Elliot Lake - Little Lake Flood Study

Report Prepared For

Shellharbour City Council

Report J1959/R1974-V5
January, 2006

Cardno Lawson Treloar Pty Ltd
Level 2
910, Pacific Highway
Gordon NSW 2072
Australia

Telephone: 61 2 9499 3000
Facsimile: 61 2 9499 3033
Email: clt@syd.cardno.com.au

A.B.N. 55 001 882 873
A.C.N. 001 882 873

REPORT STATUS

Version	Date	Status	Prepared by:	Reviewed by:	Approved by:
1	28 August 2002	Draft	HUR	PDT	NVL
2	10 December 2002	Draft	HUR	PDT/LCH	NVL
3	18 May 2003	Draft	HUR	PDT/LCH	NVL
4	July 2005	Final Draft	HUR	PDT/LCH	NVL
5	January 2006	Final Draft 2	HUR	PDT/LCH	NVL

It is the responsibility of the reader to verify the currency of the version number of this report. All subsequent releases will be made directly to Council.

REPORT COPY NUMBER: ....

TABLE OF CONTENTS

LIST OF FIGURES

LIST OF TABLES

LIST OF APPENDICES

FOREWORD

The State Government's Flood Prone Lands Policy is directed towards providing solutions to existing flood problems in developed areas utilising ecologically positive methods wherever possible and ensuring that new development is compatible with the flood hazard and does not create additional flooding problems in other areas.

Under the policy, the management of flood prone land is the responsibility of Local Government. To achieve its primary objective, the policy provides for State Government's financial assistance to Councils for flood mitigation works to alleviate existing flooding problems. The policy also provides for State Government's technical assistance to the Councils to ensure that the management of flood prone land is consistent with the flood hazard and that the future developments do not create or increase flooding problems in the flood prone land.

The Policy provides for technical and financial support by the State Government through the following sequential stages:

1. Flood Study	Determines the nature and extent of the flood problem.
2. Floodplain Risk Management Study	Evaluates management options for the floodplain in respect of both existing and proposed development.
3. Floodplain Risk Management Plan	Involves formal adoption by Council of a plan of management for the floodplain.
4. Implementation of the Plan	Construction of flood mitigation works to protect existing development.
	Use of Local Environmental Plans to ensure new development is compatible with the flood hazard.

The Elliot Lake – Little Lake Flood Study is the first stage of the management process for the Elliot Lake – Little Lake Catchment. This study, which has been prepared for Shellharbour City Council by Lawson & Treloar Pty Ltd, defines flood behaviour under April 2001 as well as fully developed/final catchment conditions.

EXECUTIVE SUMMARY

A flood study of the Elliot Lake – Little Lake catchment has been undertaken to define the nature and extent of flooding in the study area for a range of design rainfall events. The study has been carried out for the April 2001 as well as fully developed/final catchment conditions excluding the lake foreshore areas.

The Elliot Lake – Little Lake catchment, which has an area of 12.26 square kilometres, lies south of Lake Illawarra and includes the suburbs of Barrack Heights, Shellharbour, Flinders, Blackbutt and parts of Mt Warrigal and Warilla. The catchment has reached its full development potential with a mix of low density housing, commercial and light industrial land uses, except for Blackbutt Forest and a reserve in the southern parts. These areas are not presently zoned for future development. There are two major creeks in the catchment. Bensons Creek, which drains the northern half of the catchment and Tongarra Creek, which drains the southern half of the catchment. These creeks drain into Elliot Lake – Little Lake, which discharges to the ocean through a trained entrance at Warilla Beach.

The potential for flood impacts on recent developments in the catchment, in particular the lake foreshore area, has prompted Shellharbour City Council, through the Shellharbour Floodplain Risk Management Committee, to commence preparation of a comprehensive Floodplain Management Plan. The Plan, which will cover the lake foreshore areas and the floodplain of its tributaries, is being prepared as part of the State Government's program to manage major flood impacts and hazards in floodplains, in accordance with the State Government's Floodplain Development Manual (2005).

In the past, flooding in the Elliot Lake – Little Lake catchment has caused property damage and posed a high hazard to the residents living close to the foreshore area and major drainage channels. The catchment has experienced major flood events in October 1959, November 1961, June 1964, November 1966, November 1969, February 1971, May 1974, March 1975, April 1978, May 1983 and April 1998. Of these the March 1975 and the May 1983 storms were well recorded. Flooding in the catchment is caused by runoff in the upper parts of the catchment whereas in the foreshore area, ocean water levels also play a major role.

Estimation of flooding behaviour was undertaken by developing two mathematical models to simulate the hydrologic and hydraulic aspects of flooding. The hydrological modelling package RAFTS was utilised to determine catchment runoff and for routing flows through the catchment. Predicted hydrographs from RAFTS were then input to the dynamically linked one-dimensional/two-dimensional hydraulic model, SOBEK for the determination of peak flood level, velocity and discharge for various design rainfall events. The design rainfall events investigated for this study were 1%, 2%, 5%, 10%, 20% and 50% annual exceedence probability (AEP) events together with the Probable Maximum Flood (PMF).

Flood levels within the catchment for the storm events of May 1983 and March 1975 were available for calibration and validation of the hydraulic model. These events were chosen on the basis of flood level information obtained from the previous

studies and the available rainfall data. The model was calibrated to the May 1983 event and validated for the March 1975 event.

Design rainfall intensities and temporal patterns for the required rainfall events were obtained from Australian Rainfall and Runoff (1999) (AR&R). The Probable Maximum Precipitation (PMP) was estimated using the Generalised Short Duration Method recommended by the Bureau of Meteorology (1994). The embedded storm approach for hydrological analysis was also investigated.

The model results indicate that there are greater flooding impacts in Bensons Creek catchment than the Tongarra Creek catchment. In the event of a major flood, Shellharbour Road would be overtopped at the Bensons Creek crossing, Tongarra Creek crossing and the Oakley Creek crossing near Sunset Avenue. The undeveloped area south of George Street opposite the Lake Windemere Caravan Park will be completely inundated in a 1% AEP flood.

Limits of predicted flood extents for the 1%, 5% and 20% AEP events together with the PMF are provided in plan form with cadastral information. Tabulated modelling results, which include predicted flood levels, velocities and flows at a number of locations in the floodplain, are also provided.

The flood hazard has been determined for the 1%, 5% and 20% AEP events and the PMF. The hazard categorisation has been provided on the cadastral plan of the study area.

The hydraulic categories of the flood-affected area have been identified and provided as extents on the aerial photograph for each of the 1%, 5% and 20% AEP events and the PMF.

All the above information has been prepared in a GIS system, which is compatible with the Council's system. The data is available electronically and can be used in a variety of ways.

The impact of variability of significant model parameters has been assessed by carrying out a sensitivity analysis. Model parameters such as channel roughness, catchment runoff and downstream boundary conditions have been checked for sensitivity. Culvert blockage is an issue, which is increasingly receiving attention in areas near the Elliot Lake – Little Lake catchment. Although the Elliot Lake – Little Lake catchment has not experienced significant culvert blockages, a sensitivity analysis was carried out to appreciate the likely impact of this issue for the catchment.

This study has produced flood behaviour information and provides a management tool in the form of a hydraulic model for future assessment of floodplain management options in the study area.

GLOSSARY OF TERMS

Terms in this Glossary have been derived or adapted from the NSW Government *Floodplain Development Manual,* 2005.

Annual Exceedence Probability (AEP)	Refers to the probability or risk of a flood of a given size occurring or being exceeded in any given year. A 90% AEP flood has a high probability of occurring or being exceeded each year; it would occur quite often and would be relatively small. A 1% AEP flood has a low probability of occurrence or being exceeded each year; it would be fairly rare but it would be relatively large.
Australian Height Datum (AHD)	A common national surface level datum approximately corresponding to mean sea level.
Australian Rainfall and Runoff (AR&R)	Institution of Engineers publication pertaining to rainfall and flooding investigations in Australia
Cadastre, cadastral base	Information in map or digital form showing the extent and usage of land, including streets, lot boundaries, water courses etc.
Catchment	The area draining to a site. It always relates to a particular location and may include the catchments of tributary streams as well as the main stream.
Design Flood	A significant event to be considered in the design process; various works within the floodplain may have different design events: some roads may be designed to be overtopped in annual flood event.
Development	The erection of a building or the carrying out of work; or the use of land or of a building or work; or the subdivision of land.
Discharge	The rate of flow of water measured in terms of volume over time. It is to be distinguished from the speed or velocity of flow, which is a measure of how fast the water is moving rather than how much is moving.
Flash flooding	Flooding which is sudden and often unexpected because it is caused by sudden local heavy rainfall or rainfall in another area. Often defined as flooding which occurs within 6 hours of the rain that causes it.

Flood	Relatively high stream flow which overtops the natural or artificial banks in any part of a stream, river or drainage system.
Flood fringe	The remaining area of flood-prone land after floodway and flood storage areas have been defined.
Flood hazard	Potential risk to life and limb caused by flooding.
Flood Liable Land	Is synonymous with flood prone land and is land covering the entire area flooded by the probable maximum flood event.
Flood-prone land	Land susceptible to inundation by the probable maximum flood (PMF) event, i.e. the maximum extent of flood liable land. Floodplain Risk Management Plans encompass all flood-prone land, rather than being restricted to land subject to designated flood events.
Floodplain	Area of a river valley adjacent to the river channel, which is subject to inundation by the probable maximum flood event.
Floodplain management measures	The full range of techniques available to floodplain managers.
Floodplain management options	The measures which might be feasible for the management of a particular area.
Flood storages	Those parts of the floodplain that are important for the temporary storage of floodwaters during the passage of a flood.

Floodway areas	Those areas of the floodplain where a significant discharge of water occurs during floods. They are often, but not always, aligned with naturally defined channels. Floodways are areas that, even if only partially blocked, would cause a significant redistribution of flood flow, or significant increase in flood levels. Floodways are often, but not necessarily, areas of deeper flow or areas where higher velocities occur. As for flood storage areas, the extent and behaviour of floodways may change with flood severity. Areas that are benign for small floods may cater for much greater and more hazardous flows during larger floods. Hence, it is necessary to investigate a range of flood sizes before adopting a design flood event to define floodway areas.
Freeboard	Freeboard is the difference between the flood event upon which the Flood Planning Level (FPL) is based and the FPL itself. The freeboard is a factor of safety that takes into account; Uncertainties in the estimation of flood levels, the difference in water levels across the floodplain due to local factors, waves generated by wind or vehicles, climate change and the cumulative effect of subsequent infill development.
Geographical information systems (GIS)	A system of software and procedures designed to support the management, manipulation, analysis and display of spatially referenced data.
High hazard	Possible danger to life and limb; evacuation by trucks difficult; able-bodied adults would have difficulty wading to safety; potential for significant structural damage to buildings.
Hydraulics	The term given to the study of water flow in a river, channel or pipe, in particular, the evaluation of flow parameters such as stage and velocity.
Hydrograph	A graph that shows how the discharge changes with time at any particular location.
Hydrology	The term given to the study of the rainfall and runoff process as it relates to the derivation of hydrographs for given floods.

Integrated survey grid (ISG)	ISG is a global co-ordinate system based on a Transverse Mercator Projection. The globe is divided into a number of zones, with the true origin at the intersection of the Central Meridian and the Equator.
Low hazard	Should it be necessary, people and their possessions could be evacuated by trucks; able-bodied adults would have little difficulty wading to safety.
Mainstream flooding	Inundation of normally dry land occurring when water overflows the natural or artificial banks of the principal watercourses in a catchment. Mainstream flooding generally excludes watercourses constructed with pipes or artificial channels considered as stormwater channels.
Management plan	A document including, as appropriate, both written and diagrammatic information describing how a particular area of land is to be used and managed to achieve defined objectives. It may also include description and discussion of various issues, special features and values of the area, the specific management measures which are to apply and the means and timing by which the plan will be implemented.
Mathematical/computer models	The mathematical representation of the physical processes involved in runoff and stream flow. These models are often run on computers due to the complexity of the mathematical relationships. In this report, the models referred to are mainly involved with rainfall, runoff, pipe and overland stream flow.
NPER	National Professional Engineers Register. Maintained by the Institution of Engineers, Australia.
Peak discharge	The maximum discharge occurring during a flood event.
Probable maximum flood (PMF)	The flood calculated to be the maximum that is likely to occur.

Probability	A statistical measure of the expected frequency or occurrence of flooding. For a fuller explanation see Annual Exceedence Probability.
Risk	Chance of something happening that will have an impact. It is measured in terms of consequences and likelihood. For this study, it is the likelihood of consequences arising from the interaction of floods, communities and the environment.
Runoff	The amount of rainfall that actually ends up as stream or pipe flow, also known as rainfall excess.
Stage	Equivalent to 'water level'. Both are measured with reference to a specified datum.
Stage hydrograph	A graph that shows how the water level changes with time. It must be referenced to a particular location and datum.
Stormwater flooding	Inundation by local runoff. Stormwater flooding can be caused by local runoff exceeding the capacity of an urban stormwater drainage system or by the backwater effects of mainstream flooding causing the urban stormwater drainage system to overflow.
Topography	A surface which defines the ground level of a chosen area.

LIST OF ABBREVIATIONS

AAD	Average Annual Damage
AEP	Annual Exceedence Probability
AHD	Australian Height Datum
AMG	Australian Mapping Grid
ARI	Average Recurrence Interval
AWRC	Australian Water Resources Council
BoM	Bureau of Meteorology
CMB	Catchment Management Board
DCP	Development Control Plan
DLWC	Department Of Land and Water Conservation
DPWS	Department of Public Works and Services
DUAP	Department of Urban Affairs and Planning
FPL	Flood Planning Level
FRMC	Floodplain Risk Management Committee
FRMP	Floodplain Risk Management Plan
FRMS	Floodplain Risk Management Study
GIS	Geographic Information System
GSDM	Generalised Short Duration Method
ha	hectare
IEAust	Institution of Engineers Australia
IFD	Intensity Frequency Duration
km	kilometres
km^2	Square kilometres
L&T	Lawson & Treloar
LAP	Local Approvals Policy
LEP	Local Environment Plan
LGA	Local Government Area
LIC	Land Information Centre

m	metre
m^2	Square metres
m^3	Cubic metres
mAHD	Metres to Australian Height Datum
MHIs	Maximum Height Indicators
MHL	Manly Hydraulics Laboratory
MHWL	Mean High Water Level
mm	millimetre
m/s	metres per second
MSL	Mean Sea Level
NSW	New South Wales
PMF	Probable Maximum Flood
PMP	Probable Maximum Precipitation
PWD	Public Works Department New South Wales
RAFTS	RAFTS proprietary software package
RTA	Roads and Traffic Authority
SCC	Shellharbour City Council
SRA	State Rail Authority
WBNM	Watershed Boundary Network Model

1. INTRODUCTION

The Elliot Lake – Little Lake catchment lies south of Lake Illawarra and includes the suburbs of Barrack Heights, Shellharbour, Flinders, Blackbutt and major areas of Mt Warrigal and Warilla. Except for the Blackbutt Forest Reserve and another un-named reserve in the southern parts, the catchment is fully urbanised. The study area, for which the detailed flood behaviour has been assessed, lies along the two main creeks that contribute flow to Elliot Lake – Little Lake. The creek, which drains the northern half of the catchment is called Bensons Creek and the creek draining the southern half is called Tongarra Creek. The total catchment area is 12.26 square kilometres. The location of the catchment and study area is shown in Figure 1.

Rapid development along the lake foreshore area and the potential for flood damages has prompted Shellharbour City Council, through the Shellharbour Floodplain Risk Management Committee, to commence preparation of a comprehensive Risk Floodplain Management Plan for the lake foreshore areas and the floodplain of its tributaries. This plan will form part of the State Government's program to manage major flood impacts and hazards in floodplains, in accordance with the State Government's Floodplain Development Manual, 2005.

The first step in the preparation of a Floodplain Risk Management Plan is to undertake a detailed Flood Study for the catchment. Shellharbour City Council commissioned Lawson and Treloar Pty Ltd (L&T) to undertake this flood study to determine the flood behaviour for the 1%, 2%, 5%, 10%, 20%, 50% AEP floods and Probable Maximum Flood (PMF). In accordance with its objectives, the study has determined the nature and extent of flooding through the estimation of design flood flows, levels and velocities. The study has defined Provisional Flood Hazard and Hydraulic Categories for the flood affected areas.

The various components of the flood study can be grouped together in two stages. Firstly, a full hydrologic investigation was carried out on the catchment using a hydrologic computer model. This involved the collection of available historical rainfall and flood level data. Historical flood data was gathered through an extensive data search and review of Council documents. Secondly, a hydraulic computer model of the major creeks in the catchment was established and calibrated using the historical flood level data. The hydraulic model was then used with design rainfall conditions to simulate flood behaviour in the catchment.

The hydraulic model developed in this study has been used to simulate flooding which may occur under existing as well as developed catchment conditions. The model may be used to investigate various management and flood mitigation options and can assist in defining long term Floodplain Management strategies.

2. STUDY METHODOLOGY

The objectives of the Flood Study are to:

- Identify all the flood-related data by searching all relevant data sources.
- Determine the likely extent and nature of flooding and identify potential hydraulic controls by carrying out detailed site visits of the study area.
- Define April 2001 fully developed/final catchment condition flood behaviour for mainstream flooding in the catchment with due consideration to the impact of ocean levels on flooding characteristics.
- Define design flood levels, velocities and flow distributions for the catchment.
- Define the extent of flooding for the 1%, 2%, 5%, 10%, 20% and 50% AEP floods and Probable Maximum Flood (PMF) for the catchment.
- Define Provisional Flood Hazard for the flood-affected areas.
- Define the Hydraulic Categories for the flood-affected areas.

Two numerical modelling tools were developed:

- A hydrologic model to convert rainfall on the catchment into runoff. The hydrologic model combines rainfall information with local catchment characteristics to estimate a runoff hydrograph.

- A hydraulic model to convert runoff into water levels and velocities throughout the major drainage system in the study area. The model simulates the hydraulic behaviour of the water within the study area by accounting for flow in the major channels as well as potential flowpaths, which develop when the capacity of the channels is exceeded. It relies on boundary conditions, which include the runoff hydrographs produced by the hydrologic model and the appropriate downstream ocean boundary.

Section 3 of the report discusses the content and sources of relevant data, which was utilised throughout the study. This section describes historical rainfall and flood level data, which were used in the calibration of the established hydrologic and hydraulic models.

Section 4 and 5 discuss the catchment characteristics and provides a description of the hydrological model used in the study.

Section 6 provides details of the study carried out to determine the appropriate ocean water levels for historic as well as the design storms.

Section 7 describes the hydraulic model utilised for the flood study, its calibration and subsequent use for design rainfall events.

Section 8 provides the results of design flood estimation for the catchment.

Section 9 quantifies the impact of model sensitivity on design flood estimation.

Section 10 and 11 provide details of provisional flood hazard and hydraulic categorisation as per the Floodplain Development Manual (2005).

Section 12 summarises the study results and provides discussion on various aspects of the results.

Section 13 qualifies the results of the study.

A number of figures are included to illustrate the study results. Spatially referenced data such as flood extents are represented in a Geographic Information System (GIS) package, which is compatible with Council's GIS platform (MapInfo).

3. FLOOD DATA

Data has been obtained from a number of sources and includes information required for input to the hydrologic and hydraulic models, together with information required for verification of model results and the adequate representation and presentation of those results.

3.1 Previous Studies and Reports

Shellharbour City Council provided the following reports, drawings and other data:

Reports:

1. Rose Consulting Group (1994). "Flinders Catchment Shellharbour Stormwater Drainage Review and Analysis".
2. NSW Department of Public Works (1991). "Little Lake Flood Study Compendium of data"; Report No. PWD 91071, ISBN 0730586464, November 1991.
3. Forbes, Rigby & Associates (1993). "A Review of Flooding in Tongarra Creek Between Shellharbour Rd & Barrack Avenue prepared for Mr M Cohen".
4. Gutteridge Haskins & Davey (1997). "HEC-RAS Analysis of Barrack Avenue Bridge – Letter Report to Shellharbour City Council".
5. NSW Department of Public Works (1996). "Coastal and Environmental Monitoring 1995/96 Shellharbour Council". Manly Hydraulics Laboratory, Report MHL838, PWD Report No 96170.
6. NSW Department of Public Works (1994). "New South Wales Coastal Rainfall Annual Summary 1993/94". Manly Hydraulics Laboratory, Report MHL698, PWD Report No 94043, ISBN 073127345.
7. NSW Department of Public Works (1994). "New South Wales Coastal Rainfall Annual Summary 1993/94". Manly Hydraulics Laboratory, Report MHL699, PWD Report No 94044, ISBN 073127426.
8. NSW Department of Public Works (1993). "New South Wales Estuary and River Water Levels Annual Summary 1992/93". Manly Hydraulics Laboratory, Report MHL659, PWD Report No 93069, ISBN 0731027523.
9. NSW Department of Public Works (1992). "Mid New South Wales Coastal Region Tide-Storm Surge Analysis". Manly Hydraulics Laboratory, Report MHL621, PWD Report No 92028, ISBN 0730594203.
10. Forbes, Rigby & Associates (1995). "A Review of Flooding and Development Options at Little Lake prepared for Shellharbour City Council".
11. Forbes, Rigby & Associates (1993). "A Review of Flooding in Bensons Creek Between Shellharbour Rd & Lakes Entrance rd prepared for Shellharbour City Council".
12. Forbes, Rigby & Associates (1993). "A Preliminary Opinion on the Impact of Filling on Flooding in Little Lake prepared for Mr D Arap Ali".
13. Forbes, Rigby & Associates (1995). "A Review of Flooding in Tongarra Creek Between Shellharbour Rd & Barrack Avenue prepared for Lucas & Tait".
14. Forbes, Rigby & Associates (1989). "A Report on the Impact of Flooding on the Development of Land at Sparta Street, Warilla".
15. Forbes, Rigby & Associates (1982). "Impact of Flooding on the Development of Land at Little Lake, Warilla".

16. Forbes, Rigby & Associates (1993). "Flood Level Determination Adjacent to Lot 27 D.P. 23988 Shellharbour Road, Warilla".
17. Forbes, Rigby & Associates (1988). "A Report on Flooding at Surfrider Caravan Park, Junction Street, Barrack Point, Shellharbour".
18. Forbes, Rigby & Associates (1999). "Little Lake Local Environmental Study Stage Two – Warilla Sports Club Land Lot 1 DP 628864 Shellharbour Road, Warilla prepared for Shellharbour City Council".

Drawings:

1. The Housing Commission of NSW (1978). Plans for the Construction of Detention Basins and Associated Works, Shellharbour Site 7200 Section 1, Sheets 1-2.
2. NSW Department of Housing (1991). Design Plans for Tongarra Basin, Shellharbour Site 7200/3/101, Sheets 1-6.
3. NSW Department of Housing (1993). Design Plans for Landcom Junction Basin, Shellharbour Site 7200/3/104, Sheets 1-31.
4. NSW Department of Housing (Undated). Design Plans for Tongarra Basin, Shellharbour, Sheets 1-16.
5. Craven Elliston & Hayes Pty Ltd (Undated). 0.5m Contour Map, Little Lake (surrounded by Parklands Drive, Wattle Road, Shellharbour Road, and Ocean Beach Drive).
6. M.W.S.& D.B, Sydney, NSW (1983). Flooding of 27/05/1983, Shellharbour W.P.C.P, Sheets 1-4.
7. Shellharbour City Council (1954). Design Plan for MR522 Shellharbour Road Bridge, Mary St.
8. Shellharbour City Council (1997). Cross Sections, Creek Across Barrack Ave. Sheets 1-2.
9. Shellharbour City Council (1999). Design Plans of the Landcom Junction Basin, Sheets 6, 10-11, 14-16, 21-22.
10. Shellharbour City Council (1998). Proposed Sport-Fields Development at Lot 201 DP 848203, Wattle Road Shellharbour, Sheet 1 of 1, Plan No. 3D 221.
11. Shellharbour City Council (Undated). Sub-Surface Investigation Data, Lake Entrance Road.
12. Shellharbour City Council (1995). Surface Correction MR522 South Bound Adjacent Mary St. Bridge, Sheet 1, Plan No. 4A-55.
13. Shellharbour City Council (1976). Reconstruction to Dual Carriageway at Warilla. Sheets 1, 4, 7, 9 & 12.
14. Shellharbour City Council (1976). MR522 Proposed Shellharbour By-Pass, Final Plans, Sheets 1-11.

3.2 Additional Ground Survey

A ground level survey of floodplain areas of the catchment was commissioned to obtain the geometric information required for the development of the hydraulic model. The hydraulic model included one-dimensional as well as two-dimensional elements. For the one-dimensional component, ground survey was carried out to define cross-sections along the creeks. For the two-dimensional component, aerial survey with photogrammetric techniques was used.

A brief was prepared for the two surveys, which outlined the details and the methodology for surveying the hydraulic features of the catchment. A print out of the surveyed cross-sections is provided in Appendix A.

The cross-section survey information was obtained to enable definition of the flow channels, significant overland flood flow paths and floodplain storage areas. Various flow control structures such as culverts and bridges were also surveyed. Details of these structures are also provided in Appendix A.

In addition, the pit and pipe data for relevant parts of the Bensons Creek catchment was also obtained. The data is provided in Appendix A.

The aerial survey was carried out to define the flat foreshore areas of Elliot Lake – Little Lake. The survey provided definition of the terrain in fine detail, which allowed the two-dimensional component of the hydraulic model to define more accurate flood behaviour.

The survey was carried out to Australian Height Datum (AHD). All level data provided in this report is based on this datum.

3.3 Historical Storm Rainfall Data

The following rainfall stations were identified in and around the study area:

Table 1 List of Rain Gauges

Name	Station No	Location		Type	Source	Operation
		Lat S	Long E			
Dunmore	068113	34°36’	150°51’	Daily	BOM	1962-1974
Oak Flats	068199	34°34’	150°49’	Daily	BOM	1974-1988
Albion Park	068000	34°34’	150°47’	Daily	BOM	1992-current
Windang	068123	34°32’	150°52’	Daily	BOM	1962-current
Pt Kembla Steel	068131	34°28’	150°53’	Pluvio	BOM	1963-current
Pt Kembla	--	34°29’	150°54’	Pluvio	PWD	1982-current
Little Lake	--	34°34’	150°52’	Pluvio	PWD	1991-current
Pt Kembla STW	568120	34°30’	150°55’	Daily	WB	1971-current
Shellharbour STW	568119	34°34’	150°52’	Daily	WB	1975-current
Bank St Pump Stn	568118	34°26’	150°54’	Pluvio	WB	1976-current

BOM = Bureau of Meteorology, PWD = Public Works Department, WB = Water Board

The location of these stations is presented in Figure 2.

Historical storm events to be used for model calibration were identified from the availability of pluviograph data. Data for following storm events was available (Reference 1):

Table 2: Historical Rainfall Events with Pluviometer Data

Month	Year
February	1971
March	1974
March	1975
March	1978
June	1978
May	1983
February	1984
April	1988

All the above events were recorded every hour except the May 1983 storm for which the complete pluvio data was available (every 0.5mm trips of the gauge).

The May 1983 and March 1975 events were used for calibration purposes. The rainfall data from various stations was used to define the rainfall isohyetal pattern for the two storm events.

All pluviograph data used in the calibration of the hydrologic and hydraulic models is tabulated in Appendix B.

3.4 Recorded Flood Levels

Previous reports and Council records were searched for historical flood data. The 1992 Flood Study (Reference 1) carried out by PWD provided information about the historical storms. The study provided flood levels for the following events:

Table 3: Recorded Flood Levels

Month	Year	Number of Flood Levels	Pluvio Data Available?
October	1959	1	No
November	1961	2	No
June	1964	1	No
November	1966	1	No
November	1969	1	No
February	1971	1	YES
May	1974	2	No
March	1975	6	YES
April	1978	2	No
June	1978	1	YES
May	1983	22	YES
April	1988	2	YES

In addition, Reference 2 also provided flood level information for the March 1975 event.

3.5 Hydraulic Model Calibration Events

For hydraulic model calibration, only those events for which pluviometer records as well as recorded flood levels are available can be utilised. Table 3 above, provides five such events. The May 1983 and March 1975 events were the only storms, which were well recorded both in terms of rainfall and flood levels. Therefore, these events were used for calibration and validation purposes. Other data provided information about the general flooding behaviour of the catchment.

3.6 Streamflow Data

As neither Bensons Creek nor Tongarra Creek is equipped with stream gauges, no stream flow data was available in the catchment.

3.7 Ocean Water Level Data

The tidal data from the Compendium of Data (Reference 1), NSW DPW (Reference 9), wave data from Sydney Ports Corporation (Bruce Hudson, Senior Coastal Technician), and Bureau of Meteorology records provided information for the 1983 flood event), making it possible for the establishment of downstream boundary conditions for the hydraulic models. Synoptic chart data was used to assess offshore wave direction as part of the input to an existing Port Kembla region wave propagation model. This model includes refraction, shoaling and a full frequency-direction description of wave spectra, but not wave breaking. Therefore a surf zone model which includes wave breaking and wave set-up was used to propagate the waves inshore from about the 12m depth to the Little Lake entrance. In the inshore calculations an entrance depth of 0.5m relative to AHD was assumed, with depth adjusted for tide. Those depth changes affect wave set-up in the lake entrance.

Total water level at the lake entrance was determined by summing the astronomical tide, wave set-up and the inverse barometer effects; adopting 1010hPa as a regional average atmospheric pressure. Wind set up is small at this site where nearshore depths are relatively deep.

3.8 Cadastral and Topographic Data

Shellharbour City Council provided cadastral base information and contour files for use during the Flood Study in digital format. The data was input to the MapInfo mapping package.

The Council also provided information about the land-use in the catchment through the Local Environment Plan and aerial photographs.

The above information was utilised in the development of the hydrology and hydraulic models. The information was also used to develop the survey brief for additional ground survey.

Survey data including the latest cadastral map, 2-metre contour map, catchment boundary, aerial photography, and details of existing culverts, pits, bridges, and channel cross sections were collected and collated from various sources. For the purpose of hydraulic modelling, Craven, Elliston & Hayes (Dapto) Pty Ltd carried out survey of an additional 114 cross sections. The locations of these surveyed cross sections, culverts and pits are shown in Figure 3. Detailed survey is provided in Appendix A.

3.9 Other Data

Apart from these data, References 3, 4, 5, 6 and 7 also provided valuable information, which were particularly useful in the construction of hydrologic and hydraulic models.

Lawson & Treloar also conducted a site inspections from which valuable first hand information was gathered.

4. CATCHMENT DESCRIPTION

4.1 General

The Little Lake catchment is located at the heart of the Shellharbour City Council LGA (Figure 1). The catchment has experienced significant urban development in the last few decades. At present, about 60% of the catchment has residential land use including roads. Another 31% is open pasture and wetlands and the remaining 9% is forest (eucalypts). The total area of the catchment is 12.26 square kilometres. The elevation varies from 140 mAHD in the southwest to 0 mAHD near the Elliot Lake – Little Lake entrance, and the overland slopes range from 16 % in the upper reaches to 0.1% near the foreshore area.

The surface soils in the catchment are generally of a rich black clay. The soils were derived from weathering of the underlying basalt rock, which is found throughout the entire region. The soils in the area are expected to be highly expansive with more than 10% of the soil being dispersive (Reference 3).

There are two major creeks in the Elliot Lake - Little Lake catchment, Tongarra Creek and Bensons Creek. Tongarra Creek drains an area of 6.63 square kilometres. The creek passes under Shellharbour Road through Barrack Swamp before discharging into Elliot Lake – Little Lake. There are three detention basins constructed on the creek, which are located upstream of Shellharbour Road, Wattle Road and in the new development further upstream. Most of the creek is in a natural state with some segments modified to a regular section.

Bensons Creek drains an area of 5.63 square kilometres. A significant segment of Bensons Creek flows through underground pipelines or culverts. The creek passes under Lake Entrance Road, flowing through Shellharbour Council grounds before experiencing a sharp bend and then discharges into Little Lake. In the study area, the creek changes from natural state to a concrete lined channel between Shellharbour Road and a trash rack downstream of the pedestrian bridge at Joan Avenue.

The channel network of the Little Lake catchment has many man-made structures, including detention basins, culverts and road/pedestrian bridges that can potentially restrict the flow of large floods. In addition, there is an extensive pit and pipe drainage network in the developed parts of the catchment.

4.2 Flooding Behaviour and History

The urban areas of Shellharbour City Council adjacent to the major creeks face a risk of moderate to severe flooding. For the lake foreshore areas, there are two contributing factors. One is the storm runoff generated in the catchment itself and the other is the potential for elevated ocean water levels. High ocean levels coupled with storm runoff can pose a high hazard to residents living close to Barrack Swamp, Elliot Lake - Little Lake, lower Bensons Creek and lower Tongarra Creek. High ocean levels can create tailwater conditions that can block the storm runoff from exiting the lake via the entrance across Warilla Beach. The level of the beach berm at Warilla Beach can also affect the flow of storm runoff.

The Department of Public Works compiled a compendium of data on flooding in Elliot Lake - Little Lake (Reference 1). Local newspapers within the Illawarra area have reported a large number of floods with events reported as far back as March 1873. More recent floods were experienced in May 1974, March 1975, April 1978, May 1983, April 1988 and August 1998. Of these events, the March 1975 and May 1983 floods have been the most significant.

During the March 1975 flood, many houses were inundated in the Shellharbour area. The damage to Council roads and bridges was estimated to be $70,000. The Council recorded a flood level immediately upstream of the Shellharbour Road Bridge of 2.84 mAHD (Reference 4).

The then Water Board (now Sydney Water Corporation) surveyed the May 1983 flood level profile adjacent to the Shellharbour Sewage Treatment Plant. The flood level immediately downstream of the Shellharbour Road Bridge was found to be 2.70 mAHD (Reference 1). The Council also conducted a survey and a resident interview for flood levels following the May 1983 flood.

Although surveyed water levels for other floods are also available for a number of locations on the east of Shellharbour Road (Reference 1), the AEPs of the storms producing these floods were considerably less than the 1% AEP (Reference 4).

Apart from the above records, NSW Department of Public Works has been recording water levels at Elliot Lake - Little Lake since 1992 (Reference 9).

4.3 Rainfall

The average annual rainfall in the Illawarra area is approximately 1200 mm. Among the four seasons, autumn is mostly likely to experience the highest seasonal rainfall, while spring is likely to be the driest season (Reference 2). In terms of spatial variability of rainfall, since the majority of the catchment falls on the edge of the coastal belt, the rainfall is relatively uniform (Reference 3).

There are two rain gauges within the Little Lake catchment maintained by the Bureau of Meteorology and Sydney Water Corporation respectively. There are also several other rain gauges outside the catchment. Details of these rain gauges are provided in Section 3. Rainfall records for several floods are available from the compendium of data (Reference 1).

5. HYDROLOGY

5.1 General

The Elliot Lake – Little Lake catchment is a typical urbanised catchment with residential land-use and some light industrial/commercial areas. The catchment has steep slopes in the upper reaches and becomes flat near the lake foreshore area. The following attributes were considered in the hydrological analysis of the catchment:

- Rainfall intensity-frequency-duration (IFD) relationships
- Sub-catchment division
- Slopes and overland flowpath lengths, and
- Land use (pervious and impervious areas).

Two catchment development stages were considered in the study – catchment condition in April 2001 (based on the aerial photograph for the same date) and fully developed or final catchment condition. The description for both catchment development conditions is provided below.

5.2 April 2001 Catchment Development Condition

Based on the topographic features (the 2-metre contour map) and land-use (aerial photograph), the catchment was divided into 72 sub-catchments. The sub-catchment layout is shown in Figure 4 and details of these sub-catchments can be found in Table 4 below.

Table 4: Elliot Lake – Little Lake Catchment Details- April 2001 Development Condition

Subcat-chment	Subcatchment area (ha)				Catch Slope (%)	Impervious (ha)	Pervious (ha)
	Grass	Forest	Urban	Subtotal			
B1	1.58	0.00	10.76	12.34	0.79	5.38	6.96
B10	2.97	0.00	11.50	14.47	3.34	5.75	8.72
B11	2.29	0.00	14.65	16.94	4.30	7.33	9.62
B12	3.99	0.00	20.79	24.78	4.99	10.40	14.39
B13	0.49	6.14	16.04	22.67	5.80	8.02	14.65
B14	0.25	13.78	3.81	17.84	10.05	1.91	15.94
B15	0.00	5.83	17.31	23.14	4.69	8.66	14.49
B16	0.56	0.00	17.86	18.42	5.31	8.93	9.49
B2	4.11	0.00	6.56	10.67	0.83	3.28	7.39
B3	0.00	0.00	10.41	10.41	1.87	5.21	5.21
B4	3.04	0.00	12.19	15.23	1.66	6.10	9.14
B5	0.00	0.00	12.67	12.67	1.62	6.34	6.34
B7	0.00	0.00	11.77	11.77	3.80	5.89	5.89
B7A	5.98	0.00	3.76	9.74	0.00	1.88	7.86
B8	0.00	0.00	20.36	20.36	3.59	10.18	10.18
B9	0.58	0.00	14.33	14.91	1.81	7.17	7.75

Subcat-chment	Subcatchment area (ha)				Catch Slope (%)	Impervious (ha)	Pervious (ha)
	Grass	Forest	Urban	Subtotal			
BA1	0.38	0.00	10.87	11.25	3.49	5.44	5.82
BA10	9.15	0.00	4.16	13.31	8.19	2.08	11.23
BA11	6.92	0.12	13.20	20.24	5.62	6.60	13.64
BA12	1.53	0.00	13.40	14.93	8.25	6.70	8.23
BA13	0.00	0.00	14.45	14.45	9.54	7.23	7.23
BA2	0.00	0.00	7.41	7.41	1.26	3.71	3.71
BA3	0.00	0.00	11.30	11.30	2.27	5.65	5.65
BA4	2.12	0.00	17.86	19.98	5.82	8.93	11.05
BA5	2.74	0.00	3.42	6.16	12.87	1.71	4.45
BA6	3.34	0.00	11.81	15.15	3.84	5.91	9.25
BA7	0.87	0.00	27.67	28.54	3.33	13.84	14.71
BA8	0.13	0.00	14.67	14.80	4.64	7.34	7.47
BA9	5.96	0.00	5.44	11.40	4.92	2.72	8.68
BB1	0.00	0.00	9.01	9.01	1.86	4.51	4.51
BB2	1.95	0.00	8.43	10.38	2.11	4.22	6.17
BB3	1.81	0.00	27.59	29.40	2.82	13.80	15.61
BB4	4.88	0.17	40.13	45.18	4.76	20.07	25.12
BC1	7.37	0.00	7.23	14.60	2.51	3.62	10.99
BC2	0.00	0.00	13.37	13.37	1.91	6.69	6.69
T1	1.82	0.00	22.41	24.23	2.13	11.21	13.03
T10	4.53	0.00	5.48	10.01	2.72	2.74	7.27
T11	9.57	0.00	15.65	25.22	4.26	7.83	17.40
T12	6.36	0.00	10.76	17.12	4.38	5.38	11.74
T13	11.57	0.00	22.87	34.44	4.89	11.44	23.01
T14	7.75	0.00	5.23	12.98	5.05	2.62	10.37
T15	21.67	0.00	0.00	21.67	6.63	0.00	21.67
T16	17.21	0.00	0.08	17.29	7.55	0.04	17.25
T17	16.25	0.00	0.08	16.33	11.44	0.04	16.29
T18	7.76	0.00	0.00	7.76	6.82	0.00	7.76
T19	23.97	0.00	0.00	23.97	16.16	0.00	23.97
T2	0.21	0.00	26.18	26.39	1.88	13.09	13.30
T3	3.25	0.00	8.41	11.66	1.69	4.21	7.46
T4	10.73	0.00	9.27	20.00	1.56	4.64	15.37
T5	0.00	17.64	27.21	44.85	6.11	13.61	31.25
T6	10.46	3.10	7.06	20.62	4.06	3.53	17.09
T7	6.48	0.00	0.00	6.48	N/A	0.00	6.48
T8	2.08	0.00	11.91	13.99	3.87	5.96	8.04
T9	20.29	0.00	0.00	20.29	3.72	0.00	20.29
TA1	0.00	1.65	6.29	7.94	8.67	3.15	4.80
TA2	0.00	5.07	8.83	13.90	7.79	4.42	9.49
TA3	0.00	7.04	4.14	11.18	6.85	2.07	9.11
TA4	0.00	20.19	0.93	21.12	5.78	0.47	20.66
TA5	0.00	13.19	1.15	14.34	7.95	0.58	13.77
TA6	0.00	8.26	0.04	8.30	11.53	0.02	8.28
TA7	0.00	2.67	8.72	11.39	9.28	4.36	7.03

Subcat-chment	Subcatchment area (ha)				Catch Slope (%)	Impervious (ha)	Pervious (ha)
	Grass	Forest	Urban	Subtotal			
TB1	0.00	0.00	22.69	22.69	5.12	11.35	11.35
TB2	0.00	0.00	21.32	21.32	9.08	10.66	10.66
TC1	9.34	0.00	0.34	9.68	5.92	0.17	9.51
TC2	15.75	0.00	0.03	15.78	6.43	0.02	15.77
TC3	15.51	0.00	2.47	17.98	7.61	1.24	16.75
TC4	15.04	0.00	0.00	15.04	10.12	0.00	15.04
TD1	0.81	0.00	11.04	11.85	6.52	5.52	6.33
TD2	17.83	0.00	7.42	25.25	5.25	3.71	21.54
TD3	12.87	0.00	1.92	14.79	6.62	0.96	13.83
TD4	22.95	0.00	0.12	23.07	6.54	0.06	23.01
TD5	17.69	0.00	0.11	17.80	9.41	0.06	17.75
Total	**384.74**	**104.85**	**736.35**	**1225.94**	**-**	**368.17**	**857.77**

For urban areas, 50% imperviousness was assumed. This assumption was based upon site inspection and a review of previous studies. The pervious urban areas (mainly backyards) were then lumped together with grass and forest areas to form the total pervious areas for the subcatchments. The total impervious area in the Elliot Lake catchment was thus estimated as 30% of the total catchment area. In Reference 3, the imperious area for Tongarra Creek catchment was estimated to be 26% of the total area.

5.3 Fully Developed or Final Catchment Condition

The catchment had almost reached its fully developed potential under April 2001 conditions except parts of the catchment upstream of Village Green Basin on Tongarra Creek. In the final catchment condition, these areas are considered to be developed in the same manner as the rest of the developed regions of the catchment. The areas are highlighted in Figure 4. The subcatchment description where the final catchment condition varies from that of April 2001, is provided in Table 5.

Table 5: Elliot Lake – Little Lake Catchment Details Fully Developed or Final Catchment Condition

Subcat chment	Subcatchment area (ha)				Catch Slope (%)	Impervious (ha)	Pervious (ha)
	Grass	Forest	Urban	Subtotal			
T9	15.52	0.00	4.77	20.29	3.72	2.39	17.91
T13	2.87	0.00	31.57	34.44	4.89	15.79	18.66
T14	1.11	0.00	11.87	12.98	5.05	5.94	7.05
T15	2.86	0.00	20.08	22.94	6.63	10.04	12.90
TC1	0.25	0.00	9.43	9.68	5.92	4.72	4.97
TC2	0.26	0.00	15.52	15.78	6.43	7.76	8.02
TC3	2.36	0.00	15.62	17.98	7.61	7.81	10.17
TC4	1.36	0.00	13.75	15.11	10.12	6.88	8.24

Subcatchment	Subcatchment area (ha)				Catch Slope (%)	Impervious (ha)	Pervious (ha)
	Grass	Forest	Urban	Subtotal			
TD1	0.61	0.00	11.20	11.81	6.52	5.60	6.21
TD2	11.37	0.00	12.88	24.25	5.25	6.44	17.81
TD3	10.06	0.00	4.44	14.50	6.62	2.22	12.28

In the developed/final catchment condition, the area upstream of Shellharbour Road on Tongarra Creek is envisaged as wetland, which has a freshwater and a saltwater component. A horseshoe embankment laid out upstream of Shellharbour Bridge opening separates the two components. A preliminary layout of the embankment is shown in Figure 1.

After discussion with the Council, the following geometric design was adopted for the present flood study:

- Embankment crest level at 2.7m AHD
- Low level weirs for discharge of frequent flows without overtopping the embankment. Two 30m long weirs. Each weir divided into two 15m long sections at 2.0m AHD and 2.1m AHD.

5.4 The Embedded Design Storm Method

The embedded design storm method was first proposed by Phillips et al (Reference 10) to address the 'dry start' issue that caused underestimation of the 1% AEP design flood levels for the Upper Parramatta River and its tributaries.

Forbes Rigby Pty Ltd subsequently carried out another study on the rainfall IFD relationships in which the original embedded design storm method was further developed (Reference 11). The latter study questioned the AR&R (1999) procedure and suggested that the modified embedded design storm method be used in the Illawarra Region.

For the Illawarra Region, the proposed embedded design storm is a combination of a single design rainfall burst for the critical duration of the catchment and a storm envelope with the same AEP and 24-hour duration, both from AR&R. The single burst is embedded into the storm envelope in such a way that the average intensity of the original 24-hour storm is preserved and that the highest intensities in the original storm envelope are replaced. The AEP of the embedded design storm is assumed to be the same as that of the design rainfall burst.

5.4.1 Critique of the Method

The embedded design storm method may achieve good results for catchments where the typical historical storms have a comparatively much longer duration than a single design burst, and where there is a significant amount of surface storage and/or groundwater contribution to surface runoff.

However, this method has its limitations. According to Philips et al (Reference 10), the original intention was to avoid the 'dry start' problem that resulted in over-attenuation of the flood hydrograph in the lower reaches of Parramatta River. In other words, the embedded design storm method can provide a 'hot start' so that the surface runoff generated from an isolated burst can be superimposed on an assumed base flow which is otherwise non-existent. The assumed base flow would partially fill up surface and in-channel storages at the commencement of the embedded design burst.

The 'hot start' strategy indicates that for catchments with an insignificant amount of surface storages and limited groundwater contribution to surface runoff, the embedded design storm method may not be applicable.

In the proposed method, the design burst replaces the highest intensity burst in the longer duration storm. For catchments with short critical durations, the location of the burst can significantly affect the peak discharge. There is a lack of evidence in both References 10 & 11 to justify this replacement. Furthermore, the temporal patterns of the historic storms shown in Reference 10 seem to be entirely different from the temporal pattern of the proposed embedded storm shown. It can be argued that if the design burst is embedded at different time locations, the design hydrographs provided in Reference 10 would be different. Therefore, the embedded design storm method introduces uncertainties for the method itself. The burst replacement can only be justified, if there is large historic data set to support it.

The example of Elliot Lake - Little Lake catchment presented in Reference 10, shows that the ratio of predicted peak discharge from the embedded storm to that predicted by the extracted burst is in the range of 1.14 ~ 1.26. Given the nature of hydrological data and the errors involved in hydrological models, such a difference is within the error range of hydrological analysis.

Apart from the above issues, the joint probability issue that is not addressed in Reference 10 is also of concern. The assumption that the design burst has the same AEP as the 24-hour storm envelope is a topic worthy of further investigation.

For the Elliot Lake - Little Lake catchment, there is no significant surface storage, except for the lake itself located in the lower reaches of both creeks. Even the effect of the lake on surface runoff is limited, as the design storms are assumed to be coincident with high ocean water levels. The storage deficit of the lake will be satisfied by the high tailwater level rather than the 'hot start'.

References 10 and 11 are included in Appendix C.

5.4.2 Recommended Approach

The approach to embed design storm bursts in a longer duration storm has some merit in catchments where storage is significant. The approach, which effectively prevents a 'dry' start of the model for design storms, is useful for catchments where there are significant man-made storages and/or in-bank channel storages.

In the Elliot Lake – Little Lake catchment, there are two storage basins on Tongarra Creek, both of which have a small storage capacity. These basins are not likely to have any significant impact on the results if model runs are carried out with a 'dry' start. The in-bank channel storages are important in the lower reaches of the catchment (east of Shellharbour Road). The design ocean levels used in this study are such that the in-bank storage of these channels would be partially taken up by the tidal flow at the start of the design storm. Thus the impact of a dry start in the hydrological model is taken care of in the hydraulic modelling.

The results presented in References 10 & 11 were based upon limited data. Further investigations would need to be carried out on a larger data set to establish the embedded design storm approach. This approach is, therefore, not recommended for use in the current study.

Rainfall IFD relationships for all design events for the Elliot Lake - Little Lake catchment were therefore derived from AR&R Volume 2 (Reference 12).

5.5 Establishment Of Hydrological Model

Runoff hydrographs for the flood study were estimated using the RAFTS (Reference 13) rainfall-runoff modelling package. The sub-catchment layout as used in the RAFTS model is shown in Figure 4.

The RAFTS sub-catchments were established based on the contour information. Using the RAFTS utility, each sub-catchment was further divided to account for different initial/continuing rainfall loss rates for pervious/impervious areas of the urban parts of the catchment i.e. a split catchment modelling approach was adopted.

Important parameters used in the development of the RAFTS model are given below in Table 6 and the full list of parameters used in the RAFTS model are included in Appendix D.

Table 6: RAFTS Model Parameters

RAFTS parameter	Forest	Pervious (grass & backyard)	Urban Impervious Area
Manning's 'n' for subcatchments	0.04	0.02	0.01
Storage delay parameter, B	1.0	1.0	1.0
Hydrograph Routing Lag	Based on a flow velocity of 1 and 2 m/s in upper reaches of the catchment, and 0.5 m/s for the rest		

5.6 Review of Previous Studies

Rose Consulting Group carried out a study (Reference 3) for the Flinders catchment (a part of the Tongarra Creek catchment) in which a RAFTS model was set up to investigate the performance of the detention drainage system in the catchment. Since there was no historical data for model calibration, the model was evaluated using physical characteristics. The Rational method was used to verify the peak discharge for the 1% AEP event at three locations.

Forbes Rigby carried out a series of flood studies for different purposes in which three hydrological models, namely PSxRM, WBNM and RAFTS, were used (References 2, 4, 5, 6, 7, and 14). The results of these models for 1% AEP peak discharges and results of the rational method at various locations are summarised in Table 7. The critical duration in the previous studies was reported as 2 hours. It is worthwhile to note that for some locations, different peak discharge values were reported in various reports. For example, the peak discharge of the 1% AEP flood at Elliot Lake - Little Lake entrance was reported as 137.7 m^3/s (Reference 14), 148 m^3/s (Reference 5) using PSxRM, and 150/180 m^3/s using RAFTS respectively (Reference 11).

Table 7: 1% AEP Peak Discharges from Previous Studies (m^3/s)

Location	Forbes Rigby			Rose Consulting (RAFTS)	This Study
	RAFTS	PSxRM	WBNM		
Little Lake Entrance	150/180 *	148	149	-	154
Bensons Ck Georges Ave	-	86	76	-	96
Tongarra Ck Barrack Ave	-	62	66	-	51
Tongarra Ck Shellharbour Rd	-	120	97	69**	61
Bensons Ck Little Lake confluence	-	123	-	-	107
Bensons Ck Shellharbour Rd	-	97	-	-	94
Oakley Ck Shellharbour Rd	-	15	-	-	39
Bensons Ck Davidson St	-	42	-	-	51

Note: * These two numbers were derived from Figure 5 (Reference 11). They correspond to a 2-hour burst (150 m^3/s) and a 24-hour embedded design storm (180 m^3/s) respectively.
** Corresponding to post development conditions with detention basins

5.7 Model Calibration

There are no flow gauges in the study area (Section 3.6) and hence the hydrological model could not be calibrated directly. A combined hydrology/hydraulics approach was adopted, where the hydraulic model was calibrated with input from the hydrology model, thus indirectly validating the results of the hydrology model. The May 1983 and March 1975 storm events were used for this purpose.

The results of the hydrological model were further validated by comparison with the previous studies. Table 7 provides a comparison of flows at different locations in the catchment for the 1% AEP 2 hour storm.

5.8 Historical Storm Event

As discussed in Section 3, the May 1983 and March 1975 storm events were deemed suitable for calibration purposes. The rainfall data for these events was obtained from Reference 1. The data was recorded at Port Kembla pluviometer. An analysis of the rainfall data is provided in Table 8. The mass curves for the two events are shown in Figures 5 and 6.

Table 8: May 1983 and March 1975 Events - Rainfall Data (mm)

Duration	May 1983	March 1975	20% AEP	10% AEP	5% AEP
1 hr	52	53	56	64	74
2 hr	96	71	73	83	97
3 hr	111	90	85	97	113
6 hr	127	150	110	127	147
9 hr	136	167	129	148	172
12 hr	138	174	143	164	192

In addition, daily rainfall data was also sourced from the Bureau of Meteorology for a number of gauges in the area. These gauges are shown in Figure 2. The data was utilised to define rainfall isohyets for the catchment. The isohyetal map for the May 1983 storm is presented in Figure 7 and for March 1975 event in Figure 8.

The May 1983 storm intensity varied uniformly across the catchment with 130 mm near the beach to 100 mm at the upper parts of the catchment. An average value of 115 mm was assumed over the entire catchment for modelling purposes.

Similarly for the March 1975 event, an average value of 250 mm was assumed.

The temporal pattern of the rainfall event as recorded at Port Kembla pluviometer was assumed to be applicable to the Elliot Lake – Little Lake catchment.

The pluviometer data used in the calibration of the hydrological model is presented in Appendix B.

5.9 Design Rainfalls

Owing to the small area of the catchment, uniform areal distribution of design storms has been assumed in the hydrologic analysis. Design rainfall depths and temporal patterns for the 1%, 2%, 5%, 10%, 20% and 50% AEP events were developed using standard techniques provided in AR&R. Design storm rainfall intensities for the full range of storm frequencies and durations are presented in Table 9.

Table 9: Design Rainfall Intensities (mm/h)

Frequency Duration	50%	20%	10%	5%	2%	1%	PMP	
							A	B
15 min	86	110	124	142	166	184	630	540
30 min	62	80	90	104	122	136	450	400
45 min	49.8	65	74	86	101	113	380	340
1h	42.5	56	64	74	87	98	330	300
1.5h	33.2	43.6	49.7	58	69	77	290	260

Frequency Duration	50%	20%	10%	5%	2%	1%	PMP	
							A	B
2h	27.8	36.5	41.7	48.4	58	64	250	230
3h	21.5	28.4	32.4	37.7	44.7	50	200	180
6h	13.9	18.4	21.1	24.5	29.1	32.6	140	120
9h	10.8	14.3	16.4	19.1	22.7	25.4	*	*
12h	9	11.9	13.7	16	19	21.3	*	*
18h	6.85	9.26	10.7	12.6	15.1	17.1	*	*
24h	5.64	7.72	9	10.6	12.8	14.6	*	*
36h	4.24	5.92	6.97	8.31	10.1	11.5	*	*
48h	3.44	4.86	5.77	6.92	8.49	9.72	*	*
72h	2.5	3.61	4.33	5.24	6.5	7.5	*	*

*Not calculated.

The Probable Maximum Precipitation (PMP) was estimated using the Generalised Short Duration Method (Reference 18) recommended by the Bureau of Meteorology. In Table 09 above, two rainfall intensities have been presented. The two values correspond to isohyets A and B defined for spatial distribution of PMP (Reference 17). Spatial distribution of isohyets A and B for the catchment is shown in Figure 9.

5.10 Design Flows

The design rainfall estimates were applied to the hydrologic model in order to predict design runoff hydrographs. The rainfall losses were adopted in accordance with the AR&R guidelines and are provided in Table 10 below.

Table 10: Design Rainfall Losses Used in RAFTS

Catchment Type	Initial Loss (mm)	Continuing Loss (mm/hr)
Forest	30	3.0
Rural – Grass	20	1.5
Urban – Pervious	10	1.0
Urban – Impervious	1.0	0.0

For PMF estimates, an initial rainfall loss of 1.0 mm and a continuing loss of 0.0 mm/hr were adopted. These extremely low values have been adopted as per the recommendation of AR&R.

The initial and continuing loss values used in the hydrological assessment of the catchment should not be used for local flood studies. They are catchment wide values that were determined in the model calibration process and account for broad scale catchment parameters and are not suitable for local studies. Council guidelines for initial and continuing losses should be referenced when undertaking a local flood study.

The design flows were obtained for the 30min, 1hr, 2hr, 3hr, 6hr, 9hr, 12hr, 18hr, 24hr, 36hr and 48hr storm events. The flows for 2 hr duration at the lake entrance are provided in Table 11 below:

Table 11: Peak Design Flows and Rainfall Intensities for 2 Hour Storm

	Storm Event						
	1%	**2%**	**5%**	**10%**	**20%**	**50%**	**PMF**
Rainfall Intensity (mm/h)	64	58	48.4	41.7	36.5	27.8	A=250, B=230
Peak Flow at Lake Entrance (m^3/s)	154	136	115	98.7	85.2	62.4	586

The design hydrographs for the above storms are provided in Appendix E.

6. OCEAN WATER LEVELS

6.1 Coastal Processes Overview

6.1.1 Waves

Wave propagation to the outlet of the catchment at the shoreline may affect nearshore and waterway entrance water levels, as well as causing runup and shoreline erosion during a storm. Wave breaking at the shoreline leads to a phenomenon called wave set-up which increases still water levels above storm tide levels (astronomical tide + wind set-up + inverse barometer effect). In this study wave set-up at an estuarine entrance is an important water level component, but normally smaller than the storm tide. These complex nearshore wave processes are described in Appendix F.

6.1.2 Water Levels

Water level variations at the coastline result from one or more of the following natural causes:

- Tides
- Wind Set-up and the Inverse Barometer Effect
- Wave Set-up
- Wave Runup
- Fresh Water Flow
- Eustatic and Tectonic Changes
- Tsunamis
- Greenhouse Effect, and
- Global Changes in Meteorological Conditions.

These water level components are described in detail in Appendix F.

6.1.3 Summary of Processes Affecting Little Lake Entrance

Ocean water levels will often affect discharge through the entrance, especially at high tide. During very severe ocean storms elevated ocean water levels occur at the shoreline. MHL (Reference 15) describes the extremal analysis of historical peak water levels from the Sydney (Fort Denison) tide gauge. These levels include astronomical tide, the inverse barometer effect and wind set-up, as well as ocean basin processes such as the El Nino phenomenon. Ocean levels at selected average recurrence intervals (ARI) are presented in Table 12. These levels are also applicable for the open ocean region adjacent to Little Lake because Sydney and Port Kembla tides are very similar and on a statistical basis, ocean storms are similar.

Table 12: Peak Storm Ocean Levels at Sydney (Fort Denison Data 1914-1991)

Average Recurrence Interval (years)	Ocean Water Level (mAHD)
20	1.38
50	1.42
100	1.45

These elevated ocean level events are very unlikely to occur during severe fresh water flood events of short duration in small coastal catchments, but form a component of potential ocean inundation levels near the entrance to Little Lake. That is, near the entrance, inundation may be caused by ocean storms and/or fresh water flooding.

Measured Sydney water level data has also been used to describe the probability of exceedence of astronomical tide plus meteorological effects – storm tide (Appendix G).

For this study combined astronomical tide, inverse barometer and wave set-up were computed to provide ocean boundary water level data for the calibration event (May, 1983). For this event wind speeds were low and have been found previously (Reference 16) to provide only a small water level increment. Only if winds are very high (>30m/s) and there are extensive, shallow nearshore regions will there be any significant wind setup.

For design events, ocean water levels affect the driving head available for discharge through the entrance. The following logic was adopted for the design ocean level specification.

The duration of flood events in Little Lake is in the order of two to four hours, and peak ocean boundary levels, which are dominated by the astronomical tide, have a similar duration around peak water level. Therefore extreme ocean storm tides are not likely to occur at the same time as urban floods of this type. Ocean storms may cause elevated ocean levels for periods of up to four days. Nevertheless, the normal astronomical tide will cause high and low water levels and the likelihood of a flood event occurring at the same time as peak ocean high water level is small.

Recent analyses of long term rainfall and measured water levels at Newcastle (Hunter River) have shown virtually no correlation. The most likely ocean water level occurring jointly with a local flood event is mean sea level (MSL), with some minor hint of increased elevation caused by low atmospheric pressure. Therefore, one could adopt 0m AHD as the most likely or expected ocean level. However, it is recommended that a more responsible risk-based approach is to specify the ocean boundary water level to be that level which is only equalled or exceeded for 1% of the time. This level is 1.0m AHD in the Sydney region, (see Appendix G), where datum AHD is 0.9m above tide datum (LAT). This approach has been developed for Newcastle City Council, adopted for flooding at Newport Beach and is suitable for this site also. The level excludes wave set-up. It should be noted that this approach is not appropriate for major river floods, but is suitable for short duration floods in urban catchments.

6.2 Ocean Water Levels

6.2.1 Wave Climate Investigation

In order to investigate the importance of wave propagation and wave set-up at the ocean entrance of the lake, a detailed investigation of wave propagation to the nearshore entrance area was undertaken. In this analysis it was necessary to apply a wave propagation model to transfer the offshore wave climate parameters to the nearshore area of the lake entrance. The investigations are described in Appendices G and H.

6.2.2 Calibration Event

The wave coefficients described in Appendix H were used to transform the time series offshore wave data (H_s, T_z and direction) to the nearshore location for the May, 1983 event. As measured wave direction data was not available, synoptic charts were used to estimate offshore wave direction. For each time step, T_z and direction were used to determine a wave coefficient, (including interpolation), to determine the appropriate offshore to inshore wave transfer relationship. Assuming linearity, Equation G1 was then used to calculate the equivalent inshore wave height.

A surf zone model based on the work of Goda (Reference 8) was then used to calculate wave set-up. Set-up was calculated in a water depth that included tide and estimated entrance bed level. A seabed slope of 1:10 was used which is considered realistic on a beach/ocean entrance undergoing storm erosion, where back-beach sand is being transported offshore. Further landward there would be a steep erosion escarpment on the beach dune face, but that feature would not affect wave set-up at the entrance. It is not possible to know actual bed levels at the time and also, general average scour levels in the vicinity of the outlet dominate the wave set-up, as opposed to the small scale detail, which can not be confidently described. Therefore, this assumption is realistic.

The inverse barometer effects on water level were estimated by using barometric data for the event.

Wind set-up was very small because wind speeds were low during the storm and the nearshore seabed is relatively deep close to the shoreline.

A time series of combined water level were determined at three hour intervals to provide the downstream boundary water level time series for the numerical hydraulic model.

The largest offshore waves (H_s) for the calibration event reached only 2.6m. The largest met-ocean water level increment, 0.27m for the assumed entrance depth, also occurred at that time.

6.2.3 Design Events

For design events, the offshore wave climate described in Appendix H was transferred to the near shore location using the weighted average wave coefficient (0.79), see Appendix G. The surf zone model was then used to calculate wave set-up in a water depth of 1m, assuming an entrance bed level of 0m AHD.

For the PMF flood an offshore H_s of 11m with T_p of 15 seconds was adopted. Ocean level results are presented in Table 13.

Table 13: Ocean Boundary Water Levels Adopted for Design Flood Modelling
(This Level Includes Prospective Greenhouse Mean Sea Level Rise)

Flood Event (%AEP)	Offshore Wave Height (H_s-m)	Design Ocean Water Level (mAHD)
PMF	11	2.1
1	10.5	2.0
2	9.6	1.8
5	9.2	1.6
10	8.3	1.5
20	7.3	1.4
50	6.4	1.3

The design ocean water levels associated with each AEP flood event are considered to be conservative. They are based on a storm tide water level of 1.0m AHD (level exceeded for only 1% for the time), plus 0.2m Greenhouse increment in MSL plus wave set-up. These ocean levels have been specifically derived for use in the flood study, where freshwater flooding is the primary concern. Together with the design freshwater flood events, these ocean levels produce a conservative estimate of flooding.

However, these ocean levels adopted (Table 13) are not based on extreme ocean conditions and higher ocean levels may occur under those conditions. Analysis of extreme ocean conditions usually forms a part of storm surge studies or ocean hazard studies and does not fall under the scope of the present study. For similar reasons, other coastal processes such as wave run-up that can increase coastal flooding, are also not considered in the present study.

7. HYDRAULIC MODELLING

The study area for Elliot Lake – Little Lake extends from relatively steep parts of the catchment to the flat foreshore areas of the lake. The flood flows are likely to be contained within channels or defined overland flowpaths in the upper reaches of the study area whereas for areas generally east of Shellharbour Road, the flows are likely to spill over a larger area. This hydraulic behaviour was taken into consideration when deciding on the type of hydraulic model to be used for the study. After consultation with Council and DLWC, it was decided that the upper reaches of the study area would be modelled using a one-dimensional (1D) hydraulic model and the area east of Shellharbour Road would be modelled using a two-dimensional (2D) hydraulic model. The hydraulic modelling package, SOBEK 1D/2D (Reference 18) was used in the study.

7.1 Establishment of Hydraulic Model

SOBEK 1D/2D is a dynamic hydraulic-routing model developed by WL|Delft Hydraulics of the Netherlands, which is used world-wide and has been shown to provide reliable, robust simulation of flood behaviour in urban and rural areas through a vast number of applications. The model allows addition of a 2D domain to a 1D network with the two components dynamically coupled and solved simultaneously using the robust Delft Scheme. The unique solution scheme is capable of handling steep fronts, wetting and drying processes and subcritical and supercritical flow. The wide variety of hydraulic structures which the model can handle (weirs, roads, levees, culverts, bridges etc) makes it a flexible and adaptable hydraulic analysis tool.

Another important feature of the model is the ability to model the hydraulic structures in the 1D component rather than in the 2D domain. The benefit of this approach is that structure hydraulics is modelled more precisely than the approximate representation possible in a 2D domain.

The channels were described as typical one-dimensional branches with cross-sections defining the channel geometry. Once the channel capacity is exceeded, flow is able to spill into the two-dimensional overland flow grid. During the flood recession, flow is also able to drain from the overland areas back into the defined channels.

7.1.1 1D Model Set-up

For the 1D component of the hydraulic model, the branch layout was developed after a detailed site visit and thorough review of reports of historical floods and available mapping. The physical lie of the land, in addition to hydraulic controls such as roads and embankments were also taken into account. The 1D model branch layout is shown in Figure 10.

The location of cross sections in the model was determined by field inspection. Cross sections were located to be perpendicular to defined flow paths. The floodway cross sections were located so that flow controls on the floodplain could be modelled satisfactorily, with cross sections spaced to adequately represent variations in the drainage network of the floodplain. The location of cross sections in the model is shown in Figure 11. The model cross sections are provided in Appendix A.

All the major creeks/channels in the study area were modelled including the streets, which provide active flowpaths in the area. There are no known active flowpaths through any residential properties.

The detention basins upstream of Shellharbour and Wattle Road on Tongarra Creek were incorporated in the model. Further upstream, the Village Green Basin was also included in the model.

