CURRAMBENE CREEK
AND
MOONA MOONA CREEK

FLOOD STUDIES

VOLUME 1 –REPORT

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FOREWORD

The primary objective of the NSW Government's Flood Prone Land Policy is to reduce the impact of flooding and flood liability on individual owners and occupiers of flood prone property, and to reduce private and public losses resulting from floods. At the same time, the policy recognises the benefits flowing from the use, occupation and development of flood prone land.

The policy promotes the use of a merit approach which balances social, economic, environmental and flood risk parameters to determine whether particular development or use of the floodplain is appropriate and sustainable.

In this way the policy avoids the unnecessary sterilisation of flood prone land. Equally it ensures that flood prone land is not the subject of uncontrolled development inconsistent with its exposure to flooding.

The policy highlights that primary responsibility of floodplain risk management rests with Councils which are provided with financial and technical support by the State Government. The Commonwealth has also historically shown a willingness to be involved by providing financial assistance to local government in partnership with the State Government.

The Currambene Creek and Moona Moona Creek Flood Studies constitute the first two stages of the process for this area and have been prepared for Shoalhaven City Council with financial assistance from the Department of Natural Resources, to define flood behaviour under current conditions.

NOTE ON FLOOD FREQUENCY

The frequency of floods may be referred to in terms of their Average Recurrence Interval (ARI) or Annual Exceedance Probability (AEP). For example, for a flood having a 5 year ARI there will be a flood of equal or greater magnitude once in 5 years on the average. For a flood having a 5% AEP magnitude, there is a 5% probability that there will be floods of equal or greater magnitude each year. The approximate correspondence between these two systems is:

ANNUAL EXCEEDANCE PROBABILITY (AEP) %	AVERAGE RECURRENCE INTERVAL (ARI) YEARS
0.5	200
1	100
5	20
20	5

In this report floods are referred to in terms of their Average Recurrence Interval. Reference is also made in the report to the Probable Maximum Flood (PMF). This flood occurs as a result of the Probable Maximum Precipitation (PMP). The PMP is the result of the optimum combination of the available moisture in the atmosphere and the efficiency of the storm mechanism as regards rainfall production. The PMP is used to estimate PMF discharges using a model which simulates the conversion of rainfall to runoff. The PMF is defined as the limiting value of floods that could reasonably be expected to occur.

ABBREVIATIONS

AEP	Annual Exceedance Probability (%)
AHD	Australian Height Datum
ARI	Average Recurrence Interval (years)
ARR	Australian Rainfall and Runoff
BOM	Bureau of Meteorology
DNR	Department of Natural Resources (formerly Department of Infrastructure Planning and Natural Resources (formerly Department of Land and Water Conservation)

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1. INTRODUCTION

1.1 Study Background

A comprehensive *Floodplain Risk Management Plan (FRMP)* is to be prepared for the Currambene and Moona Moona Creek catchments as part of a Government program to manage the flood related risks of human occupation of the floodplains. Two important steps in the process of preparing a *FRMP* **(Figure 1.1)** are the undertaking of data collection and flood studies for the two streams and their main tributaries. Data collection and the flood studies are the formal starting points of defining management measures for flood prone land and represent a detailed technical investigation of flood behaviour.

Using flood data collected for the purpose of this study, plus detailed field surveys of the estuarine areas and tributaries of the two creeks, mathematical models were developed and interpreted to present a comprehensive picture of flooding on the two creeks under present day conditions.

The study objective was to define flood behaviour in the creeks and their main tributaries in terms of flows, levels and flooding behaviour for floods ranging between 5 and 200 years Average Recurrence Interval (ARI), as well as for the Probable Maximum Flood (PMF).

Flood behaviour was defined using computer based hydrologic models of the catchments and hydraulic models of the two main streams and their tributaries. The hydrologic models were based on a runoff routing approach and in the case of Currambene Creek, calibrated against recorded rainfall and stream flow data. Parameters derived from calibrating the Currambene Creek model gave guidance for the parameters selected for design flood estimation on that catchment and also on Moona Moona Creek.

Design storms were applied to the models to generate discharge hydrographs within the study area. These hydrographs constituted the upstream boundaries and tributary inflow inputs to the hydraulic models.

A dynamic hydraulic modelling approach was adopted for the analysis to account for the time varying effects of flows in the streams, the routing effects of the floodplain storage and the potential impact of entrance and storm tidal conditions on flooding in the estuarine areas. A one-dimensional link-node modelling approach was chosen which allowed for the interaction of flows between the channels and the floodplains, flow through culverts and flow over control structures such as road embankments and the channel outlets to Jervis Bay.

After testing, the models were used to define the extents of flooding and produce water surface profiles, the distribution of flow across the floodplain and flow velocities for the design events.

In accordance with current engineering practice and documentation supplied by *Department of Natural Resources (DNR)*, a *"Flood Envelope"* approach was adopted for defining design water surface profiles. The procedure involved running the model for two alternative scenarios to define the upper limit of expected flooding for each design flood frequency. The two scenarios were: catchment runoff derived from the design storm events, in conjunction with a normal semi-diurnal tidal hydrograph at the downstream boundary; or storm tide hydrographs of each design frequency in conjunction with catchment runoff from a minor storm event.

It was found that elevated ocean levels due to storm tides and wave action controlled design flood levels in the lower reaches of the two creeks. Catchment runoff controlled flooding in the middle to upper reaches of the main streams and their tributaries.

1.2 Study Tasks

The flood study had three components:

- ***Review of available hydrologic and hydraulic data and previous investigations.*** Rainfall, and stream flow data for Currambene Creek were supplied from various sources including *Bureau of Meteorology (BOM), DNR* and a previous investigation of flooding on Currambene Creek *(Brian Lyall and Associates, 1983).* The hydrologic data were collated for three historic floods which occurred in the 1970's and used in the testing of the hydrologic and hydraulic models for that stream.

  A brief was prepared for cross sectional survey of the streams in the study catchments. Gallagher Odell & Garey Consulting Surveyors & Engineers undertook the survey. A considerable length of the study reach on Currambene Creek, which extended from a location about 1 km downstream of the Princes Highway Bridge below The Falls to the outlet to Jervis Bay at Huskisson had been surveyed for the 1983 study. This information was also used for the present investigation.

  Central Mapping Authority supplied a Digital Terrain Model (DTM) of the study area, which had limited contour information within the floodplain areas. This information was used to define the sub-catchments for the development of the catchment model and also assisted in the estimation of the flood extents and presentation of study results.

- ***A hydrologic component*** which included flood frequency analysis of streamflow records on Currambene Creek, preparation of hydrologic models of the two catchments, calibration of the Currambene Creek model for the three historic floods and adoption of model parameters for design flood estimation, derivation of design storms and their application to the models to define design discharge hydrographs.

- ***A hydraulic component*** which comprised the preparation and testing of hydraulic models of the main streams and floodplain areas on Currambene Creek and Moona Moona Creek and the application of discharge hydrographs to the models to define water surface profiles, flows and velocities for the design floods.

1.3 Overview of Report

This Report (**Volume 1**) summarises the investigations and presents plans showing water surface profiles and the indicative extents of flooding. The Report is supported by three **Appendices,** which provide additional details on flooding patterns and are bound in **Volume 2**.

Section 2 of the Report contains background information including a description of the catchments, a review of the data base available for the study and a discussion on the history of flooding in the two catchments. This led to the selection of the historic floods for calibration and testing of the RORB hydrologic model of Currambene Creek.

Streamflows recorded at The Falls gauging station, together with pluviographic data recorded at the RAN Air Station at HMAS Albatross and daily rainfall data were used for this purpose. **Appendix A** provides further details of the calibration process.

Section 3 deals with the derivation of design runoff hydrographs from the study catchments. The RORB runoff-routing program was adopted for this study. Models of the catchments of Currambene and Moona Moona Creeks were developed.

This step involved the determination of design storm rainfall depths over the catchments for a range of storm durations, and conversion of the rainfall hyetographs to discharge hydrographs. Further details of this phase of the investigation are presented in **Appendix B**.

Section 4 deals with the development of the hydraulic models of the main streams and their estuarine surrounds. The unsteady flow version of the HEC-RAS software was used for this purpose. There were no historic flood level data available for calibration of the hydraulic models. Accordingly, selection of model parameters required a detailed review of the engineering literature and previous studies of a similar nature undertaken by the Study Team, supplemented by sensitivity analysis. The results of model testing are presented in the first part of **Appendix C**.

Section 5 details the results of the hydraulic modelling of the design floods using HEC-RAS. Results are presented as water surface profiles and plans showing indicative extents of inundation. **Appendix C** presents further details on the derivation of design flood information and contains diagrams showing zones of provisional high and low hazard on the floodplain for selected design floods events, categorisation of the floodplain into floodway and flood storage areas and tabulations of peak levels and flow velocities at each cross section in the hydraulic models.

Section 6 summarises the flood study investigations and identifies several flood related issues on the tributaries of Currambene Creek which will require further study during the Floodplain Risk Management Study and Plan.

Section 7 contains a list of References.

