3 to 0.5 m lower than HEC-2 peak levels for the 100 year ARI flood.

(2) **Hydraulic roughness values adopted.** In the 1983 study, the hydraulic roughness values adopted were 0.06 for the channel and 0.12 for the floodplain. The Currambene Creek channel in its estuarine areas is a wide, smooth, sand bed channel, although it narrows and has tree lined banks with correspondingly higher roughness values further upstream. The best estimate values of roughness shown on **Table 3.1** are considered to be more representative of present day conditions on Currambene Creek. The various sensitivity studies described in **Section 3.6** indicate that increases in best estimate hydraulic roughness in both the channel and floodplain would not lead to levels as high as those of the 1983 study.

TABLE 4.5
COMPARISON OF PEAK
100 YEAR ARI FLOOD LEVELS

Location	1983 Flood Study RL m	2006 Flood Study RL m	Difference m
RS 15206 (d/s Princes Highway)	7.3	6.58	– 0.72
RS 11308 (Falls Rd/Woollomia Rd)	4.85	4.43	– 0.42
RS 9196 u/s Georges Creek and Tributary 1)	4.35	4.16	– 0.19
RS 7517 (Goodland Road)	4.0	3.38	– 0.62
RS 5935 (Willowford Road)	3.7	3.1	– 0.6
RS 1094	2.15	1.9	– 0.15
Outlet at Huskisson	1.15	1.89	+ 0.74

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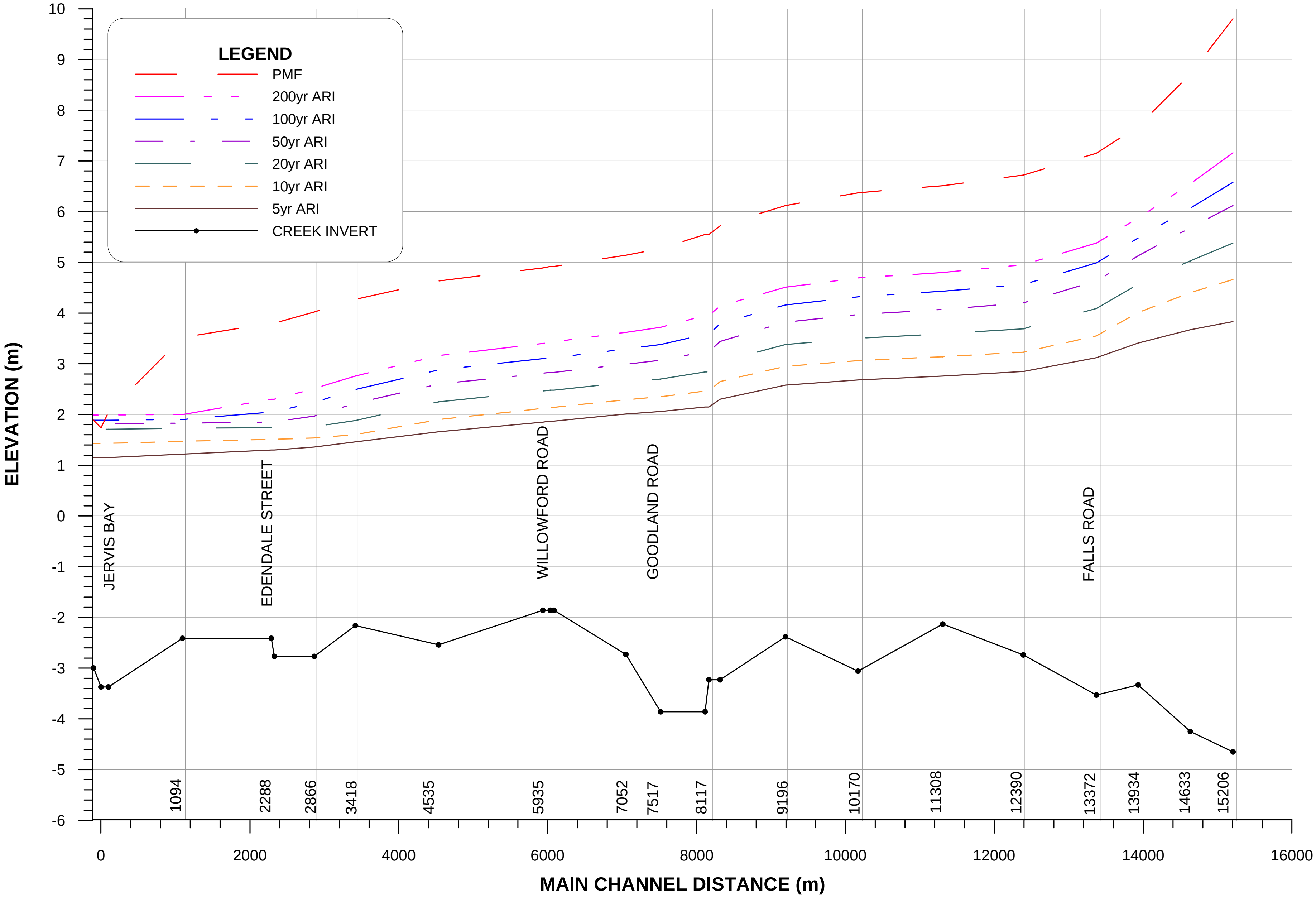
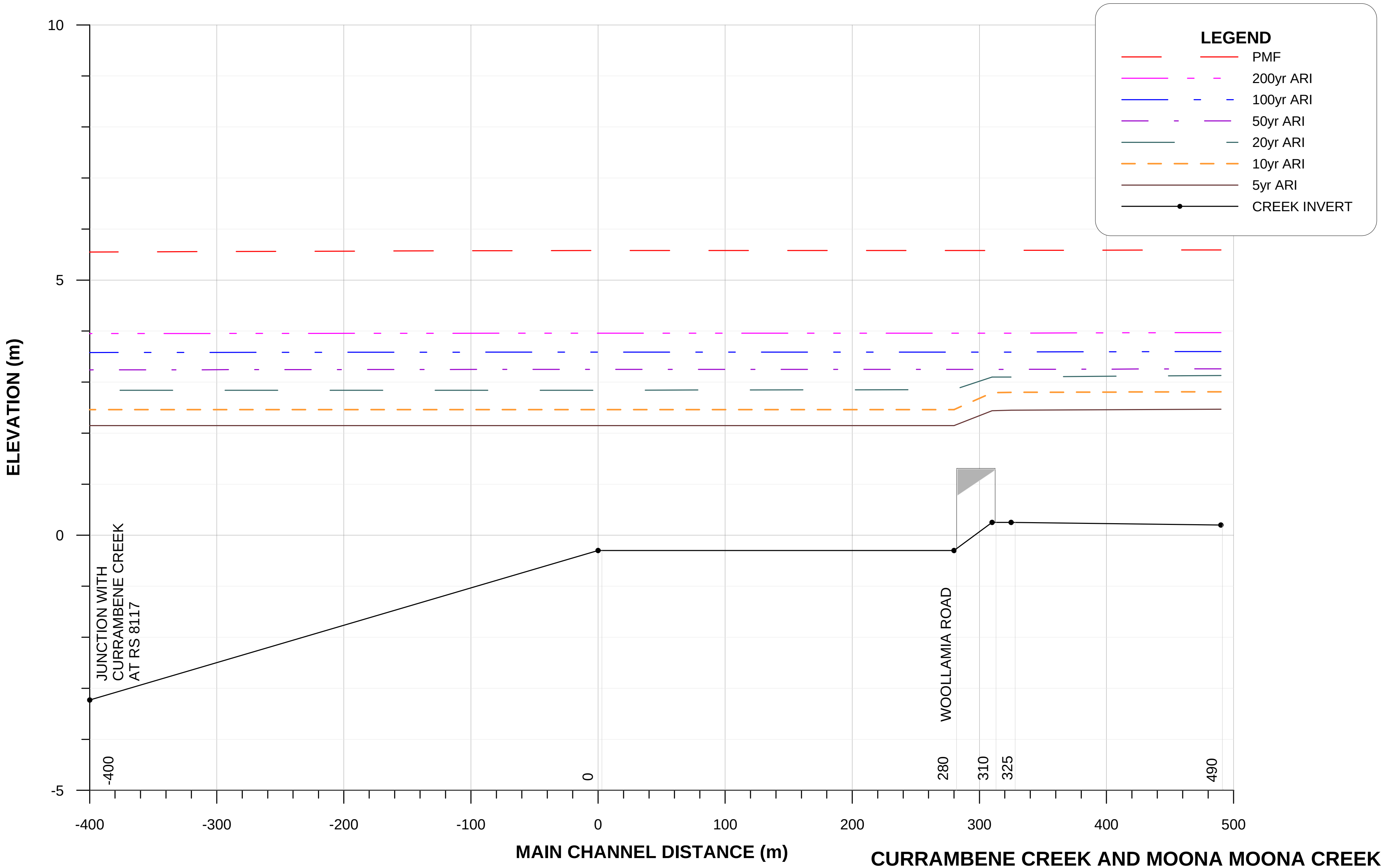


CURRAMBENE CREEK AND MOONA MOONA CREEK FLOOD STUDIES

Figure 4.1

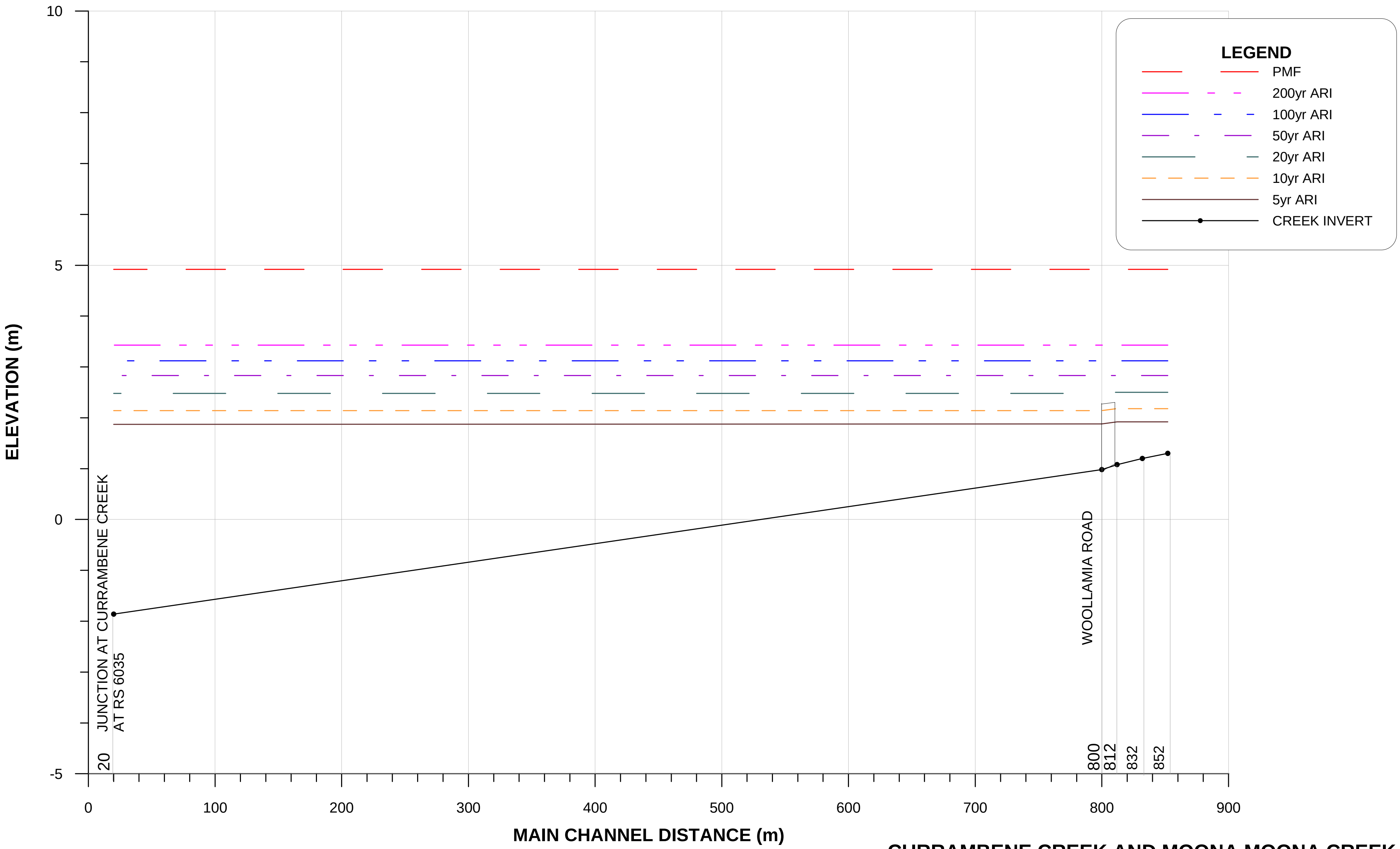
CURRAMBENE CREEK
DESIGN WATER SURFACE PROFILES
5 YEAR ARI TO PMF



CURRAMBENE CREEK AND MOONA MOONA CREEK FLOOD STUDIES

Figure 4.2

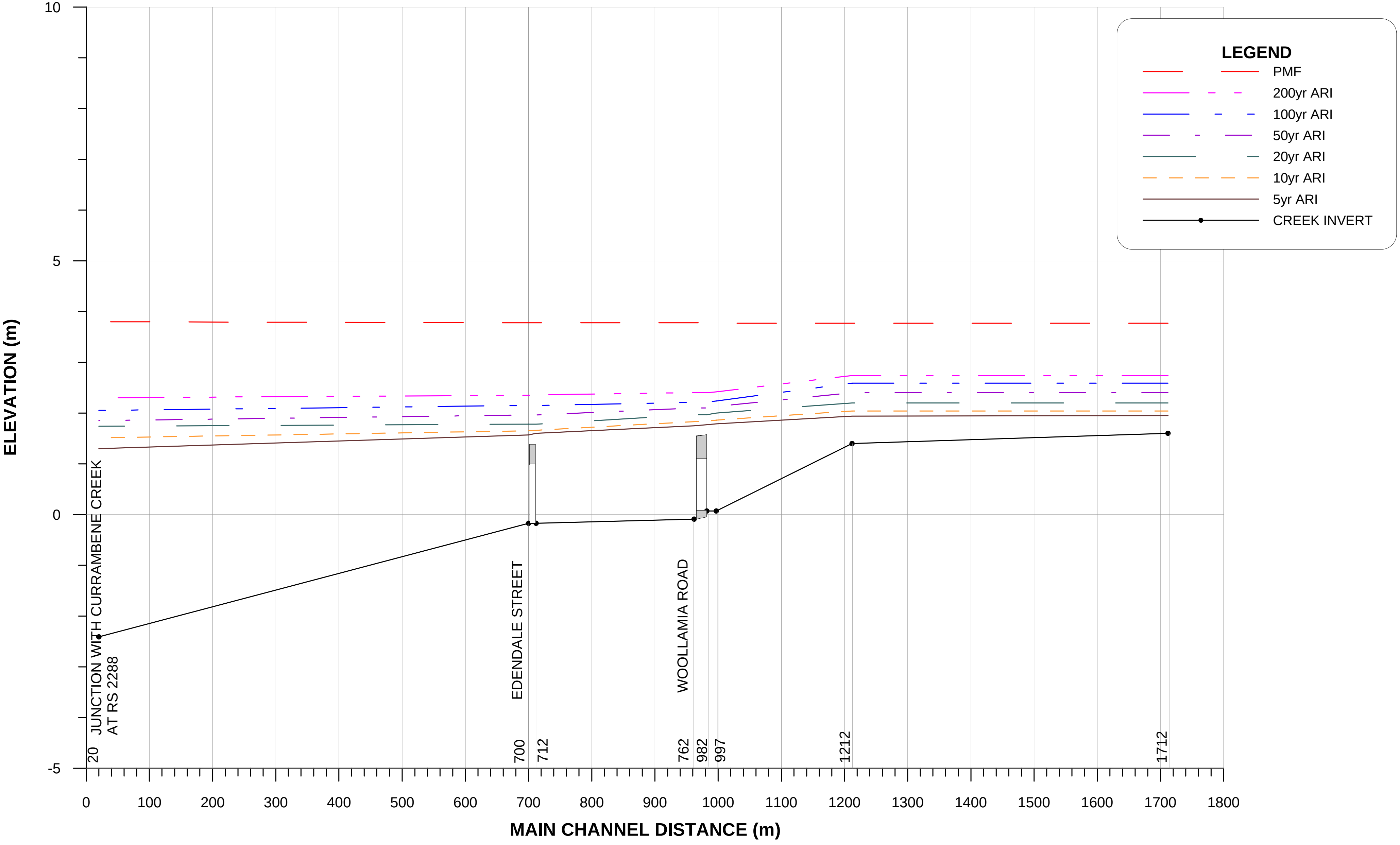
DESIGN WATER SURFACE PROFILES
TRIBUTARY 1 OF CURRAMBENE CREEK
5 YEAR ARI TO PMF



CURRAMBENE CREEK AND MOONA MOONA CREEK
FLOOD STUDIES

Figure 4.3

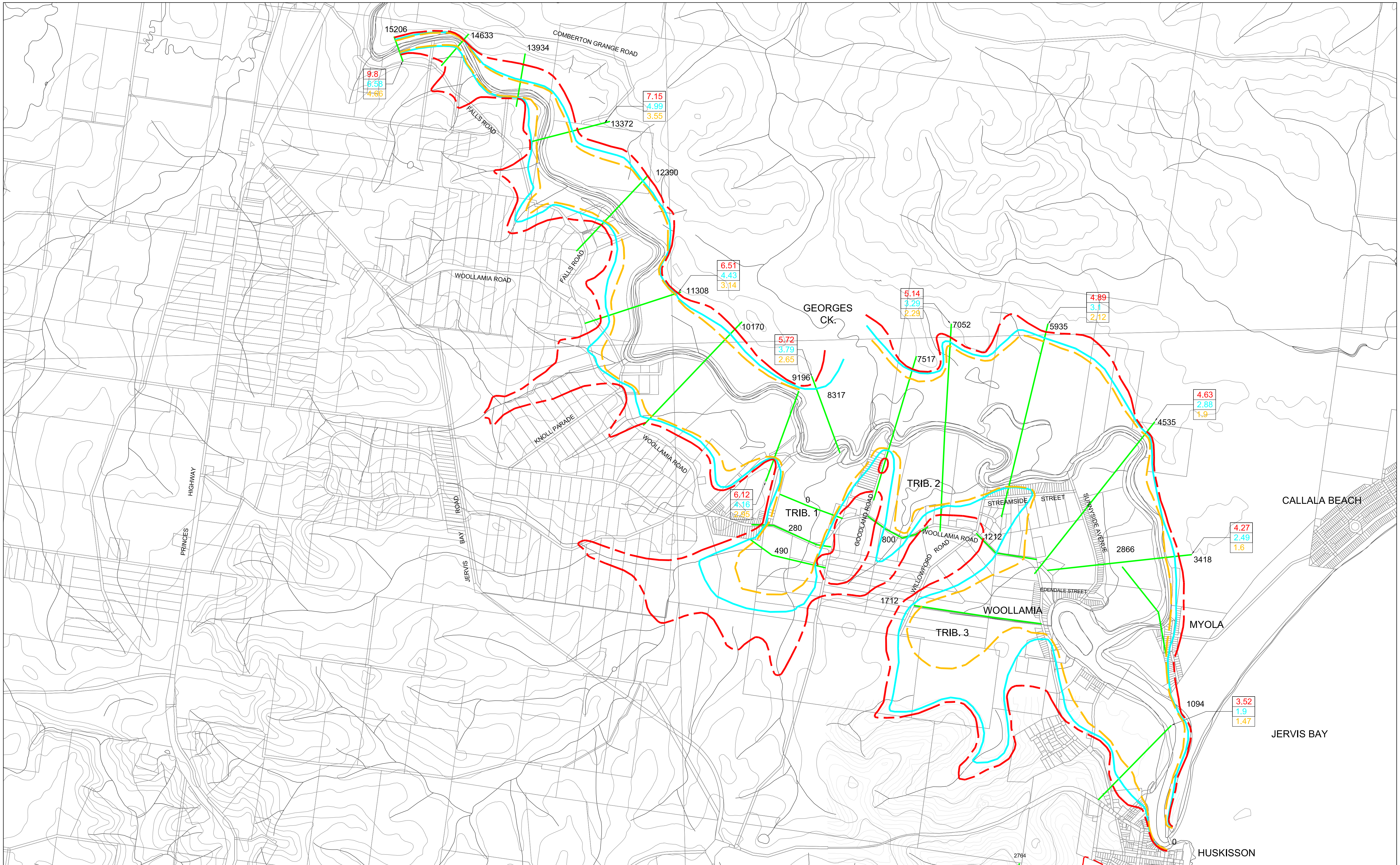
DESIGN WATER SURFACE PROFILES
TRIBUTARY 2 OF CURRAMBENE CREEK
5 YEAR ARI TO PMF



CURRAMBENE CREEK AND MOONA MOONA CREEK
FLOOD STUDIES

Figure 4.4

DESIGN WATER SURFACE PROFILES
TRIBUTARY 3 OF CURRAMBENE CREEK
5 YEAR ARI TO PMF



NOTE

THE EXTENTS OF FLOODING SHOWN WERE DETERMINED FROM SURVEYED CROSS SECTIONS OF THE CREEK AND FLOODPLAIN AND AVAILABLE CONTOUR DATA AND ARE APPROXIMATE ONLY. THE EXTENT OF INUNDATION OF INDIVIDUAL ALLOTMENTS NEAR THE FLOOD FRINGE SHOULD BE CONFIRMED BY SITE SPECIFIC SURVEY.