7.2 2D Model Set-up

The major component of a two-dimensional model set-up is the model grid or topography. The model topography was developed from a detailed DTM of the study area, which was obtained through aerial survey. The DTM consisted of feature strings and points to define the major terrain features of the area to be modelled. The accuracy of the DTM is +/- 0.1m (95% confidence limits). A 10m grid (dm1) was generated for the model.

Subsequent to the presentation of the Draft Elliot Lake – Little Lake Flood Study, the hydraulic model was updated for another study undertaken for Council in Memorial Park, Warilla. The update involved the addition of a 2m grid (dm2) covering Memorial Park. Additional ground survey which covered the entire park was provided by Council. The creeks flowing through Memorial Park were represented within the 1D component of the model while the 2D component accounted for the interaction between the numerous flow paths converging within Memorial Park.

The two model grid specifications are given in Table 14 below.

Table 14: Two-dimensional Grid Parameters

Grid	Grid Parameter	Dimension
dm1	Origin *	286698, 1174982
	Grid Size	2 m
	X-dimension (east-west)	450 m
	Y-dimension (north-south)	510 m
	Rotation	0.0 deg (relative to GN)
dm2	Origin *	286925, 1173565
	Grid Size	10 m
	X-dimension (east-west)	1300 m
	Y-dimension (north-south)	2000 m
	Rotation	0.0 deg (relative to GN)

* ISG Coordinate System

Figures 12A and 12B show the model topography for the 10m and 2m grid respectively. For the area east of Shellharbour Road the topography was sampled from the DTM on a 10m grid. The grid resolution was deemed appropriate as the creeks were not represented in the 2D domain, which are sub-grid scale, and were represented by the 1D component of the hydraulic model.

The buildings in the 2D area are likely to be flooded during major floods and therefore provide temporary storage for the floodwaters. Since, in general, no active flowpath exists through the buildings, the grid cells representing the building were modelled as high roughness areas as opposed to removing the areas occupied by the buildings from the 2D domain. This practice of applying high roughness is generally used when flood waters are expected to be temporarily stored within the buildings during the flood events.

7.3 Hydraulic Roughness

The hydraulic roughness for the 2D component of the model was defined using a two-dimensional roughness map of Manning's "n" values. The map was developed by using the land-use zones as described in the Council LEP and from digital orthophotos. The data was processed within a GIS package. Figures 13A and 13B show the hydraulic roughness map for the 10m and 2m grid respectively. Table 15 below summarises the classification of roughness with land use.

Table 15: 2D Grid Roughness Classification

Land Use	Hydraulic Roughness (Manning's "n")
Roads	0.03
Grassed/open parks	0.03
Caravan Parks	0.05
Residential Housing Areas	0.10
Large Buildings	0.50

7.4 Model Boundaries

The model boundaries were located at the model extremities. The upstream boundaries were defined as discharge boundaries, which were applied to the 1D branches of the model, whereas the downstream boundary was defined at the ocean near the lake entrance.

The upstream boundary discharge hydrographs were obtained from the RAFTS model. In addition, inflow hydrographs were also applied at various locations along the 1D branches of the model.

The downstream boundary was represented by ocean water level. The approach adopted for the development of historic and design flood events is described in Section 6. Table 13 provides ocean water levels, which have been used in the modelling of various design events.

7.5 Model Calibration

The storm events of May, 1983 and March, 1975 were selected for calibration and validation purposes. The flow hydrographs for these events were obtained from the RAFTS modelling.

Recorded flood level data used in the calibration was obtained from Reference 1. A number of flood levels were available for the May 1983 event. The levels were obtained through resident interviews. In addition, the then Water Board (now Sydney Water Corporation) recorded a number of flood levels along Tongarra Creek between Shellharbour Road and Barrack Avenue Bridge. Only a few levels were available for March 1975 event. Further details of calibration levels are given in Section 3.

As the May 1983 event provided the maximum historic flood level data, it was used for calibration of the model. The March 1975 event was used for validation.

A model topography was developed which represented the best estimate of the floodplain features at the time of the flood. Reference 1 lists the improvements carried out in the catchment in the 1960's and 1970's to reduce flooding. These improvements included:

- construction of rock groyne at the entrance
- dredging of creek beds
- raising of George Street, Beverley Avenue and Shellharbour Road (Tongarra Creek) bridges
- concrete lining of Bensons Creek beside Woodford Avenue
- levee bank construction beside Bass Street, and
- residential development constructed on filled land above known historical flood levels.

These modifications to the floodplain were identified from interviews consolidated as part of the Little Lake Flood Study Compendium of data (1991). There is no survey data available for ground conditions prior to any of the above modifications and hence these modifications could not be incorporated in the model.

This study identified two other modifications, which include construction of the Junction/Village Green/Landcom Basin and modification to Wattle Street (previously Tongarra Road) Bridge. The data for the previous Wattle Street Bridge was available in Reference 1. These two modifications were therefore incorporated in the model.

In the calibration process, the model parameters such as channel roughness were adjusted and a match was obtained between the recorded and modelled flood levels. Downstream boundary of the model was based on the ocean water levels for the historic events as discussed in Section 6. The results of calibration are presented in Table 16. The location of recorded flood levels along with the recorded and modelled flood levels is shown in Figures 14 and 15 for the May 1983 and March 1975 events respectively.

Table 16: Calibration - May 1983 Event

Location of Historic Flood Level	Branch	Historic Flood Level (mAHD)	Modelled Flood Level (mAHD)	Difference (m)
38 Woodford Ave Warilla	Bensons	2.70	2.58	-0.12
40 Woodford Ave Warilla	Bensons	2.77	2.57	-0.20
16 Joan Ave Warilla	Bensons	2.95	2.54	-0.41
6 Joan Ave Warilla	Bensons	2.53	2.52	-0.01
33 George St Warilla	Bensons	2.63	2.52	-0.11
11 Stephanie Ave Warilla	Bensons	2.67	2.54	-0.13
11 Stephanie Ave Warilla	Bensons	2.55	2.54	-0.01
9 Stephanie Ave Warilla	Bensons	2.41	2.54	0.13
19 Terry Ave Warilla	Bensons	2.21	2.08	-0.13
106 Osborne Pde Warilla	Bensons	2.20	2.12	-0.08
4 Ramond St Warilla	Bensons	2.16	2.13	-0.03
10 Ramond St Warilla	Bensons	2.20	2.14	-0.06
25 Headland Pde Barrack Point	Lake	1.55	1.77	0.22
29 Headland Pde Barrack Point	Lake	1.50	1.75	0.25
50 Jason Ave Barrack Heights	Tongara	1.73	Not flooded 1.78[1]	0.05
54 Jason Ave Barrack Heights	Tongara	1.98	Not flooded 1.78[1]	-0.20
72 Jason Ave Barrack Heights	Tongara	2.11	Not flooded[2] 1.78[1]	-0.33
74 Jason Ave Barrack Heights	Tongara	2.06	Not flooded[2] 1.78[1]	-0.28
56 Bass St Barrack Heights	Tongara	1.30[3]	2.5	1.20
Surfrider Caravan Park	Tongara	2.30	2.5	0.20
Surfrider Caravan Park - Junction Rd	Tongara	2.25	2.5	0.25
Lake Windemere Caravan Park	Bensons	2.00	2.12	0.12

[1] Nearest flood level in the lake
[2] Likely flooding from a local creek
[3] The reported flood level across the creek in Surfrider Caravan Park is ~2.3 mAHD

In addition, the then Water Board (now Sydney Water Corporation) also collected flood level data along Tongarra Creek between Shellharbour Bridge and Barrack Avenue Bridge. A comparison of recorded and modelled flood levels is shown in Figure 16.

The results of the calibration are discussed in Section 12.

The calibrated model was then validated using the March 1975 event. Catchment conditions were assumed similar to May 1983 event. Historic storm data was applied to the RAFTS hydrology model and inflow hydrographs were obtained for the hydraulic model. The hydraulic model parameters were kept the same as those used in the May 1983 model. An appropriate ocean water level boundary was used. The result of validation is provided in Table 17.

Table 17: Validation – March 1975 Event

Location of Historic Flood Level	Branch	Historic Flood Level (mAHD)	Modelled Flood Level (mAHD)	Difference (m)
Jason Ave. Warilla	Bensons	1.63	1.7	0.07
Shellharbour Rd Shellharbour	Tongarra	2.84	2.42	-0.42
3 Stephanie Ave. Warilla	Bensons	2.41	2.32	-0.09
68 Benaud Crescent Warilla	Bensons Tributary	6.4	-[1]	-
25 Susan Ave Warilla	Bensons	3.13	2.46	-0.67
19 Susan Ave Warilla	Bensons	3.14	2.46	-0.68
Surfrider Caravan Park	Tongarra	2.55	2.32	-0.23

[1] Most likely a local flooding issue as nearby houses did not report any flooding (Reference 1)

8. DESIGN FLOOD ESTIMATION

8.1 General

Design flooding behaviour was estimated for two development stages of the catchment - April 2001 and fully developed/final catchment conditions. The two development conditions are described in Table 18 below:

Table 18: Catchment Development Conditions

April 2001 Catchment Conditions	Fully Developed/Final Catchment Conditions
All urban areas fully developed except parts of the catchment upstream of Village Green Basin on Tongarra Creek	All urban areas fully developed including the development upstream of the Village Green Basin
Wetland/detention basin upstream of Shellharbour Road in a natural state	Wetland/detention basin modified to accommodate freshwater/saltwater wetland, sports fields developed along the wetland

Design inflow hydrographs, obtained from the RAFTS model, were applied to the hydraulic model, which represented the floodplain and catchment conditions for the two development stages of the catchment. A range of hydrographs with different storm durations was applied to the model in order to estimate the critical storm duration for different areas in the floodplain.

Based on the results of the coastal processes analysis (Section 6), a set of design water levels was developed for use as a downstream boundary in the hydraulic model as a downstream boundary. Table 13 (Section 6) presents the downstream boundary levels used in the design flood event modelling. Due to the short critical duration of the catchment flooding, a constant ocean water level is used as downstream boundary in the model.

The use of constant ocean water level as a model boundary ensures that the modelling results represent the envelope of water levels for both fresh water flooding and salt water flooding from the design ocean water levels.

8.2 Results

The model results for all duration design storms were compiled into a single result list for each AEP for the fully developed/final catchment condition. The reported results are the envelope of results for all durations for each AEP (Appendix J).

Model results for predicted flood behaviour at significant locations in the floodplain are summarised below. Table 19 contains design flood level information at various locations in the flood-affected areas.

The results are also provided as longitudinal profiles in Figures 17 to 20 of the major channel/drain and main flow paths in the floodplain. The flood profiles are provided for:

1. Tongarra Creek
2. Bensons Creek
3. Tributary of Bensons Creek
4. Oakley Creek near Sunset Ave.

The results presented in Table 19 below and Figures 17 to 20 represent the peak flood level at each location when all storm durations have been considered. Therefore the flood profiles represent the peak water level envelope.

Table 19: Summary of Peak Design Flood Levels
Fully Developed/Final Catchment Condition

Location	Branch	Water Level (mAHD)						
		1%	2%	5%	10%	20%	50%	PMF
Junction Basin	Tongarra	13.20	12.82	12.40	11.98	11.63	10.94	16.03
U/S of Wattle Road	Tongarra	7.83	7.64	7.39	7.16	6.98	6.58	9.12
U/S of Shellharbour Road	Tongarra	3.22	3.09	2.92	2.76	2.63	2.37	4.38
Shellharbour Sewage Treatment Works	Tongarra	3.16	2.99	2.80	2.64	2.52	2.29	4.23
Surfrider Caravan Park	Tongarra	3.16	2.99	2.79	2.64	2.51	2.27	4.21
U/S of Barrack Ave. Bridge	Tongarra	3.13	2.95	2.74	2.57	2.44	2.21	4.16
U/S of Madigan Blvd	Bensons	18.45	18.38	18.30	18.20	18.10	17.46	19.21
U/S of Landy Drive	Bensons	13.35	13.24	13.15	13.06	12.98	12.71	14.13
Andrew Cr. Cul-de-sac	Bensons	11.49	11.44	11.36	11.30	11.23	11.09	12.07
U/S of Shellharbour Road	Bensons	4.96	4.90	4.80	4.60	4.46	4.19	5.54
U/S of Beverley Ave.	Bensons	3.72	3.66	3.57	3.47	3.35	3.05	4.39
Joan Ave. Pedestrian Bridge	Bensons	2.93	2.85	2.77	2.69	2.83	2.56	3.99
U/S of George St.	Bensons	2.90	2.82	2.73	2.63	2.56	2.35	3.98
Raymond St.	Bensons	2.57	2.45	2.32	2.22	2.15	2.09	3.92
Lake Windemere Caravan Park	Bensons	2.50	2.40	2.29	2.21	2.15	2.06	3.91
U/S of Kingsway	Bensons Tributary	13.21	13.13	13.04	12.93	12.84	12.50	14.23
U/S of Leawarra Ave.	Bensons Tributary	10.39	10.33	10.26	10.18	10.10	9.81	11.23
U/S of Lake Entrance Rd.	Bensons Tributary	6.12	6.05	5.90	5.65	5.45	5.00	6.49
Tenpin Bowl, Sunset Ave.	Oakley Creek	3.68	3.54	3.39	3.20	3.03	2.58	4.82
Intersection of Sunset Ave. and Shellharbour Rd.	Oakley Creek	2.45	2.32	2.19	2.11	2.08	2.02	3.91
Bowling Club	Lake	2.28	2.11	1.94	1.84	1.76	1.59	3.85
Lake Entrance	Lake	2.10	2.01	1.61	1.51	1.41	1.31	2.24

U/S - Upstream

A summary of model results for the fully developed/final catchment conditions is provided in Appendix J. The results include peak water level, flow, velocity and critical duration for all the model cross-sections. It should be noted that the flow velocities are average values for the whole channel section. These values vary across the channel section due to localised effects.

Flood extents and flood contours for the fully developed/final catchment conditions are provided for 1%, 5% and 20% AEP floods and PMF in Figures 21 through 24. These figures represent the maximum extent for each AEP for the fully developed/final catchment condition (see section 8.1 for a description of the fully developed/final catchment condition). It is to be noted that the extents have been based on the surveyed information where available. Elsewhere 2m LIC contour data has been used. The above figures should not be used for the assessment of flood extent on individual properties.

The model results for the April 2001 catchment conditions (see section 8.1 for description of this condition) were also processed and compared with the results for fully developed/final catchment conditions. The comparison is provided in Appendix K. The results for both the catchment conditions are for equivalent locations within the floodplain. Changes to the floodplain between the April 2001 condition and the fully developed/final condition result in changes to the cross sections used to represent the creek system. The change has been significant in the area between Shellharbour Road and Wattle Road on Tongarra Creek, where sports fields and wetland with a horse-shoe embankment has recently been constructed.

9. SENSITIVITY ANALYSIS

Hydraulic model sensitivity was tested to demonstrate the range of uncertainty in the model results for the 1% and 50% AEP 2-hour design events. The following model parameters were tested for sensitivity:

- Catchment runoff - increased/decreased by 20%
- Channel roughness - increased/decreased by 20%
- Downstream boundary - increased/decreased by 20%
- Lake entrance scour - as per the 1991 survey (Reference 1)
- Culvert blockage - 100% (see section 9.1)
- Time varying downstream boundary (see section 9.2).

The fully developed/final catchment condition was assessed for sensitivity analysis, except for the time varying downstream boundary where the April 2001 catchment condition was assessed.

The sensitivity results are presented in Figures 25 to 28 for the 1D components of the model. The sensitivity results for the 2D area of the model are presented in Table 20 and 21 below:

Table 20: Model Sensitivity for the 2D Component of Hydraulic Model 1% AEP Event

Location	1% AEP – change in peak water level						
	Flow		DS Boundary		Roughness		Scour
	+20%	-20%	+20%	-20%	+20%	-20%	
D/S Joan Ave - Bensons Creek	0.10	-0.10	0.06	-0.02	0.07	-0.14	0.00
U/S George St - Bensons Creek	0.10	-0.11	0.06	-0.02	0.03	-0.05	0.00
D/S George St - Bensons Creek	0.11	-0.13	0.09	-0.03	0.05	-0.12	-0.01
Sparta St - Bensons Creek	0.09	-0.10	0.19	-0.08	0.03	-0.04	-0.02
Confluence of Bensons and tributary	0.09	-0.10	0.22	-0.10	0.03	-0.03	-0.02
Confluence of Bensons and Tongarra Creek	0.08	-0.08	0.28	-0.17	0.02	-0.02	-0.05
Lake Entrance	0.00	0.00	0.41	-0.01	0.00	-0.01	0.00
D/S Sunset Avenue	0.16	-0.19	0.05	-0.04	0.05	-0.06	0.00
Outlet Terry Ave Drain	0.09	-0.10	0.19	-0.08	0.03	-0.04	-0.02
D/S Barrack Avenue	0.08	-0.08	0.16	-0.08	0.01	-0.02	-0.02
U/S Barrack Avenue	0.16	-0.16	0.13	-0.07	-0.04	0.03	-0.01
U/S of Confluence on Tongarra Creek	0.09	-0.08	0.27	-0.16	0.02	-0.02	-0.05
D/S Shellharbour Rd - Bensons Creek	0.12	-0.15	0.01	0.00	0.04	-1.05	0.00

D/S Downstream
U/S Upstream

Table 21: Model Sensitivity for the 2D Component of Hydraulic Model 50% AEP Event

Location	50% AEP – change in peak water level						
	Flow		DS Boundary		Roughness		Scour
	+20%	-20%	+20%	-20%	+20%	-20%	
D/S Joan Ave - Bensons Creek	0.03	-0.16	0.01	-0.01	0.05	-0.21	0.00
U/S George St - Bensons Creek	0.04	-0.18	0.02	-0.01	0.03	-0.13	0.00
D/S George St - Bensons Creek	0.03	-0.14	0.02	-0.01	0.04	-0.15	0.00
Sparta St - Bensons Creek	0.04	-1.92	0.05	-0.09	0.00	0.03	-0.02
Confluence of Bensons and tributary	0.04	-0.09	0.08	-0.05	0.00	0.01	-0.03
Confluence of Bensons and Tongarra Creek	0.05	-0.07	0.17	-0.11	0.02	-0.01	-0.08
Lake Entrance	0.00	0.00	0.26	-0.25	0.00	0.00	0.00
D/S Sunset Avenue	0.11	-0.06	0.02	-0.01	0.03	-0.02	0.00
Outlet Terry Ave Drain	0.04	-1.92	0.05	-0.04	0.00	0.03	-0.01
D/S Barrack Avenue	0.13	-0.15	0.17	-0.13	0.00	0.00	-0.02
U/S Barrack Avenue	0.15	-0.18	0.17	-0.14	-0.01	0.01	-0.02
U/S of Confluence on Tongarra Creek	0.05	-0.08	0.17	-0.12	0.02	-0.02	-0.08
D/S Shellharbour Rd - Bensons Creek	0.02	-0.09	0.00	0.00	0.13	-0.06	0.00

D/S Downstream
U/S Upstream

There is no historical data available for the lake entrance scour. The survey carried out in 1991 (Reference 1) showed a more scoured entrance than the survey carried out for this study. Hence 1991 data was adopted as representative of scoured entrance.

Detailed results of the sensitivity analysis for flow, downstream boundary, roughness and entrance scour are provided in Appendix L. The sensitivity analysis presented in Appendix L was carried out for the April 2001 catchment condition.

9.1 Culvert Blockage

Culvert blockage can cause significant rise in flood levels and can result in inundation of areas, which would otherwise be unaffected. However, the degree of blockage for various culverts is uncertain. Recent studies in Wollongong catchments have indicated certain blockage intensities for various culvert sizes. Based on this information, a 100% culvert blockage was assumed for the sensitivity testing of the Elliot Lake – Little Lake catchment.

Culvert blockage can not be predicted at any given culvert for a given storm, as there are many factors that contribute to a culvert blocking. In addition to the unpredictability of a single culvert blocking, the combination of culverts blocking is also unpredictable. Studies in the Wollongong area have shown that blocked culvert combinations can lead to significantly higher water levels than if each culvert were to block individually. Given all permutations of culvert blockage possible in the Elliot Lake – Little Lake catchment it is not possible to model each combination of culvert blockage. After discussion with the Council a risk/consequence-based approach was adopted whereby individual culverts were blocked. The following culverts were modelled as blocked for the PMF 1h design storm (fully developed/final catchment condition) and the 1% AEP 2h design storm (fully developed/final catchment condition):

1. Bensons Tributary at Lake Entrance Road
2. Bensons Creek at Shellharbour Road
3. Bensons Creek at George Street
4. Tongarra Creek at Wattle Street
5. Tongarra Creek at Shellharbour Road
6. Tongarra Creek at Barrack Avenue.

Table 22 provides the difference in peak water levels due to culvert blockage for the 1% AEP 2 hour design event and the PMF 1 hour event.

Table 22: Impact of Culvert Blockage

Location of Blocked Culverts	PMF 1h - Fully Developed/ Final Catchment Condition			1% AEP 2h - Fully Developed/Final Catchment Condition		
	Culverts Open (mAHD)	Culvert Blocked (mAHD)	Difference (m)	Culverts Open (mAHD)	Culvert Blocked (mAHD)	Difference (m)
Lake Entrance Road	6.44	6.76	0.32	6.12	6.51	0.38
Shellharbour Road Bensons Creek	5.15	5.38	0.23	4.60	4.93	0.34
George Street	3.67	3.69	0.02	2.89	3.05	0.16
Wattle Street	9.10	9.17	0.07	7.83	8.53	0.70
Shellharbour Road Tongarra Creek	4.14	4.21	0.07	3.07	3.37	0.31
Barrack Avenue	3.86	4.00	0.14	2.83	3.30	0.47

The blockage impact of the individual culverts is shown as a profile in Figures 29A and 29B. It is to be noted that impact of individual culverts is combined in a single profile for presentation purposes. The profile therefore is an envelope of blockage impact at individual culverts.

9.2 Influence of Time Varying Downstream Boundary

For design flood estimation, a constant ocean water level was used as the downstream boundary of the hydraulic model. In addition to carrying out the sensitivity analysis for variations in constant ocean level, the impact of a time varying boundary was also investigated.

The downstream boundary for the shorter duration storms (4-6 hours) can be assumed constant without any significant impact on the peak flood levels. However, as discussed in Section 5 above, there is a need to consider longer duration design storms, for which a constant water level boundary may not be an appropriate assumption. On the other hand, if a time varying boundary is to be used, it introduces the issues regarding phasing of the flood peak with the ocean water level i.e. what would be the ocean water level when the flood peak arrives.

To determine the relative impact of a time varying boundary and also to consider various phasing scenarios, a number of model runs were carried out for a 1% AEP 2 hour (April 2001 Catchment condition) event using a 1D hydraulic model. The following ocean water level conditions were modelled:

1. Ocean water level peak coincides with the flood peak
2. Ocean water level peak occurs earlier than the flood peak
3. Ocean water level peak occurs later than the flood peak
4. Constant ocean water level.

The results of the ocean water-level-timing sensitivity-analysis modelling are presented in Appendix M.

10. PROVISIONAL FLOOD HAZARD

10.1 General

Flood hazard can be defined as the risk to life and limb and damage caused by a flood in a floodplain. The hazard caused by a flood varies both in time and place across the floodplain. The Floodplain Development Manual (Reference 19) describes various factors to be considered in determining the degree of hazard. These factors are:

1. Size of the flood
2. Depth and velocity of floodwaters
3. Effective warning time
4. Flood awareness
5. Rate of rise of floodwaters
6. Duration of flooding
7. Evacuation problems
8. Access.

Hazard categorisation based on all the above factors is part of establishing a Floodplain Risk Management Plan. The scope of the present study calls for determination of provisional flood hazards only, which when considered in conjunction with the above listed factors provides comprehensive analysis of the flood hazard.

10.2 Provisional Flood Hazard

Provisional flood hazard is determined through a relationship developed between the depth and velocity of floodwaters (Reference 19). The Floodplain Development Manual defines two categories for provisional hazard - High and Low.

The model results were processed using an in-house developed program, which utilises the model results of flood level and velocity to determine hazard. Provisional flood hazard for the final or fully developed catchment condition for the 1%, 5% and 20% AEP floods and PMF are presented in Figures 30 to 33. The corresponding flood extents are also provided. The area enclosed within the hazard extent represents high hazard area. Elsewhere it is low hazard up to the flood extent. The provisional hazard is based on the envelope of the hazard calculation at each location. Hazard calculations are undertaken for each discrete time step for each duration for all AEP's presented.

For the 1D component of the model, the hazard is defined only at each of the model cross-sections. Between cross-sections, the hazard has been interpolated. The hazard definition beyond the cross-section limits is solely based on the depth criteria for hazard. Figure 33b shows a typical cross section along with hazard definitions.

11. HYDRAULIC CATEGORISATION

11.1 General

Hydraulic categorisation of the floodplain is used in the development of the Floodplain Risk Management Plan. The Floodplain Development Manual defines flood affected land to fall into one of the following three hydraulic categories:

- **Floodway -** Areas that convey a significant portion of the flow. These are areas that, even if partially blocked, would cause a significant increase in flood levels or a significant redistribution of flood flows, which may adversely affect other areas.

- **Flood Storage -** Areas that are important in the temporary storage of the floodwater during the passage of the flood. If the area is substantially removed by levees or fill it will result in elevated water levels and/or elevated discharges. Flood Storage areas, if completely blocked would cause peak flood levels to increase by 0.1m and/or would cause the peak discharge to increase by more than 10%.

- **Flood Fringe -** Remaining area of flood prone land, after Floodway and Flood Storage areas have been defined. Blockage or filling of this area will not have any significant affect on the flood pattern or flood levels.

11.2 Hydraulic Category Identification

Floodways were determined for the 1%, 5% and 20% AEP and PMF by considering those model branches that conveyed a significant portion of the total flow. These branches, if blocked or removed, would cause a significant redistribution of the flow. The criteria used to define the floodways are described below.

As a minimum, the floodway was assumed to follow the creekline from bank to bank. In addition, the following depth and velocity criteria was used to define a floodway:

- Velocity * Depth must be greater than 0.25 m^2/s **and** velocity must be greater than 0.25 m/s **OR**
- Velocity is greater than 1 m/s

Flood storage was defined as those areas outside the floodway, which if completely filled would cause peak flood levels to increase by 0.1 m and/or would cause peak discharge anywhere to increase by more than 10%. This criteria was applied to the model results as described below.

In the 1D part of the model, it was assumed that if the cross-sectional area is reduced such that 10% of the conveyance is lost, the criteria for flood storage would be satisfied. To determine the limits of 10% conveyance in a cross-section, the depth was determined at which 10% of the flow was conveyed. This depth, averaged over several cross-sections, was found to be 0.2 m. Thus the criteria used to determine the flood storage is:

- Depth greater than 0.2m, and
- Not classified as floodway.

The same criteria was used to define the flood storage in the 2D part of the model.

All areas that were not categorised as floodway or flood storage, but still fell within the flood extent are represented as flood Fringe.

The hydraulic categories for the fully developed/final catchment condition for the 1%, 5%, and 20% AEP and PMF are provided as plans in Figures 34 to 37. The hydraulic categories are based on the envelope of the hydraulic categorisation at each location. The hydraulic categorisation was undertaken for each discrete time step for all duration for the AEP's presented. Figure 33b shows a typical cross section with hydraulic categories mapped.

12. DISCUSSION OF RESULTS

12.1 Model Set-up and Calibration

Flooding behaviour of the Elliot Lake – Little Lake catchment was investigated using hydrologic and hydraulic models. The flooding in the study area was modelled using a dynamically linked one-dimensional/two-dimensional hydraulic model, SOBEK 1D/2D. The one-dimensional component of the model represented the drainage channels in the steeper parts of the catchment west of Shellharbour Road, whereas the two-dimensional component represented the flatter areas east of Shellharbour Road including the lake foreshore areas.

Inflows to the hydraulic model were developed using the RAFTS hydrologic model. The output from RAFTS was compared with the results of previous studies. The comparison shows similar peak flows and supports the results from the RAFTS model.

The hydraulic model was calibrated to the May 1983 event and validated using the March 1975 event. The calibration results (Table 16) indicate a reasonable match of historic and modelled flood levels. However, there are a few locations where there are differences. Some of the factors likely to affect the calibration results are:

1. The aerial survey is accurate to +/- 0.1m. The modelled flood levels can approximately vary by the same amount.
2. The state of catchment development was unknown at the time of flooding events. Hydrological analysis was based on the existing state of development for the catchment.
3. Although the Port Kembla rainfall station is close to the catchment, given the history of storm spatial variability in the Illawarra Region (August 1998 and October 1999 are recent examples), the use of a temporal pattern from Port Kembla may have caused an unknown error in the hydrological analysis.
4. There have been a number of modifications to the catchment as reported in Reference 1. Except accounting for the Junction Basin on Tongarra Creek and Wattle Street Bridge, the hydraulic model was based on existing catchment features and topography, as historic data was not available. In addition, there is anecdotal evidence that the bridge at Barracks Avenue has also been modified. Lack of bridge details for the historic events will have an unknown error in the calibration results.
5. The lake entrance bathymetry at the time of historic events was unknown.
6. The quality of some of the historic data may be doubtful. All flood level data was obtained from Resident Interviews which were undertaken a significant time after the historic events. Therefore the flood levels are likely to have some margin of error. As an example, for the May 1983 event, the flood level reported at 56 Bass Street is 1.3 mAHD whereas at the same location across the creek in Surfriders Caravan Park the reported level is 2.3 mAHD. Similar anomalies exist at 16 and 6 Joan Avenue, Warilla and 50 and 54 Jason Avenue Barrack Point.

For the March 1975 event, the above factors would have played a role in the differences between observed and modelled flood levels. The underestimation of the flood levels in Bensons Creek indicate that the creek might have been in a natural state resulting in high roughness and consequently higher modelled flood levels.

If the creek was in a natural state, it is also possible that it would have carried a significant load of debris causing blockage of the culverts/bridges and resulting in higher flood levels. Collection of debris (Reference 1) has been reported for the May 1983 event upstream of George Street Bridge.

Despite some differences in the observed and modelled flood levels, the calibration modelling produced a flood behaviour, which correspond to what has generally been reported in the area. The calibration results were achieved through a reasonable set of model parameters, which are expected for the Elliot Lake – Little Lake catchment.

12.2 Design Flood Behaviour

The 'calibrated' model was used to estimate design flood levels for the April 2001 catchment conditions as well as the fully developed/final catchment and floodplain conditions. The storm durations of 1-3 hours were generally found to be critical. Design flood profiles of peak water levels for the fully developed/final catchment and floodplain condition are presented in Figures 17 to 20. Model results are detailed in Appendix J. Figures 21 to 24 indicate areas likely to be inundated by the PMF, 1%, 5% and 20% AEP storm events for the fully developed/final catchment and floodplain conditions.

The model results for the April 2001 catchment conditions were also processed and comparison with the results for fully developed/final catchment conditions is provided in Appendix K. Changes to the floodplain between the April 2001 condition and the fully developed/final condition resulted in changes to the cross sections used to represent the creek system. The results presented in Appendix K for the two catchment conditions are for equivalent locations within the floodplain. In some locations the change in invert has produced a decrease in the water level. This decrease is due to the lowering of the creek invert rather than any change in the flow regime.

The flood extents have been developed based on the survey information, which is limited to areas near the model cross-sections. The extent definition between model cross-sections is based on 2m LIC contour data, which has a limited accuracy. Therefore the interpretation of the flood extents presented in this study should be treated with caution.

12.3 Bensons Creek

The Bensons Creek catchment is subjected to widespread flooding. The floodwaters are generally contained within the creek reserve in the area between Lake Entrance Road and the location where the creek enters the major pipe drainage near the Andrew Crescent cul-de-sac. The pipe drainage has limited capacity, with the result that the floodwaters spill over into the area around Lindwall Street and Johnston Street. The flow is carried downstream through Spofforth Street, Grimmet Street and O'Neil Street and finally enters the Shellharbour War Memorial Park where it is joined by the Bensons Creek Tributary, a concrete channel draining the suburb of Barrack Heights.

On the Bensons Creek Tributary, the flow is likely to spill over at Leawarra Avenue crossing and carried towards the Lake Entrance Road where part of it flows down the roadway towards Shellharbour Road and part would be diverted into King Street.

Downstream of Shellharbour Road, Bensons Creek is concrete lined and hydraulically steep. The flow downstream of the culverts is supercritical which results in a hydraulic jump upstream of the Beverley Avenue crossing. The hydraulic jump is shown in design flood profiles in Figure 19. The jump is submerged for the 5% AEP and rarer events.

East of Shellharbour Road, the area between George Street and Woodford Avenue would be affected by floods. Near the George Street crossing, the areas around Stephanie Avenue and Joan Avenue would be inundated. Downstream of George Street, the Mount Warrigal nursing home would generally be flood-free, whereas the properties along Osborne Parade, Arcadia Street and Terry Avenue would be affected by floods. Further downstream, the Lake Windemere Caravan Park would be inundated along with the Bowling Club.

The flooding in the lake is controlled by the design ocean water levels and the width of the entrance along Headland Parade. The impact of entrance scour is discussed in section 12.6.4.

12.4 Tongarra Creek

Flooding in Tongarra Creek is modified by the presence of a number of detention basins. The study area starts at the Landcom/Village Green/Junction Basin. The basin routes all floods up to the 1% AEP without overtopping. Downstream of the basin, the creek is relatively steep and the flow is contained in a narrow width. Further downstream, the flow spreads out over the playing fields, which act as a basin upstream of Wattle Road. This basin also routes all floods up to the 1% AEP without overtopping. Downstream of Wattle Road, the flow enters the Myimbar Community Park Reserve where it slows down appreciably. Some areas along Ocean Beach Drive near Shellharbour Road are likely to be affected by major floods.

The Shadforth wetlands which are located on Tongara Creek upstream of Shellharbour Road, have recently undergone some rehabilitation works. A horseshoe shaped embankment upstream of Shellharbour Road prevents salt water from penetrating upstream to the wetland and ensures a minimum fresh water level is maintained within the wetland. The embankment includes a supplementary weir on each side of the embankment which are set at 15 and 25 metres respectively, stepped at 2.0 and 2.1 mAHD. The main weir level is set at 2.7 mAHD. Downstream of Shellharbour Road, Barracks Swamp is completely inundated along with the Sewage Treatment Works. The dunes along Junction Road would prevent any outbreak of floodwaters towards the beach.

The bridge at Barrack Avenue is a major control on Tongarra Creek. For low tides, the flooding upstream of Barrack Avenue is entirely controlled by the bridge.

However, with high tides the bridge control becomes less prominent and flooding can be attributed to a combination of high tide and catchment runoff. The Surfriders Caravan Park and properties along Bass Street would be affected by flooding.

Downstream of Barrack Avenue, properties along Jason Avenue and Headland Parade would be affected by the floods.

12.5 Sunset Avenue

Sunset Avenue is likely to be a major flowpath in the event of flooding. The floodwaters would travel towards the large surcharge pit in the drainage reserve between Sunset Avenue and Shellharbour Road. The flow in the reserve then enters a short concrete channel, which terminates at the Shellharbour Road crossing. The culverts under the Shellharbour Road have a limited capacity with the result that the road will be overtopped even in moderate events such as the 10% AEP.

12.6 Model Sensitivity

The uncertainty in the model results was tested by carrying out a sensitivity analysis. The sensitivity of various parameters needs to be considered carefully because it defines the variability of model results. Sensitivity to catchment runoff, channel roughness, downstream boundary, culvert blockage and lake entrance scour was investigated for the 1% and 50% AEP 2 hour floods for the fully developed/final catchment condition. Blockage sensitivity analysis was undertaken for the fully developed/final catchment condition for the 1% AEP 2hr and PMF 1hr event.

12.6.1 Channel Roughness

Channel roughness was increased by 20%, which had a varying impact on the peak water levels. Detailed results are provided in Appendix L. For the 1% AEP flood the greatest increase is on Bensons Creek upstream of Madigan Boulevard where the flood level increased by 0.20 m. For the 50% AEP the greatest increase occurred in Bensons Creek upstream of Madigan Boulevard where the increase in flood levels was 0.31 m.

The maximum increase in Tongarra Creek occurred at upstream of Wattle Road for the 1% AEP, where the flood level increased by 0.13 m. The maximum flood level increase for the 50% AEP was 0.14 m and occurred upstream of Wattle Road.

12.6.2 Catchment Runoff

Sensitivity of the hydraulic model results to changes in catchment runoff points to the significance of output from the hydrological model. The impact of increased runoff is more widely distributed throughout the study area as compared to the impact of increased roughness. The greatest increase for the 1% AEP was observed in the Village Green/Landcom Basin where the level increases by 0.65 m. Flood level increases of 0.35 m were observed in this location for the 50% AEP but the greatest increase occurred in Bensons Creek upstream of Madigan Boulevard where the level increased by 0.43 m.

On Bensons Creek, between Lake Entrance Road and Landy Drive, the flood levels increase by 0.18 m for the 1% AEP and 0.43 m for the 50% AEP flood.

12.6.3 Downstream Boundary

The downstream boundary of the model is represented by the ocean water level. The design ocean levels used for various floods are provided in Section 6. The impact of an increase in downstream water levels is generally controlled by the tidal limit. In Tongarra Creek upstream of Barrack Avenue Bridge, the water levels increase by 0.10 m for the 1% AEP and 0.16 m for the 50% AEP flood. The influence of the boundary extends to the reserve upstream of Shellharbour Road.

The flood levels in Oakley Creek downstream of Sunset Avenue increase by 0.20 m for the 1% AEP and 0.08 m for the 50% AEP floods. There is no impact on the Bensons Creek flood levels upstream of Shellharbour Road.

12.6.4 Lake Entrance Scour

In significant flood events, the lake entrance is likely to scour and create a wider flow area for release of floodwaters from the lake. In the absence of any detailed historic data, the entrance was scoured to the cross section levels surveyed in Reference 1 in 1991. The entrance was approximately lowered from 0.0 mAHD to –0.5 mAHD as per the available cross section details.

The results show that there is a low impact on flood levels in both Tongarra and Bensons Creeks. In the lake area, the water levels drop by 0.05m for the 1% AEP and 0.07m for the 50% AEP flood.

12.6.5 Culvert Blockage

Incidents of culvert blockage in the catchment during storm events have been reported in the past (Reference 1). However, the blockage is not likely to be excessive as most of the catchment is developed and as such a significant quantity of the blockage material is likely to be anthropogenic in nature rather than from natural sources.

An analysis of culvert blockage was carried out to develop an understanding of the magnitude of the problem and to this end a blockage factor of 100% was used. The blockage factor used in the analysis is supported by observations in the Wollongong catchments.

Various culvert blockage combinations are possible within the catchment. Each blockage combination can have a different effect on the flood behaviour by not only providing a local obstruction to flow but also modifying the flow routing lag along various creeks. After discussions with Council, only limited blockage combinations were investigated.

The greatest increases in water levels are observed upstream of the blocked culverts with corresponding decrease downstream of these structures. The results of the culvert blockage analysis are presented in Table 19, with profiles of the blockage envelope presented in Figure 29A and 29B. The profiles presented in Figures 29A and 29B are the envelopes of the individual blockage analysis runs and the decrease in water level downstream is not apparent as the higher backwater levels for the downstream blocked culvert are presented in the flood profile envelope. Flood levels from lake entrance to the first upstream culverts are controlled by the entrance condition and design ocean water level, and as such the flood levels in this area experience minimal impact due to culvert blockage.

Barrack Avenue is overtopped for the 1% AEP 2 hour event. The large storage capacity of the creek and the associated floodplain upstream of the bridge accommodates a large volume of the floodwaters for the given design event. Further analysis of blockage for longer duration storms where the hydrograph has a greater volume may result in greater overtopping depths at this location.

The results indicate that those culverts that are capable of carrying a large proportion or the total flow in the unblocked state will lead to the greatest increase in water levels when blocked. Those culverts that are already overtopped under unblocked conditions will have a smaller impact on flood levels when blocked. This is shown by the smaller increase in water level at each of the culverts for the PMF as compared to the 1% AEP event, where all culverts are overtopped by the PMF in the unblocked condition.

The culvert blockage analysis carried out for this study is not exhaustive and does not take into account every possible culvert blockage in the catchment. The purpose of the analysis was to provide an indication of the impact of culvert blockage. Thus other culverts in the catchment can block and result in higher flood levels than the unblocked results presented in this report. To indicate the change in flooding extent under culvert blockage conditions, an extent map showing the increase in flooded area have been produced. Figure 42 shows the envelope of the increase in flood extent due to blockage for the 1% AEP 2hr (fully developed/final catchment conditions) and Figure 43 shows the envelope of the increase in flood extent due to blockage for the PMF 1hr (fully developed/final catchment conditions).

The results indicate that the extent of flooding is not modified appreciably for the culverts that have been investigated. However, it is possible that other blocked culverts in the catchment may have more severe impact.

12.6.6 Impact of Time-Varying Ocean Water Level Boundary

As longer duration storms (up to 48 hours) were used in the hydraulic modelling, the impact of time-varying ocean water level boundary was determined. The results are presented in Appendix M for the April 2001 catchment condition 1% 2hr storm run through a fully 1d model. The results indicate that, in general, there is only a small difference between the application of a time varying boundary and a constant water level. The use of a time-varying boundary also raises the question of the phasing of the flood peak with the ocean water levels, which was also investigated. The results suggest that a constant water level boundary would produce the highest flood levels. With the time-varying boundary, the scenario where the ocean levels peak before the flood peak arrives, would produce the highest flood level.

Lawson and Treloar have carried out a joint probability analysis of ocean water levels and catchment rainfall in a separate study (Reference 20). The study found no correlation between the two events for short duration storms.

Since there is no justification nor any standard practice to phase in the ocean water level boundary with the flood peaks for any of the modelled scenarios and the fact that the constant water level boundary did not produce overly conservative flood levels, the use of constant ocean water level boundary was deemed appropriate.

12.7 Embedded Design Storms

There has been some criticism of the use of design storms, as provided in AR&R, in hydrological analysis of the catchments in the Illawarra region. Doubts have been raised as to the accuracy of the flow data obtained for short duration storms, which actually represent a short burst in a longer duration storm. An approach has been suggested where a shorter duration design storm for a given ARI is embedded in a longer duration storm of the same ARI to prevent the ‘dry’ start in hydrological model. The main argument for using the embedded design approach is the assumption that some of the storage volume within the catchment would be taken up before the actual short duration storm burst occurs.

Though the approach has some merit, it is based on very limited data and it is not justified to adopt this approach for the current study. Further investigations based on a larger datasets are required to develop this approach. Further discussion of this matter can be found in section 5.4.

12.8 Hydraulic and Hazard Categorisation

Hydraulic and Hazard categories were defined for the design events of 1%, 5% and 20% AEP together with the PMF. The definition of these categories was based on the guidelines provided in the Floodplain Development Manual (Reference 19).

Figures 30 to 33 provide extents of high and low hazards in the study area. There is more area with high hazard in Tongarra Creek than in the Bensons Creek catchment. In Tongarra Creek, high hazard exists upstream of all the detention basins as well as through Barracks Swamp and the Surfriders Caravan Park.

In the Bensons Creek catchment, War Memorial Park upstream of Shellharbour Road is subject to high hazard. In addition, parts of King Mickey Park are also classified as high hazard.

Sunset Avenue and the adjacent reserve are high hazard areas.

The hydraulic categories are shown in Figures 34 to 37. For the PMF, most of the flooded area is either floodway or flood storage. For the 1% and 5 % AEP floods, large areas around Woodford Avenue, Joan Avenue, Susan Avenue and Stephanie Avenue would act as storage. On Tongarra Creek, large areas between the Shellharbour Road Bridge and the Barrack Avenue Bridge would act as storage.

The impact of culvert blockage is not taken into account in the definition of hydraulic and hazard categorisation.

Draft Flood Risk Precincts as defined by Council have been supplied in Figure 40. This figure comprises of the following extents:

- High flood risk precinct - the 1% AEP high hydraulic hazard plus the 1% AEP floodway extent
- Medium flood risk precinct - the 1% AEP flood extent plus 500 mm freeboard
- Low flood risk precinct - the PMF flood extent.

13. REPORT QUALIFICATIONS

This report has been prepared for Shellharbour City Council to define the nature and extent of flooding for the study area in the Elliot Lake – Little Lake catchment. The report defines the flooding behaviour for the major flow paths in the catchment.

The investigation and modelling procedures adopted for this study follow current best practice and considerable care has been applied to the preparation of the results. However, model set-up and calibration depends on the quality of data available and there will always be some uncertainties. The flow regime and the flow control structures are very complicated and can only be represented by schematised model layouts.

Hence there will be an unknown level of uncertainty in the results and this should be borne in mind in their application.

The results of the study are based on the following assumptions/conditions:-

- The hydraulic model results are based on the survey data and as such the accuracy of the survey data is reflected in the model results.
- Calibration and validation of the model was undertaken using available historic information about the catchment modifications.
- Design flood extents are approximate between 1D cross sections of the model. Where surveyed levels are not available, flood extents are based on the land contour data provided by Council and interpolation of model results.
- Design flood extents in the 2D domain of the model are developed for the average depth and level of the cell (10m x 10m), and as such may vary slightly within the cell.
- The properties adjacent to the modelled flowpaths are not modelled as active flowpaths and are assumed to provide only storage for floodwaters.
- The Elliot Lake – Little Lake Flood Study is a broadscale catchment-wide study. The study results should be used with caution to determine the flood levels for individual properties.

Study results should not be used for purposes other than those for which they were prepared.

14. ACKNOWLEDGMENTS

This study was jointly funded by Shellharbour City Council, the Commonwealth and the NSW State Government under the Floodplain Management Program administered by the NSW Department of Land and Water Conservation.

In compiling this report, Lawson & Treloar Pty Ltd. has been assisted by advice and information from relevant sections of Shellharbour City Council and various other public authorities. Their assistance is gratefully acknowledged.

15. REFERENCES

1. NSW Department of Public Works (1991). "Little Lake Flood Study Compendium of Data; Report No. PWD 91071, ISBN 0730586464, November 1991".

2. Forbes, Rigby and Associates (1989). "A Report on the Impact of Flooding on the Development of Land at Sparta Street, Warilla".

3. Rose Consulting Group (1994). "Flinders Catchment Shellharbour Stormwater Drainage Review and Analysis".

4. Forbes, Rigby & Associates (1993). "A Review of Flooding in Tongarra Creek Between Shellharbour Rd & Barrack Avenue for Mr M Cohen".

5. Forbes Rigby Consulting Engineers (1995). "A Review of Flooding and Development Options at Little Lake for Shellharbour City Council".

6. Forbes, Rigby & Associates (1993). "A Review of Flooding in Bensons Creek Between Shellharbour Rd & Lakes Entrance Road".

7. Forbes, Rigby & Associates (1993). "A Preliminary Opinion on the Impact of Filling on Flooding in Little Lake for Mr D Arap Ali".

8. Goda, Y. (2000): Random Seas and Design of Maritime Structures. World Scientific, Advanced Series on Ocean Engineering – Volume 15.

9. NSW Department of Public Works (1993). "New South Wales Estuary and River Water Levels Annual Summary 1992/93". Manly Hydraulic Laboratory, Report MHL659. PWD Report No 93069, ISBN 0731027523.

10. Phillips, B., Lees, S., Lynch, S (1994). "Embedded Design Storms – An Improved Procedure for Design Flood Level Determination", Proc. Water Down Under' 94, Adelaide, IE Aust.

11. Rigby, E. H., and Bannigan, D. J (1996). "The Embedded Design Storm Concept – A Critical Review". Proc. Hydrology and Water Resources Symposium'96, Hobart, IE Aust.

12. The Institution of Engineers, Australia (Revised Edition 1999) "Australian Rainfall and Runoff - A Guide to Flood Estimation".

13. WP Software (1994). "RAFTS-XP. Runoff Analysis & Flow Training Simulation with XP Graphical Interface. User Manual". Belconnen ACT.

14. Forbes, Rigby & Associates (1995). "A Review of Flooding in Tongarra Creek Between Shellharbour Rd & Barrack Avenue for Lucas & Tait".

15. Manly Hydraulics Laboratory (1992): Mid New South Wales Coastal Region. Tide Storm Surge Analysis. Report MHL621.

16. Lawson & Treloar (1984): Wave Design Criteria – Wamberal and Avoca Beaches. Report Prepared for NSW Public Works Department.

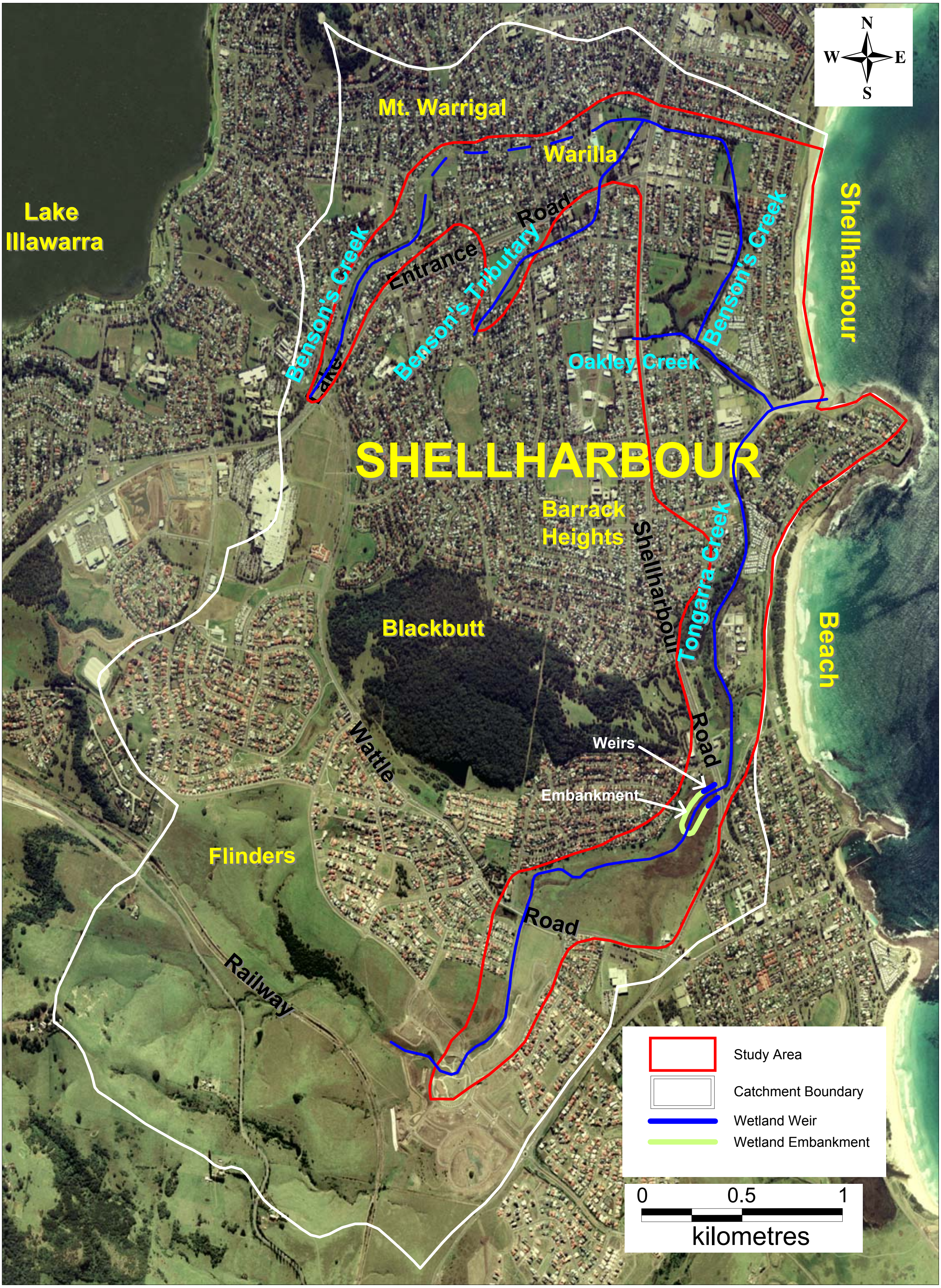
17. Bureau of Meteorology (1994): "The Estimation of Probable Maximum Precipitation in Australia; Generalised Short Duration Method." Australian Government Publishing Service, Canberra.