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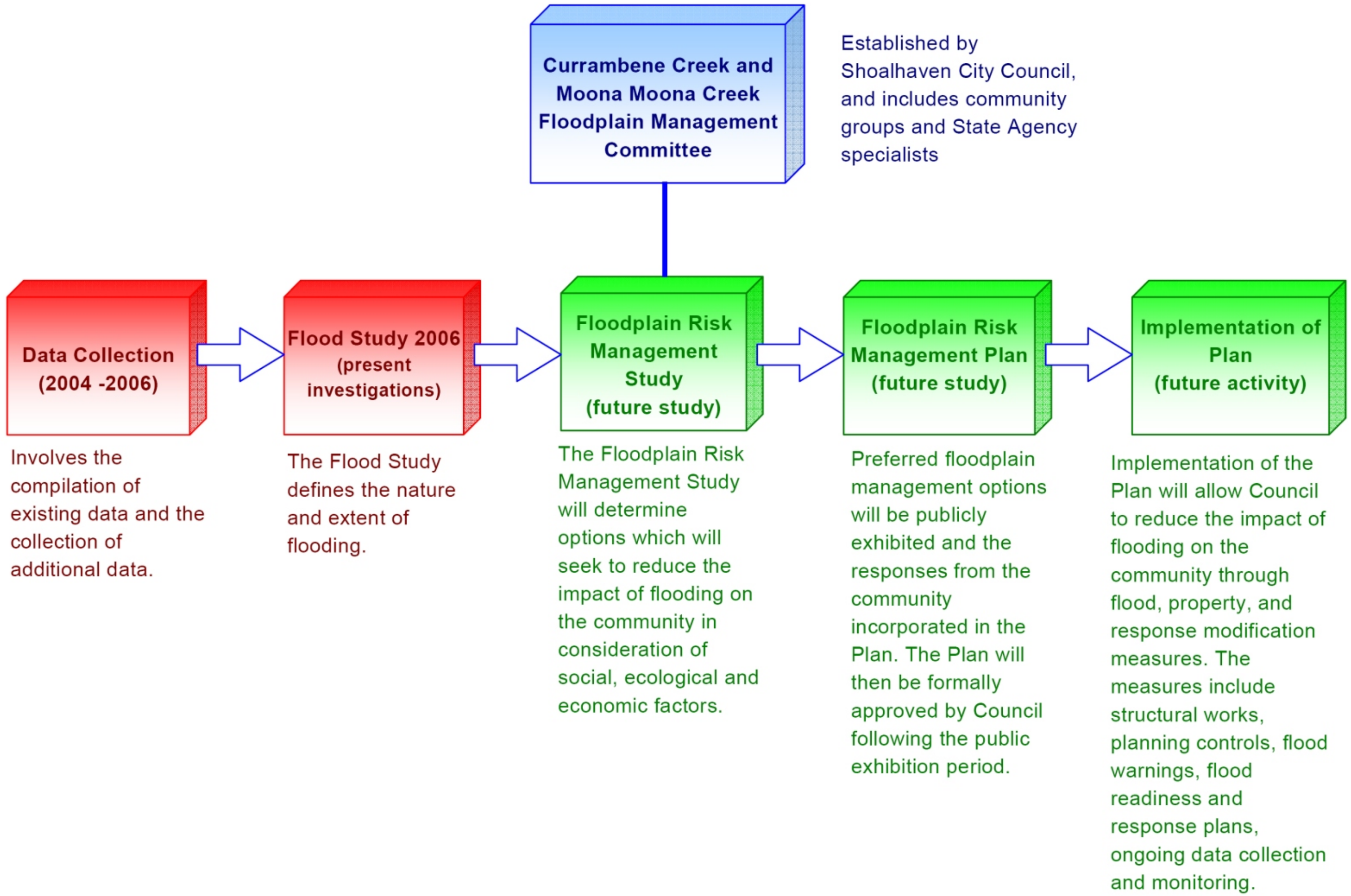

CURRAMBENE CREEK AND MOONA MOONA CREEK FLOOD STUDY

Figure 1.1

FLOODPLAIN RISK MANAGEMENT PROCESS

2. BACKGROUND

2.1 Catchment Description

2.1.1 Currambene Creek

Currambene Creek is one of a series of short streams which drain the coastal strip of the South Coast of NSW. These streams rise in heavily forested, mountainous country along the escarpment where land slopes are generally greater than eight degrees. There is no continuous coastal plain, but small alluvial flats have developed along the lower parts of many of the streams, which become more extensive near their mouths. The lower sections of these streams are tidal and as the bed slope flattens, extensive areas of coastal swamps are formed.

The centroid of the Currambene Creek catchment is located about 12 km to the south of Nowra. The Turpentine Range, which runs along the northern and western boundaries separates the catchment from that of several streams which drain northwards to the Shoalhaven River and several small creeks which drain eastwards to St Georges Basin.

Currambene Creek rises in the plateau area occupied by the RAN Air Station, HMAS Albatross at an elevation of 100 m and falls through 90 m along a stream length of 7 km to the Princes Highway crossing **(Figure 2.1)**. Just upstream of the highway, Currambene Creek is joined by Parma Creek which rises to the south-west at an elevation of 300 m and has a stream length of 20 km. The combined catchment area at the Princes Highway is 95 km^2 of which Parma Creek, the major arm, contributes 75 km^2.

At a location known as The Falls, immediately upstream of the Princes Highway, the stream bed drops about 8 m. The stream is tidal from this point to its outlet to Jervis Bay at Huskisson. The length of this reach is about 16 km. The tidal section of Currambene Creek is fed by a sub-catchment of 64 km^2, giving a total catchment of 160 km^2 at the outlet.

On the northern bank, the main sub-catchment is Georges Creek which drains the Currambene State Forest and enters Currambene Creek via an extensive swampy area about 7 km downstream of the highway opposite Goodland Road, ref. **Figure 5.5** which shows tributary stream, local roads and indicative extents of inundation derived from the investigation.

Several un-named streams drain the southern part of the catchment and cross Woollamia Road before joining the right bank of Currambene Creek. The most important of these streams drains the Tomerong State Forest area and enters the main stream opposite the Georges Creek junction. The three main streams are denoted Tributaries 1 to 3 on **Figure 5.5**.

For the first 4 km below The Falls, the Currambene Creek waterway comprises a tree lined main channel of uniform width of around 40 m rising relatively steeply to cleared grazing land. Below this point the tidal channel gradually opens out to an estuarine area, with a typical width below mean sea level of 100 to 200 m. During major floods, the extent of inundation could reach in excess of 1 km. A large volume of storage is contained in the swampy areas above normal high tide level which attenuates the flood peaks resulting from major storm events.

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

In its southward passage to Huskisson, the creek passes the townships of Woollamia 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 successful stabilisation (re-grassing) programme has been undertaken. However, should the creek break through the beach to form a new outlet, 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.

2.1.2 Moona Moona Creek

Moona Moona Creek has a small catchment, which drains the area to the south of Currambene Creek and outfalls to Jervis Bay at the northern end of Collingwood Beach. The total catchment area at the outfall is 28 km^2. The catchment is undeveloped apart from the urbanised strip of Vincentia running along the dune adjacent to Elizabeth Drive and the portion to the south of Vincentia Road. The southern and western parts of Huskisson also drain to Moona Moona Creek.

Moona Moona Creek and its main tributary Duck Creek drain the foothills comprising the western portion of the catchment and cross Jervis Bay Road, before entering a low lying heavily overgrown, swampy area which occupies the middle reaches of the catchment upstream of the bridge at Elizabeth Drive, ref. **Figure 5.8** which shows local roads, tributary streams and indicative extents of inundation. 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. The flood gradient in this area would be very low as it mainly functions as a basin for the temporary storage of runoff.

A minor tributary conveys runoff from the southern portion of the catchment extending to Vincentia. It runs to the east of the Sewage Treatment Plant before joining the southern bank of Moona Moona Creek about 600 m upstream of Elizabeth Drive bridge, 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 immediately south of an unnamed point at the northern end of Collingwood Beach. The width of the lagoon averages 100 – 120 m and has a sandy bed, which is likely to show considerable variation in level over time, although local opinion suggests the presence of a rock shelf beneath the sand which would limit the depth of erosion during flood periods. The highest invert elevation within the lagoon at the time of the creek survey was RL 0 m. The lagoon outlet was about 40 – 60 m wide at the time of the survey carried out for this investigation in mid-2004 and had an invert of RL –2 m AHD.

2.2 Community Consultation

At the commencement of the Flood Study, a Community Newsletter and questionnaire was delivered to residents in the study catchments to firstly introduce the study and to secondly invite the community to provide any historic flood information in their possession.

Respondents to the questionnaire noted that their properties had not experienced recent flooding from the creeks. Several instances of historic flooding in the 1970's were identified.

Follow up discussions with local residents within the catchments provided additional anecdotal evidence on the extent of flooding in the February 1971 event. However, no information was uncovered of assistance in the calibration of the computer-based models set up for this study.

2.3 Data Base

2.3.1 Rainfall Data

A pluviographic station has been in operation at HMAS Albatross since the late 1960's. This station is located in the upstream portion of the catchment west of the Princes Highway **(Figure 2.1)**. Several daily read rainfall stations are located in the proximity of the catchment. Data from these stations were obtained from the BOM.

2.3.2 Streamflow Data

A stream gauging station has operated on Currambene Creek at The Falls since 1969. The site is located on the western side of the Princes Highway, where the catchment area amounts to 95 km^2. DNR supplied peak flow data, which were used to carry out flood frequency analysis and also supplied discharge hydrographs for several major historic flood events.

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 2.1**. 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 for those floods.

TABLE 2.1
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).

Based on an annual series analysis of flood peaks at The Falls, February 1971 flood had a recurrence interval between 50 and 100 years ARI. The March 1975 and October 1978 floods were in the range 5 and 20 years ARI.

No stream gauging stations are located on Moona Moona Creek.