N

SCALE

0 100 500 1000m

LEGEND

PMF

100 YEAR ARI

10 YEAR ARI

PEAK FLOOD LEVELS

4.27 PMF

2.49 100 YEAR ARI

1.6 10 YEAR ARI

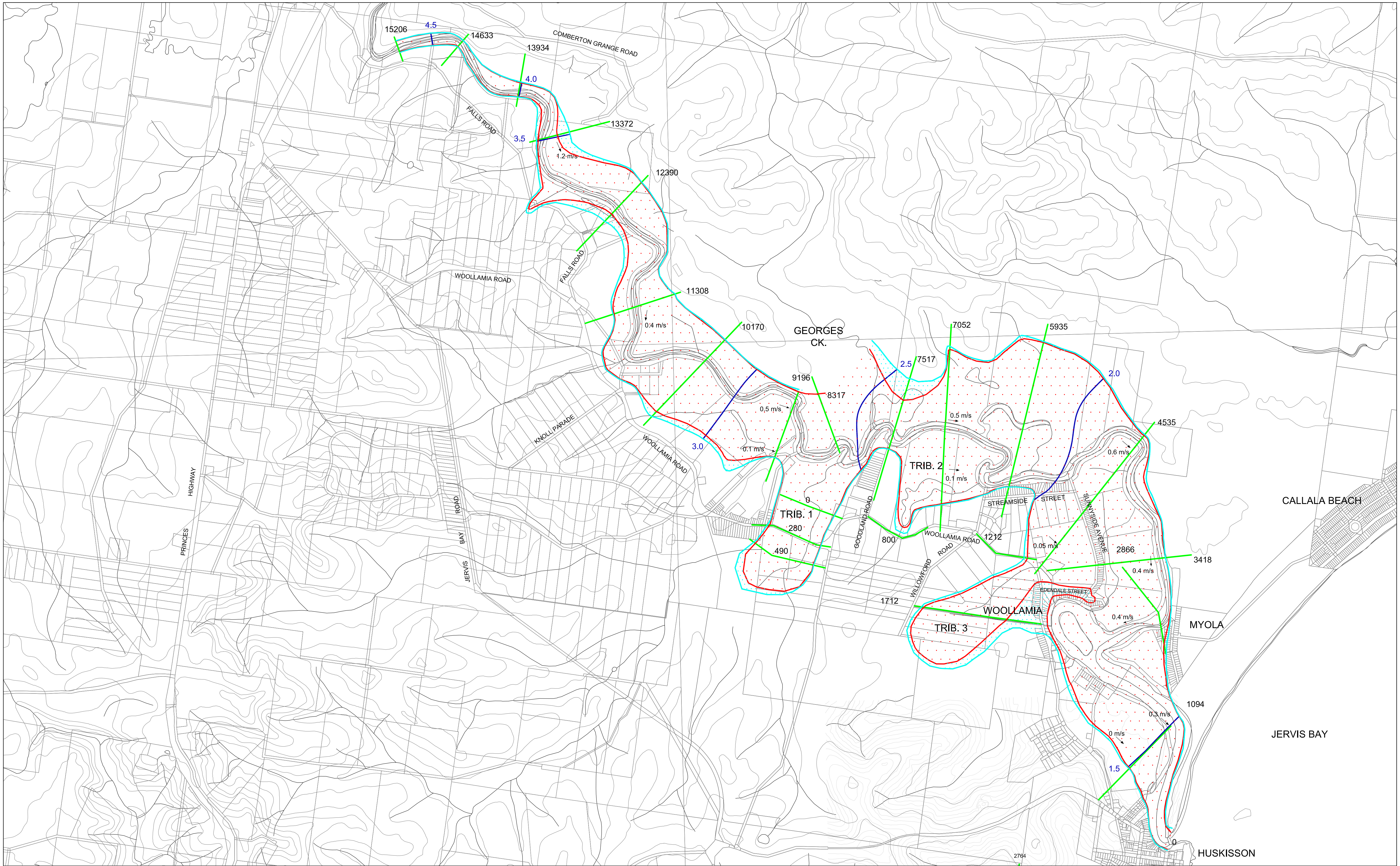
MODEL RIVER STATION

3418

CURRAMBENE CREEK AND MOONA MOONA CREEK
FLOOD STUDIES

Figure 4.5

CURRAMBENE CREEK
INDICATIVE EXTENTS OF INUNDATION
10 YEAR, 100 YEAR ARI AND PMF



NOTE

THE EXTENTS OF FLOODING SHOWN WERE DETERMINED FROM SURVEYED CROSS SECTIONS OF THE CREEK AND FLOODPLAIN AND AVAILABLE CONTOUR DATA AND ARE APPROXIMATE ONLY. THE EXTENT OF INUNDATION OF INDIVIDUAL ALLOTMENTS NEAR THE FLOOD FRINGE SHOULD BE CONFIRMED BY SITE SPECIFIC SURVEY.

N

SCALE

0 100 500 1000m

LEGEND

FLOODWAY ZONE

FLOOD STORAGE ZONE

10 YEAR ARI

2.5 PEAK FLOOD LEVEL CONTOUR - m AHD

0.1 m/s FLOW VELOCITY - m/s

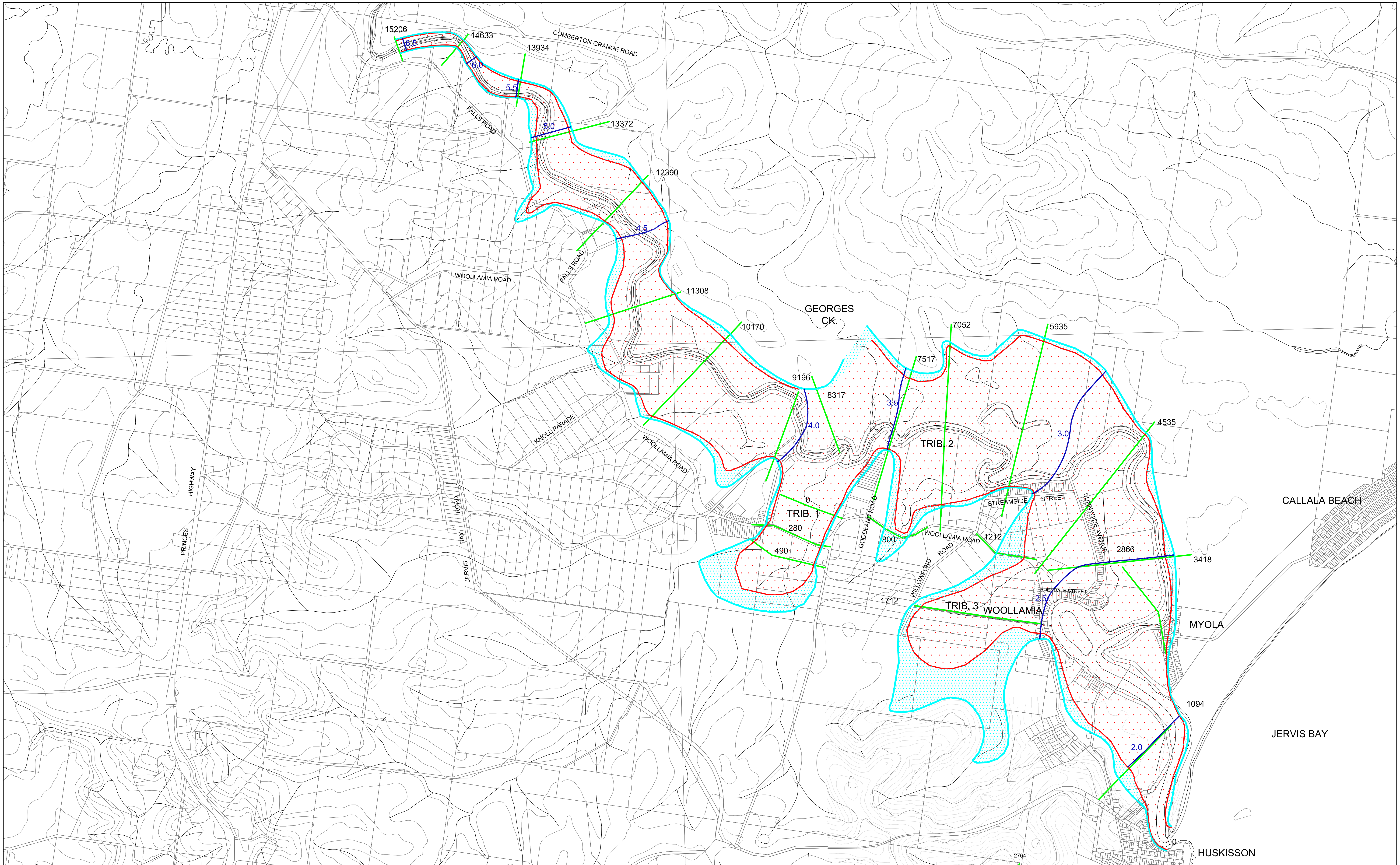
MODEL RIVER STATION

3418

CURRAMBENE CREEK AND MOONA MOONA CREEK FLOOD STUDIES

Figure 4.6

CURRAMBENE CREEK
HYDRAULIC CATEGORISATION OF FLOODPLAIN
10 YEAR ARI FLOOD



NOTE

THE EXTENTS OF FLOODING SHOWN WERE DETERMINED FROM SURVEYED CROSS SECTIONS OF THE CREEK AND FLOODPLAIN AND AVAILABLE CONTOUR DATA AND ARE APPROXIMATE ONLY. THE EXTENT OF INUNDATION OF INDIVIDUAL ALLOTMENTS NEAR THE FLOOD FRINGE SHOULD BE CONFIRMED BY SITE SPECIFIC SURVEY.

N

SCALE
0 100 500 1000m

LEGEND

FLOODWAY ZONE

FLOOD STORAGE ZONE

100 YEAR ARI

2.5 PEAK FLOOD LEVEL CONTOUR - m AHD

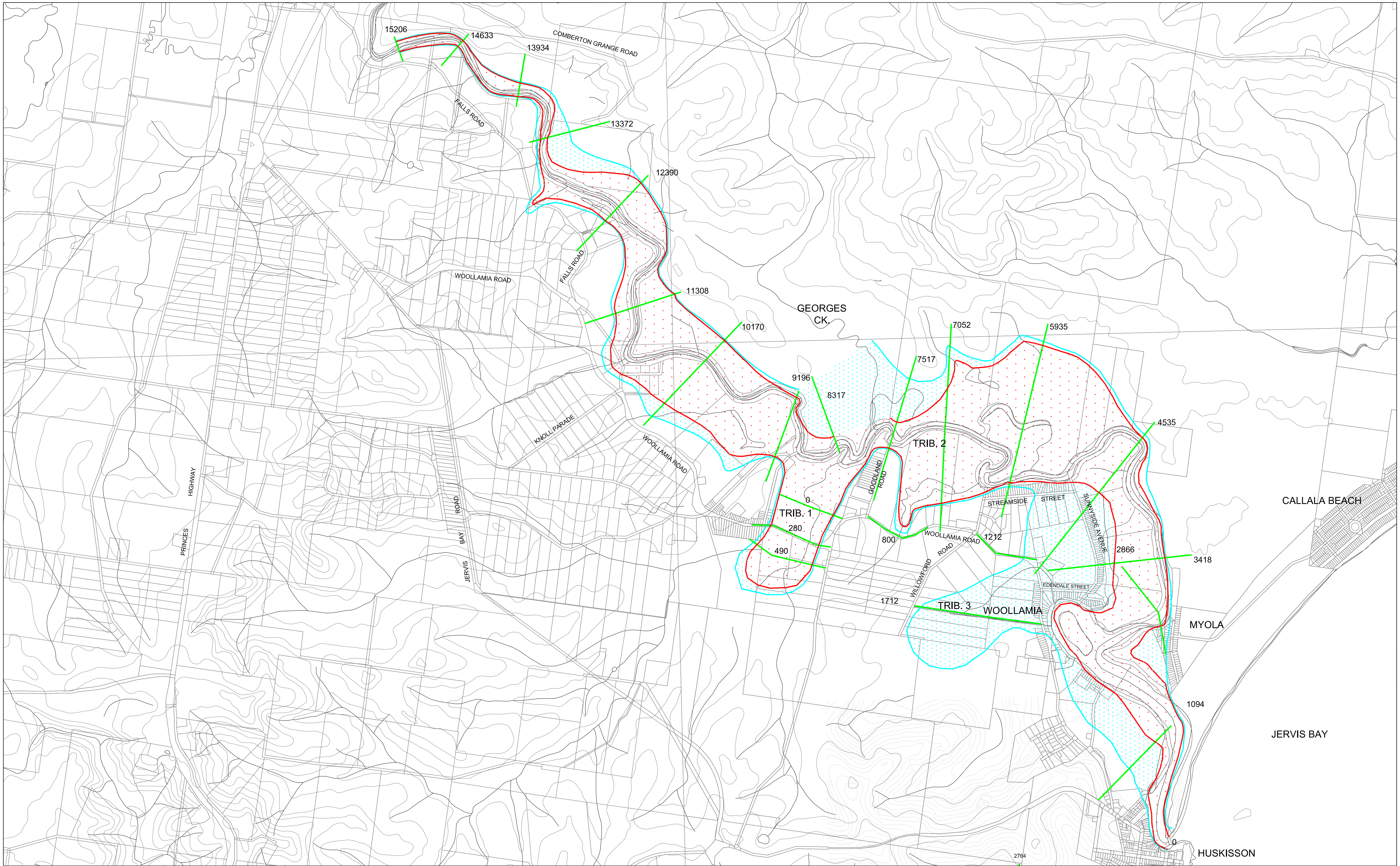
0.1 m/s FLOW VELOCITY - m/s

MODEL RIVER STATION 3418

CURRAMBENE CREEK AND MOONA MOONA CREEK FLOOD STUDIES

Figure 4.7

CURRAMBENE CREEK
HYDRAULIC CATEGORISATION OF FLOODPLAIN
100 YEAR ARI FLOOD



LEGEND

INDICATIVE FLOOD EXTENT

LOW HAZARD AREA

HIGH HAZARD AREA

MODEL RIVER STATION

3418

SCALE

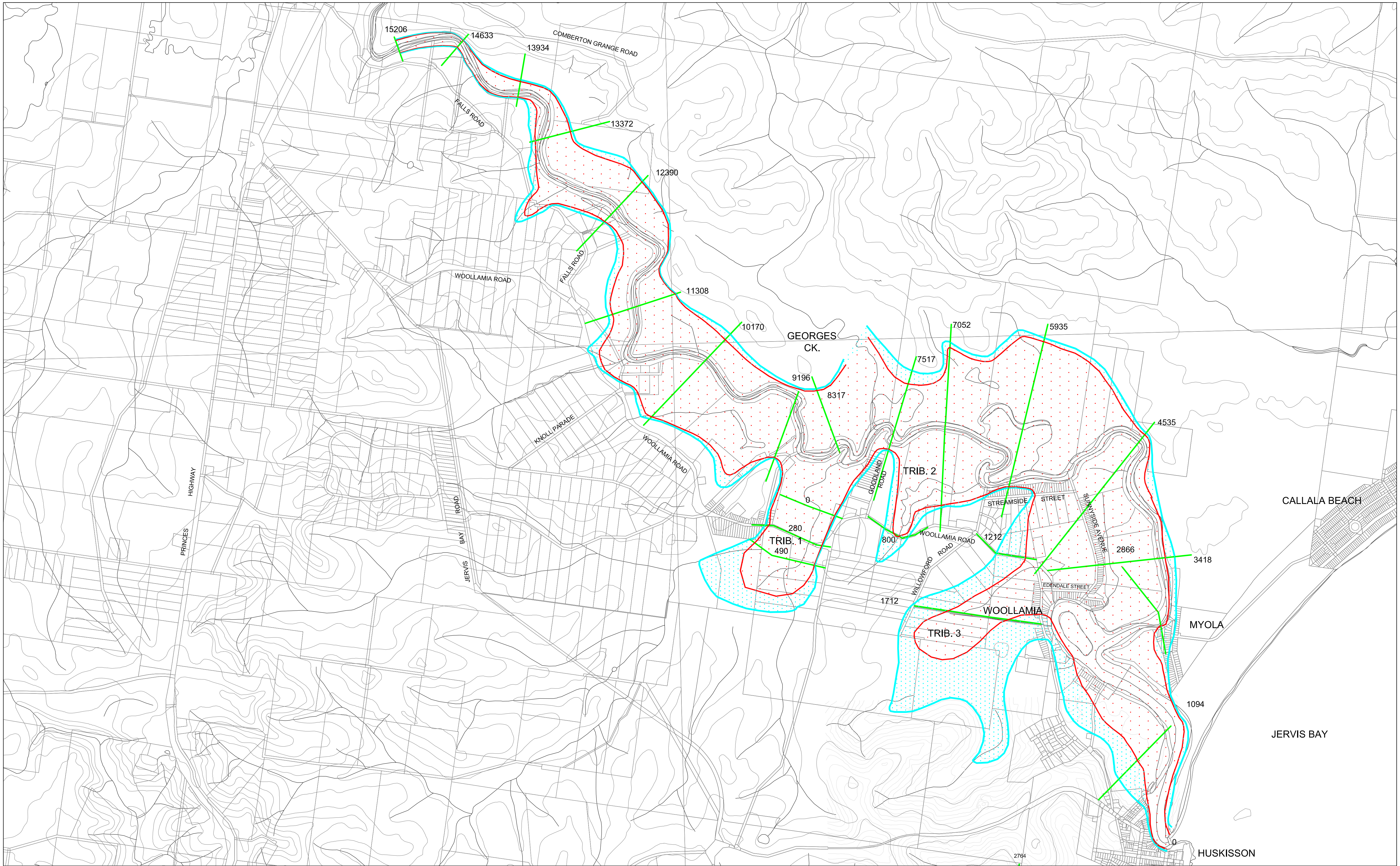
0 100 500 1000m

N

CURRAMBENE CREEK AND MOONA MOONA CREEK
FLOOD STUDIES

Figure 4.8

CURRAMBENE CREEK
PROVISIONAL FLOOD HAZARD
10 YEAR ARI DESIGN FLOOD



LEGEND

INDICATIVE FLOOD EXTENT

LOW HAZARD AREA

HIGH HAZARD AREA

MODEL RIVER STATION

3418

SCALE

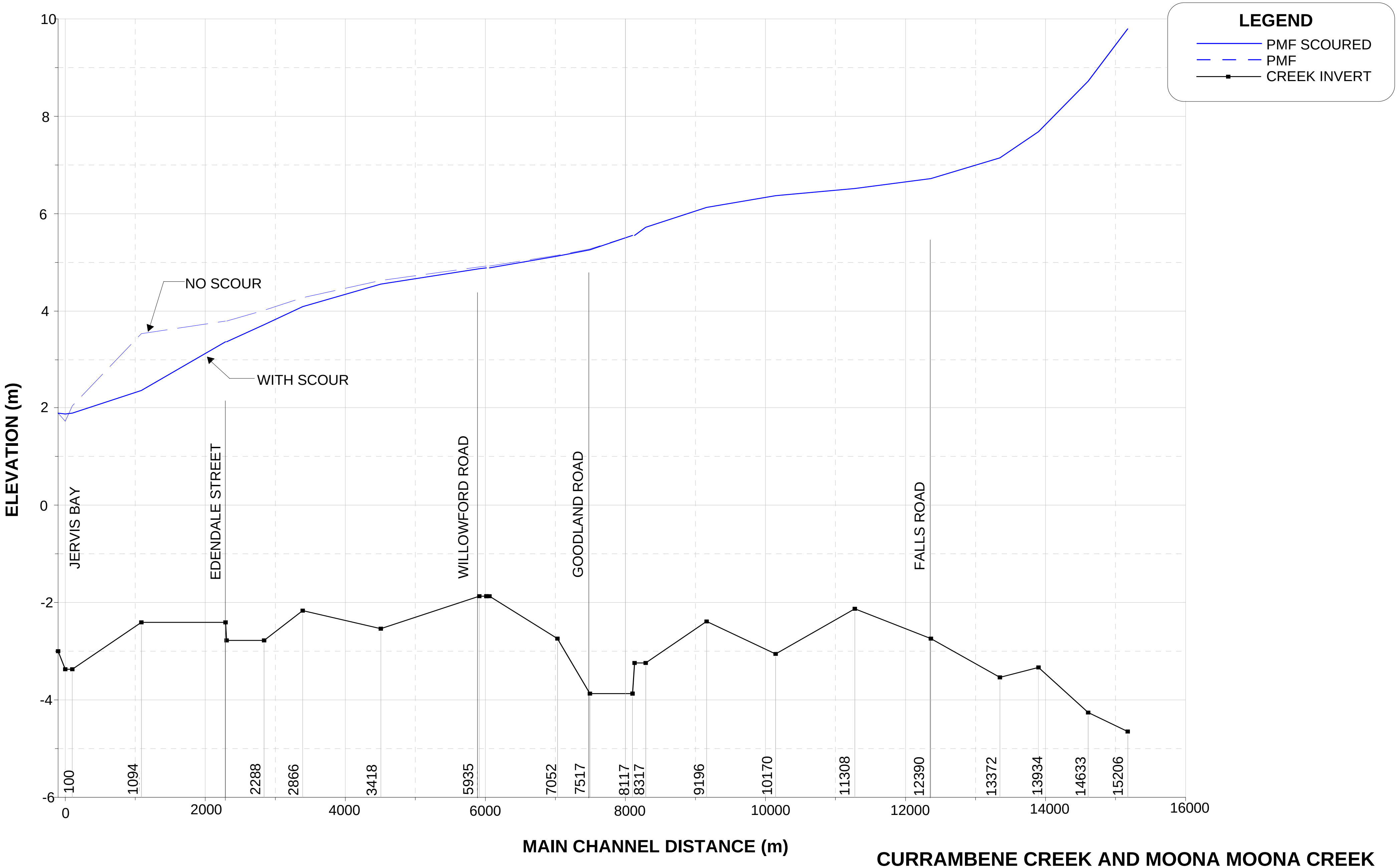
0 100 500 1000m

N

CURRAMBENE CREEK AND MOONA MOONA CREEK
FLOOD STUDIES

Figure 4.9

CURRAMBENE CREEK
PROVISIONAL FLOOD HAZARD
100 YEAR ARI DESIGN FLOOD



CURRAMBENE CREEK AND MOONA MOONA CREEK
FLOOD STUDIES

Figure 4.10

CURRAMBENE CREEK
SENSITIVITY OF PMF LEVELS TO ENTRANCE SCOUR

5. MOONA MOONA CREEK MODEL

5.1 Model Structure

A schematic layout of the Moona Moona Creek HEC-RAS model is shown on **Figure 5.1**. The model comprises the main arm of the creek and a tributary denoted "moona 2" which conveys runoff from the southern portion of the catchment and runs to the east of the Sewage Treatment Plant before joining the southern bank of Moona Moona Creek about 600 m upstream of the Elizabeth Drive bridge.