18. WL|Delft Hydraulics (2002): "SOBEK 1D/2D User Guide and Technical Reference" PO Box 177, 2600 M.H. Delft Netherlands

19. NSW Government (2005): "Floodplain Development Manual" - Management of Flood-Liable Land.

20. Lawson & Treloar Pty Ltd (1999): "Design Water Levels in Newcastle Harbour Joint Possibility Study".

FIGURES



Cardno Lawson Treloar

J1959/R1974/V5
January 2006

Elliot Lake - Little Lake
Flood Study

FIGURE 1
STUDY AREA

J1959/Figures V5/Fig_1_Locality.Wor

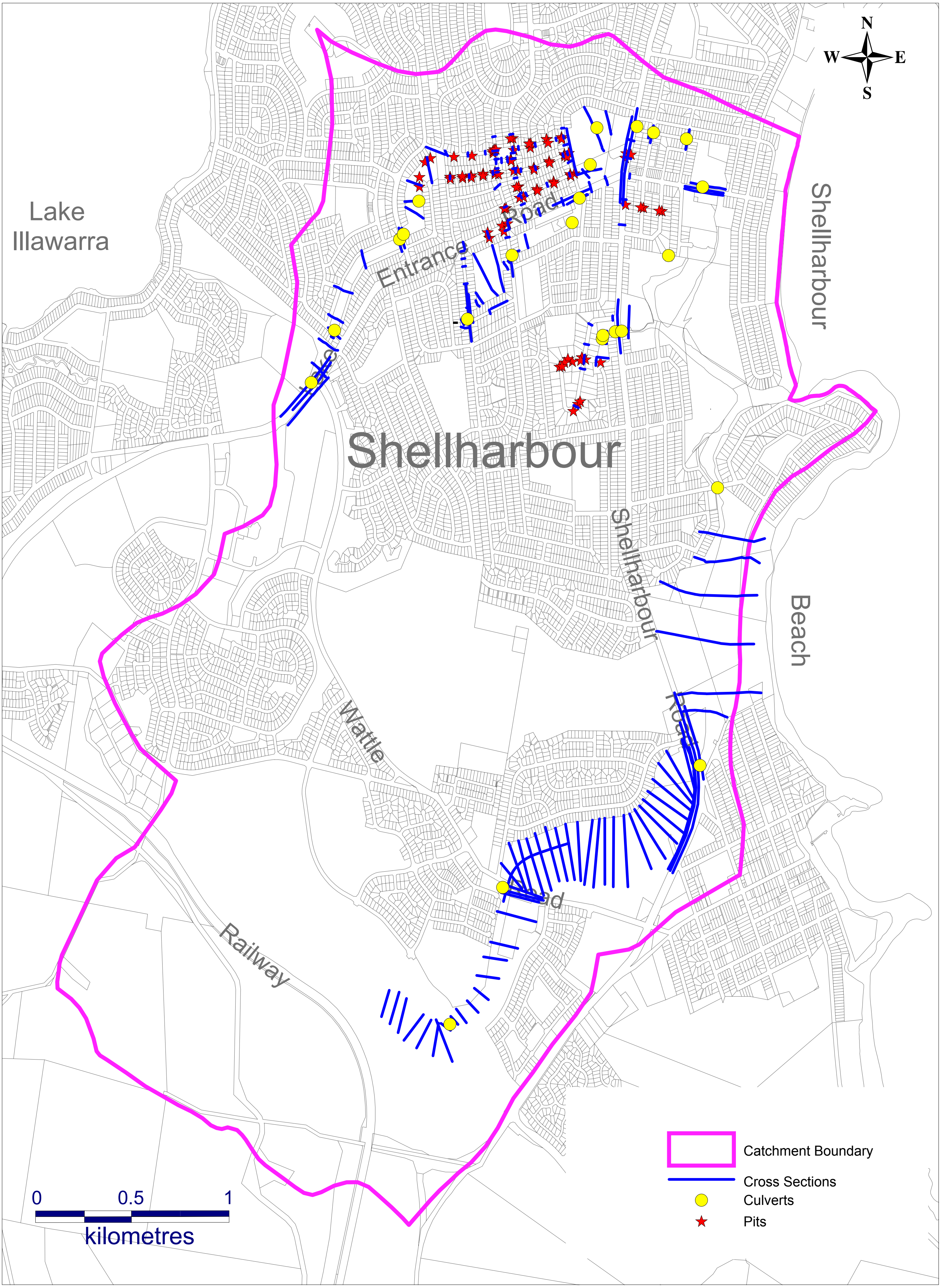
Cardno Lawson Treloar

J1959/R1974/V5
January 2006

Elliot Lake - Little Lake
Flood Study

FIGURE 2
LOCATION OF RAINFALL STATION

J1959/Figures V5/Fig_2_Rainstation.Wor



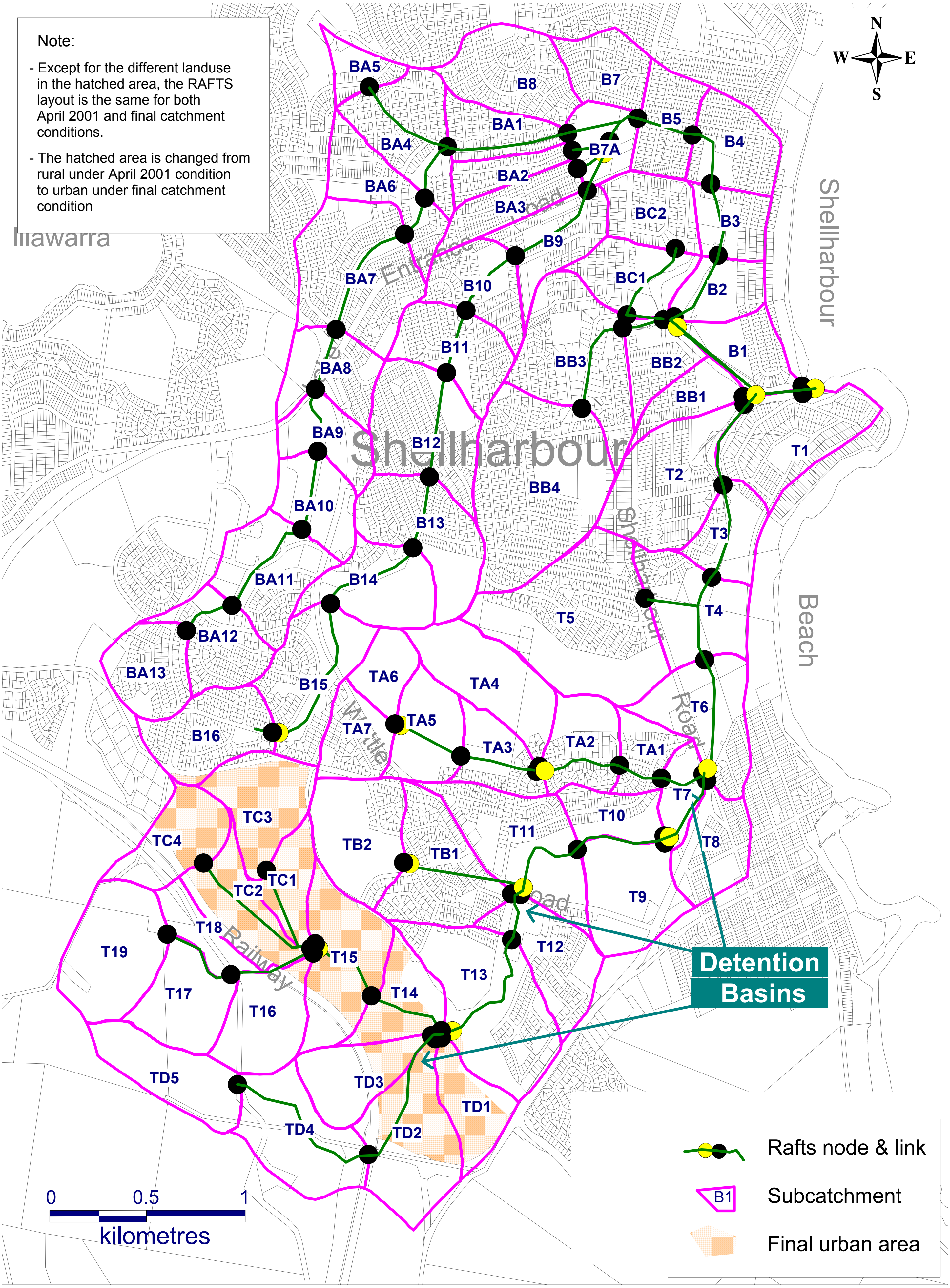
Cardno Lawson Treloar

J1959/R1974/V5
January 2006

Elliot Lake - Little Lake
Flood Study

FIGURE 3
LOCATION OF CROSS SECTIONS, PITS AND CULVERTS

J1959/Figures V5/Fig_3_XSecs_Culverts_Pits.Wor



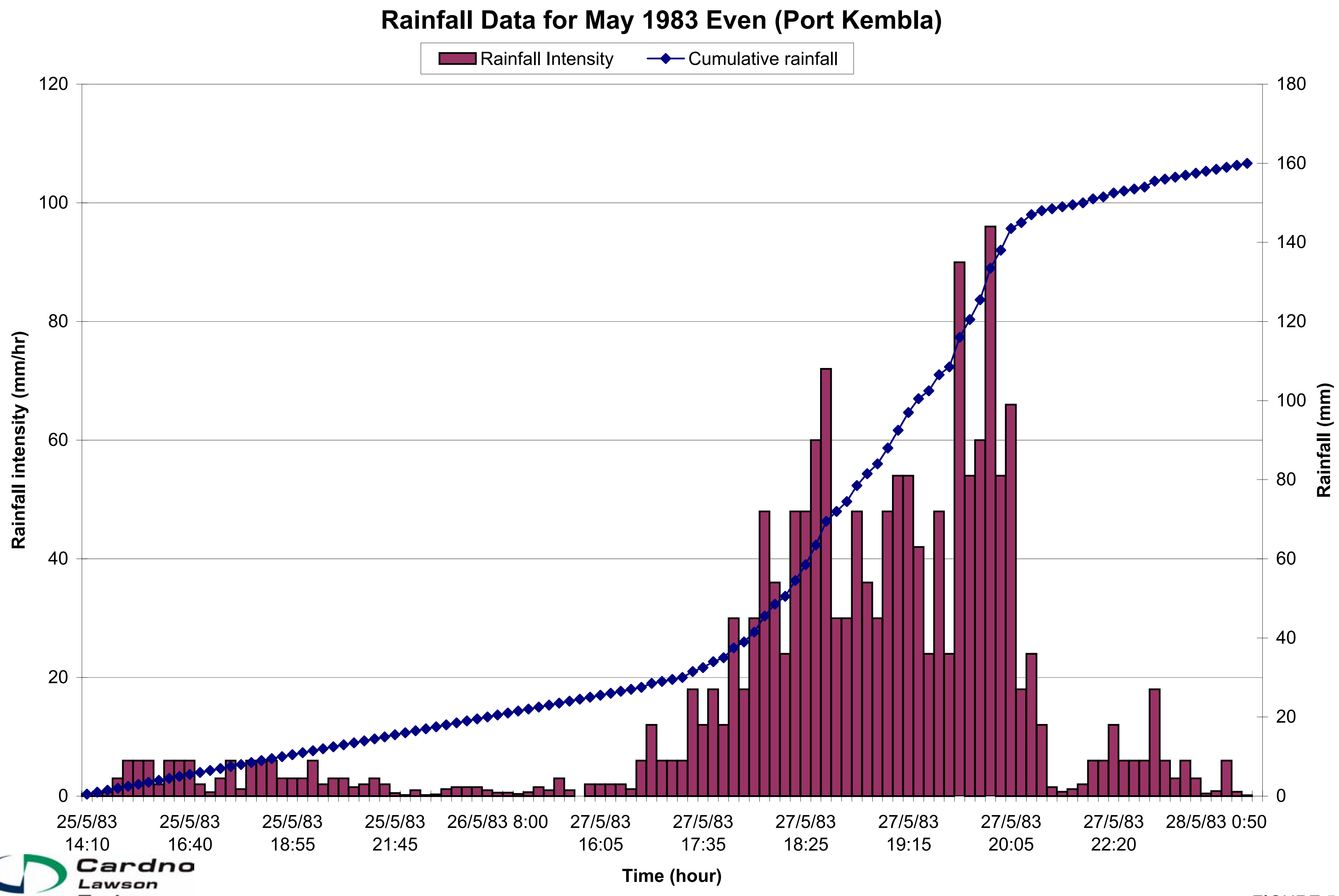
Cardno Lawson Treloar

J1959/R1974/V5
January 2006

Elliot Lake - Little Lake Flood Study

FIGURE 4
RAFTS MODEL SUBCATCHMENT LAYOUT - APRIL 2001 AND FINAL CATCHMENT CONDITIONS

J1959/Figures V5/Fig_4_RAFTS_Layout.Wor

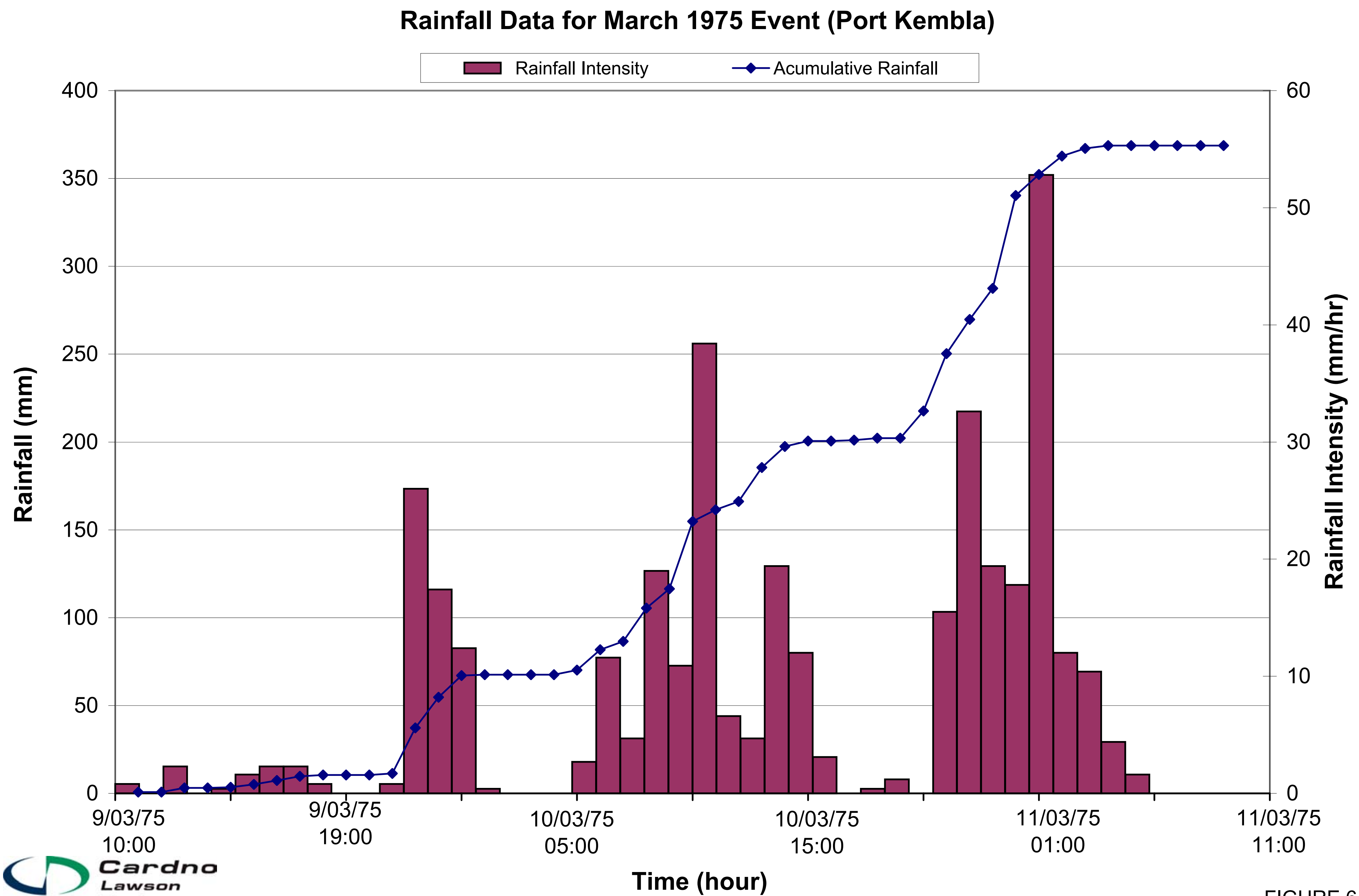


Cardno Lawson Treloar

J1959/R1974/V5
January 2006

Elliot Lake - Little Lake
Flood Study

FIGURE 5
RAINFALL DATA FOR MAY 1983 EVENT
J1959\Figures V5\rainfall-M75-M83

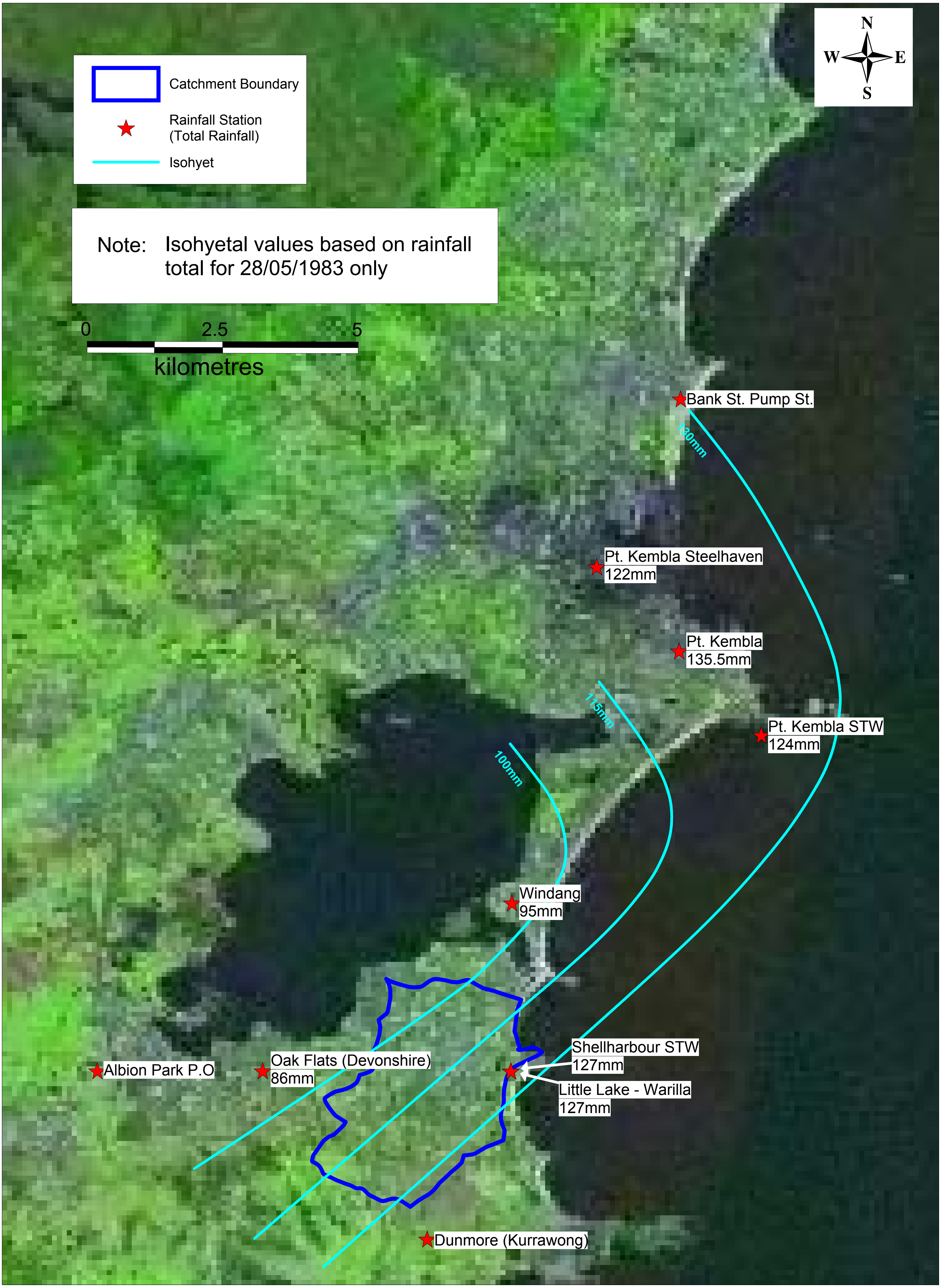


Cardno Lawson Treloar

J1959/R1974/V5
January 2006

Elliot Lake - Little Lake
Flood Study

FIGURE 6
RAINFALL DATA FOR MARCH 1975 EVENT
J1959\Figures V5 rainfall-M75-M83



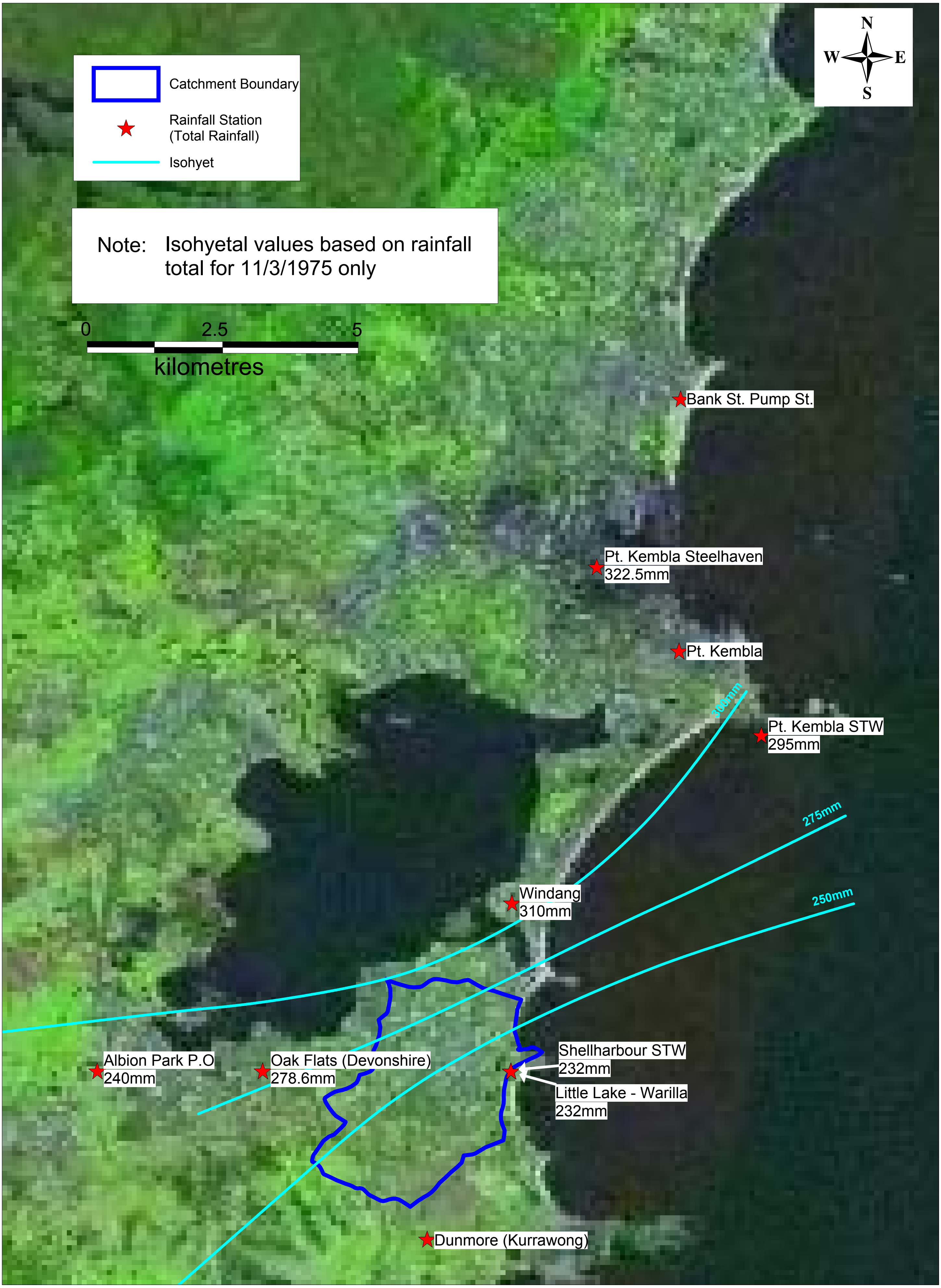
Cardno Lawson Treloar

J1959/R1974/V5
January 2006

Elliot Lake - Little Lake
Flood Study

FIGURE 7
MAY 1983 STORM ISOHYETAL MAP

J1959/Figures V5/Fig_7_May83_Isohyetal.Wor



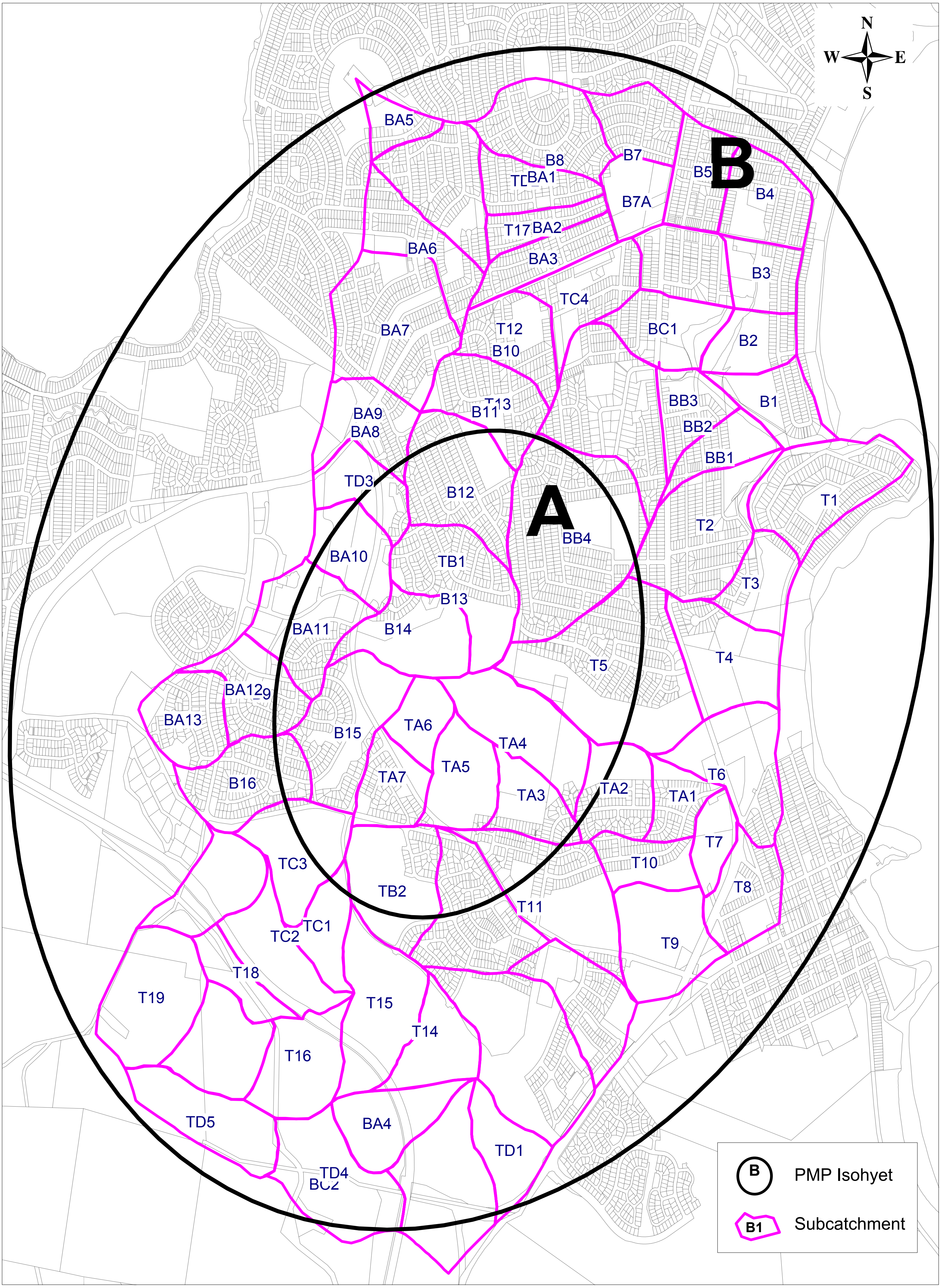
Cardno Lawson Treloar

J1959/R1974/V5
January 2006

Elliot Lake - Little Lake
Flood Study

FIGURE 8
MARCH 1975 STORM ISOHYETAL MAP

J1959/Figures V5/Fig_8_Mar75_Isohyetal.Wor



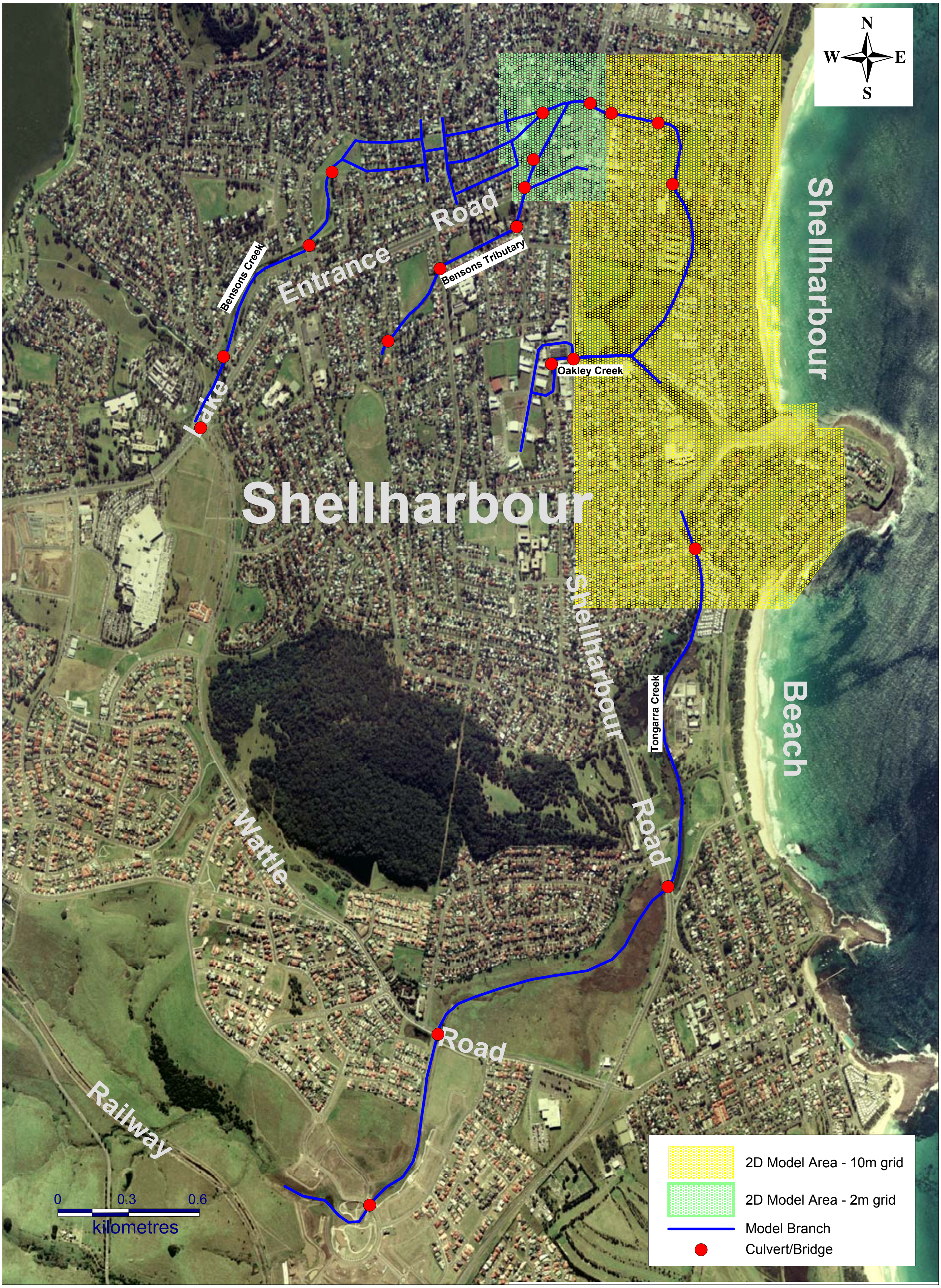
Cardno Lawson Treloar

J1959/R1974/V5
January 2006

Elliot Lake - Little Lake
Flood Study

FIGURE 9
SPATIAL DISTRIBUTION OF PMP

J1959/Figures V5/Fig_9_PMP_Rings.Wor



Cardno Lawson Treloar

J1959/R1974/V5
January 2006

Elliot Lake - Little Lake
Flood Study

FIGURE 10
HYDRAULIC MODEL LAYOUT

J1959/Figures V5/Fig_10_Hydraulic_Layout.Wor

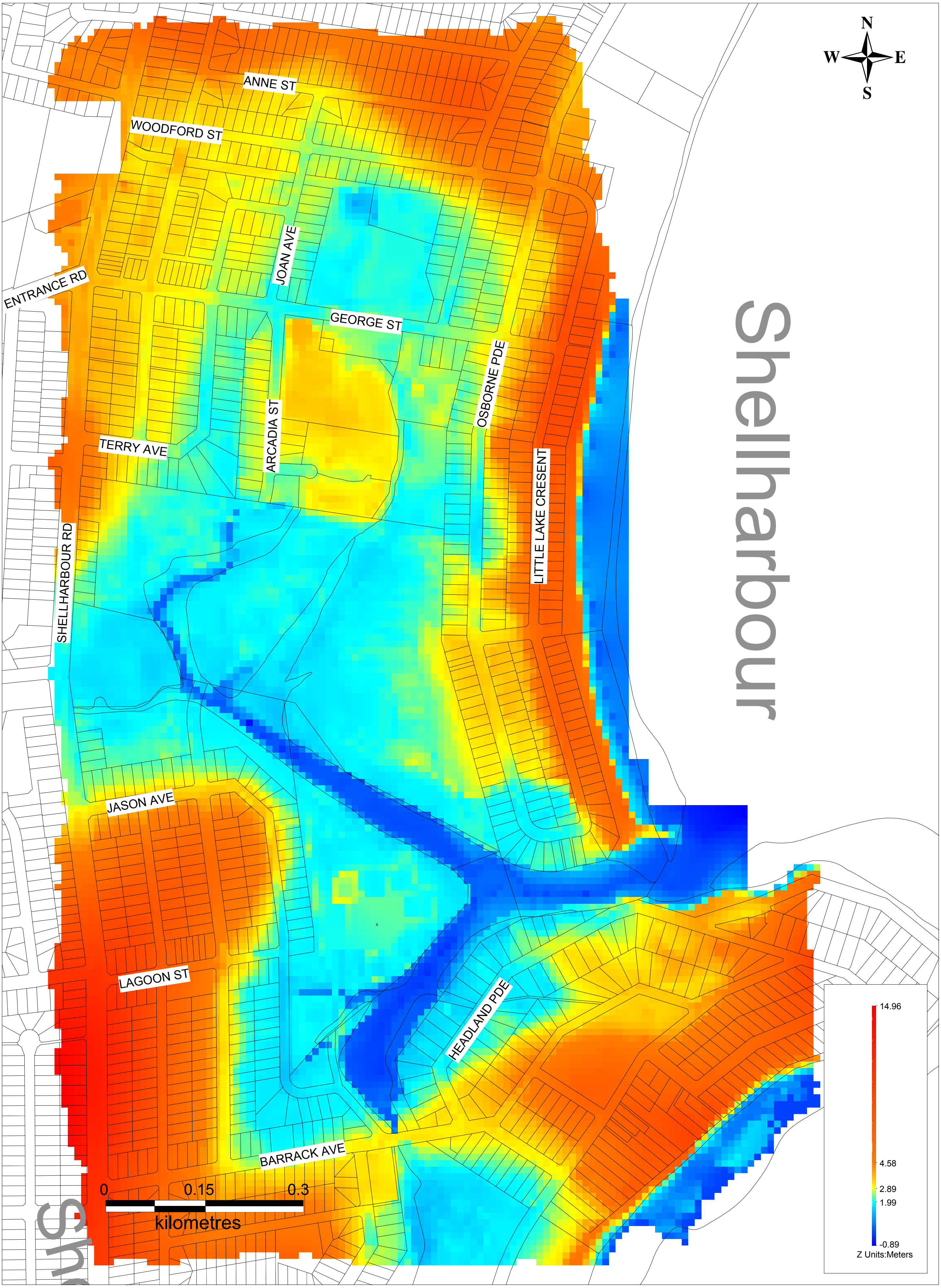
Cardno Lawson Treloar

J1959/ R1974/V4
May 2005

Elliot Lake - Little Lake
Flood Study

FIGURE 11
CREEK CROSS SECTION LAYOUT

J1959/Figures V4/Fig11 All x-sec.Wor



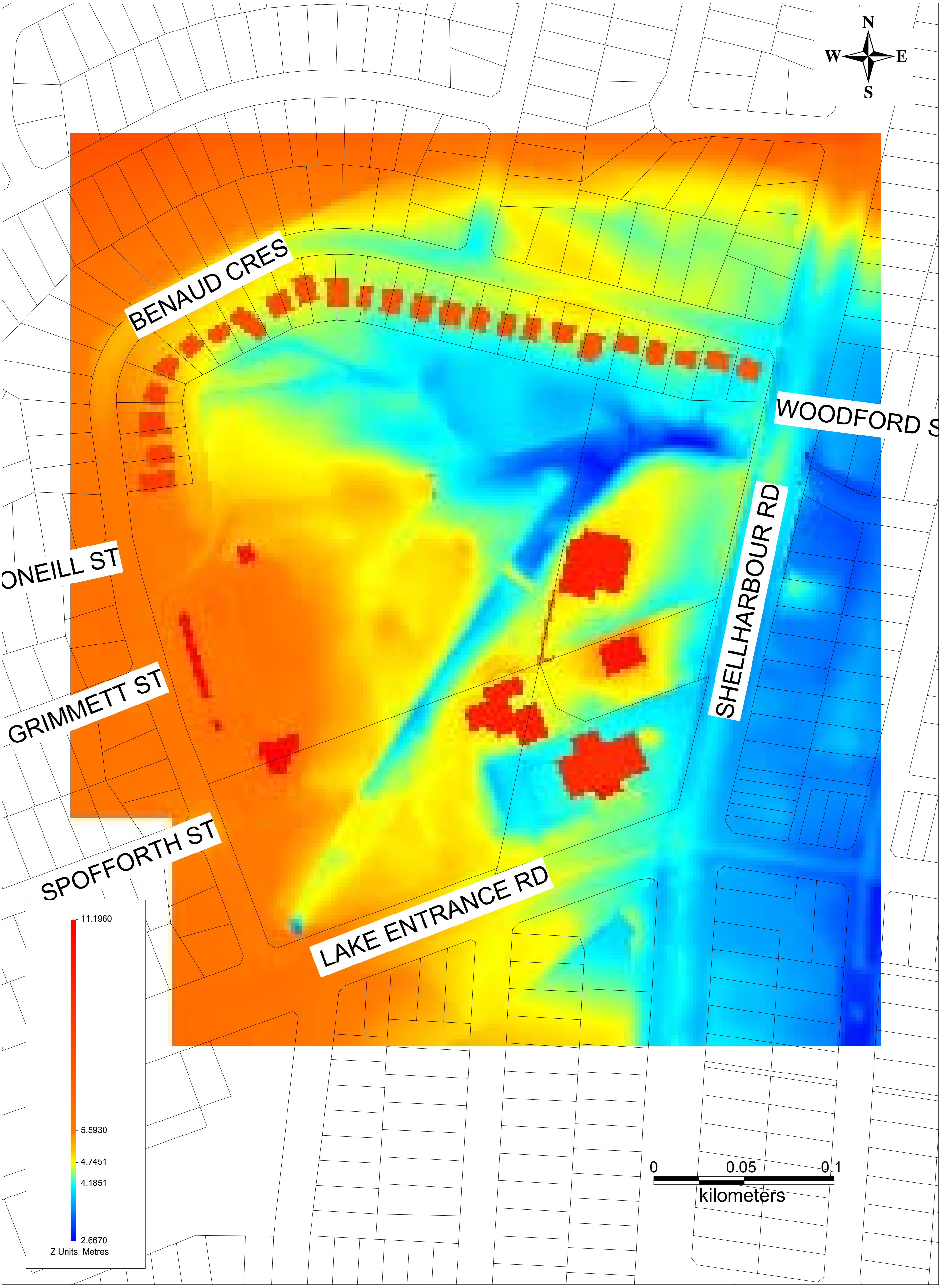
Cardno Lawson Treloar

J1959/R1974/V5
January 2006

Elliot Lake - Little Lake
Flood Study

FIGURE 12A
2D 10-m TOPOGRAPHY GRID

J1959/Figures V5/Fig_12A_10m_Topo_grid.Wor



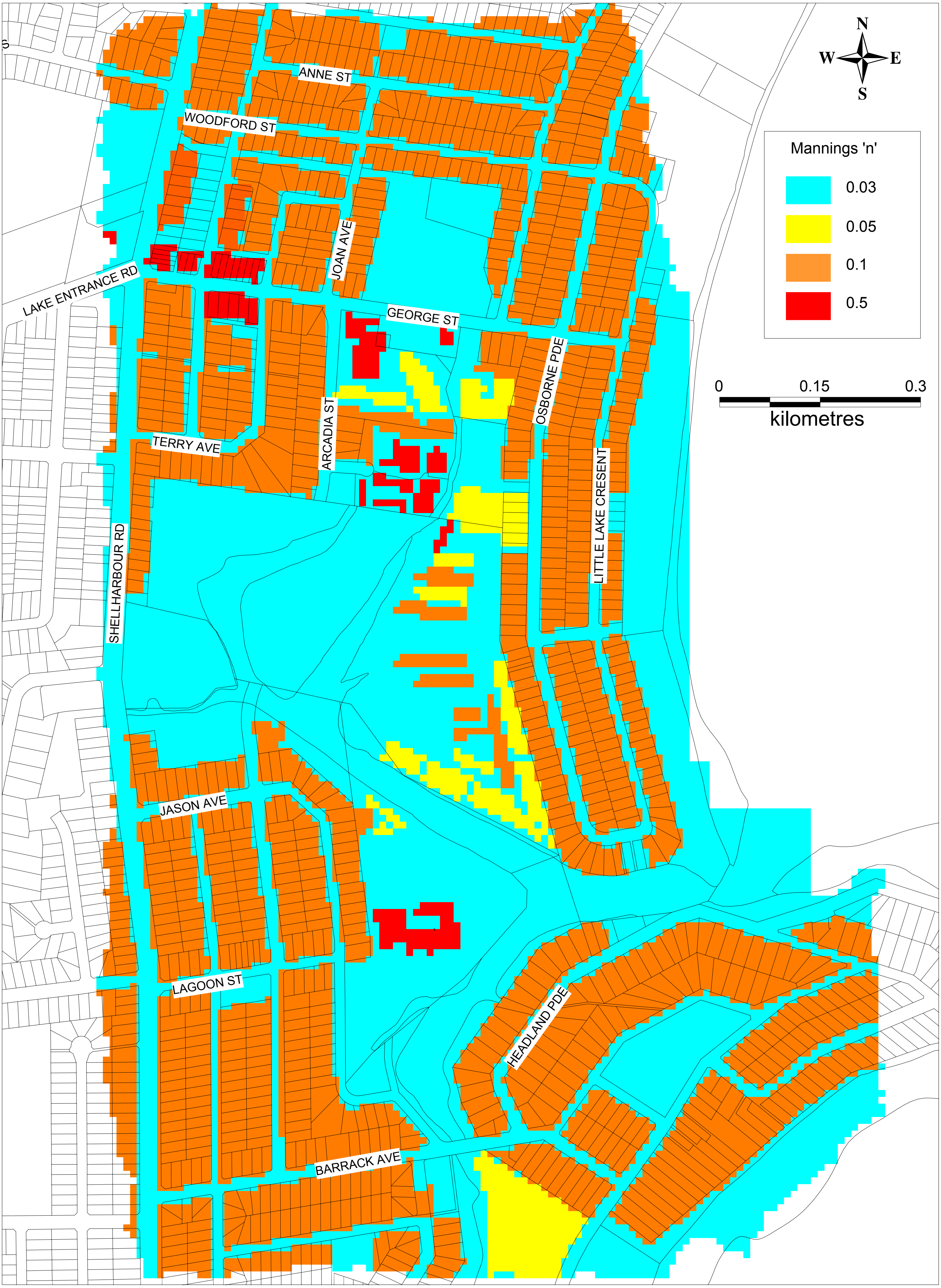
Cardno Lawson Treloar

J1959/R1974/V5
January 2006

Elliot Lake - Little Lake
Flood Study

FIGURE 12B
2D 2-m TOPOGRAPHY GRID

J1959/Figures V5/Fig_12B_2m_Topo_Grid.Wor



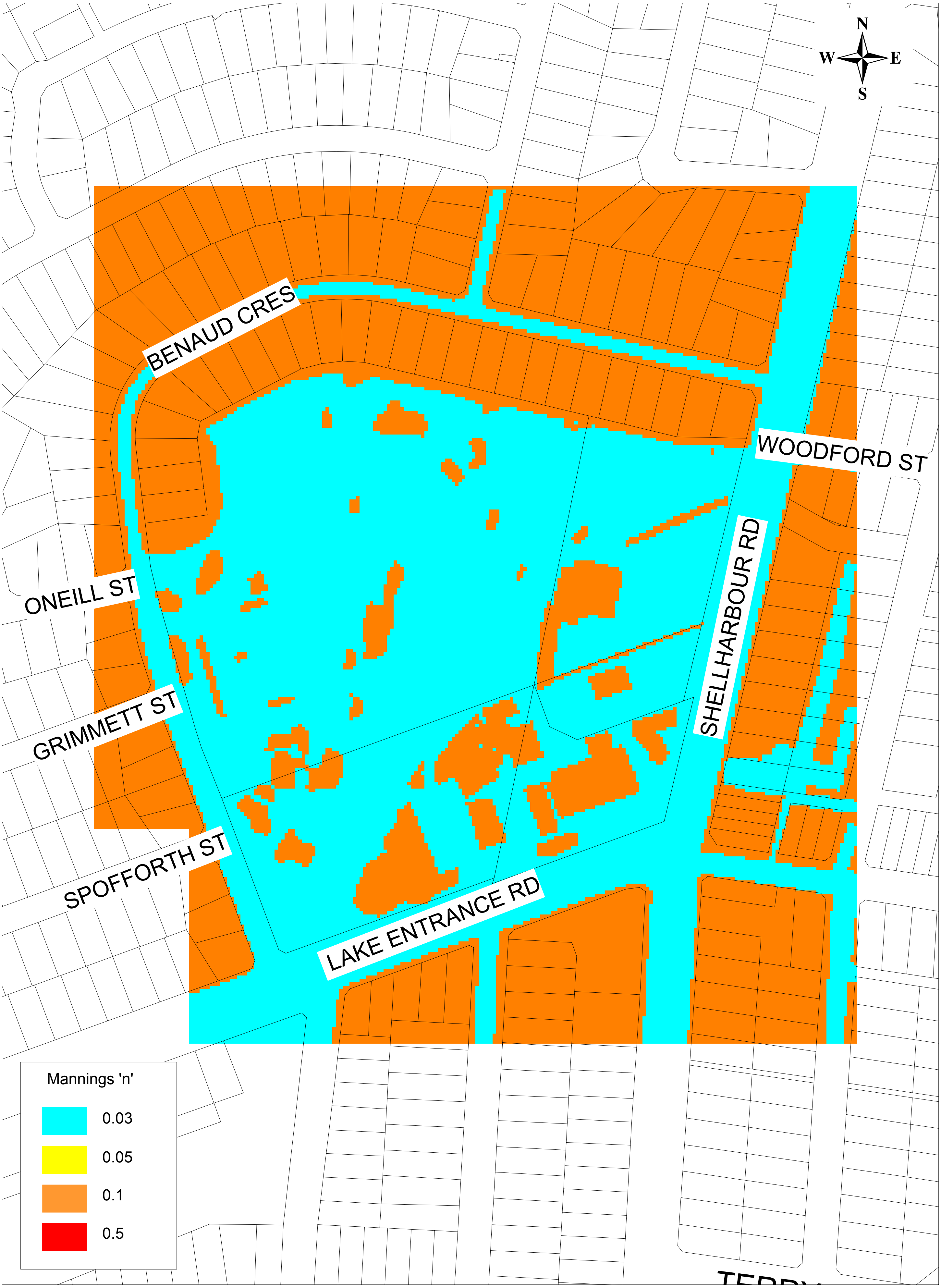
Cardno Lawson Treloar

J1959/R1974/V5
January 2006

Elliot Lake - Little Lake
Flood Study

FIGURE 13A
2D 10-m ROUGHNESS GRID

J1959/Figures V5/Fig_13A_10m_Rough_Grid.Wor



Cardno Lawson Treloar

J1959/R1974/V5
January 2006

Elliot Lake - Little Lake
Flood Study

FIGURE 13B
2D 2-m ROUGHNESS GRID

J1959/Figures V5/Fig_13B_2m_Rough_Grid.Wor

Cardno Lawson Treloar

J1959/R1974/V5
January 2006

Elliot Lake - Little Lake
Flood Study

FIGURE 14
MAY 1983 EVENT
RECORDED FLOOD LEVEL DATA

J1959\Figures V5\Fig 14 M83 level.Wor

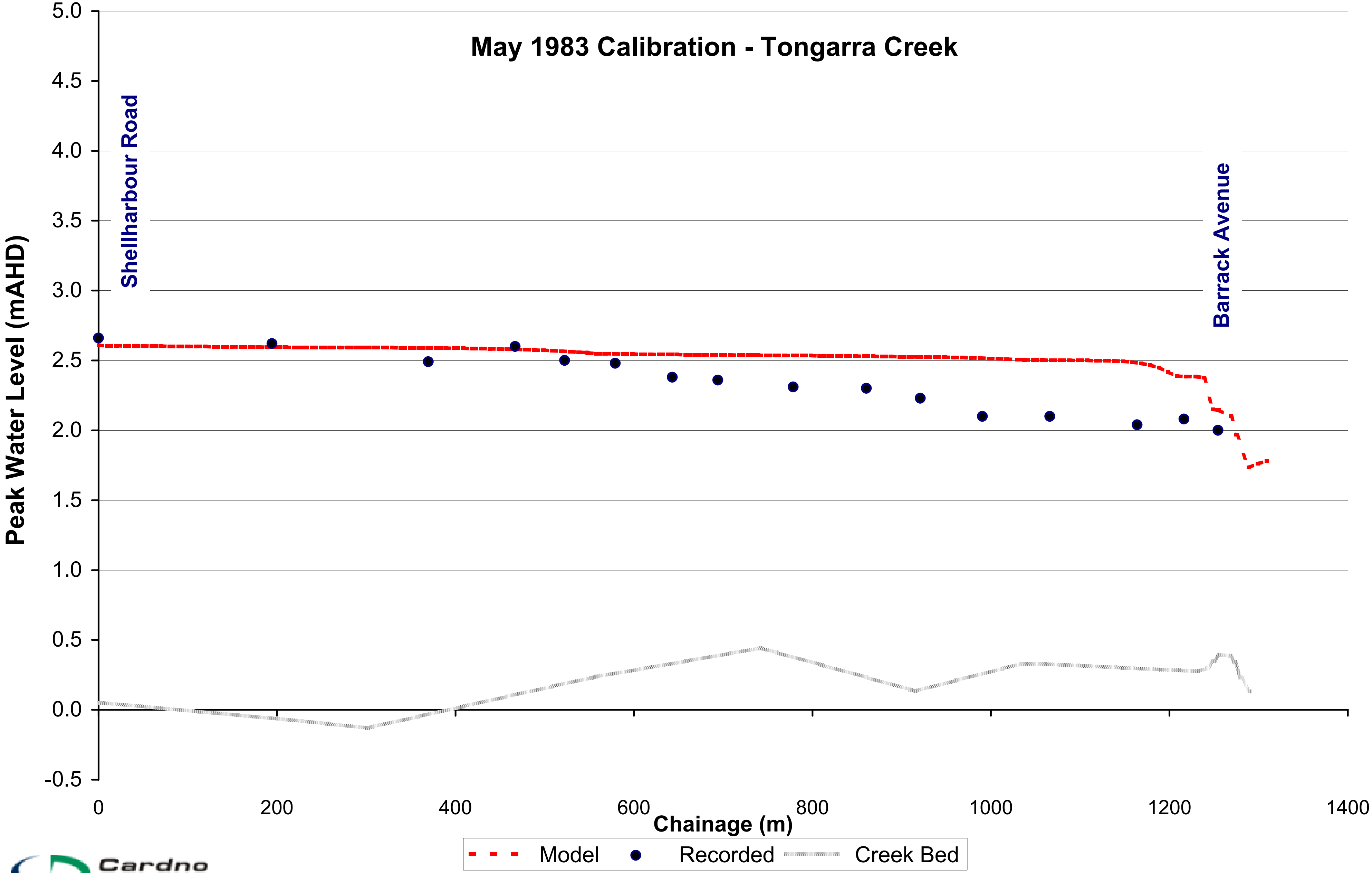
Cardno Lawson Treloar

J1959/R1974/V5
January 2006

Elliot Lake - Little Lake
Flood Study

FIGURE 15
MARCH 1975 EVENT
RECORDED FLOOD LEVEL DATA

J1959\Figures V5\Fig 15 M75 level.Wor

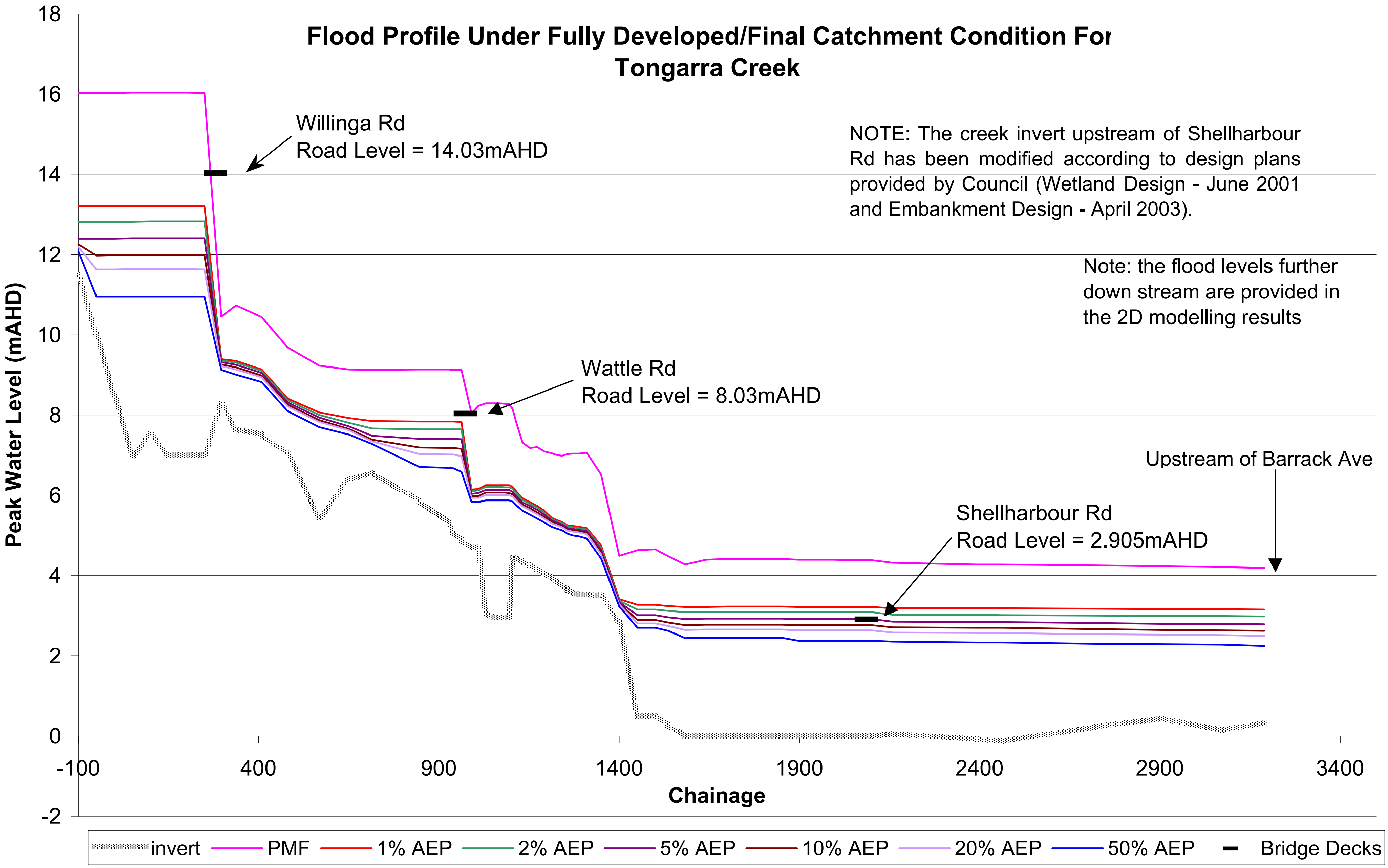


Cardno Lawson Treloar

J1959/R1974/V5
January 2006

Elliot Lake - Little Lake
Flood Study

FIGURE 16
MAY 1983 CALIBRATION
J1959\Figures V5\ Fig_16_May83 Profile



Cardno Lawson Treloar

FIGURE 17
FLOOD PROFILE UNDER FINAL CATCHMENT CONDITION FOR
TONGARRA CREEK

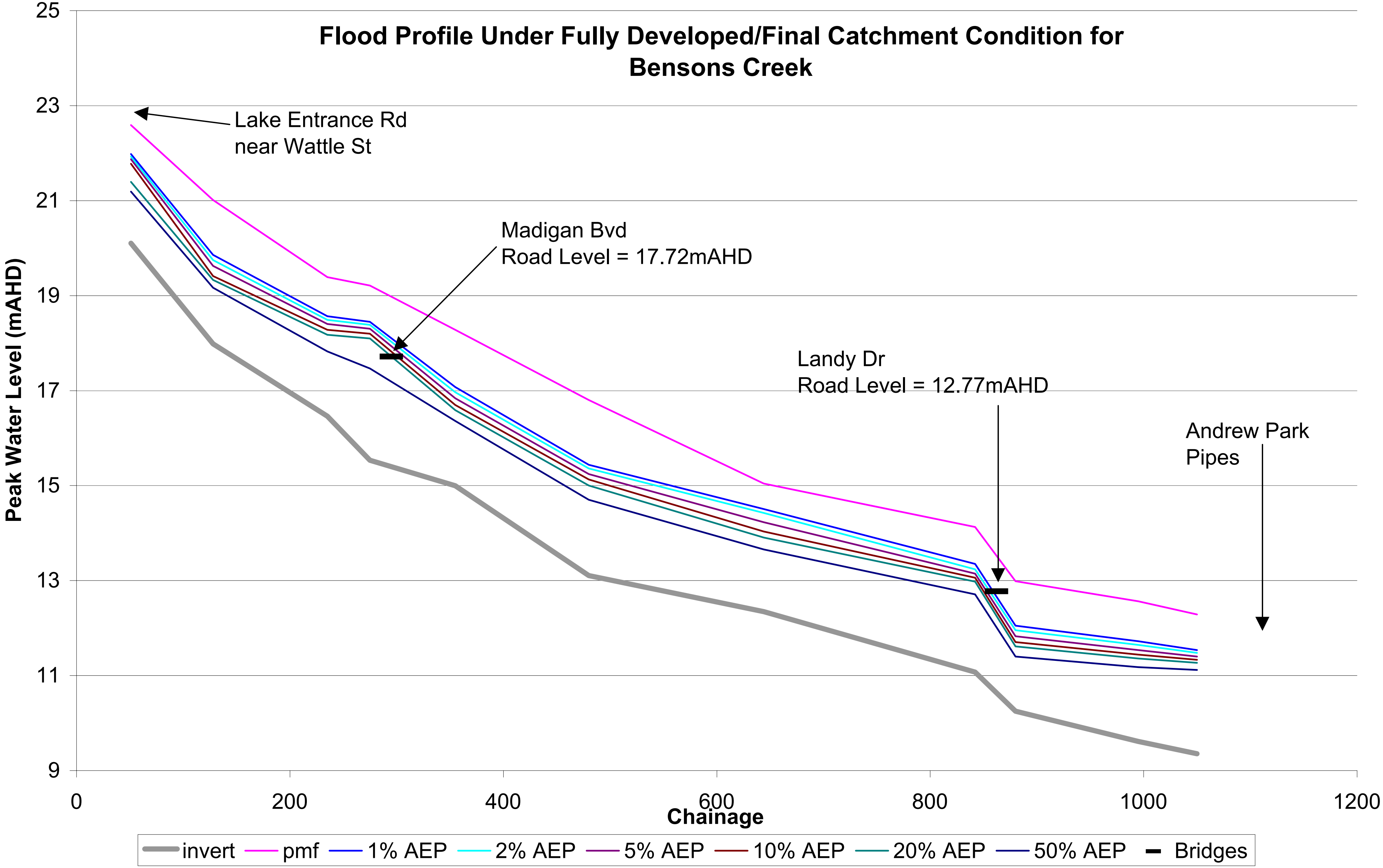


FIGURE 18
FLOOD PROFILE UNDER FINAL CATCHMENT CONDITION FOR BENSONS CREEK
J1959/Figures V5/Fig_17-20_Water Level Profiles

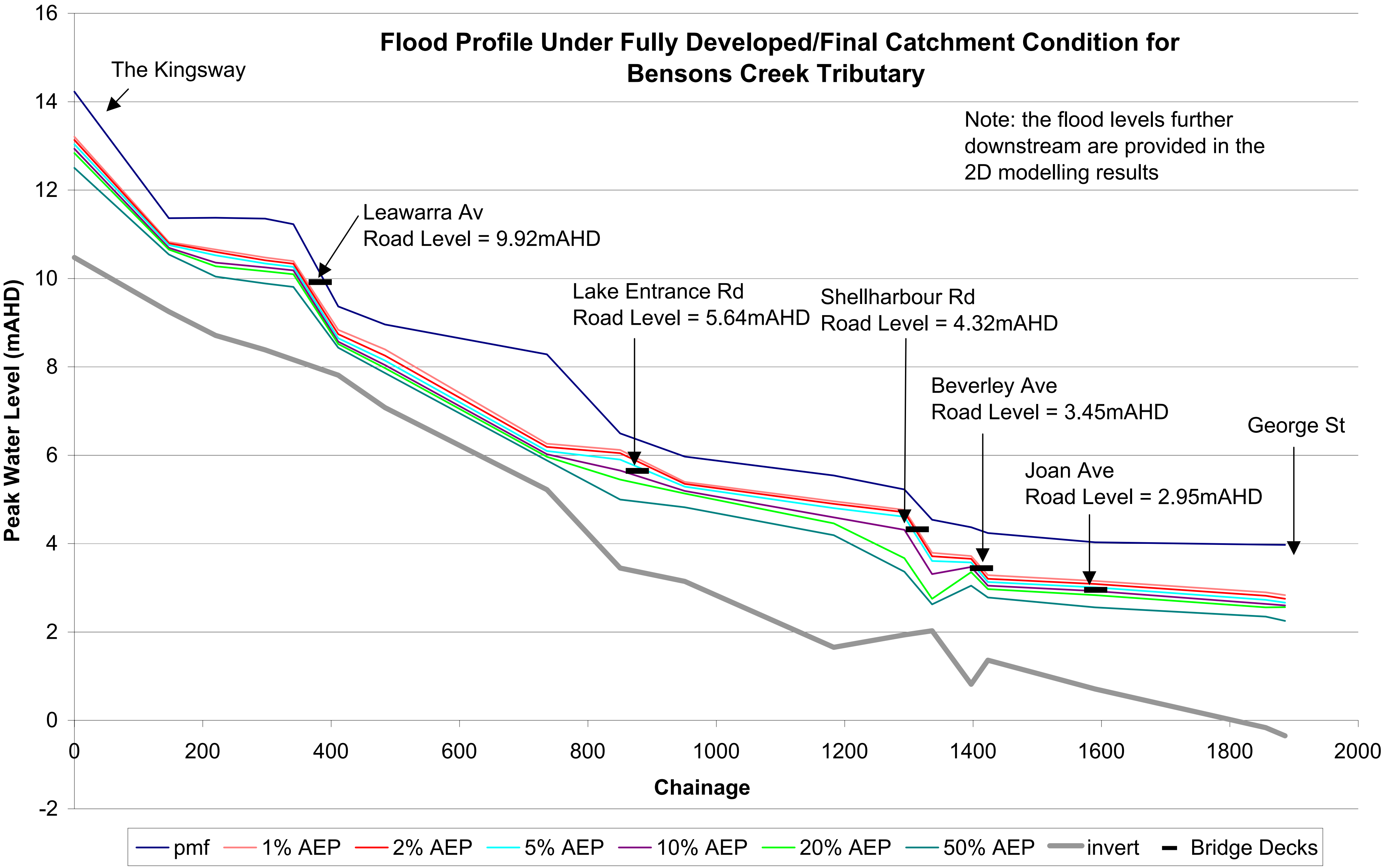


FIGURE 19
FLOOD PROFILE UNDER FINAL CATCHMENT CONDITION FOR
BENSONS CREEK TRIBUTARY

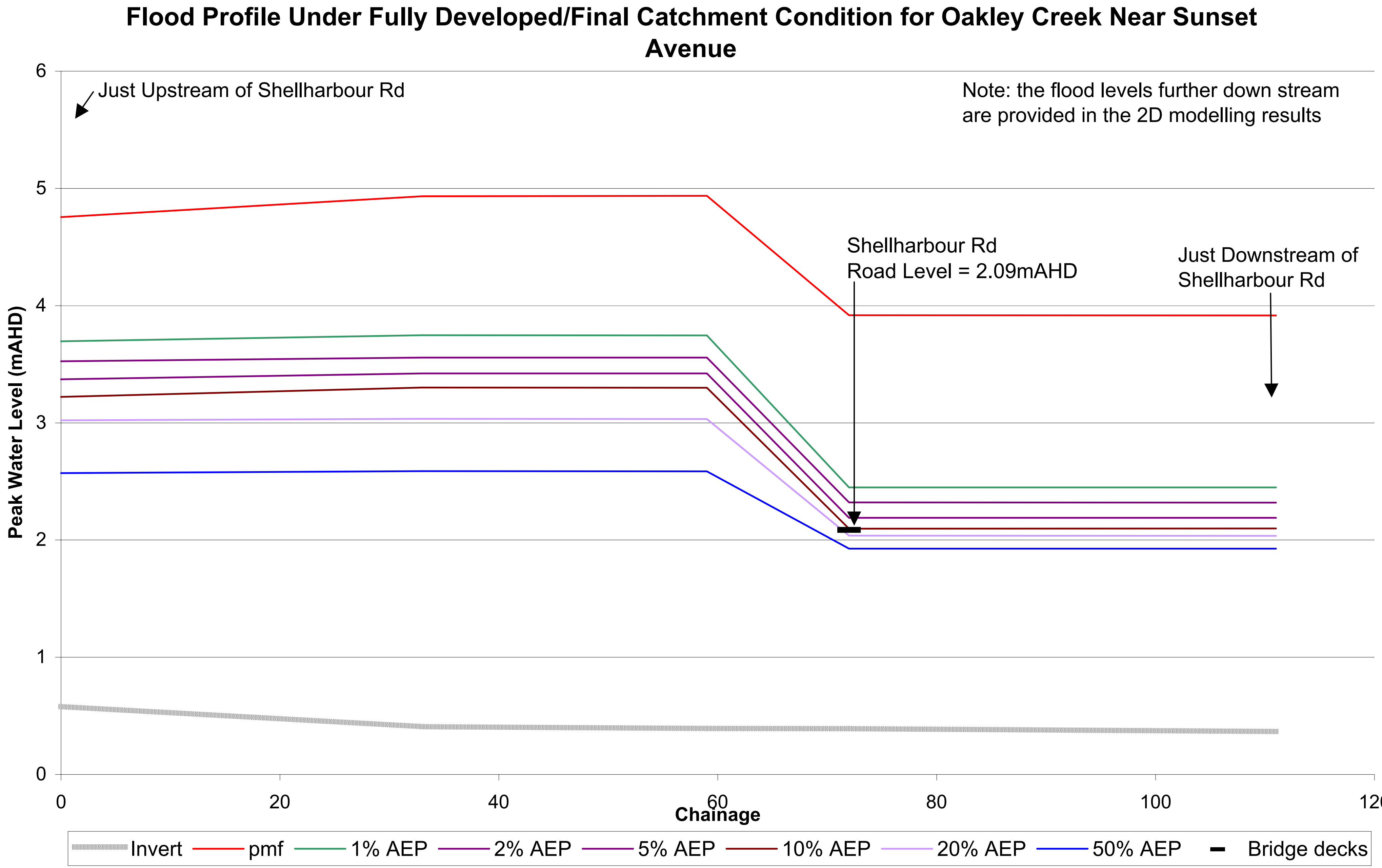


FIGURE 20
FLOOD PROFILE UNDER FINAL CATCHMENT CONDITION FOR SUNSET AVE

Cardno Lawson Treloar
J1959/R1974/V5
January 2006

Flood Study

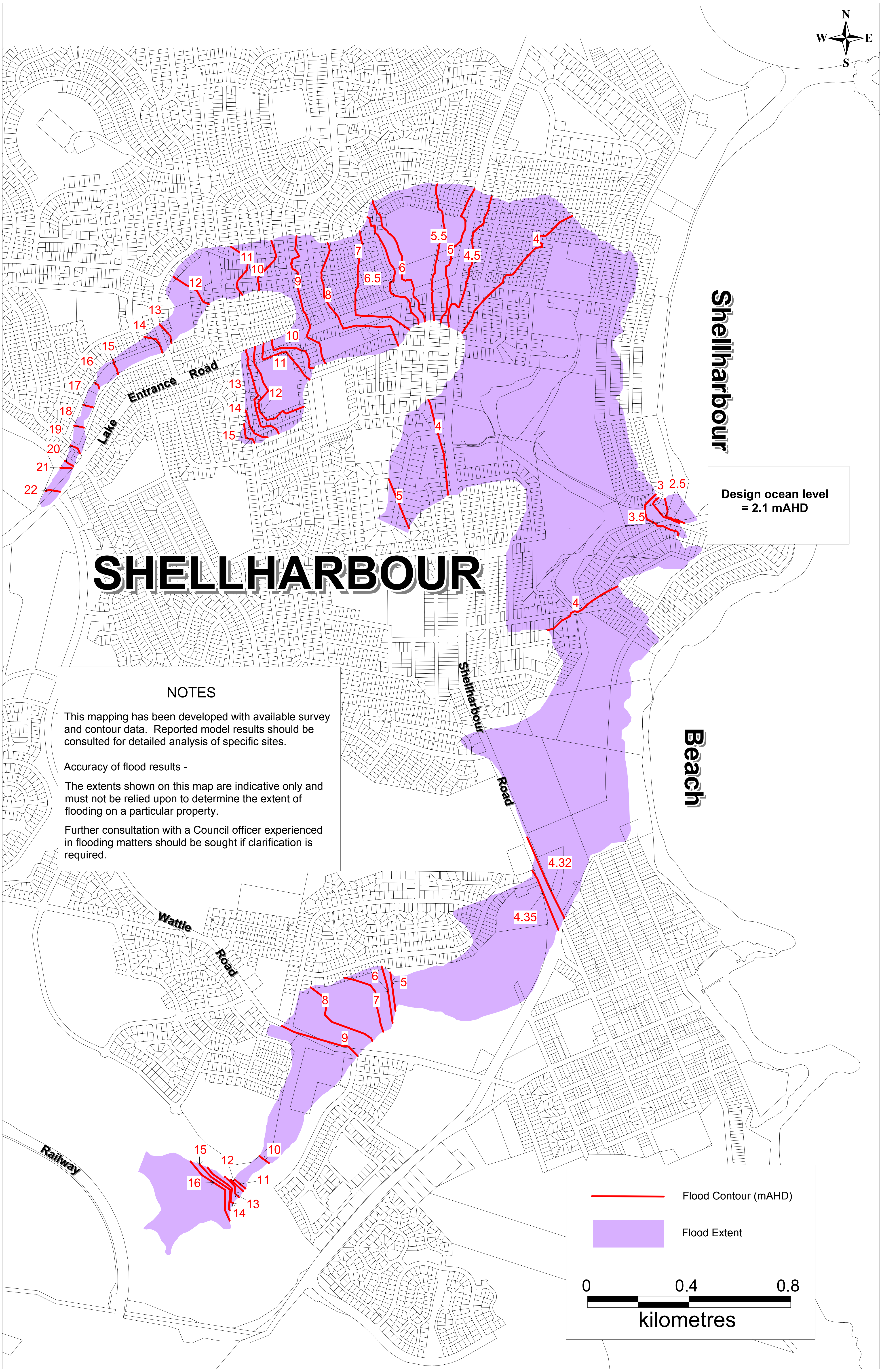
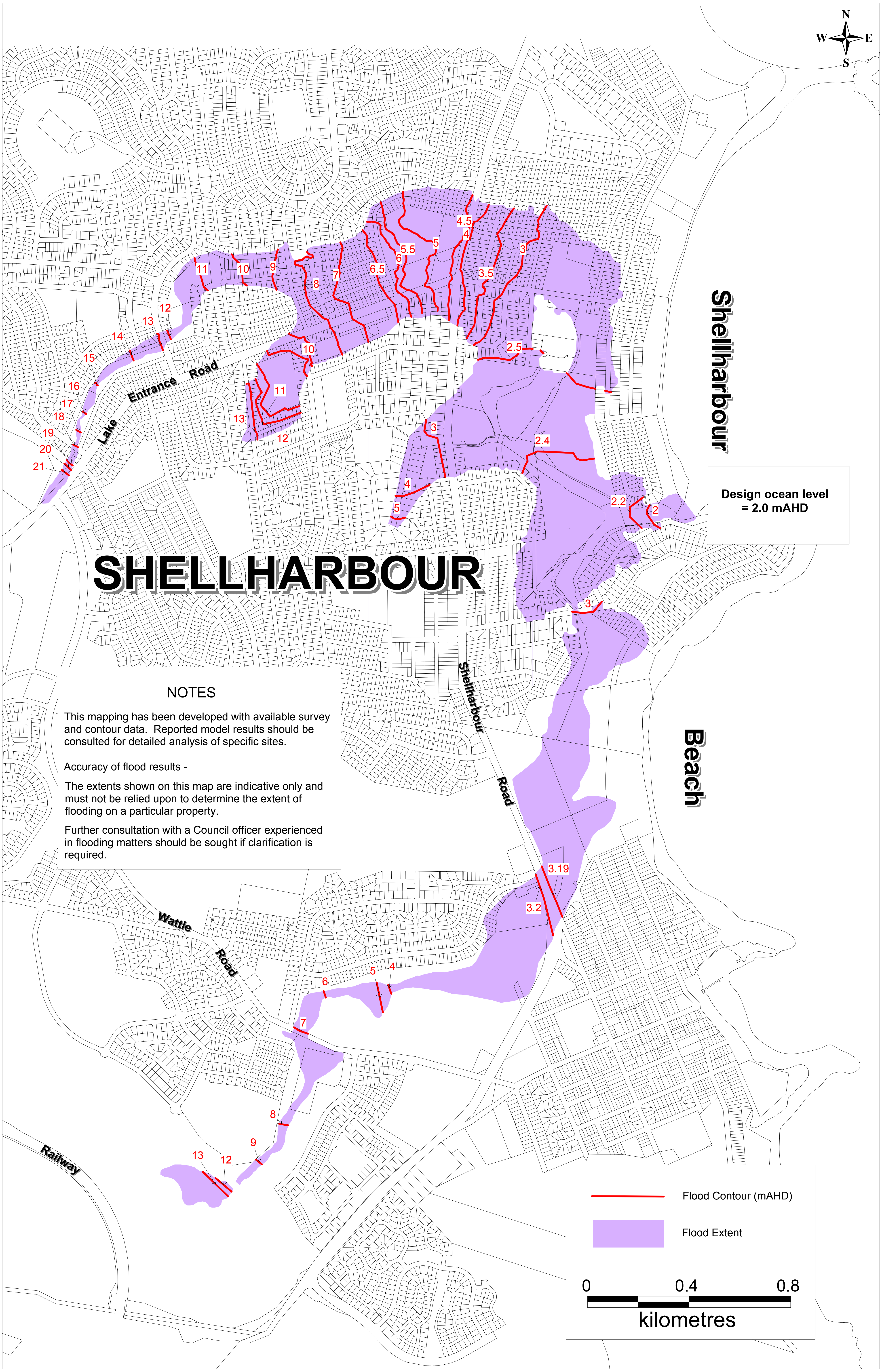


FIGURE 21
PMF FLOOD EXTENT AND CONTOURS
FULLY DEVELOPED/FINAL CATCHMENT CONDITION

J1959/Figures V5/Fig_21_PMF_extent.Wor

Cardno Lawson Treloar
J1959/R1974/V5
January 2006

Elliot Lake - Little Lake
Flood Study



NOTES

This mapping has been developed with available survey and contour data. Reported model results should be consulted for detailed analysis of specific sites.

Accuracy of flood results -

The extents shown on this map are indicative only and must not be relied upon to determine the extent of flooding on a particular property.

Further consultation with a Council officer experienced in flooding matters should be sought if clarification is required.