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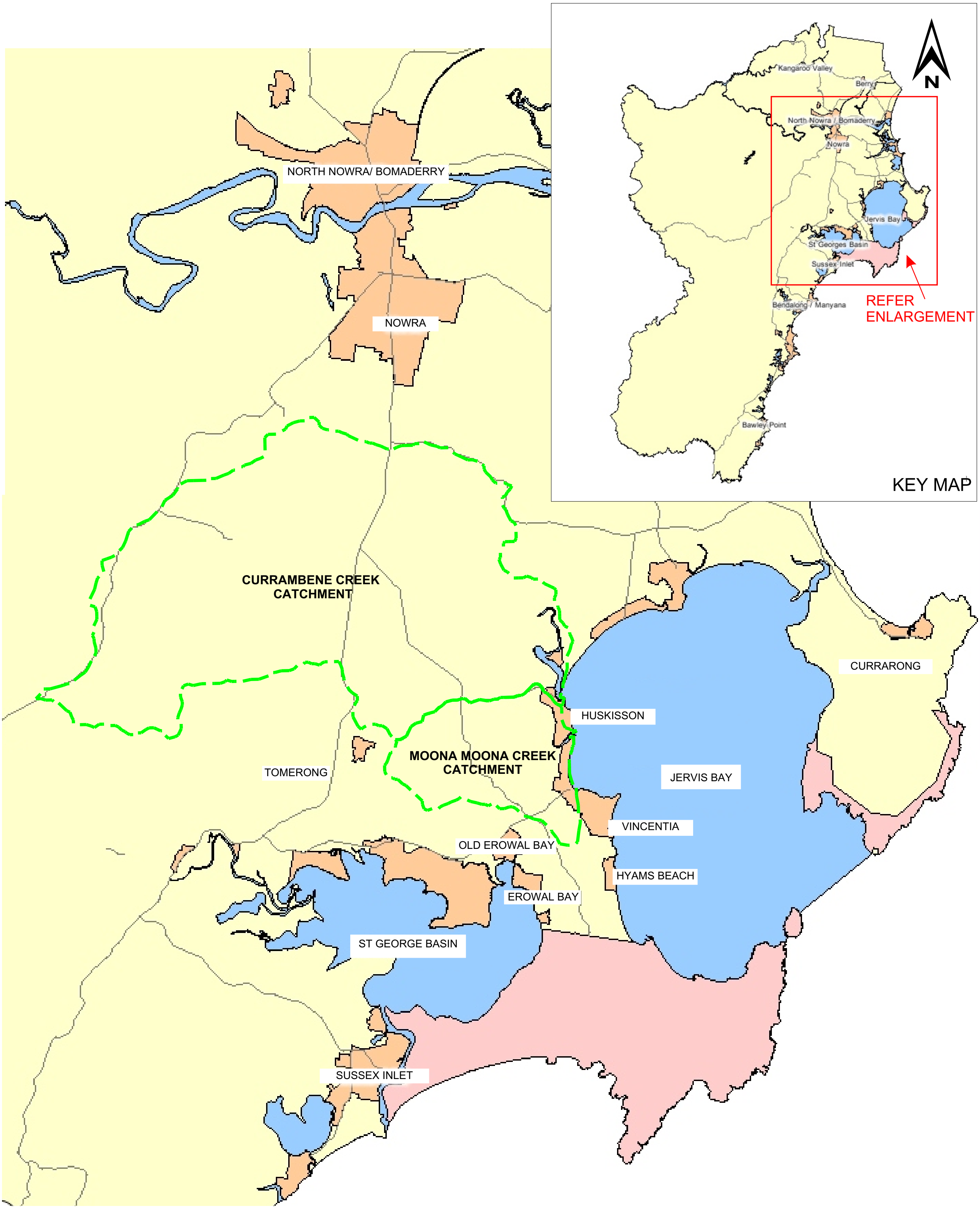

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CURRAMBENE CREEK AND MOONA MOONA CREEK FLOOD STUDIES

Figure 2.1

LOCATION PLAN

3. HYDROLOGY

3.1 Review of RORB Modelling Approach

The RORB software converts storm rainfall to discharge hydrographs using a procedure known as runoff-routing and envisages the catchment to be comprised of a series of concentrated storages which represent sub-catchments defined on watershed lines, plus concentrated special storages which could simulate flood storage areas.

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

$$S = kQ^m \qquad (3.1)$$

where S = volume of storage (m^3).

Q = Discharge (m^3/s)

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, i.e. discharge increases at a proportional rate to rainfall intensity.

The storage parameter "k" in the storage equation 3.1 is modified to reflect the catchment storage and the reach storage as follows:

$$k = kc.kr \qquad (3.2)$$

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.

An advantage of using RORB is that the software has been used extensively throughout Australia on a wide range of rural and urban catchments. Relationships between the catchment storage parameter kc and catchment areas for a large number of regions have been developed and may be used as a check on results achieved for calibrated catchments, or alternatively used to estimate flows on ungauged catchments.

3.2 RORB Model Calibration and Testing

Although it was possible in **Appendix A** to achieve a good correspondence between recorded and modelled discharge hydrographs for *individual* floods, it was not possible to achieve a consistent set of kc and m parameters for all three events. This may have been partly due to errors in defining the average temporal pattern of rainfall over the catchment area. Even though there is a pluviograph located within the Currambene Creek catchment, it is on the northern fringe and therefore may not have recorded the average temporal patterns of rainfall for all of the storms.

A second analysis was undertaken in which design storms of various frequencies were applied to the RORB model to reproduce the recorded flood frequency relationship. The results of this curve fitting analysis gave a small range of kc values (between 10.2 and 10.8) for recurrence intervals ranging between 10 and 100 years ARI and a larger value of 12.6 for the 5 year ARI. These results were achieved with a value of 0.8 for the exponent m of the storage – discharge equation 3.1, a continuing loss of 2.5 mm/h and initial losses recommended by Walsh et al, 1991.

The RORB model parameters shown on **Table 3.1** were adopted for the purposes of deriving inflow hydrographs to the hydraulic model of Currambene Creek at its upstream boundary near The Falls.

Lateral inflow hydrographs for the sub-catchments downstream of The Falls were derived from a "Big" RORB model of the catchment, which extended as far as the outlet at Huskisson. The kc value for the "Big" model of the catchment (160 km^2) were derived using the relationship suggested in the RORB manual, whereby kc is proportional to the square root of the catchment area (equation 3.3). The results of the model calibration at The Falls gave a proportionality constant C in the range 1.05 to 1.1 for medium and major flood events and by application of this constant to equation 3.3, a kc value of 13.9 for the "Big" RORB model.

$$kc = CA^{0.5} \quad \ldots\ldots\ldots\ldots\ldots\ldots \; (3.3)$$

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

This equation was also used to estimate kc for the Moona Moona Creek RORB model.

TABLE 3.1
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 ("Big" Model)	160	40 – 60	2.5	13.9	0.8
Moona Moona Creek at Jervis Bay	28	40	2.5	5.4	0.8

3.3 Derivation of Design Storms

3.3.1 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 Book 2 of ARR (Reprinted 2001). Design storms for frequencies between 5 and 200 year ARI were derived for storm durations ranging between 3 hours and 24 hours.

3.3.2 Areal Reduction Factors

The rainfalls derived using the processes outlined in ARR are applicable strictly to a point. An areal reduction factor (ARF) is typically applied to obtain an intensity that is applicable over the entire catchment area.

For this present study, ARR indicates that a value of 0.95 could be justified for the ARF on the Currambene Creek catchment as an appropriate value for the 12 to 24 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, which are reviewed in **Appendix B**.

As the catchment area of Moona Moona Creek is smaller than for Currambene Creek, negligible reduction in rainfall intensity would result, thus the point values were adopted.

3.3.3 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 is extrapolated to this ARI.

3.4 Design Hydrographs

The RORB models for the two catchments were run with the above parameters (**Table 3.1**) to obtain design hydrographs for input to the hydraulic model.

<u>Currambene Creek</u>

At most locations on Currambene Creek, 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.

The RORB model contained a lumped storage versus discharge relationship downstream of the confluence with Georges Creek, which simulated the floodplain storage and resulted in a considerable reduction in the peak flow between its upstream and downstream locations. However, in reality, the storage is distributed along the entire reach of Currambene Creek. Its impacts on flows are more accurately assessed in the hydraulic model.

In the hydraulic modelling described in **Chapter 4**, storm durations up to 24 hours in length were found to be critical in terms of generating peak flood levels on Currambene Creek. This effect is probably due to the more accurate modelling of flood storage inherent in a dynamic hydraulic model, compared with the lumped approach in RORB.

Moona Moona Creek

Below the confluence with Duck Creek, downstream of Jervis Bay Road, the floodplain of Moona Moona Creek comprises a large overgrown storage area. The storage characteristics of this area were not explicitly modelled by RORB, but are incorporated in the cross sections of the floodplain comprising the hydraulic model.

3.5 Probable Maximum Flood

Estimates of Probable Maximum Precipitation for the catchments were made using the Generalised Short Duration Method (GSDM) as described in the Bureau of Meteorology's publication (BOM, 2003). This method is appropriate for estimating extreme rainfall depths for catchments up to 1000 km^2 in area and storm durations up to 6 hours.

In the RORB analysis for Currambene Creek, there was the trend for discharges to increase for longer duration PMP 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 derived. However, to explore this effect further would have required estimation of longer duration PMP's by BOM and was not justified. The discharge hydrographs derived in this investigation are considered to allow a reasonable estimate of extreme flood levels as determined in **Chapter 5.**

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 for a previous flood study 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 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.

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. The results, which are described in **Appendix B**, gave PMF peaks which were slightly smaller than those derived from the non-linear modelling approach.

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

4. HYDRAULICS

4.1 The HEC-RAS Modelling Approach

The HEC-RAS modelling software routes flows through the main streams and their tributaries, and produces time series of flows, velocities and water surface elevations at nominated locations. The model developed for the present study is capable of adjustment in a future Floodplain Risk Management Study so as to 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.

The complete non-linear Saint-Venant equations of open channel flow are solved numerically between user defined grid arrangements (more typically, cross section locations) at specified time intervals for given boundary conditions such as inflow 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 quasi-steady state.

4.2 Model Layouts

The models consisted of cross sections derived from ground survey. The choice of section locations depended on the need to accurately represent features on the floodplains 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.

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

Schematic layouts of the Currambene Creek and Moona Moona Creek HEC-RAS models are shown on **Figures 4.1 and 4.2**. These figures also show the locations of tributary inflows to the models.

4.3 Flood Producing Mechanisms

Flooding in the lower portion of the two streams 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 semi diurnal 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 of several days. This elevation is superimposed upon the normal tidal variation.

A rigorous joint probability analysis of the two flood producing mechanisms is not practicable, as there are negligible level data available in the lower reaches of the creeks. Therefore, although stream flow data on Currambene Creek (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 hydraulic modelling procedure was adopted for design flood estimation. This approach is supported by the *"Floodplain Management Guideline No. 5 Ocean Boundary Conditions*" supplied by DIPNR.