Modelling commences at RS 4025 which is located downstream of the confluence of Moona Moona and Duck Creeks, about 1 km east of Jervis Bay Road. Above the tidal limit, the creek is overgrown and ill defined with little evidence of a defined channel.

The tidal channel commences about 3 km upstream of the outlet to Jervis Bay and progressively widens to about 80 m in width at the bridge. The overbanks are heavily overgrown with little conveyance capacity and mainly function as areas for the temporary storage of runoff during flood events.

Downstream of Elizabeth Drive, which comprises a two span crossing about 20 m wide at spring tide level. The creek invert within the immediate vicinity of the bridge waterway has scoured to an elevation of RL –3.5 m. Downstream of the bridge, the creek traverses a sandy lagoon area about 350 m in length and outfalls to Jervis Bay at the northern end of Collingwood Beach. The width of the lagoon averages 100 – 120 m and at a point mid-way between the bridge and the outlet, the invert level when surveyed was at RL 0 m. The outlet is about 40 – 60 m wide and when surveyed had an invert of RL –2 m AHD.

The "moona 2" tributary is about 1.6 km long and extends upstream to the low lying area on the northern side of Vincentia Road. This tributary crosses Berry Street, the access road to the Sewage Treatment Works. The culverts beneath the road have been included in the model.

5.2 Moona Moona Creek Survey Data

Cross sections of the main arm of Moona Moona Creek were surveyed from the outlet through the bridge over Elizabeth Drive and extending to a location about 3 km upstream of the crossing.

Due to the difficulties associated with the surveyor working in the swampy areas adjacent to the tidal channel, only three sections of the overbank areas were taken upstream of the bridge. Fortunately, there was some contour information available on both sides of the creek at 2 metre intervals, which assisted with extension of the survey into areas inaccessible to the survey party.

The area upstream of Elizabeth Drive mainly functions as a storage area and consequently, extension of the cross sections beyond the limits surveyed, was mainly aimed at gaining a reasonably accurate assessment of flood storage in the overbank areas.

Four cross sections were also surveyed on the tributary of Moona Moona Creek, including measuring the dimensions of the Berry Street crossing and its culverts.

5.3 Model Parameters

Hydraulic roughness parameters were estimated using similar procedures to those outlined in **Section 3.3.2** for Currambene Creek. **Table 5.1** shows the best estimate of roughness.

TABLE 5.1
BEST ESTIMATE OF HYDRAULIC ROUGHNESS
MOONA MOONA CREEK

	Channel	Floodplain
d/s Elizabeth Drive Bridge	0.025	NA
u/s Bridge	0.03 – 0.035	0.12

5.4 Upstream Boundary Conditions

Discharge hydrographs derived from RORB provided the boundary conditions at the upstream end of the model. Lateral inflow hydrographs were added at various locations to account for runoff from the sub-catchments. In all, a total of 7 discharge hydrographs were applied to the model for each flood event.

5.5 Downstream Boundary Conditions

To assess the reach of the creek where flooding may be influenced by a backwater due to storm tides in Jervis Bay, the following procedure was adopted. This approach is supported by the *"Floodplain Management Guideline No. 5 Ocean Boundary Conditions"* published by DNR and is similar to that adopted for Currambene Creek in **Section 3.6.1**.

(1) Hydraulic modelling was carried out with normal semi-diurnal tidal hydrographs for a range of catchment floods. Initially, the 5 and 100 year ARI catchment floods were modelled for a range of storm durations.

(2) Modelling was then carried out with storm tidal hydrographs of 1 in 20 and 1 in 100 year return periods, in conjunction with a minor catchment flood of 5 year ARI. Storm tides were derived using the generalised procedures set out in Guideline No. 5, in conjunction with the site specific assessment in *"Estimates of Tailwater Levels in Currambene Creek and Moona Moona Creek"* obtained from DNR. (**Table 3.5**).

5.6 Hydraulic Model Testing

5.6.1 Catchment Flooding in Association with Normal Tides

Results for condition (1) above i.e. catchment runoff in conjunction with Normal Tides are shown in **Tables 5.2 and 5.3**. Several cases of hydraulic roughness were analysed: the "best estimate" values of **Table 5.1**, with roughness values in the channel and floodplain increased by 20 per cent, and with the floodplain roughness only increased by 20 per cent.

For the 100 year ARI, the 12 hour storm is generally critical in the upper reaches to a point about 400 m upstream of the bridge. Downstream of that point, the 18 hour storm is critical.

For the 5 year ARI, the 18 hour storm is critical throughout the model with peak levels between 60 mm to 180 mm higher than for the 12 hour storm in the storage area upstream of the bridge.

Table 5.4 shows the sensitivity of model results to hydraulic roughness at representative locations on Moona Moona Creek. The results are not sensitive to variations in hydraulic roughness due to the low velocities of flow in Moona Moona Creek.

5.6.2 Storm Tides in Association with Minor Catchment Floods

This section deals with condition (2) of **Section 5.4**, i.e. Storm Tides in conjunction with minor catchment floods.

Storm tidal hydrographs derived according to point (2) of **Section 5.5** were used as the downstream boundary condition and run with minor 5 year ARI storms on the Moona Moona Creek catchment of 12 and 18 hours duration.

Figure 5.2 shows the results of conditions (1) and (2) for the 100 year ARI event. In the case of the 100 year ARI, the storm tidal backwater controls design peak flood as far as the Elizabeth Drive bridge Upstream of this point, the situation is reversed and catchment flooding controls design flood levels. A similar result is experienced for the 5 year ARI (**Figure 5.4**).

For the 20 year ARI, the influence of the Storm Tide extends further upstream to 2.4 km upstream of Elizabeth Drive (**Figure 5.3**).

For all cases modelled, the flood gradient upstream of Elizabeth Drive is very low, confirming that this area functions mainly as a flood storage with very low flow velocities.

TABLE 5.2
HEC-RAS TEST RUNS ON MOONA MOONA CREEK
5 YEAR ARI 18 HOUR STORM IN CONJUNCTION WITH NORMAL TIDES

River	Reach	River Station	Best Estimate Manning's 'n'	Sensitivity Run 1 Manning's 'n' 20% Increase Channel and Floodplain	Sensitivity Run 2 Manning's 'n' 20% Increase Floodplain Only
			RL m AHD	RL m AHD	RL m AHD
Moona	MA	4064	2.02	2.1	2.05
Moona	MA	2764	1.76	1.82	1.78
Moona	MA	2309	1.68	1.74	1.70
Moona	MA	1844	1.55	1.61	1.56
Moona	MA	1459	1.47	1.53	1.47
Moona	MA	1009	1.43	1.48	1.42
Moona	MA	937	1.41	1.46	1.40
Moona	MB	874	1.41	1.46	1.40
Moona	MB	514	1.34	1.39	1.34
Moona	MB	374	1.33	1.37	1.33
Moona	MB	336	1.31	1.36	1.31
Moona	MB	329		Elizabeth Drive Bridge	
Moona	MB	324	1.28	1.33	1.28
Moona	MB	271	1.30	1.34	1.30
Moona	MB	115	1.29	1.33	1.29
Moona	MB	68	1.28	1.32	1.28
Moona	MB	0	1.01	1.03	1.01
Moona	MB	– 100	0.87	0.87	0.87
				Jervis Bay	
Moona2	LG	1640	2.34	2.34	2.34
Moona2	LG	1300	2.07	2.08	2.08
Moona2	LG	570	1.97	1.98	1.98
Moona2	LG	560	1.97	1.97	1.98
Moona 2	LG	555		Berry Street	
Moona2	LG	550	1.83	1.82	1.83
Moona2	LG	150	1.41	1.46	1.40
Moona2	LG	20	1.41	1.46	1.40

Notes: "Moona" is the main arm of Moona Moona Creek; "Moona 2" is the tributary running between the STP and Elizabeth Drive.

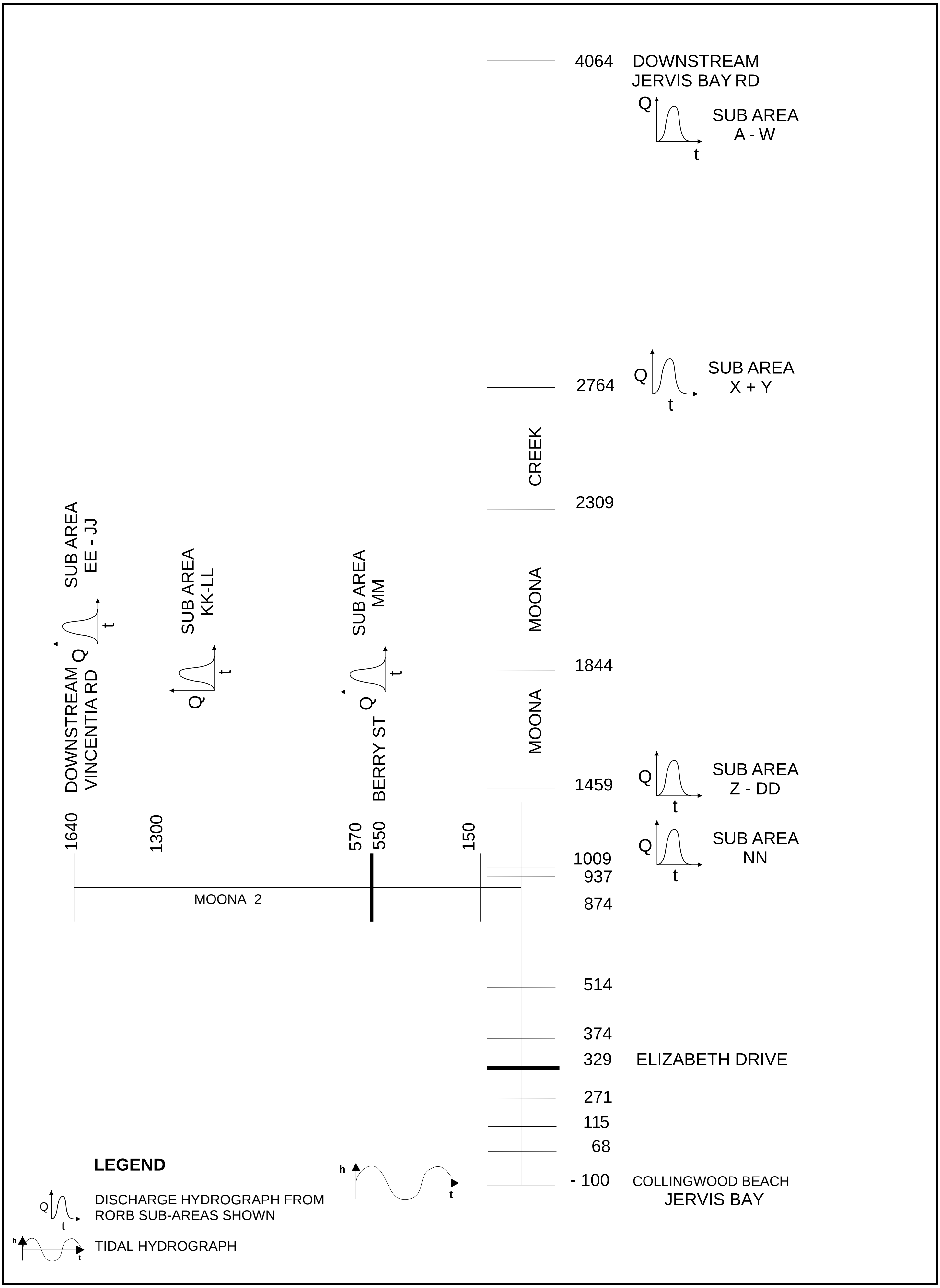
TABLE 5.3
HEC-RAS TEST RUNS ON MOONA MOONA CREEK
100 YEAR ARI 18 HOUR STORM IN CONJUNCTION WITH NORMAL TIDES

River	Reach	River Station	Best Estimate Manning's 'n'	Manning's 'n' 20% Increase Channel and Floodplain	Manning's 'n' 20% Increase Floodplain Only
			RL m AHD	RL m AHD	RL m AHD
Moona	MA	4064	2.60	2.72	2.66
Moona	MA	2764	2.40	2.48	2.43
Moona	MA	2309	2.35	2.43	2.38
Moona	MA	1844	2.29	2.36	2.31
Moona	MA	1459	2.25	2.31	2.26
Moona	MA	1009	2.22	2.27	2.23
Moona	MA	937	2.21	2.26	2.21
Moona	MB	874	2.21	2.26	2.21
Moona	MB	514	2.16	2.20	2.16
Moona	MB	374	2.14	2.18	2.14
Moona	MB	336	2.10	2.13	2.10
Moona	MB	329	Elizabeth Drive Bridge		
Moona	MB	324	2.02	2.05	2.02
Moona	MB	271	2.07	2.09	2.06
Moona	MB	115	2.05	2.07	2.05
Moona	MB	68	2.04	2.06	2.04
Moona	MB	0	1.14	1.17	1.14
Moona	MB	– 100	0.87	0.87	0.87
			Jervis Bay		
Moona2	LG	1640	2.68	2.69	2.69
Moona2	LG	1300	2.51	2.52	2.52
Moona2	LG	570	2.33	2.35	2.33
Moona2	LG	560	2.33	2.35	2.32
Moona2	LG	555	Berry Street		
Moona2	LG	550	2.27	2.31	2.27
Moona2	LG	150	2.21	2.26	2.21
Moona2	LG	20	2.21	2.26	2.21

Notes: "Moona" is the main arm of Moona Moona Creek; "Moona 2" is the tributary running between the STP and Elizabeth Drive.

TABLE 5.4
SENSITIVITY OF MODEL RESULTS TO HYDRAULIC ROUGHNESS ON MOONA MOONA CREEK STORMS OF RETURN PERIOD SHOWN IN CONJUNCTION WITH NORMAL TIDES

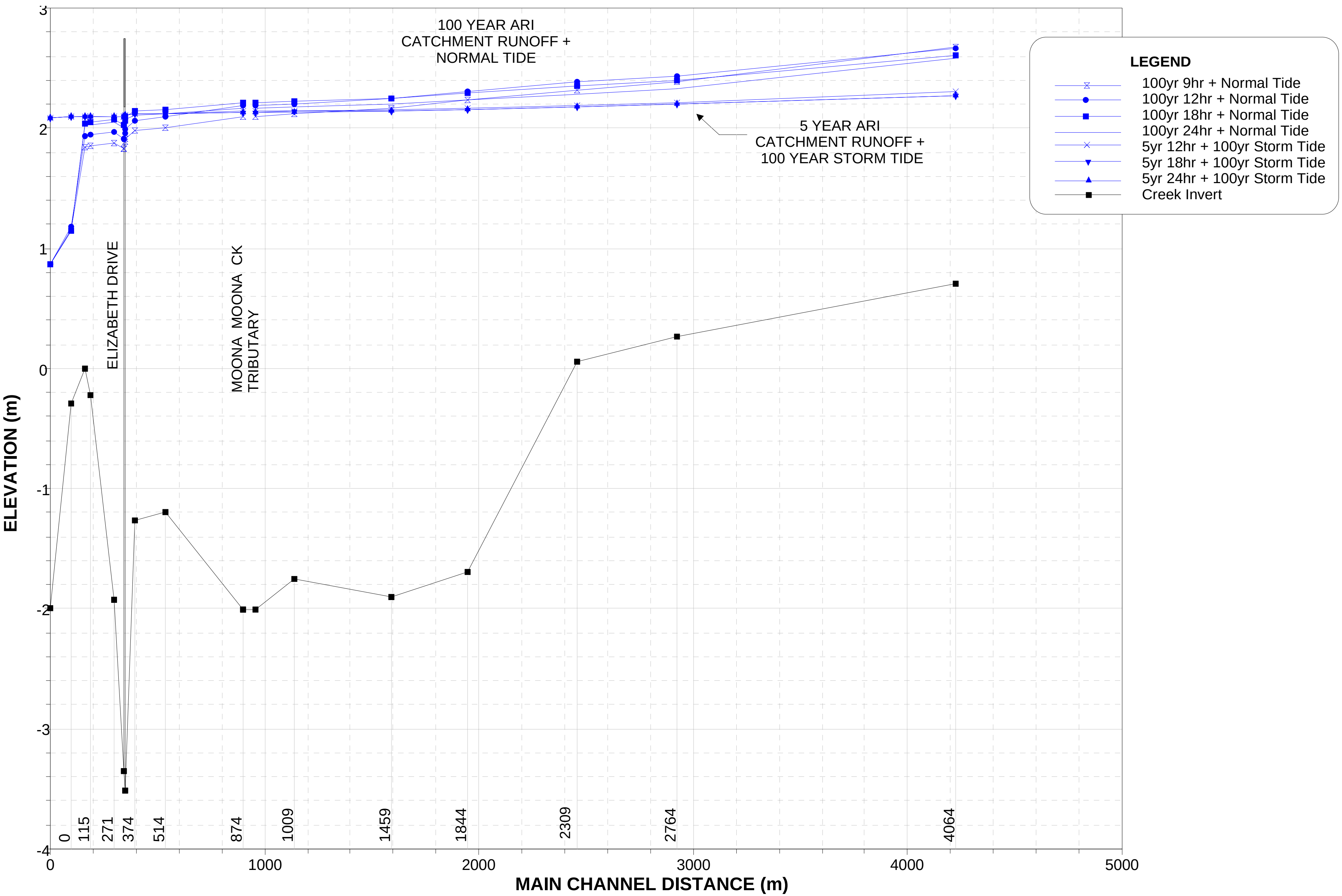
River Station	Flood Level Best Estimate Hydraulic Roughness	Flood Level 20% Increase in Roughness Channel and Floodplain	Increase in Flood Level m	Flood Level 20% Increase in Roughness Floodplain Only	Increase in Flood Level m
	5 YEAR ARI 18 HOUR STORM				
4064	2.02	2.10	+ 0.08	2.05	+0.03
937	1.41	1.46	+ 0.05	1.40	0
336	1.31	1.36	+0.05	1.31	0
	100 YEAR ARI 18 HOUR STORM				
4064	2.60	2.72	+ 0.12	2.66	+0.06
937	2.21	2.26	+ 0.05	2.21	0
336	2.10	2.13	+0.03	2.10	0