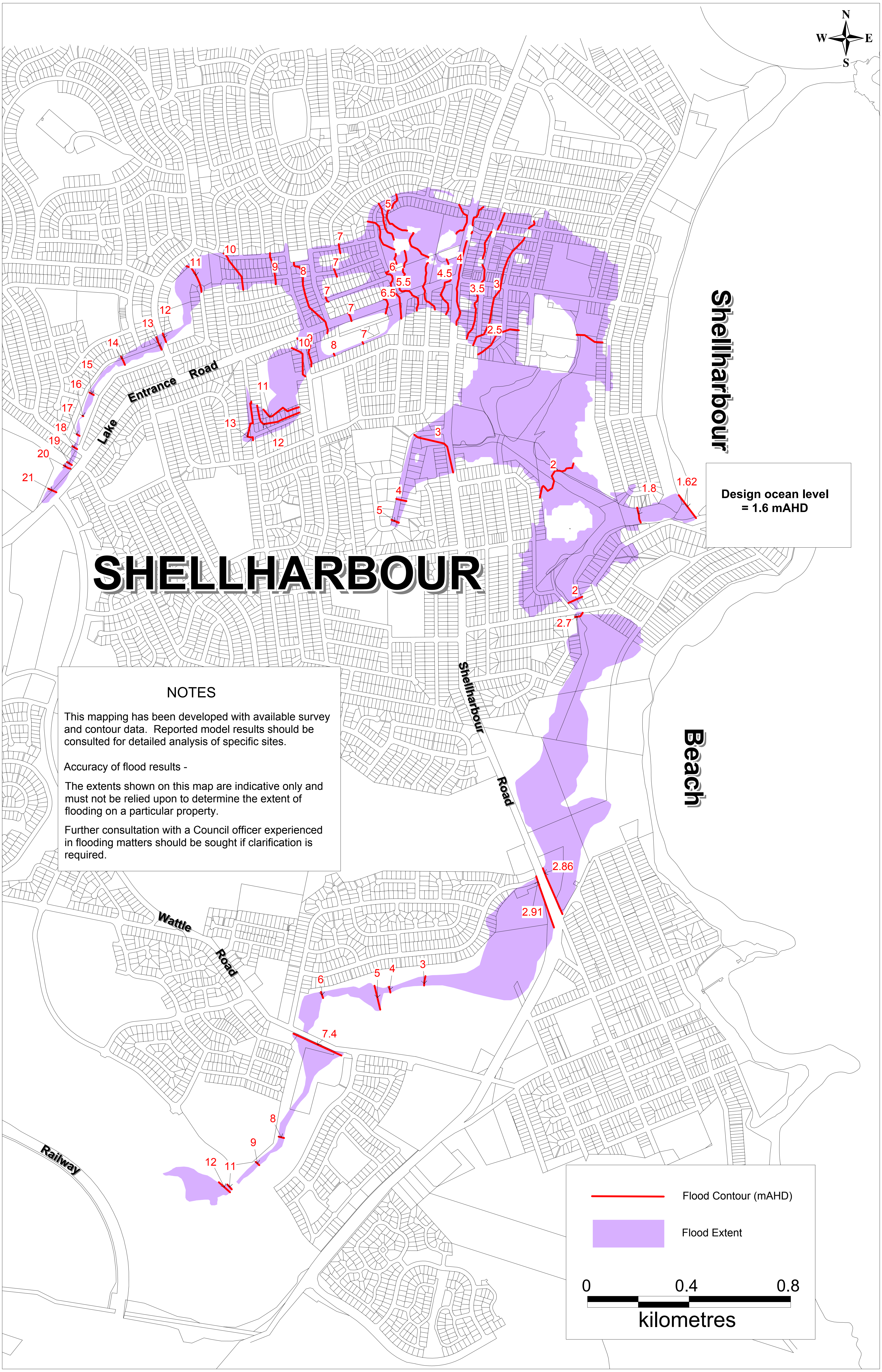
FIGURE 22
1% AEP FLOOD EXTENT AND CONTOURS
FULLY DEVELOPED/FINAL CATCHMENT CONDITION

J1959/Figures V5/Fig_22_1%_extent.Wor

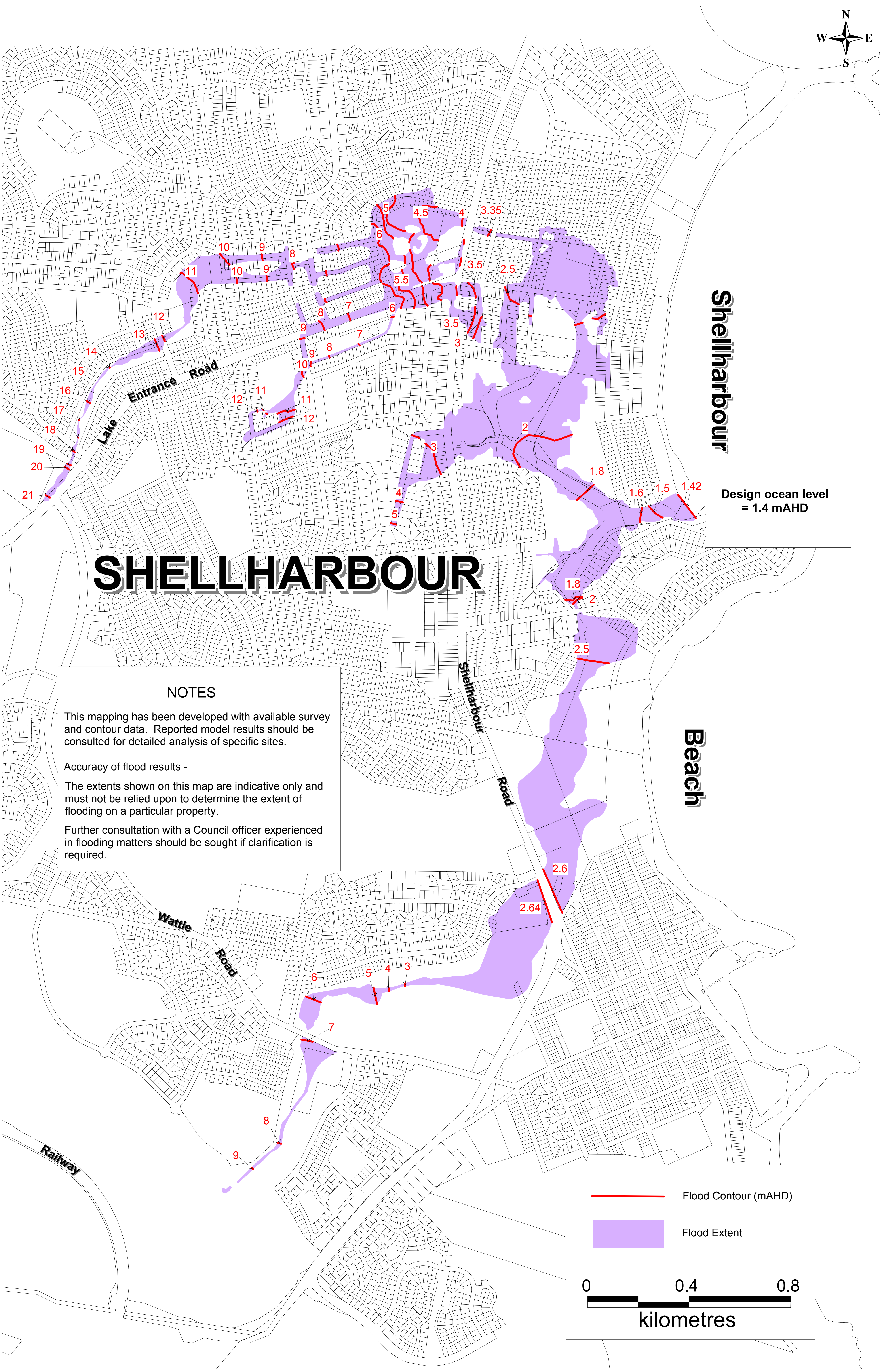
FIGURE 23

5% AEP FLOOD EXTENT AND CONTOURS

FULLY DEVELOPED/FINAL CATCHMENT CONDITION



Cardno Lawson Treloar
J1959/R1974/V5
January 2006
Elliot Lake - Little Lake Flood Study
J1959/Figures V5/Fig_23_5%_extent.Wor



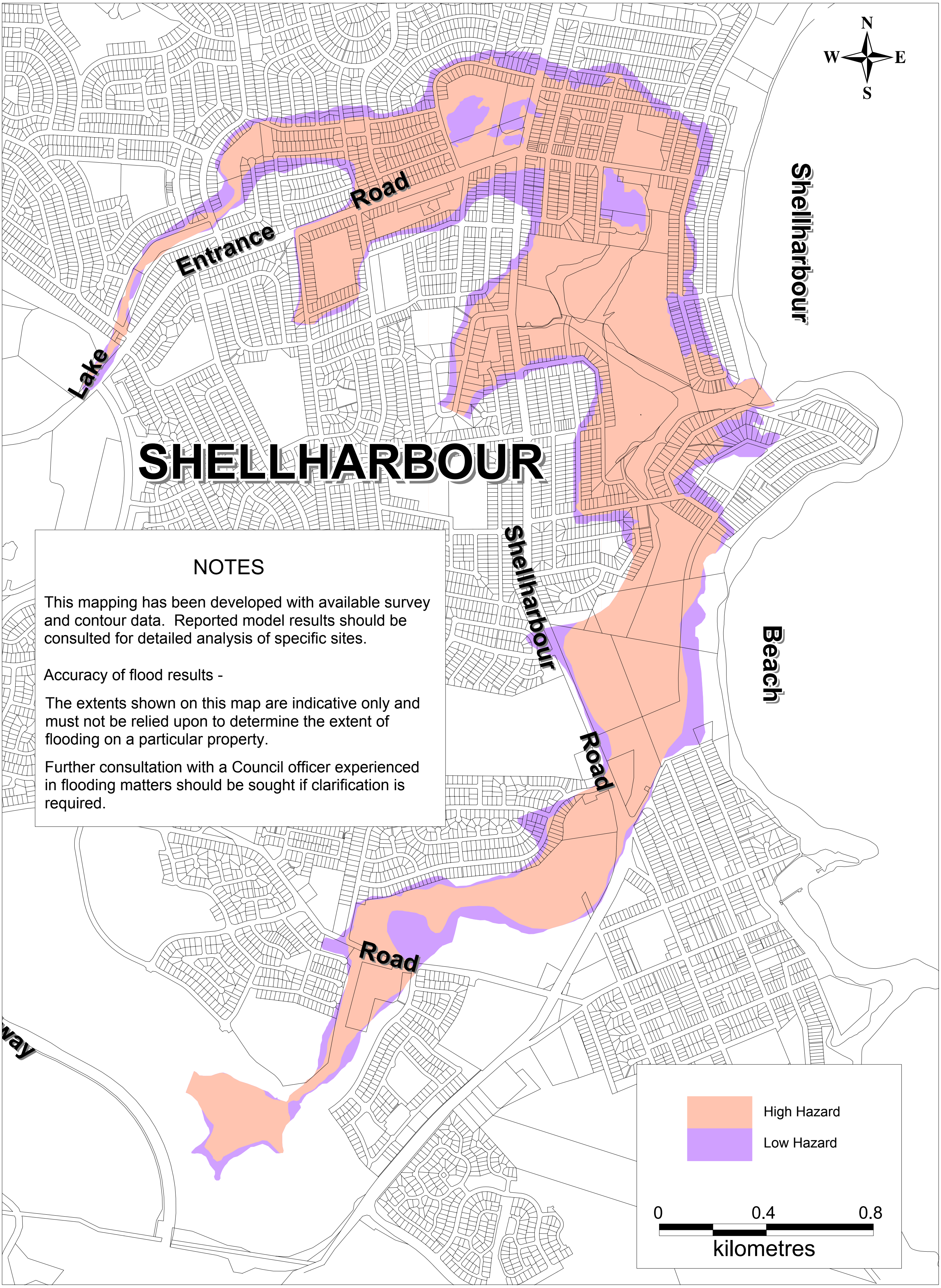
Cardno Lawson Treloar

J1959/R1974/V5
January 2006

Elliot Lake - Little Lake
Flood Study

FIGURE 24
5% AEP FLOOD EXTENT AND CONTOURS
FULLY DEVELOPED/FINAL CATCHMENT CONDITION

J1959/Figures V5/Fig_24_20%_extent.Wor



NOTES

This mapping has been developed with available survey and contour data. Reported model results should be consulted for detailed analysis of specific sites.

Accuracy of flood results -

The extents shown on this map are indicative only and must not be relied upon to determine the extent of flooding on a particular property.

Further consultation with a Council officer experienced in flooding matters should be sought if clarification is required.

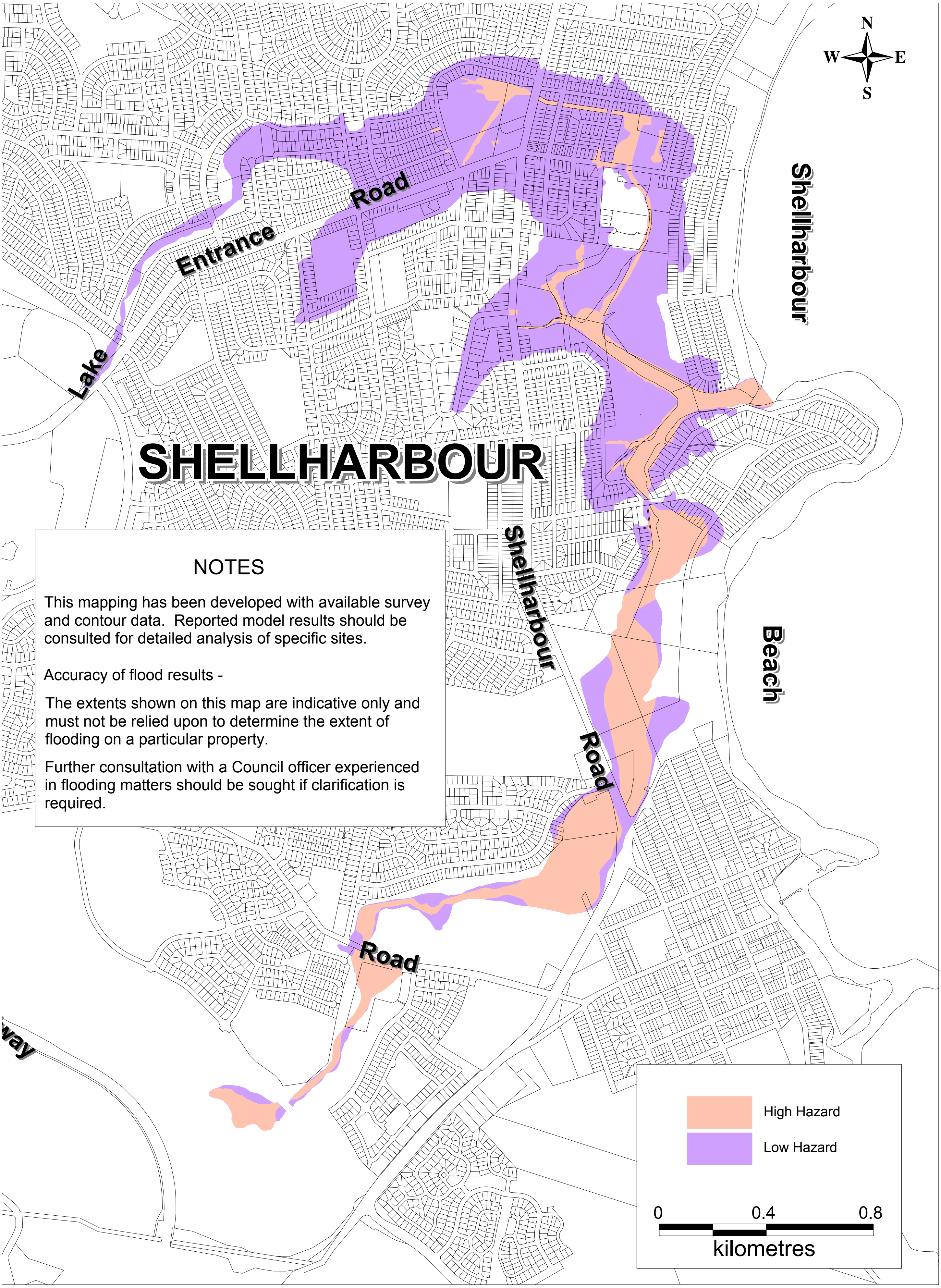
Cardno Lawson Treloar

J1959/R1974/V5
January 2006

Elliot Lake - Little Lake
Flood Study

FIGURE 30
PMF HAZARD CATEGORIES
FINAL CATCHMENT CONDITIONS

J1959/Figures V5/Fig_30_PMF_Hazard.Wor



NOTES

This mapping has been developed with available survey and contour data. Reported model results should be consulted for detailed analysis of specific sites.

Accuracy of flood results -

The extents shown on this map are indicative only and must not be relied upon to determine the extent of flooding on a particular property.

Further consultation with a Council officer experienced in flooding matters should be sought if clarification is required.

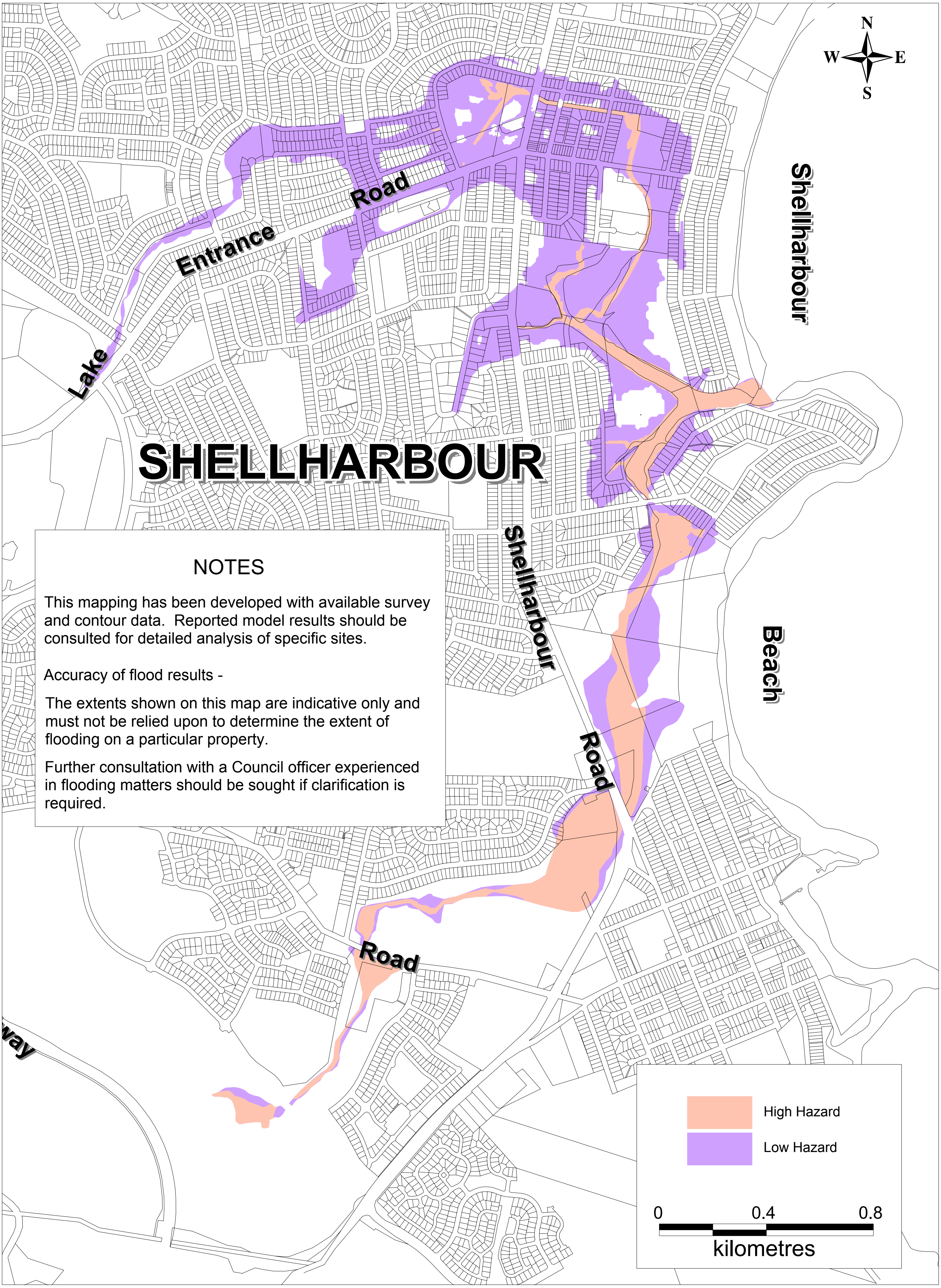
Cardno
Lawson
Treloar

J1959/R1974/V5
January 2006

Elliot Lake - Little Lake
Flood Study

FIGURE 31
1% AEP HAZARD CATEGORIES
FINAL CATCHMENT CONDITIONS

J1959/Figures V5/Fig_31_1%_Hazard.Wor



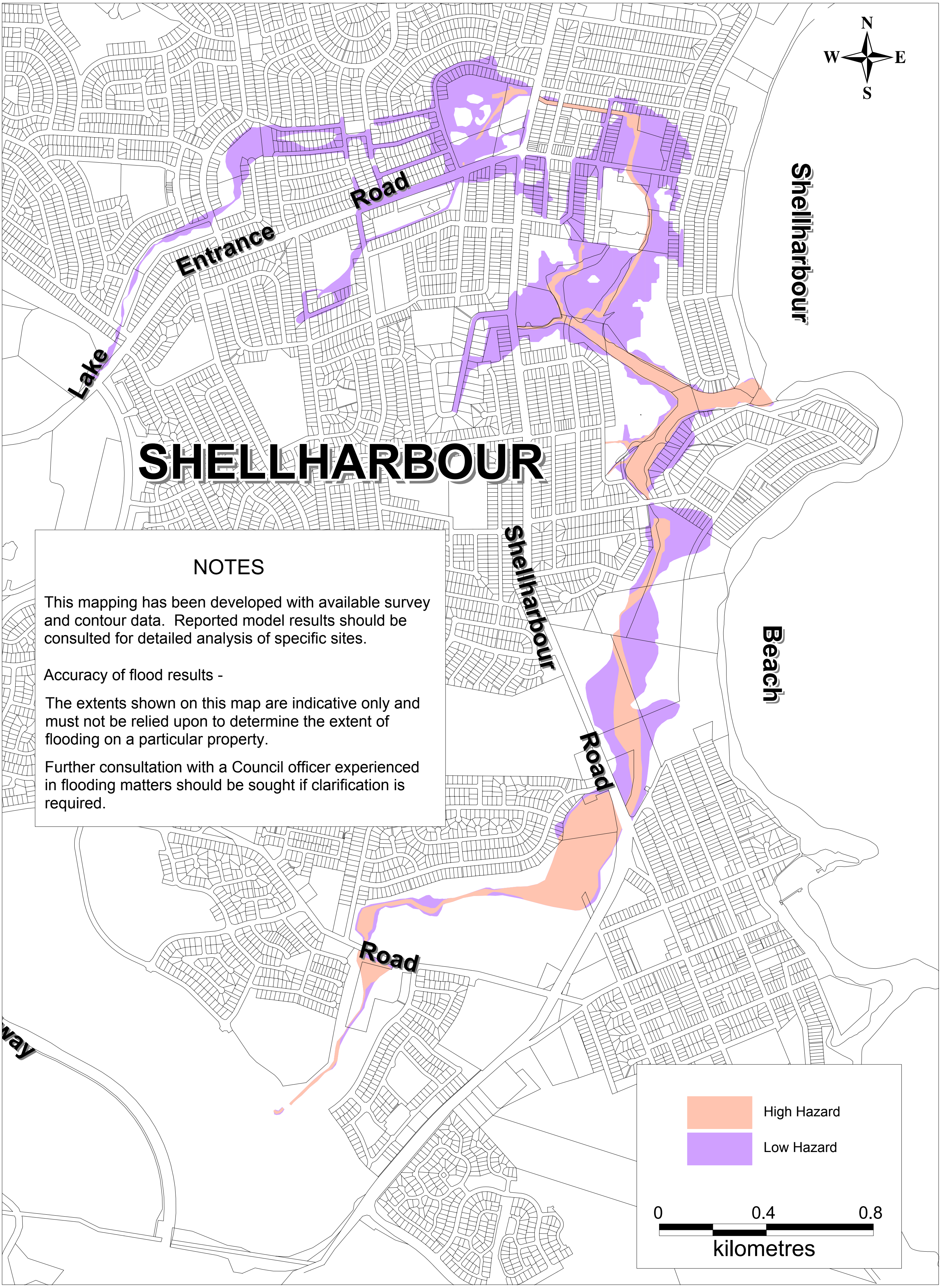
Cardno Lawson Treloar

J1959/R1974/V5
January 2006

Elliot Lake - Little Lake
Flood Study

FIGURE 32
5% AEP HAZARD CATEGORIES
FINAL CATCHMENT CONDITIONS

J1959/Figures V5/Fig_32_5%_Hazard.Wor



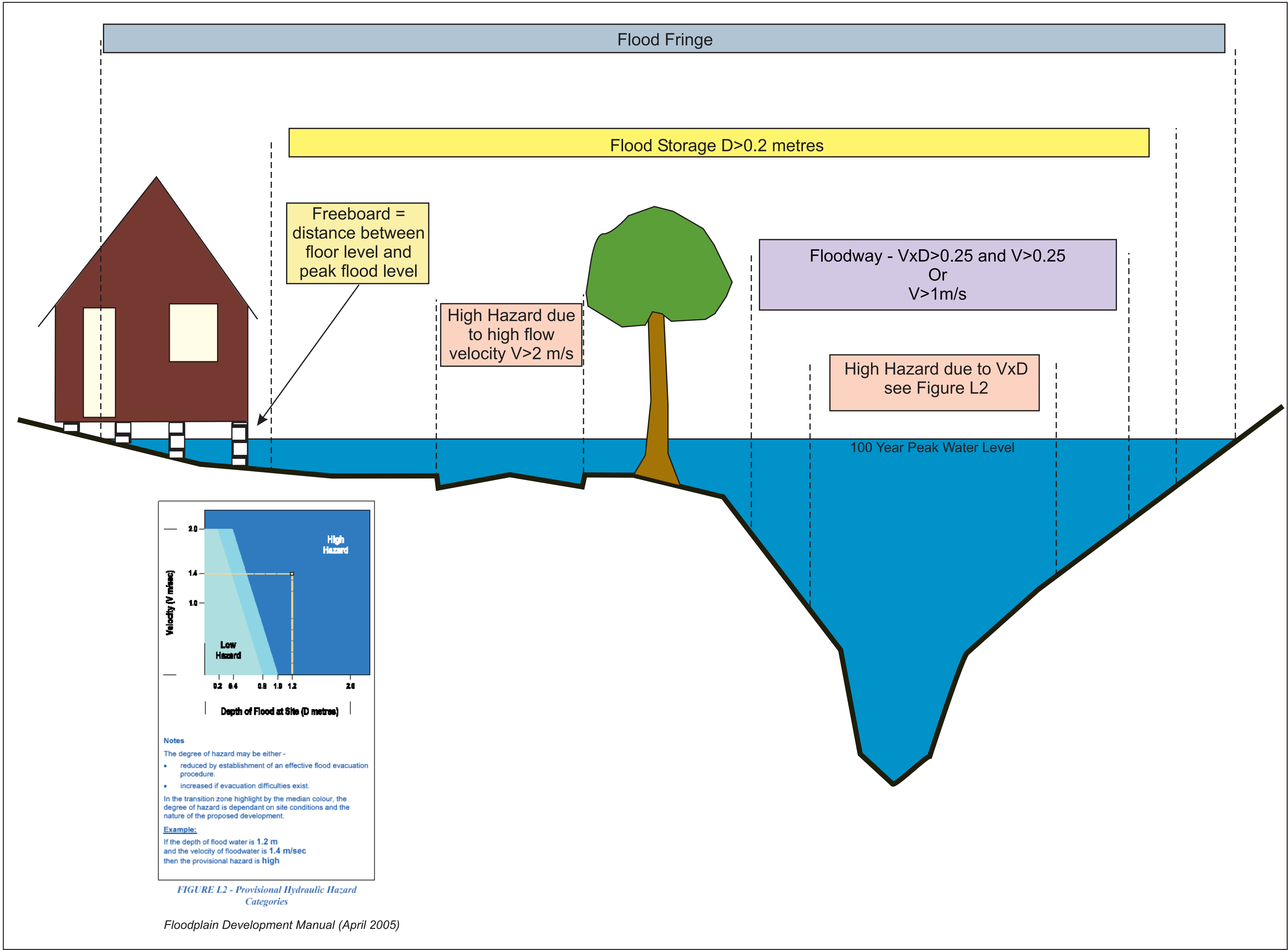
Cardno Lawson Treloar

J1959/R1974/V5
January 2006

Elliot Lake - Little Lake
Flood Study

FIGURE 33
20% AEP HAZARD CATEGORIES
FINAL CATCHMENT CONDITIONS

J1959/Figures V5/Fig_33_20%_Hazard.Wor



Cardno Lawson Treloar

J1959/R1974/V5

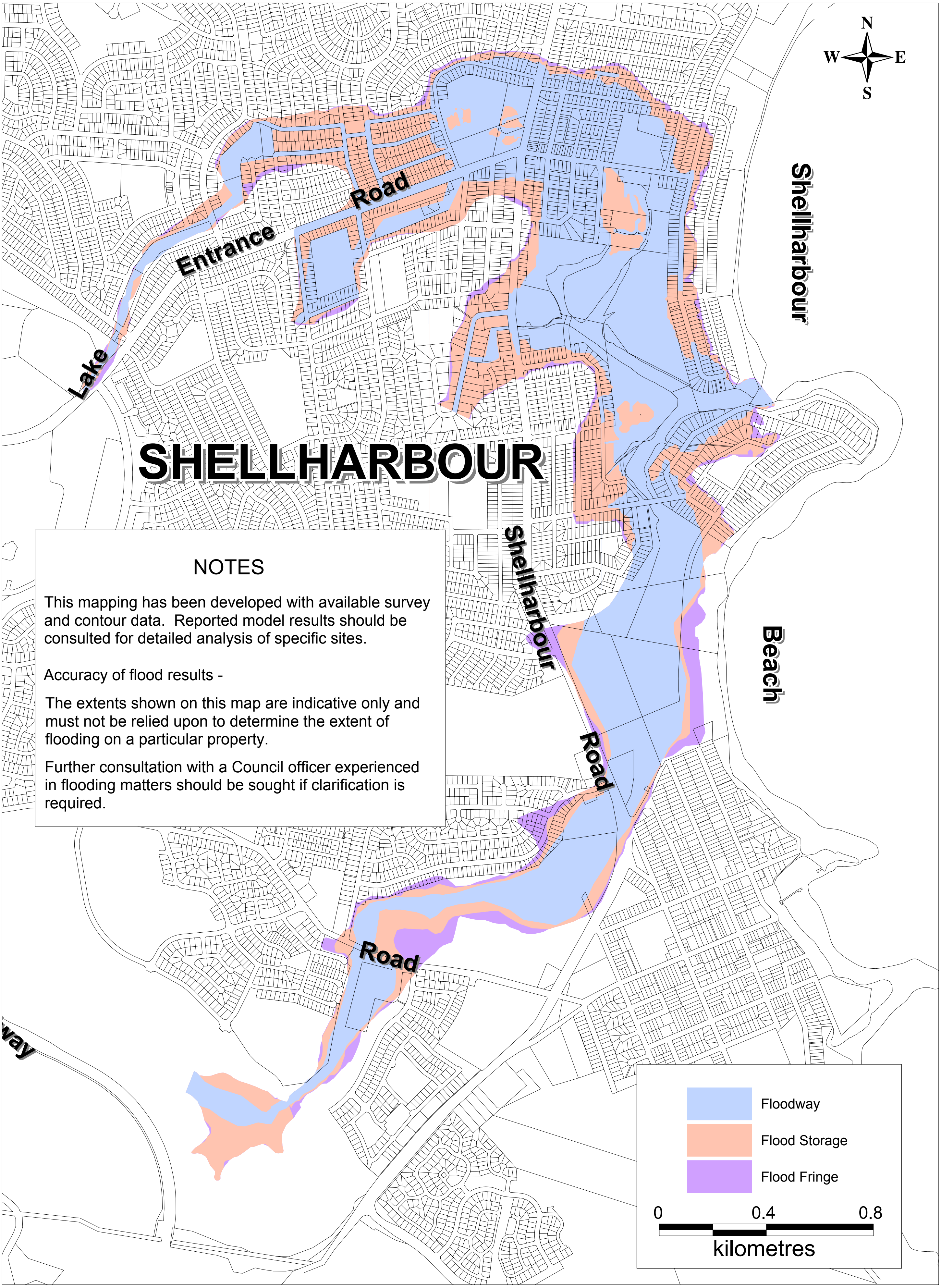
January 2006

Elliot Lake - Little Lake Flood Study

FIGURE 33b

HYDRAULIC CATEGORIES

J1959\Figures V5\Fig 33b hydraulic categories.cdr



NOTES

This mapping has been developed with available survey and contour data. Reported model results should be consulted for detailed analysis of specific sites.

Accuracy of flood results -

The extents shown on this map are indicative only and must not be relied upon to determine the extent of flooding on a particular property.

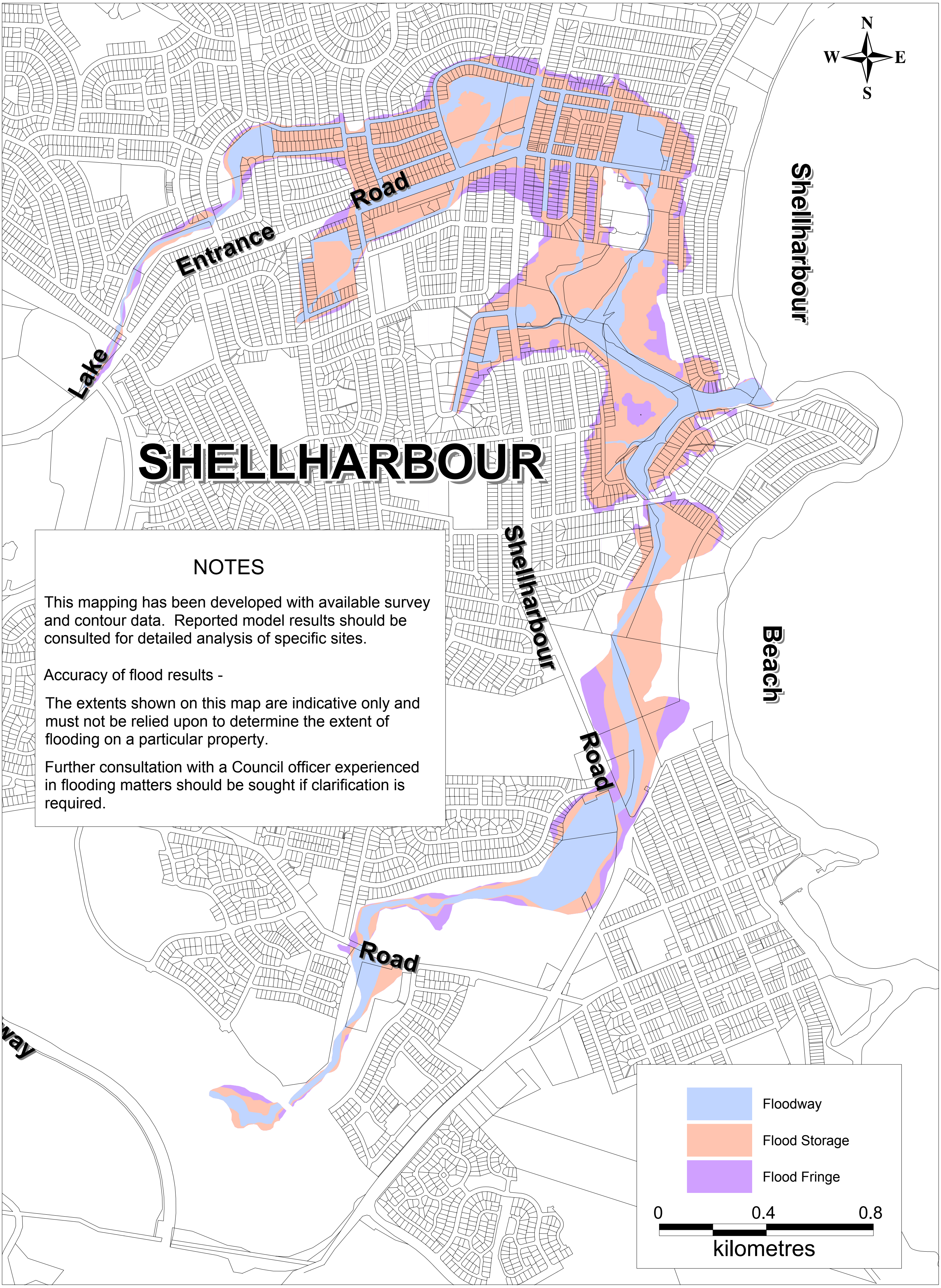
Further consultation with a Council officer experienced in flooding matters should be sought if clarification is required.

Cardno Lawson Treloar

J1959/R1974/V5
January 2006

Elliot Lake - Little Lake Flood Study

FIGURE 34
PMF HYDRAULIC CATEGORIES
FINAL CATCHMENT CONDITIONS
J1959/Figures V5/Fig_34_PMF_Hyd_Cat.Wor



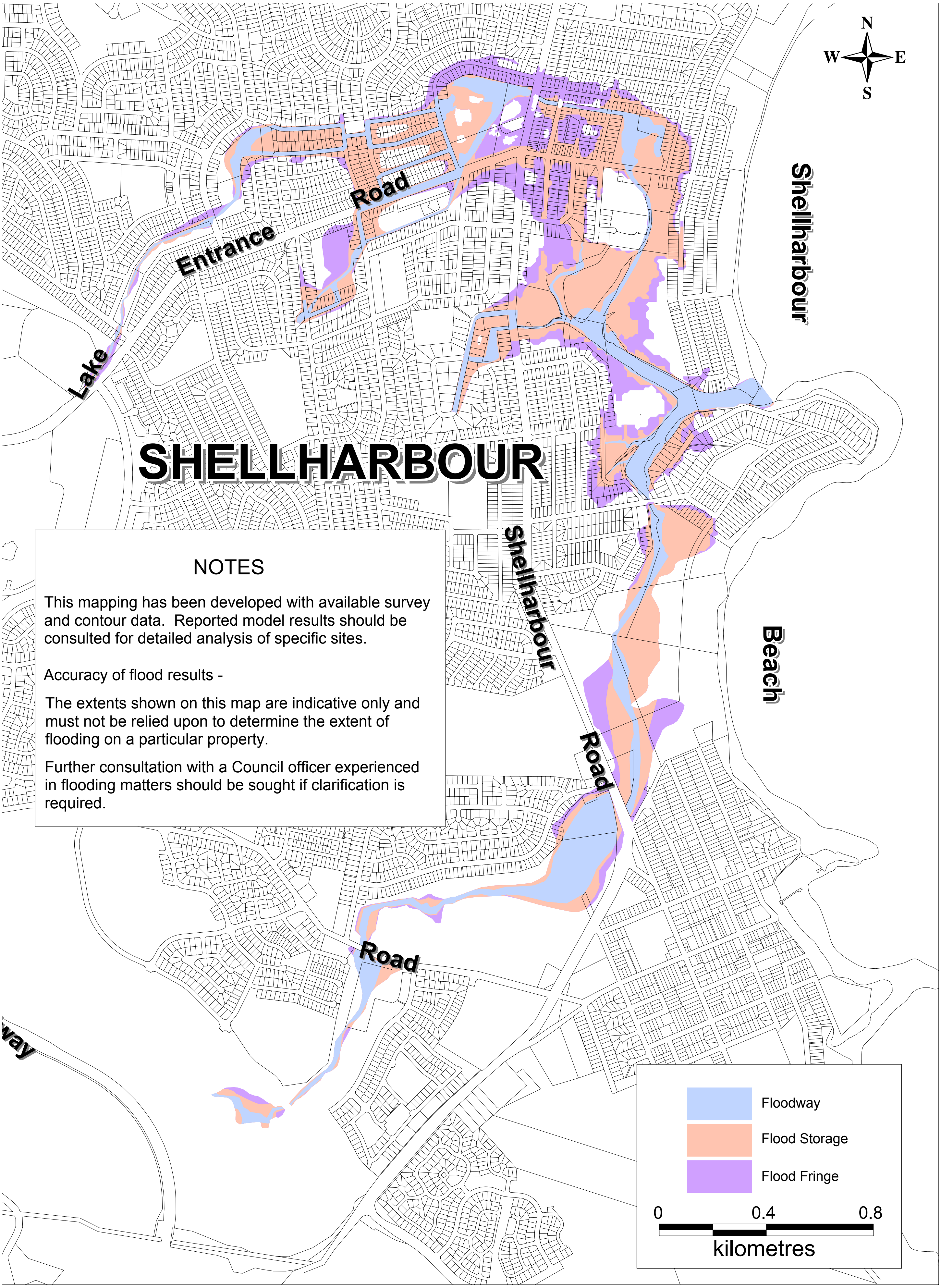
Cardno Lawson Treloar

J1959/R1974/V5
January 2006

Elliot Lake - Little Lake Flood Study

FIGURE 35
1% AEP HYDRAULIC CATEGORIES
FINAL CATCHMENT CONDITIONS

J1959/Figures V5/Fig_35_1%_Hyd_Cat.Wor



NOTES

This mapping has been developed with available survey and contour data. Reported model results should be consulted for detailed analysis of specific sites.

Accuracy of flood results -

The extents shown on this map are indicative only and must not be relied upon to determine the extent of flooding on a particular property.

Further consultation with a Council officer experienced in flooding matters should be sought if clarification is required.

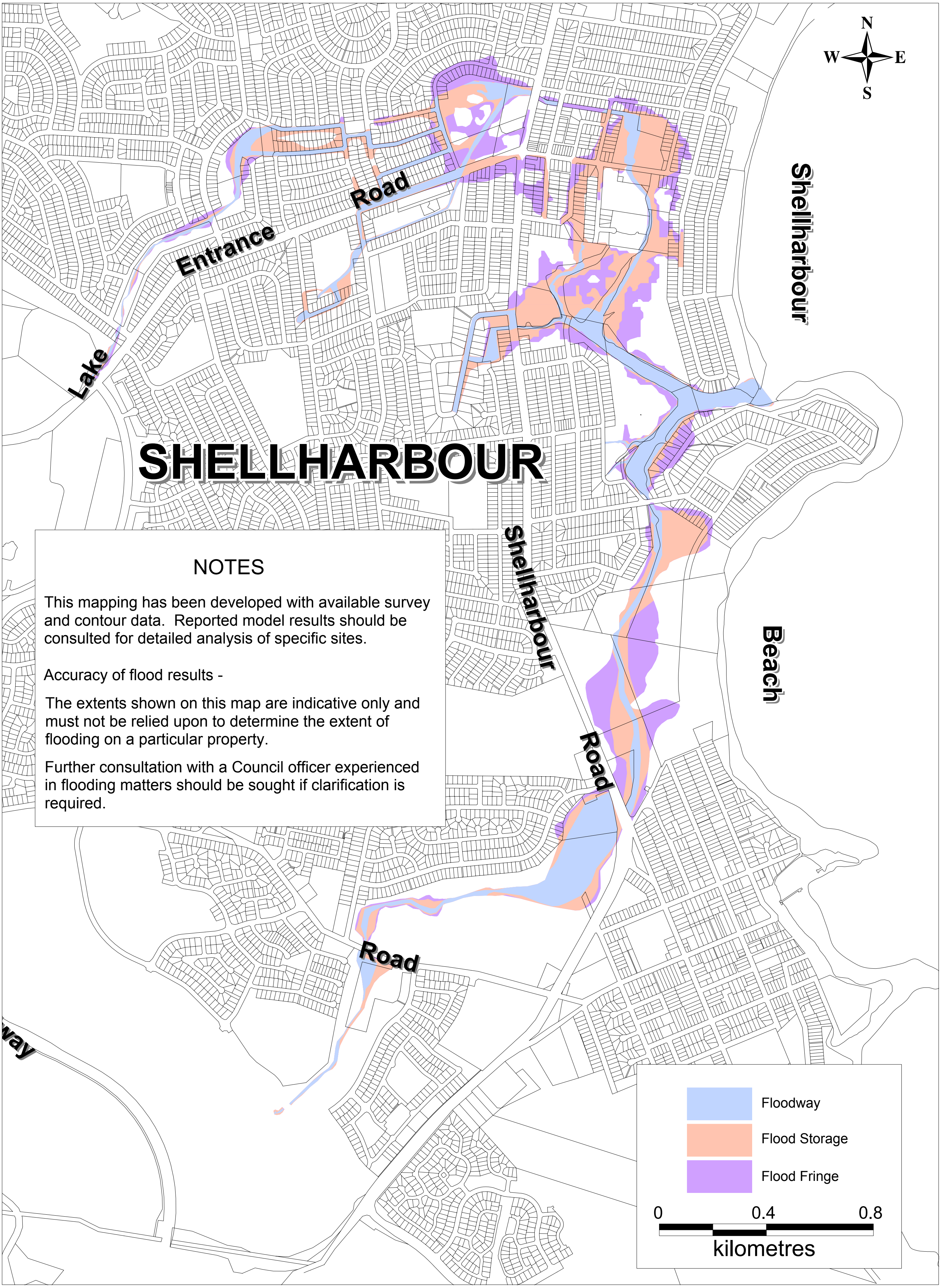
Cardno Lawson Treloar

J1959/R1974/V5
January 2006

Elliot Lake - Little Lake
Flood Study

FIGURE 36
5% AEP HYDRAULIC CATEGORIES
FINAL CATCHMENT CONDITIONS

J1959/Figures V5/Fig_36_5%_Hyd_Cat.Wor



NOTES

This mapping has been developed with available survey and contour data. Reported model results should be consulted for detailed analysis of specific sites.

Accuracy of flood results -

The extents shown on this map are indicative only and must not be relied upon to determine the extent of flooding on a particular property.

Further consultation with a Council officer experienced in flooding matters should be sought if clarification is required.

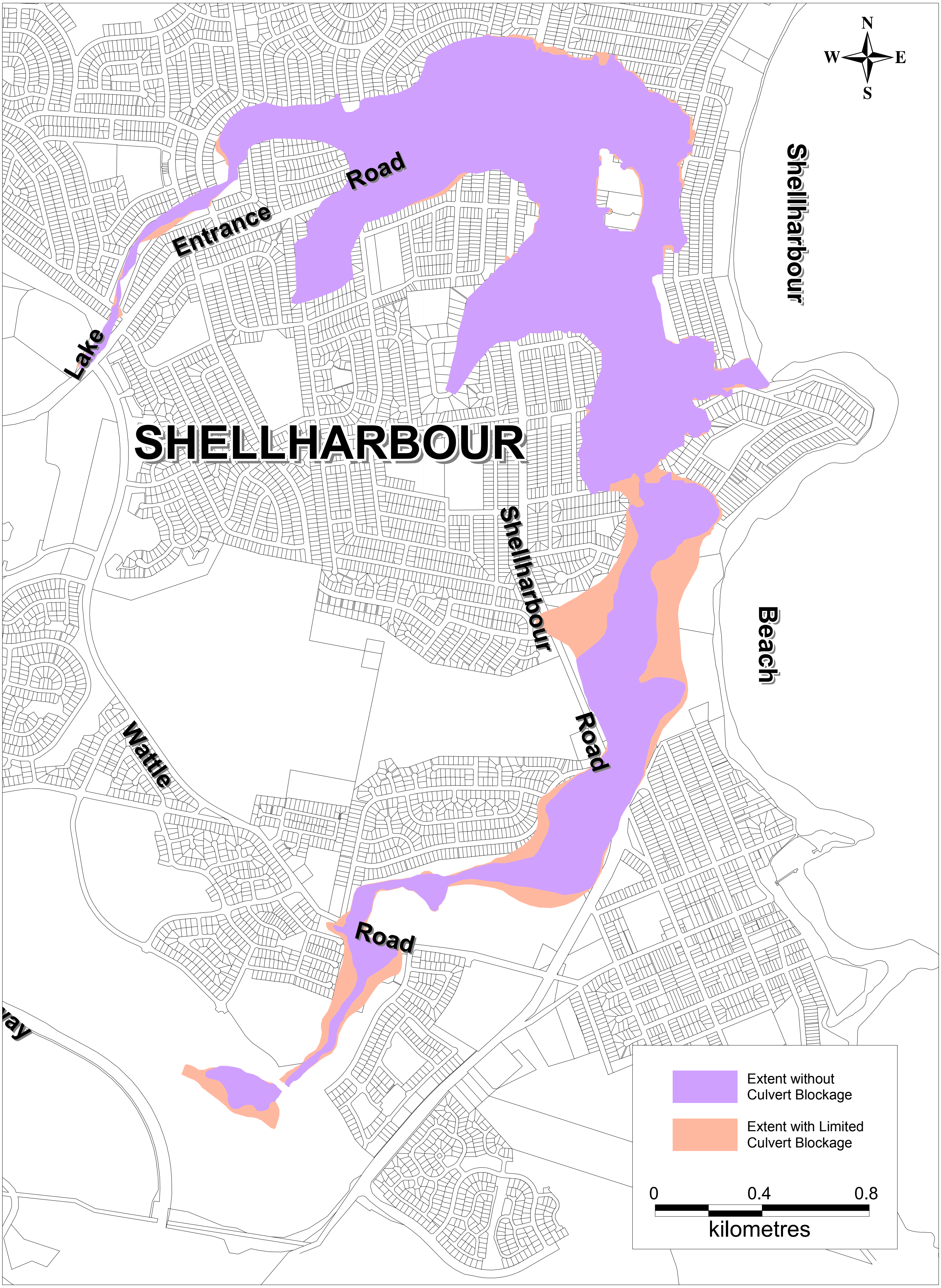
Cardno Lawson Treloar

J1959/R1974/V5
January 2006

Elliot Lake - Little Lake Flood Study

FIGURE 37
20% AEP HYDRAULIC CATEGORIES
FINAL CATCHMENT CONDITIONS

J1959/Figures V5/Fig_37_20%_Hyd_Cat.Wor



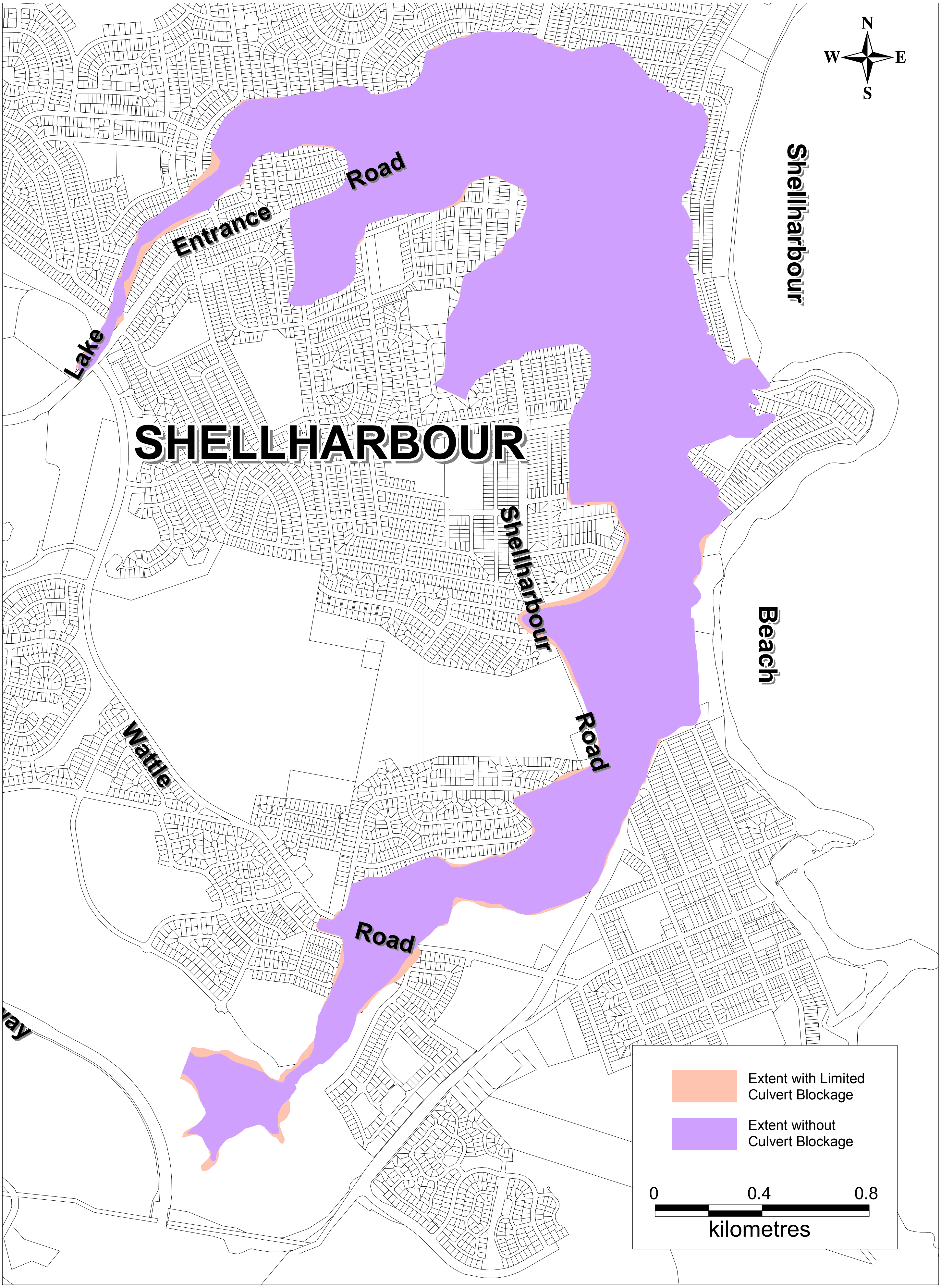
Cardno Lawson Treloar

J1959/R1974/V5
January 2006

Elliot Lake - Little Lake Flood Study

FIGURE 38
1% AEP EVENT CULVERT BLOCKAGE
FINAL CATCHMENT CONDITIONS

J1959/Figures V5/Fig_38_1%_Blockage.Wor



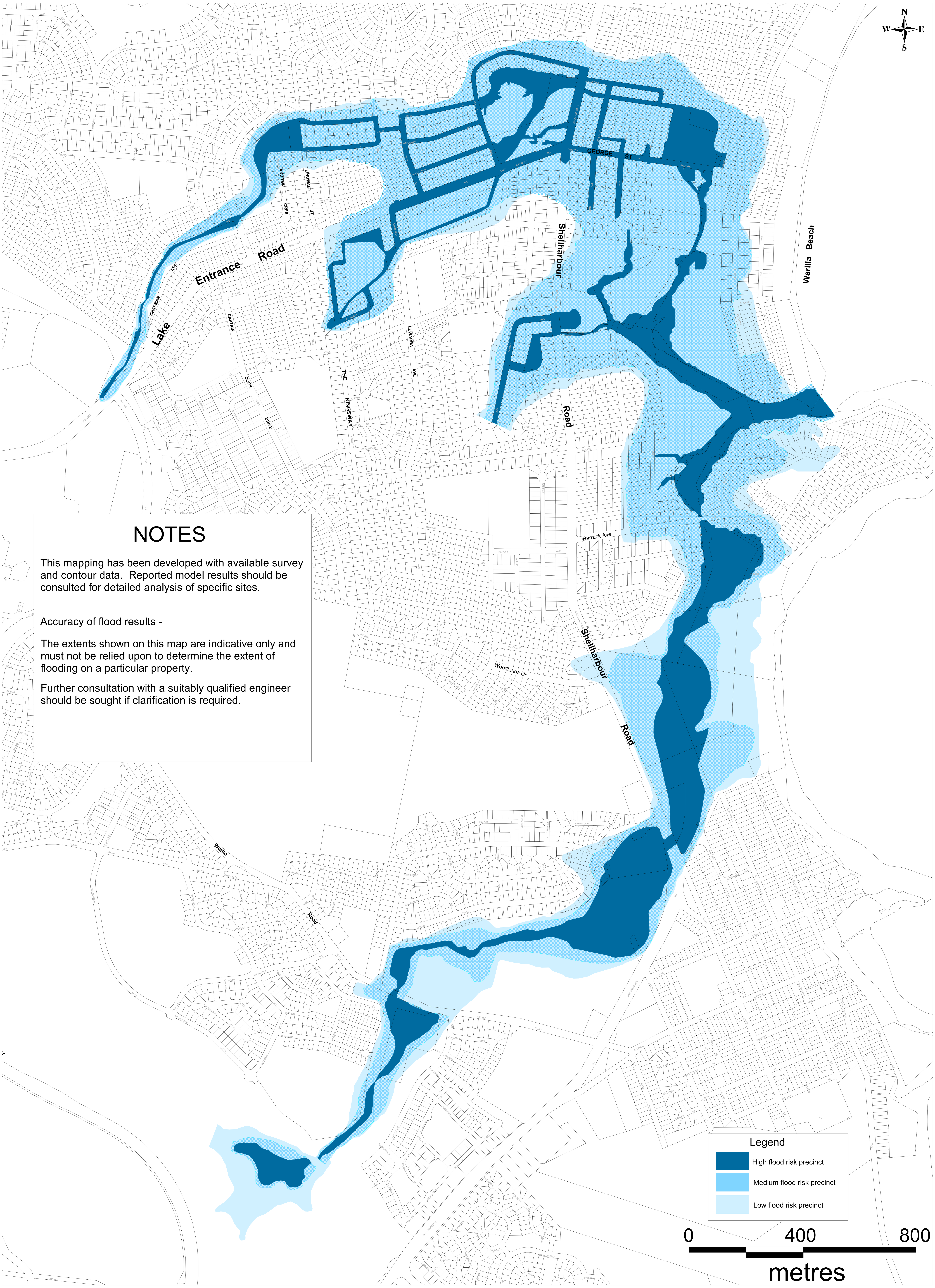
Cardno Lawson Treloar

J1959/R1974/V5
January 2006

Elliot Lake - Little Lake
Flood Study

FIGURE 39
PMF EVENT CULVERT BLOCKAGE
FINAL CATCHMENT CONDITIONS

J1959/Figures V5/Fig_39_PMF_Blockage.Wor



NOTES

This mapping has been developed with available survey and contour data. Reported model results should be consulted for detailed analysis of specific sites.

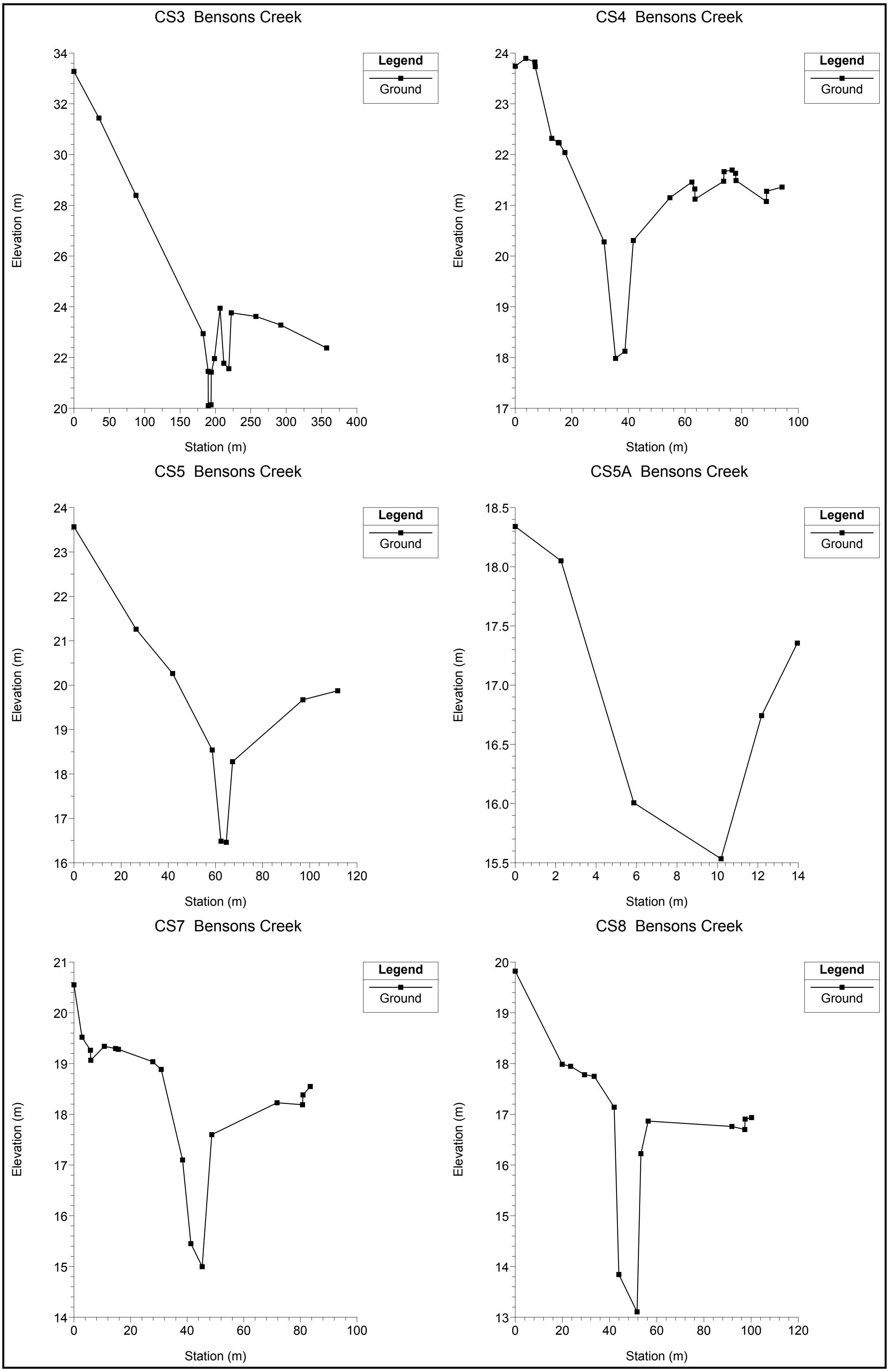
Accuracy of flood results -

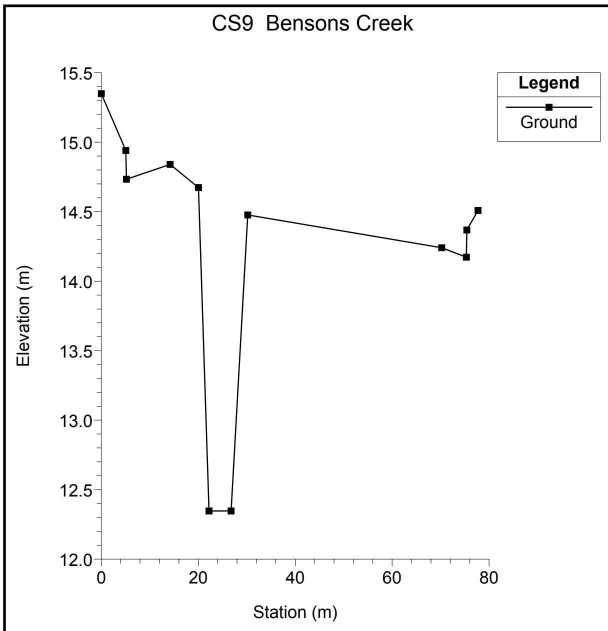
The extents shown on this map are indicative only and must not be relied upon to determine the extent of flooding on a particular property.

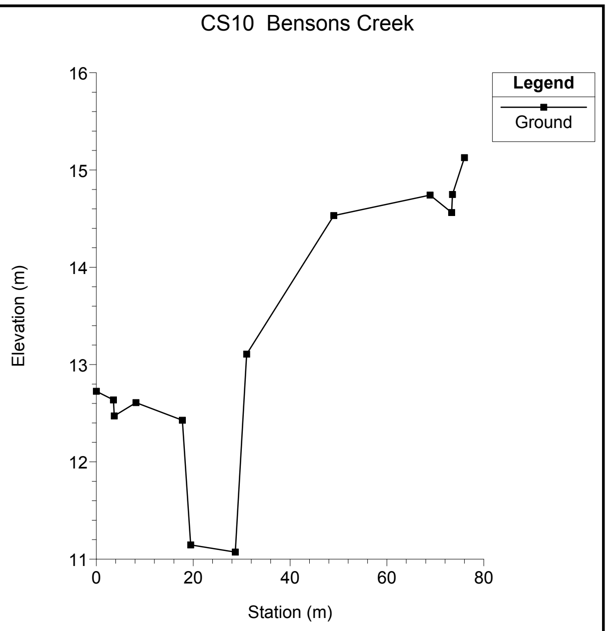
Further consultation with a suitably qualified engineer should be sought if clarification is required.