Scenario *Tide – Catchment Flood Combination*

1. Normal Semi-Diurnal Tide Hydrograph + Catchment Flood of Relevant Frequency

2. Storm Tide Hydrograph of Relevant Frequency + Minor (5 year ARI) Catchment Flood

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 DNR and entitled *"Estimates of Tail Water Levels in Currambene Creek and Moona Moona Creek".*

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 catchment runoff 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 upstream discharge, is well established in situations where there are insufficient data to undertake a more rigorous joint probability analysis of the two flood producing mechanisms.

4.4 Model Parameters

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 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 by up to 20 per cent in the channel, where it was considered that roughness values could be estimated with greater accuracy.

The peak flood levels on Currambene Creek were 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. At the upstream end of the creek at RS 15206, the maximum modelled increase in peak flood levels over the range of the sensitivity studies was 0.48 m for the 100 year ARI flood. Further downstream at RS 7515, which is located near Goodland Road, the maximum increase in peak flood level amounted to 0.34 m.

In the case where only the roughness of the floodplain was increased, the corresponding increase in flood levels were quite small, because there was a re-direction of flows from the floodplain into the channel, which became relatively more hydraulically efficient.

The increases in flood levels in the upstream to middle reaches of the creek that are associated with increasing hydraulic roughness simulate a retarding basin and result in a very small reduction in peak floods levels compared with the "best estimate" roughness values near the downstream end of the model at RS 2288 opposite Myola.

The sensitivity studies presented in **Appendix C** cover the range of roughness 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. 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 **Chapter 5**.

Tables 4.1 and **4.2** summarise the "best estimate" values of hydraulic roughness adopted for the investigation.

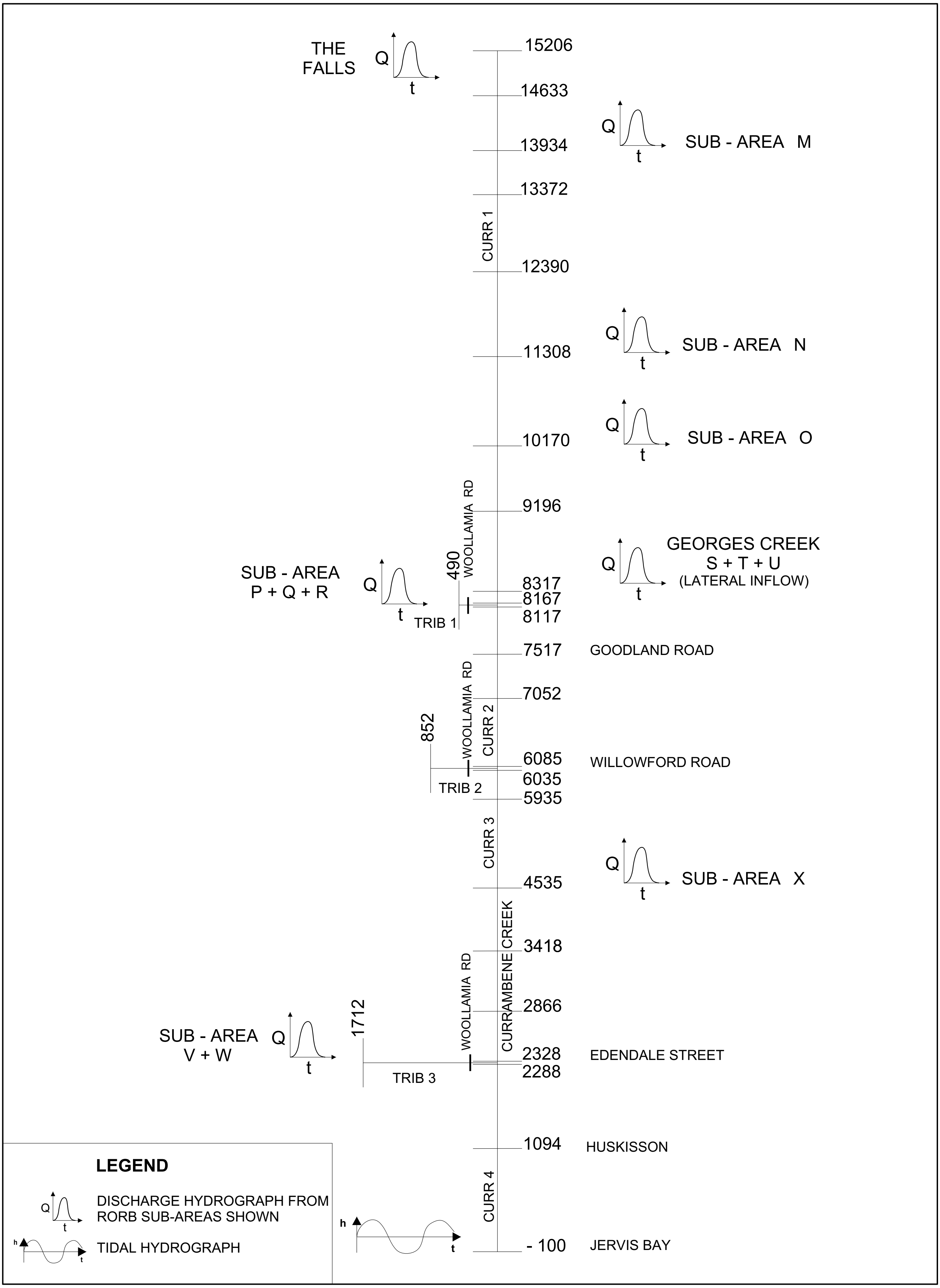
TABLE 4.1
"BEST ESTIMATE" OF HYDRAULIC ROUGHNESS VALUES
CURRAMBENE CREEK

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

On Moona Moona Creek, for all cases modelled, the flood gradient upstream of Elizabeth Drive was very low, confirming that most of the waterway cross section in this area functions as a flood storage with very low flow velocities. Flood levels were not sensitive to variations in hydraulic roughness.

TABLE 4.2
BEST ESTIMATE OF HYDRAULIC ROUGHNESS
MOONA MOONA CREEK

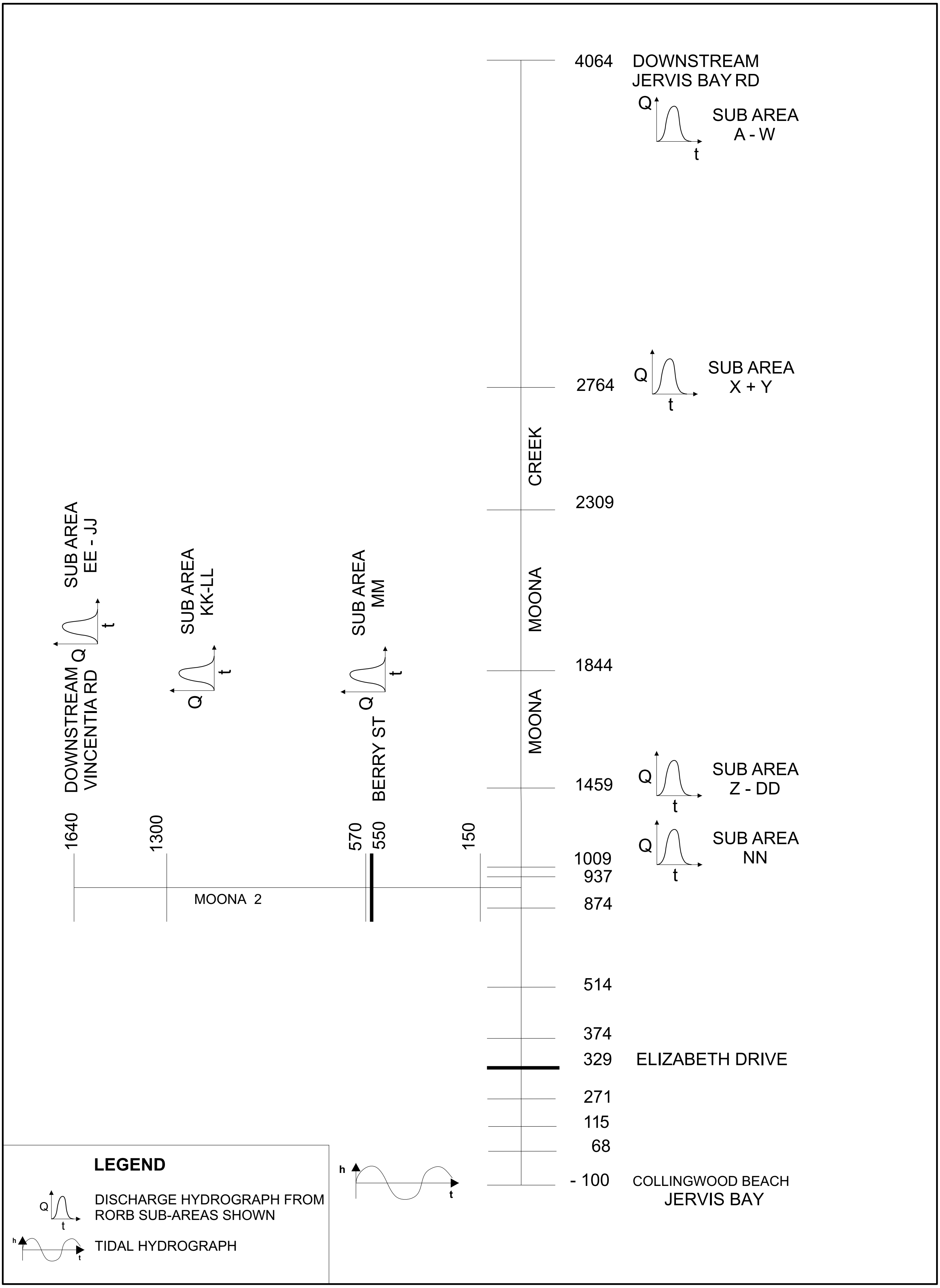
	Channel	**Floodplain**
d/s Elizabeth Drive Bridge	0.025	NA
u/s Bridge	0.03 – 0.035	0.12



CURRAMBENE CREEK AND MOONA MOONA CREEK FLOOD STUDIES

Figure 4.1

CURRAMBENE CREEK
HEC - RAS SCHEMATIC LAYOUT



CURRAMBENE CREEK AND MOONA MOONA CREEK FLOOD STUDIES

Figure 4.2

MOONA MOONA CREEK
HEC - RAS SCHEMATIC LAYOUT

5. HYDRAULIC MODELLING OF DESIGN FLOODS

5.1 Introduction

This chapter summarises the results of modelling the design flood events of 5 to 200 year ARI and the Probable Maximum Flood Event. The procedure is described in more detail in **Appendix C** and involved running storms for each frequency ranging between 9 hours and 24 hours durations, both with *"Normal Tides"* 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.