CURRAMBENE CREEK AND MOONA MOONA CREEK
FLOOD STUDIES

Figure 5.1

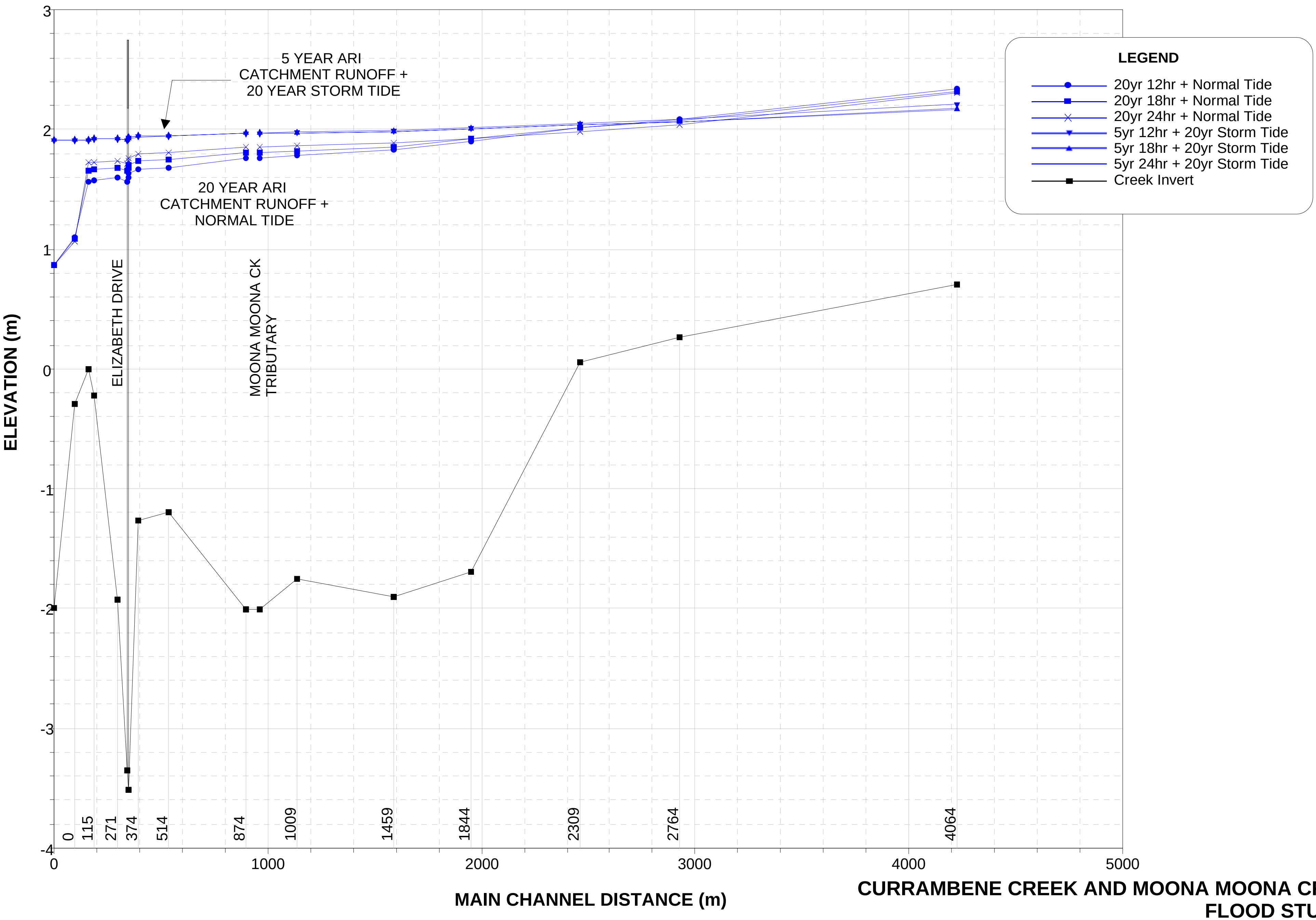
MOONA MOONA CREEK
HEC - RAS SCHEMATIC LAYOUT



CURRAMBENE CREEK AND MOONA MOONA CREEK FLOOD STUDIES

Figure 5.2

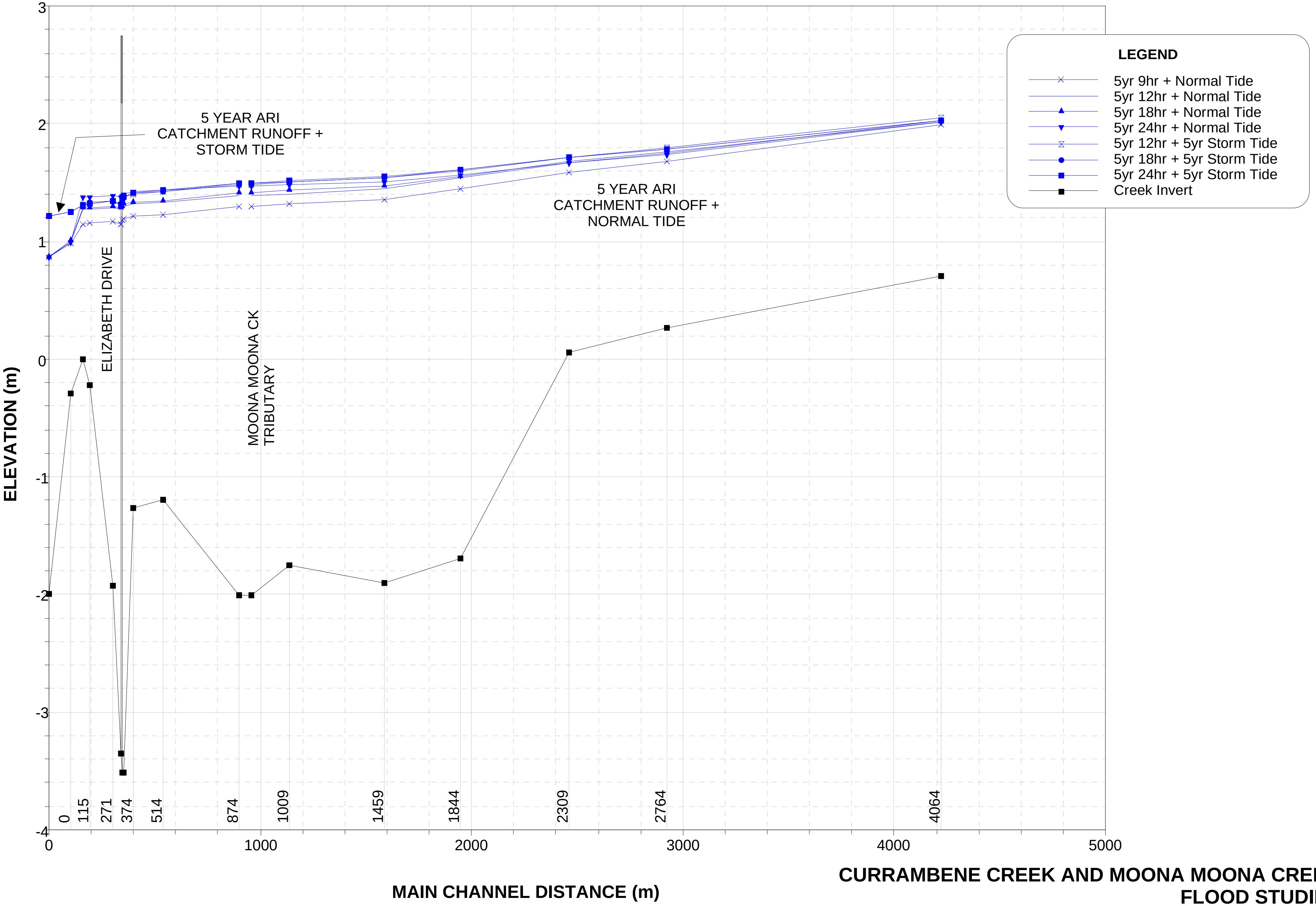
MOONA MOONA CREEK
100 YEAR ARI FLOOD ENVELOPE



CURRAMBENE CREEK AND MOONA MOONA CREEK FLOOD STUDIES

Figure 5.3

MOONA MOONA CREEK
20 YEAR ARI FLOOD ENVELOPE



CURRAMBENE CREEK AND MOONA MOONA CREEK FLOOD STUDIES

Figure 5.4

MOONA MOONA CREEK
5 YEAR ARI FLOOD ENVELOPE

6. MOONA MOONA CREEK – RESULTS OF HYDRAULIC MODELLING

6.1 General

Figures 6.1 and 6.2 show the flood envelopes for each flood along the 4 km length of Moona Moona Creek and the tributary.

Figure 6.3 is a plan of the Moona Moona Creek floodplain showing the indicative extents of inundation, and peak flood levels for the 10 and 100 year ARI floods and the PMF.

Figures 6.4 and 6.5 relate to the 10 and 100 year ARI floods. These figures show the sub-division of the inundated areas into floodway and flood storage zones and also include flood contour and flow velocity information at typical locations.

Figures 6.6 and 6.7 present provisional flood hazard information for the 10 and 100 year ARI floods, based on flow velocity and depth data obtained from the hydraulic analyses.

Addendum B presents tabulated data of flood levels and distribution of flow and velocities across the waterway sections.

The extents of inundation shown on **Figure 6.3** are necessarily indicative only. On Moona Moona Creek, the extent of inundation is based on several surveyed sections downstream of Elizabeth Drive and several sections through the swampy area on the upstream side of the bridge. The GIS data also contained 2 m contour information over a small portion of the floodplain upstream of Elizabeth Drive. Overall, however, due to lack of data, the extent of inundation is less precise than for Currambene Creek.

Whilst the flood level and velocity data derived from the analyses are accurate at the sections comprising the model, the flood extent diagrams should not be used to determine the flood affectation in individual allotments.

6.2 Discussion of Results

Storm tides control flood levels in the lower portion of Moona Moona Creek between Elizabeth Drive bridge and the outlet to Jervis Bay. Upstream of the bridge in the storage area, flood gradients are very low. However, towards the upstream end of the modelled reach the higher catchment flows associated with the Normal Semi-Diurnal Tide scenario result in flood levels 200-300 mm higher than for the Storm Tide case. A similar situation occurs on the southern tributary.

Flow velocities over the modelled reaches are quite low, generally less than 0.5 m/s. The Elizabeth Drive bridge conveys flows up to the 100 year ARI event without overtopping.

The water surface level versus discharge relationship experienced at each cross section within the overgrown area upstream of Elizabeth Drive is characteristic of the looped rating curve often exhibited in rivers with a low channel capacity and large volume of floodplain storage. The stage hydrograph lags the discharge hydrograph by several hours. The travel time of the flood peak through the storage area is about 3 hours.

The attenuating effects of the flood storage in the overbank areas offset the increase in flows arising from the contributions from the sub-catchments. **Table 6.1** shows peak flows along Moona Moona Creek for the 100 year ARI 12 hour storm, which is critical for flood levels over the upper and middle reaches. This table also shows peak flows generated by the RORB catchment model. In contrast to the Currambene Creek case, the floodplain storage was not specifically modelled in the RORB model for Moona Moona Creek. Comparison of the HEC-RAS and RORB flows presented in **Table 6.1** shows the attenuation achieved by the floodplain storage upstream of Elizabeth Drive.

At Elizabeth Drive the storage within the HEC-RAS model attenuates the peak flow by about 100 m^3/s from 268 m^3/s (as derived by RORB) to 165 m^3/s (HEC-RAS).

TABLE 6.1
PEAK FLOWS ON MOONA MOONA CREEK
100 YEAR ARI, 12 HOUR STORM
Values in m^3/s

Location	HEC-RAS Model (Incorporating Distributed Flood Storage)	RORB Hydrologic Model (Flood Storage not incorporated in Model)
D/s Jervis Bay Road	118	118
U/s Confluence with Tributary	138	188
Tributary d/s Vincentia Road	41	41
Tributary u/s Confluence with Moona Moona Creek	33	48
Elizabeth Drive bridge	165	268

Similar comparative results were achieved in the case of the Probable Maximum Flood as shown on **Table 6.2**.

TABLE 6.2
PEAK FLOWS ON MOONA MOONA CREEK
PROBABLE MAXIMUM FLOOD
Values in m^3/s

Location	HEC-RAS Model (Incorporating Distributed Flood Storage)	RORB Hydrologic Model (Flood Storage not Incorporated in Model)
D/s Jervis Bay Road	410	415
U/s Confluence with Tributary	390	604
Tributary d/s Vincentia Road	138	138
Tributary u/s Confluence with Moona Moona Creek	210	173
Elizabeth Drive bridge	585	883

6.3 Hydraulic Categorisation of the Floodplain

The subdivision of the floodplain according to the Floodplain Development Manual, 2005 is discussed in detail in **Section 4.4** in the case of Currambene Creek.

The extent of inundation reached by the 10 year ARI flood was adopted as the 100 year ARI floodway on Moona Moona Creek. The 5 year ARI flood extent was adopted as the floodway for the 10 year ARI flood.

These hydraulic categorisations are shown on **Figures 6.4** and **6.5**.

6.4 Provisional Definition of Flood Hazard

Flood hazard categories were assigned to flood affected areas on Moona Moona in accordance with the procedures outlined in **Section 4.5** for Currambene Creek.

A depth of 1 m was adopted as the boundary between *Low* and *High Hazard* zones. Provisional hazard diagrams for the 10 and 100 year ARI floods are shown on **Figures 6.6** and **6.7**.

As noted in the Floodplain Development Manual, 2005, other considerations such as rate of rise of floodwaters and access to high ground for evacuation from the floodplain should also be taken into consideration before a final determination of Flood Hazard can be made. These factors are normally taken into account in the *Floodplain Risk Management Study* for the catchment.

6.5 Impacts of Entrance Scour on Flood levels

As mentioned previously in **Section 4.6**, the hydraulic analyses described in this study have been carried out assuming that the dimensions of the channel are maintained over the duration of the simulation i.e. on the assumption of a “rigid boundary” for all cross sections comprising the model.

With the long duration storms which were found to maximise flows on the two streams, it is likely that the an intermittently opened entrance such as exists on Moona Moona Creek outlet downstream of Elizabeth Drive would have been scoured in the early stages of the flood so that by the time the peak arrived, the erosion process would have been largely completed. An unsteady flow analysis with a rigid boundary representing the eroded channel could therefore be used to provide a reasonable assessment of the impacts of scour.

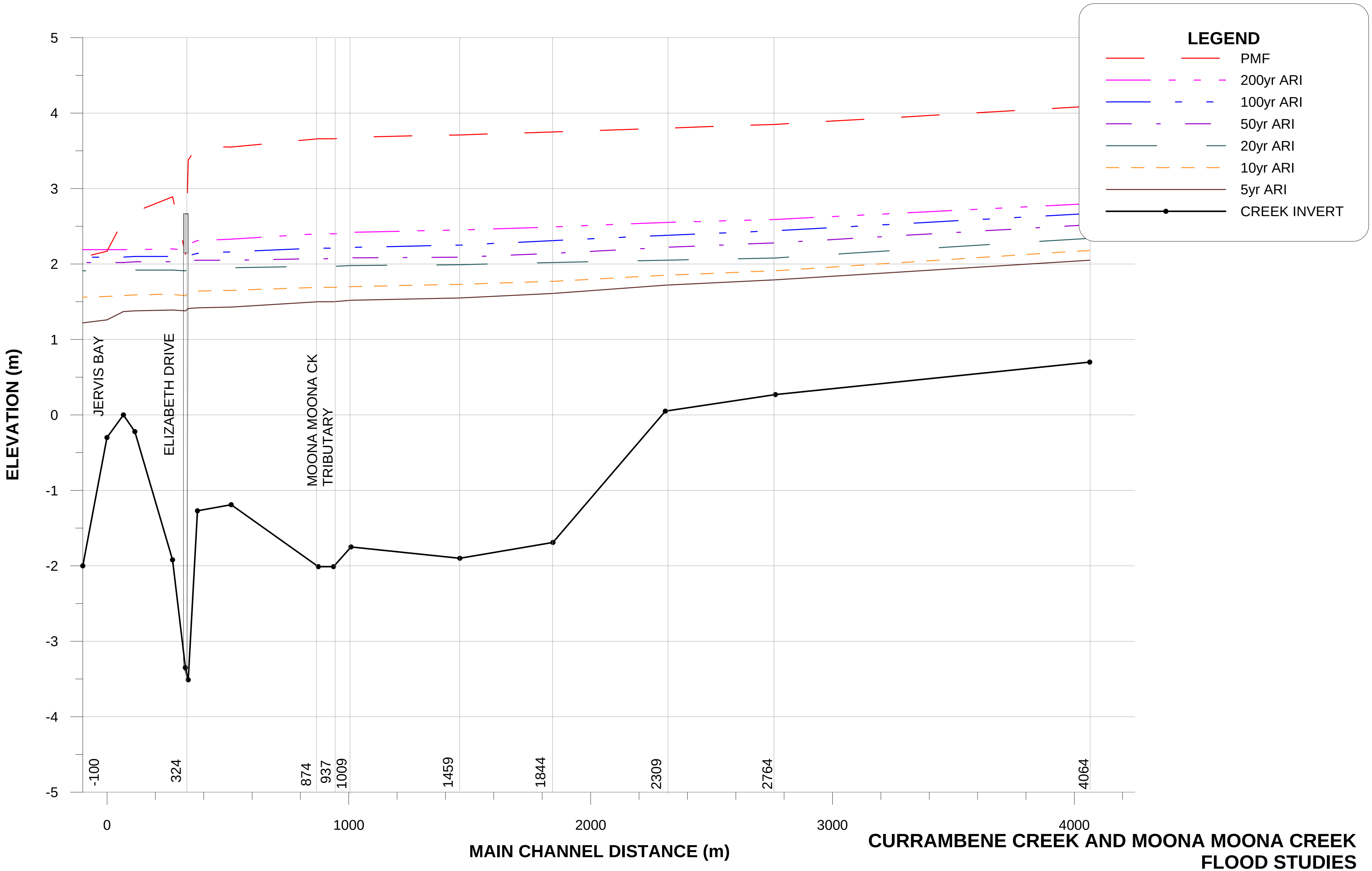
At the time of the survey the invert of the sand bed channel on the downstream side of Elizabeth Drive reach a maximum elevation of about RL 0 m AHD. Local knowledge indicates that there may be a rock shelf at a shallow depth beneath the sand, which would limit the depth of erosion during flood periods. However, there are no quantitative data available regarding sub-surface conditions.

For the purposes of sensitivity analysis of the effects of scour, a channel with invert eroded to RL –1 m AHD was assumed. **Figure 6.8** shows 100 year ARI water surface profiles with a Normal Tide tailwater, for both the surveyed and scoured channel. The water surface profile resulting from a 100 year ARI tailwater, in conjunction with a 5 year ARI catchment flood is also shown.

Water levels with a scoured channel would be around 1 metre lower than for the surveyed channel from the entrance to Elizabeth Drive. The two profiles would not merge until the upstream model boundary was reached about 4 km upstream of Elizabeth Drive.

However, the Storm Tide water surface profile, which commences at the entrance with an elevation of about RL 2.1 m AHD, would then control design flood levels. This profile would not be affected by scour as it drowns out the entrance to Moona Moona Creek. That profile would then control design flood levels for about 2 km upstream of the entrance.

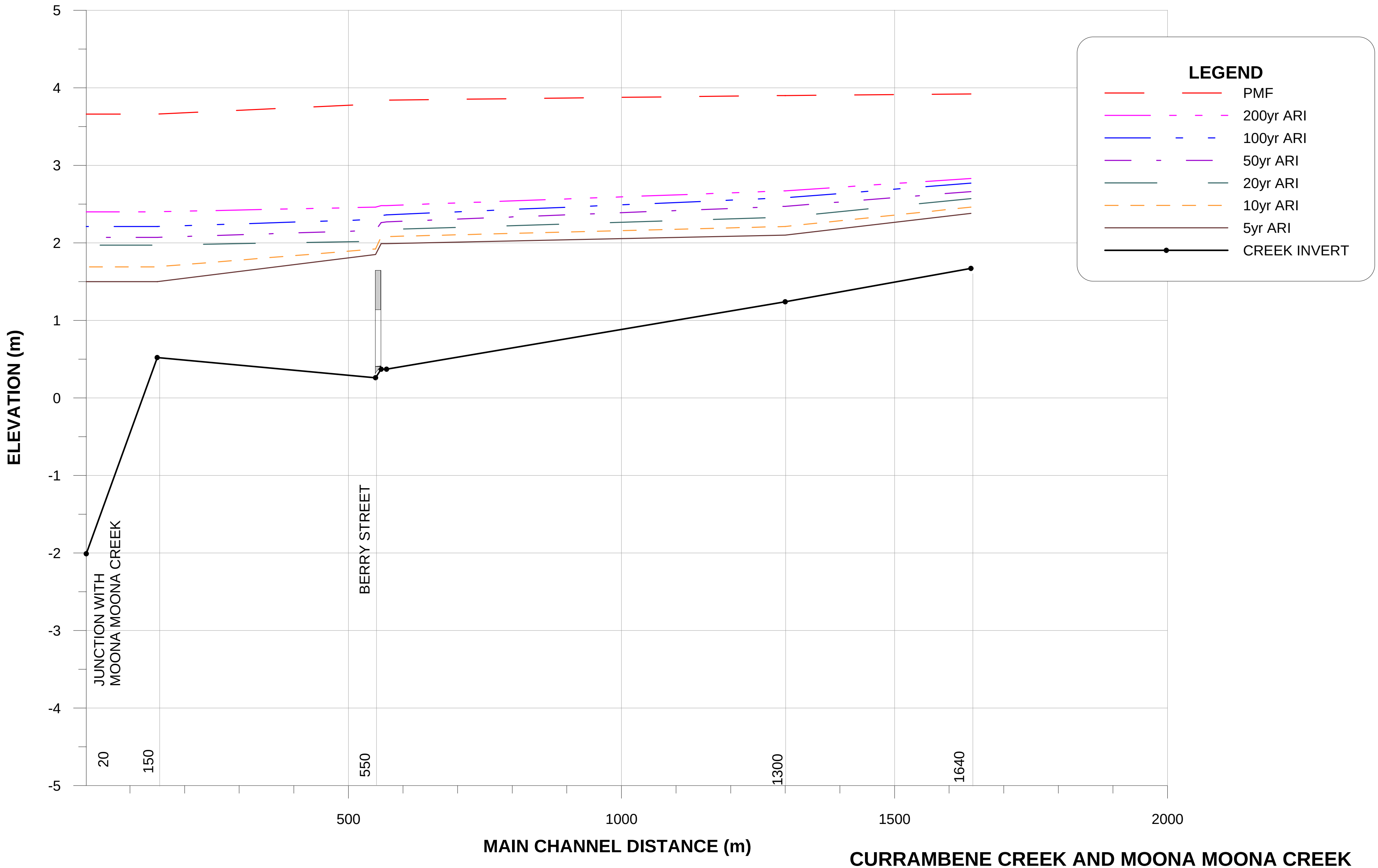
By inspection of **Figure 6.8**, it will be appreciated that adoption of the results for a scoured channel would not result in a significant reduction in design flood levels on Moona Moona Creek, as they would then be controlled by the Storm Tide scenario presented on that diagram.