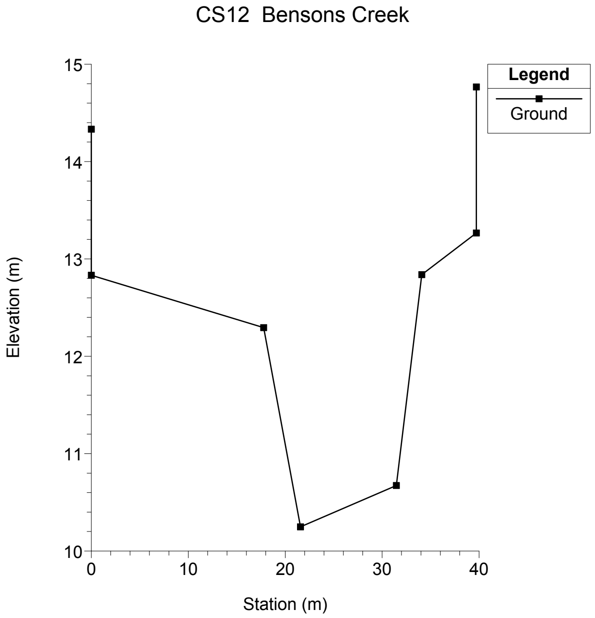
APPENDIX A

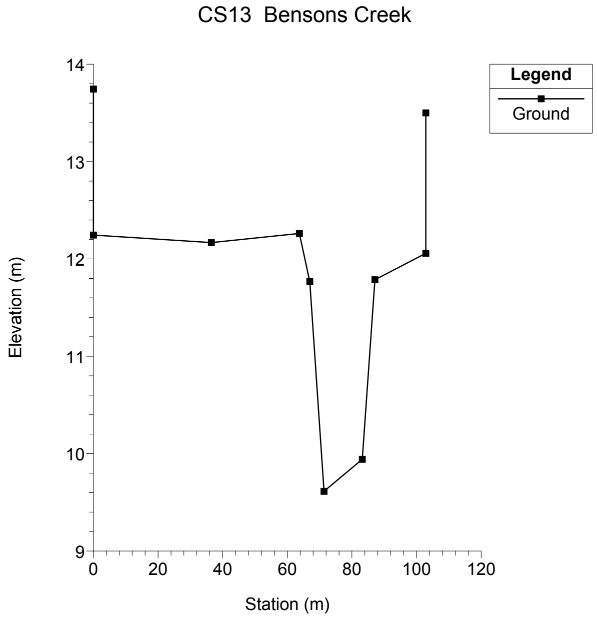
SURVEY DATA

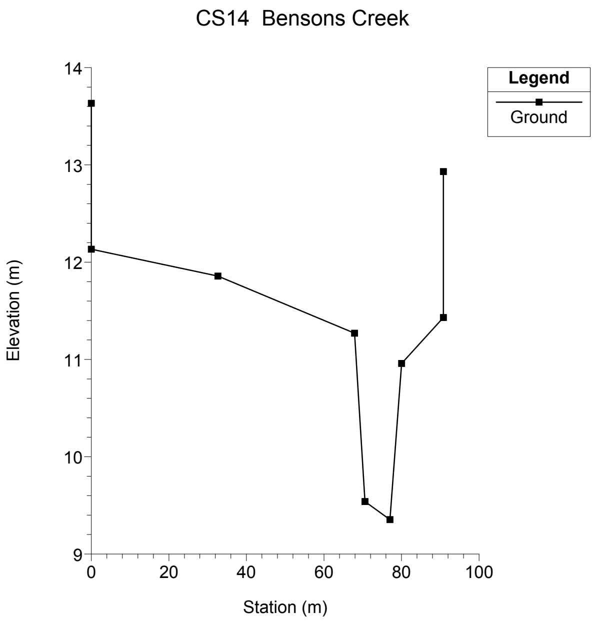

CS3 Bensons Creek
CS4 Bensons Creek
CS5 Bensons Creek
CS5A Bensons Creek
CS7 Bensons Creek
CS8 Bensons Creek
Legend
Ground
Elevation (m)
Station (m)


CS9 Bensons Creek
Legend
Ground
Elevation (m)
Station (m)
15.5
15.0
14.5
14.0
13.5
13.0
12.5
12.0
0
20
40
60
80

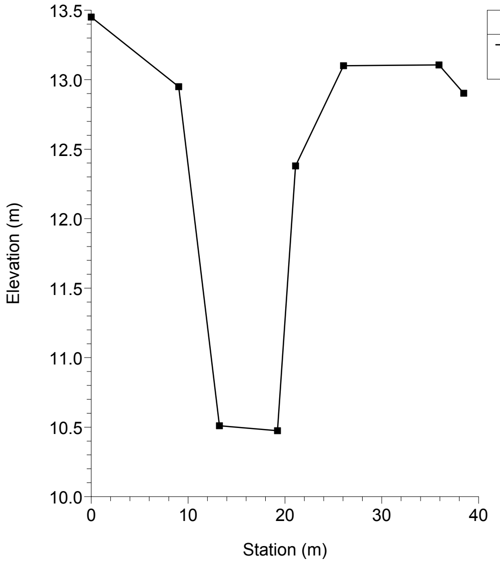

CS10 Bensons Creek
Legend
Ground
Elevation (m)
Station (m)
16
15
14
13
12
11
0
20
40
60
80


CS12 Bensons Creek
Legend
Ground
Elevation (m)
Station (m)
15
14
13
12
11
10
0
10
20
30
40

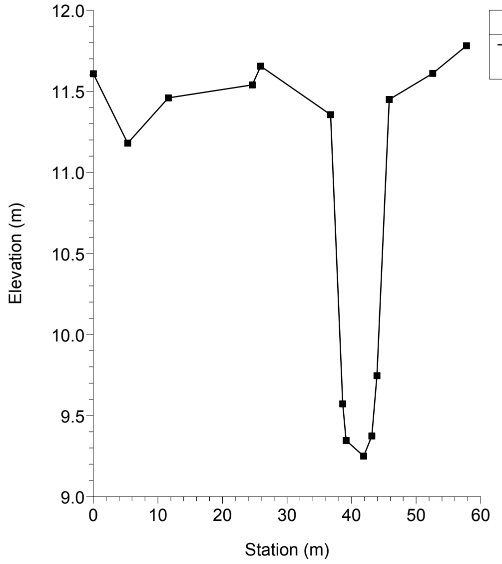

CS13 Bensons Creek
Legend
Ground
Elevation (m)
Station (m)
14
13
12
11
10
9
0
20
40
60
80
100
120


CS14 Bensons Creek
Legend
Ground
Elevation (m)
Station (m)
14
13
12
11
10
9
0
20
40
60
80
100

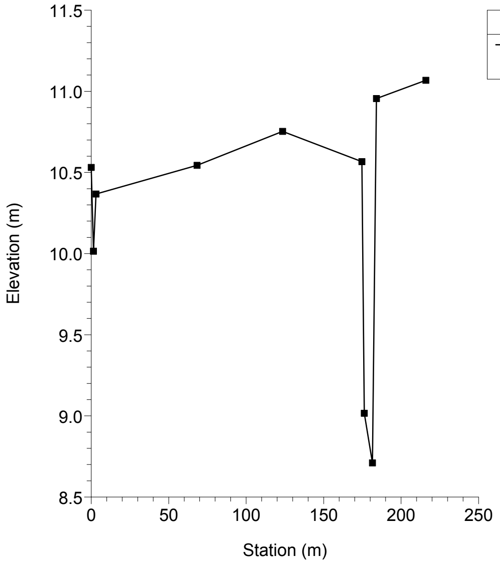
CS50 Bensons Tributary



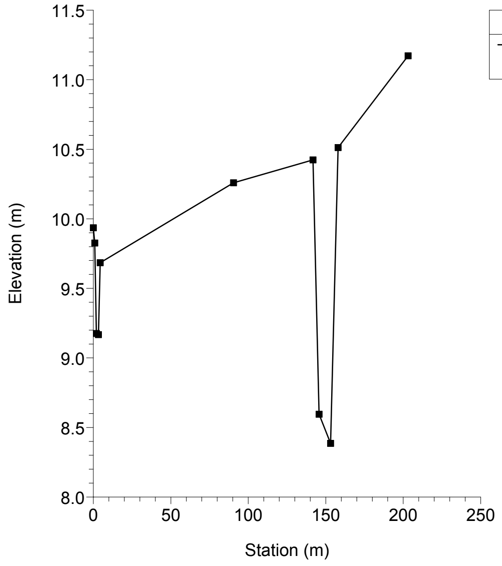
CS52A Bensons Tributary



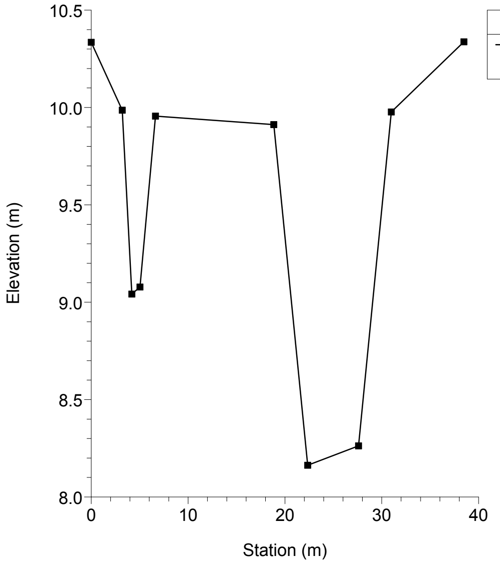
CS53 Bensons Tributary



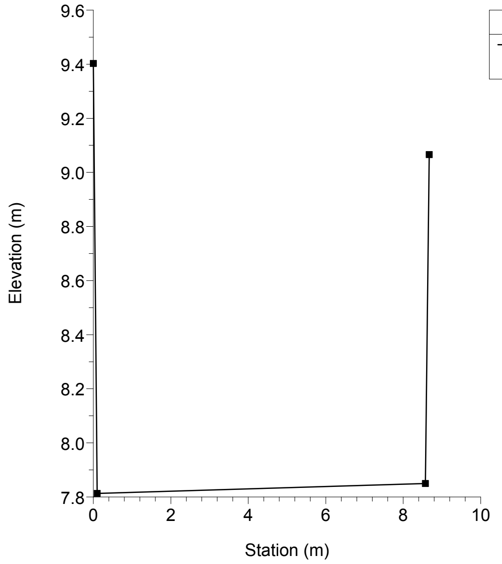
CS54 Bensons Tributary

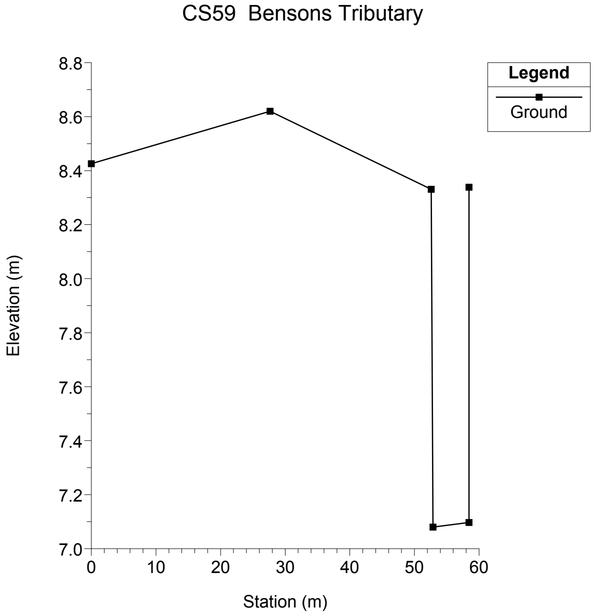


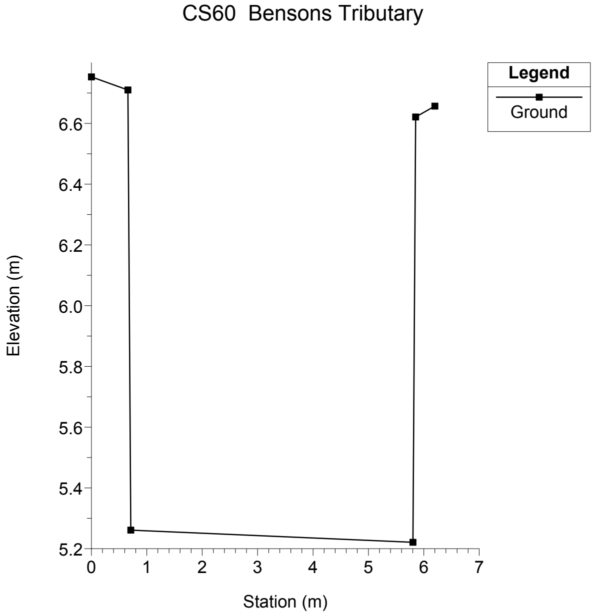
CS55 Bensons Tributary

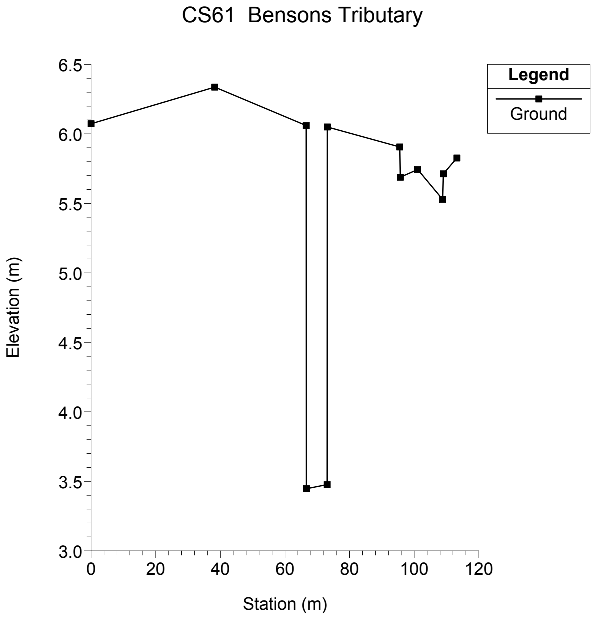


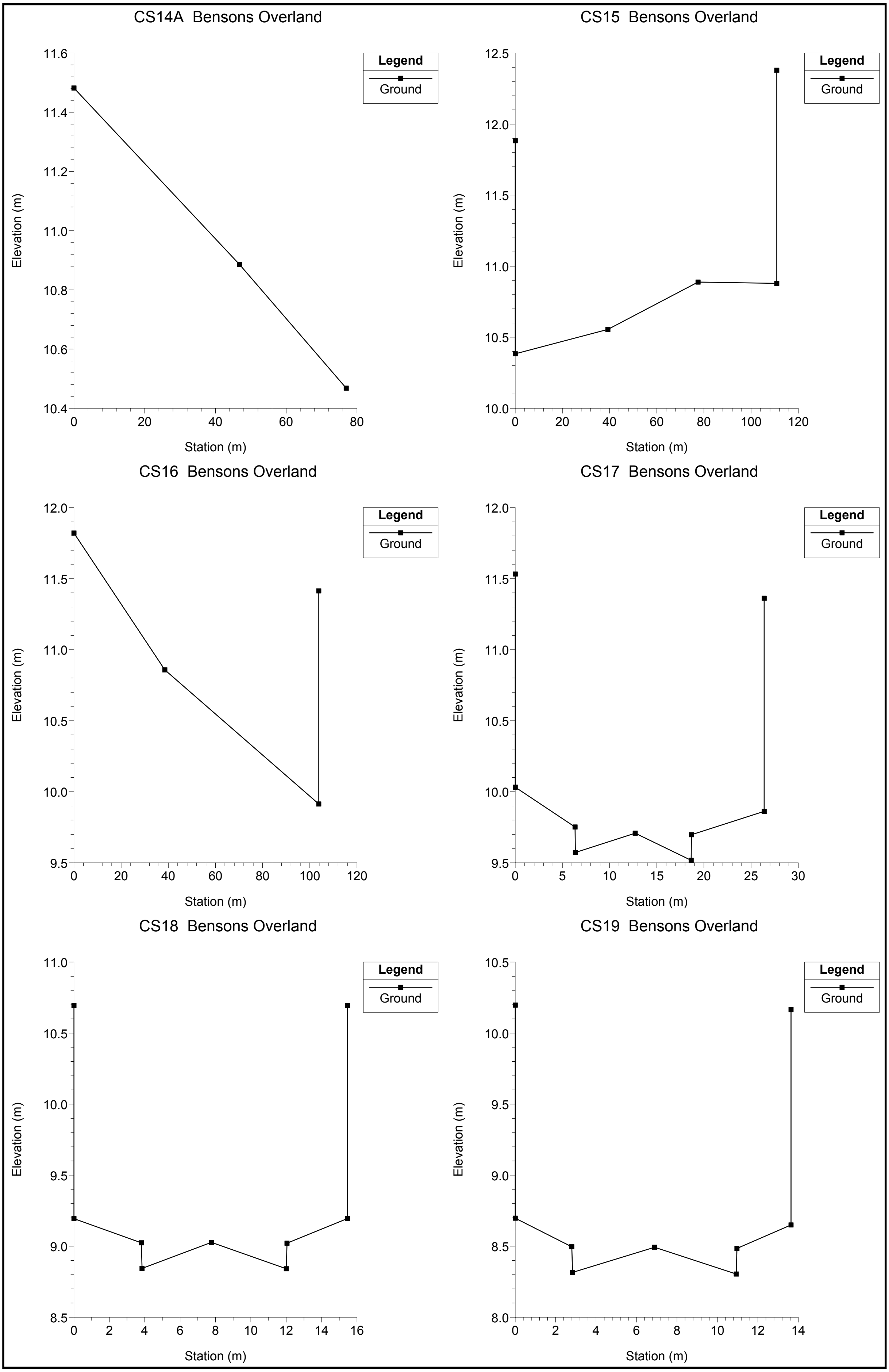
CS58 Bensons Tributary

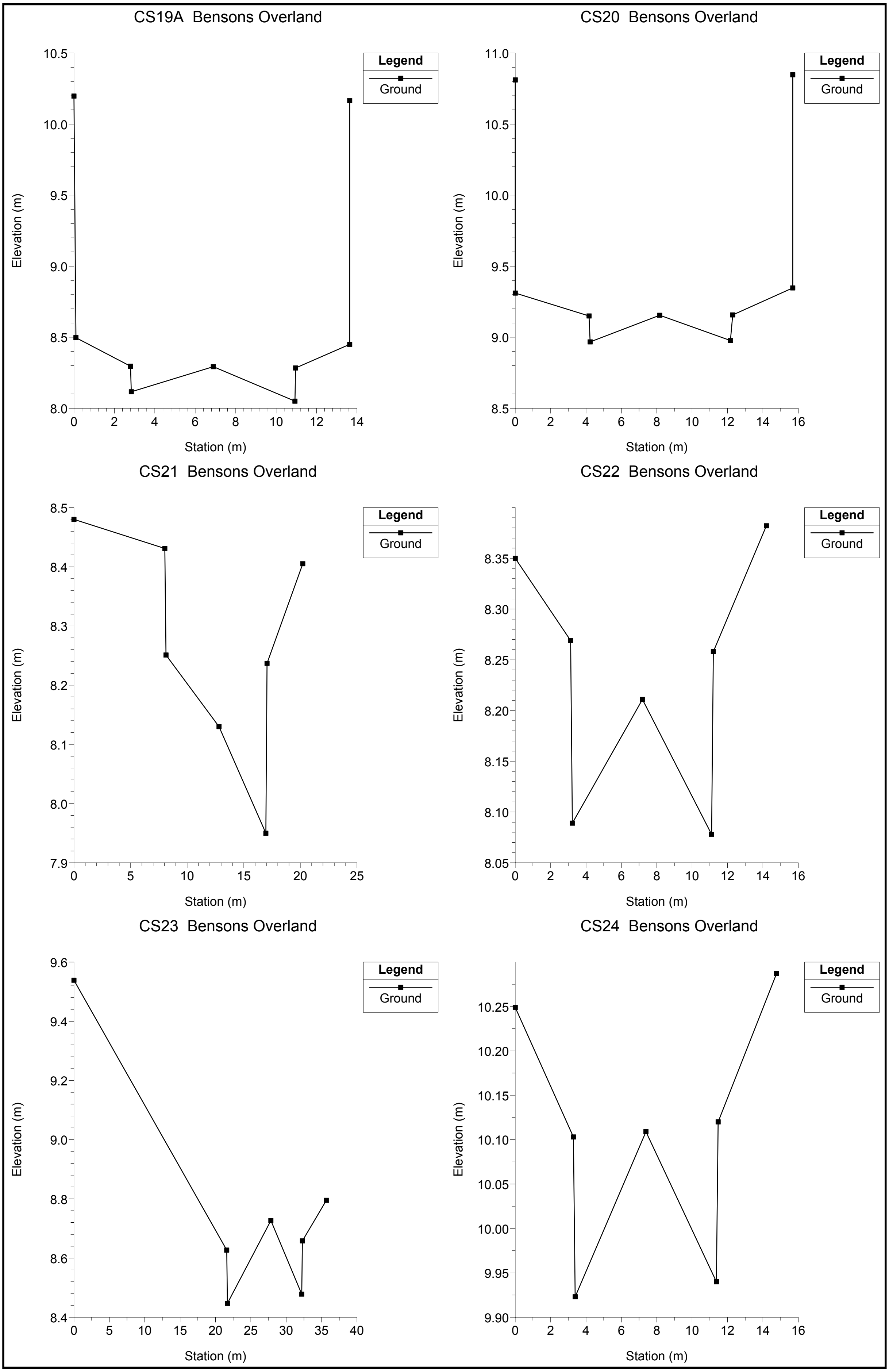


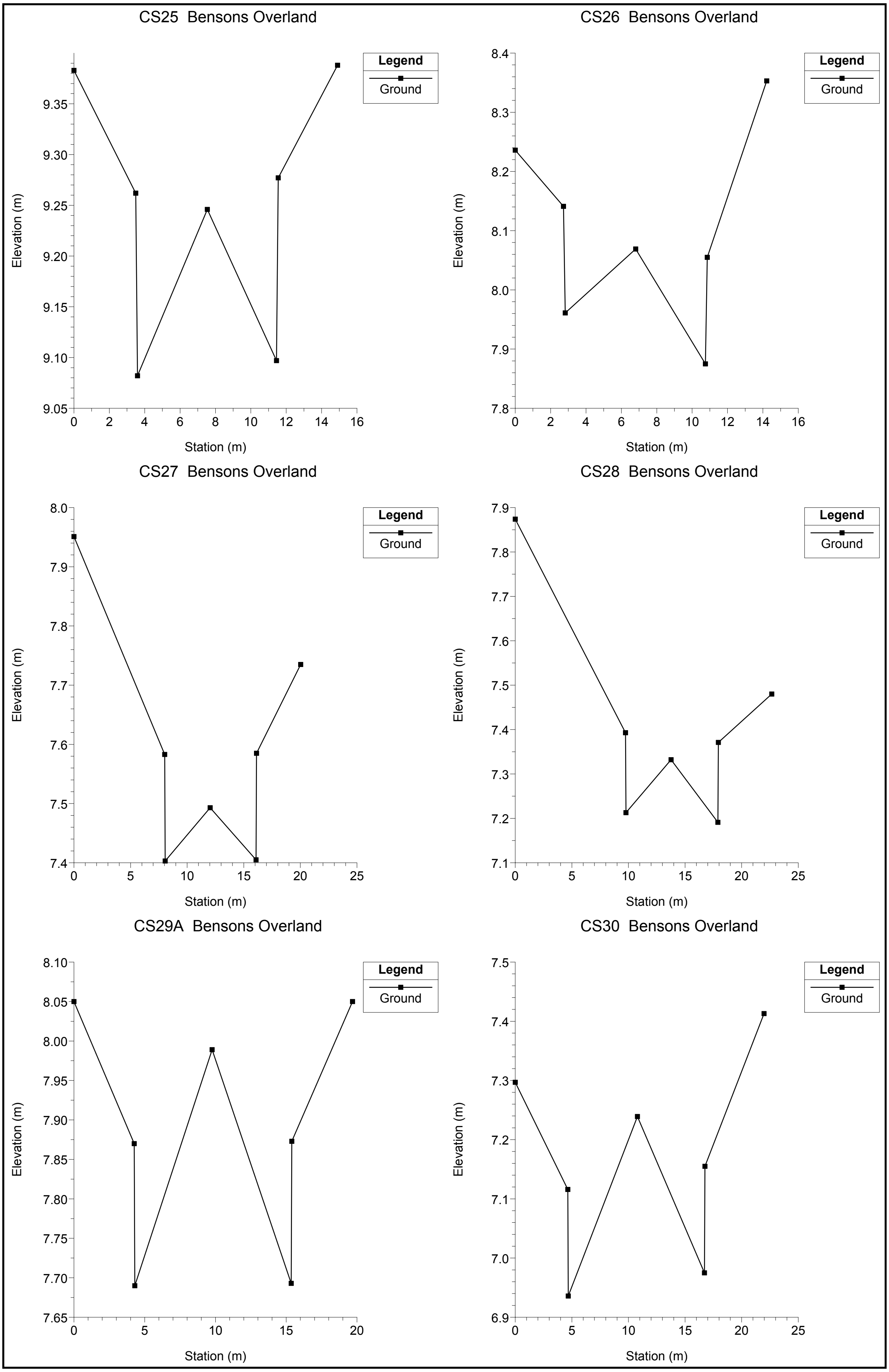

CS59 Bensons Tributary
Legend
Ground
Elevation (m)
Station (m)


CS60 Bensons Tributary
Legend
Ground
Elevation (m)
Station (m)

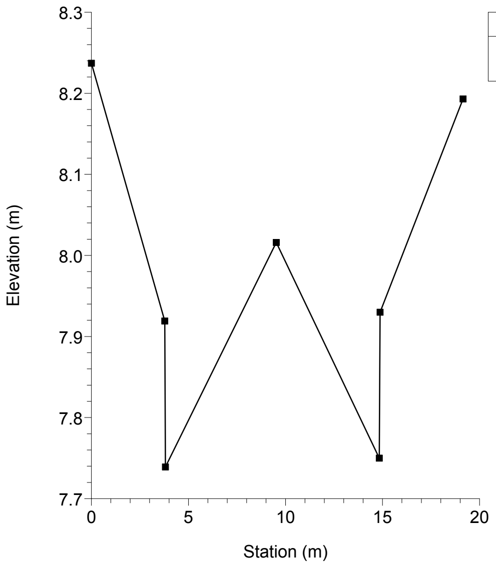

CS61 Bensons Tributary
Legend
Ground
Elevation (m)
Station (m)


CS14A Bensons Overland
CS15 Bensons Overland
CS16 Bensons Overland
CS17 Bensons Overland
CS18 Bensons Overland
CS19 Bensons Overland
Legend
Ground
Elevation (m)
Station (m)

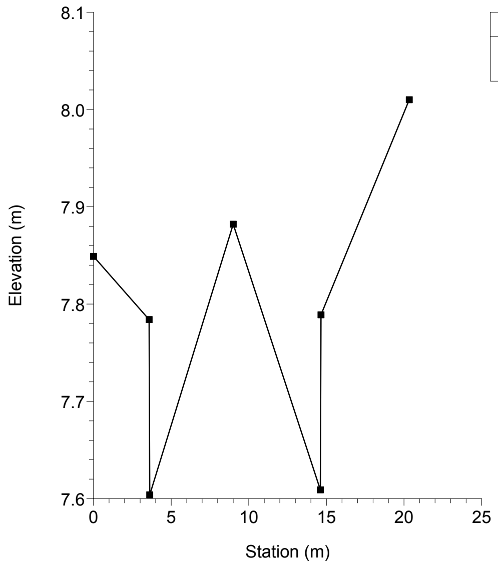

CS19A Bensons Overland
CS20 Bensons Overland
CS21 Bensons Overland
CS22 Bensons Overland
CS23 Bensons Overland
CS24 Bensons Overland
Legend
Ground
Elevation (m)
Station (m)


CS25 Bensons Overland
CS26 Bensons Overland
CS27 Bensons Overland
CS28 Bensons Overland
CS29A Bensons Overland
CS30 Bensons Overland
Legend
Ground
Elevation (m)
Station (m)

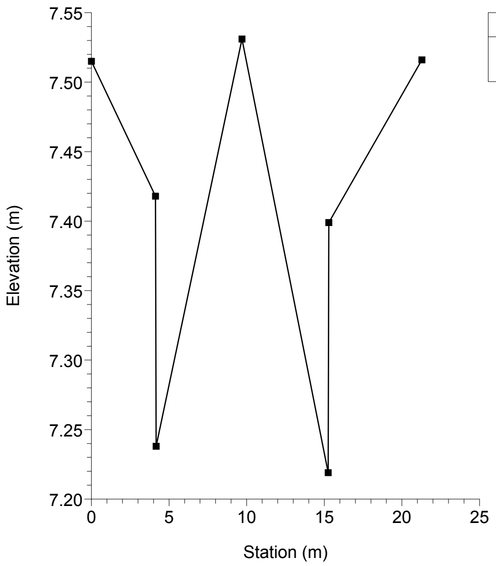
CS31 Bensons Overland



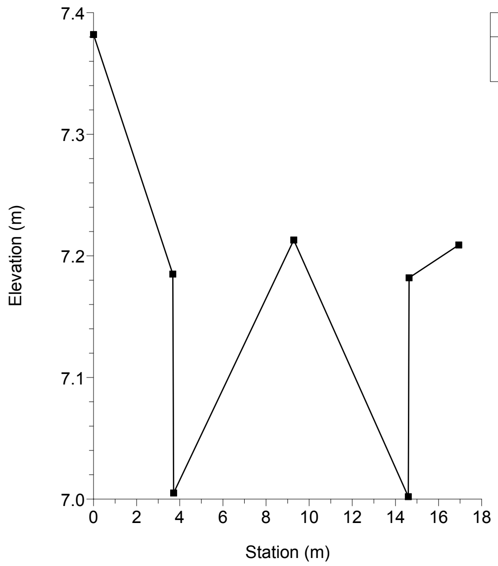
CS32 Bensons Overland



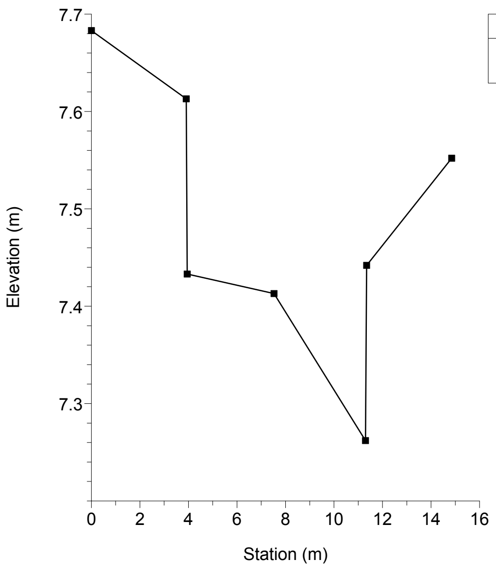
CS33 Bensons Overland



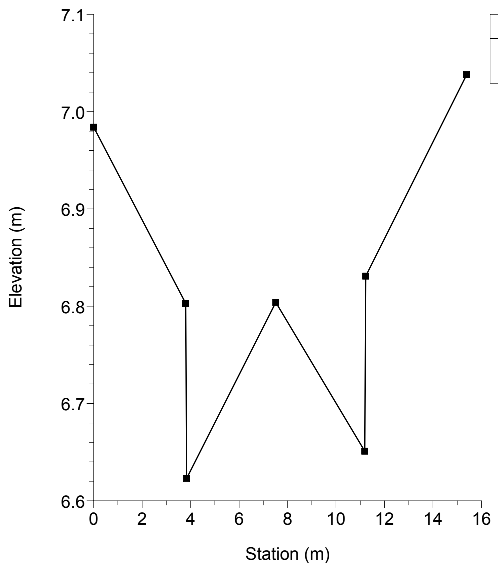
CS34 Bensons Overland

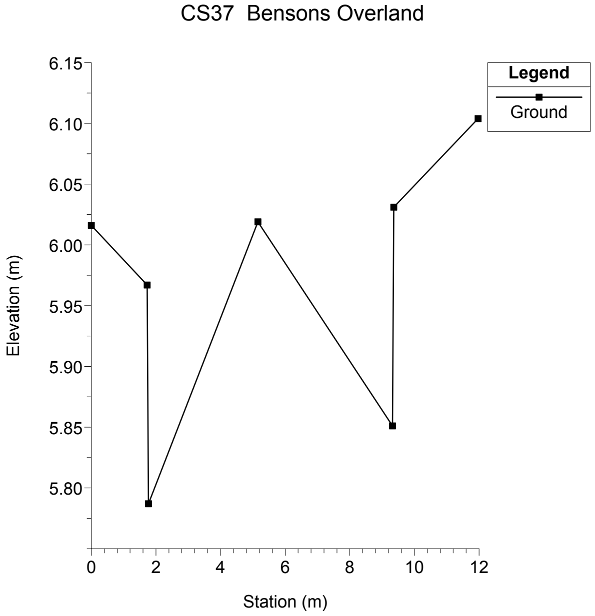


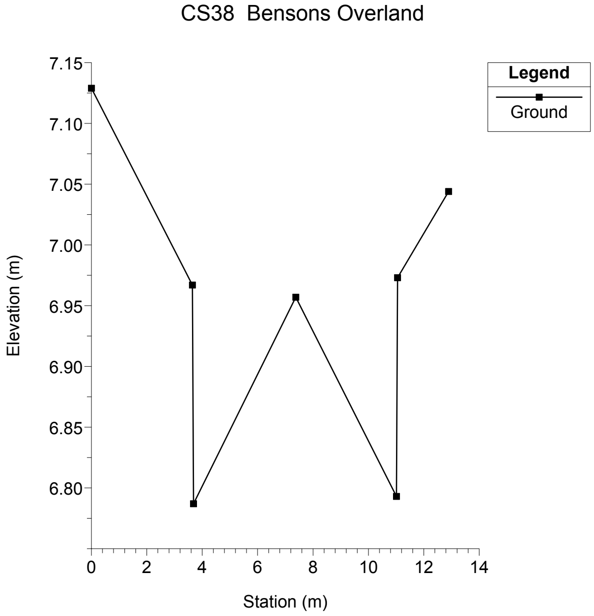
CS35 Bensons Overland

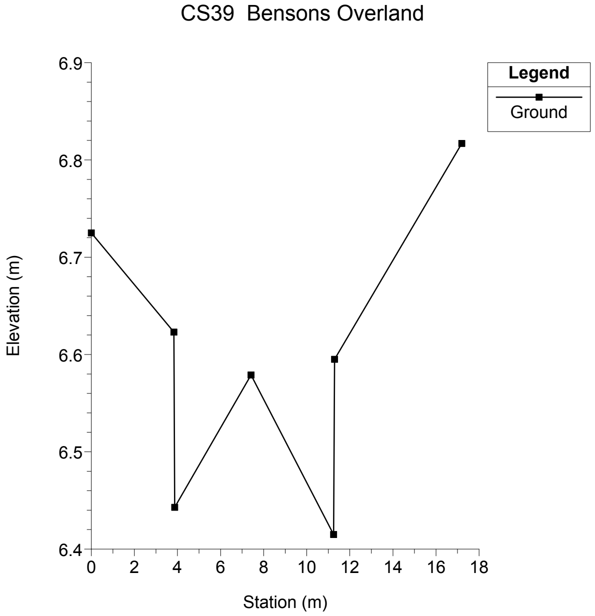


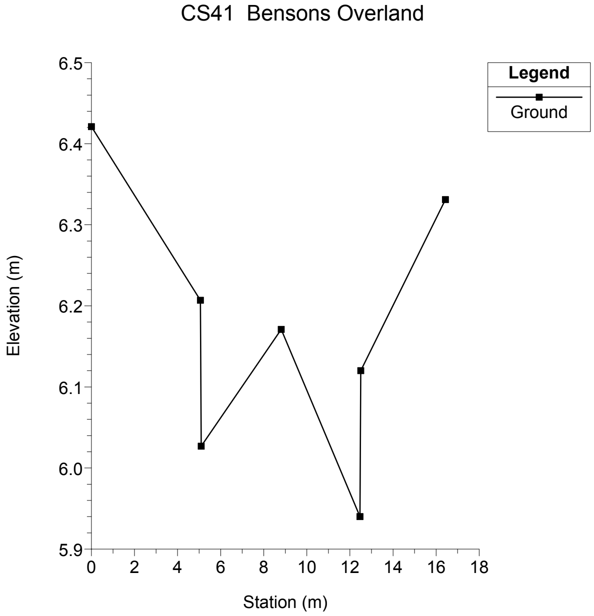
CS36 Bensons Overland

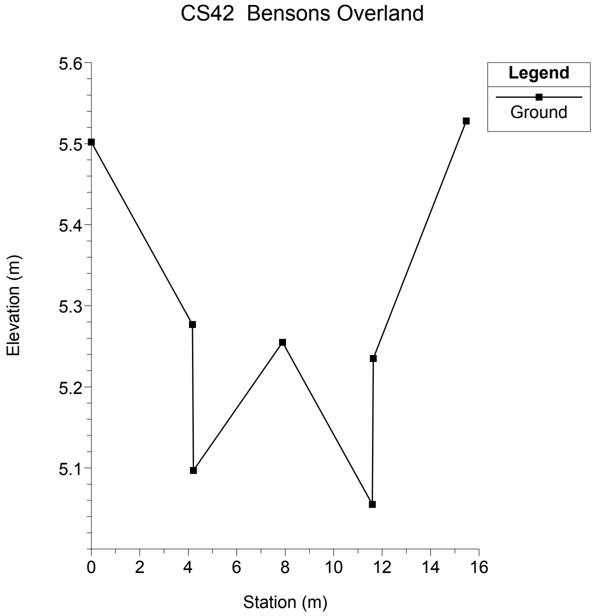


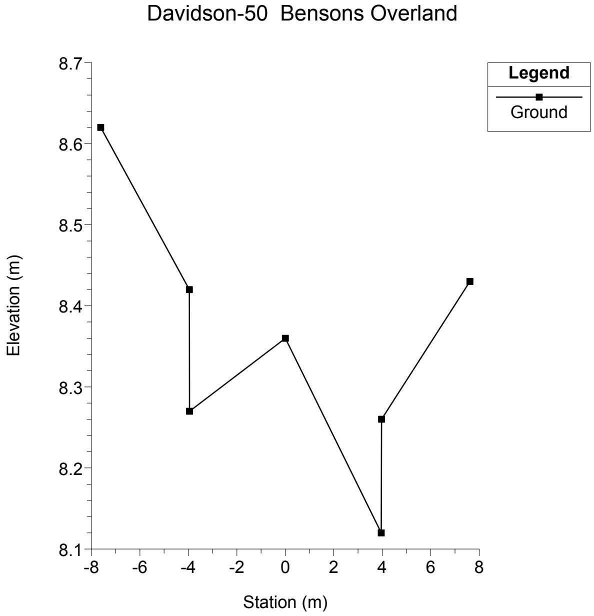

CS37 Bensons Overland
Legend
Ground
Elevation (m)
Station (m)

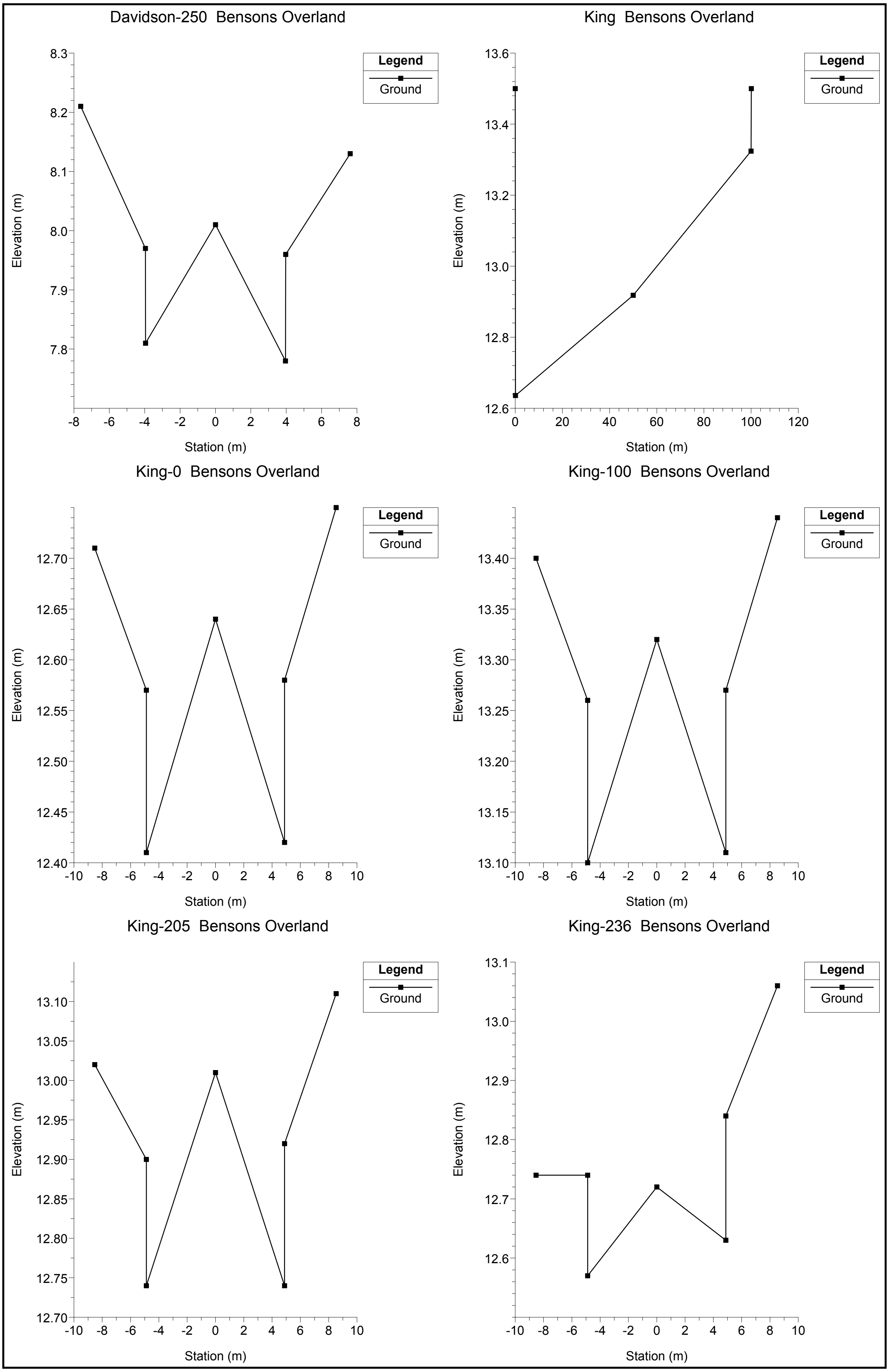

CS38 Bensons Overland
Legend
Ground
Elevation (m)
Station (m)


CS39 Bensons Overland
Legend
Ground
Elevation (m)
Station (m)

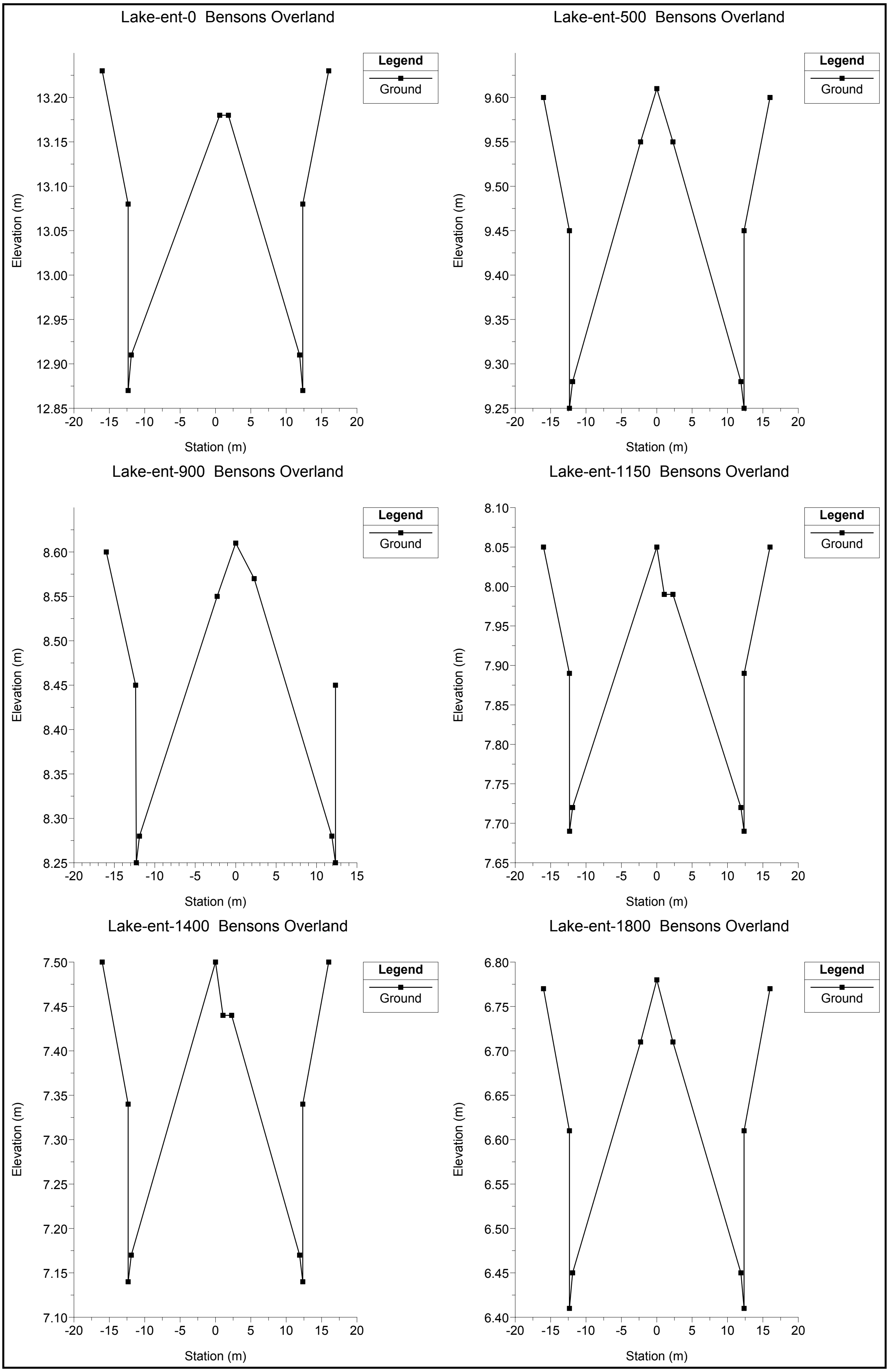

CS41 Bensons Overland
Legend
Ground
Elevation (m)
Station (m)

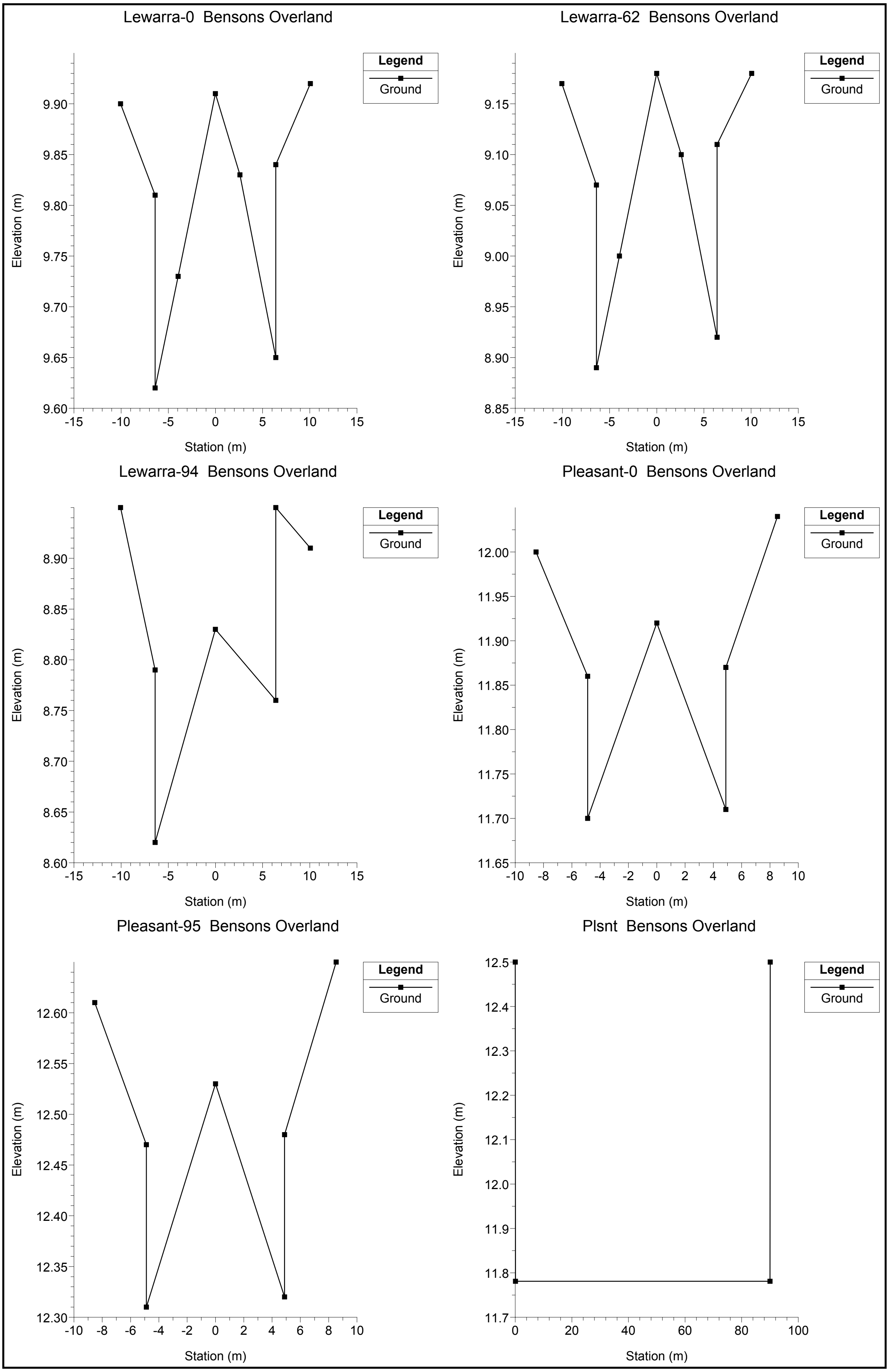

CS42 Bensons Overland
Legend
Ground
Elevation (m)
Station (m)

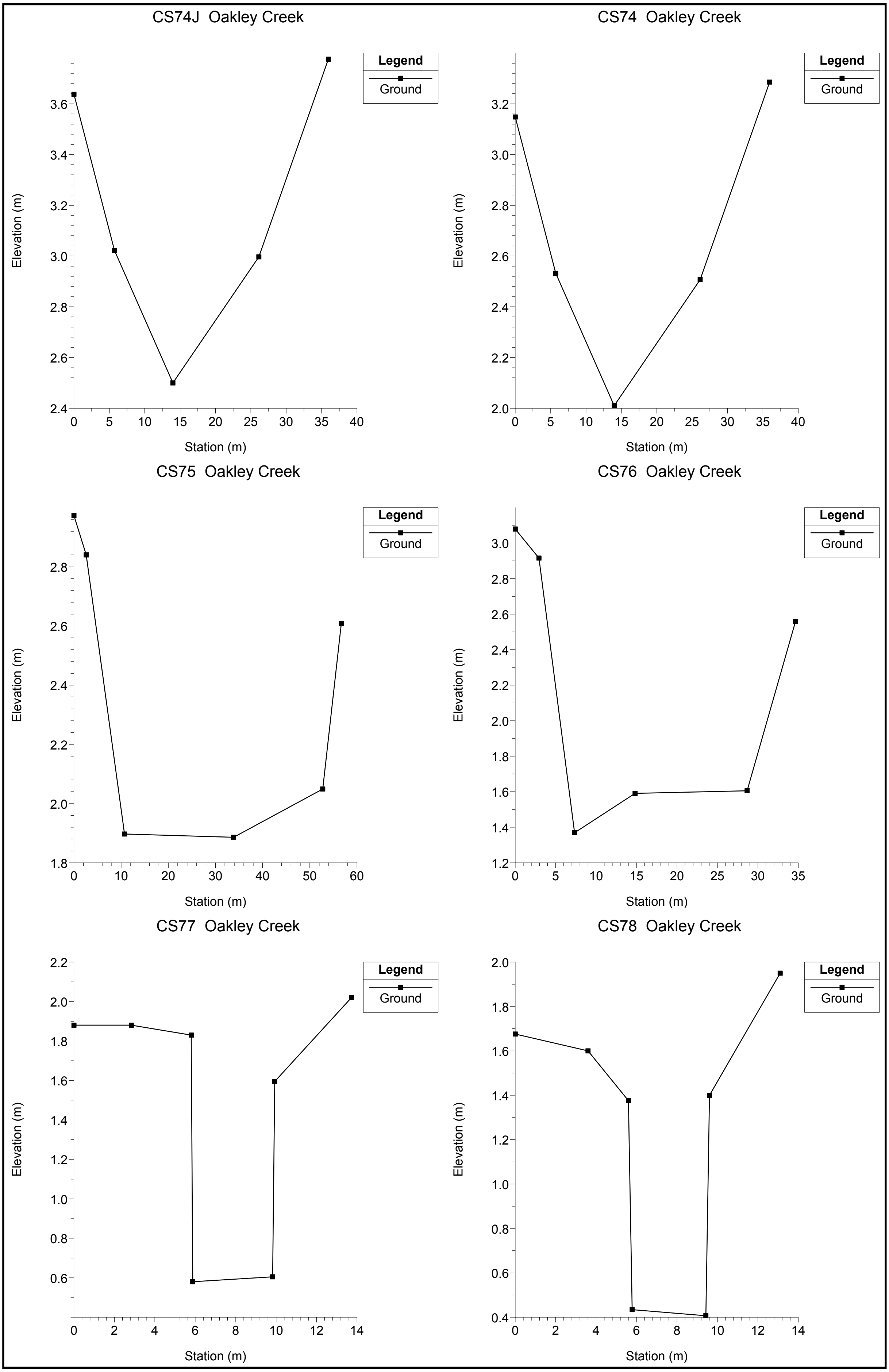

Davidson-50 Bensons Overland
Legend
Ground
Elevation (m)
Station (m)



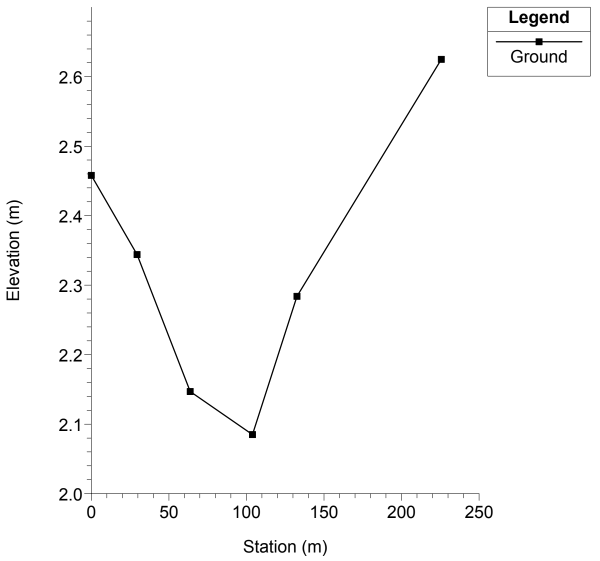
Davidson-250 Bensons Overland
King Bensons Overland
King-0 Bensons Overland
King-100 Bensons Overland
King-205 Bensons Overland
King-236 Bensons Overland
Legend
Ground
Elevation (m)
Station (m)


Lake-ent-0 Bensons Overland
Legend
Ground
Elevation (m)
Station (m)
Lake-ent-500 Bensons Overland
Lake-ent-900 Bensons Overland
Lake-ent-1150 Bensons Overland
Lake-ent-1400 Bensons Overland
Lake-ent-1800 Bensons Overland

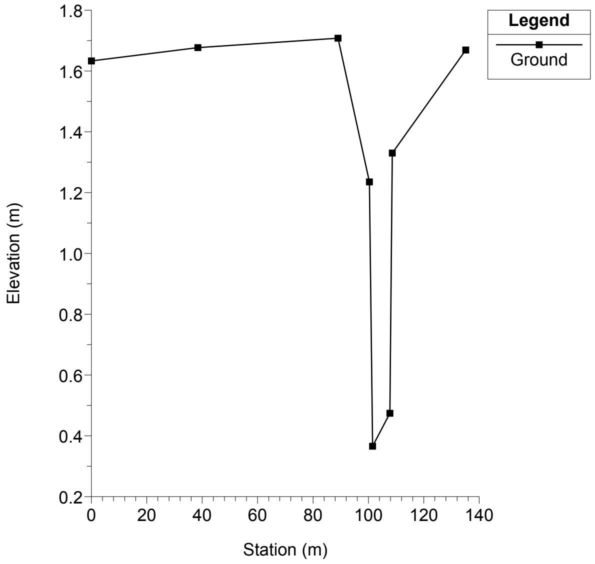

Lewarra-0 Bensons Overland
Lewarra-62 Bensons Overland
Lewarra-94 Bensons Overland
Pleasant-0 Bensons Overland
Pleasant-95 Bensons Overland
Plsnt Bensons Overland
Legend
Ground
Elevation (m)
Station (m)

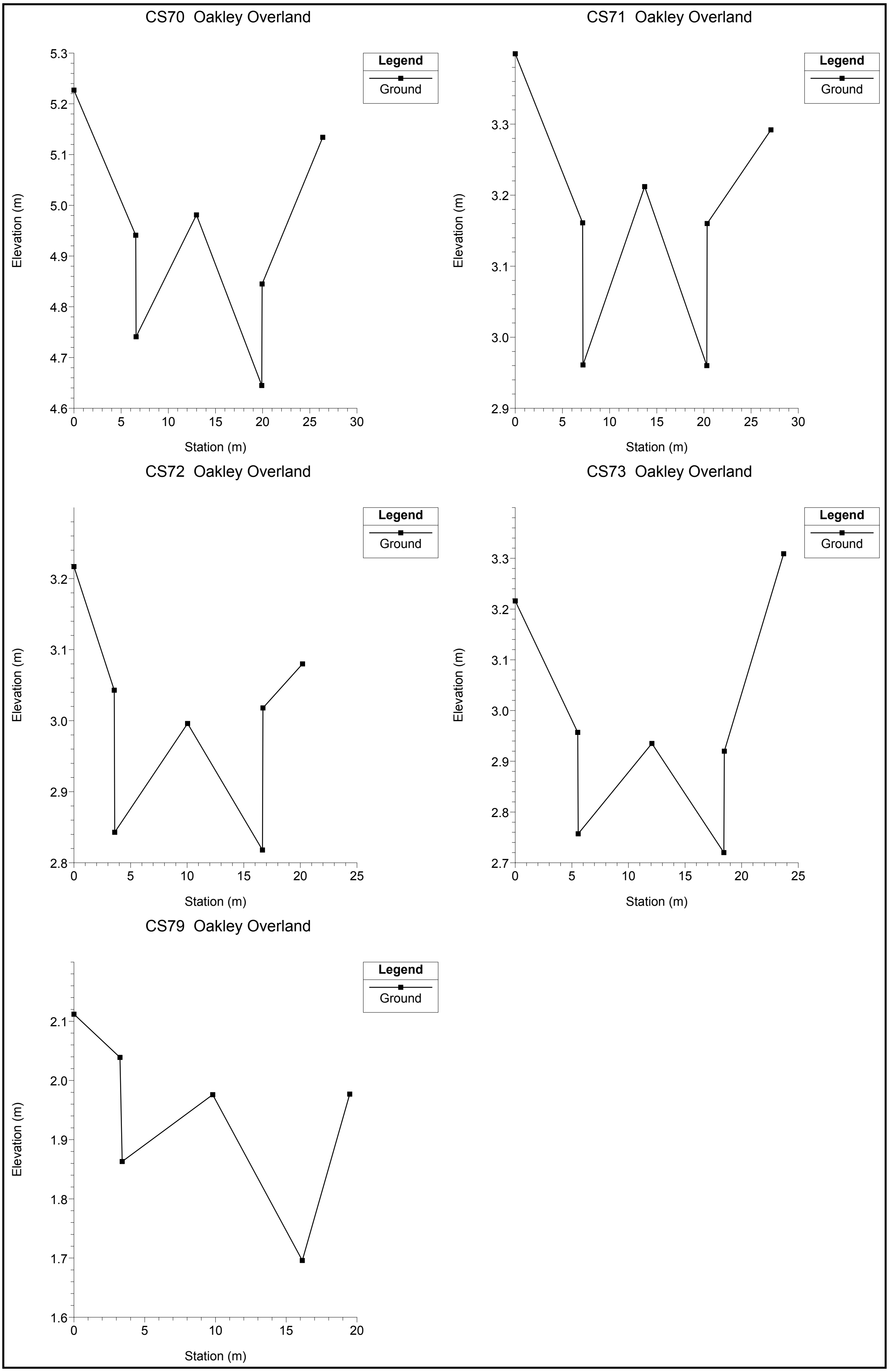

CS74J Oakley Creek
CS74 Oakley Creek
CS75 Oakley Creek
CS76 Oakley Creek
CS77 Oakley Creek
CS78 Oakley Creek
Legend
Ground
Elevation (m)
Station (m)

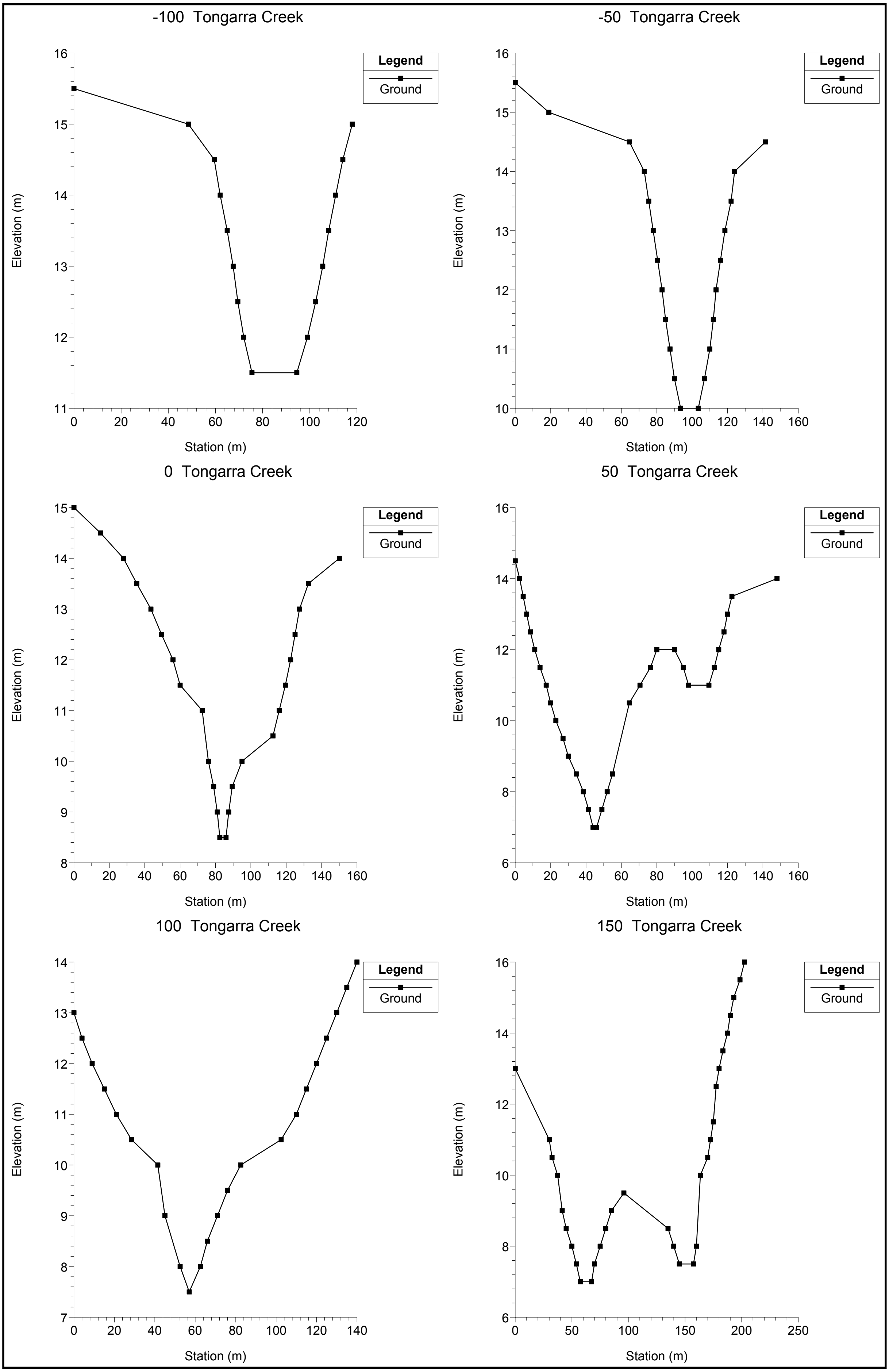
CS80 Oakley Creek

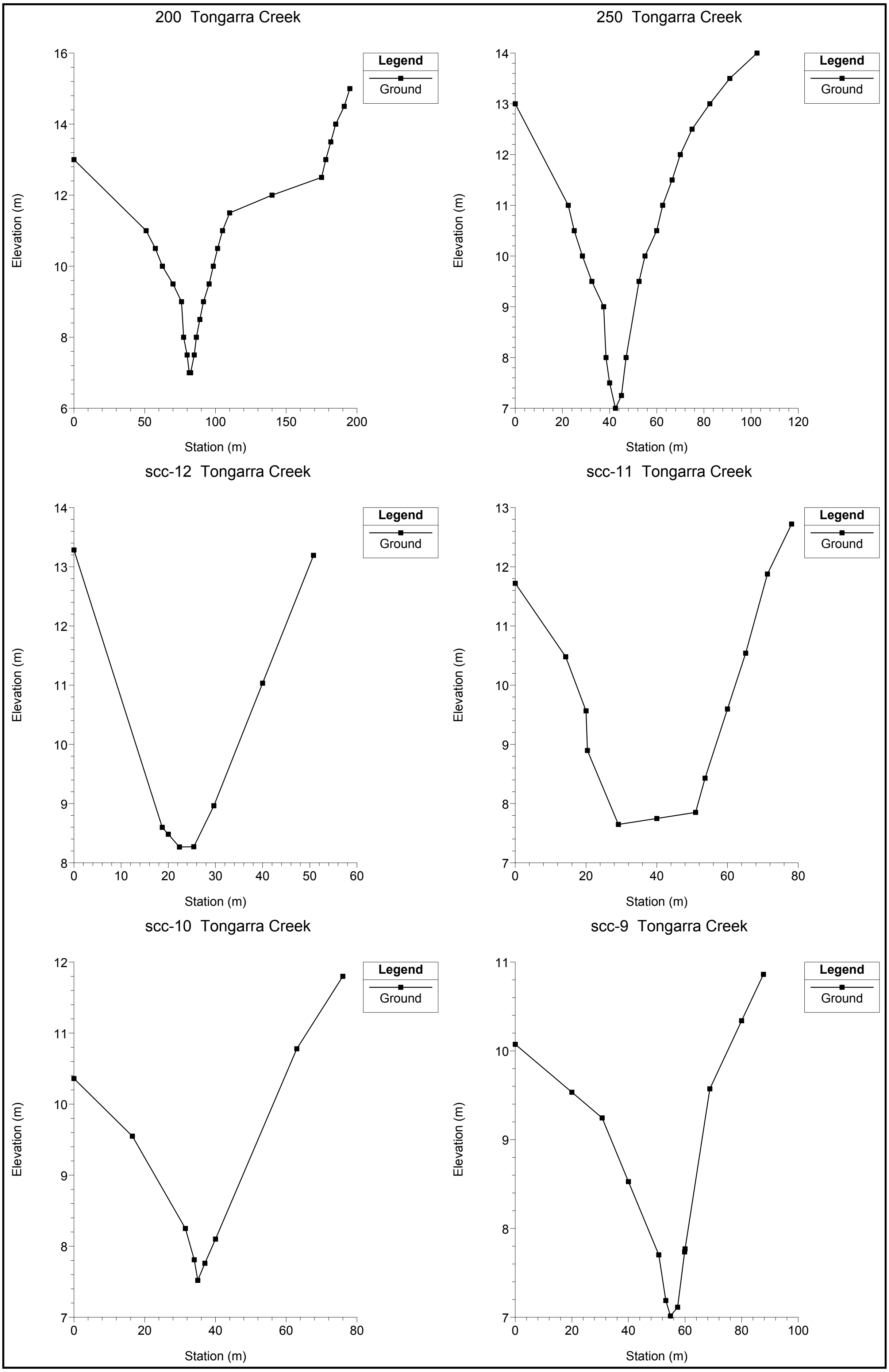


CS81 Oakley Creek

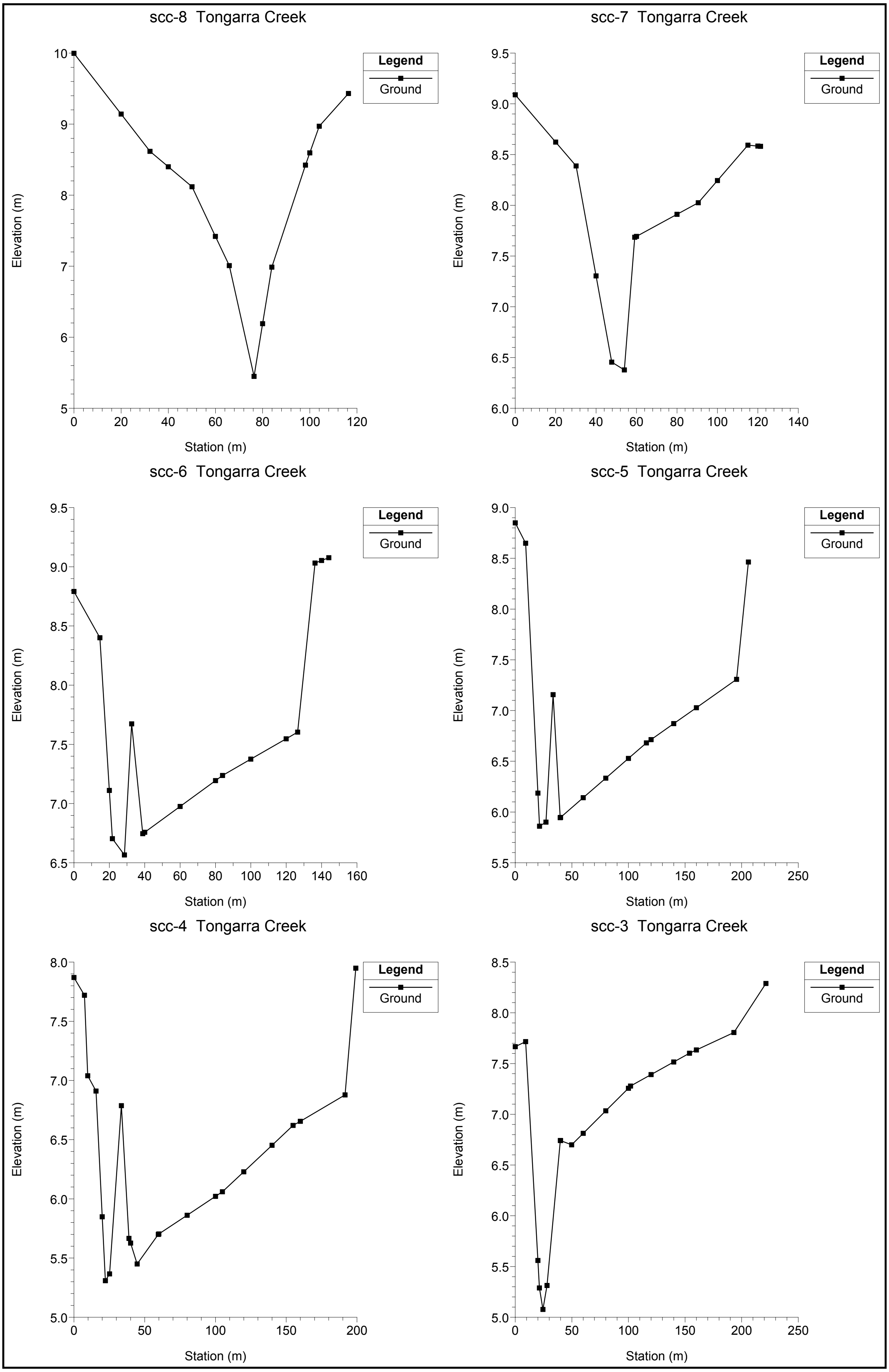


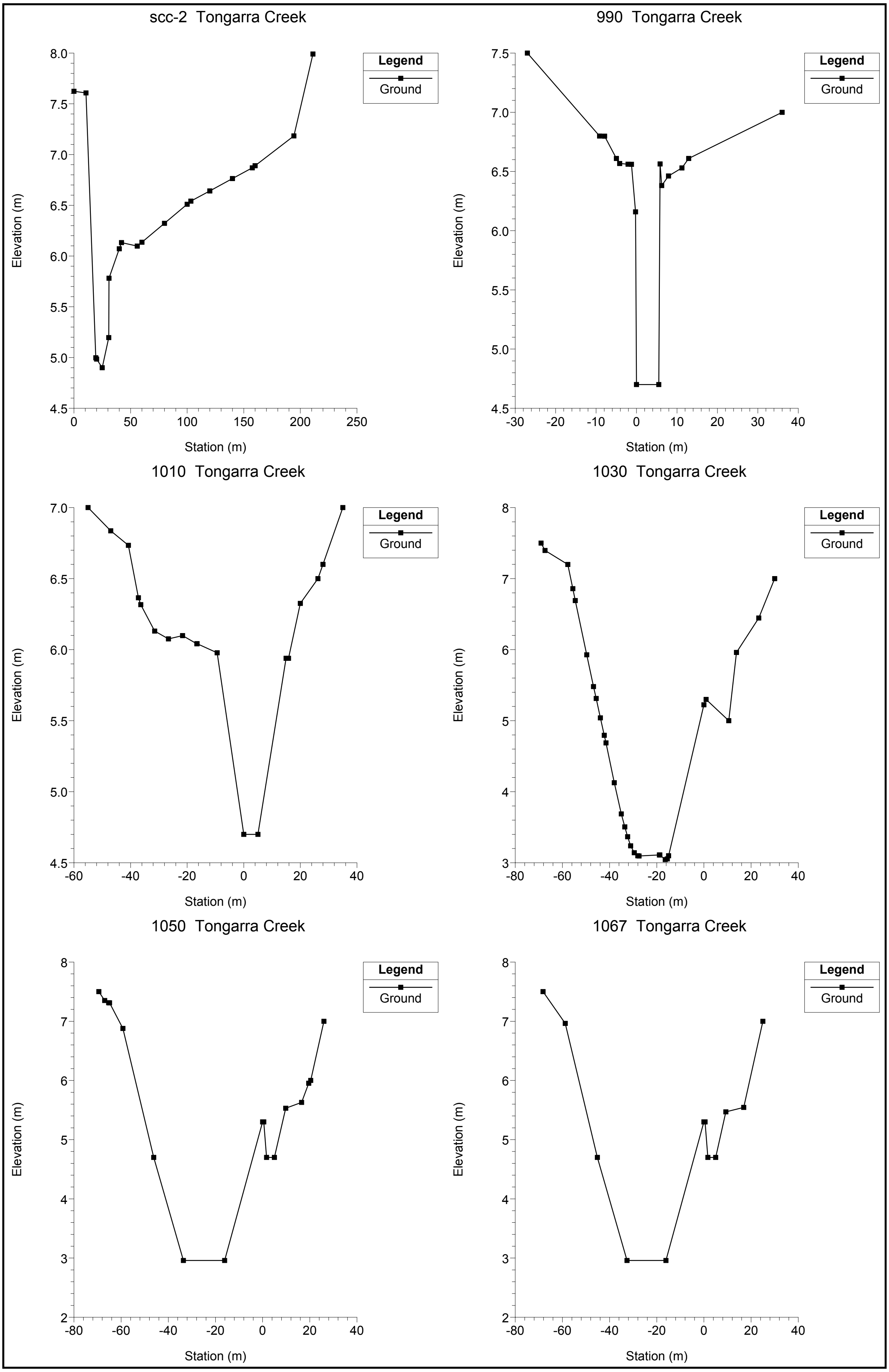

CS70 Oakley Overland
Legend
Ground
Elevation (m)
Station (m)
CS71 Oakley Overland
Legend
Ground
Elevation (m)
Station (m)
CS72 Oakley Overland
Legend
Ground
Elevation (m)
Station (m)
CS73 Oakley Overland
Legend
Ground
Elevation (m)
Station (m)
CS79 Oakley Overland
Legend
Ground
Elevation (m)
Station (m)