5.2 Currambene Creek

5.2.1 Indicative Extents of Inundation

Figures 5.1 to **5.4** show water surface profiles on Currambene Creek and Tributaries 1 to 3 for the full range of flood events from 5 year ARI to PMF. **Figure 5.5** is a plan of the floodplain showing the extents of inundation and flood levels for the 10 and 100 year ARI floods and the PMF.

The extent of inundation of each flood event is necessarily indicative only. It is 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 Goodland Road, Willowford Road, Streamside Street and Edendale Street. A line of levels was also surveyed along Myola Road on the northern 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.

5.2.2 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 within the confines 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 0.7 m/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 (ref **Appendix A**), enters Currambene Creek from the southern side just upstream of Goodland Road. The offstream storage in this tributary is modelled by Tributary 1 in the hydraulic model. On the northern side, flows from another major tributary, Georges Creek, join Currambene Creek. Georges Creek has not been hydraulically modelled in this study. Whilst the storage characteristics of the floodplain at its confluence with Currambene Creek have been modelled, there is presently no detailed survey available to define the extent of flooding on this stream upstream of its confluence with Currambene Creek.

Together the two streams, 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 offstream storage areas on both sides of the creek largely offsets any resulting increase in downstream 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 minor flood events **(Table 5.1)**.

At Goodland Road (RS 7517), the floodplain of Currambene Creek 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.

TABLE 5.1
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

The horizontal water surface profiles presented in **Figures 5.2** to **5.4** show that peak flood levels in the tributaries are caused by backwater flooding from Currambene Creek. Peak flows from the tributaries 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 is 400 mm below its subsequent peak. On Tributary 3, the corresponding peak discharge is 58 m^3/s and occurs when the water level in Currambene Creek is 250 mm below its subsequent peak. Although the tributaries contribute flood storage and attenuate the floodwave on Currambene Creek as backwaters, they also function as floodways for the conveyance of runoff from their respective sub-catchments.

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 a peak of 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.

5.3 Moona Moona Creek

5.3.1 Extents of Inundation

Figures 5.6 and **5.7** shows water surface profiles on Moona Moona Creek and its southern Tributary for the full range of flood events from 5 year ARI to PMF. **Figure 5.8** is a plan of the floodplain showing the extents of inundation and flood levels for the 10 and 100 year ARI floods and the PMF.

5.3.2 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 of the modelled reach, the higher catchment flows associated with the Normal Tide scenario result in flood levels 200-300 mm higher than for the Storm Tide case. A similar situation occurs on the southern tributary of Moona Moona Creek.

Flow velocities over the modelled reaches are quite low, generally less than 0.5 m/s in the channel and less than 0.2 m/s in the floodplain. 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 streams 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 runoff contributions from the major tributaries.

5.4 Preliminary Hydraulic Categories

According to the Floodplain Development Manual, 2005, the floodplain may be subdivided into the following hydraulic zones:

- 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 level 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 of 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.

Based on the procedures described above, the hydraulic models were used to estimate the floodway/flood storage boundaries for both the 100 year ARI and 10 year ARI floods on Currambene and Moona Moona Creeks. The results are presented in **Appendix C**.

5.5 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 1 m in the absence of significant flow velocities 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 the provisional *Low* and *High Hazard* zones. Hazard diagrams for the 10 and 100 year ARI floods are shown in **Appendix C**.

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.

5.6 Impacts of Entrance Scour on Flood Levels

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 during future major storm events, with the actual scour depending on the tailwater level in Jervis Bay. 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.

The impacts on peak flood levels of potential scour at the two outlets are modelled in **Appendix C**. As shown therein, the uncertainties regarding the likely scour at the two openings are somewhat academic, as the Storm Tide scenario, which is uninfluenced by scour due to the backwater effects of the tide, was found to govern design flood levels in the lower reaches of the two streams.

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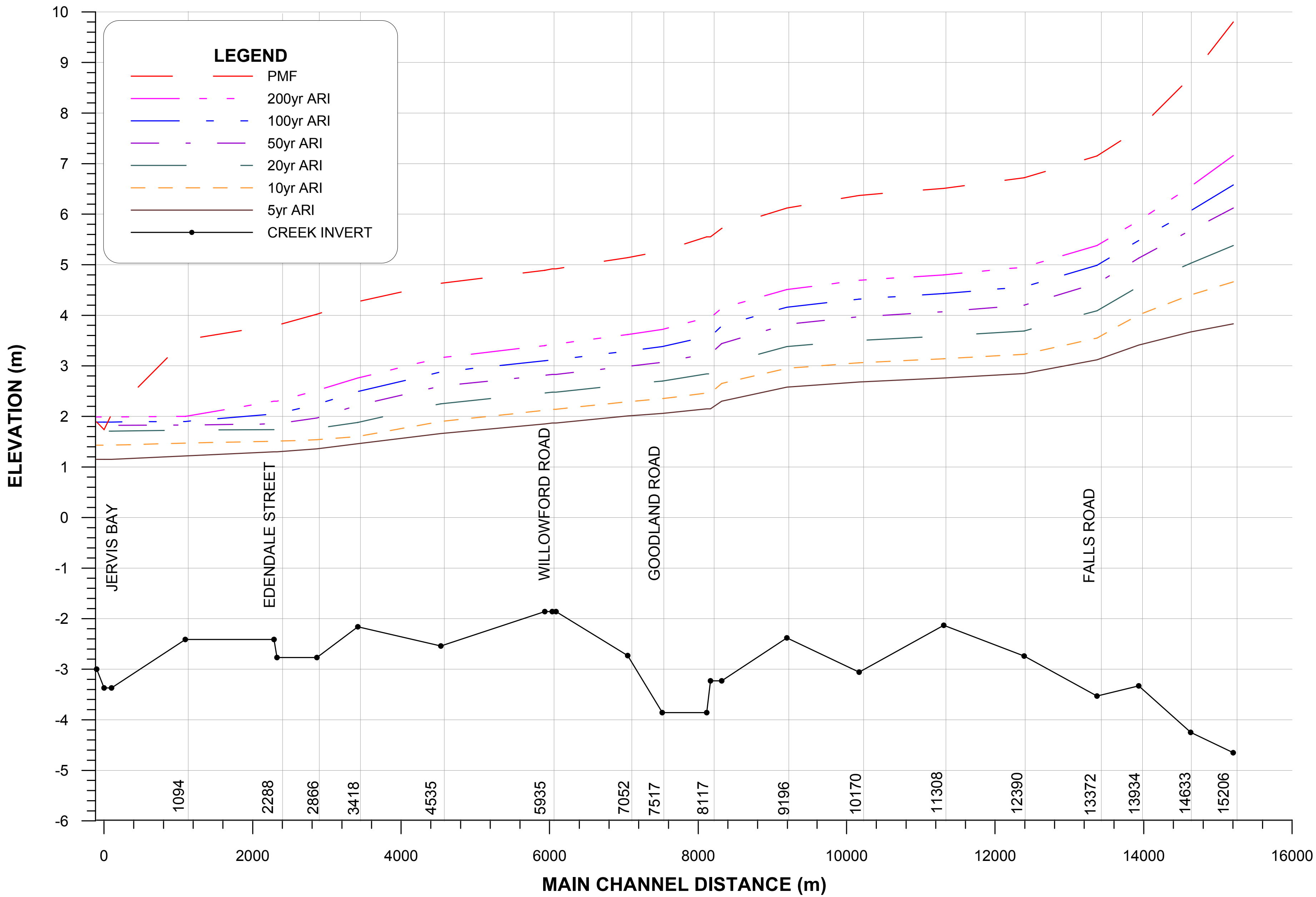
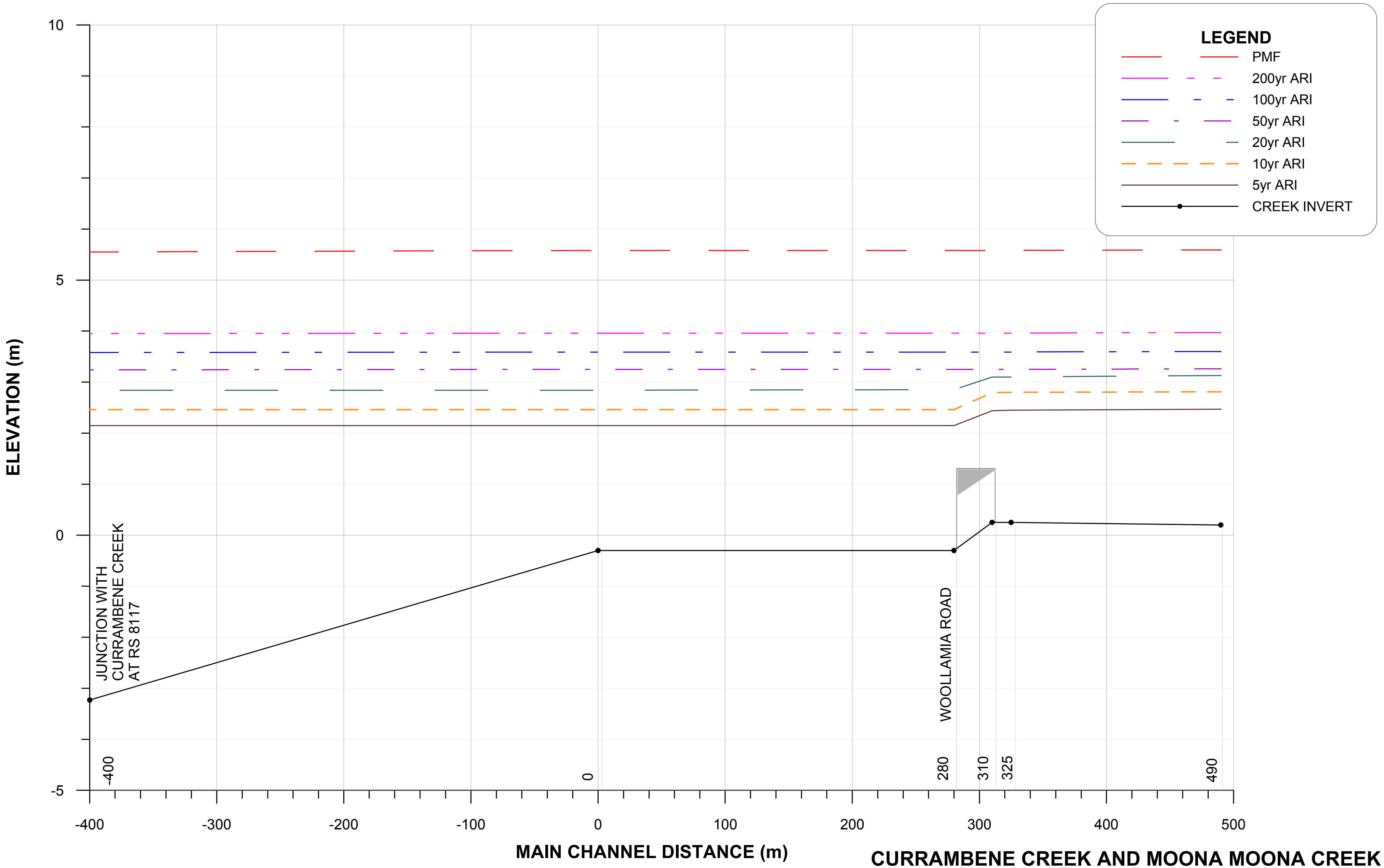