CURRAMBENE CREEK AND MOONA MOONA CREEK FLOOD STUDIES

Figure 6.1

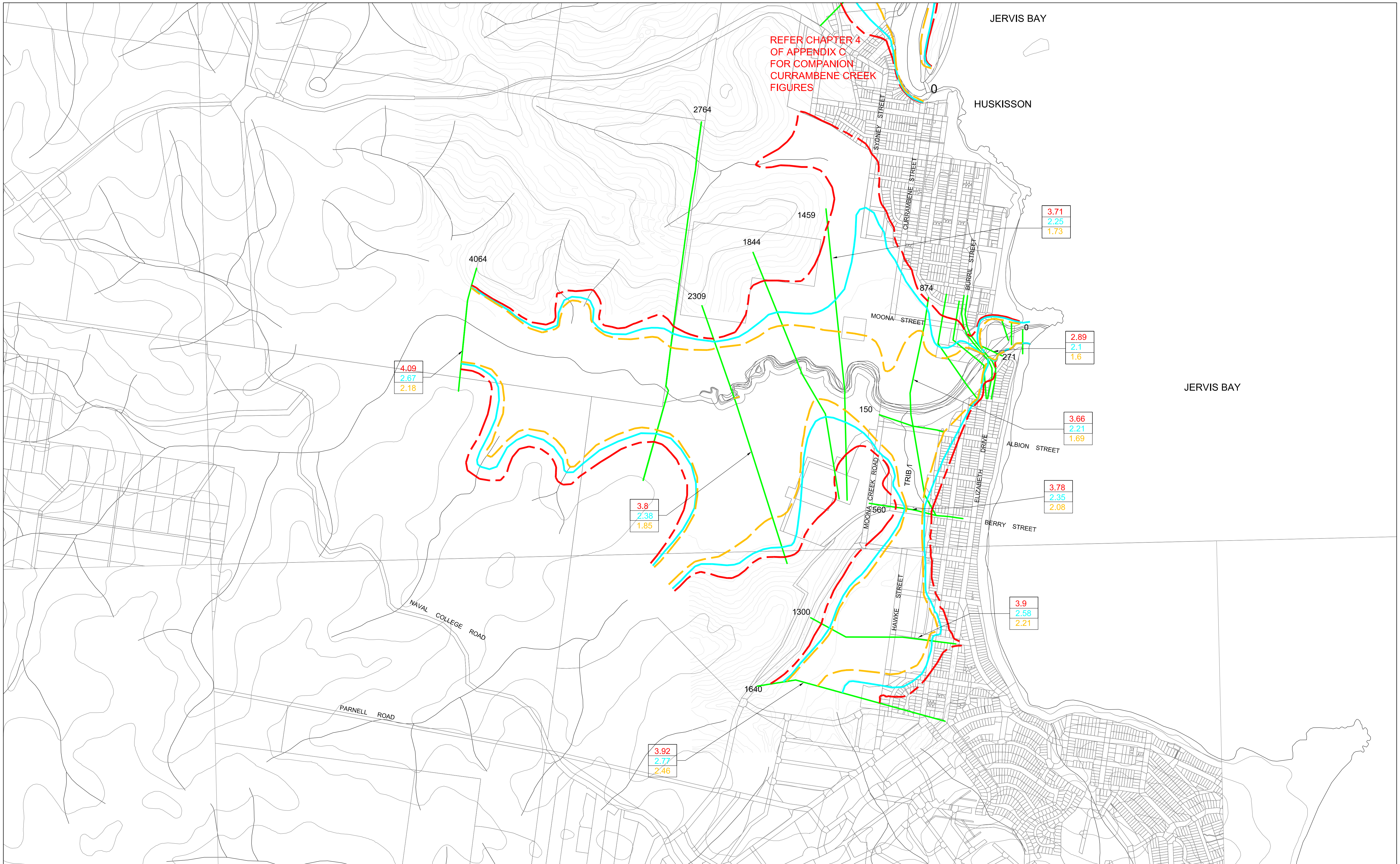
MOONA MOONA CREEK
DESIGN WATER SURFACE PROFILES
5 YEAR ARI TO PMF



CURRAMBENE CREEK AND MOONA MOONA CREEK FLOOD STUDIES

Figure 6.2

DESIGN WATER SURFACE PROFILES
TRIBUTARY 1 OF MOONA MOONA CREEK
5 YEAR ARI TO PMF



NOTE

THE EXTENTS OF FLOODING SHOWN WERE DETERMINED FROM SURVEYED CROSS SECTIONS OF THE CREEK AND FLOODPLAIN AND AVAILABLE CONTOUR DATA AND ARE APPROXIMATE ONLY. THE EXTENT OF INUNDATION OF INDIVIDUAL ALLOTMENTS NEAR THE FLOOD FRINGE SHOULD BE CONFIRMED BY SITE SPECIFIC SURVEY.

N

SCALE 0 250 500m

LEGEND

PMF

100 YEAR ARI

10 YEAR ARI

PEAK FLOOD LEVELS

1.85 10 YEAR ARI

2.38 100 YEAR ARI

3.8 PMF

MODEL RIVER STATION

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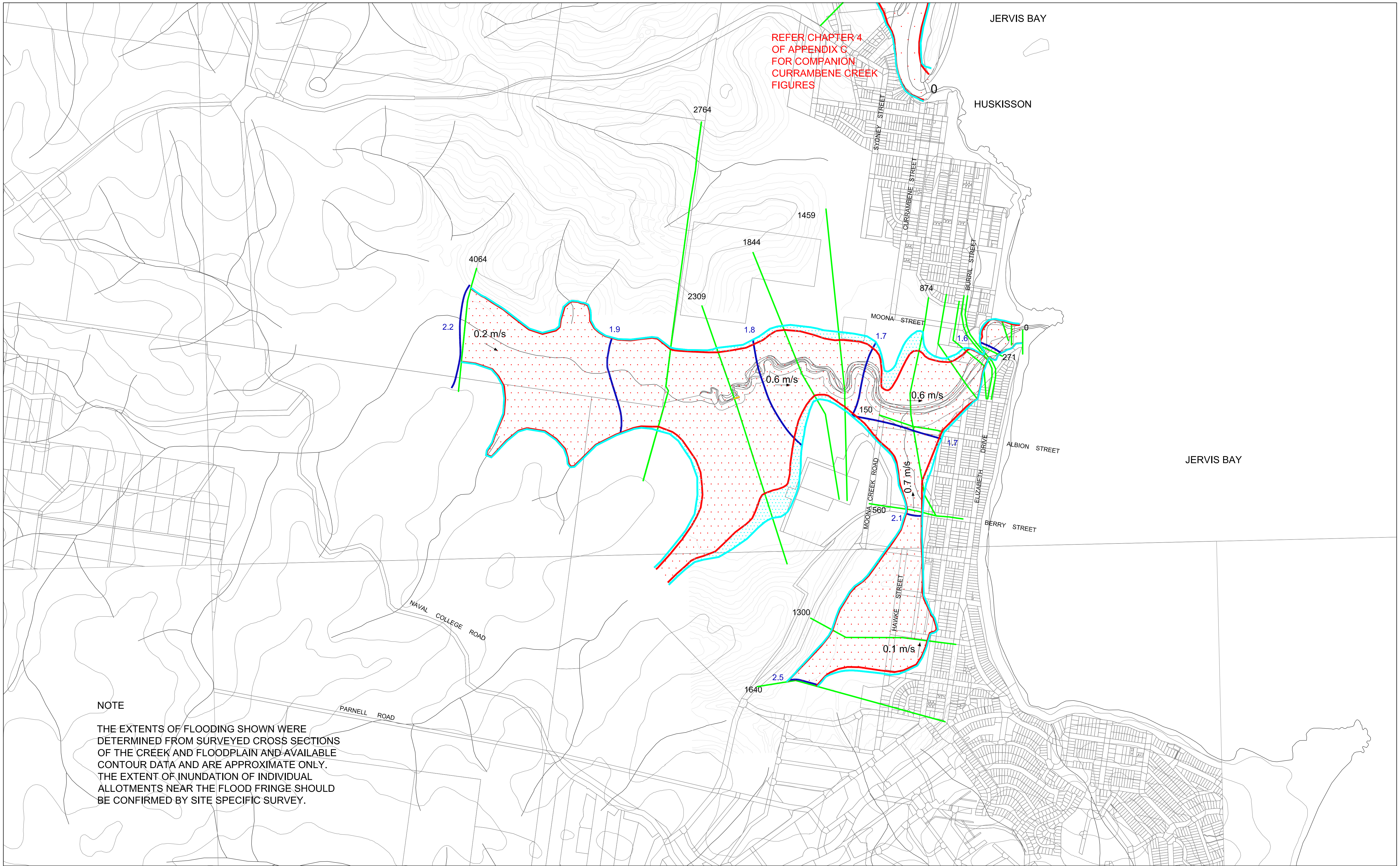
CURRAMBENE CREEK AND MOONA MOONA CREEK FLOOD STUDIES

Figure 6.3

MOONA MOONA CREEK INDICATIVE EXTENTS OF INUNDATION 10 YEAR, 100 YEAR ARI AND PMF



LEGEND

N

SCALE
0 250 500m

FLOODWAY ZONE

FLOOD STORAGE ZONE

10 YEAR ARI

2.5 PEAK FLOOD LEVEL CONTOUR - m AHD

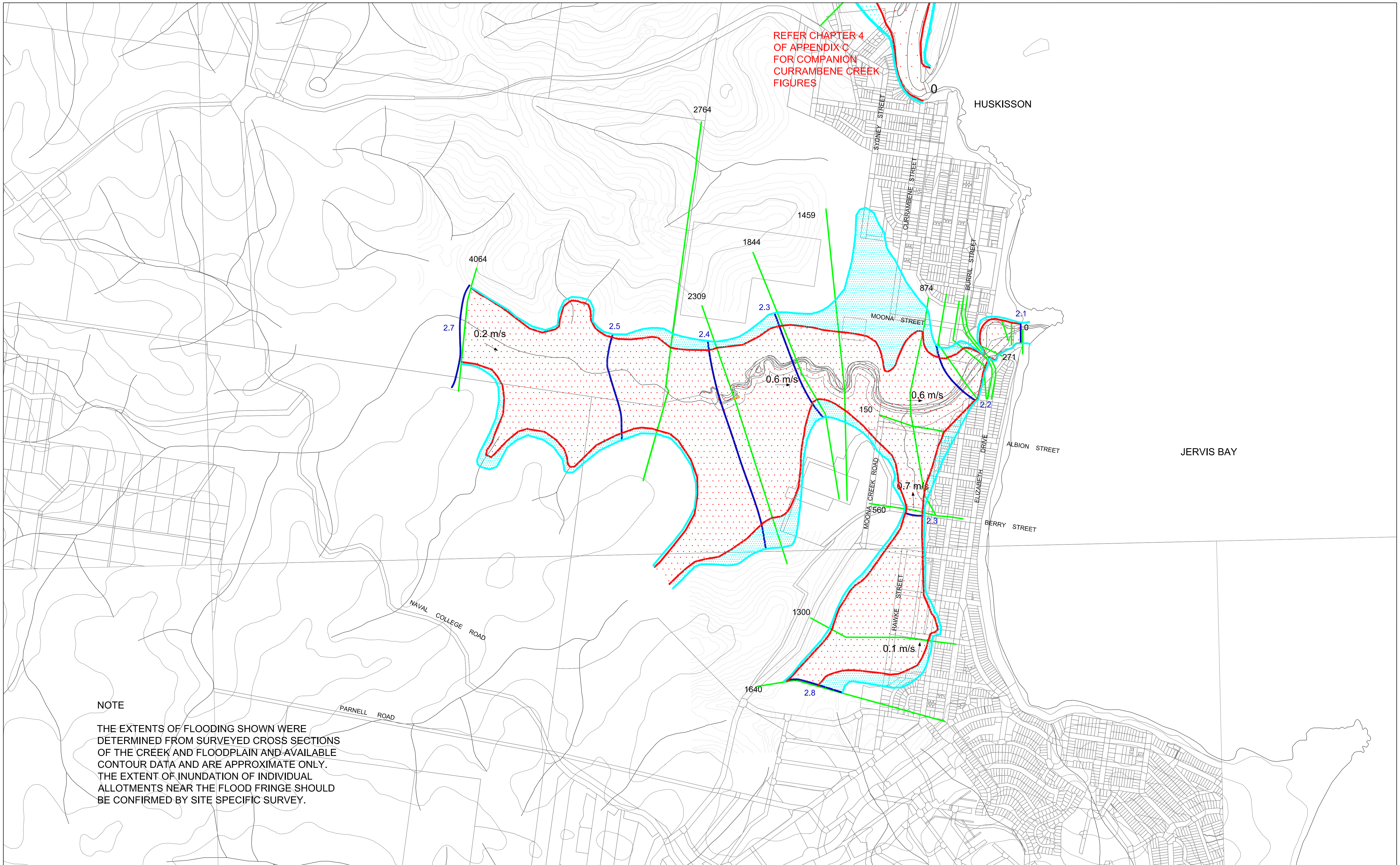
0.1 m/s FLOW VELOCITY - m/s

MODEL RIVER STATION 2309

CURRAMBENE CREEK AND MOONA MOONA CREEK
FLOOD STUDIES

Figure 6.4

MOONA MOONA CREEK
HYDRAULIC CATEGORISATION OF FLOODPLAIN
10 YEAR ARI FLOOD



LEGEND

N

SCALE 0 250 500m

FLOODWAY ZONE

FLOOD STORAGE ZONE

100 YEAR ARI

2.5 PEAK FLOOD LEVEL CONTOUR - m AHD

0.1 m/s FLOW VELOCITY - m/s

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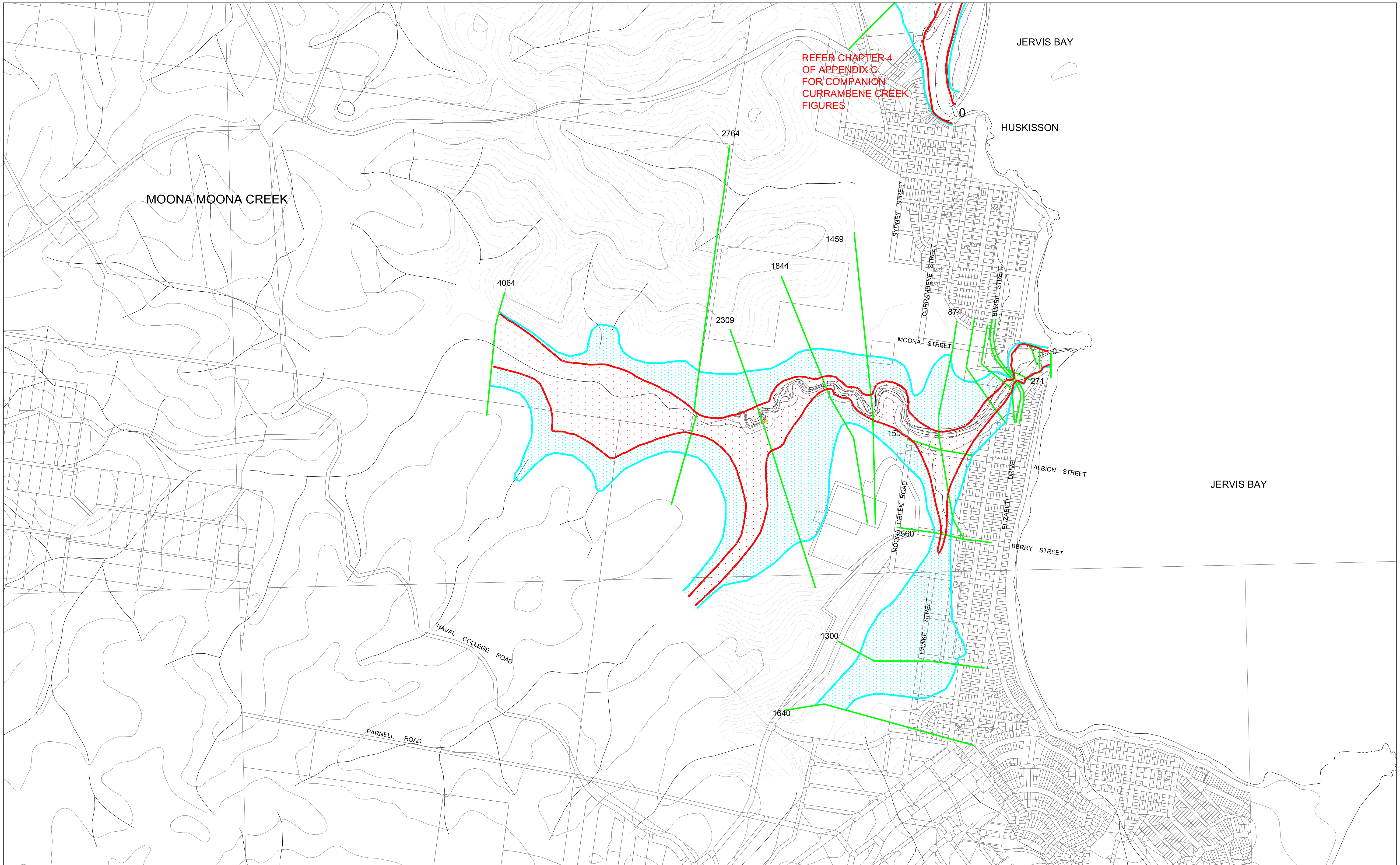
CURRAMBENE CREEK AND MOONA MOONA CREEK FLOOD STUDIES