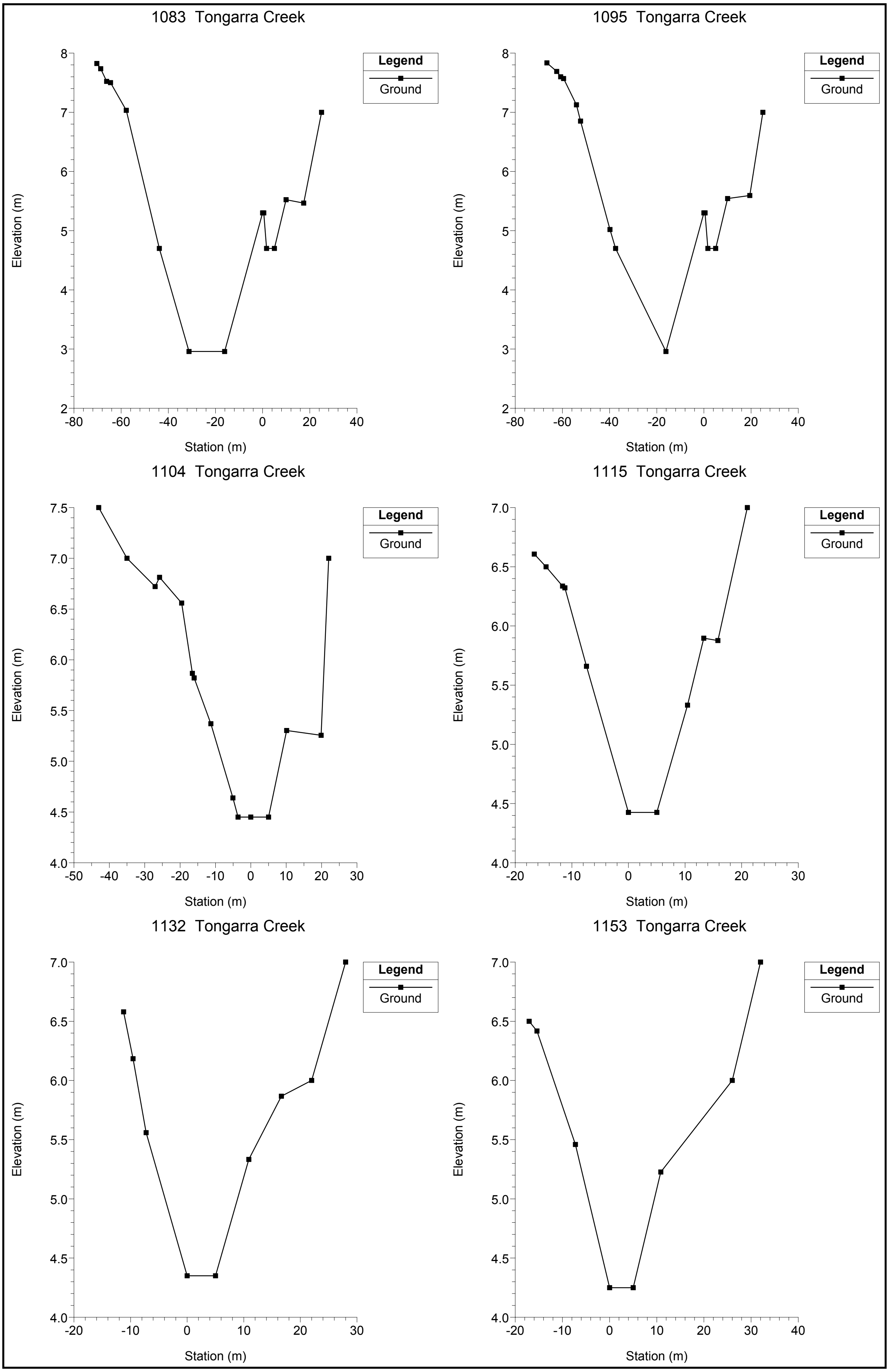

-100 Tongarra Creek
-50 Tongarra Creek
0 Tongarra Creek
50 Tongarra Creek
100 Tongarra Creek
150 Tongarra Creek
Legend
Ground
Elevation (m)
Station (m)

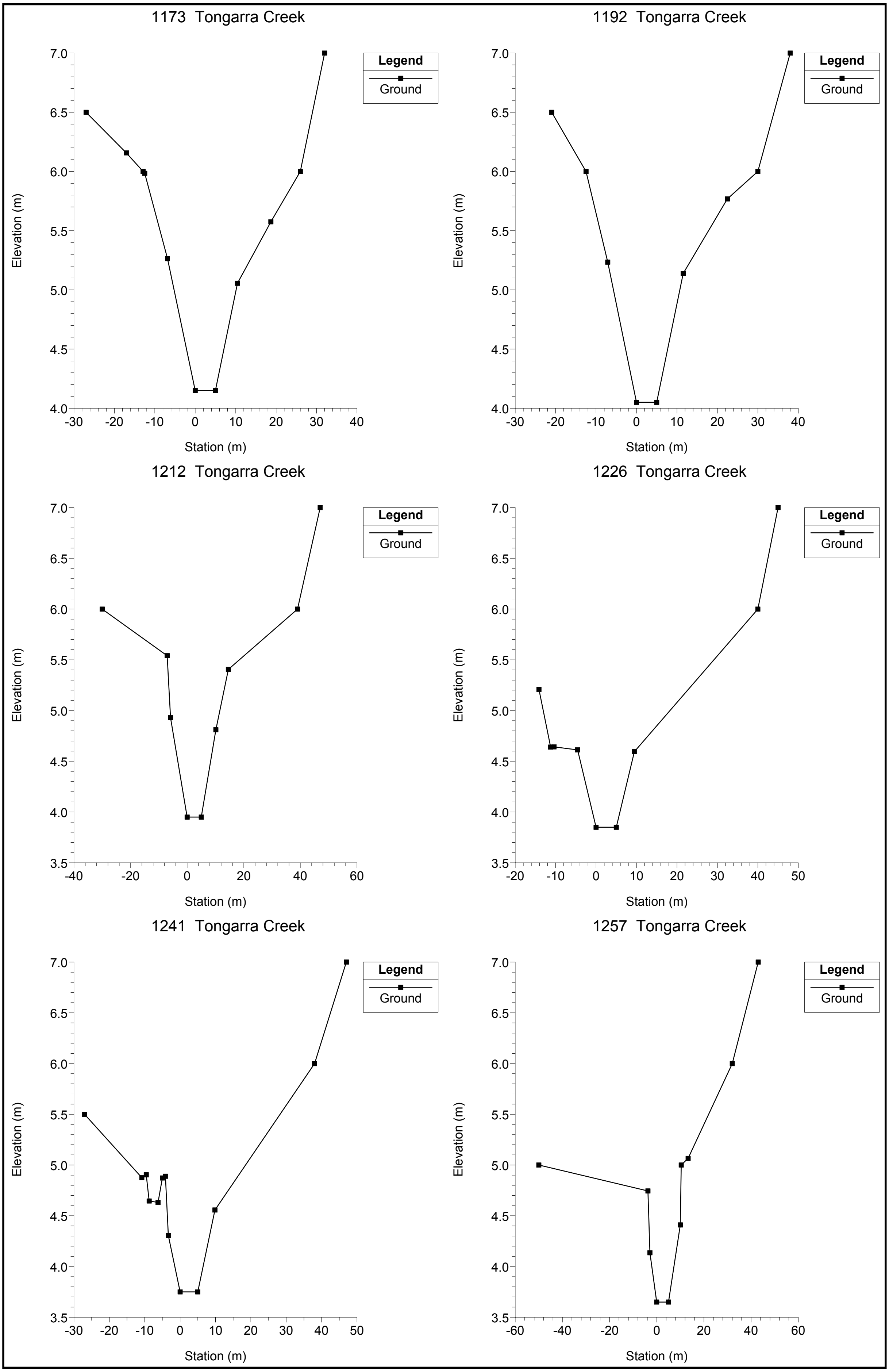


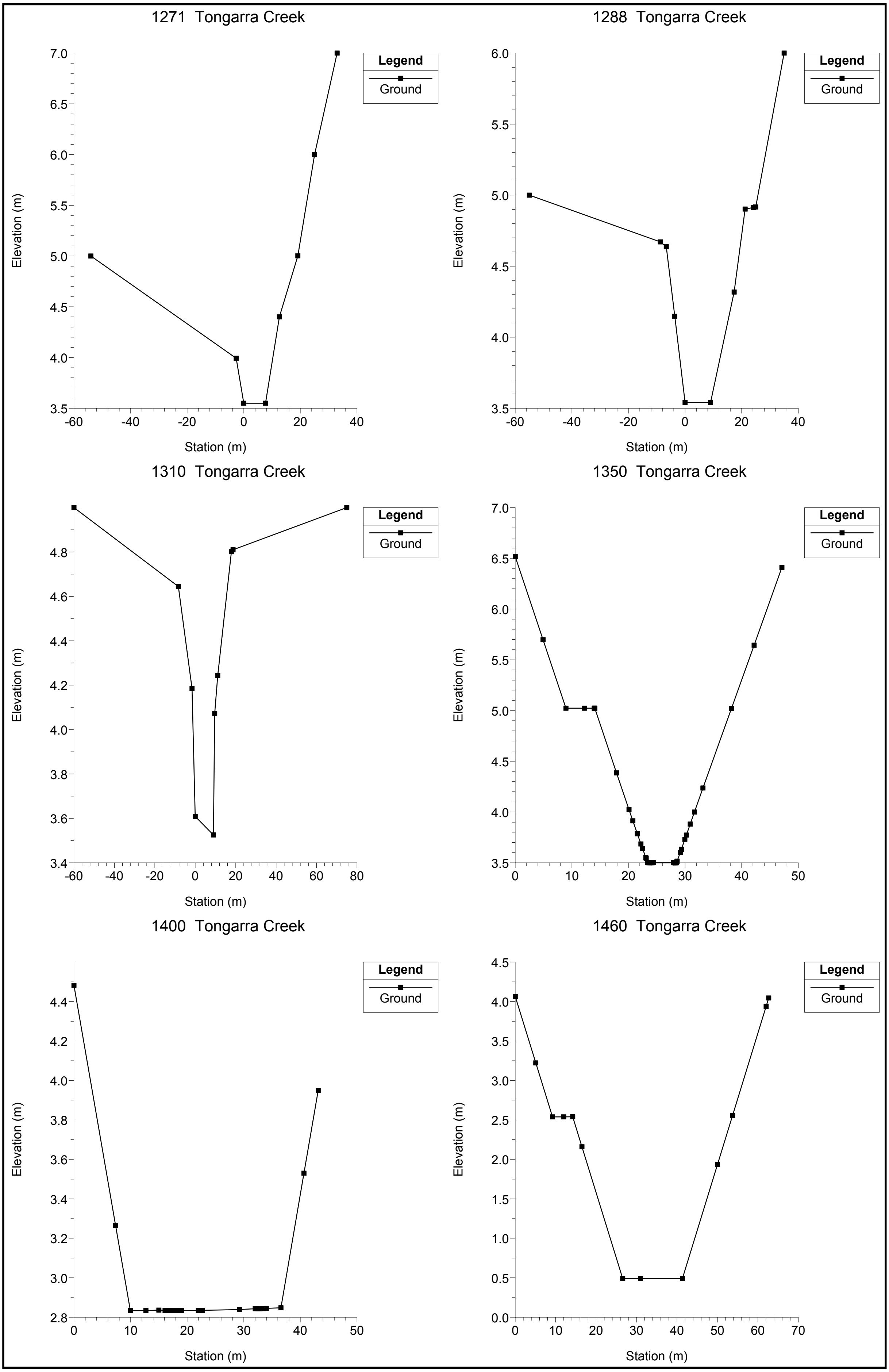
200 Tongarra Creek
250 Tongarra Creek
scc-12 Tongarra Creek
scc-11 Tongarra Creek
scc-10 Tongarra Creek
scc-9 Tongarra Creek
Legend
Ground
Elevation (m)
Station (m)

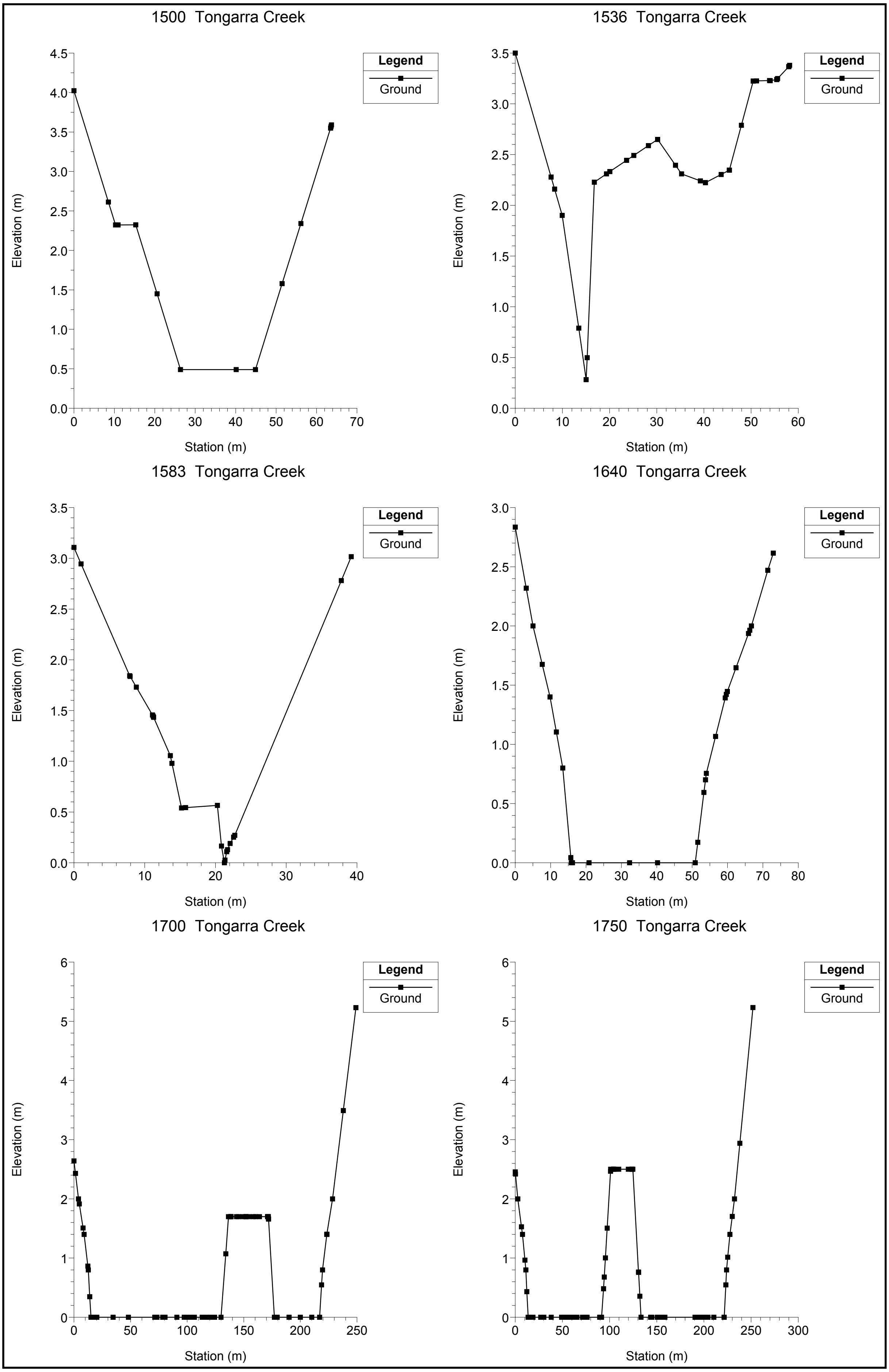

scc-8 Tongarra Creek
scc-7 Tongarra Creek
scc-6 Tongarra Creek
scc-5 Tongarra Creek
scc-4 Tongarra Creek
scc-3 Tongarra Creek
Legend
Ground
Elevation (m)
Station (m)

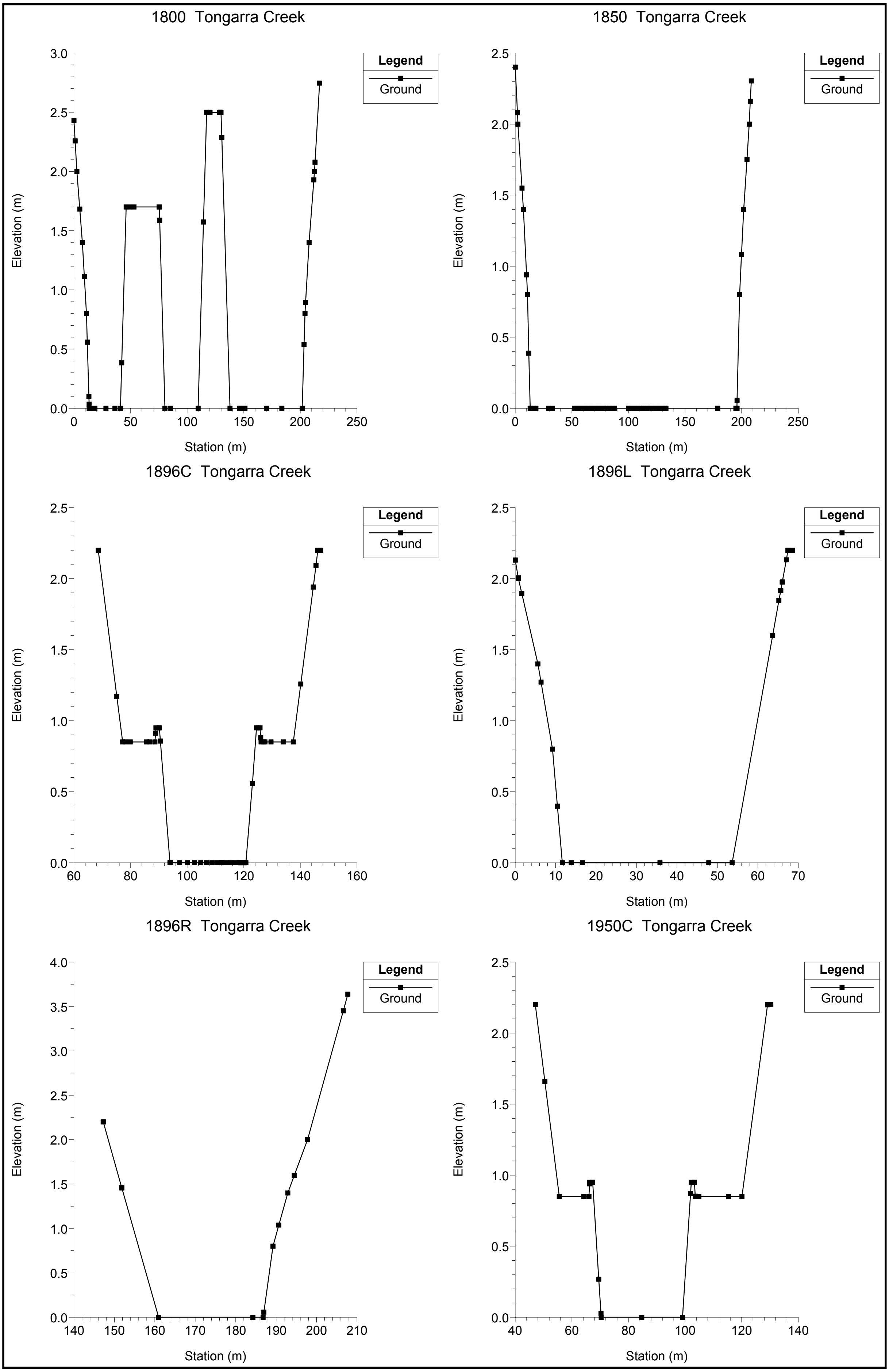

scc-2 Tongarra Creek
990 Tongarra Creek
1010 Tongarra Creek
1030 Tongarra Creek
1050 Tongarra Creek
1067 Tongarra Creek
Legend
Ground
Elevation (m)
Station (m)

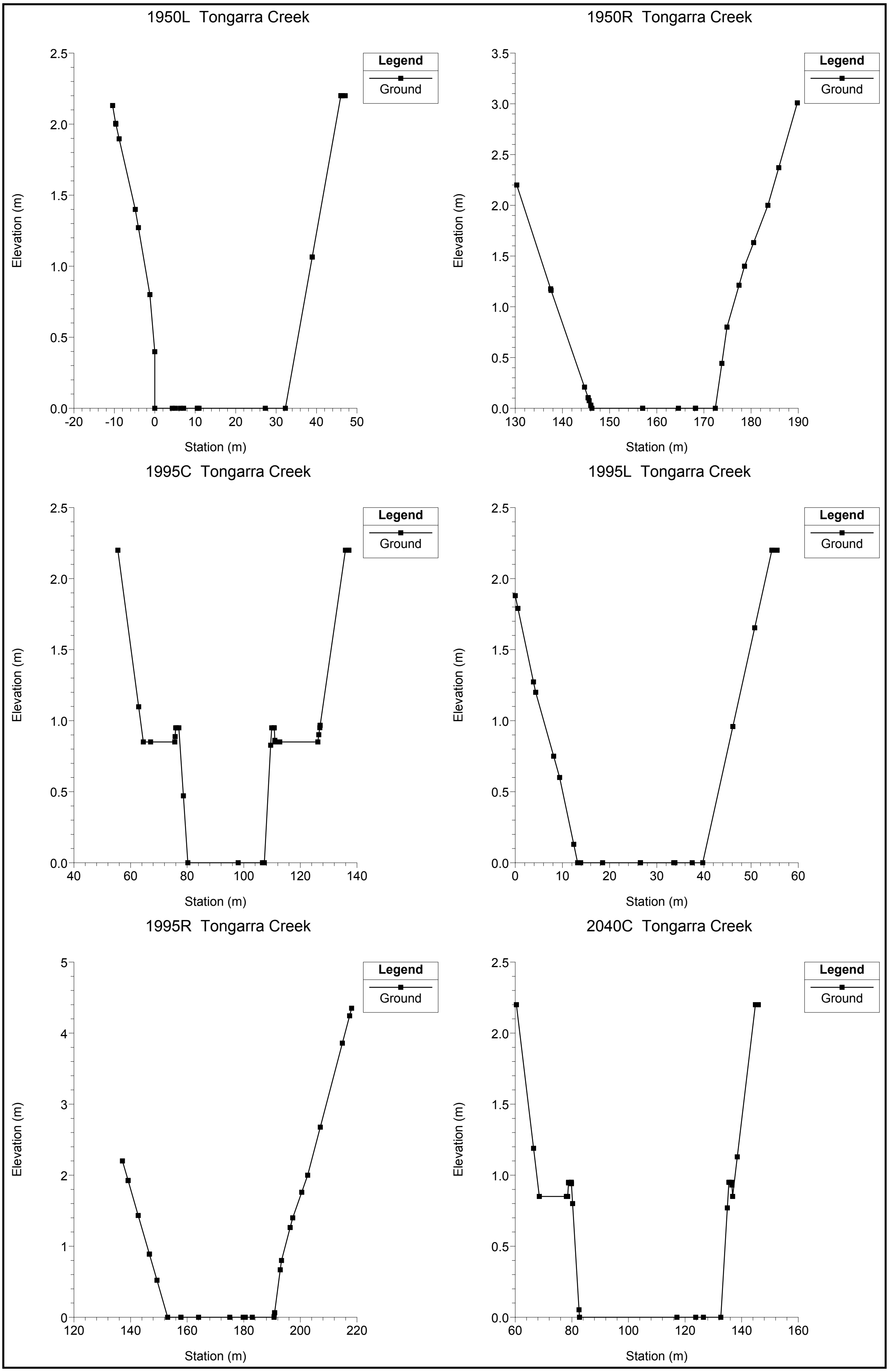

1083 Tongarra Creek
1095 Tongarra Creek
1104 Tongarra Creek
1115 Tongarra Creek
1132 Tongarra Creek
1153 Tongarra Creek
Legend
Ground
Elevation (m)
Station (m)

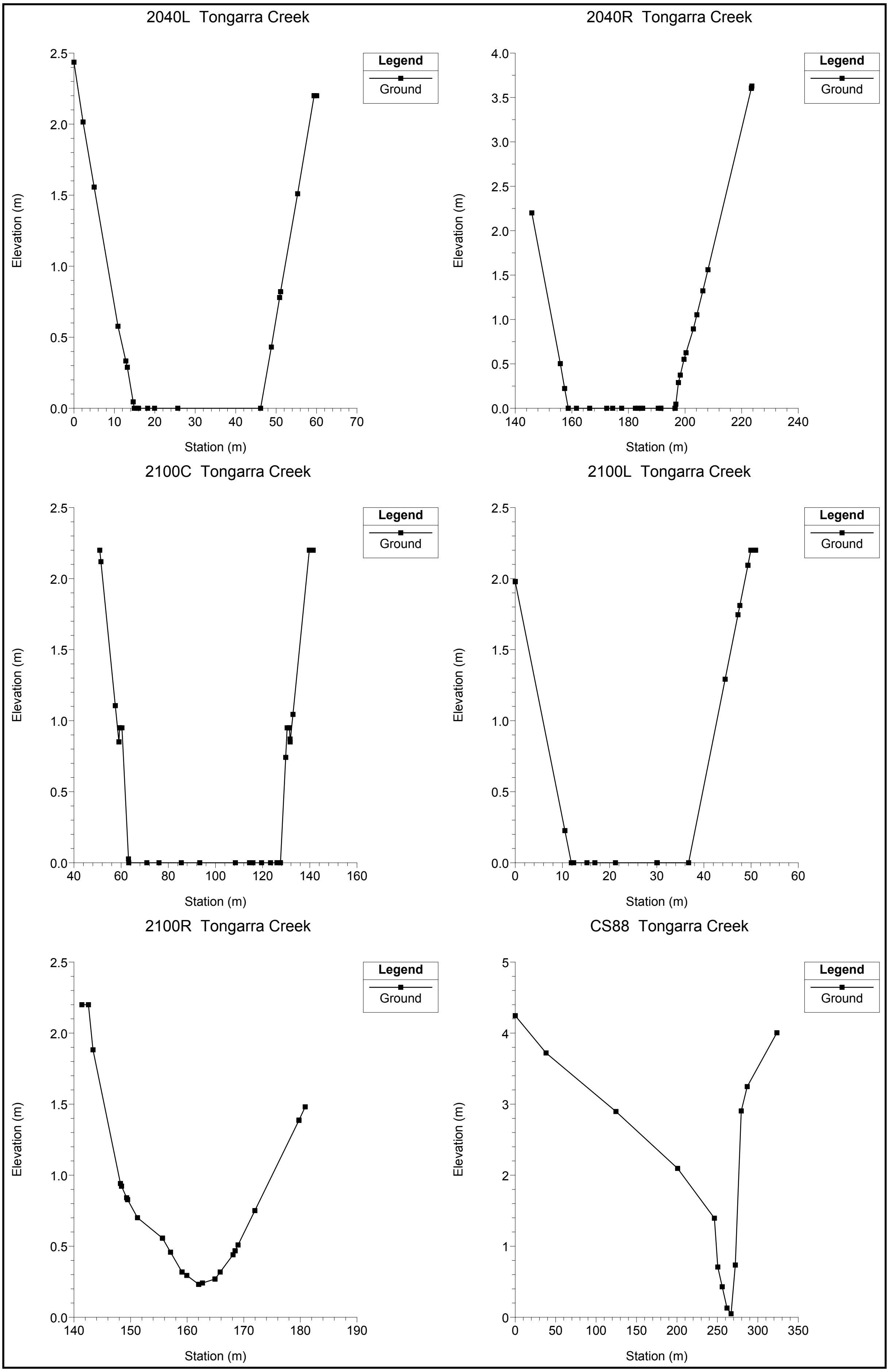

1173 Tongarra Creek
1192 Tongarra Creek
1212 Tongarra Creek
1226 Tongarra Creek
1241 Tongarra Creek
1257 Tongarra Creek
Legend
Ground
Elevation (m)
Station (m)

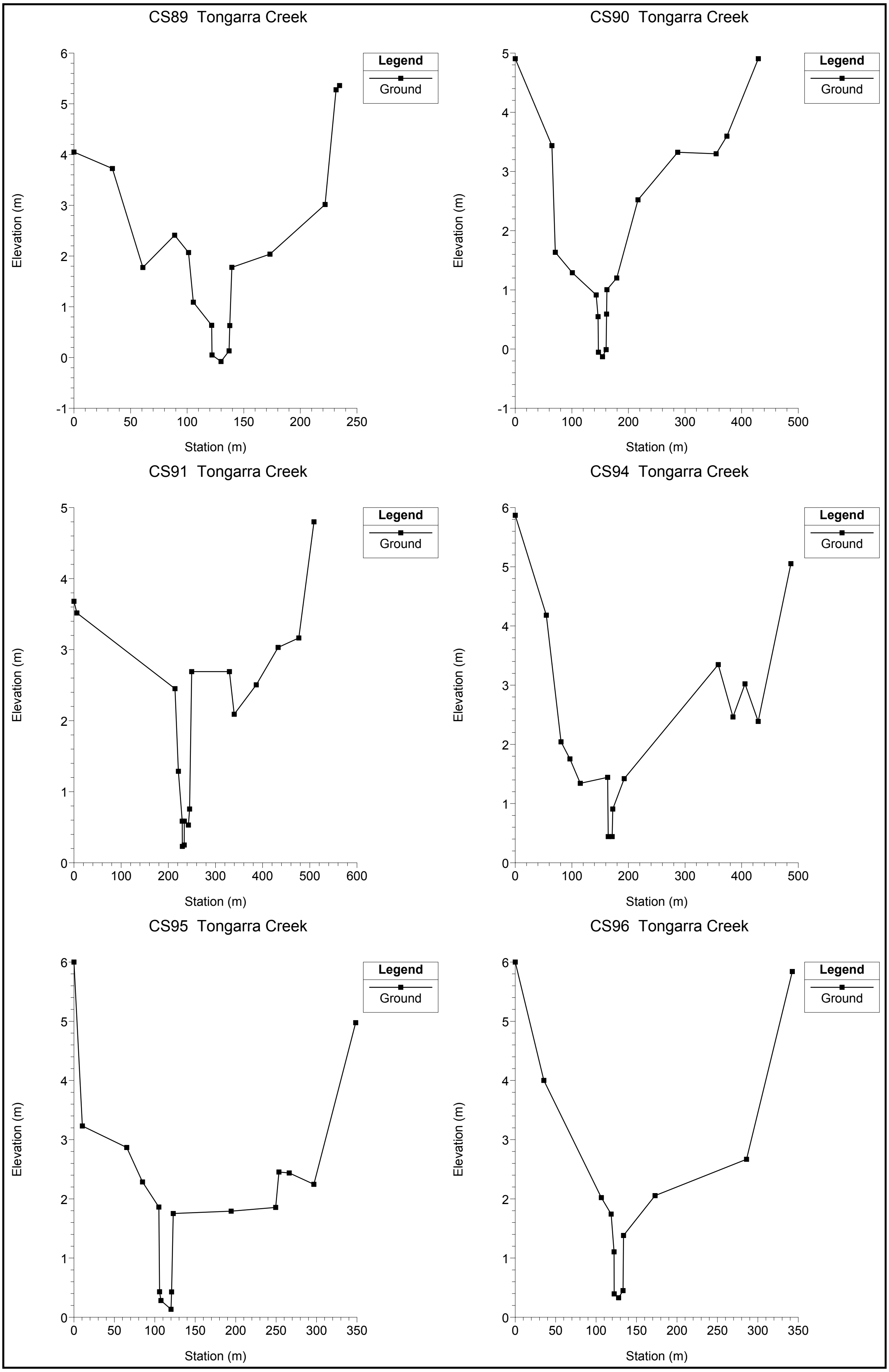

1271 Tongarra Creek
1288 Tongarra Creek
1310 Tongarra Creek
1350 Tongarra Creek
1400 Tongarra Creek
1460 Tongarra Creek
Legend
Ground
Elevation (m)
Station (m)


1500 Tongarra Creek
Legend
Ground
Elevation (m)
Station (m)
1536 Tongarra Creek
Legend
Ground
Elevation (m)
Station (m)
1583 Tongarra Creek
Legend
Ground
Elevation (m)
Station (m)
1640 Tongarra Creek
Legend
Ground
Elevation (m)
Station (m)
1700 Tongarra Creek
Legend
Ground
Elevation (m)
Station (m)
1750 Tongarra Creek
Legend
Ground
Elevation (m)
Station (m)


1800 Tongarra Creek
1850 Tongarra Creek
1896C Tongarra Creek
1896L Tongarra Creek
1896R Tongarra Creek
1950C Tongarra Creek
Legend
Ground
Elevation (m)
Station (m)


1950L Tongarra Creek
Elevation (m)
Station (m)
Legend
Ground
1950R Tongarra Creek
Elevation (m)
Station (m)
Legend
Ground
1995C Tongarra Creek
Elevation (m)
Station (m)
Legend
Ground
1995L Tongarra Creek
Elevation (m)
Station (m)
Legend
Ground
1995R Tongarra Creek
Elevation (m)
Station (m)
Legend
Ground
2040C Tongarra Creek
Elevation (m)
Station (m)
Legend
Ground


2040L Tongarra Creek
Legend
Ground
Elevation (m)
Station (m)
2040R Tongarra Creek
Legend
Ground
Elevation (m)
Station (m)
2100C Tongarra Creek
Legend
Ground
Elevation (m)
Station (m)
2100L Tongarra Creek
Legend
Ground
Elevation (m)
Station (m)
2100R Tongarra Creek
Legend
Ground
Elevation (m)
Station (m)
CS88 Tongarra Creek
Legend
Ground
Elevation (m)
Station (m)


CS89 Tongarra Creek
CS90 Tongarra Creek
CS91 Tongarra Creek
CS94 Tongarra Creek
CS95 Tongarra Creek
CS96 Tongarra Creek
Legend
Ground
Elevation (m)
Station (m)

APPENDIX A SURVEY DATA

A2 - Pits

Pit No.	Depth from top of kerb (m)	Pit Length (m)	Pit Width (m)	Gap (gutter to kerb) (m)	Opening (gutter to underside of lintel) (m)	Lintel size (m)	Grate	Top of Pit R.L.	Invert Pit R.L.
3	1.624	1.2	0.83	0.2	0.13	1.83	No	6.324	4.7
4	1.35	0.95	0.95	0.18	0.12	1.83	No	6.14	4.79
5	1.49	0.95	0.8	0.18	0.11	1.83	No	6.99	5.5
6	1.21	0.95	0.8	0.18	0.12	1.83	No	6.8	5.59
7	1.28	0.9	0.8	0.2	0.15	1.83	No	6.831	5.551
8	1.05	1	0.8	0.18	0.13	1.83	No	7.625	6.575
9	1	1	0.8	0.18	0.12	1.83	No	7.589	6.589
10	0.95	0.9	0.8	0.2	0.11	1.83	No	7.346	6.396
11	0.95	0.95	0.8	0.2	0.12	1.83	No	7.309	6.359
12	1.2	0.95	0.8	0.2	0.13	1.83	No	7.353	6.153
13	1.1	0.95	0.8	0.2	0.14	1.83	No	7.077	5.977
14	1.15	0.95	0.8	0.2	0.12	1.83	No	6.599	5.449
15	1.15	0.95	0.8	0.2	0.12	1.83	No	6.566	5.416
16	1.34	0.95	0.8	0.18	0.13	1.83	No	6.187	4.847
17	0.5	0.95	0.8	0.2	0.15	1.83	No	6.153	5.653
18	1.41	0.95	0.8	0.16	0.12	1.83	No	5.49	4.08
19	1.1	0.95	0.8	0.16	0.16	1.83	No	5.435	4.335
20	1.15	0.9	0.7	0.16	0.02	3.1	Yes	5.664	4.514
21	1.96	0.9	0.7	0.16	0.02	3.1	Yes	5.402	3.442
22	1.365	0.9	0.7	0.19	0.13	1.83	No	5.165	3.8
23	1.15	0.9	0.7	0.19	0.12	1.83	No	5.09	3.94
24	1.3	0.9	0.8	0.19	0.12	1.83	No	5.98	4.68
25	1.2	0.9	0.8	0.19	0.12	1.83	No	5.969	4.769
26	1.25	0.9	0.8	0.2	0.13	1.83	No	6.452	5.202
27	1.25	0.9	0.8	0.2	0.13	1.83	No	6.489	5.239
29	1.1	0.9	0.7	0.19	0.115	1.83	No	6.871	5.771
30	1.1	0.9	0.7	0.19	0.11	1.83	No	7.078	5.978
31	0.95	0.8	0.7	0.18	0.11	1.83	No	6.897	5.947
32	1.14	0.85	0.7	0.18	0.11	1.83	No	6.882	5.742
33	1.08	1.2	0.83	0.22	0.15	1.83	No	8.27	7.19
34	1.05	1.2	0.83	0.2	0.15	1.83	No	8.246	7.196
35	0.89	1.2	0.83	0.2	0.15	1.83	No	8.971	8.081
36	0.93	1.2	0.83	0.2	0.15	NA	Letter box	9.515	8.585
37	1.64	1.2	0.9	0	NA	NA	Letter box	10.59	8.95
38	1.3	1.2	0.9	0	0.2	0.3	Letter box	10.624	9.324
39	1.38	1.2	0.9	0	NA	NA	Letter box	10.88	9.5
40	1	1.6	1.2	0	0.2	4.8	Letter box	10.676	9.676
41	0.58	1.2	0.83	0.2	0.15	1.83	No	10.113	9.533
42	1	1.2	0.83	0.2	0.15	1.83	No	10.1	9.1
43	0.63	1.2	0.83	0.2	0.15	1.83	No	9.542	8.912
44	1.12	1.2	0.83	0.2	0.12	1.83	No	9.573	8.453
45	1.13	1.2	0.83	0.2	0.15	1.83	No	9.19	8.06
46	0.93	1.2	0.83	0.2	0.15	1.83	No	9.127	8.197
47	0.78	1.2	0.83	0.2	0.15	1.83	No	8.558	7.778
48	1.15	1.2	0.83	0.2	0.15	1.83	No	8.558	7.408
49	0.97	1.2	0.83	0.2	0.15	1.83	No	8.067	7.097
50	1.32	1.2	0.83	0.2	0.15	1.83	No	7.976	6.656
51	1.88	0.86	0.85	0.2	0.15	0.9	Yes	8.434	6.554
52	1.1	1.2	0.83	0.2	0.15	1.83	No	8.071	6.971

APPENDIX A SURVEY DATA
A2 - Pits

Pit No.	Depth from top of kerb (m)	Pit Length (m)	Pit Width (m)	Gap (gutter to kerb) (m)	Opening (gutter to underside of lintel) (m)	Lintel size (m)	Grate	Top of Pit R.L.	Invert Pit R.L.
53	1.44	1.2	0.83	0.2	0.15	1.83	No	8.063	6.623
54	1.27	0.9	0.8	0.2	0.12	1.9	Yes	8.762	7.492
55	1.29	0.9	0.8	0.2	0.12	2.4	Yes	8.955	7.665
56	1.26	0.9	0.8	0.2	0.12	2.4	Yes	9.573	8.313
57	1.08	0.9	0.8	0.2	0.12	0.9	Yes	9.559	8.479
58	0.89	0.9	0.8	0.2	0.12	2.4	Yes	8.924	8.034
100	1.17	2	0.65	0.2	0.11	2.8	Yes	2.76	1.59
101	1.08	0.85	0.85	0.18	0.12	2.8	Yes	2.807	1.727
102	1.09	1	0.85	0.18	0.11	2.8	Yes	2.927	1.837
103	1.38	1.25	1.15	0.18	0.12	2.8	Yes	3.535	2.155
104	1.38	0.9	0.9	0.18	0.12	2.8	Yes	3.865	2.485
105	1.47	1.25	1.25	0.18	0.12	2.83	Yes	4.159	2.689
106	1.72	1	1	0.12	0.11	2.8	Yes	4.547	2.827
107	1.6	1	1	0.18	0.12	2.8	Yes	4.5	2.9
108	2.24	4	3.2	0.2	0.11	2.8	Yes	5.175	2.935
109	1.18	0.9	0.9	0.2	0.12	2.8	Yes	4.729	3.549
150	1.28	0.9	0.9	0.26	0.16	0.9	Yes	4.088	2.808
151	1.36	0.9	0.7	0.26	0.16	3.2	Yes	3.983	2.623
152	1.26	0.85	1.5	0.26	0.16	3.2	Yes	3.785	2.525
159	0.79	0.6	0.6	0.22	0.12	2	No	4.159	3.369
160	0.65	NA	0.55	0.55	0.55	NA	Letter box	3.754	3.104
161	0.6	0.47	0.4	0.47	0.47	NA	Letter box	3.583	2.983
162	0.4	0.5	0.4	0.1	NA	NA	Letter box	2.486	2.086
163	0.46	0.47	0.4	0.12	0.15	NA	Letter box	2.449	1.989
30A	0.82	1.2	0.83	0.22	0.15	1.83	No	8.535	7.715
32A	1.03	1.2	0.83	0.22	0.15	1.83	No	8.539	7.509

APPENDIX B

MARCH 1975 AND MAY 1983 PLUVIOGRAPH DATA

Appendix B

PLUVIO RAINFALL DATA AT PORT KEMBLA

(May 1983 and March 1975)

May 1983 Rainfall					
Time	**Rainfall (mm)**	**Time**	**Rainfall (mm)**	**Time**	**Rainfall (mm)**
05/25/83 14:10	0.50	05/27/83 16:05	0.50	05/27/83 22:20	1.00
05/25/83 15:00	0.50	05/27/83 16:20	0.50	05/27/83 22:25	0.50
05/25/83 15:45	0.50	05/27/83 16:35	0.50	05/27/83 22:30	0.50
05/25/83 15:55	0.50	05/27/83 17:00	0.50	05/27/83 22:35	0.50
05/25/83 16:00	0.50	05/27/83 17:05	0.50	05/27/83 22:40	1.50
05/25/83 16:05	0.50	05/27/83 17:10	1.00	05/27/83 22:45	0.50
05/25/83 16:10	0.50	05/27/83 17:15	0.50	05/27/83 22:55	0.50
05/25/83 16:25	0.50	05/27/83 17:20	0.50	05/27/83 23:00	0.50
05/25/83 16:30	0.50	05/27/83 17:25	0.50	05/27/83 23:10	0.50
05/25/83 16:35	0.50	05/27/83 17:30	1.50	05/28/83 00:15	0.50
05/25/83 16:40	0.50	05/27/83 17:35	1.00	05/28/83 00:50	0.50
05/25/83 16:55	0.50	05/27/83 17:40	1.50	05/28/83 00:55	0.50
05/25/83 17:40	0.50	05/27/83 17:45	1.00	05/28/83 01:35	0.50
05/25/83 17:50	0.50	05/27/83 17:50	2.50	05/28/83 05:00	0.50
05/25/83 17:55	0.50	05/27/83 17:55	1.50		
05/25/83 18:20	0.50	05/27/83 18:00	2.50		
05/25/83 18:25	0.50	05/27/83 18:05	4.00		
05/25/83 18:30	0.50	05/27/83 18:10	3.00		
05/25/83 18:35	0.50	05/27/83 18:15	2.00		
05/25/83 18:45	0.50	05/27/83 18:20	4.00		
05/25/83 18:55	0.50	05/27/83 18:25	4.00		
05/25/83 19:05	0.50	05/27/83 18:30	5.00		
05/25/83 19:10	0.50	05/27/83 18:35	6.00		
05/25/83 19:25	0.50	05/27/83 18:40	2.50		
05/25/83 19:35	0.50	05/27/83 18:45	2.50		
05/25/83 19:45	0.50	05/27/83 18:50	4.00		
05/25/83 20:05	0.50	05/27/83 18:55	3.00		
05/25/83 20:20	0.50	05/27/83 19:00	2.50		
05/25/83 20:30	0.50	05/27/83 19:05	4.00		
05/25/83 20:45	0.50	05/27/83 19:10	4.50		
05/25/83 21:45	0.50	05/27/83 19:15	4.50		
05/26/83 00:25	0.50	05/27/83 19:20	3.50		
05/26/83 00:55	0.50	05/27/83 19:25	2.00		
05/26/83 03:35	0.50	05/27/83 19:30	4.00		
05/26/83 05:15	0.50	05/27/83 19:35	2.00		
05/26/83 05:40	0.50	05/27/83 19:40	7.50		
05/26/83 06:00	0.50	05/27/83 19:45	4.50		
05/26/83 06:20	0.50	05/27/83 19:50	5.00		
05/26/83 06:40	0.50	05/27/83 19:55	8.00		
05/26/83 07:10	0.50	05/27/83 20:00	4.50		
05/26/83 08:00	0.50	05/27/83 20:05	5.50		
05/26/83 08:50	0.50	05/27/83 20:10	1.50		
05/26/83 10:10	0.50	05/27/83 20:15	2.00		
05/26/83 10:55	0.50	05/27/83 20:20	1.00		
05/26/83 11:15	0.50	05/27/83 20:40	0.50		
05/26/83 11:45	0.50	05/27/83 21:20	0.50		
05/26/83 11:55	0.50	05/27/83 21:45	0.50		
05/26/83 12:25	0.50	05/27/83 22:00	0.50		
05/27/83 15:35	0.50	05/27/83 22:10	1.00		
05/27/83 15:50	0.50	05/27/83 22:15	0.50		

Appendix B
PLUVIO RAINFALL DATA AT PORT KEMBLA
(May 1983 and March 1975)

March 1975 Rainfall	
Time	**Rainfall (mm)**
09/03/75 10:00	1.0
09/03/75 11:00	0.0
09/03/75 12:00	3.0
09/03/75 13:00	0.0
09/03/75 14:00	0.5
09/03/75 15:00	2.0
09/03/75 16:00	3.0
09/03/75 17:00	3.0
09/03/75 18:00	1.0
09/03/75 19:00	0.0
09/03/75 20:00	0.0
09/03/75 21:00	1.0
09/03/75 22:00	33.5
09/03/75 23:00	22.5
10/03/75 00:00	15.5
10/03/75 01:00	0.5
10/03/75 02:00	0.0
10/03/75 03:00	0.0
10/03/75 04:00	0.0
10/03/75 05:00	3.5
10/03/75 06:00	15.0
10/03/75 07:00	6.0
10/03/75 08:00	24.5
10/03/75 09:00	14.0
10/03/75 10:00	49.5
10/03/75 11:00	8.5
10/03/75 12:00	6.0
10/03/75 13:00	25.0
10/03/75 14:00	15.5
10/03/75 15:00	4.0
10/03/75 16:00	0.0
10/03/75 17:00	0.5
10/03/75 18:00	1.5
10/03/75 19:00	0.0
10/03/75 20:00	20.0
10/03/75 21:00	42.0
10/03/75 22:00	25.0
10/03/75 23:00	23.0
11/03/75 00:00	66.0
11/03/75 01:00	15.0
11/03/75 02:00	13.0
11/03/75 03:00	5.5
11/03/75 04:00	2.0
11/03/75 05:00	0.0
11/03/75 06:00	0.0
11/03/75 07:00	0.0
11/03/75 08:00	0.0
11/03/75 09:00	0.0

APPENDIX C

EMBEDDED DESIGN STORMS

The Embedded Design Storm Concept - A Critical Review

E. H. Rigby, Director, Forbes Rigby Pty Ltd, Consulting Engineers, Wollongong, NSW
D. J. Bannigan, Drainage Engineer, Forbes Rigby Pty Ltd, Consulting Engineers, Wollongong, NSW

Summary In 1958 Australian Rainfall and Runoff introduced a procedure for estimation of design discharges from storm burst intensity - frequency - duration and temporal data which has now served the profession well for over 30 years. Difficulties in applying this procedure with the current generation of rainfall runoff models has however generated a need to review this procedure. This paper explores these dificulties and attempts to locally quantify the impact of extracting a burst from a storm, on predicted discharges. The IFD profile of historic storms in the Illawarra Region is examined and an alternate procedure for creation of a design storm presented. The impact of embedment on predicted discharges is shown to be most significant when short duration design bursts are applied to catchments with substantial natural or man made storages. The implementation of an embedded design storm procedure in the rainfall runoff model WBNM is discussed and the Authors' conclusions in respect to the need for an alternative to the present burst approach presented.

1. INTRODUCTION

In 1958, Australian Rainfall and Runoff (AR&R)(1) introduced a procedure for estimating a flood discharge of given Average Recurrence Interval (ARI) from storm burst Intensity - Frequency - Duration (IFD) data.

D[illegible]ng the course of evolution of this procedure, much discussion and research took place with subsequent editions of AR&R in 1977(2) and 1987(3) formalising the procedure and incorporating substantial improvements in the quality of the IFD and temporal pattern data upon which the storm burst procedure was based.

Notwithstanding the considerable time over which this procedure has evolved, the fundamental presumption that a flood discharge can be reliably derived from a storm burst (as distinct from a storm) has received little attention. With use [illegible] computer based rainfall/runoff models to calculate design [illegible]ood discharges now common place, it is the authors' view that it is time to review this presumption.

Whilst simple flood estimation models dealing only with the peak of the flood hide the distinction between a storm and storm burst - almost all of the present range of computer based rainfall/runoff models need some rather arbitrary adjustment to losses and starting storage, if a design storm burst is to be used to predict flooding with an ARI similar to that of the storm burst.

This problem is most evident in catchments with relatively sh[illegible]rt response times, significant flood storage and high losses. I[illegible] [illegible]ch circumstances, the response of the catchment to a burst, relative to a storm 'typical' of the dataset from which the burst was extracted, is likely to differ in the following areas.

- The critical design burst is likely to be of much shorter duration than the 'typical' storm it was extracted from, with consequential impact on flow volume and duration.
- Lack of flow over catchment surfaces, in channels and on floodplains and lack of storage in basins at the commencement of a design burst, all result in attenuation of peak flow relative to that of the 'typical' parent storm.
- Inclusion of losses in a design burst based on losses in historic storm events does not reflect the impact of lead up rainfall on losses, further reducing peak flows relative to the 'typical' parent storm.

In a recent review of flooding in the Parramatta River by Phillips, Lees and Lynch (4), it was concluded that the above factors were responsible for an underprediction of the 100yr ARI flood surface by up to 500mm.

In the following sections, the evolution of the present storm burst based procedure is reviewed and the limitations of this procedure explored. An alternate procedure, to construct an embedded design storm from existing burst IFD and temporal data is presented and implementation of this procedure in the Watershed Bounded Network Model (WBNM)(5) discussed.

2. THE AR&R STORMBURST PROCEDURE

In the 1987 edition of AR&R, a procedure is set out for the derivation of a design peak flood discharge of given ARI from a design storm burst of equivalent ARI.

In summary this procedure includes :

- Selection of appropriate design storm burst IFD data for the site.

- Selection of an appropriate temporal pattern for the design storm burst.

- Selection of appropriate design storm burst losses for the site and chosen ARI/Duration.

- Application of an appropriate rainfall/runoff model to establish the design storm burst duration producing the greatest discharge - this maximised design discharge then being accepted as the design discharge of the specified ARI.

It is important to note that both the IFD data and temporal patterns presented in AR&R are for storm bursts, not storms. This is made clear in Section 3.2 of the 1987 edition of AR&R where the "NATURE OF DESIGN PATTERNS", is discussed.

The procedure outlined above first appeared in the 1958 edition of AR&R as a means of determining the peak discharge of a given ARI using a unit hydrograph based model. As pointed by French and Walsh (6), this procedure appeared without explanation, at a time when respected texts such as Linsley et al (7) made no mention of such a procedure.

In the 1977 edition of AR&R, the quality of data had much improved but the distinction between storm bursts and storms remained blurred. Losses were treated much more deliberately but without distinction between design and historic storms, other than to recommend *"Selection of design values of initial and continuing loss is best based upon consideration of many such values derived for recorded storms on the catchment concerned"*.

Whilst more clearly stated in the 1977 edition of AR&R, the design storm burst procedure in AR&R 1977 is the same as that set out for the unit hydrograph procedure in the 1958 edition of AR&R. Considerable discussion and explanation is provided in AR&R 1977 on the derivation and limitations of both the IFD data and design storm burst temporal patterns. No comment is made on the appropriateness of using a design storm burst of a given ARI to calculate the peak design flood discharge of a similar ARI.

In the current (1987) edition of AR&R a considerable body of additional data has been collected and collated, markedly increasing the level of confidence inherent in the IFD data and storm burst temporal patterns. The distinction between storm bursts and storms is now clearly stated and consistently presented in the document. No further explanation as to the rationale behind the use of a storm burst in lieu of a storm to predict a peak discharge of given ARI is however provided.

With respect to losses, the 1987 edition of AR&R notes that *"for estimating the runoff excess from an actual storm as distinct from the design situation, allowance must be made for the condition and wetness of the catchment immediately pri[or] to the event"* and *"for design, as discussed in Chapter I, a[n] average value is usually needed and .... the median of th[e] derived values is probably the most appropriate to design"*.

3. DESIGN STORM BURST LIMITATIONS

Whilst the procedure set out in the 1987 edition of AR&R i[s] quite clear and readily implemented, it does present some difficulties in practice.

For the most part these difficulties arise as a consequence of the differences between the response of a catchment to a design storm burst and a design storm.

Given that the present design storm burst IFD data and temporal patterns have been extracted from a range of historic storms, it does not seem reasonable to expect a design storm burst to be able to simulate the discharge occurring in the storm from whence it was extracted.

In short duration design bursts (typically extracted from storms of duration greater than the burst under consideration) losses would, at the instant the burst commences in the storm, be much reduced relative to losses applicable to the start of the storm.

Many practitioners have responded to this concern by setting initial losses to zero when modelling short duration design storm bursts. This is however rather arbitrary and leaves unresolved the question as to when the burst duration is long enough for the initial loss to be reintroduced.

A further concern arises from the impact of lead up rainfall on natural and man made flood storages. In modelling the behaviour of such systems, using the design burst approach, the system commences its response with all storages empty - attenuating peak discharges relative to discharges that would have been calculated had the full storm been modelled. Where man made storages are present, some models permit an analysis to be made with the storages partly full, to reflect the likelihood that some storage will be present at the commencement of the design storm burst. As with initial losses, this is however an arbitrary approach which at best can only indicate the sensitivity of discharge to starting storage.

It should be noted that the above concerns are strongest when modelling short duration design storm bursts. For larger catchments with "critical" burst durations of a day or more, it seems reasonable to assume that the average intensity and temporal pattern of the design storm burst is typical of a full storm.

4. REGIONAL STORMS AND BURSTS

Given the basic derivation of both the present 1987 AR&R IFD and storm burst temporal data, it is relevant to ask what the differences in predicted discharge are between the full storm from whence storms bursts were extracted and the burst themselves. If the extracted bursts can not adequately produce peak discharges occurring in their parent storms, then it is not reasonable to assume that a storm burst can be used to model storm discharges.

Since only limited data was available to the author, a review of this relationship was made for four storms only, all responsible for significant flooding in the Illawarra Region. As such, the results are not intended to reflect a national position, but to indicate the order with which a storm burst is able to simulate a full storm discharge. A series of four historic storms were applied to seven catchments ranging in size from about 0.5 km² to 10 km². This exercise was then repeated by extracting the most intense burst with a duration equal to the critical duration for the catchment, from the storm, and rerunning the model with this storm burst. An outline of catchments considered is presented in Table 1. The ratio of predicted discharge from the full storm to that predicted by the extracted burst is reproduced in Table 2.

Catchment (Subcatchment)	Catchment Area	Critical Duration
1. Carricks Ck (Full catchment)	0.64 km²	120 min
2. Bellambi Ck (Raths Gully)	0.91 km²	120 min
3. Bellambi Lake (Full Catchment)	2.83 km²	120 min
4. Byarong Ck (Millbrook Site)	3.96 km²	120 min
5. Hewitts Ck (Full Catchment)	6.14 km²	120 min
6. Horsley Creek (Full Catchment)	9.60 km²	120 min
7. Little Lake (Full Catchment)	12.10 km²	120 min

Table 1 : REGIONAL TEST CATCHMENTS

Given the significant under prediction in discharge produced by the extracted burst relative to the parent storm, on some catchments, one must conclude that the extracted storm burst can not reliably simulate runoff from the parent storm. Whilst there is a considerable spread in results, both across catchments and across storms, it is relevant to note that both catchments exhibiting consistent underprediction (Bellambi Creek and Little Lake) contain significant natural and man made storages.

Catchment No.	Feb 1975 Port Kembla	Mar 1984 Wongawilli	April 1988 Bulli	June 1991 Port Kembla
1	1.00	1.01	1.08	1.04
2	1.60	1.48	1.46	1.72
3	1.06	1.15	1.12	1.08
4	1.05	1.13	1.13	-
5	1.01	1.16	1.08	-
6	1.06	1.12	1.12	1.07
7	1.19	1.20	1.14	1.26

Table 2 : RATIO OF DESIGN STORM TO BURST DISCHARGES FOR REGIONAL STORMS

0

In modelling discharges from the storms, an initial loss of 15 mm with 2.5 mm/hr continuing loss was assumed. In modelling discharges from storm bursts, an initial loss of zero was assumed with 2.5 mm/hr continuing loss.

Had the 'full storm' initial loss been applied to the 'burst', burst discharges would reduce, further increasing the gap between storm and burst discharges.

5. THE EMBEDDED DESIGN STORM

Traditionally, earlier design procedures such as the rational method were based on the concept of critical duration bursts and IFD data for these bursts. It is only the more recent rainfall runoff models that make the distinction between a full storm and storm burst, a significant consideration.

In developing a design storm based procedure, the basic objective is to recreate for a given ARI and duration, a design storm with the average intensity and pattern that would be extracted from a rainfall data set appropriate to the site. Whilst simply stated, the complexity of deciding when a storm starts and finishes, and magnitude of the data extraction task makes achievement of this objective very difficult.

The following procedure is a relatively simple compromise, for creation of an embedded design storm using existing design burst IFD and temporal data, to simulate the above design storm. This procedure assumes :

- Existing design burst IFD and temporal pattern data are typically those of storms for the longer durations and of bursts within storms for the shorter durations.

That in order to best relate the ARI of the design discharge to that of the design rainfall using an embedded design storm approach;

- Average losses should apply as determined from a range of historic storms.

- The temporal pattern of both the critical duration burst and storm envelope into which it is to be embedded should be similar to the patterns of average variability as presently defined in AR&R 1987.

- The peak of the critical duration burst temporal pattern should coincide with the peak of the longer duration storm temporal pattern.

- The average intensity of the critical duration burst should have an ARI equal to that of the desired flood discharge. The design burst should be embedded in a design storm envelope with an intensity/duration profile similar to that of storms common to the area.

- A range of embedded burst durations may need to be considered (as in the present AR&R 1987 procedure) to establish the critical storm burst duration to embed in the design storm envelope.

A review of data for storms in the Illawarra Region associated with past flooding, indicates most such storms had storm durations in the range of 12 to 24 hours. Whilst some flood producing storms had durations longer than 24 hours, few storms for which data is available had storm durations less than 12 hours. Hyetographs and intensity/duration plots for four of the most significant recent flood producing rain storms in the Illawarra region are reproduced in Figures 1 and 2.

Whilst a limited data set, the intensity/duration relationships presented in Figure 2 suggest that in typical flood producing rains in the Illawarra region, the 1 to 2 hour burst intensity is generally of the order of or of lower ARI than that of the 24 hour burst in the same storm.

Notwithstanding the small number of events analysed, the frequency with which high ARI long duration burst or storm intensities occurred relative to short duration burst intensities of similar ARI, does raise some additional questions as to the appropriateness of the present IFD curves in the Illawarra Region.

Based finally on a largely subjective view of the duration of past storms and their IFD relationships, an embedded design storm procedure has been developed for the Illawarra Region incorporating the previously outlined assumptions in which a design burst of critical duration for the catchment under consideration is embedded in a design storm envelope of 24 hours duration. The ARI of both the critical storm burst and design storm envelope are set equal to that of the flood discharge being modelled.