CURRAMBENE CREEK AND MOONA MOONA CREEK FLOOD STUDIES

Figure 5.1

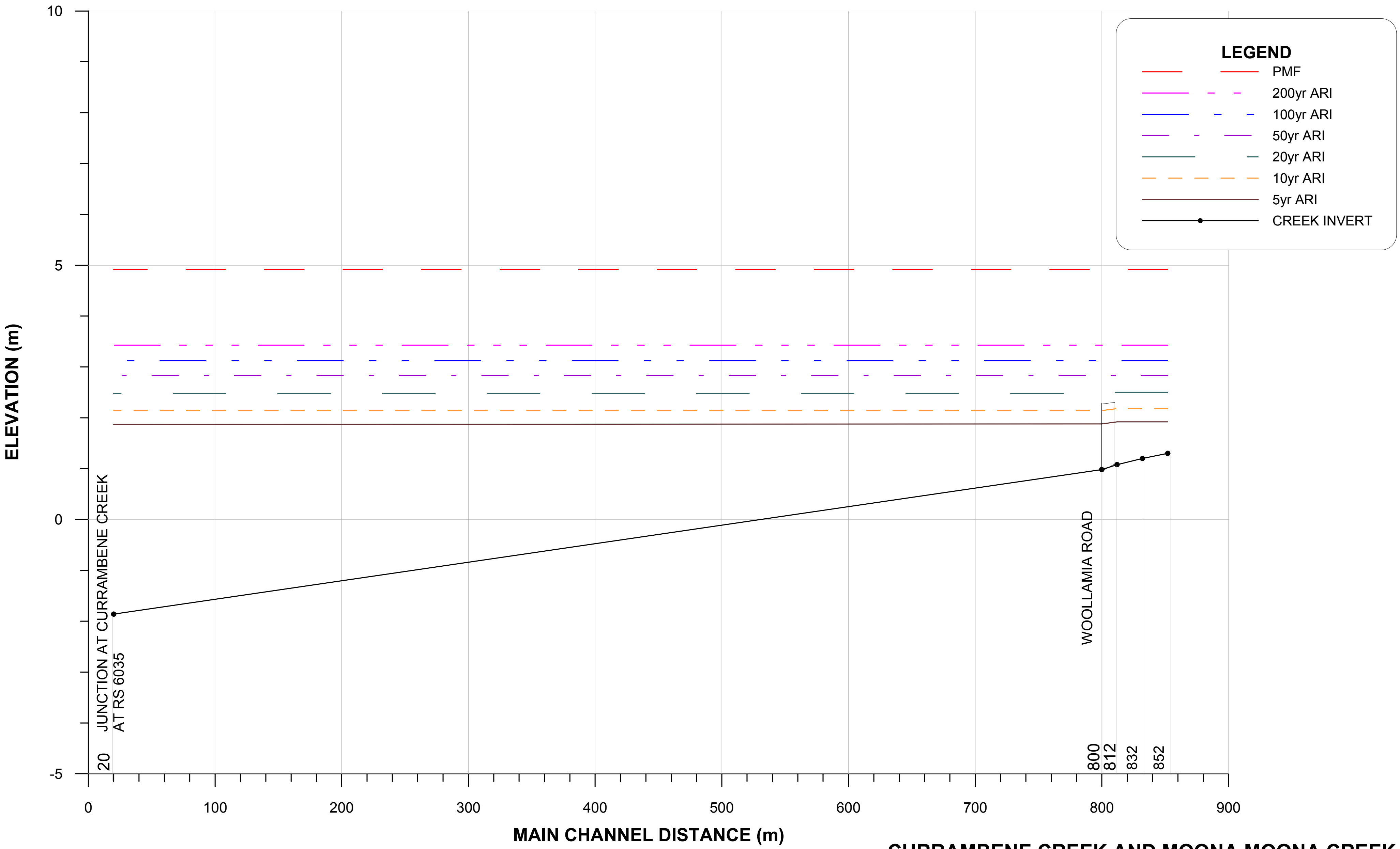
CURRAMBENE CREEK
DESIGN WATER SURFACE PROFILES
5 YEAR ARI TO PMF



CURRAMBENE CREEK AND MOONA MOONA CREEK FLOOD STUDIES

Figure 5.2

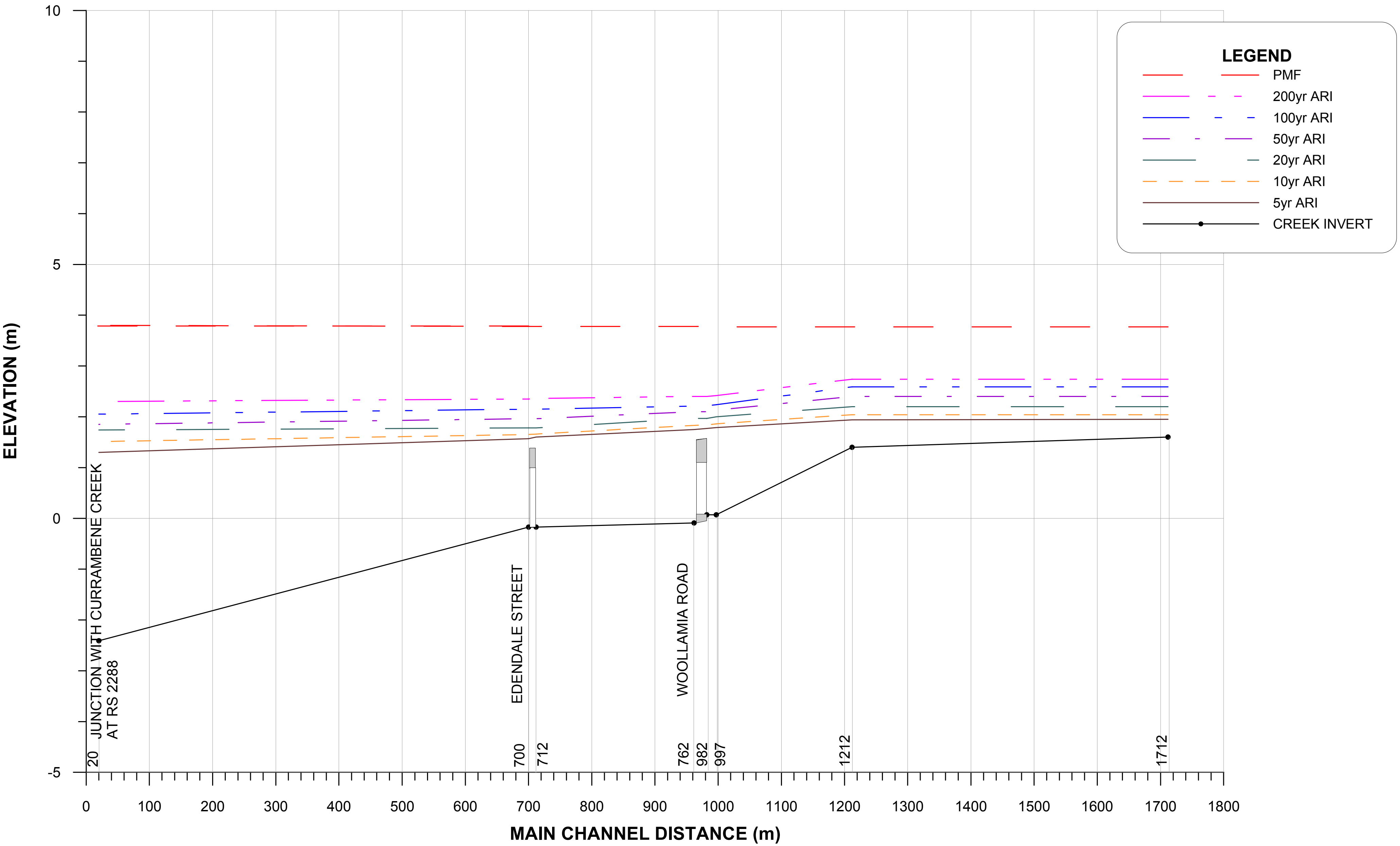
DESIGN WATER SURFACE PROFILES
TRIBUTARY 1 OF CURRAMBENE CREEK
5 YEAR ARI TO PMF



CURRAMBENE CREEK AND MOONA MOONA CREEK FLOOD STUDIES

Figure 5.3

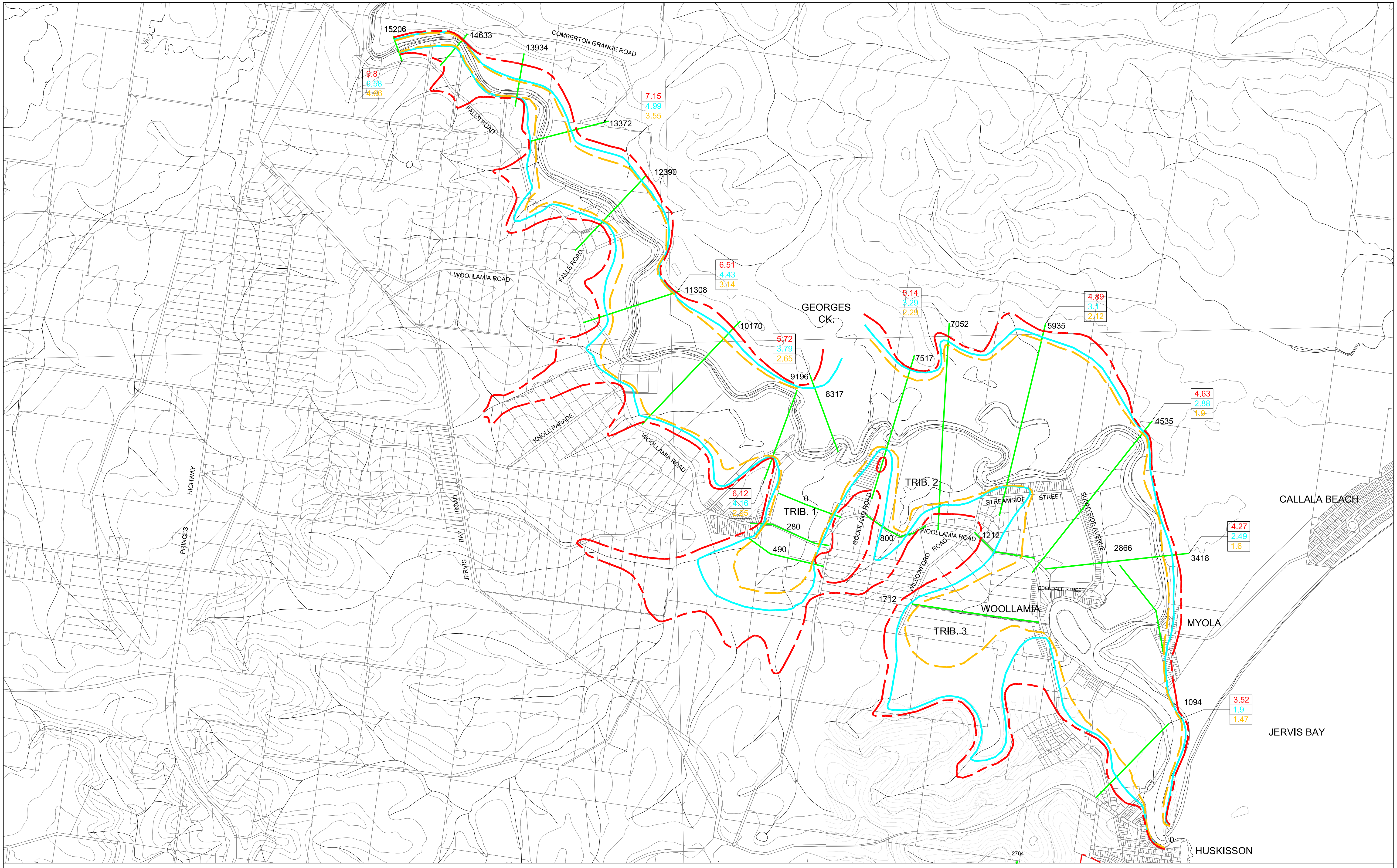
DESIGN WATER SURFACE PROFILES
TRIBUTARY 2 OF CURRAMBENE CREEK
5 YEAR ARI TO PMF



CURRAMBENE CREEK AND MOONA MOONA CREEK FLOOD STUDIES

Figure 5.4

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.