Figure 6.5

MOONA MOONA CREEK HYDRAULIC CATEGORISATION OF FLOODPLAIN 100 YEAR ARI FLOOD



LEGEND

INDICATIVE FLOOD EXTENT

LOW HAZARD AREA

HIGH HAZARD AREA

MODEL RIVER STATION

2309

N

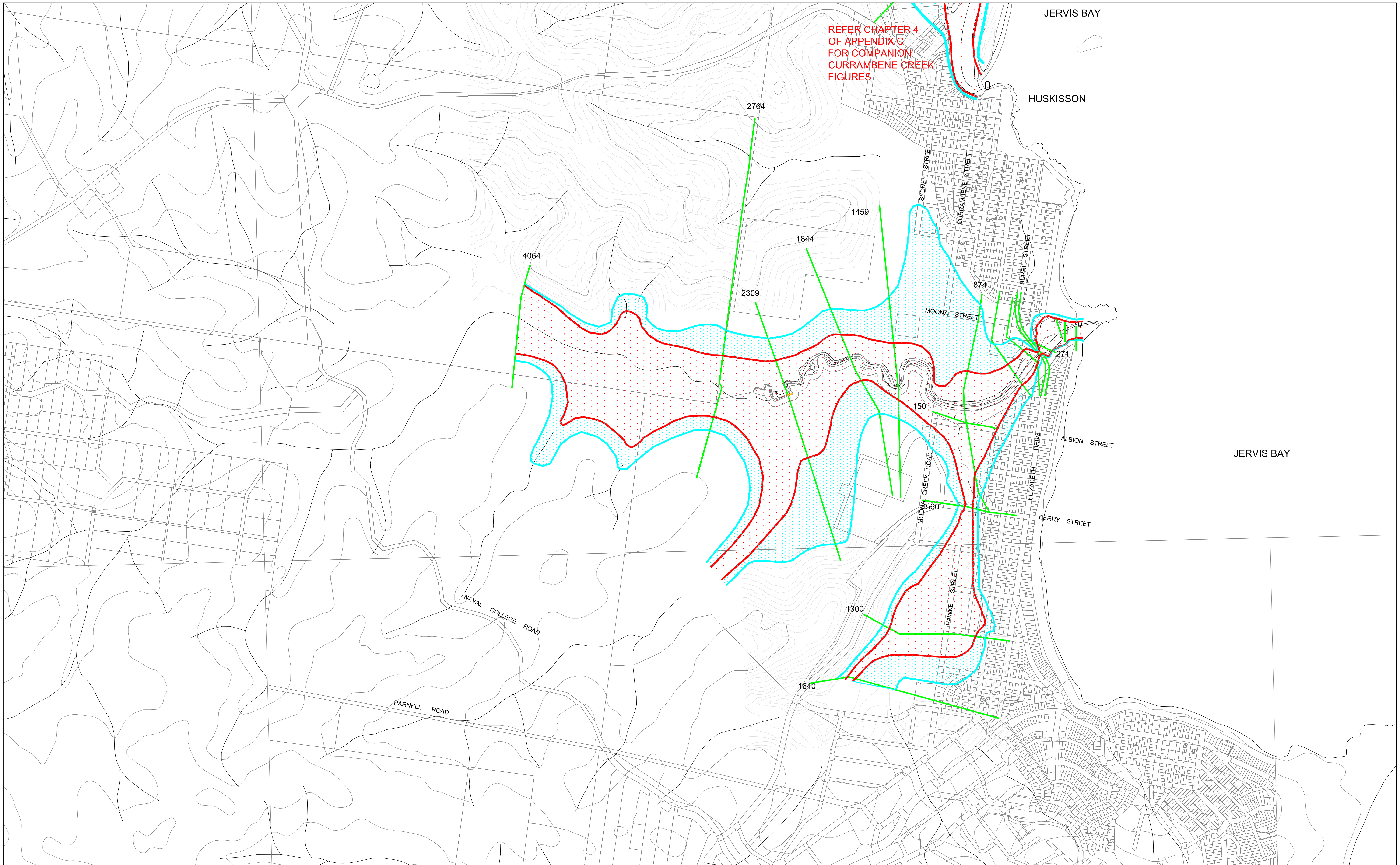
SCALE

0 250 500m

CURRAMBENE CREEK AND MOONA MOONA CREEK
FLOOD STUDIES

Figure 6.6

MOONA MOONA CREEK
PROVISIONAL FLOOD HAZARD
10 YEAR ARI DESIGN FLOOD



N

SCALE

0 250 500m

LEGEND

INDICATIVE FLOOD EXTENT

LOW HAZARD AREA

HIGH HAZARD AREA

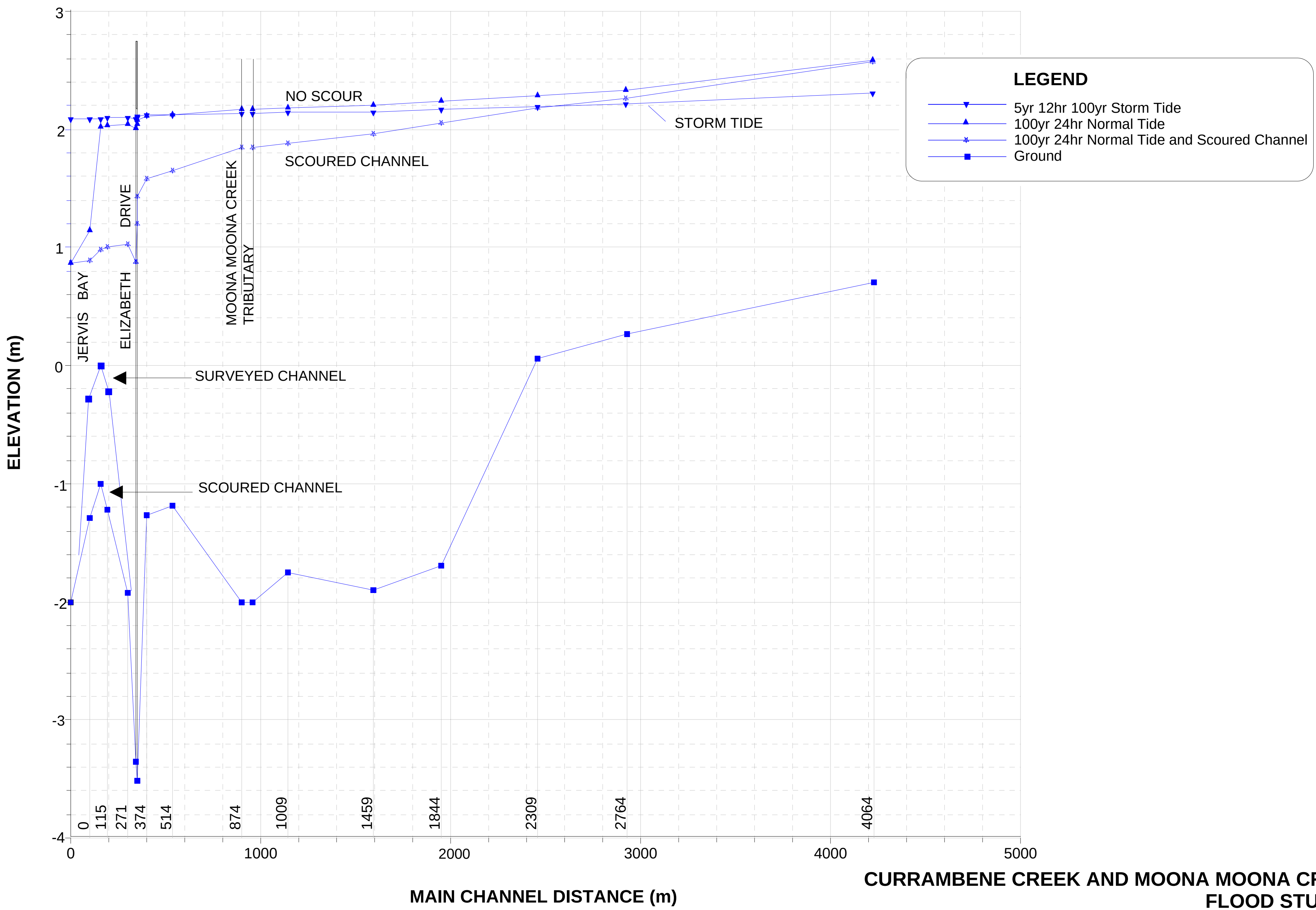
MODEL RIVER STATION

2309

CURRAMBENE CREEK AND MOONA MOONA CREEK
FLOOD STUDIES

Figure 6.7

MOONA MOONA CREEK
PROVISIONAL FLOOD HAZARD
100 YEAR ARI DESIGN FLOOD



CURRAMBENE CREEK AND MOONA MOONA CREEK FLOOD STUDIES

Figure 6.8

MOONA MOONA CREEK
SENSITIVITY OF 100 YEAR ARI
FLOOD LEVELS TO ENTRANCE SCOUR

7. REFERENCES

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ADDENDUM A

CURRAMBENE CREEK PEAK FLOOD LEVELS AND FLOW PATTERNS

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Reach	River Station	Critical Storm	Peak Water Level (m AHD)	5 yr ARI					
				Velocity (m/s)			Flow (m^3/s)		
				Left	Channel	Right	Left	Channel	Right
Curr1	15206	Storm Tide	3.83	0.22	0.69	0.17	12.24	198.38	0.92
Curr1	14633	Storm Tide	3.67	0.12	0.97	0.19	0.05	208.27	2.7
Curr1	13934	Storm Tide	3.41	0.28	0.93	0.09	15.41	194.28	0.02
Curr1	13372	Storm Tide	3.12	0.08	0.94	0.14	0.23	215.61	0.55
Curr1	12390	Storm Tide	2.85	0.21	0.41	0.19	102.28	63.27	32.64
Curr1	11308	Storm Tide	2.76	0.19	0.34	0.17	99.61	51.81	40.56
Curr1	10170	Storm Tide	2.68	0.16	0.36	0.14	45.82	48.68	98.61
				Knoll Parade					
Curr1	9196	Storm Tide	2.58	0.05	0.47	0.14	1.02	80.66	113.45
Curr1	8317	Storm Tide	2.3	0.01	0.87	-	0.01	195.08	-
Curr1	8167	Storm Tide	2.15	-	1.06	-	-	217.39	-
Curr2	8117	Storm Tide	2.15	0.07	0.54	0.07	11.86	223.58	0.33
Curr2	7517	Storm Tide	2.06	0.07	0.55	0.07	9.84	218.68	0.28
				Goodland Road					
Curr2	7052	Storm Tide	2.01	0.08	0.47	0.09	64.98	97.29	60.66
Curr2	6085	Storm Tide	1.87	0.11	0.61	0.06	123.78	84.5	0.62
Curr3	6035	Storm Tide	1.87	0.11	0.61	0.06	124.36	84.9	0.63
Curr3	5935	Storm Tide	1.85	0.11	0.62	0.05	122.41	85.29	0.58
Curr3	4535	Storm Tide	1.66	0.04	0.52	0.09	0.13	98.72	94.93
Curr3	3418	Storm Tide	1.46	-	0.68	0.11	-	147.05	42.81
Curr3	2866	Storm Tide	1.36	-	0.52	0.08	-	155.01	34.78
Curr3	2328	Storm Tide	1.3	-	0.54	0.08	-	156.83	32.7
				Edendale Street					
Curr4	2288	Storm Tide	1.3	-	0.36	0.03	-	193.01	1.65
Curr4	1094	Storm Tide	1.22	-	0.37	0.02	-	187.95	1.01
Curr4	100	Storm Tide	1.15	0.01	0.48	-	0.01	185.89	-
Curr4	0	Storm Tide	1.15	0.01	0.48	-	0.01	185.75	-
Curr4	-100	Storm Tide	1.15	-	0.09	-	-	185.54	-
				Jervis Bay					
TRIB1	490	Storm Tide	2.47	0.01	0.02	0.1	0.91	0.09	90.66
TRIB1	325	Storm Tide	2.45	0.11	0.36	1.19	29.23	19.01	43.4
TRIB1	310	Storm Tide	2.44	0.11	0.36	1.19	29.19	19.05	43.4
				Woollamia Road Crossing					
TRIB1	280	Storm Tide	2.15	0.03	0.11	0.31	5.33	2.64	10.91
TRIB1	0	Storm Tide	2.15	0.01	0.02	0.08	4.87	0.39	13.29
TRIB1	-400	Storm Tide	2.15	0.02	0.11	0.13	0.85	17.42	0.12
TRIB2	852	Storm Tide	1.92	-	0	-	-	1	-
TRIB2	832	Storm Tide	1.92	-	0	-	-	1	-
TRIB2	812	Storm Tide	1.92	0.02	0.04	0.02	0.12	0.78	0.11
				Woollamia Road Crossing					
TRIB2	800	Storm Tide	1.88	0.02	0.05	0.02	0.08	0.84	0.07
TRIB2	20	Storm Tide	1.87	0	0.01	0	0.15	0.83	0.01
TRIB3	1712	Storm Tide	1.95	-	0.02	-	-	7.01	-
TRIB3	1212	Storm Tide	1.94	-	0.01	-	-	6.7	-
TRIB3	997	Storm Tide	1.79	0.06	0.48	0.06	1.32	3.77	1.53
TRIB3	982	Storm Tide	1.77	0.06	0.54	0.06	1.12	4.16	1.26
				Woollamia Road Crossing					
TRIB3	962	Storm Tide	1.75	0.02	0.21	0.02	0.25	5.6	0.68
TRIB3	712	Storm Tide	1.6	0.05	0.49	0.06	0.22	4.36	0.74
				Edendale Street Crossing					
TRIB3	700	Storm Tide	1.57	0.05	0.52	0.05	0.15	4.57	0.51
TRIB3	20	Storm Tide	1.3	-	0.01	0	-	5.05	0.09

\- Refer Note 1 on page 44

Reach	River Station	Critical Storm	Peak Water Level (m AHD)	10 yr ARI					
				Velocity (m/s)			Flow (m^3/s)		
				Left	Channel	Right	Left	Channel	Right
Curr1	15206	12 hr	4.66	0.36	0.93	0.27	32.98	339.71	3.11
Curr1	14633	12 hr	4.4	0.26	1.33	0.39	0.48	339.23	13.4
Curr1	13934	12 hr	4.01	0.46	1.21	0.22	44.45	338.46	0.32
Curr1	13372	12 hr	3.55	0.2	1.26	0.27	5.62	336.25	3.87
Curr1	12390	24 hr	3.23	0.24	0.45	0.22	145.95	75.82	49.23
Curr1	11308	24 hr	3.14	0.22	0.37	0.19	142.2	62.97	59.83
Curr1	10170	24 hr	3.06	0.18	0.37	0.16	67.06	55.19	146.44
					Knoll Parade				
Curr1	9196	24 hr	2.95	0.07	0.54	0.17	2.93	101.41	167.8
Curr1	8317	24 hr	2.65	0.07	1.02	0.04	3.59	268.13	0.01
Curr1	8167	24 hr	2.46	0.06	1.23		0.73	297.92	
Curr2	8117	24 hr	2.46	0.09	0.66	0.09	25.79	300.02	0.73
Curr2	7517	24 hr	2.35	0.09	0.66	0.09	21.38	291.48	0.6
					Goodland Road				
Curr2	7052	24 hr	2.29	0.1	0.52	0.1	99.24	114.93	91.4
Curr2	6085	24 hr	2.14	0.13	0.67	0.08	191.58	101.36	1.68
Curr3	6035	24 hr	2.14	0.13	0.67	0.08	192.2	101.69	1.68
Curr3	5935	24 hr	2.12	0.13	0.68	0.08	190.12	102.36	1.62
Curr3	4535	24 hr	1.9	0.05	0.6	0.11	0.47	124.8	162.58
Curr3	3418	Storm Tide	1.6	-	0.56	0.1	-	173.13	45.49
Curr3	2866	Storm Tide	1.54	-	0.41	0.06	-	172.64	36.47
Curr3	2328	Storm Tide	1.51	-	0.42	0.06	-	171.41	35.39
					Edendale Street				
Curr4	2288	Storm Tide	1.51	-	0.29	0.03	-	176.2	3.36
Curr4	1094	Storm Tide	1.47	-	0.28	0.03	-	167.42	2.82
Curr4	100	Storm Tide	1.43	0.01	0.38	0.01	0.15	162.97	0.01
Curr4	0	Storm Tide	1.43	0.01	0.38	0.01	0.14	162.79	0
Curr4	-100	Storm Tide	1.43	-	0.07	-	-	162.55	-
					Jervis Bay				
TRIB1	490	24 hr	2.81	0.01	0.02	0.11	1.22	0.11	114.05
TRIB1	325	24 hr	2.8	0.11	0.33	1.17	40.34	20.48	54.53
TRIB1	310	24 hr	2.79	0.11	0.33	1.17	40.32	20.5	54.53
					Woollamia Road Crossing				
TRIB1	280	24 hr	2.46	0.03	0.11	0.34	9.01	3.03	16.11
TRIB1	0	24 hr	2.46	0.01	0.02	0.1	7.55	0.55	19.88
TRIB1	-400	24 hr	2.46	0.03	0.15	0.18	1.96	25.71	0.21
TRIB2	852	24 hr	2.18		0			1	
TRIB2	832	24 hr	2.18		0			1	
TRIB2	812	24 hr	2.18	0.01	0.03	0.01	0.13	0.76	0.11
					Woollamia Road Crossing				
TRIB2	800	24 hr	2.14	0.01	0.03	0.01	0.1	0.79	0.09
TRIB2	20	24 hr	2.14	0	0.01	0	0.17	0.76	0.02
TRIB3	1712	24 hr	2.04		0.02			8.38	
TRIB3	1212	24 hr	2.04		0.01			8.28	
TRIB3	997	24 hr	1.86	0.07	0.41	0.07	2.17	3.35	2.67
TRIB3	982	24 hr	1.84	0.07	0.46	0.07	2.01	3.73	2.44
					Woollamia Road Crossing				
TRIB3	962	24 hr	1.83	0.03	0.22	0.03	0.7	6.13	1.35
TRIB3	712	24hr Storm Tide	1.66	0.05	0.38	0.06	0.31	3.5	0.98
					Edendale Street Crossing				
TRIB3	700	24hr Storm Tide	1.65	0.05	0.38	0.06	0.31	3.51	0.97
TRIB3	20	24hr Storm Tide	1.51		0.01	0		4.61	0.18

Refer Notes 2 and 3 on page 44

Reach	River Station	Critical Storm	Peak Water Level (m AHD)	20 yr ARI					
				Velocity (m/s)			Flow (m^3/s)		
				Left	Channel	Right	Left	Channel	Right
Curr1	15206	12 hr	5.38	0.47	1.12	0.35	60.05	403.75	6.65
Curr1	14633	12 hr	5.03	0.38	1.63	0.56	1.52	437.63	30.49
Curr1	13934	12 hr	4.57	0.6	1.41	0.33	83.79	382.66	1.24
Curr1	13372	18 hr	4.09	0.32	1.32	0.36	35.2	393.99	12.98
Curr1	12390	18 hr	3.69	0.3	0.54	0.28	224.72	99.3	79.06
Curr1	11308	18 hr	3.59	0.27	0.44	0.24	215.07	82.73	93.31
Curr1	10170	18 hr	3.5	0.21	0.4	0.19	98.84	65.63	229.1
					Knoll Parade				
Curr1	9196	18 hr	3.38	0.11	0.64	0.21	7.83	134.29	254.88
Curr1	8317	24 hr	3.03	0.12	1.13	0.09	23.94	347.43	0.33
Curr1	8167	24 hr	2.84	0.13	1.41	0.09	14.04	401.46	0.13
Curr2	8117	24 hr	2.84	0.12	0.79	0.12	53.94	403.75	1.51
Curr2	7517	24 hr	2.7	0.12	0.8	0.12	44.73	388.89	1.27
					Goodland Road				
Curr2	7052	24 hr	2.63	0.12	0.56	0.12	149.81	138.02	136.7
Curr2	6085	24 hr	2.48	0.16	0.72	0.11	287.77	119.72	3.52
Curr3	6035	24 hr	2.48	0.16	0.72	0.11	288.46	120.01	3.53
Curr3	5935	24 hr	2.46	0.16	0.73	0.11	286.73	120.78	3.45
Curr3	4535	24 hr	2.25	0.07	0.63	0.13	1.36	146.9	257.08
Curr3	3418	24 hr	1.88	0.05	1.09	0.2	0.13	276.5	135.44
Curr3	2866	Storm Tide	1.76	-	0.3	0.05	-	108.43	34.02
Curr3	2328	Storm Tide	1.74	-	0.29	0.05	-	106.31	33.03
					Edendale Street				
Curr4	2288	Storm Tide	1.74	-	0.2	0.02	-	138.07	4.53
Curr4	1094	Storm Tide	1.73	-	0.18	0.02	-	123.23	3.91
Curr4	100	Storm Tide	1.71	0.02	0.26	0.01	0.42	119.21	0.05
Curr4	0	Storm Tide	1.71	0.02	0.26	0.01	0.41	118.91	0.05
Curr4	-100	Storm Tide	1.71	-	0.05	-	-	118.98	-
					Jervis Bay				
TRIB1	490	12 hr	3.13	0.01	0.03	0.13	1.78	0.14	158.52
TRIB1	325	12 hr	3.1	0.13	0.36	1.31	56.99	24.97	73.01
TRIB1	310	24 hr	3.1	0.11	0.31	1.15	50	21.94	64.09
					Woollamia Road Crossing				
TRIB1	280	24 hr	2.85	0.04	0.13	0.42	15.57	3.88	25.39
TRIB1	0	24 hr	2.84	0.01	0.03	0.13	12.18	0.83	31.23
TRIB1	-400	24 hr	2.84	0.05	0.2	0.26	4.42	38.74	0.4
TRIB2	852	24 hr	2.5	-	0	-	-	1	-
TRIB2	832	24 hr	2.5	-	0	-	-	1	-
TRIB2	812	24 hr	2.5	0.01	0.02	0.01	0.14	0.74	0.12
					Woollamia Road Crossing				
TRIB2	800	24 hr	2.48	0.01	0.02	0.01	0.12	0.77	0.1
TRIB2	20	24 hr	2.48	0	0	0	0.2	0.75	0.03
TRIB3	1712	24 hr	2.2	-	0.02	-	-	11.35	-
TRIB3	1212	18 hr	2.2	-	0.01	-	-	11.17	-
TRIB3	997	18 hr	2	0.07	0.32	0.07	3.61	2.78	4.63
TRIB3	982	18 hr	1.97	0.08	0.35	0.07	3.51	3.04	4.47
					Woollamia Road Crossing				
TRIB3	962	18 hr	1.97	0.04	0.22	0.04	1.76	6.51	2.74
TRIB3	712	24 hr	1.78	0.09	0.58	0.12	1.31	5.73	3.4
					Edendale Street Crossing				
TRIB3	700	24 hr	1.78	0.09	0.58	0.12	1.31	5.73	3.39
TRIB3	20	24 hr Storm Tide	1.74	-	0	0	-	3.06	0.2