The 24 hour and 2 hour IFD profiles for a design storm burst of 100 Yr ARI at Port Kembla are shown in Figure 3, together with the IFD profile of a 2 in 24 hr 100Yr ARI embedded design storm. As will be apparent from a comparison of these

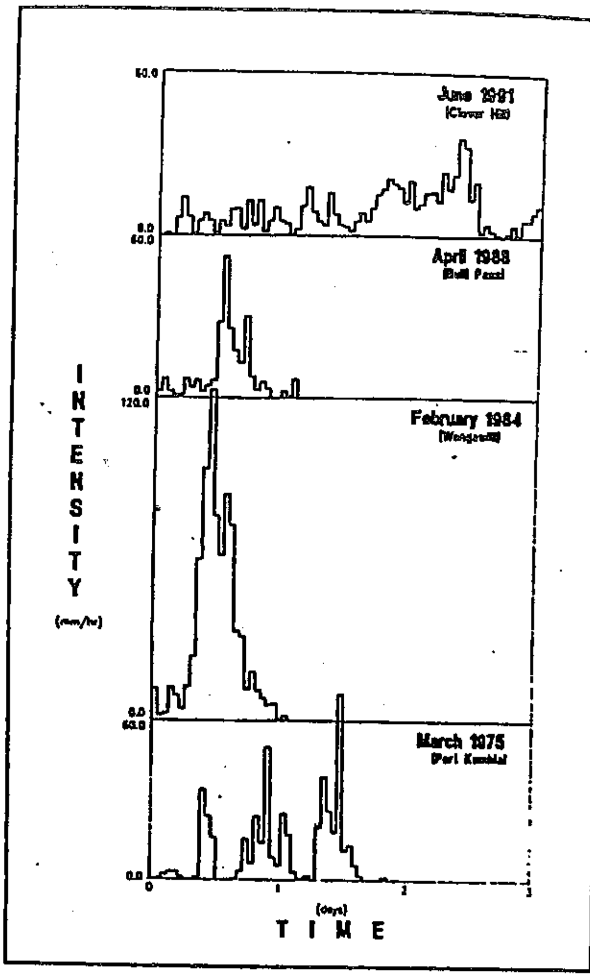


Figure 1 : HISTORIC STORMS HYETOGRAPHS

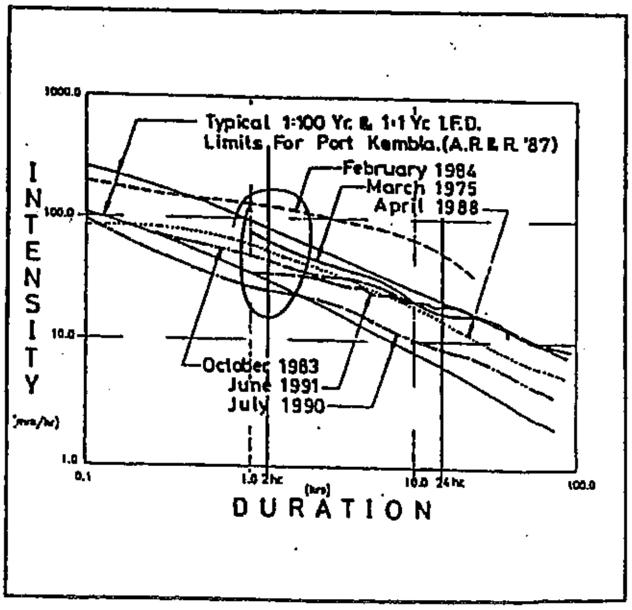
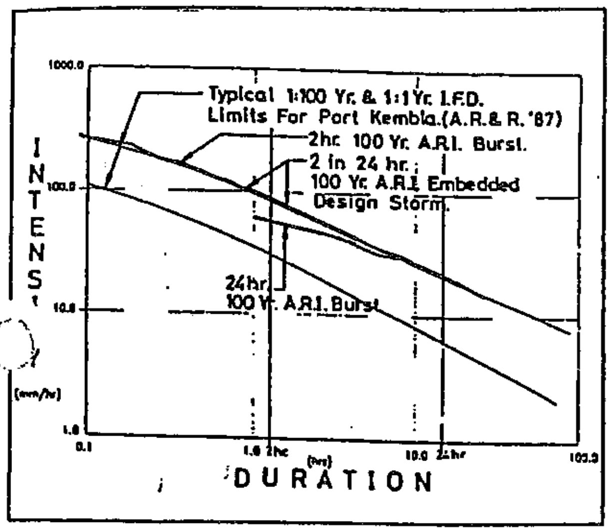


Figure 2 : HISTORIC STORMS INTENSITY - DURATION PROFILES

˜ves, the embedded design storm is close to a 100Yr ARI across the full range of durations up to 24 hours. .ilar to the US "Chicago" style design storm.) Whilst this ›u .ormally be expected to produce a flood event of :reased ARI, due to joint probability considerations - the ents examined suggest such a profile does typically exist in .od producing rains in the Illawarra. In Figure 4, the etograph of a 100Yr ARI 2 hour design storm burst is own embedded in a 24 hour duration burst of similar ARI, :ating a 2 in 24 hour, 100Yr ARI embedded design storm.



F.,ure 3 : AR&R BURST & EMBEDDED DESIGN STORM IFD PROFILES

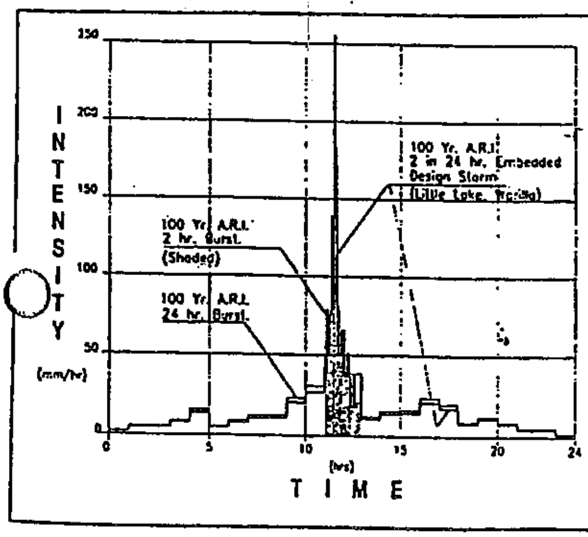


Figure 4 : EMBEDDED DESIGN STORM HYETOGRAPH

ite ies are adjusted each side of the embedded burst, to :aintain the specified average intensity for the design storm, ›llowing the AR&R temporal pattern for a design burst of the ›ecified storm duration.

Figure 5, the various design hydrographs resulting from a 10˜ ARI 2 hour design storm burst, design 24 hour storm burst, and 2 in 24 hour embedded design storm burst are contrasted. It is of relevance to note that the ratio of the 2 in 24 hour embedded design storm burst discharge to that of the 2 hour design storm burst discharge is of the order of 1.2 - a similar ratio to that of a historic storm on the catchment to an extracted 2 hour burst.

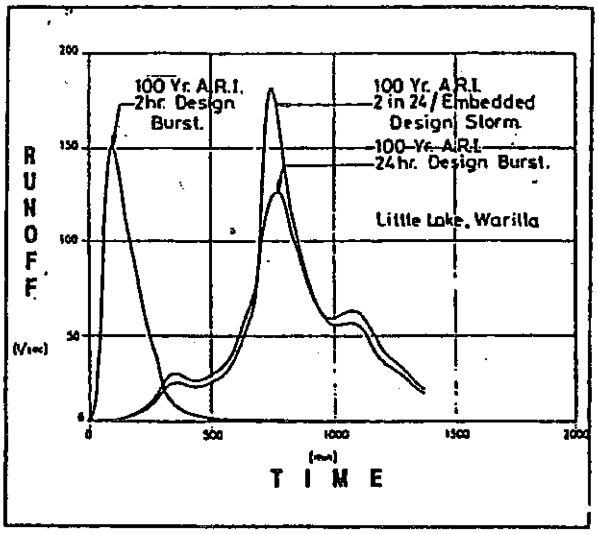


Figure 5 : DESIGN HYDROGRAPHS

6. WBNM IMPLEMENTATION

The above procedure for creation of an embedded design storm has been incorporated into the rainfall runoff model WBNM. Since the procedure outlined above is likely to change as the procedure evolves, the present implementation of embedded design storms in WBNM is both optional (users may follow the standard AR&R burst procedures if they wish) and generic (both burst duration and storm duration being input by the user together with a factor to adjust the average intensity of the design storm envelope). To automatically invoke the embedded design storm routines, a user adds the specified storm duration and desired multiplication factor for the intensity of the design storm envelope to the line containing the design storm burst ARI, duration and areal reduction factor flag;

When these optional data entries are absent, design follows the standard 1987 AR&R burst procedure.

In operation, WBNM subdivides the design storm in to time steps matching those of the design burst, locates the central portion of the peak of the design storm and overlays the storm with the burst, centred on the design storm peak.

The average intensity of the design storm is then calculated, factored up or down by the user specified multiplication factor, and the remaining ordinates of the design storm adjusted such that the factored average embedded design storm intensity is as specified.

On completion, an embedded design storm with a storm duration equal to that of the requested storm, containing the specified design storm burst is available for further processing by WBNM. (in the same way as for an AR&R 1987 design storm burst).

7. CONCLUSIONS

It is the authors' conclusions that;

1. There is need to re-examine the fundamental logic behind the present AR&R 1987 design storm burst procedure.

2. Using modern computer based simulation models, it is not reasonable to expect storm bursts to reliably predict discharges of a given ARI without significant and for the most part arbitrary adjustment to model losses and starting storage levels.

3. In order to overcome difficulties inherent in the design storm burst approach, either a consistent means of adjusting models for the loss and storage problems present needs to be developed <u>or</u> a design storm based approach needs to be adopted.

4. Whilst losses can be adjusted to partly overcome the problem of applicable storm burst losses, adjustment of models to accommodate a burst with substantially less volume than the historic event from which it was extracted, is a much more difficult issue to resolve. It seems more logical and valuable in the long term to pursue development of a procedure for creation of design storms to replace the storm burst procedure, than to patch the present procedure.

5. Existing IFD and storm burst temporal data can be used to create an embedded design storm by merging a specified design storm burst of given ARI with a longer duration design storm(burst) pattern (typical of storms in the region). Care is needed to ensure that the relationship of the average intensity of the design storm to that of the design burst is typical of storms in the region. Joint probability issues may otherwise create runoff with an ARI different from that of the design burst.

6. In the Illawarra region a 24 hour base appears reasonable for the design storm duration into which a design burst of the specified (critical) duration is to be embedded. To reflect the pattern of historic flood producing storms in the region, the average intensity of the embedded design storm should be equal to that of a 24 hour burst of the same ARI as that of the embedded design storm burst (and flood to be modelled).

7. Whilst the impact of embedment on discharge depends greatly on the characteristics of the catchment, design discharges in the Illawarra are typically increased by 10 to 30 percent when a design storm burst is embedded in a design storm complying with the above recommendations. Where major storages (natural or man made) are present this factor increases.

8. The procedure outlined should avoid most of the deficiencies of the design burst approach.

   In particular;

   - Rainfall losses are subtracted from the design storm rather than the short duration design burst.

   - Surface and channel flows from a design storm with a temporal pattern of near average variability and appropriate IFD profile exist at the commencement of the embedded burst, with natural and man made storages partly full.

   - The catchment is 'warmed up' by this procedure in a logical manner consistent with the present concepts and data of AR&R 1987.

9. Whilst the above procedure avoids most of the deficiencies of the storm burst procedure it does not address the question of whether maximised discharge from a range of burst durations is necessarily the best estimate of a flood discharge of given ARI. This aspect is in need of further investigation.

9. REFERENCES

1. INSTITUTION OF ENGINEERS, AUSTRALIA "First report of The Stormwater Standards Committee of The Institution on Australian Rainfall and Runoff," 1958, IE Aust.

2. INSTITUTION OF ENGINEERS, AUSTRALIA "Australian Rainfall and Runoff. Flood analysis and design," ed. by A. Pattison, J.K.C. Ward, T.A. McMahon and B. Watson, 1977, IE Aust. ACT.

3. INSTITUTION OF ENGINEERS, AUSTRALIA "Australian Rainfall and Runoff. A guide to flood estimation Volume 1", ed. by D.H. Pilgrim, 1987, IE Aust.

4. PHILLIPS, B., LEES, S., LYNCH, S. "Embedded Design Storms - An Improved Procedure for Design Flood Level Determination", <u>Proc. Water Down Under '94</u>, 1994, Adelaide, IE Aust.

5. BOYD, M., RIGBY, E., SHARPIN, M., VAN DREI,

R. "Enhanced Runoff Routing Model WBNM94" Proc Water Down Under '94, 1994, Adelaide IE Aust.

FRENCH, R. & WALSH, M., "Increasing the Confidence of Australian Flood Design", Proc. Hydrology and Water Resources Symposium Christchurch NZ., 1989, IE Aust.

7. LINSLEY K, KOHLER M, PAULUS J "Hydrology for Engineers", 1958, McGraw Hill, New York.

Water Down Under '94
Adelaide, Australia, 21-25 November 1994

Embedded Design Storms - An Improved Procedure for Design Flood Level Estimation?

B.C. PHILLIPS, BE (Civil), MEngSc, PhD, MIEAust.
Associate Director, Willing & Partners Pty Ltd
S.J. LEES, BE (Hons), Ph.D., MIEAust.
Executive Officer, Upper Parramatta River Catchment Trust
S.J. LYNCH, BE (Civil)
Engineer, Upper Parramatta River Catchment Trust

SUMMARY When the traditional design storm approach was used to generate design flood hydrographs for an unsteady flood routing model of the Parramatta River and tributary creeks it was noted that 1% AEP design flood levels estimated by the model were in the order of 1.0 m lower than previously estimated in the lower reaches of the study area in the vicinity of the Parramatta CBD.

It was eventually concluded that the apparent over-attenuation of the design flood wave was due to the fact that the creeks and channels were being started "dry" and that the available channel storage particularly in the lower reaches was significant in comparison with the volume of the critical 90 minute duration storm. Consequently a decision was made to embed the design rainfall burst obtained from AR&R, 1987 into the historical April 1988 storm. This approach gave design flood levels which were more in keeping with historical flood levels and previously estimated design flood levels and confirmed the benefits of this approach. The adoption of the "embedded" design storm approach raises a number of questions which have potential implications for current design flood estimation practices particularly in urban catchments with short duration critical storms including:

(i) is an "embedded" design storm more realistic than a stand-alone (isolated) design storm burst?,
(ii) is channel and floodplain storage leading to under-estimates of design flood levels because creeks and channels are being started "dry" when in reality antecedent rainfall and runoff are partially filling creeks and stream prior to higher rainfall bursts within storms?;
(iii) are the required storage volumes for retarding basins being under-estimated because it is also assumed that retarding basins are "dry" at the commencement of design rainfall?

It was concluded that in the case of the Upper Parramatta River catchment that an "embedded" design storm approach provides more realistic design flood levels than the traditional design flood estimation approach.

1. INTRODUCTION

The upper catchments of the Parramatta River are some of the most urbanised catchments in Australia. Urbanisation has brought flood problems caused by increased runoff from storm rainfall, faster response times and development in flood prone areas.

The catchment of the Parramatta River above the Charles Street Weir has an area of 110 km². The greater part of the catchment is drained by the Toongabbie Creek while the Darling Mills Creek and Hunts Creek are also important tributaries (refer Figure 1). Investigations into potential flood problems in the catchment were first instigated in 1976 by the then Australian Government's Department of Urban and Regional Development.

In 1976, a study recommended a flood mitigation and stormwater drainage scheme comprising 22 retarding basins and minimum enlargement works to existing creeks (SMEC (1)). In 1980 a further study was carried out to detail changes in urbanisation since the 1976 report and to formulate retarding basin modifications in response to the changes (SMEC and Willing & Partners (2)).

Prior to the 1976 and 1980 studies reasonably large floods occurred in 1956 and again in 1967 but property damage was not widespread.

Subsequently in August 1986, April 1988 and June 1991 major floods occurred within the catchment.

In 1989 the Department of Water Resources released a report on1% Annual Exceedance Probability (AEP) design flood levels throughout the catchment (DWR (3)). The 1% AEP flood levels were derived using the RAFTS rainfall/runoff model to estimate design flood flows and the steady state HEC-2 water surface profile model to estimate flood levels. In view of the size of the catchment and the very large number of cross sections (over 500) the HEC-2 model was segmented into a series of sub-models of all the major creeks and streams within the catchment.

In recent years further studies have been undertaken to convert the former HEC-2 models into EXTRAN-XP models to provide improved modelling of the overbank/floodplains to enable the Upper Parramatta River Catchment Trust to issue revised 1% AEP flood levels. The aim of the modelling has been to better account for dynamic floodplain storage and to better model flow diversions.

2. EXTRAN-XP

The XP-EXTRAN package is the EXPERT (XP) graphical implementation of the EXTRAN program. EXTRAN is a hydraulic flow routing model for both open channel and closed conduits in dendritic and looped networks. The model can simulate branched or looped networks, backwater due to tidal or nontidal conditions, free-surface flow, pressure or surcharge flow, flow reversals, flow transfer by weirs, orifices and pumping facilities, and ponds or lake storages.

A major feature of Version 4 of EXTRAN which was released in 1988 was its ability to model both closed conduits and irregular cross sections. Consequently, the EXTRAN model has been used to investigate a number of large drainage systems including Toongabbie, Pendle Hill, Greystanes and Grantham Creeks and the Parramatta River and Parramatta CBD (Willing & Partners (4), (5), (6), (7)) in Sydney. The application of the EXTRAN-XP program to the hydraulic investigation of the Toongabbie Creek confluence area has also been previously described by Phillips, et al. (8).

3. MODELLING APPROACH

3.1 Hydrology

Inflow hydrographs for the historical and design floods are estimated using the RAFTS-XP rainfall/runoff model and outputted in the form of RAFTS interface files. These files enable flood hydrographs to be directly imported into the EXTRAN-XP unsteady flood routing model. These inflow hydrographs include both total inflow hydrographs at the boundaries of the study area and local inflow hydrographs at various locations within the study area.

The RAFTS model which is used by the UPRCT to estimate historical and design floods is a catchment model which the Trust has assembled over a number of years. This model determines flood hydrographs at 255 locations throughout the 110 km² catchment to the tidal limit of the Parramatta River. The model also includes all retarding basins which have been constructed as part of the strategies proposed in 1976 and 1980.

Since issuing the 1% AEP flood levels in 1989, the RAFTS-XP rainfall/runoff model of the catchment has undergone further development. The current model differs from the RAFTS model used in the previous studies in a number of ways, including:

(i) considerably more subcatchments have been defined;
(ii) current impervious areas have been determined by detailed examination of recent air photos correlated to Council planning instruments;
(iii) the adoption of the Australian Representative Basins Model (ARBM) for the determination of rainfall losses instead of the initial loss / continuing loss model; and
(iv) the definition of 16 rainfall zones across the catchment.

The RAFTS model, which has been calibrated to the August 1986, April 1988 and June 1991 floods is used to provide inflow hydrographs throughout the study area for both the historical storms and the design storms.

3.2 Hydraulics

The approach adopted when converting former HEC-2 models into EXTRAN-XP models includes:

The direct comparison of the results from the existing main channel HEC-2 models and the EXTRAN-XP model has been facilitated by adopting a specific convention for entering HEC-2 cross section data so that EXTRAN-XP computes water surface elevations at the same locations as those previously reported by the HEC-2 model.

The adopted convention is to:-

(i) locate junctions (nodes) at existing HEC-2 cross section locations and adopt the cross section invert as the junction invert level, and

(ii) input the HEC-2 cross section at which a junction is defined into the downstream conduit.

In order to contain the size of the EXTRAN-XP within the model limits, it has been necessary on occassions to rationalise the number of HEC-2 cross sections which have been input. This rationalisation of cross sections also assists to limit instability problems due to very short conduits.

All road and rail crossings and culverts within the study area are included within the model(s). In the case of road crossings these sections are modelled as multiple conduits with the bridge waterway and/or culverts being input separately from the cross section or weir section which is input to represent the road geometry under overtopping flow conditions.

'eas where historical flooding records indicate that ..ificant breakouts of floodwaters occur .a network model arrangement is used to model the overbank area. The main overland flow channels (eg. roads, depressions, etc) are identified and nodes are located in areas where it is considered that the flow may converge or diverge. These flow paths are modelled using simplified natural sections with weir section connections used where it is anticipated that flows would be restricted by fence lines or there was a natural ridge between adjacent flowpaths.

4. HISTORICAL AND DESIGN FLOODS

4.1 Historical Floods

The calibration of the EXTRAN-XP model(s) has been undertaken for the April 1988 and August 1986 floods. The June 1991 flood data has been subsequently used to verify the model(s). The order in which model parameters are adjusted to achieve a model calibration is:

(i) adjustment of Manning roughness values for cross sections and culverts,
(ii) adjustment of weir discharge coefficients,
(i" adjustment of entry and exit loss terms, and
(adjustment of the widths of overbank flow to represent ineffective flow areas.

Manning roughness values are adjusted within physically realistic limits using site inspection photographs and available orthophotomaps. These adjustments are primarily in vegetated channels and/or overbank areas.

Weir sections were used to model the breakout of floodwaters from the main channels into the overbank 'reas. These flows are typically through property boundaries and fences hence the adopted weir crest level is generally slightly above the natural surface level and weir discharge coefficients are low to represent the impedance to overland flow by fences. Excellent agreement has been obtained between the predicted and observed flood levels in various areas within the catchment for the three recent historical floods.

4.2 Design Floods

Traditionally the approach used to estimate design flood levels was to input design flood hydrographs generated by the RAFTS-XP rainfall/runoff model using the design storms contained in the 1987 edition of Australian Rainfall & Runoff.

These design flood hydrographs were input into the various EXTRAN-XP models and it has been determined that the 90 minute design storm is the critical storm for most locations within the catchment.

In recent design flood runs it was noted that 1% AEP design flood levels were approximately 1.0 m lower than previously estimated in the lower reaches of the study area in the vicinity of the Parramatta CBD in spite of small increases in peak flows resulting from improved RAFTS-XP modelling. Sensitivity runs of the hydraulic model disclosed that reducing the channel roughness, changing loss coefficients or further subdividing the river reaches in the model had no effect on the calculated attenuation of the flood wave.

After much discussion and re-running of the RAFTS-XP rainfall/runoff model with conceptual storages to represent in-stream flood storage it was concluded that the apparent over-attenuation of the design flood wave was due to the fact that the creeks and channels were being started "dry" and that the available channel storage particularly in the lower reaches was significant in comparison with the volume of the critical 90 minute duration storm.

The UPRCT is moving towards continuous, rather than event modelling of the catchment hydrologic and hydraulic behaviour. To this end the RAFTS-XP hydrologic model has been configured so that it can be linked to input from the 16 automatic rain gauges in the catchment via radio telemetry. To allow for rainfall losses the ARBM model is used to allow for the recovery in infiltration capacity between rainfall bursts. When the RAFTS-XP model was calibrated to historical storms, the entire event, extending over 2 or 3 days is considered. By adopting this approach the resulting inflow hydrographs exported to the hydraulic model are hydrographs of total runoff including groundwater inflows; not simply the surface runoff which would result from isolated rainfall bursts.

Because of the approach taken to calibrate the models, it seemed logical that the design storm should also cover an extended period before and after the storm peak. As a result, a decision was made in early 1993 to adopt a new approach to the estimation of design flood flows. This new approach is to embed design rainfall bursts obtained from Australian Rainfall & Runoff into the April 1988 storm. The April 1988 storm was adopted for several reasons including:-

(i) the April 1988 flood was similar in magnitude to a 1%AEP flood within the catchment; and
(ii) the available rainfall data for the April 1988 storm was more comprehensive than for the August 1986 and the June 1991 floods.

Rainfall for a 42 hour subset of the 60 hour period of rainfall from the 28 April 1988 to 30 April 1988 for two representative locations within the catchment are given in Figure 2.

The procedure which has been adopted is to replace the most intense rainfall burst within the 1988 storm which corresponds to each design rainfall duration with the design rainfall determined from AR&R, 1987 and to run the RAFTS-XP model to generate flood hydrographs which incorporate the embedded design rainfall.

The 1988 storm has been previously segmented into three sections to facilitate the estimation of design flows. The first rainfall segment is run and the conditions at the end of the rainfall record are saved and become the initial conditions for the next segment. Likewise, the second segment sets the intial conditions for the flood producing third segment of rainfall. Consequently it is only necessary to re-run the third segment each time a new embedded design hydrograph is needed.

Re-running the hydraulic model with the embedded design storm gave design flood levels which were more in keeping with historical flood levels and previously estimated design flood levels and confirmed the benefits of this approach.

It should be noted that in the upper reaches of the catchment where there is limited in-channel storage, the differences between the 1% AEP flood levels under isolated and embedded design storms is only 0.1-0.3 m in total depths of 3 to 4 m.

The adoption of the "embedded" design storm approach raises a number of questions which have potential implications for current design flood estimation practices particularly in urban catchments with short duration critical storms. These include:

(i) is an "embedded" design storm more realistic because it recognises the process which was followed to determine design rainfalls in the first instance ie. rainfall bursts were excised from real storms and analysed statistically to provide the design rainfalls presented in AR&R, 1987;

(ii) when applying unsteady flood routing models to design flood level estimation for short duration design storms is channel and floodplain storage leading to under-estimates of design flood levels because the creeks and channels are started "dry" when in reality antecedent rainfall and runoff are partially filling creeks and stream prior to higher rainfall bursts within storms; and

(iii) are the required storage volumes for retarding basins being under-estimated because it is also assumed that retarding basins are "dry" at the commencement of design rainfall when in reality antecedent rainfall and runoff are partially filling retarding basins prior to higher rainfall bursts within storms

5. CONCLUSIONS

The traditional approach to design flood estimation has been to input design rainfalls into a rainfall/runoff model, adopt rainfall losses and to generate design flood hydrographs. When this approach was used to generate design flood hydrographs for an unsteady flood routing model it was noted that the resulting 1% AEP design flood levels estimated by the model were more than 0.5 m lower than previously estimated particularly in the lower reaches of the study area in the vicinity of the Parramatta CBD.

It was eventually concluded that the apparent over-attenuation of the design flood wave was due to the fact that the creeks and channels were being started "dry" and that the available channel storage particularly in the lower reaches was significant in comparison with the volume of the critical 90 minute duration storm. As a result of these investigations a decision was made to embed the design rainfall burst obtained from AR&R, 1987 into the historical April 1988 storm. Re-running the hydraulic model with the "embedded" design storm gave design flood levels which were more in keeping with historical flood levels and previously estimated design flood levels and confirmed the benefits of this approach.

The adoption of the "embedded" design storm approach raises a number of questions which have potential implications for current design flood estimation practices particularly in urban catchments with short duration critical storms. These include:

(i) is an "embedded" design storm more realistic than a stand-alone design storm burst?

(ii) is channel and floodplain storage leading to under-estimation of design flood levels because creeks and channels are being started "dry" when in reality antecedent rainfall and runoff are partially filling creeks and stream prior to higher rainfall bursts within storms?

(iii) are the required storage volumes for retarding basins being under-estimated bacause it is also assumed that retarding basins are "dry" at the commencement of design rainfall?

It is concluded that in the case of the Upper Parramatta River catchment that an "embedded" design storm approach is providing more realistic design flood levels than the traditional design flood estimation approach.

6. ACKNOWLEDGMENT

The permission of the Upper Parramatta River Catchment Trust to publish this paper to promote further discussion is gratefully acknowledged. The views expressed in the paper are those of the authors and are not necessarily the views of the Trust.

7. REFERENCES

1. Snowy Mountains Engineering Corporation "Parramatta River Basin Drainage Study", Final Report, prepared for the Sydney Western Regional Organisation of Local Authorities, 1976.

2. Snowy Mountains Engineering Corporation and Willing & Partners "Parramatta River Flood Mitigation Investigation", Final Report, prepared for the NSW Water Resources Commission, October, 1980.

3. Department of Water Resources "Toongabbie Creek Flood Study", Draft Report No. 2, May, 1989.

4. Willing & Partners Pty Ltd "EXTRAN-XP Model of Toongabbie Creek, Pendle Hill Creek and Greystanes Creek ", Final Report, prepared for UPRCT, October 1992, Sydney.

5. Willing & Partners Pty Ltd "EXTRAN-XP Model of Toongabbie Creek, Blacktown Creek, Lalor Creek and Grantham Creek ", Draft Final Report, prepared for UPRCT, July 1993, Sydney.

6. Willing & Partners Pty Ltd "Parramatta River EXTRAN Model ", Final Report, prepared for UPRCT, March 1991, Sydney.

7. Willing & Partners Pty Ltd "Parramatta River CBD Flood Model", Final Report, prepared for UPRCT, November 1992, Sydney.

8. Phillips, B.C., Haege, M., Lynch, S.J. and Dickinson, R.E. "Unsteady Flow Modelling of Toongabbie Creek Confluence Area Using XP-EXTRAN", Proceedings, Watercomp'93, IEAust. Conf. Pub. No. 93/2, 30 March - 1 April 1993, Melbourne, pp. 111-116.

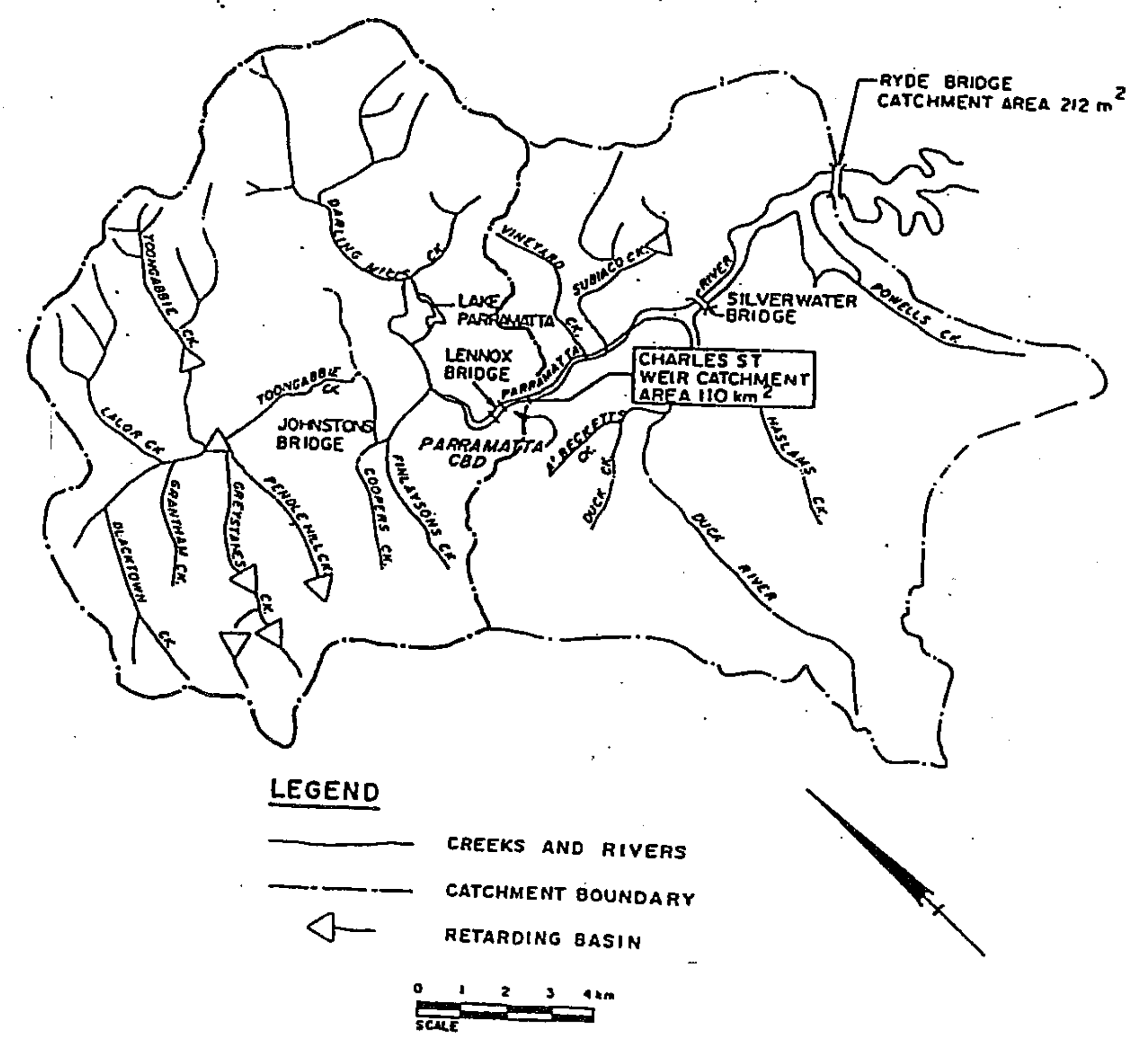


Figure 1 The Parramatta River Catchment

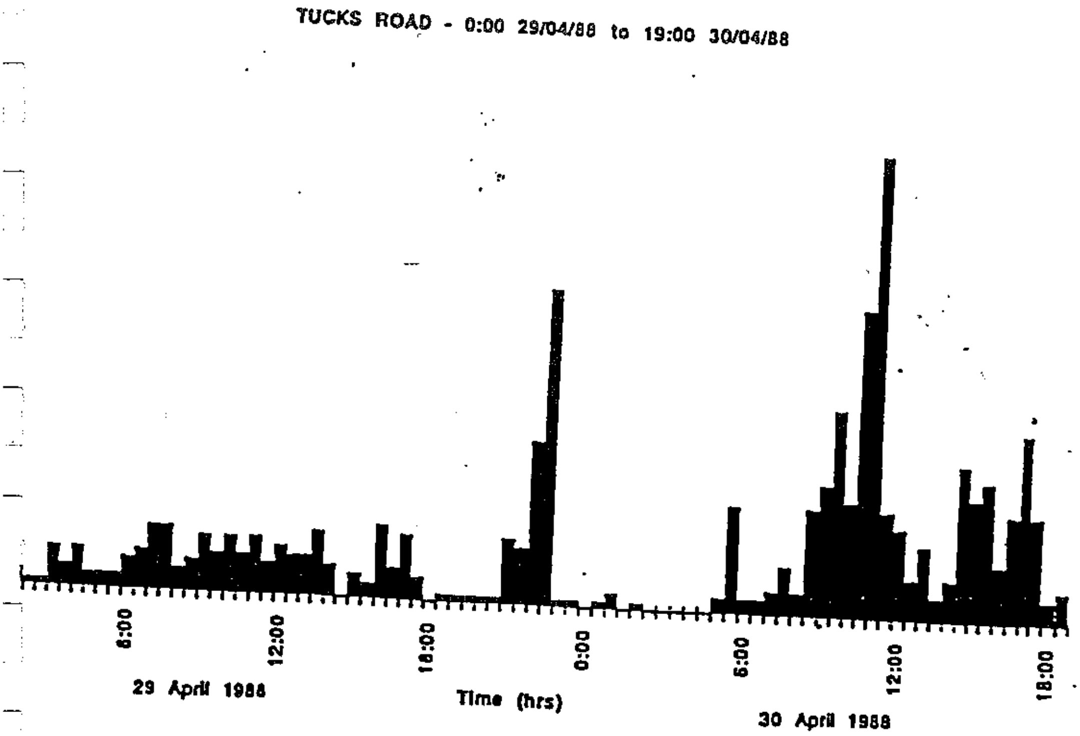
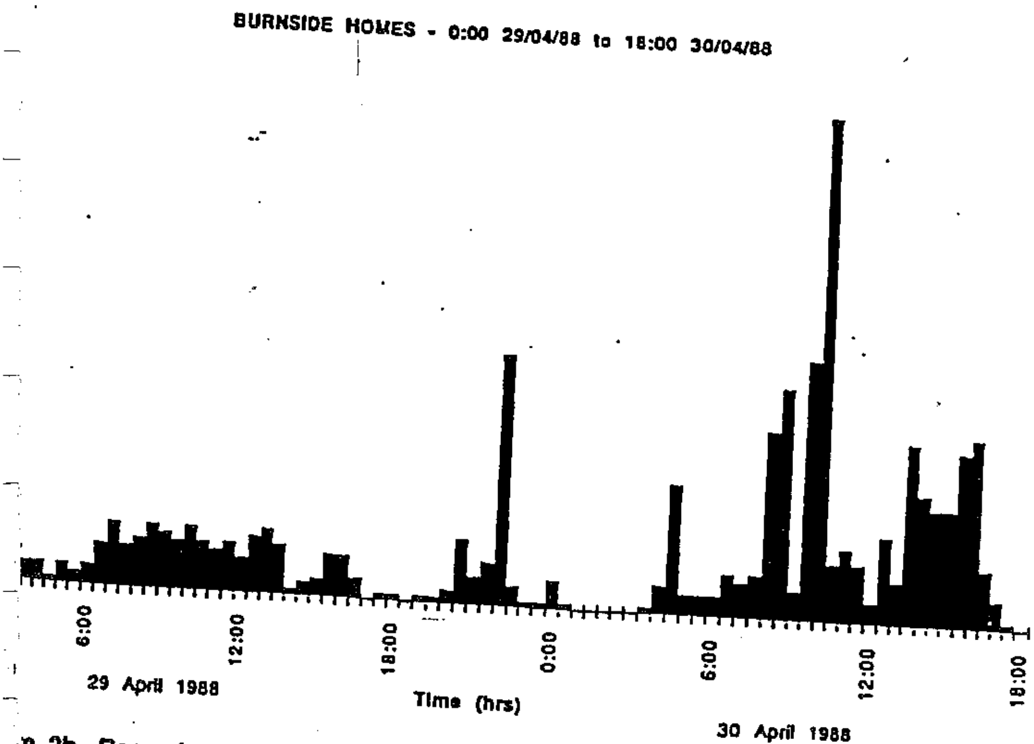


Figure 2a Recorded Rainfall at Tucks Road 00:00 29/04/88 to 18:00 30/4/88



e 2b Recorded Rainfall at Burnside Homes 00:00 29/04/88 to 18:00 30/4/88

APPENDIX D

RAFTS MODEL PARAMETERS FOR SUB-CATCHMENTS

Appendix D
SUMMARY OF CATCHMENT DATA

Link Label	Catch Area (ha)		Slope (%)		% Impervious		Pern		B		Initial Loss (mm)		Cont. Loss (mm/h)		Link Lag
	#1	#2	#1	#2	#1	#2	#1	#2	#1	#2	#1	#2	#1	#2	(mins)
T5	31.24	13.60	6.11	6.11	5	100	0.04	0.01	0.07	0.00	10	5	2	1	2.60
TA4	20.65	0.46	5.78	5.78	5	100	0.04	0.01	0.06	0.00	10	5	2	1	0.00
TA7	7.03	4.36	9.28	9.28	5	100	0.02	0.01	0.01	0.00	10	5	2	1	0.00
TA6	8.28	0.02	11.53	11.53	5	100	0.04	0.01	0.03	0.00	10	5	2	1	0.00
DP10	0.00	0.00	0.00	0.00	0	0	0.03	0.00	0.02	0.00	5	0	1	0	0.04
TA5	13.76	0.57	7.95	7.95	5	5	0.04	0.01	0.04	0.00	10	5	2	1	0.31
TA3	9.11	2.07	6.85	6.85	5	100	0.04	0.01	0.04	0.00	10	5	2	1	0.00
DP9	0.00	0.00	0.00	0.00	0	0	0.03	0.00	0.02	0.00	5	0	1	0	0.06
TA2	9.48	4.41	7.79	7.79	5	100	0.04	0.01	0.03	0.00	10	5	2	1	0.80
TA1	4.79	3.15	8.67	8.67	5	100	0.04	0.01	0.02	0.00	10	5	2	1	0.00
T8	8.04	5.95	3.87	3.87	5	100	0.02	0.01	0.02	0.00	10	5	2	1	0.00
T10	7.27	2.74	2.72	2.72	5	100	0.02	0.01	0.03	0.00	10	5	2	1	0.00
T9	20.29	0.00	3.72	0.00	5	0	0.02	0.00	0.04	0.00	10	0	2	0	0.00
TB2	10.66	10.66	9.08	9.08	5	100	0.02	0.01	0.02	0.00	10	5	2	1	0.13
TB1	11.35	11.35	5.12	5.12	5	100	0.02	0.01	0.02	0.00	10	5	2	1	0.00
TD5	17.75	0.00	9.41	0.00	5	0	0.02	0.00	0.02	0.00	10	0	2	0	0.69
TD4	23.01	0.00	6.54	0.00	5	0	0.02	0.00	0.03	0.00	10	0	2	0	0.93
TD2	21.54	3.71	5.25	5.25	5	100	0.02	0.01	0.03	0.00	10	5	2	1	0.00
TC4	15.04	0.00	10.12	0.00	5	0	0.02	0.00	0.02	0.00	10	0	2	0	0.51
TC2	15.76	0.00	6.43	0.00	5	0	0.02	0.00	0.03	0.00	10	0	2	0	0.00
TC3	16.74	1.23	7.61	7.61	5	100	0.02	0.01	0.02	0.00	10	5	2	1	4.28
TC1	9.51	0.17	5.92	5.92	5	100	0.02	0.01	0.02	0.00	10	5	2	1	0.00
T16	17.25	0.04	7.55	7.55	5	100	0.02	0.01	0.03	0.00	10	5	2	1	0.00
T18	7.76	0.00	6.82	0.00	5	0	0.02	0.00	0.02	0.00	10	0	2	0	0.00
T19	23.97	0.00	16.16	0.00	5	0	0.02	0.00	0.02	0.00	10	0	2	0	0.00
T17	16.29	0.04	11.44	11.44	5	100	0.02	0.01	0.02	0.00	10	5	2	1	0.00
DP8	0.00	0.00	0.00	0.00	0	0	0.03	0.00	0.02	0.00	5	0	1	0	0.34
DP7	0.00	0.00	0.00	0.00	0	0	0.03	0.00	0.02	0.00	5	0	1	0	0.50
T15	21.67	0.00	6.63	0.00	5	0	0.02	0.00	0.05	0.00	10	0	2	0	0.65
T14	10.37	2.61	5.05	5.05	5	100	0.02	0.01	0.02	0.00	10	5	2	1	0.00
TD3	13.83	0.96	6.62	6.62	5	100	0.02	0.01	0.02	0.00	10	5	2	1	0.00
TD1	6.33	5.52	6.52	6.52	5	100	0.02	0.01	0.02	0.00	10	5	2	1	0.00
DP6	0.00	0.00	0.00	0.00	0	0	0.03	0.00	0.02	0.00	5	0	1	0	0.50

Appendix D
SUMMARY OF CATCHMENT DATA

Link	Catch Area (ha)		Slope (%)		% Impervious		Pern		B		Initial Loss (mm)		Cont. Loss (mm/h)		Link Lag
Label	#1	#2	#1	#2	#1	#2	#1	#2	#1	#2	#1	#2	#1	#2	(mins)
T13	23.01	11.44	4.89	4.89	5	100	0.02	0.01	0.04	0.00	10	5	2	1	0.00
T12	11.74	5.38	4.38	4.38	5	100	0.02	0.01	0.03	0.00	10	5	2	1	0.00
DP5	0.00	0.00	0.00	0.00	0	0	0.03	0.00	0.02	0.00	5	0	1	0	5.94
T11	17.40	7.82	4.26	4.26	5	100	0.02	0.01	0.03	0.00	10	5	2	1	5.30
DP4	0.00	0.00	0.00	0.00	0	0	0.03	0.00	0.02	0.00	5	0	1	0	0.00
T7	6.99	0.00	0.01	0.00	0	0	0.02	0.00	0.53	0.00	10	0	2	0	6.54
T6	17.10	3.53	4.06	4.06	5	100	0.02	0.01	0.03	0.00	10	5	2	1	2.60
T4	15.37	4.64	1.56	1.56	5	100	0.02	0.01	0.05	0.00	10	5	2	1	3.69
T3	7.46	4.21	1.69	1.69	5	100	0.02	0.01	0.03	0.00	10	5	2	1	3.54
BB1	4.50	4.50	1.86	1.86	5	100	0.02	0.01	0.03	0.00	10	5	2	1	0.00
T2	13.30	13.09	1.88	1.88	5	100	0.02	0.01	0.04	0.00	10	5	2	1	0.00
T1	13.03	11.21	2.13	2.13	5	100	0.02	0.01	0.04	0.00	10	5	2	1	0.00
B1	6.96	5.38	0.79	0.79	5	100	0.02	0.01	0.05	0.00	10	5	2	1	0.00
BC2	6.69	6.69	1.91	1.91	5	100	0.02	0.01	0.03	0.00	10	5	2	1	2.41
BC1	10.99	3.62	2.51	2.51	5	100	0.02	0.01	0.03	0.00	10	5	2	1	0.00
BB2	6.16	4.21	2.11	2.11	5	100	0.02	0.01	0.03	0.00	10	5	2	1	0.00
BA2	3.70	3.70	1.26	1.26	5	100	0.02	0.01	0.04	0.00	10	5	2	1	0.00
BA3	5.65	5.65	2.27	2.27	5	100	0.02	0.01	0.03	0.00	10	5	2	1	0.00
BA5	4.48	1.71	12.87	12.87	5	100	0.02	0.01	0.01	0.00	10	5	2	1	0.66
BA4	11.05	8.93	5.82	5.82	5	100	0.02	0.01	0.02	0.00	10	5	2	1	0.00
BA13	7.22	7.22	9.54	9.45	5	100	0.02	0.01	0.01	0.00	10	5	2	1	0.29
BA12	8.23	6.70	8.25	8.25	5	100	0.02	0.01	0.02	0.00	10	5	2	1	0.18
BA11	13.64	6.60	5.62	5.62	5	100	0.02	0.01	0.03	0.00	10	5	2	1	0.65
BA10	11.23	2.08	8.19	8.19	5	100	0.02	0.01	0.03	0.00	10	5	2	1	0.66
BA9	8.68	2.72	4.92	4.92	5	100	0.02	0.01	0.03	0.00	10	5	2	1	0.55
BA8	7.46	7.34	4.64	4.64	5	100	0.02	0.01	0.02	0.00	10	5	2	1	0.29
BA7	14.71	13.84	3.33	3.33	5	100	0.02	0.01	0.03	0.00	10	5	2	1	0.18
BA6	9.24	5.91	3.84	3.84	5	100	0.02	0.01	0.03	0.00	10	5	2	1	0.66
DP3	0.00	0.00	0.00	0.00	0	0	0.03	0.00	0.02	0.00	5	0	1	0	0.21
BA1	5.82	5.43	3.49	3.49	5	100	0.02	0.01	0.03	0.00	10	5	2	1	0.00
B8	10.18	10.18	3.59	3.59	5	100	0.02	0.01	0.03	0.00	10	5	2	1	0.00
B7	5.89	5.89	3.80	3.80	5	100	0.02	0.01	0.02	0.00	10	5	2	1	0.00
B16	9.49	8.93	5.31	5.31	5	100	0.02	0.01	0.02	0.00	10	5	2	1	2.03

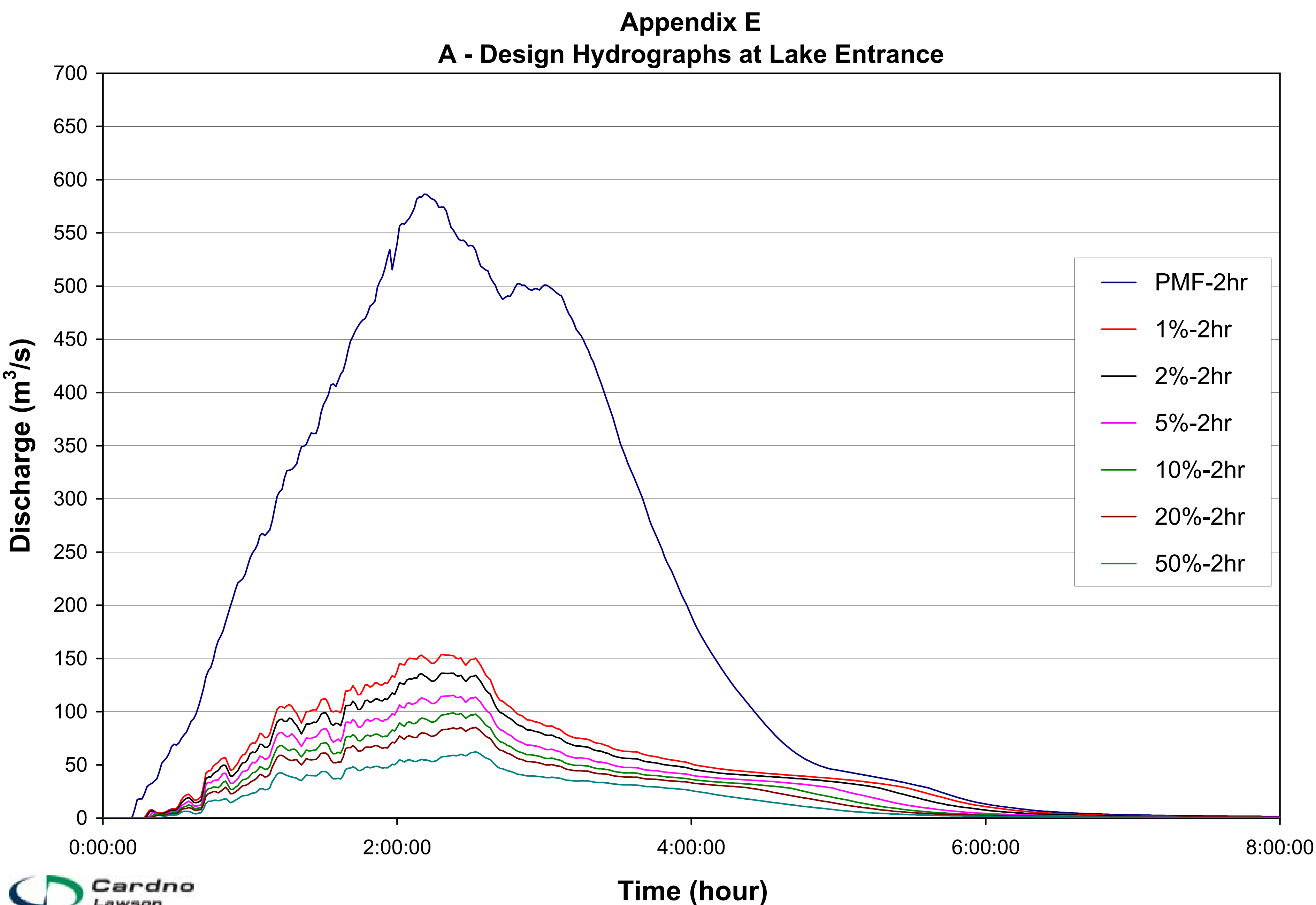
Appendix D
SUMMARY OF CATCHMENT DATA

Link Label	Catch Area (ha)		Slope (%)		% Impervious		Pern		B		Initial Loss (mm)		Cont. Loss (mm/h)		Link Lag (mins)
	#1	#2	#1	#2	#1	#2	#1	#2	#1	#2	#1	#2	#1	#2	
B15	14.48	8.65	4.69	4.69	5	100	0.02	0.01	0.03	0.00	10	5	2	1	0.42
B14	15.93	1.90	10.05	10.05	5	100	0.04	0.01	0.04	0.00	10	5	2	1	0.10
B13	14.66	8.02	5.80	5.80	5	100	0.04	0.01	0.05	0.00	10	5	2	1	0.18
B12	14.39	10.40	4.99	4.99	5	100	0.02	0.01	0.03	0.00	10	5	2	1	0.59
B11	9.62	7.33	4.30	4.30	5	100	0.02	0.01	0.02	0.00	10	5	2	1	0.06
B10	8.73	5.75	3.34	3.34	5	100	0.02	0.01	0.03	0.00	10	5	2	1	0.00
B9	7.74	7.16	1.81	1.81	5	100	0.02	0.01	0.03	0.00	10	5	2	1	0.00
B7A	7.86	1.88	0.00	0.00	5	100	0.02	0.01	1.44	0.05	10	5	2	1	0.81
B5	6.34	6.34	1.62	1.62	5	100	0.02	0.01	0.03	0.00	10	5	2	1	0.42
B4	9.13	6.09	1.66	1.66	5	100	0.02	0.01	0.04	0.00	10	5	2	1	0.37
B3	5.21	5.21	1.87	1.87	5	100	0.02	0.01	0.03	0.00	10	5	2	1	2.41
B2	7.39	3.28	0.83	0.83	5	100	0.02	0.01	0.05	0.00	10	5	2	1	0.00
BB4	25.12	20.07	4.76	4.76	5	100	0.02	0.01	0.04	0.00	10	5	2	1	0.34
BB3	15.61	13.80	2.82	2.82	5	100	0.02	0.01	0.04	0.00	10	5	2	1	0.02
DP2	0.00	0.00	0.00	0.00	0	0	0.03	0.00	0.02	0.00	5	0	1	0	5.00
DP1	0.00	0.00	0.00	0.00	0	0	0.03	0.00	0.02	0.00	5	0	1	0	0.00
DP0	0.00	0.00	0.00	0.00	0	0	0.03	0.00	0.02	0.00	5	0	1	0	0.00

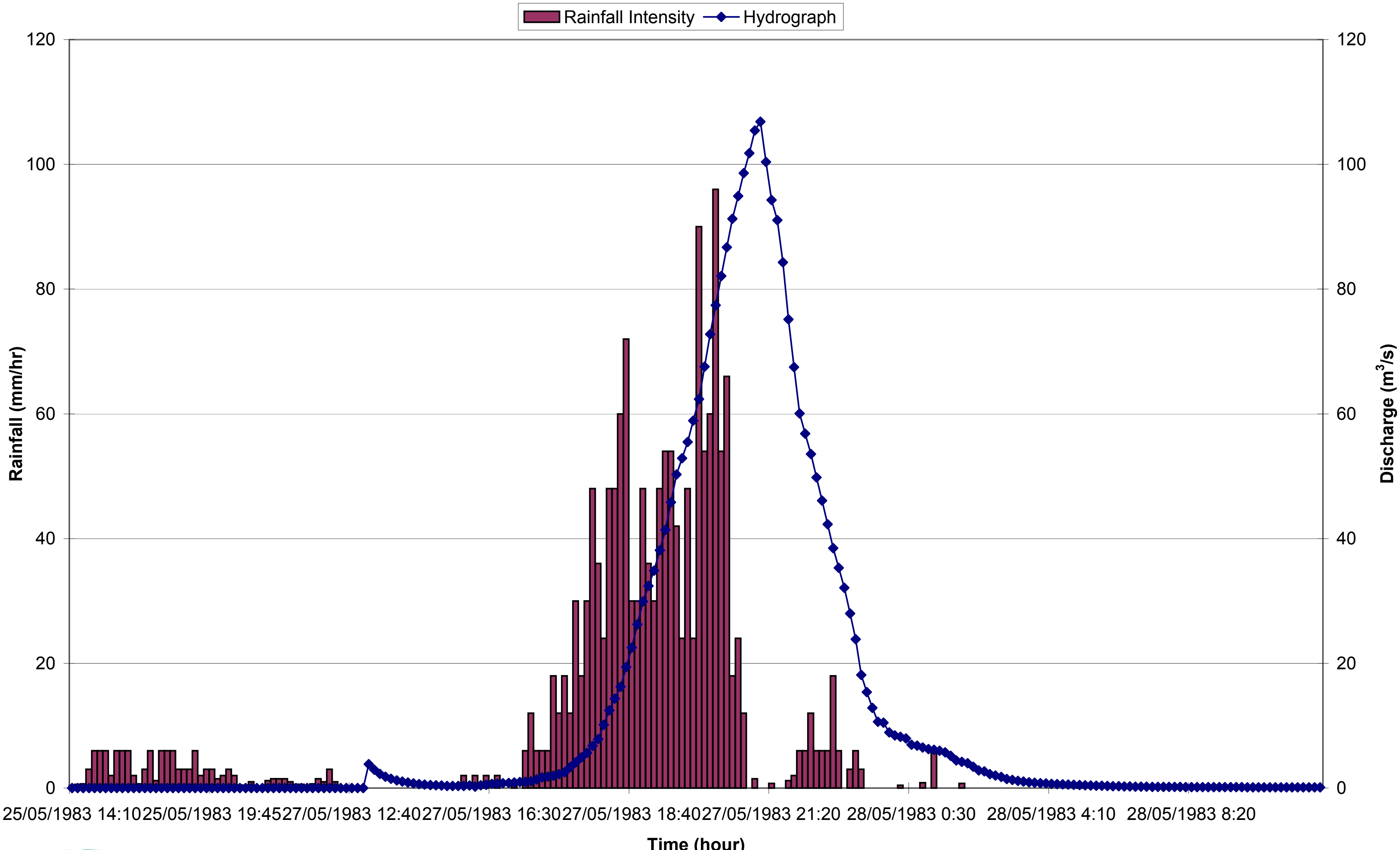
APPENDIX E

DESIGN HYDROGRAPHS AT LAKE ENTRANCE

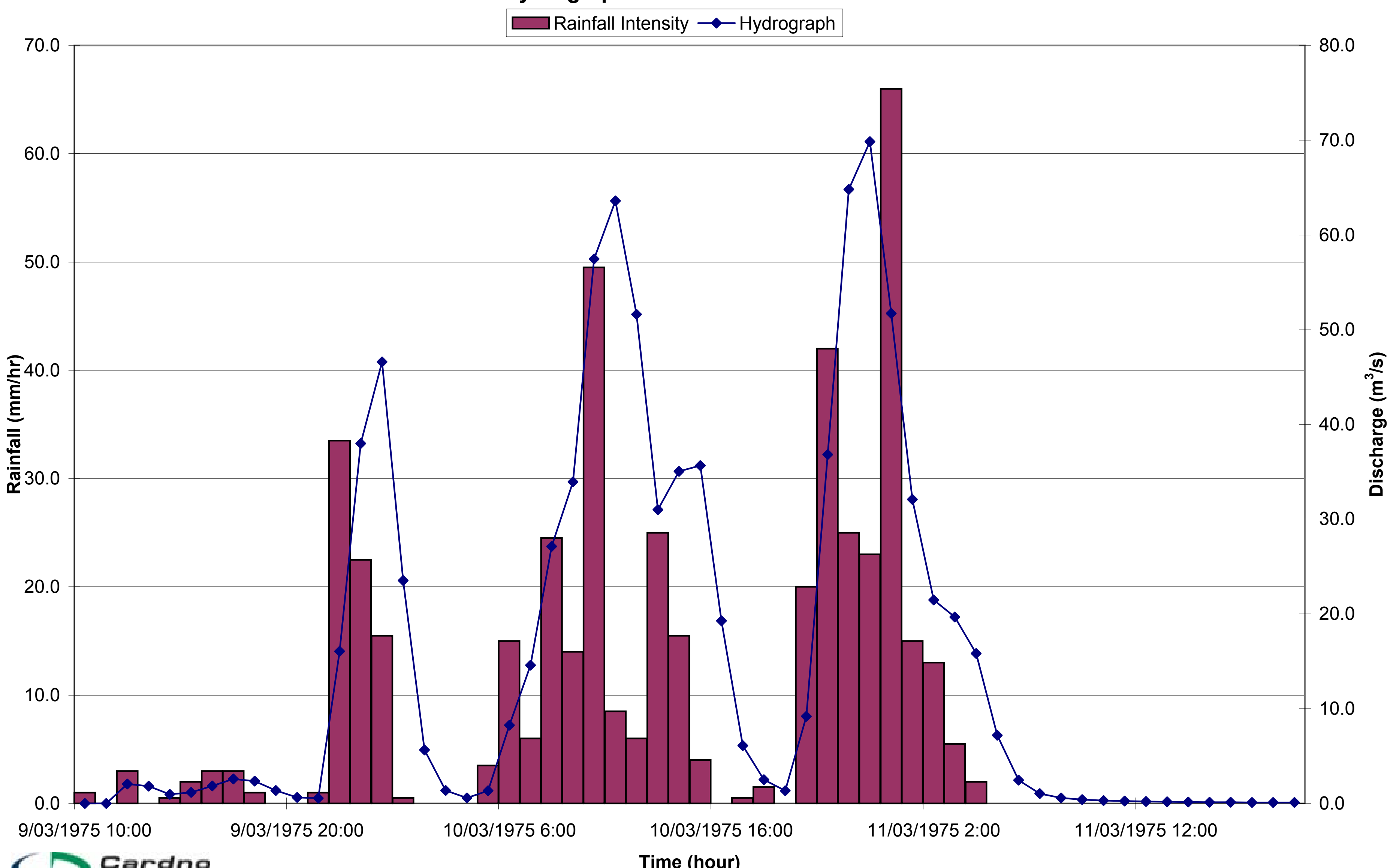
Appendix E
A - Design Hydrographs at Lake Entrance



Appendix E
B - Estimated Hydrograph at Lake Entrance for May 1983 Event



Appendix E
C - Estimated Hydrograph at Lake Entrance for March 1975 Event



Cardno Lawson Treloar

J1959/R1974/V5
January 2006

Elliot Lake - Little Lake
Flood Study

APPENDIX E (C)
ESTIMATED HYDROGRAPH AT LAKE ENTRANCE
FOR MARCH 1975 EVENT
J1959\Appendices V5\Appendix E - Hydrographs.xls

APPENDIX F

COASTAL PROCESSES

F1 WAVES

Waves which propagate to the study area may have energy in two distinct frequency bands. These are related to the generation and propagation of Pacific Ocean swell and local sea. Large waves generated by a storm are generally categorised as sea because wind energy is still being transferred to the ocean, but this distinction was not made in this study for storm waves. Waves of importance to this flood study will be the larger storm waves which only occur every few years or so.

Real waves are irregular in height and period and so it is necessary to describe wave conditions using a range of statistical parameters. In this study the following have been used:-

- H_{mo} significant wave height (H_s) based on $4\sqrt{M_o}$ where M_o is the zeroth moment of the wave energy spectrum (rather than the time domain $H_{1/3}$ parameter).
- H_{max} maximum wave height in a specified time period
- T_p wave energy spectral peak period, that is, the wave period related to the highest ordinate in the wave spectrum
- T_z average zero crossing period based on upward zero crossings of the still water line. An alternative definition is based on the zeroth and second spectral moments.

Wave heights defined by zero upcrossings of the still water line fulfil the Rayleigh Distribution in deep water and thereby provide a basis for estimating other wave height parameters from H_s. Significant wave height defined from the wave spectrum, H_{mo}, is normally larger (typically 5% to 8%) than $H_{1/3}$ defined from a time series analysis.

Real waves also have a dominant direction of wave propagation and directional spread about that direction which can be defined by a Gaussian or generalised cosine ($\cos^n$) distribution (amongst others), and a wave grouping tendency. Directional spread is reduced by refraction as waves propagate into the shallow nearshore regions and the wave crests become more parallel with each other and the seabed contours. Although neither of these characteristics is addressed explicitly in this study, directional spreading was included in the numerical wave modelling work. Directional spreading causes the sea surface to have a shorter crested wave structure in deep water.

Waves propagating into shallow water may undergo changes caused by refraction, shoaling, bed friction, wave breaking and, to some extent, diffraction.

Wave refraction is caused by differential wave propagation speeds. That part of the shoreward propagating wave which is in the more shallow water has a lower speed than those parts in deeper water. When waves approach a coastline obliquely these differences cause the wave fronts to turn and become more coast parallel.

Associated with this directional change there are changes in wave heights. On irregular seabeds wave refraction becomes a very complex process.

Waves propagating shoreward develop reduced speeds in shallow water. In order to maintain constancy of wave energy flux (ignoring energy dissipation processes) their heights must increase. This phenomenon is termed shoaling and leads to a significant increase in wave height near the shoreline.

A turbulent boundary layer forms above the seabed with associated wave energy losses which are manifested as a continual reduction in wave height in the direction of wave propagation - leaving aside further wind input, refraction and shoaling.

Wave breaking occurs in shallow water when the wave crest speed becomes greater than the phase speed. For irregular waves this breaking occurs in different depths so that there is a breaker zone rather than a breaker line. Sea bed slopes and wave steepness are important parameters affecting the wave breaking phenomenon. As a consequence of this energy dissipation, wave set-up (a rise in still water level caused by wave breaking), develops shoreward from the breaker zone in order to maintain conservation of momentum flux. This rise in water level increases non-linearly in the shoreward direction and allows larger waves to propagate shoreward before breaking. Field measurements have shown that the slope of the water surface is normally concave upward. Wave set-up at the shoreline can be in the order of 15% of the equivalent deep water significant wave height, Ho'. Less set-up occurs in estuarine entrances, but the momentum flux remains the same. Wave grouping and the consequent surf beats also cause fluctuations in the still water level. Wave set-up is particularly important for this study because the nearshore design wave heights are relatively high.

Wave diffraction will not be particularly important for this study because there are no large promontories immediately near the study location.

In a random wave field each wave may be considered to have a period different from its predecessors and successors and the distribution of wave energy is often described by a wave energy spectrum. In fact, the whole wave train structure changes continuously and individual waves appear and disappear until quite shallow water is reached and dispersive processes are reduced. In developed sea states, for example, swell, the Bretschneider modified Pierson-Moskowitz spectral form has generally been found to provide a realistic wave energy description. For developing sea states the JONSWAP spectral form, which is generally more 'peaky', has been found to provide a better spectral description.

For structural design in the ocean environment it is necessary to define the H_{max} parameter related to storms having average recurrence intervals of up to 100 years. However, the expected H_{max}, relative to H_s in statistically stationary wave conditions, increases as storm duration increases. Based on the Rayleigh Distribution the usual relationship is:-

$$H_{max} = H_s \sqrt{(0.5 \, \ell n Nz)} \qquad \textbf{(F1)}$$

where Nz is the number of waves occurring during the time period being considered, where waves are defined by Tz.

This relationship has been found to overestimate H_{max} by about 10% in severe ocean storms. In shallow water the relationship described by equation (F1) is not fulfilled. In very shallow water H_{max} is replaced by the breaking wave height, H_b.

Waves propagating through an area affected by a current field are caused to turn in the direction of the current. The extent of this direction change depends on wave celerity, current speed and relative directions. Wave height is also changed. Opposing currents cause wave lengths to shorten and wave heights to increase and may lead to wave breaking. When the current speed is greater than one quarter of the phase speed the waves are blocked. Conversely, a following current reduces wave heights and extends wave lengths. This phenomenon is not important to this study.

F2 WATER LEVELS

Water level variations at the coastline result from one or more of the following natural causes:

- Eustatic and Tectonic Changes
- Tides
- Wind Set-up and the Inverse Barometer Effect
- Wave Set-up
- Wave Run-up
- Fresh Water Flow
- Tsunamis
- Greenhouse Effect
- Global Changes in Meteorological Conditions

Eustatic sea level changes are long term world wide changes in sea level relative to the land mass and are generally caused by changes to the polar ice caps. No rapid changes are believed to be occurring at present and this aspect has not been addressed. Nevertheless, a minimum rise of 1mm per annum is now generally accepted.

Tides are caused by the relative motions of the Earth, Moon and Sun and their gravitational attractions. While the vertical tidal fluctuations are generated as a result of these forces, the distribution of land masses, bathymetric variation and the Coriolis force determine the local tidal characteristics. Tidal level time series included as part of storm tide simulations for this study have been based on tidal constants for Sydney presented in Australian National Tide Tables (1999) and application of the so-called Canadian tidal prediction package. There is little variation in the astronomical tide in the NSW Central Coast region and those determined at Sydney are suitable for Shellharbour.

Wind set-up and the inverse barometer effect are caused by regional meteorological conditions. When the wind blows over an open body of water, drag forces develop between the air and the water surface. These drag forces are proportional to the square of the wind speed. The result is that a wind drift current is generated. This current may transport water towards the coast upon which it piles up causing wind set-up. Wind set-up is inversely proportional to depth. It is not particularly high in the South Coast region of NSW because of the relatively deep water close to the shoreline.

In addition, the drop in atmospheric pressure, which accompanies severe meteorological events, causes water to flow from high pressure areas on the periphery of the meteorological formation to the low pressure area. This is called the "inverse barometer effect" and results in water level increases up to 1cm for each millibar drop in central pressure below the average sea level pressure in the area for the particular time of year, typically 1010 hPa. The actual increase depends on the speed of the meteorological system and 1cm is only achieved if it is moving slowly. The phenomenon causes daily variations from predicted tide levels up to 0.1m. The combined result of wind set-up and the inverse barometer effect is called storm surge.

Wave run-up is the vertical distance between the maximum height a wave runs up the beach and the still water level, composed from tide plus storm surge. It is not independent of wave set-up, but their relative importance varies according to the site. That is, there is a gradual change from spilling, set-up only, to reflecting, run-up only conditions. Most wave run-up equations include wave set-up implicitly.