SCALE

0 100 500 1000m

N

LEGEND

PMF

100 YEAR ARI

10 YEAR ARI

PEAK FLOOD LEVELS

4.27 PMF

2.49 100 YEAR ARI

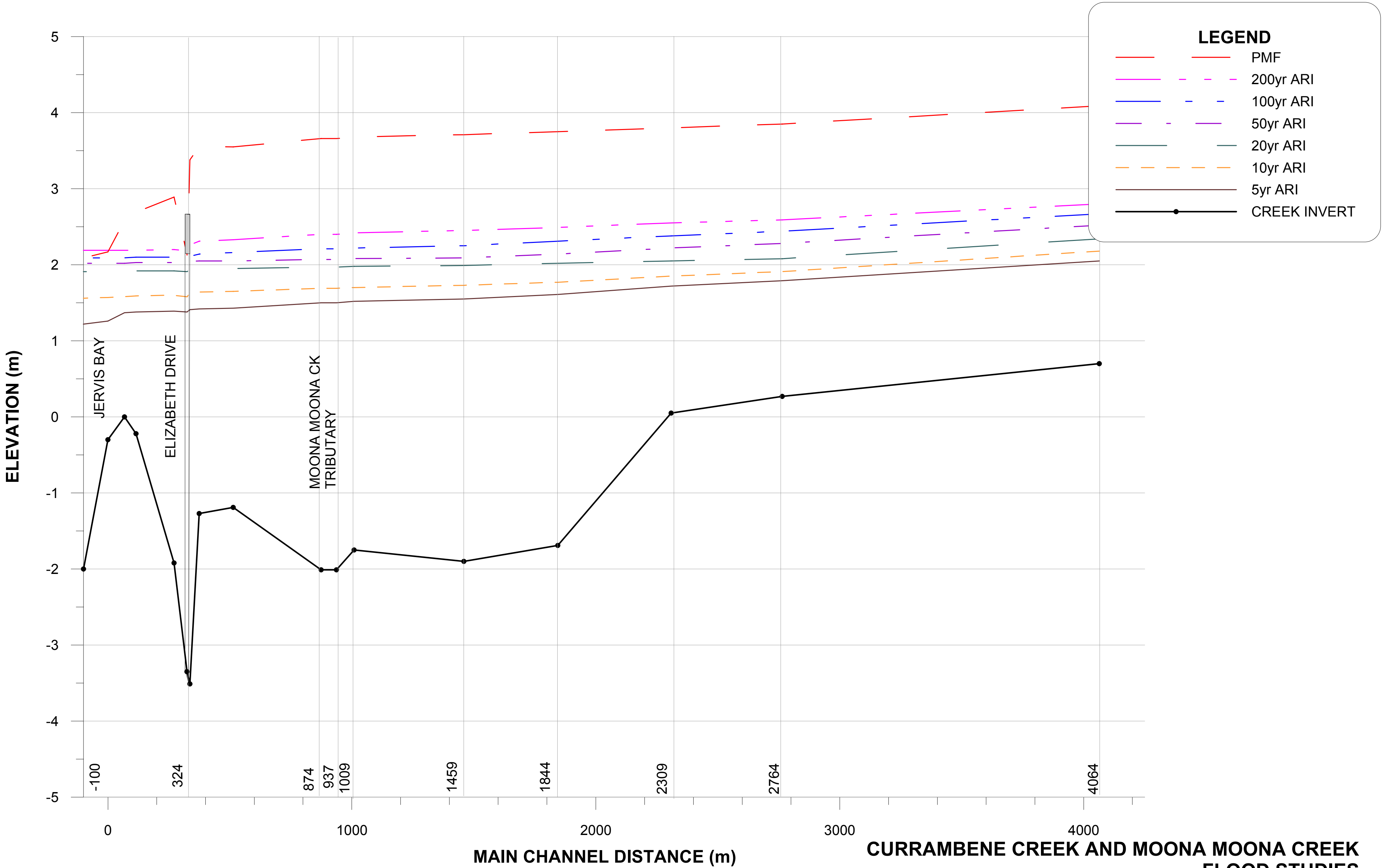
1.6 10 YEAR ARI

MODEL RIVER STATION 3418

CURRAMBENE CREEK AND MOONA MOONA CREEK
FLOOD STUDIES

Figure 5.5

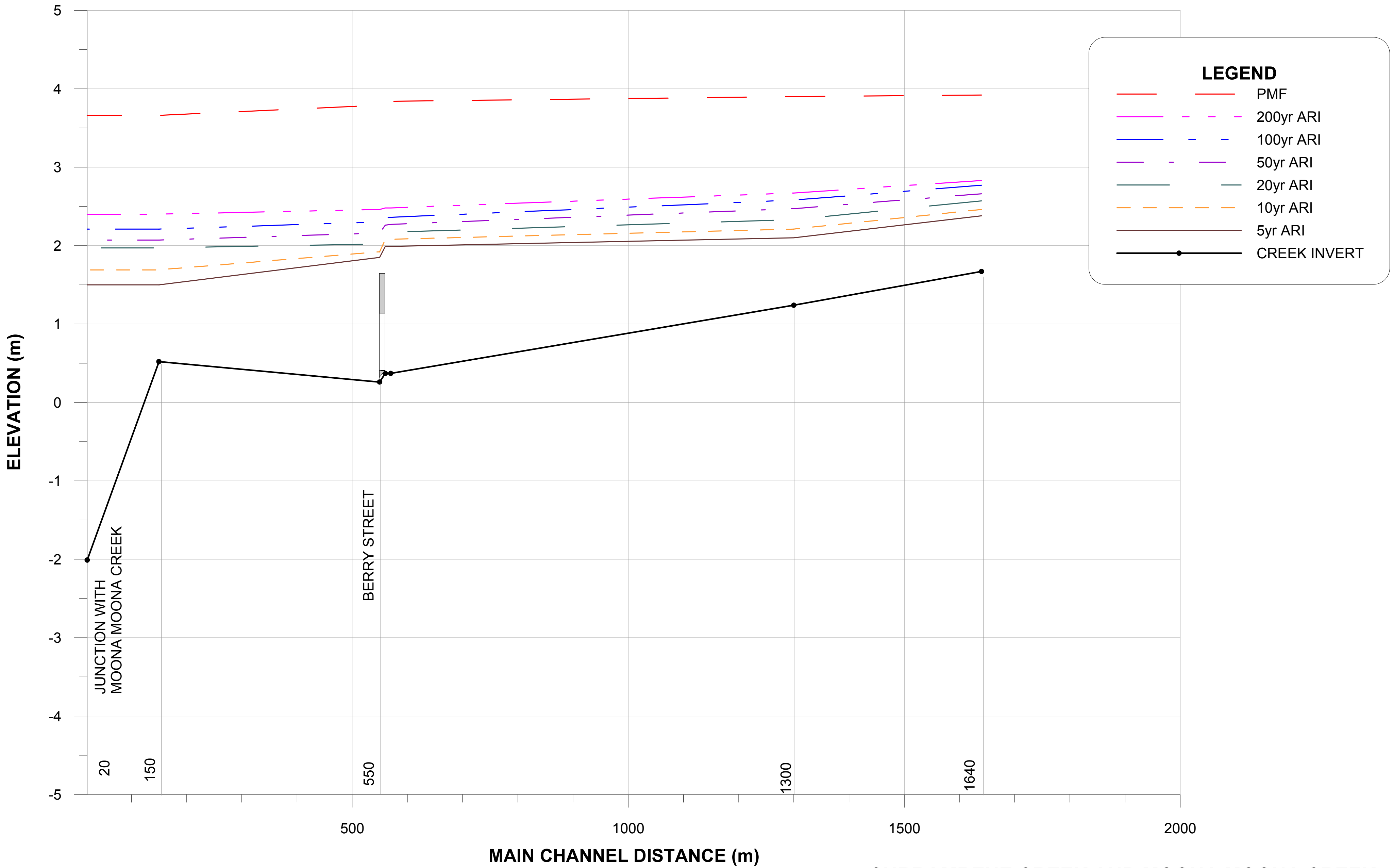
CURRAMBENE CREEK
INDICATIVE EXTENTS OF INUNDATION
10 YEAR, 100 YEAR ARI AND PMF



CURRAMBENE CREEK AND MOONA MOONA CREEK FLOOD STUDIES

Figure 5.6

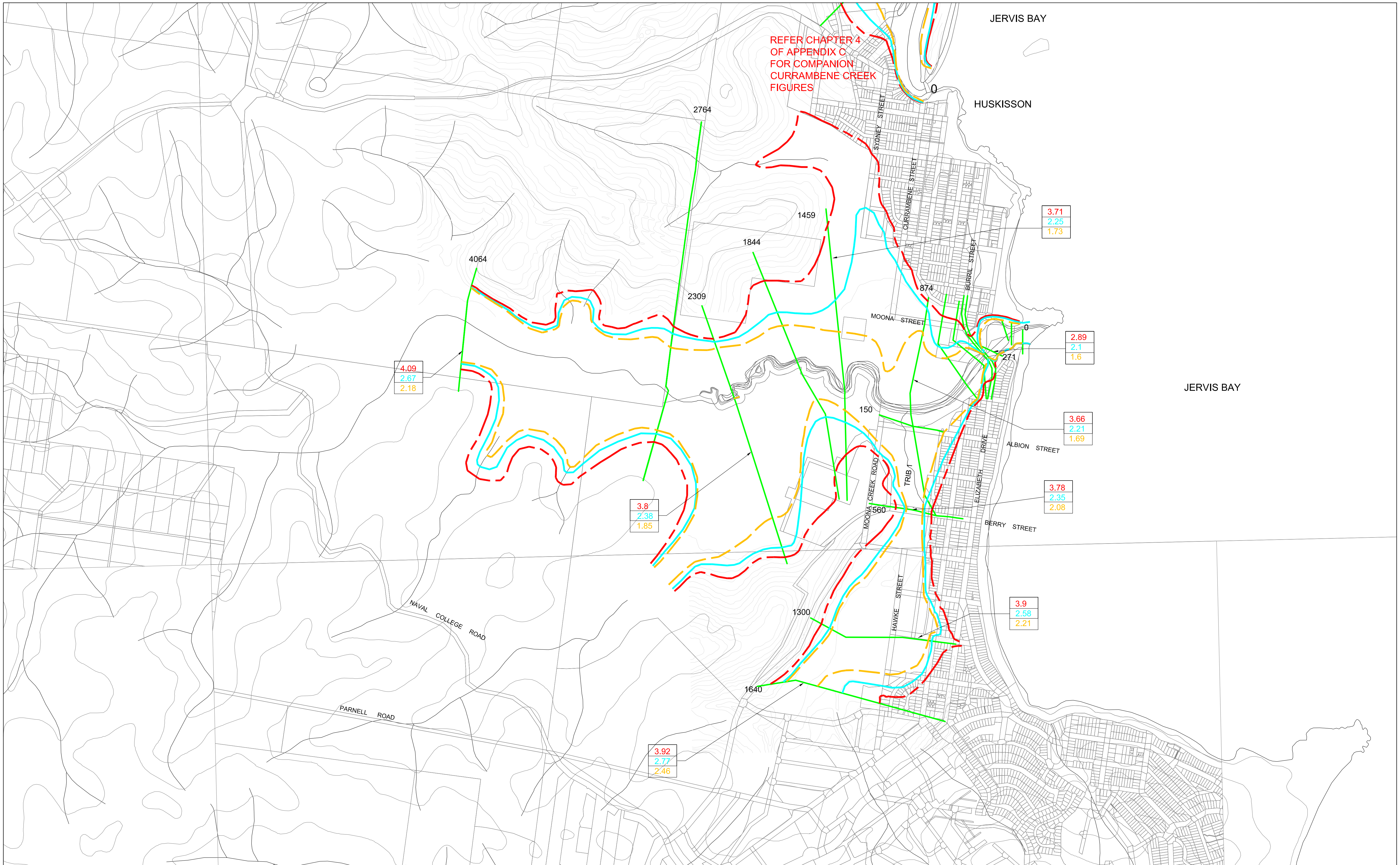
MOONA MOONA CREEK
DESIGN WATER SURFACE PROFILES
5 YEAR ARI TO PMF



CURRAMBENE CREEK AND MOONA MOONA CREEK FLOOD STUDIES

Figure 5.7

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



N

SCALE

0 250 500m

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.

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

2309

CURRAMBENE CREEK AND MOONA MOONA CREEK
FLOOD STUDIES

Figure 5.8

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

6. SUMMARY AND RECOMMENDATIONS

6.1 Summary

Flood behaviour on Currambene Creek and its contiguous catchment Moona Moona Creek **(Figure 2.1)** has been defined using computer based hydrologic models of the catchments and hydraulic models of the two streams and main tributaries.

The hydrologic models were based on a runoff routing approach and in the case of Currambene Creek, calibrated against rainfall and runoff data recorded on that catchment. Parameters derived from calibrating the Currambene Creek model gave guidance for the selection of parameters for design flood estimation on that catchment and also on Moona Moona Creek.

A dynamic hydraulic modelling approach was adopted for the conversion of design hydrographs to flood levels, flow distribution and velocities along the streams. The hydraulic model allowed for the time varying effects of flows over the duration of the flood as well as the routing effects of the floodplain storage and the potential effects of entrance and storm tidal conditions in the estuarine areas and outlets of both streams.