- Refer Notes 4 and 5 on page 44

Reach	River Station	Critical Storm	Peak Water Level (m AHD)	50 yr ARI					
				Velocity (m/s)			Flow (m^3/s)		
				Left	Channel	Right	Left	Channel	Right
Curr1	15206	12 hr	6.12	0.59	1.31	0.45	98.58	522.92	12.58
Curr1	14633	12 hr	5.68	0.52	1.95	0.75	3.57	572.86	57.14
Curr1	13934	12 hr	5.13	0.74	1.63	0.44	138.61	490.35	3.14
Curr1	13372	12 hr	4.63	0.47	1.49	0.46	99.76	501.7	31.24
Curr1	12390	12 hr	4.2	0.37	0.62	0.34	331.1	128.21	120.23
Curr1	11308	12 hr	4.07	0.33	0.52	0.28	316.66	108.74	138.83
Curr1	10170	12 hr	3.97	0.23	0.45	0.23	134.83	81.24	349.95
					Knoll Parade				
Curr1	9196	12 hr	3.82	0.16	0.76	0.26	16.2	177	374.87
Curr1	8317	12 hr	3.44	0.21	1.35	0.15	85.7	480.72	1.65
Curr1	8167	18 hr	3.24	0.21	1.58	0.15	65.18	526.07	1.07
Curr2	8117	18 hr	3.24	0.16	0.95	0.13	108.08	534.63	2.73
Curr2	7517	18 hr	3.07	0.15	0.97	0.14	87.58	521.11	2.3
					Goodland Road				
Curr2	7052	24 hr	2.99	0.14	0.62	0.14	214.96	165.75	195.12
Curr2	6085	24 hr	2.83	0.18	0.78	0.13	411.36	142.56	6.15
Curr3	6035	24 hr	2.83	0.18	0.78	0.13	412.09	142.81	6.16
Curr3	5935	24 hr	2.81	0.18	0.79	0.13	410.16	143.47	6.07
Curr3	4535	24 hr	2.6	0.09	0.67	0.15	3.2	173.86	374.87
Curr3	3418	24 hr	2.22	0.09	1.25	0.22	1.67	352.49	212.98
Curr3	2866	24 hr	1.97	-	1.05	0.18	-	418.78	147.68
Curr3	2328	Storm Tide	1.85	-	0.28	0.05	-	106.08	35.1
					Edendale Street				
Curr4	2288	Storm Tide	1.85	-	0.19	0.02	-	137.55	5.32
Curr4	1094	Storm Tide	1.83	-	0.18	0.02	-	128.69	4.86
Curr4	100	Storm Tide	1.82	0.02	0.27	0.02	0.61	128.32	0.08
Curr4	0	Storm Tide	1.82	0.02	0.27	0.02	0.61	128.13	0.08
Curr4	-100	Storm Tide	1.82	-	0.05	-	-	128.57	-
					Jervis Bay				
TRIB1	490	18 hr	3.26	0	0.01	0.04	0.61	0.05	53.3
TRIB1	325	24 hr	3.25	0.05	0.14	0.53	25.25	10.46	31.7
TRIB1	310	24 hr	3.25	0.05	0.14	0.53	25.25	10.46	31.7
					Woollamia Road Crossing				
TRIB1	280	18 hr	3.25	0.04	0.11	0.4	19.61	3.96	30.07
TRIB1	0	18 hr	3.25	0.01	0.03	0.13	14.89	0.95	37.53
TRIB1	-400	18 hr	3.24	0.06	0.22	0.28	6.94	45.64	0.55
TRIB2	852	24 hr	2.83	-	0	-	-	1	-
TRIB2	832	24 hr	2.83	-	0	-	-	1	-
TRIB2	812	24 hr	2.83	0.01	0.02	0.01	0.14	0.74	0.12
					Woollamia Road Crossing				
TRIB2	800	24 hr	2.83	0.01	0.02	0.01	0.13	0.76	0.11
TRIB2	20	24 hr	2.83	0	0	0	0.23	0.73	0.04
TRIB3	1712	12 hr	2.4	-	0.02	-	-	15.44	-
TRIB3	1212	12 hr	2.4	-	0.02	-	-	15.15	-
TRIB3	997	12 hr	2.13	0.08	0.28	0.07	5.34	2.64	6.98
TRIB3	982	12 hr	2.1	0.08	0.31	0.08	5.25	2.85	6.84
					Woollamia Road Crossing				
TRIB3	962	12 hr	2.1	0.05	0.23	0.05	3.17	7.26	4.51
TRIB3	712	24 hr	1.96	0.09	0.48	0.12	2.78	5.18	5.6
					Edendale Street Crossing				
TRIB3	700	24 hr	1.96	0.09	0.48	0.12	2.77	5.2	5.6
TRIB3	20	24 hr Storm Tide	1.85	-	0	0	-	1.93	0.15

- Refer Notes 6 and 7 on page 44

Reach	**River Station**	**Critical Storm**	**Peak Water Level (m AHD)**	**100 yr ARI**					
				Velocity (m/s)			**Flow (m^3/s)**		
				Left	**Channel**	**Right**	**Left**	**Channel**	**Right**
Curr1	15206	12 hr	6.58	0.67	1.44	0.51	129.12	606.86	17.78
Curr1	14633	12 hr	6.07	0.6	2.16	0.86	5.55	667.68	78.5
Curr1	13934	12 hr	5.48	0.82	1.76	0.51	180.43	562.84	4.99
Curr1	13372	12 hr	4.99	0.55	1.54	0.52	159	557.14	49.61
Curr1	12390	12 hr	4.56	0.41	0.68	0.39	413.02	148.96	153.48
Curr1	11308	12 hr	4.43	0.36	0.56	0.31	396.67	127.53	175.43
Curr1	10170	12 hr	4.32	0.24	0.48	0.26	167.41	91.82	446.54
					Knoll Parade				
Curr1	9196	12 hr	4.16	0.19	0.84	0.3	24.43	210.37	475.98
Curr1	8317	12 hr	3.79	0.26	1.41	0.18	150.14	556.66	3.67
Curr1	8167	12 hr	3.58	0.3	1.75	0.2	140.29	650.94	2.99
Curr2	8117	12 hr	3.58	0.21	1.1	0.16	178.25	671.92	4.56
Curr2	7517	18 hr	3.38	0.19	1.05	0.15	140.63	615.8	3.54
					Goodland Road				
Curr2	7052	18 hr	3.29	0.16	0.68	0.16	284.98	195.42	258.08
Curr2	6085	18 hr	3.12	0.21	0.84	0.15	539.15	166.18	9.02
Curr3	6035	18 hr	3.12	0.21	0.84	0.15	539.91	166.42	9.04
Curr3	5935	18 hr	3.1	0.21	0.85	0.15	537.32	166.96	8.93
Curr3	4535	18 hr	2.88	0.11	0.72	0.17	5.3	200.99	491.61
Curr3	3418	18 hr	2.49	0.12	1.28	0.25	4.56	390.28	304.4
Curr3	2866	18 hr	2.24	0.01	1.12	0.2	0.01	497.06	196.85
Curr3	2328	18 hr	2.05	-	1.23	0.22	-	506.97	185.43
					Edendale Street				
Curr4	2288	18 hr	2.05	0.03	0.85	0.11	0.03	673.75	33.35
Curr4	1094	Storm Tide	1.9	0	0.14	0.02	0	101.18	4.18
Curr4	100	Storm Tide	1.89	0.02	0.2	0.01	0.55	98.24	0.08
Curr4	0	Storm Tide	1.89	0.02	0.2	0.01	0.55	98	0.08
Curr4	-100	Storm Tide	1.89	-	0.04	-	-	98.3	-
					Jervis Bay				
TRIB1	490	12 hr	3.6	0	0.01	0.04	0.78	0.06	65.01
TRIB1	325	18 hr	3.59	0.05	0.13	0.48	27.85	10.38	33.65
TRIB1	310	18 hr	3.59	0.05	0.13	0.48	27.81	10.36	33.61
					Woollamia Road Crossing				
TRIB1	280	18 hr	3.59	0.04	0.13	0.45	26.92	4.8	39.85
TRIB1	0	12 hr	3.59	0.01	0.03	0.13	17.56	1.08	43.87
TRIB1	-400	12 hr	3.58	0.06	0.23	0.29	9.2	50.65	0.66
TRIB2	852	18 hr	3.12	-	0	-	-	1	-
TRIB2	832	18 hr	3.12	-	0	-	-	1	-
TRIB2	812	18 hr	3.12	0.01	0.02	0.01	0.15	0.73	0.12
					Woollamia Road Crossing				
TRIB2	800	18 hr	3.12	0.01	0.02	0.01	0.13	0.75	0.11
TRIB2	20	18 hr	3.12	0	0	0	0.24	0.71	0.05
TRIB3	1712	12 hr	2.59	-	0.02	-	-	19.89	-
TRIB3	1212	12 hr	2.59	-	0.02	-	-	19.75	-
TRIB3	997	24 hr	2.24	0.07	0.24	0.07	6.06	2.33	8
TRIB3	982	24 hr	2.21	0.07	0.25	0.07	5.93	2.44	7.81
					Woollamia Road Crossing				
TRIB3	962	24 hr	2.21	0.05	0.21	0.05	3.97	6.81	5.41
TRIB3	712	18 hr	2.15	0.08	0.34	0.1	4.22	4.01	6.43
					Edendale Street Crossing				
TRIB3	700	18 hr	2.15	0.08	0.34	0.1	4.22	4.01	6.43
TRIB3	20	18 hr	2.05	0	0.02	0	0	13.39	1.33

Refer Notes 8 and 9 on page 44

Reach	River Station	Critical Storm	Peak Water Level (m AHD)	200 yr ARI					
				Velocity (m/s)			Flow (m^3/s)		
				Left	Channel	Right	Left	Channel	Right
Curr1	15206	12 hr	7.16	0.77	1.61	0.59	174.14	720.8	26.03
Curr1	14633	12 hr	6.55	0.72	2.44	0.99	8.89	799.73	110.16
Curr1	13934	12 hr	5.87	0.94	1.95	0.6	239.91	663.76	7.97
Curr1	13372	12 hr	5.38	0.64	1.61	0.61	237.82	627.14	76.7
Curr1	12390	12 hr	4.95	0.46	0.75	0.44	512.82	175.77	197.02
Curr1	11308	12 hr	4.8	0.41	0.62	0.34	495.09	150.17	222.01
Curr1	10170	12 hr	4.69	0.26	0.51	0.28	211.15	104.17	562.77
					Knoll Parade				
Curr1	9196	12 hr	4.51	0.22	0.93	0.34	35.2	249.43	598.18
Curr1	8317	12 hr	4.14	0.3	1.47	0.2	229.92	644.01	6.85
Curr1	8167	12 hr	3.95	0.35	1.79	0.24	230.65	742.42	6.2
Curr2	8117	12 hr	3.95	0.25	1.19	0.16	260.19	789.99	6.74
Curr2	7517	12 hr	3.72	0.24	1.21	0.17	222.02	765.1	5.71
					Goodland Road				
Curr2	7052	12 hr	3.62	0.19	0.76	0.19	379.77	235.14	343.53
Curr2	6085	12 hr	3.43	0.24	0.93	0.18	705.77	196.95	12.93
Curr3	6035	12 hr	3.43	0.24	0.93	0.18	706.52	197.16	12.94
Curr3	5935	12 hr	3.4	0.24	0.94	0.18	703	197.61	12.79
Curr3	4535	12 hr	3.16	0.14	0.79	0.2	8.11	237.06	641.66
Curr3	3418	12 hr	2.76	0.15	1.34	0.28	9.28	440.64	415.53
Curr3	2866	12 hr	2.51	0.05	1.22	0.23	1.18	592.34	258.63
Curr3	2328	12 hr	2.3	0.03	1.32	0.24	0.1	599.69	243.02
					Edendale Street				
Curr4	2288	12 hr	2.3	0.06	0.92	0.13	0.18	805.25	50.3
Curr4	1094	Storm Tide	2	0	0.12	0.02	0	92.21	4.3
Curr4	100	Storm Tide	1.99	0.01	0.18	0.01	0.62	90.1	0.1
Curr4	0	Storm Tide	1.99	0.01	0.18	0.01	0.61	89.87	0.1
Curr4	-100	Storm Tide	1.99	-	0.04	-	-	90.32	-
					Jervis Bay				
TRIB1	490	12 hr	3.97	0	0.01	0.05	1.06	0.07	83.45
TRIB1	325	12 hr	3.96	0.05	0.13	0.49	33.68	11.48	39.38
TRIB1	310	12 hr	3.96	0.05	0.13	0.49	33.67	11.48	39.38
					Woollamia Road Crossing				
TRIB1	280	12 hr	3.96	0.04	0.12	0.45	31.85	5.08	45.71
TRIB1	0	12 hr	3.96	0.02	0.03	0.15	22.78	1.36	56.61
TRIB1	-400	12 hr	3.95	0.08	0.26	0.33	13.38	63.39	0.88
TRIB2	852	12 hr	3.43	-	0	-	-	1	-
TRIB2	832	12 hr	3.43	-	0	-	-	1	-
TRIB2	812	12 hr	3.43	0.01	0.01	0.01	0.15	0.73	0.12
					Woollamia Road Crossing				
TRIB2	800	12 hr	3.43	0.01	0.01	0.01	0.14	0.75	0.11
TRIB2	20	12 hr	3.43	0	0	0	0.25	0.66	0.06
TRIB3	1712	12 hr	2.74	-	0.02	-	-	24.14	-
TRIB3	1212	12 hr	2.74	-	0.02	-	-	23.94	-
TRIB3	997	18 hr	2.42	0.06	0.17	0.06	6.27	1.78	8.34
TRIB3	982	18 hr	2.4	0.06	0.17	0.06	6.17	1.81	8.21
					Woollamia Road Crossing				
TRIB3	962	18 hr	2.4	0.04	0.16	0.04	4.62	5.56	6.01
TRIB3	712	18 hr	2.36	0.07	0.26	0.09	5.72	3.29	7.05
					Edendale Street Crossing				
TRIB3	700	18 hr	2.35	0.07	0.26	0.09	5.7	3.31	7.05
TRIB3	20	12 hr	2.3	0	0.01	0	0.01	11.48	1.43

Refer Notes 10 and 11 on page 44

Reach	River Station	Critical Storm	Peak Water Level (m AHD)	PMF					
				Velocity (m/s)			Flow (m^3/s)		
				Left	Channel	Right	Left	Channel	Right
Curr1	15206	6 hr	9.8	1.23	2.38	0.89	499.87	1375.89	95.04
Curr1	14633	6 hr	8.71	1.32	3.74	1.59	42.19	1545.63	375.17
Curr1	13934	6 hr	7.68	1.39	2.79	1.02	655.97	1216.27	36.23
Curr1	13372	6 hr	7.15	0.91	1.87	0.92	727.77	964.96	248.36
Curr1	12390	6 hr	6.72	0.66	1.05	0.62	1149.37	319.07	441.61
Curr1	11308	6 hr	6.51	0.59	0.85	0.49	1070.45	272.16	537.49
Curr1	10170	6 hr	6.37	0.36	0.64	0.39	491.95	168.4	1246.35
				Knoll Parade					
Curr1	9196	6 hr	6.12	0.37	1.34	0.51	114.03	468.48	1339.5
Curr1	8317	6 hr	5.72	0.47	1.81	0.38	741.55	1122.27	42.06
Curr1	8167	6 hr	5.55	0.54	2.1	0.43	799.78	1260.42	43.79
Curr2	8117	6 hr	5.55	0.41	1.64	0.25	805.42	1450.34	36.07
Curr2	7517	6 hr	5.27	0.41	1.66	0.24	733.46	1408.87	29.68
				Goodland Road					
Curr2	7052	6 hr	5.14	0.28	1	0.27	890.14	408.19	808.45
Curr2	6085	6 hr	4.92	0.34	1.2	0.25	1644.76	337.79	37.97
Curr3	6035	6 hr	4.92	0.34	1.2	0.25	1645.57	337.95	37.99
Curr3	5935	6 hr	4.89	0.34	1.2	0.25	1637.71	337.68	37.88
Curr3	4535	6 hr	4.63	0.22	1	0.29	28.96	406.67	1519.99
Curr3	3418	6 hr	4.27	0.29	1.53	0.4	65.13	703.51	1163.54
Curr3	2866	6 hr	4.02	0.2	1.59	0.34	74.31	1165.89	665.46
Curr3	2328	6 hr	3.8	0.2	1.68	0.35	63.21	1171.98	652.3
				Edendale Street					
Curr4	2288	6 hr	3.8	0.13	1.19	0.2	3.57	1646.5	215.46
Curr4	1094	6 hr	3.52	0.14	1.29	0.21	2.91	1662.26	197.19
Curr4	100	6 hr	2.05	0.3	3.63	0.25	13.91	1837.82	2.23
Curr4	0	6 hr	1.74	0.3	3.63	0.25	13.91	1837.82	2.23
Curr4	-100	6 hr	1.89	-	0.75	-	-	1824.82	-
				Jervis Bay					
TRIB1	490	6 hr	5.59	0.01	0.02	0.08	3.04	0.16	198.25
TRIB1	325	6 hr	5.58	0.07	0.18	0.71	84.71	23.32	90.78
TRIB1	310	6 hr	5.58	0.07	0.18	0.7	84.54	23.27	90.6
				Woollamia Road Crossing					
TRIB1	280	6 hr	5.58	0.07	0.17	0.67	81.17	9.85	107.39
TRIB1	0	6 hr	5.58	0.02	0.05	0.22	55	3.01	137.12
TRIB1	-400	6 hr	5.55	0.16	0.45	0.52	44.16	141.45	2.24
TRIB2	852	6 hr	4.92	-	0	-	-	1	-
TRIB2	832	6 hr	4.92	-	0	-	-	1	-
TRIB2	812	6 hr	4.92	0	0.01	0	0.15	0.73	0.12
				Woollamia Road Crossing					
TRIB2	800	6 hr	4.92	0	0.01	0	0.14	0.74	0.11
TRIB2	20	6 hr	4.92	0	0	0	0.29	0.61	0.09
TRIB3	1712	6 hr	3.77	-	0	-	-	2.21	-
TRIB3	1212	6 hr	3.77	-	0	-	-	3.21	-
TRIB3	997	6 hr	3.77	-0.02	-0.04	-0.02	-5.6	-0.63	-7.61
TRIB3	982	6 hr	3.78	-0.02	-0.04	-0.02	-6.35	-0.71	-8.63
				Woollamia Road Crossing					
TRIB3	962	6 hr	3.78	-0.02	-0.05	-0.02	-6.06	-2.66	-7.2
TRIB3	712	6 hr	3.78	-0.03	-0.08	-0.04	-9.14	-1.58	-7.8
				Edendale Street Crossing					
TRIB3	700	6 hr	3.78	-0.03	-0.08	-0.04	-9.2	-1.59	-7.85
TRIB3	20	6 hr	3.8	0	-0.01	0	-0.08	-17.34	-4.54

- Refer Note 12 on page 44

Notes Relating to Previous Tables for Currambene Creek

5 Year ARI

Note 1: Critical flood levels result from catchment flooding due to 5 year ARI storm of 24 hours duration in combination with 1 in 5 year Storm Tide.

10 Year ARI

Note 2: Critical flood levels upstream of RS 3418 result from catchment flooding due to 10 year ARI storms of the durations shown in combination with Normal Semi-Diurnal Tidal Hydrographs.

Note 3: Critical flood levels downstream of RS 3412 result from 1 in 10 year Storm Tides in conjunction with catchment flooding from a minor 5 year ARI storm of 24 hours duration.

20 Year ARI

Note 4: Critical flood levels upstream of RS 2866 result from catchment flooding due to 20 year ARI storms of the durations shown in combination with Normal Semi-Diurnal Tidal Hydrographs.

Note 5: Critical flood levels downstream of RS 2866 result from 1 in 20 year Storm Tides in conjunction with catchment flooding from a minor 5 year ARI storm of 24 hours duration.

50 Year ARI

Note 6: Critical flood levels upstream of RS 2328 result from catchment flooding due to 50 year ARI storms of the durations shown in combination with Normal Semi-Diurnal Tidal Hydrographs.

Note 7: Critical flood levels downstream of RS 2328 result from 1 in 50 year Storm Tides in conjunction with catchment flooding from a minor 5 year ARI storm of 24 hours duration.

100 Year ARI

Note 8: Critical flood levels upstream of RS 2288 result from catchment flooding due to 100 year ARI storms of the durations shown in combination with Normal Semi-Diurnal Tidal Hydrographs.

Note 9: Critical flood levels downstream of RS 2288 result from 1 in 100 year Storm Tides in conjunction with catchment flooding from a minor 5 year ARI storm of 24 hours duration.

200 Year ARI

Note 10: Critical flood levels upstream of RS 2288 result from catchment flooding due to 200 year ARI storms of the durations shown in combination with Normal Semi-Diurnal Tidal Hydrographs.

Note 11: Critical flood levels downstream of RS 2288 result from 1 in 200 year Storm Tides in conjunction with catchment flooding from a minor 5 year ARI storm of 24 hours duration.

Probable Maximum Flood

Note 12: Critical flood levels result from 1 in 100 year storm tides in conjunction with catchment flooding from PMP storm of 6 hours duration.