In accordance with recognised procedures, a Flood Envelope approach was adopted for defining design water surface profiles for floods ranging between 5 and 200 years ARI and the Probable Maximum Flood. This procedure involved selection of the upper limit of expected flooding for each frequency resulting from two alternative scenarios:

- Catchment runoff derived from design storm events of the relevant frequency in conjunction with a Normal Semi-Diurnal Tide.
- Storm tide hydrographs of the relevant frequency in conjunction with a minor 5 year ARI catchment flood.

Elevated ocean levels due to storm tides and wave action controlled design flood levels in the lower reaches of both creeks, whereas catchment flooding controlled further upstream.

6.2 Recommendations for Further Hydraulic Analysis

The investigation described in this report and appendices represent a detailed investigation of flood behaviour in the two catchments under present day conditions.

Several additional issues will need to be investigated in the future *Floodplain Risk Management Study.*

- Definition of flooding on Georges Creek, an important tributary of Currambene Creek, which joins the left (northern bank) in its middle reaches opposite Goodland Road. There is presently no survey data available, consequently a detailed cross sectional survey of the channel and floodplain upstream of the confluence will be required to model this stream.
- Further consideration of the hydraulic categorisation to define floodway-flood storage areas on the tributaries of Currambene Creek denoted Tributaries 1 and 3. As discussed in **Section 5.2**, these streams along with Tributary 2, provide storage for flooding from Currambene Creek. Design flood levels on the tributaries are controlled by backwater influences from that stream. However, Tributaries 1 and 3 also have substantial sub-catchments which generate significant flows which peak prior to the arrival of the flood peak on Currambene Creek. Further analysis will be required to define the floodway widths on the tributaries.

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