ADDENDUM B

MOONA MOONA CREEK
PEAK FLOOD LEVELS
AND FLOW PATTERNS

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Reach	River Station	Critical Storm	Peak Water Level (m AHD)	5 yr ARI					
				Velocity (m/s)			Flow (m^3/s)		
				Left	Channel	Right	Left	Channel	Right
Moona	4064	Storm Tide	2.05	0.10	0.11	0.03	7.06	36.37	0.02
Moona	2764	Storm Tide	1.79	0.08	0.16	0.1	8.81	17.39	12.18
Moona	2309	Storm Tide	1.72	0.08	0.15	0.08	14.69	2.7	25.08
Moona	1844	Storm Tide	1.61	0.11	0.51	0.09	10.91	21.89	8.54
Moona	1459	Storm Tide	1.55	0.05	0.61	0.09	7.37	27.17	5.92
Moona	1009	Storm Tide	1.52	0.07	0.49	0.06	4.57	41.09	5.8
Moona	937	Storm Tide	1.5	0.04	0.52	0.05	4.57	46.62	9
			Junction with Tributary (Moona 2)						
Moona	874	Storm Tide	1.5	0.05	0.63	0.06	5.53	56.44	10.9
Moona	514	Storm Tide	1.43	0.11	0.91	0.15	10.51	56.06	3.18
Moona	374	24 hr	1.42	0.08	0.43	0.02	2	50.87	0
Moona	336	24 hr	1.41	-	0.65	-	-	52.88	-
Moona	324	24 hr	1.38	-	0.82	-	-	52.72	-
Moona	271	24 hr	1.39	-	0.2	-	-	52.8	-
Moona	115	24 hr	1.38	-	0.4	-	-	52.77	-
Moona	68	24 hr	1.37	-	0.49	-	-	52.75	-
Moona	0	Storm Tide	1.26	-	0.92	-	-	65.96	-
Moona	-100	Storm Tide	1.22	-	0.08	-	-	65.85	-
			Jervis Bay						
Moona 2	1640	9 hr	2.38	-	0.24	-	-	18.24	-
Moona 2	1300	12 hr	2.1	-	0.04	-	-	14.02	-
Moona 2	570	12 hr	1.99	0.08	0.4	0.08	1.35	11.11	1.29
Moona 2	560	12 hr	1.99	0.09	0.46	0.09	1.56	12.89	1.48
			Berry Street						
Moona 2	550	12 hr	1.85	0.02	0.26	0.03	0.02	14.24	0.06
Moona 2	150	Storm Tide	1.5	0.05	0.15	0.03	11.77	0.62	0.29
Moona 2	20	Storm Tide	1.5	-	0.13	-	-	12.68	-
			Junction with Moona Moona Creek						

Notes:
1. Critical flood levels between RS 68 and 374 on Moona Moona Creek and on the middle and upper reaches of the Tributary (Moona 2) result from catchment flooding due to 5 year ARI storms of 12 and 24 hour duration.
2. Elsewhere, critical flood levels result from 1 in 5 year Storm Tides in conjunction with catchment flooding from 5 year ARI storms of between 12 and 24 hour duration.
3. “Moona” is the main arm of Moona Moona Creek; “Moona 2” is the tributary running between the STP and Elizabeth Drive.

Reach	River Station	Critical Storm	Peak Water Level (m AHD)	10 yr ARI					
				Velocity (m/s)			Flow (m^3/s)		
				Left	Channel	Right	Left	Channel	Right
Moona	4064	12 hr	2.18	0.12	0.13	0.04	9.31	47.62	0.06
Moona	2764	18 hr	1.91	0.08	0.16	0.1	11.19	18.4	13.88
Moona	2309	18 hr	1.85	0.08	0.16	0.1	17.25	2.8	28.68
Moona	1844	Storm Tide	1.77	0.09	0.43	0.08	12.76	20.15	10.73
Moona	1459	Storm Tide	1.73	0.06	0.56	0.04	13.64	27.91	0.78
Moona	1009	Storm Tide	1.7	0.06	0.44	0.05	4.61	40.06	7.1
Moona	937	Storm Tide	1.69	0.04	0.43	0.04	6.74	42.04	11.18
			Junction with Tributary (Moona 2)						
Moona	874	Storm Tide	1.69	0.04	0.52	0.05	8.23	51.3	13.64
Moona	514	Storm Tide	1.65	0.1	0.8	0.11	12.4	54.38	3.69
Moona	374	Storm Tide	1.64	0.1	0.49	0.05	4.23	65.49	0.07
Moona	336	Storm Tide	1.62	-	0.75	-	-	67.34	-
Moona	324	Storm Tide	1.58	-	0.88	-	-	64.01	-
Moona	271	Storm Tide	1.6	-	0.22	-	-	64	-
Moona	115	Storm Tide	1.59	-	0.41	-	-	63.83	-
Moona	68	Storm Tide	1.58	-	0.49	-	-	63.77	-
Moona	0	Storm Tide	1.57	-	0.59	-	-	63.69	-
Moona	-100	Storm Tide	1.56	-	0.07	-	-	63.52	-
			Jervis Bay						
Moona 2	1640	12 hr	2.46	-	0.22	-	-	19.76	-
Moona 2	1300	12 hr	2.21	-	0.04	-	-	17.53	-
Moona 2	570	12 hr	2.08	0.09	0.46	0.09	1.89	13.54	1.8
Moona 2	560	12 hr	2.08	0.11	0.53	0.11	2.16	15.62	2.06
			Berry Street						
Moona 2	550	12 hr	1.92	0.04	0.33	0.05	0.04	19.66	0.12
Moona 2	150	Storm Tide	1.69	0.04	0.14	0.03	13.39	0.79	0.49
Moona 2	20	Storm Tide	1.69	0	0.13	-	0.01	14.65	-
			Junction with Moona Moona Creek						

Notes:
1. Critical flood levels upstream of RS 2309 result from local catchment flooding due to 10 year ARI storms of the durations shown in combination with Normal Semi-Diurnal Tidal Hydrographs.
2. Critical flood levels downstream of RS 2309 on Moona Moona Creek and in the lower reaches of the Tributary (Moona 2) result from 1 in 10 year Storm Tides in conjunction with catchment flooding from 5 year ARI storms of between 12 to 24 hour durations.
3. “Moona” is the main arm of Moona Moona Creek; “Moona 2” is the tributary running between the STP and Elizabeth Drive.

Reach	River Station	Critical Storm	Peak Water Level (m AHD)	20 yr ARI					
				Velocity (m/s)			Flow (m^3/s)		
				Left	Channel	Right	Left	Channel	Right
Moona	4064	12 hr	2.34	0.13	0.13	0.05	11.56	58.64	0.16
Moona	2764	12 hr	2.08	0.1	0.18	0.12	17.55	23.81	19.82
Moona	2309	Storm Tide	2.05	0.06	0.1	0.05	16.03	2.18	28.04
Moona	1844	Storm Tide	2.02	0.08	0.34	0.07	14.75	17.22	13.41
Moona	1459	Storm Tide	1.99	0.05	0.44	0.03	16.92	24.72	1.57
Moona	1009	Storm Tide	1.98	0.04	0.36	0.05	5.05	37.43	8.78
Moona	937	Storm Tide	1.97	0.03	0.32	0.04	9.72	35.07	14.4
			Junction with Tributary (Moona 2)						
Moona	874	Storm Tide	1.97	0.04	0.39	0.04	11.84	42.74	17.56
Moona	514	Storm Tide	1.95	0.08	0.63	0.07	14.31	49.06	3.67
Moona	374	Storm Tide	1.94	0.08	0.37	0.05	6.12	57.33	0.34
Moona	336	Storm Tide	1.93	-	0.59	-	-	60.53	-
Moona	324	Storm Tide	1.91	-	0.69	-	-	60.49	-
Moona	271	Storm Tide	1.92	-	0.18	-	-	60.49	-
Moona	115	Storm Tide	1.92	-	0.31	-	-	60.29	-
Moona	68	Storm Tide	1.92	-	0.36	-	-	60.21	-
Moona	0	Storm Tide	1.91	-	0.39	-	-	60.12	-
Moona	-100	Storm Tide	1.91	-	0.06	-	-	59.94	-
			Jervis Bay						
Moona 2	1640	12 hr	2.57	-	0.22	0.03	-	24.27	0.01
Moona 2	1300	12 hr	2.33	-	0.04	-	-	22.01	-
Moona 2	570	12 hr	2.17	0.11	0.53	0.11	2.64	16.75	2.52
Moona 2	560	12 hr	2.16	0.13	0.63	0.13	3.06	19.68	2.93
			Berry Street						
Moona 2	550	Storm Tide	2.02	0.02	0.2	0.03	0.06	12.79	0.12
Moona 2	150	Storm Tide	1.97	0.03	0.1	0.02	11.65	0.75	0.57
Moona 2	20	Storm Tide	1.97	0.01	0.09	0	0.23	12.59	0.15
			Junction with Moona Moona Creek						

Notes:
1. Critical flood levels upstream of RS 2764 result from local catchment flooding due to 20 year ARI storms of the durations shown in combination with Normal Semi-Diurnal Tidal Hydrographs.
2. Critical flood levels downstream of RS 2764 on Moona Moona Creek and in the lower reaches of the Tributary (Moona 2) result from 1 in 20 year Storm Tides in conjunction with catchment flooding from a 5 year ARI storm of 12 hours duration.
3. “Moona” is the main arm of Moona Moona Creek; “Moona 2” is the tributary running between the STP and Elizabeth Drive.

Reach	**River Station**	**Critical Storm**	**Peak Water Level (m AHD)**	**50 yr ARI**					
				Velocity (m/s)			**Flow (m^3/s)**		
				Left	**Channel**	**Right**	**Left**	**Channel**	**Right**
Moona	4064	12 hr	2.52	0.14	0.15	0.06	14.45	72.72	0.33
Moona	2764	12 hr	2.28	0.11	0.18	0.12	24.03	27.02	25.78
Moona	2309	12 hr	2.22	0.09	0.14	0.08	28.85	3.36	54.41
Moona	1844	12 hr	2.14	0.13	0.55	0.12	29.2	29.77	27.19
Moona	1459	Storm Tide	2.09	0.05	0.4	0.03	17.62	23.21	1.84
Moona	1009	Storm Tide	2.08	0.03	0.32	0.04	5.63	35.37	9.2
Moona	937	Storm Tide	2.07	0.03	0.29	0.03	10.83	32.53	15.1
			Junction with Tributary (Moona 2)						
Moona	874	Storm Tide	2.07	0.03	0.34	0.04	13.02	39.1	18.15
Moona	514	Storm Tide	2.05	0.07	0.55	0.06	14	44.37	3.86
Moona	374	Storm Tide	2.05	0.07	0.32	0.05	6.32	51.59	0.43
Moona	336	Storm Tide	2.04	-	0.54	-	-	58.06	-
Moona	324	Storm Tide	2.02	-	0.63	-	-	58.06	-
Moona	271	Storm Tide	2.03	-	0.17	-	-	58.02	-
Moona	115	Storm Tide	2.03	-	0.28	-	-	57.8	-
Moona	68	Storm Tide	2.02	-	0.32	-	-	57.75	-
Moona	0	Storm Tide	2.02	-	0.34	-	-	57.61	-
Moona	-100	Storm Tide	2.02	-	0.06	-	-	57.46	-
			Jervis Bay						
Moona 2	1640	12 hr	2.66	-	0.23	0.05	-	29.19	0.27
Moona 2	1300	12 hr	2.47	-	0.05	-	-	27.34	-
Moona 2	570	12 hr	2.27	0.13	0.63	0.13	3.74	20.84	3.58
Moona 2	560	12 hr	2.26	0.14	0.67	0.14	3.95	22.25	3.78
			Berry Street						
Moona 2	550	12 hr	2.16	0.06	0.4	0.07	0.26	28.94	0.44
Moona 2	150	Storm Tide	2.07	0.02	0.08	0.02	10.57	0.69	0.55
Moona 2	20	Storm Tide	2.07	0	0.08	0	0.35	11.16	0.29
			Junction with Moona Moona Creek						

Notes:
1. Critical flood levels upstream of RS 1844 result from local catchment flooding due to 50 year ARI storms of the durations shown in combination with Normal Semi-Diurnal Tidal Hydrographs.
2. Critical flood levels downstream of RS 1844 on Moona Moona Creek and in the lower reaches of the Tributary (Moona 2) result from 1 in 20 year Storm Tides in conjunction with catchment flooding from a 5 year ARI storm of 12 hours duration.
3. "Moona" is the main arm of Moona Moona Creek; "Moona 2" is the tributary running between the STP and Elizabeth Drive.

Reach	River Station	Critical Storm	Peak Water Level (m AHD)	100 yr ARI					
				Velocity (m/s)			Flow (m^3/s)		
				Left	Channel	Right	Left	Channel	Right
Moona	4064	9 hr	2.67	0.16	0.17	0.07	18.8	94.1	0.6
Moona	2764	12 hr	2.44	0.11	0.19	0.13	29.76	29.75	30.81
Moona	2309	12 hr	2.38	0.09	0.13	0.08	33.39	3.46	66.15
Moona	1844	12 hr	2.31	0.13	0.55	0.12	36.32	31.41	34.87
Moona	1459	12 hr	2.25	0.1	0.84	0.07	45.7	51.28	5.61
Moona	1009	18 hr	2.22	0.05	0.56	0.08	13.33	65.23	19.43
Moona	937	18 hr	2.21	0.05	0.46	0.06	22.83	54.79	29.42
			Junction with Tributary (Moona 2)						
Moona	874	18 hr	2.21	0.06	0.55	0.07	27.07	64.98	34.88
Moona	514	18 hr	2.16	0.13	1.03	0.12	29.29	86.03	8.52
Moona	374	18 hr	2.14	0.14	0.63	0.1	14.79	107.78	1.18
Moona	336	Storm Tide	2.11	-	0.51	-	-	56.54	-
Moona	324	Storm Tide	2.09	-	0.59	-	-	56.49	-
Moona	271	Storm Tide	2.1	-	0.16	-	-	56.44	-
Moona	115	Storm Tide	2.1	-	0.26	-	-	56.26	-
Moona	68	Storm Tide	2.09	-	0.3	-	-	56.2	-
Moona	0	Storm Tide	2.09	-	0.32	-	-	56.04	-
Moona	-100	Storm Tide	2.09	-	0.05	-	-	55.9	-
			Jervis Bay						
Moona 2	1640	9 hr	2.77	-	0.27	0.08	-	40.18	1.32
Moona 2	1300	9 hr	2.58	-	0.05	-	-	33.82	-
Moona 2	570	12 hr	2.36	0.14	0.65	0.14	4.54	22.76	4.34
Moona 2	560	12 hr	2.35	0.15	0.71	0.15	4.88	24.72	4.68
			Berry Street						
Moona 2	550	12 hr	2.3	0.06	0.41	0.07	0.47	32.57	0.71
Moona 2	150	18 hr	2.21	0.04	0.13	0.03	17.75	1.19	0.99
Moona 2	20	18 hr	2.21	0.01	0.12	0.01	1.04	17.92	0.93
			Junction with Moona Moona Creek						

Notes:
1. Critical flood levels upstream of RS 374 result from local catchment flooding due to 100 year ARI storms of the durations shown in combination with Normal Semi-Diurnal Tidal Hydrographs.
2. Critical flood levels downstream of RS 374 result from 1 in 100 year Storm Tides in conjunction with catchment flooding from a 5 year ARI storm of 12 hours duration.
3. “Moona” is the main arm of Moona Moona Creek; “Moona 2” is the tributary running between the STP and Elizabeth Drive.

Reach	**River Station**	**Critical Storm**	**Peak Water Level (m AHD)**	**200 yr ARI**					
				Velocity (m/s)			**Flow (m^3/s)**		
				Left	**Channel**	**Right**	**Left**	**Channel**	**Right**
Moona	4064	12 hr	2.8	0.15	0.16	0.07	19.1	95.18	0.78
Moona	2764	12 hr	2.59	0.11	0.18	0.13	34.88	31.17	34.65
Moona	2309	12 hr	2.55	0.08	0.12	0.07	36.26	3.41	74.64
Moona	1844	12 hr	2.49	0.13	0.5	0.11	41.36	30.61	40.95
Moona	1459	12 hr	2.45	0.09	0.77	0.07	53.75	50.71	7.73
Moona	1009	12 hr	2.42	0.06	0.65	0.1	21.59	81.38	28.18
Moona	937	12 hr	2.4	0.06	0.52	0.07	35.6	66.68	42.39
			Junction with Tributary (Moona 2)						
Moona	874	12 hr	2.4	0.07	0.62	0.08	42.28	79.19	50.34
Moona	514	12 hr	2.33	0.16	1.29	0.17	43.9	114.42	13.54
Moona	374	12 hr	2.31	0.19	0.8	0.13	23.91	145.58	2.38
Moona	336	12 hr	2.26	-	1.48	-	-	171.86	-
Moona	324	Storm Tide	2.19	-	0.5	-	-	50.09	-
Moona	271	Storm Tide	2.2	-	0.14	-	-	50.03	-
Moona	115	Storm Tide	2.19	-	0.22	-	-	49.75	-
Moona	68	Storm Tide	2.19	-	0.25	-	-	49.67	-
Moona	0	Storm Tide	2.19	-	0.26	-	-	49.53	-
Moona	-100	Storm Tide	2.19	-	0.05	-	-	49.38	-
			Jervis Bay						
Moona 2	1640	12 hr	2.83	-	0.23	0.08	-	36.95	1.89
Moona 2	1300	12 hr	2.67	-	0.05	-	-	36.15	-
Moona 2	570	12 hr	2.48	0.14	0.62	0.14	5.23	23.2	5.01
Moona 2	560	12 hr	2.48	0.14	0.64	0.14	5.32	23.75	5.1
			Berry Street						
Moona 2	550	12 hr	2.46	0.06	0.33	0.06	0.66	29.67	0.89
Moona 2	150	12 hr	2.4	0.04	0.16	0.03	24.02	1.66	1.45
Moona 2	20	12 hr	2.4	0.01	0.14	0.01	2.53	22.53	2.08
			Junction with Moona Moona Creek						

Notes:
1. Critical flood levels upstream of RS 336 result from catchment flooding due to 100 year ARI storms of 12 hours duration in combination with Normal Semi-Diurnal Tidal Hydrographs.
2. Critical flood levels downstream of RS 336 result from 1 in 200 year Storm Tides in conjunction with catchment flooding from a 5 year ARI storm of 12 hours duration.
3. “Moona” is the main arm of Moona Moona Creek; “Moona 2” is the tributary running between the STP and Elizabeth Drive.

Reach	River Station	Critical Storm	Peak Water Level (m AHD)	PMF					
				Velocity (m/s)			Flow (m^3/s)		
				Left	Channel	Right	Left	Channel	Right
Moona	4064	PMF	4.09	0.24	0.27	0.14	57.45	275.1	8.05
Moona	2764	PMF	3.85	0.18	0.25	0.19	136.74	68.74	105.65
Moona	2309	PMF	3.8	0.12	0.15	0.11	101.45	6.47	237.52
Moona	1844	PMF	3.75	0.16	0.59	0.15	137.16	50.72	157.01
Moona	1459	PMF	3.71	0.13	1.02	0.12	193.58	89.31	61.93
Moona	1009	PMF	3.68	0.12	0.83	0.14	160.79	151.2	89.82
Moona	937	PMF	3.66	0.12	0.74	0.12	154.89	131.98	152.17
			Junction with Tributary (Moona 2)						
Moona	874	PMF	3.66	0.16	1	0.16	210.04	178.98	206.36
Moona	514	PMF	3.55	0.2	1.5	0.25	136.48	188.73	42.09
Moona	374	PMF	3.56	0.22	0.82	0.18	87.52	224.99	17.22
Moona	336	PMF	3.38	0.22	2.14	0.29	11.3	390.44	16.87
Moona	324	PMF	2.13	-	5.98	-	-	583.97	-
Moona	271	PMF	2.89	-	1.28	-	-	595.15	-
Moona	115	PMF	2.69	-	2.12	-	-	595.11	-
Moona	68	PMF	2.58	-	2.45	-	-	595.08	-
Moona	0	PMF	2.17	-	2.72	-	-	511.98	-
Moona	-100	PMF	2.09	-	0.57	-	-	585.65	-
			Jervis Bay						
Moona 2	1640	PMF	3.92	0.04	0.12	0.07	0.82	49.86	23.14
Moona 2	1300	PMF	3.9	-	0.05	-	-	72.99	-
Moona 2	570	PMF	3.84	0.13	0.56	0.13	19.26	35.08	18.42
Moona 2	560	PMF	3.78	0.36	1.51	0.36	49.75	92.83	47.58
			Berry Street						
Moona 2	550	PMF	3.79	0.2	0.94	0.21	17.17	155.34	18.02
Moona 2	150	PMF	3.66	0.14	0.52	0.12	136.05	10.3	10.21
Moona 2	20	PMF	3.66	0.04	0.31	0.05	51.57	78.74	26.03
			Junction with Moona Moona Creek						

Note:
1. Critical flood levels result from 1 in 100 year Storm tides in conjunction with catchment flooding from PMP storm of 6 hours duration.
2. “Moona” is the main arm of Moona Moona Creek; “Moona 2” is the tributary running between the STP and Elizabeth Drive.