Additionally, both set-up and run-up vary with surf beat which arises from wave group effects.

Tsunamis are caused by sudden crustal movements of the earth and are commonly, but incorrectly, called "tidal waves". They are very infrequent and unlikely to occur during a storm and so have not been included in this study. Nevertheless, in the context of events having recurrence intervals in the order of 100 years, one should keep this point in mind as such events have been observed in Australia on a number of occasions. For example, the 1960 tsunamis, which were caused by a severe earthquake near Chile, caused water level oscillations at Fort Denison, Sydney at approximately 45 minutes period and maximum crest to trough height of 0.84m. Other more severe tsunamis have been observed in north-western Australia.

Global meteorological and oceanographic changes, such as the El Nino Southern Oscillation phenomenon in the eastern southern Pacific ocean, and continental shelf waves, cause medium term variations in mean sea level. The former phenomenon may persist for a year or more. The causes are not properly understood, but analyses of long term data from Australian tide gauges indicate that annual mean sea level may vary up to 0.1m from the long term trend, whilst mean sea level may vary by more than 0.2m over the time scale of weeks as a result of coastal trapped wave activity.

Many scientists believe that global warming of the Earth's atmosphere will lead to a rise in mean sea level. Predictions of global sea level rise due to the Greenhouse effect vary considerably. It is impossible to state conclusively by how much the sea may rise, and no policy yet exists regarding the appropriate provision which should be made in the design of new coastal developments.

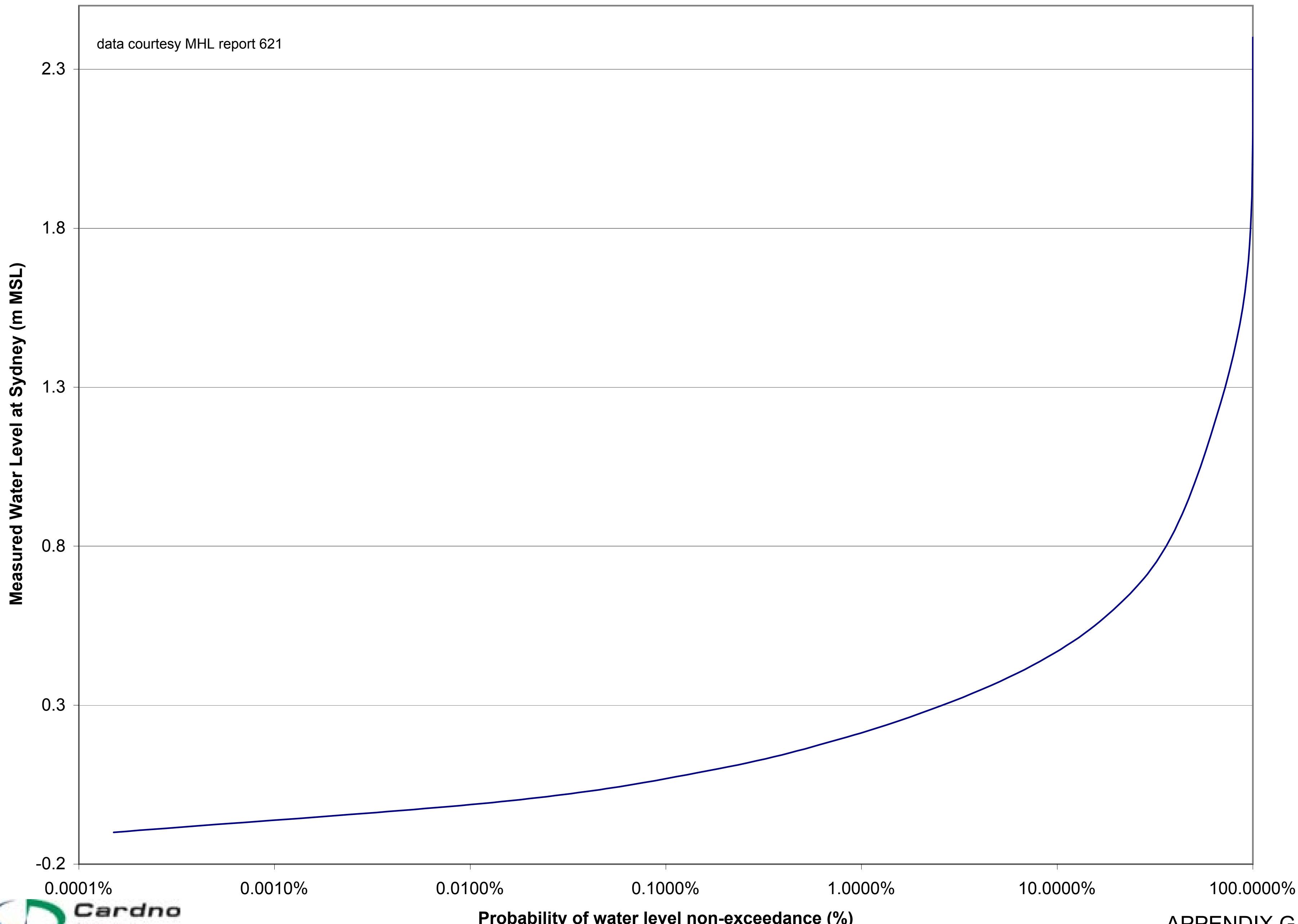
Based on models developed by the American National Academy of Science and the American National Research Council incorporating relevant environmental factors, a guide to future ocean level rises is presented in Table F1. For a fifty years planning period for Elliot Lake-Little Lake a MSL rise of 0.2m has been included for preparation of design flood levels.

Table F1: Predicted Greenhouse Related Mean Sea Level Rises.

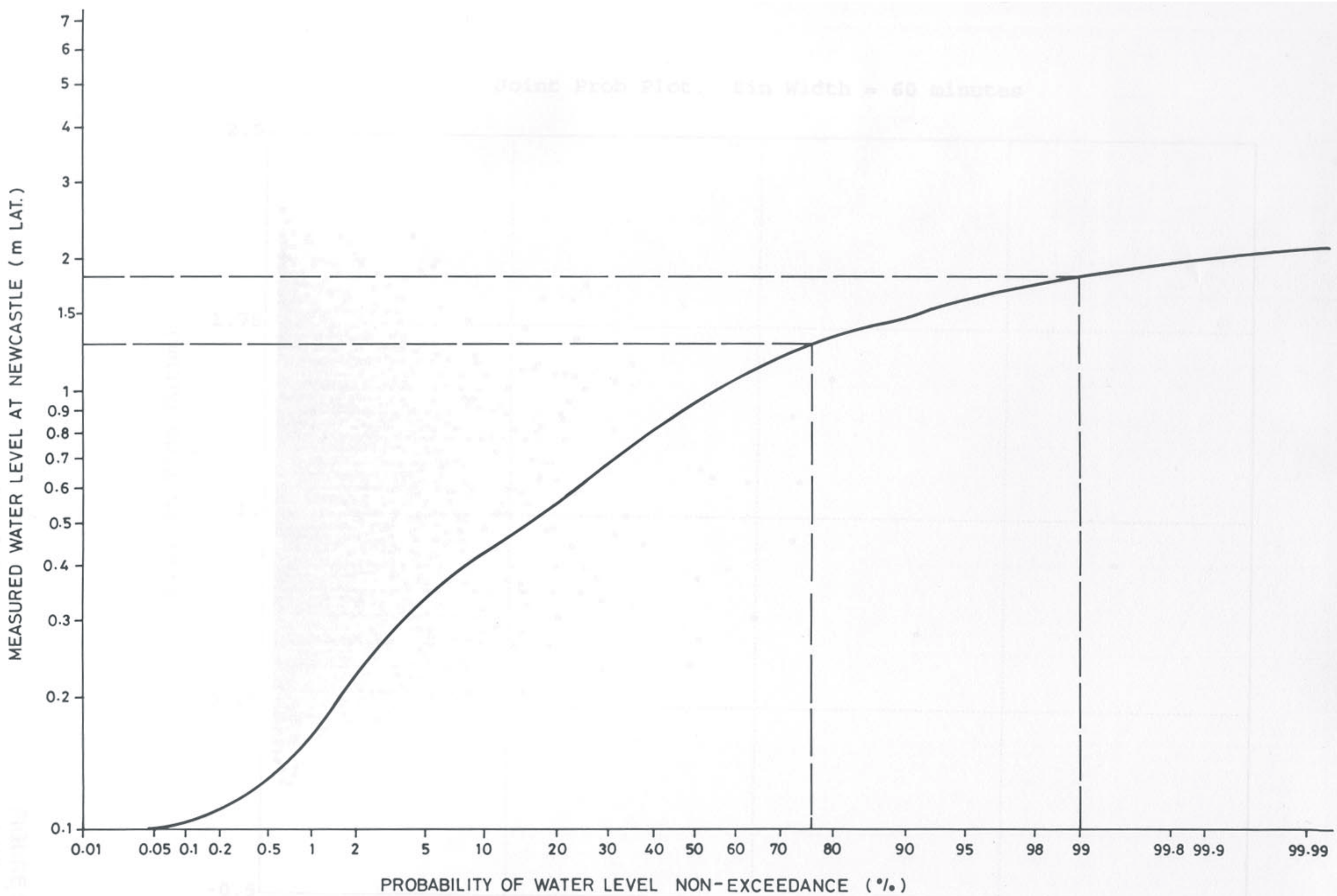
Estimate	Sea Level Rise (m) to Year Shown				
	2000	**2025**	**2050**	**2075**	**2100**
Low	0.02	0.09	0.19	0.32	0.49
Mean	0.03	0.14	0.34	0.62	0.98
High	0.03	0.20	0.49	0.92	1.48

APPENDIX G

RECORDED TIDE LEVELS AT SYDNEY



APPENDIX G
PROBILITY OF WATER LEVEL
NON-EXCEEDANCE



MEASURED WATER LEVEL AT NEWCASTLE (m LAT.)
7
6
5
4
3
2
1.5
1
0.9
0.8
0.7
0.6
0.5
0.4
0.3
0.2
0.1
0.01
0.05
0.1
0.2
0.5
1
2
5
10
20
30
40
50
60
70
80
90
95
98
99
99.8
99.9
99.99
PROBABILITY OF WATER LEVEL NON-EXCEEDANCE (%)

APPENDIX H

WAVE CLIMATE INVESTIGATION

H.1 WAVE CLIMATE INVESTIGATION

In order to describe the propagation of offshore deep water waves to the study area a numerical wave modelling system was applied. This system was based on the RAYTRK reverse ray frequency-direction spectral wave model developed at HRS, UK. This model includes refraction, shoaling, bed friction and directional spread. The model operates on a triangular grid and grid size may vary from offshore to onshore. A grid size of 60m was used in the nearshore area of the entrance. Model grid systems extended offshore to a depth of 200m. Bathymetric data from AUS charts was used. This model system was used to determine wave coefficients for a nearshore location at the -6.5m CD contour immediately east of the entrance. Wave coefficient, K_w, is defined as:-

$$H_i = H_o K_w \qquad \text{(H1)}$$

where H_i is inshore wave height
H_o is offshore wave height.

Wave coefficients were prepared for nine offshore wave directions (north through east to south) and nine wave periods (T_z from 3 to 11). They are generally suitable for non-breaking waves. Where wave breaking occurs a surf zone model must be used together with these wave coefficients to include wave set-up and wave breaking caused changes to the Rayleigh Distribution.

For the calibration event, available offshore Botany Bay wave data (H_s, T_z), provided by SPC was used; 6 hourly data. Wave directions were determined from daily synoptic charts for the period. Wave coefficients were then interpolated from the Table in Appendix I and used to transfer the offshore Botany Bay wave parameters to the nearshore model point (6.5m CD). From there waves were propagated further inshore using a surf zone model (Goda, 2000) previously wave setup in the lake entrance. In this analysis an overall depth of 0.5m at MSL was adopted. Depths were adjusted for tide level.

In addition to wave setup at the shoreline, the inverse barometer effect was estimated from the synoptic charts and assessing an average atmospheric pressure of 1010 hPa. All three water level components were then combined to provide the ocean boundary level at the site.

For design water levels these wave coefficients were used, together with a parametric description of the offshore Botany Bay wave climate, see Appendix I, to determine the peak storm H_s exceeded for no more than 24 hours per year, on average, at this nearshore location to be 4.1m. For the offshore region, the equivalent parameter is 5.2m. Thus a weighted average wave coefficient for this location is 0.79 - (4.1/5.2). This parameter embodies all of the offshore directional and wave height and period occurrence probabilities described by Appendix I. This data was prepared using over twenty years of offshore Botany Bay wave data. Wave directions were determined from synoptic charts for each record before analysis.

APPENDIX I

PARAMETERISED OFFSHORE SYDNEY WAVE CLIMATE AND EXTREME WAVE CLIMATE

APPENDIX I: Parametric Offshore Wave Climate - Botany Bay

PARAMETRIC OFFSHORE WAVE CLIMATE

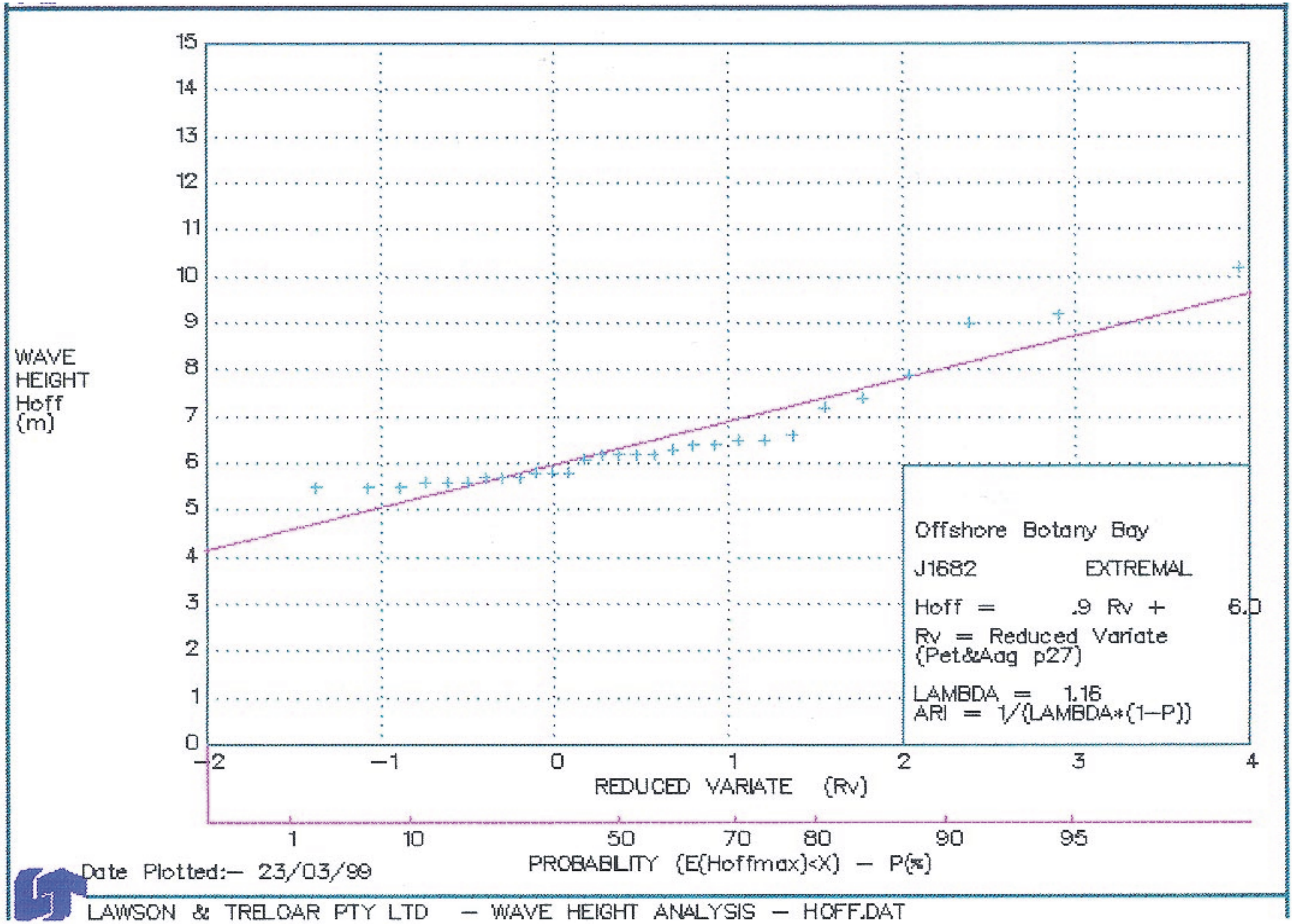
θ	Tz	P1	H_{10}(m)	H_{50}(m)	H_{90}(m)	σ_y
N (348.75-11.25)						
	3.0	0.000	-	-	-	-
	4.0	0.001	1.45	0.92	0.59	0.35
	5.0	0.003	1.77	1.06	0.63	0.40
	6.0	0.001	1.98	1.34	0.91	0.30
	7.0	0.000	-	-	-	-
	8.0	0.000	-	-	-	-
	9.0	0.000	-	-	-	-
	10.0	0.000	-	-	-	-
	11.0	0.000	-	-	-	-
NNE (11.25-33.75)						
	3.0	0.000	-	-	-	-
	4.0	0.003	1.32	0.99	0.75	0.22
	5.0	0.009	1.73	1.28	0.95	0.23
	6.0	0.007	2.30	1.45	0.91	0.36
	7.0	0.003	2.89	1.72	1.02	0.41
	8.0	0.001	3.26	1.87	1.07	0.43
	9.0	0.000	-	-	-	-
	10.0	0.000	-	-	-	-
	11.0	0.000	-	-	-	-
NE (33.75-56.25)						
	3.0	0.000	-	-	-	-
	4.0	0.020	1.29	0.94	0.69	0.24
	5.0	0.048	1.63	1.17	0.84	0.26
	6.0	0.034	2.05	1.36	0.90	0.32
	7.0	0.010	2.18	1.37	0.86	0.36
	8.0	0.003	2.73	1.56	0.89	0.44
	9.0	0.000	-	-	-	-
	10.0	0.000	-	-	-	-
	11.0	0.000	-	-	-	-
ENE (56.25-78.75)						
	3.0	0.000	-	-	-	-
	4.0	0.008	1.35	0.97	0.70	0.26
	5.0	0.027	1.73	1.19	0.84	0.28
	6.0	0.033	2.14	1.44	0.97	0.31
	7.0	0.012	3.07	1.87	1.14	0.39
	8.0	0.007	3.33	2.20	1.46	0.32
	9.0	0.001	4.20	2.53	1.53	0.39
	10.0	0.001	3.96	2.80	1.98	0.27
	11.0	0.000	-	-	-	-
E (78.75-101.25)						
	3.0	0.000	-	-	-	-
	4.0	0.009	1.23	0.92	0.69	0.23
	5.0	0.034	1.70	1.19	0.84	0.28
	6.0	0.040	2.21	1.43	0.92	0.34
	7.0	0.021	2.85	1.73	1.05	0.39
	8.0	0.005	3.77	2.16	1.24	0.43
	9.0	0.001	4.62	2.73	1.61	0.41
	10.0	0.000	-	-	-	-
	11.0	0.000	-	-	-	-

θ	Tz	P1	H_{10}(m)	H_{50}(m)	H_{90}(m)	σ_y
ESE (101.25-123.75)						
	3.0	0.000	-	-	-	-
	4.0	0.004	1.36	1.00	0.73	0.24
	5.0	0.021	1.84	1.22	0.81	0.32
	6.0	0.022	2.55	1.56	0.95	0.39
	7.0	0.014	3.39	1.99	1.17	0.42
	8.0	0.007	4.27	2.36	1.30	0.46
	9.0	0.001	5.65	2.94	1.53	0.51
	10.0	0.000	-	-	-	-
	11.0	0.000	-	-	-	-
SE (123.75-146.25)						
	3.0	0.000	-	-	-	-
	4.0	0.009	1.36	0.91	0.61	0.31
	5.0	0.039	1.84	1.20	0.78	0.33
	6.0	0.053	2.58	1.61	1.00	0.37
	7.0	0.038	3.08	1.97	1.26	0.35
	8.0	0.014	4.06	2.39	1.41	0.41
	9.0	0.004	4.30	2.48	1.43	0.43
	10.0	0.001	5.01	2.80	1.56	0.46
	11.0	0.000	-	-	-	-
SSE (146.25-168.75)						
	3.0	0.000	-	-	-	-
	4.0	0.008	1.18	0.84	0.60	0.26
	5.0	0.033	1.85	1.25	0.85	0.30
	6.0	0.059	2.58	1.68	1.09	0.34
	7.0	0.046	3.20	1.96	1.20	0.38
	8.0	0.018	4.08	2.61	1.67	0.35
	9.0	0.005	4.81	2.83	1.67	0.41
	10.0	0.001	5.37	3.13	1.83	0.42
	11.0	0.000	-	-	-	-
S (168.75-191.25)						
	3.0	0.000				
	4.0	0.014	1.17	0.86	0.63	0.24
	5.0	0.059	1.77	1.18	0.79	0.31
	6.0	0.083	2.47	1.57	1.00	0.35
	7.0	0.063	3.21	2.07	1.33	0.34
	8.0	0.026	3.80	2.43	1.56	0.28
	9.0	0.007	4.14	2.68	1.73	0.34
	10.0	0.001	5.77	2.70	1.26	0.59
	11.0	0.000	-	-	-	-

θ-is offshore dominant wave direction.
Tz-is average zero upcrossing period.
P1-is the probability that a particular offshore direction-wave period (θ -Tz) Combination occurs
H_{10}, H_{50}, H_{90}- significant wave heights exceed 10% 50% and 90% of the time based on a log normal distribution.
σ_y is standard deviation of y : y=lnH.

TABLE 6: Parametric Offshore Wave Climate Description - Botany Bay

APPENDIX I: Peak Storm Extremal Analyses Off Shore Botany Bay



APPENDIX J

SUMMARY OF MODEL RESULTS

PMF					
Cross Section	Chainage	Max WL (mAHD)	Critical Duration Event	Max Velocity (m/sec)	Max Discharge (m^3/sec)
Bensons Creek					
CS3	51	22.59	45min	3.74	100.78
CS4	128	21.01	45min	4.15	116.49
CS5	235	19.39	30min	3.68	115.58
CS5A	275	19.21	30min	3.16	117.36
CS7	355	18.28	30min	3.54	120.12
CS8	480	16.80	30min	4.12	145.40
CS9	644	15.04	30min	2.69	143.07
CS10	842	14.13	45min	2.36	140.70
CS12	880	12.99	45min	3.37	143.05
CS13	995	12.57	45min	2.28	156.73
CS14	1050	12.29	45min	1.94	148.27
Bensons Tributary					
CS50	0	14.23	1hr	2.05	129.63
CS52A	147	11.37	1hr	1.81	17.62
CS53	220	11.37	1hr	2.04	116.35
CS54	297	11.35	1hr	1.51	123.74
CS55	341	11.23	1hr	1.72	123.40
CS58	411	9.37	1hr	5.55	73.44
CS59	484	8.96	1.5hr	4.00	73.41
CS60	736	8.28	1.5hr	5.21	82.80
CS61	850	6.49	30min	5.81	82.85
Bensons - Overland					
CS14A	-	12.07	45min	1.73	147.95
CS15	-	11.77	45min	1.65	163.77
CS16	-	11.69	45min	1.75	109.49
CS17	-	11.32	45min	2.73	109.62
CS18	-	10.86	45min	3.59	109.69
CS19	-	10.14	45min	4.74	109.71
CS19A	-	9.56	45min	5.31	96.35
CS20	-	9.60	45min	0.47	0.08
CS21	-	9.23	45min	5.56	96.36
CS22	-	9.32	45min	2.55	39.60
CS23	-	9.25	45min	1.02	4.09
CS24	-	11.42	45min	2.17	51.81
CS25	-	9.99	45min	4.64	51.86
CS26	-	9.02	45min	4.71	58.16
CS27	-	8.64	45min	6.69	136.00
CS28	-	8.41	45min	6.48	136.04
CS29A	-	8.23	45min	2.20	18.69
CS30	-	8.12	45min	2.26	18.79
CS31	-	8.52	45min	0.07	0.13
CS32	-	8.75	45min	1.09	4.81
CS33	-	8.49	45min	3.92	98.28
CS34	-	8.22	45min	0.54	4.76
CS35	-	8.51	45min	2.94	39.12
CS36	-	7.59	45min	3.95	46.92
CS37	-	6.83	45min	4.34	46.97
CS38	-	8.07	45min	4.01	61.17
CS39	-	7.38	45min	4.63	61.26

PMF					
Cross Section	Chainage	Max WL (mAHD)	Critical Duration Event	Max Velocity (m/sec)	Max Discharge (m^3/sec)
CS41	-	7.01	45min	4.45	60.97
CS42	-	6.43	45min	4.06	60.94
Davidson-50	-	9.16	45min	1.08	3.63
Davidson-250	-	9.22	45min	0.26	3.83
King	-	13.59	1hr	0.58	40.74
King-0	-	13.75	1hr	2.04	47.95
King-100	-	13.57	1hr	1.11	6.98
King-205	-	13.21	1hr	1.51	6.90
King-236	-	13.23	1hr	1.02	6.88
Lake-ent-0	-	13.21	1hr	0.96	6.88
Lake-ent-500	-	9.55	1hr	2.58	7.01
Lake-ent-900	-	9.16	45min	0.35	7.17
Lake-ent-1150	-	8.49	45min	3.64	69.57
Lake-ent-1400	-	7.84	45min	3.26	50.84
Lake-ent-1800	-	5.59	1hr	3.13	50.77
Lewarra-0	-	10.53	1hr	3.20	49.77
Lewarra-62	-	9.66	1hr	4.33	49.75
Lewarra-94	-	9.37	1hr	4.10	49.72
Pleasant-0	-	13.70	1hr	2.60	62.16
Pleasant-95	-	12.72	1hr	4.12	61.78
Plsnt	-	12.65	1hr	0.82	61.73
Oakley Creek					
CS74J	-215	4.79	15min	2.55	67.51
CS74	-152	4.82	15min	3.00	74.44
CS75	-110	4.82	15min	2.36	63.69
CS76	-50	4.80	15min	1.08	52.82
CS77	0	4.75	30min	3.51	51.23
CS78	33	4.93	30min	1.27	38.81
CS80 us	59	4.94	30min	-	-
CS80 ds	72	3.92	2hr	-	-
CS81	111	3.92	2hr	-	-
Oakley - Overland					
CS70	-	5.79	15min	3.64	87.40
CS71	-	4.88	30min	4.46	88.62
CS72	-	4.23	30min	3.29	89.08
CS73	-	4.67	15min	2.37	103.43
CS79	-	3.91	2hr	5.01	94.02
Tongarra Creek					
-100	-100	16.02	1hr	3.14	181.73
-50	-50	16.02	1hr	3.41	183.09
0	0	16.03	1hr	2.85	171.19
50	50	16.03	1hr	1.47	214.38
100	100	16.03	1hr	2.35	218.70
150	150	16.03	1hr	1.67	213.79
200	200	16.03	1hr	0.74	210.51
250	250	16.03	1hr	1.30	210.38
scc-12	297	10.45	1hr	6.02	210.86
scc-11	337	10.74	1hr	2.11	210.86
scc-10	408	10.45	1hr	2.58	210.98
scc-9	480	9.69	1hr	3.29	211.06

PMF					
Cross Section	Chainage	Max WL (mAHD)	Critical Duration Event	Max Velocity (m/sec)	Max Discharge (m^3/sec)
scc-8	568	9.23	1.5hr	1.93	211.13
scc-7	650	9.13	1.5hr	1.69	232.27
scc-6	715	9.13	1.5hr	1.35	232.05
scc-5	846	9.13	1.5hr	0.95	243.41
scc-4	928	9.13	1.5hr	0.83	279.59
scc-3	940	9.13	1.5hr	0.88	279.55
scc-2	963	9.12	1.5hr	1.68	279.49
990	990	8.05	1.5hr	5.57	279.19
1010	1010	8.23	1.5hr	4.02	279.12
1030	1030	8.29	1.5hr	3.15	279.04
1050	1050	8.30	1.5hr	1.22	279.33
1067	1067	8.29	1.5hr	0.99	279.62
1083	1083	8.29	1.5hr	0.91	279.91
1095	1095	8.27	1.5hr	0.98	280.20
1104	1104	8.16	1.5hr	1.61	280.45
1115	1115	7.79	1.5hr	2.77	280.60
1132	1132	7.31	1.5hr	3.70	280.68
1153	1153	7.18	1.5hr	3.52	280.73
1173	1173	7.20	1.5hr	2.95	280.80
1192	1192	7.09	1.5hr	2.87	280.89
1212	1212	7.05	1.5hr	2.56	281.00
1226	1226	7.01	1.5hr	2.35	281.13
1241	1241	6.99	1.5hr	2.13	281.26
1257	1257	7.03	1.5hr	1.65	281.44
1271	1271	7.04	1.5hr	1.38	281.63
1288	1288	7.03	1.5hr	1.31	281.81
1310	1310	7.06	1.5hr	1.05	282.04
1350	1350	6.52	1.5hr	2.81	300.02
1400	1400	4.49	1.5hr	5.07	300.16
1460	1460	4.64	1.5hr	2.01	299.88
1500	1500	4.66	1.5hr	1.61	299.66
1536	1536	4.48	2hr	2.37	299.44
1583	1583	4.27	2hr	3.01	299.00
1640	1640	4.40	2hr	1.34	304.38
1700	1700	4.42	2hr	0.34	301.97
1750	1750	4.42	2hr	0.35	312.61
1800	1800	4.41	2hr	0.40	308.83
1850	1850	4.41	2hr	0.37	306.29
1896C	1896	4.39	2hr	0.18	82.19
1896L	1896	4.41	2hr	0.23	105.35
1896R	1896	4.41	2hr	0.27	113.74
1950C	1950	4.39	2hr	0.28	80.96
1950L	1950	4.40	2hr	0.48	105.54
1950R	1950	4.39	2hr	0.61	121.25
1995C	1995	4.39	2hr	0.49	139.36
1995L	1995	4.40	2hr	0.30	62.90
1995R	1995	4.39	2hr	0.41	102.20
2040C	2040	4.39	2hr	0.65	203.40
2040L	2040	4.40	2hr	0.13	26.86
2040R	2040	4.39	2hr	0.29	73.65
2100C	2100	4.38	2hr	1.08	371.84

APPENDIX J - SUMMARY OF MODEL RESULTS
FULLY DEVELOPED / FINAL CATCHMENT CONDITION

PMF					
Cross Section	Chainage	Max WL (mAHD)	Critical Duration Event	Max Velocity (m/sec)	Max Discharge (m^3/sec)
2100L	2100	4.39	2hr	0.35	62.22
2100R	2100	4.39	2hr	0.38	45.84
CS88	2159	4.32	2hr	0.88	347.93
CS89	2390	4.27	2hr	0.90	350.56
CS90	2463	4.27	2hr	0.65	350.00
CS91	2718	4.25	2hr	0.71	373.03
CS94	2905	4.23	2hr	0.47	366.84
CS95	3071	4.21	2hr	0.59	372.98
CS96	3189	4.19	2hr	0.76	370.87

1% ARI

Cross Section	Chainage	Max WL (mAHD)	Critical Duration Event	Max Velocity (m/sec)	Max Discharge (m^3/sec)
Bensons Creek					
CS3	51	21.98	2hr	3.77	36.31
CS4	128	19.86	2hr	3.77	38.96
CS5	235	18.56	2hr	3.14	39.04
CS5A	275	18.45	1hr	2.60	38.84
CS7	355	17.08	1hr	3.06	38.38
CS8	480	15.44	1hr	2.86	47.89
CS9	644	14.50	1hr	2.54	47.52
CS10	842	13.35	1hr	1.85	45.81
CS12	880	12.05	1hr	2.33	46.47
CS13	995	11.72	1hr	1.66	50.10
CS14	1050	11.54	1hr	1.54	42.26
Bensons Tributary					
CS50	0	13.21	1hr	1.64	40.33
CS52A	147	10.83	2hr	1.82	15.81
CS53	220	10.65	2hr	2.15	37.54
CS54	297	10.47	2hr	1.35	39.32
CS55	341	10.39	2hr	1.48	38.77
CS58	411	8.84	2hr	2.99	25.48
CS59	484	8.40	2hr	3.35	25.55
CS60	736	6.26	2hr	5.04	27.53
CS61	850	6.12	2hr	4.52	27.54
Bensons - Overland					
CS14A	-	11.49	1hr	0.96	42.34
CS15	-	11.00	1hr	1.12	43.99
CS16	-	10.58	1hr	1.53	26.89
CS17	-	10.21	1hr	2.02	26.84
CS18	-	9.63	2hr	2.70	26.65
CS19	-	9.08	2hr	3.23	26.78
CS19A	-	8.76	2hr	2.07	14.44
CS20	-	8.97	30min	0.00	0.00
CS21	-	8.73	2hr	1.81	14.49
CS22	-	8.80	2hr	1.86	15.58
CS23	-	8.79	2hr	0.65	0.57
CS24	-	10.69	1hr	1.49	15.60
CS25	-	9.58	1hr	3.04	15.55
CS26	-	8.67	2hr	2.61	17.47
CS27	-	7.99	2hr	4.20	30.08
CS28	-	7.90	2hr	3.35	30.05
CS29A	-	7.97	2hr	1.15	3.34
CS30	-	7.38	1hr	1.77	3.34
CS31	-	7.78	2hr	0.06	0.00
CS32	-	8.03	2hr	0.40	0.17
CS33	-	7.83	2hr	2.40	24.89
CS34	-	7.50	2hr	0.21	0.89
CS35	-	7.76	2hr	1.23	5.49
CS36	-	7.07	2hr	2.04	7.94
CS37	-	6.26	2hr	2.26	7.98
CS38	-	7.48	2hr	2.52	18.30
CS39	-	6.99	2hr	2.79	18.27

APPENDIX J - SUMMARY OF MODEL RESULTS
FULLY DEVELOPED / FINAL CATCHMENT CONDITION

1% ARI					
Cross Section	**Chainage**	**Max WL (mAHD)**	**Critical Duration Event**	**Max Velocity (m/sec)**	**Max Discharge (m^3/sec)**
CS41	-	6.43	1hr	2.09	8.58
CS42	-	6.14	1hr	0.69	8.52
Davidson-50	-	8.73	2hr	0.90	2.17
Davidson-250	-	8.73	2hr	0.28	0.65
King	-	13.09	1hr	0.30	6.52
King-0	-	13.16	1hr	0.54	6.79
King-100	-	13.14	1hr	0.07	0.01
King-205	-	12.78	1hr	0.34	0.01
King-236	-	12.69	1hr	0.35	0.01
Lake-ent-0	-	12.87	30min	0.00	0.00
Lake-ent-500	-	9.25	30min	0.00	0.00
Lake-ent-900	-	8.73	2hr	0.05	0.08
Lake-ent-1150	-	8.13	2hr	1.90	13.76
Lake-ent-1400	-	7.54	2hr	1.73	10.34
Lake-ent-1800	-	5.02	2hr	1.73	10.39
Lewarra-0	-	10.11	2hr	1.89	13.12
Lewarra-62	-	9.34	2hr	2.72	13.16
Lewarra-94	-	9.04	2hr	2.42	13.19
Pleasant-0	-	12.98	1hr	1.48	15.77
Pleasant-95	-	12.24	1hr	2.45	15.88
Plsnt	-	12.14	1hr	0.42	15.84
Oakley Creek					
CS74J	-215	3.69	2hr	1.63	23.02
CS74	-152	3.68	2hr	1.98	25.77
CS75	-110	3.69	2hr	0.88	23.49
CS76	-50	3.69	2hr	0.39	17.95
CS77	0	3.70	2hr	1.45	16.37
CS78	33	3.75	2hr	0.76	15.19
CS80 us	59	3.75	2hr	-	-
CS80 ds	72	2.45	2hr	-	-
CS81	111	2.45	2hr	-	-
Oakley - Overland					
CS70	-	5.27	1hr	2.09	21.26
CS71	-	3.58	2hr	2.99	20.56
CS72	-	3.41	2hr	1.93	19.42
CS73	-	3.64	2hr	1.11	19.84
CS79	-	2.45	2hr	2.96	19.01
Tongarra Creek					
-100	-100	13.20	2hr	2.64	70.62
-50	-50	13.20	2hr	2.55	70.44
0	0	13.20	2hr	2.22	67.39
50	50	13.20	2hr	0.90	76.43
100	100	13.20	2hr	1.59	68.37
150	150	13.20	2hr	0.93	56.32
200	200	13.20	2hr	0.37	46.91
250	250	13.20	2hr	0.39	38.75
scc-12	297	9.39	2hr	3.04	37.47
scc-11	337	9.35	2hr	0.78	37.47
scc-10	408	9.14	2hr	1.64	37.47
scc-9	480	8.41	2hr	2.36	37.47

APPENDIX J - SUMMARY OF MODEL RESULTS
FULLY DEVELOPED / FINAL CATCHMENT CONDITION

1% ARI					
Cross Section	**Chainage**	**Max WL (mAHD)**	**Critical Duration Event**	**Max Velocity (m/sec)**	**Max Discharge (m^3/sec)**
scc-8	568	8.06	2hr	0.94	37.44
scc-7	650	7.92	2hr	1.48	42.26
scc-6	715	7.84	2hr	1.15	41.59
scc-5	846	7.84	2hr	0.87	40.62
scc-4	928	7.84	2hr	0.82	43.83
scc-3	940	7.83	2hr	0.91	43.53
scc-2	963	7.83	2hr	1.34	42.96
990	990	6.14	2hr	4.87	42.20
1010	1010	6.16	2hr	3.22	42.20
1030	1030	6.25	2hr	1.88	42.19
1050	1050	6.26	2hr	0.39	42.19
1067	1067	6.25	2hr	0.29	42.19
1083	1083	6.25	2hr	0.30	42.19
1095	1095	6.25	2hr	0.35	42.19
1104	1104	6.21	2hr	0.79	42.19
1115	1115	6.09	2hr	1.44	42.19
1132	1132	5.92	2hr	1.78	42.19
1153	1153	5.83	2hr	1.69	42.19
1173	1173	5.73	2hr	1.64	42.19
1192	1192	5.61	3hr	1.73	42.19
1212	1212	5.45	3hr	1.95	42.19
1226	1226	5.38	3hr	1.54	42.19
1241	1241	5.34	3hr	1.34	42.19
1257	1257	5.25	3hr	1.30	42.19
1271	1271	5.23	3hr	0.73	42.19
1288	1288	5.21	3hr	0.74	42.20
1310	1310	5.19	3hr	1.05	42.20
1350	1350	4.75	3hr	2.16	44.47
1400	1400	3.41	3hr	2.61	44.46
1460	1460	3.27	9hr	0.63	44.37
1500	1500	3.27	9hr	0.48	44.26
1536	1536	3.24	9hr	1.07	44.17
1583	1583	3.21	9hr	0.90	44.05
1640	1640	3.22	9hr	0.31	44.92
1700	1700	3.22	9hr	0.08	44.43
1750	1750	3.22	9hr	0.08	45.82
1800	1800	3.22	9hr	0.09	45.03
1850	1850	3.22	9hr	0.08	44.52
1896C	1896	3.22	9hr	0.03	8.47
1896L	1896	3.22	9hr	0.05	16.82
1896R	1896	3.22	9hr	0.12	29.91
1950C	1950	3.22	9hr	0.04	8.38
1950L	1950	3.22	9hr	0.12	16.98
1950R	1950	3.22	9hr	0.27	31.42
1995C	1995	3.22	9hr	0.09	17.83
1995L	1995	3.22	9hr	0.13	15.21
1995R	1995	3.22	9hr	0.21	30.28
2040C	2040	3.22	9hr	0.12	26.09
2040L	2040	3.22	9hr	0.12	15.20
2040R	2040	3.22	9hr	0.20	29.41
2100C	2100	3.22	9hr	0.26	57.38

APPENDIX J - SUMMARY OF MODEL RESULTS
FULLY DEVELOPED / FINAL CATCHMENT CONDITION

1% ARI					
Cross Section	Chainage	Max WL (mAHD)	Critical Duration Event	Max Velocity (m/sec)	Max Discharge (m^3/sec)
2100L	2100	3.22	9hr	0.27	30.11
2100R	2100	3.22	9hr	0.35	28.43
CS88	2159	3.19	36hr	0.43	53.93
CS89	2390	3.18	36hr	0.36	53.86
CS90	2463	3.18	36hr	0.23	52.86
CS91	2718	3.17	36hr	0.63	52.95
CS94	2905	3.16	36hr	0.22	47.58
CS95	3071	3.16	36hr	0.25	47.72
CS96	3189	3.16	36hr	0.36	47.34

APPENDIX J - SUMMARY OF MODEL RESULTS
FULLY DEVELOPED / FINAL CATCHMENT CONDITION

2% ARI					
Cross Section	Chainage	Max WL (mAHD)	Critical Duration Event	Max Velocity (m/sec)	Max Discharge (m^3/sec)
Bensons Creek					
CS3	51	21.93	2hr	3.77	32.38
CS4	128	19.75	2hr	3.77	34.78
CS5	235	18.49	2hr	3.07	34.84
CS5A	275	18.38	1hr	2.60	34.64
CS7	355	16.96	1hr	2.96	34.19
CS8	480	15.36	1hr	2.66	42.18
CS9	644	14.42	1hr	2.53	41.23
CS10	842	13.24	1hr	1.63	40.84
CS12	880	11.96	1hr	2.22	41.16
CS13	995	11.65	1hr	1.55	44.32
CS14	1050	11.48	1hr	1.45	36.57
Bensons Tributary					
CS50	0	13.13	1hr	1.61	35.68
CS52A	147	10.80	2hr	1.82	15.51
CS53	220	10.60	2hr	2.20	33.18
CS54	297	10.41	2hr	1.33	34.76
CS55	341	10.33	2hr	1.53	34.11
CS58	411	8.74	2hr	2.95	23.03
CS59	484	8.25	2hr	3.29	23.05
CS60	736	6.19	2hr	4.88	24.81
CS61	850	6.05	2hr	4.46	24.78
Bensons - Overland					
CS14A	-	11.44	1hr	0.91	36.52
CS15	-	10.96	1hr	1.05	36.14
CS16	-	10.51	2hr	1.44	21.32
CS17	-	10.14	2hr	1.87	21.40
CS18	-	9.55	2hr	2.55	21.56
CS19	-	9.00	2hr	3.01	21.63
CS19A	-	8.69	2hr	1.57	9.33
CS20	-	8.97	30min	0.00	0.00
CS21	-	8.68	2hr	1.28	9.34
CS22	-	8.76	2hr	1.76	13.88
CS23	-	8.76	2hr	0.62	0.46
CS24	-	10.64	2hr	1.42	13.54
CS25	-	9.56	2hr	2.89	13.59
CS26	-	8.66	1hr	2.35	15.18
CS27	-	7.93	2hr	3.88	23.23
CS28	-	7.86	2hr	2.93	23.23
CS29A	-	7.96	2hr	1.08	2.75
CS30	-	7.30	1hr	1.71	2.77
CS31	-	7.74	30min	0.00	0.00
CS32	-	7.96	2hr	0.33	0.09
CS33	-	7.77	1hr	2.21	19.86
CS34	-	7.44	1hr	0.32	0.70
CS35	-	7.71	1hr	0.99	3.37
CS36	-	7.02	1hr	1.77	5.44
CS37	-	6.20	1hr	1.95	5.44
CS38	-	7.43	1hr	2.33	15.47
CS39	-	6.95	1hr	2.59	15.48

APPENDIX J - SUMMARY OF MODEL RESULTS
FULLY DEVELOPED / FINAL CATCHMENT CONDITION

2% ARI					
Cross Section	**Chainage**	**Max WL (mAHD)**	**Critical Duration Event**	**Max Velocity (m/sec)**	**Max Discharge (m^3/sec)**
CS41	-	6.36	1hr	1.80	5.37
CS42	-	6.09	1hr	0.56	5.27
Davidson-50	-	8.70	2hr	0.87	1.59
Davidson-250	-	8.70	2hr	0.40	0.65
King	-	13.04	1hr	0.27	4.71
King-0	-	13.10	1hr	0.43	4.96
King-100	-	13.10	30min	0.00	0.00
King-205	-	12.74	30min	0.00	0.00
King-236	-	12.57	30min	0.00	0.00
Lake-ent-0	-	12.87	30min	0.00	0.00
Lake-ent-500	-	9.25	30min	0.00	0.00
Lake-ent-900	-	8.70	2hr	0.05	0.07
Lake-ent-1150	-	8.11	2hr	1.75	11.29
Lake-ent-1400	-	7.52	2hr	1.61	8.64
Lake-ent-1800	-	4.97	2hr	1.63	8.72
Lewarra-0	-	10.08	2hr	1.77	11.22
Lewarra-62	-	9.31	2hr	2.56	11.25
Lewarra-94	-	9.02	2hr	2.27	11.27
Pleasant-0	-	12.93	1hr	1.37	13.51
Pleasant-95	-	12.21	1hr	2.28	13.36
Plsnt	-	12.10	1hr	0.38	13.34
Oakley Creek					
CS74J	-215	3.54	2hr	1.65	22.17
CS74	-152	3.54	2hr	1.96	24.57
CS75	-110	3.54	2hr	1.26	22.06
CS76	-50	3.53	2hr	0.38	17.36
CS77	0	3.52	2hr	1.43	16.90
CS78	33	3.56	2hr	0.77	15.76
CS80 us	59	3.56	2hr	-	-
CS80 ds	72	2.32	2hr	-	-
CS81	111	2.32	2hr	-	-
Oakley - Overland					
CS70	-	5.23	1hr	1.94	17.57
CS71	-	3.48	2hr	2.79	16.85
CS72	-	3.30	2hr	1.64	12.47
CS73	-	3.50	2hr	0.94	13.72
CS79	-	2.32	2hr	2.54	13.05
Tongarra Creek					
-100	-100	12.82	2hr	2.59	63.03
-50	-50	12.82	2hr	2.58	63.21
0	0	12.82	2hr	2.21	60.53
50	50	12.82	2hr	0.90	68.92
100	100	12.82	2hr	1.51	61.75
150	150	12.82	2hr	0.66	50.72
200	200	12.82	2hr	0.37	42.31
250	250	12.82	2hr	0.41	35.83
scc-12	297	9.36	2hr	2.96	34.87
scc-11	337	9.31	2hr	0.75	34.87
scc-10	408	9.10	2hr	1.60	34.88
scc-9	480	8.37	2hr	2.32	34.87

APPENDIX J - SUMMARY OF MODEL RESULTS
FULLY DEVELOPED / FINAL CATCHMENT CONDITION

2% ARI					
Cross Section	Chainage	Max WL (mAHD)	Critical Duration Event	Max Velocity (m/sec)	Max Discharge (m^3/sec)
scc-8	568	8.00	2hr	0.92	34.87
scc-7	650	7.81	2hr	1.47	39.17
scc-6	715	7.67	2hr	1.11	38.77
scc-5	846	7.65	2hr	0.88	37.99
scc-4	928	7.65	2hr	0.80	40.83
scc-3	940	7.64	2hr	0.90	40.58
scc-2	963	7.64	2hr	1.37	40.11
990	990	6.10	2hr	4.69	39.35
1010	1010	6.12	2hr	3.14	39.35
1030	1030	6.20	2hr	2.04	39.34
1050	1050	6.20	2hr	0.48	39.33
1067	1067	6.20	2hr	0.32	39.32
1083	1083	6.20	2hr	0.28	39.32
1095	1095	6.20	2hr	0.34	39.33
1104	1104	6.16	2hr	0.77	39.33
1115	1115	6.04	2hr	1.41	39.33
1132	1132	5.88	2hr	1.75	39.33
1153	1153	5.79	2hr	1.66	39.33
1173	1173	5.69	2hr	1.62	39.33
1192	1192	5.57	2hr	1.70	39.33
1212	1212	5.41	2hr	1.90	39.34
1226	1226	5.36	2hr	1.48	39.34
1241	1241	5.31	2hr	1.30	39.34
1257	1257	5.22	2hr	1.30	39.34
1271	1271	5.20	2hr	0.70	39.35
1288	1288	5.18	2hr	0.72	39.35
1310	1310	5.15	2hr	1.04	39.36
1350	1350	4.70	3hr	2.14	41.21
1400	1400	3.37	3hr	2.55	41.20
1460	1460	3.15	9hr	0.63	41.12
1500	1500	3.15	9hr	0.48	41.03
1536	1536	3.11	9hr	1.08	40.94
1583	1583	3.08	9hr	0.92	40.83
1640	1640	3.09	9hr	0.31	41.56
1700	1700	3.09	9hr	0.08	41.01
1750	1750	3.09	9hr	0.08	42.08
1800	1800	3.09	9hr	0.09	41.20
1850	1850	3.09	9hr	0.08	40.60
1896C	1896	3.09	9hr	0.02	6.80
1896L	1896	3.09	9hr	0.06	17.12
1896R	1896	3.09	9hr	0.12	28.89
1950C	1950	3.09	9hr	0.03	6.70
1950L	1950	3.09	9hr	0.13	17.25
1950R	1950	3.09	9hr	0.27	30.64
1995C	1995	3.09	9hr	0.08	14.59
1995L	1995	3.09	9hr	0.14	17.14
1995R	1995	3.09	9hr	0.22	30.19
2040C	2040	3.09	9hr	0.10	21.24
2040L	2040	3.09	9hr	0.13	17.05
2040R	2040	3.09	9hr	0.20	29.77
2100C	2100	3.09	9hr	0.27	52.35

APPENDIX J - SUMMARY OF MODEL RESULTS
FULLY DEVELOPED / FINAL CATCHMENT CONDITION

2% ARI					
Cross Section	Chainage	Max WL (mAHD)	Critical Duration Event	Max Velocity (m/sec)	Max Discharge (m^3/sec)
2100L	2100	3.09	9hr	0.28	30.21
2100R	2100	3.09	9hr	0.37	29.28
CS88	2159	3.02	9hr	0.46	46.91
CS89	2390	3.02	9hr	0.39	46.84
CS90	2463	3.02	9hr	0.23	46.02
CS91	2718	3.00	9hr	0.72	46.75
CS94	2905	2.99	9hr	0.25	42.78
CS95	3071	2.99	9hr	0.29	41.60
CS96	3189	2.98	9hr	0.45	41.09

APPENDIX J - SUMMARY OF MODEL RESULTS
FULLY DEVELOPED / FINAL CATCHMENT CONDITION

5% ARI					
Cross Section	**Chainage**	**Max WL (mAHD)**	**Critical Duration Event**	**Max Velocity (m/sec)**	**Max Discharge (m^3/sec)**
Bensons Creek					
CS3	51	21.87	2hr	3.78	28.62
CS4	128	19.62	2hr	3.81	30.67
CS5	235	18.40	2hr	2.96	30.68
CS5A	275	18.30	2hr	2.52	30.46
CS7	355	16.84	1hr	2.85	29.64
CS8	480	15.24	1hr	2.41	34.98
CS9	644	14.23	1hr	2.54	34.78
CS10	842	13.15	1hr	1.58	34.82
CS12	880	11.83	1hr	2.07	34.60
CS13	995	11.54	1hr	1.40	37.18
CS14	1050	11.40	1hr	1.33	29.30
Bensons Tributary					
CS50	0	13.04	2hr	1.57	30.34
CS52A	147	10.76	2hr	1.83	15.16
CS53	220	10.52	2hr	2.07	28.21
CS54	297	10.34	2hr	1.31	29.30
CS55	341	10.26	2hr	1.54	28.92
CS58	411	8.65	2hr	2.88	20.18
CS59	484	8.14	2hr	3.20	20.18
CS60	736	6.10	2hr	4.67	21.50
CS61	850	5.90	2hr	4.49	21.48
Bensons - Overland					
CS14A	-	11.36	1hr	0.83	29.05
CS15	-	10.90	2hr	1.01	27.19
CS16	-	10.46	2hr	1.37	16.62
CS17	-	10.08	2hr	1.72	16.61
CS18	-	9.46	2hr	2.36	16.54
CS19	-	8.91	2hr	2.74	16.46
CS19A	-	8.60	2hr	0.93	4.33
CS20	-	8.97	30min	0.00	0.00
CS21	-	8.60	2hr	0.72	4.31
CS22	-	8.70	2hr	1.55	10.86
CS23	-	8.70	2hr	0.59	0.35
CS24	-	10.56	2hr	1.31	10.59
CS25	-	9.51	2hr	2.61	10.51
CS26	-	8.65	2hr	1.89	11.68
CS27	-	7.84	2hr	3.50	15.02
CS28	-	7.80	2hr	2.30	14.94
CS29A	-	7.93	2hr	1.00	2.11
CS30	-	7.23	1hr	1.60	2.11
CS31	-	7.74	30min	0.00	0.00
CS32	-	7.87	2hr	0.28	0.05
CS33	-	7.69	2hr	1.93	13.70
CS34	-	7.42	1hr	0.44	0.55
CS35	-	7.69	1hr	0.73	1.32
CS36	-	7.00	1hr	1.72	4.86
CS37	-	6.18	1hr	1.85	4.66
CS38	-	7.35	2hr	2.04	11.45
CS39	-	6.90	2hr	2.28	11.47

APPENDIX J - SUMMARY OF MODEL RESULTS
FULLY DEVELOPED / FINAL CATCHMENT CONDITION

5% ARI					
Cross Section	**Chainage**	**Max WL (mAHD)**	**Critical Duration Event**	**Max Velocity (m/sec)**	**Max Discharge (m^3/sec)**
CS41	-	6.28	2hr	1.49	2.74
CS42	-	6.01	2hr	0.81	2.71
Davidson-50	-	8.67	2hr	0.80	1.40
Davidson-250	-	8.67	2hr	0.42	0.62
King	-	12.97	2hr	0.22	2.71
King-0	-	13.02	2hr	0.29	2.93
King-100	-	13.10	30min	0.00	0.00
King-205	-	12.74	30min	0.00	0.00
King-236	-	12.57	30min	0.00	0.00
Lake-ent-0	-	12.87	30min	0.00	0.00
Lake-ent-500	-	9.25	30min	0.00	0.00
Lake-ent-900	-	8.67	2hr	0.05	0.06
Lake-ent-1150	-	8.08	2hr	1.57	8.66
Lake-ent-1400	-	7.49	2hr	1.47	6.61
Lake-ent-1800	-	4.89	2hr	1.49	6.59
Lewarra-0	-	10.04	2hr	1.61	8.88
Lewarra-62	-	9.28	2hr	2.33	8.89
Lewarra-94	-	8.99	2hr	2.07	8.90
Pleasant-0	-	12.86	2hr	1.19	10.34
Pleasant-95	-	12.16	2hr	2.06	10.40
Plsnt	-	12.06	2hr	0.34	10.42
Oakley Creek					
CS74J	-215	3.40	2hr	1.69	20.28
CS74	-152	3.39	2hr	2.08	23.85
CS75	-110	3.39	2hr	1.72	21.31
CS76	-50	3.38	2hr	0.48	16.13
CS77	0	3.37	2hr	1.63	16.67
CS78	33	3.42	2hr	0.88	15.52
CS80 us	59	3.42	2hr	-	-
CS80 ds	72	2.19	2hr	-	-
CS81	111	2.19	2hr	-	-
Oakley - Overland					
CS70	-	5.19	1hr	1.79	14.22
CS71	-	3.37	1hr	2.59	13.47
CS72	-	3.21	2hr	1.35	7.68
CS73	-	3.37	2hr	0.70	7.88
CS79	-	2.19	2hr	2.07	8.04
Tongarra Creek					
-100	-100	12.40	2hr	2.51	55.62
-50	-50	12.39	2hr	2.60	56.07
0	0	12.40	2hr	2.19	53.57
50	50	12.40	2hr	0.91	61.37
100	100	12.40	2hr	1.41	54.80
150	150	12.40	2hr	0.60	44.16
200	200	12.40	2hr	0.42	36.88
250	250	12.40	2hr	0.45	32.44
scc-12	297	9.32	2hr	2.87	31.74
scc-11	337	9.25	2hr	0.72	31.73
scc-10	408	9.05	2hr	1.56	31.72
scc-9	480	8.32	2hr	2.27	31.69

APPENDIX J - SUMMARY OF MODEL RESULTS
FULLY DEVELOPED / FINAL CATCHMENT CONDITION

5% ARI					
Cross Section	Chainage	Max WL (mAHD)	Critical Duration Event	Max Velocity (m/sec)	Max Discharge (m^3/sec)
scc-8	568	7.93	2hr	0.88	31.71
scc-7	650	7.72	2hr	1.47	35.29
scc-6	715	7.48	6hr	1.10	35.07
scc-5	846	7.41	6hr	0.92	34.60
scc-4	928	7.40	6hr	0.79	37.14
scc-3	940	7.40	6hr	0.88	36.89
scc-2	963	7.39	6hr	1.30	36.49
990	990	6.04	6hr	4.45	35.66
1010	1010	6.05	6hr	3.06	35.66
1030	1030	6.13	6hr	2.09	35.66
1050	1050	6.13	6hr	0.58	35.66
1067	1067	6.13	6hr	0.44	35.65
1083	1083	6.13	6hr	0.34	35.65
1095	1095	6.13	6hr	0.32	35.65
1104	1104	6.09	6hr	0.73	35.65
1115	1115	5.98	6hr	1.37	35.65
1132	1132	5.82	6hr	1.70	35.65
1153	1153	5.73	6hr	1.62	35.64
1173	1173	5.63	6hr	1.58	35.64
1192	1192	5.51	6hr	1.65	35.64
1212	1212	5.37	6hr	1.82	35.64
1226	1226	5.32	6hr	1.40	35.64
1241	1241	5.27	6hr	1.25	35.64
1257	1257	5.18	6hr	1.31	35.64
1271	1271	5.16	6hr	0.67	35.64
1288	1288	5.14	6hr	0.69	35.64
1310	1310	5.11	6hr	1.04	35.65
1350	1350	4.64	6hr	2.10	37.29
1400	1400	3.34	6hr	2.46	37.29
1460	1460	3.01	9hr	0.61	37.25
1500	1500	3.01	9hr	0.46	37.20
1536	1536	2.96	9hr	1.09	37.16
1583	1583	2.91	9hr	0.95	37.08
1640	1640	2.92	9hr	0.31	37.76
1700	1700	2.92	9hr	0.07	37.24
1750	1750	2.92	9hr	0.08	38.65
1800	1800	2.92	9hr	0.09	38.80
1850	1850	2.92	9hr	0.08	39.09
1896C	1896	2.92	9hr	0.02	4.54
1896L	1896	2.92	9hr	0.06	16.02
1896R	1896	2.92	9hr	0.10	25.14
1950C	1950	2.92	9hr	0.02	4.48
1950L	1950	2.92	9hr	0.13	16.08
1950R	1950	2.92	9hr	0.25	26.46
1995C	1995	2.92	9hr	0.06	10.11
1995L	1995	2.92	9hr	0.14	15.98
1995R	1995	2.92	9hr	0.20	26.37
2040C	2040	2.92	9hr	0.07	14.82
2040L	2040	2.92	9hr	0.13	15.90
2040R	2040	2.92	9hr	0.19	26.25
2100C	2100	2.92	9hr	0.25	46.75

APPENDIX J - SUMMARY OF MODEL RESULTS
FULLY DEVELOPED / FINAL CATCHMENT CONDITION

5% ARI					
Cross Section	Chainage	Max WL (mAHD)	Critical Duration Event	Max Velocity (m/sec)	Max Discharge (m^3/sec)
2100L	2100	2.92	9hr	0.25	26.69
2100R	2100	2.92	9hr	0.35	26.17
CS88	2159	2.85	9hr	0.48	41.43
CS89	2390	2.84	9hr	0.42	41.18
CS90	2463	2.84	9hr	0.24	40.35
CS91	2718	2.81	9hr	0.73	40.87
CS94	2905	2.80	9hr	0.27	38.44
CS95	3071	2.79	9hr	0.34	37.41
CS96	3189	2.78	9hr	0.53	36.71

APPENDIX J - SUMMARY OF MODEL RESULTS
FULLY DEVELOPED / FINAL CATCHMENT CONDITION

10% ARI					
Cross Section	Chainage	Max WL (mAHD)	Critical Duration Event	Max Velocity (m/sec)	Max Discharge (m^3/sec)
Bensons Creek					
CS3	51	21.78	2hr	3.79	24.14
CS4	128	19.41	2hr	3.76	25.25
CS5	235	18.28	1hr	2.89	25.13
CS5A	275	18.20	1hr	2.58	25.16
CS7	355	16.70	1hr	2.72	24.99
CS8	480	15.13	1hr	2.20	29.41
CS9	644	14.03	1hr	2.48	29.31
CS10	842	13.06	1hr	1.58	29.25
CS12	880	11.71	1hr	1.93	28.90
CS13	995	11.44	1hr	1.25	30.96
CS14	1050	11.33	1hr	1.15	23.17
Bensons Tributary					
CS50	0	12.93	2hr	1.44	25.81
CS52A	147	10.70	2hr	1.84	14.79
CS53	220	10.36	2hr	2.12	23.22
CS54	297	10.25	2hr	1.27	24.24
CS55	341	10.18	2hr	1.49	24.15
CS58	411	8.57	2hr	2.79	17.71
CS59	484	8.04	2hr	3.10	17.69
CS60	736	6.02	2hr	4.48	18.80
CS61	850	5.65	2hr	4.29	18.71
Bensons - Overland					
CS14A	-	11.30	1hr	0.76	23.26
CS15	-	10.81	1hr	0.95	18.51
CS16	-	10.37	1hr	1.27	10.78
CS17	-	9.99	1hr	1.49	10.73
CS18	-	9.35	2hr	2.06	10.73
CS19	-	8.81	2hr	2.36	10.76
CS19A	-	8.49	2hr	0.06	0.12
CS20	-	8.97	30min	0.00	0.00
CS21	-	8.49	2hr	0.03	0.13
CS22	-	8.63	2hr	1.31	7.84
CS23	-	8.63	2hr	0.54	0.23
CS24	-	10.45	2hr	1.16	7.18
CS25	-	9.46	2hr	2.25	7.20
CS26	-	8.61	2hr	1.40	8.11
CS27	-	7.73	2hr	3.10	7.98
CS28	-	7.74	2hr	1.49	7.97
CS29A	-	7.90	2hr	0.92	1.46
CS30	-	7.21	1hr	1.39	1.46
CS31	-	7.74	30min	0.00	0.00
CS32	-	7.75	2hr	0.21	0.03
CS33	-	7.59	2hr	1.57	7.65
CS34	-	7.39	1hr	0.33	0.39
CS35	-	7.67	1hr	0.34	0.25
CS36	-	6.99	1hr	1.66	4.34
CS37	-	6.16	1hr	1.78	4.14
CS38	-	7.22	2hr	1.56	6.21
CS39	-	6.81	1hr	1.76	6.20

APPENDIX J - SUMMARY OF MODEL RESULTS
FULLY DEVELOPED / FINAL CATCHMENT CONDITION

10% ARI					
Cross Section	**Chainage**	**Max WL (mAHD)**	**Critical Duration Event**	**Max Velocity (m/sec)**	**Max Discharge (m^3/sec)**
CS41	-	6.24	2hr	1.38	1.99
CS42	-	5.90	2hr	0.78	1.61
Davidson-50	-	8.63	2hr	0.76	1.28
Davidson-250	-	8.63	2hr	0.28	0.49
King	-	12.89	2hr	0.16	1.21
King-0	-	12.93	2hr	0.18	1.32
King-100	-	13.10	30min	0.00	0.00
King-205	-	12.74	30min	0.00	0.00
King-236	-	12.57	30min	0.00	0.00
Lake-ent-0	-	12.87	30min	0.00	0.00
Lake-ent-500	-	9.25	30min	0.00	0.00
Lake-ent-900	-	8.63	2hr	0.05	0.05
Lake-ent-1150	-	8.04	2hr	1.39	6.08
Lake-ent-1400	-	7.46	2hr	1.33	4.67
Lake-ent-1800	-	4.71	2hr	1.37	4.70
Lewarra-0	-	10.00	2hr	1.41	6.43
Lewarra-62	-	9.25	2hr	2.05	6.43
Lewarra-94	-	8.95	2hr	1.82	6.42
Pleasant-0	-	12.79	2hr	1.04	7.66
Pleasant-95	-	12.11	2hr	1.84	7.75
Plsnt	-	12.01	2hr	0.29	7.77
Oakley Creek					
CS74J	-215	3.23	2hr	1.59	15.82
CS74	-152	3.20	2hr	2.17	19.85
CS75	-110	3.20	2hr	1.74	18.93
CS76	-50	3.21	2hr	0.67	14.75
CS77	0	3.22	2hr	1.71	15.85
CS78	33	3.30	2hr	1.01	14.58
CS80 us	59	3.30	2hr	-	-
CS80 ds	72	2.10	2hr	-	-
CS81	111	2.11	2hr	-	-
Oakley - Overland					
CS70	-	5.14	1hr	1.62	10.56
CS71	-	3.33	1hr	2.32	9.77
CS72	-	3.11	2hr	1.05	3.75
CS73	-	3.23	2hr	0.52	3.80
CS79	-	2.11	2hr	1.40	3.34
Tongarra Creek					
-100	-100	12.26	2hr	2.41	47.15
-50	-50	11.97	2hr	2.51	48.11
0	0	11.98	2hr	2.15	45.95
50	50	11.98	2hr	0.89	53.03
100	100	11.98	2hr	1.40	47.46
150	150	11.99	2hr	0.87	38.18
200	200	11.99	2hr	0.44	32.15
250	250	11.98	2hr	0.56	28.65
scc-12	297	9.26	2hr	2.76	28.27
scc-11	337	9.18	2hr	0.67	28.27
scc-10	408	8.99	2hr	1.51	28.25
scc-9	480	8.26	2hr	2.22	28.23

APPENDIX J - SUMMARY OF MODEL RESULTS
FULLY DEVELOPED / FINAL CATCHMENT CONDITION

10% ARI					
Cross Section	Chainage	Max WL (mAHD)	Critical Duration Event	Max Velocity (m/sec)	Max Discharge (m^3/sec)
scc-8	568	7.87	2hr	0.83	28.24
scc-7	650	7.66	2hr	1.45	31.16
scc-6	715	7.38	2hr	1.10	30.99
scc-5	846	7.18	6hr	0.91	31.10
scc-4	928	7.18	6hr	0.80	33.48
scc-3	940	7.18	6hr	0.87	33.29
scc-2	963	7.16	6hr	1.24	33.03
990	990	5.98	6hr	4.22	32.22
1010	1010	5.99	6hr	2.96	32.22
1030	1030	6.06	6hr	1.94	32.22
1050	1050	6.06	6hr	0.66	32.22
1067	1067	6.06	6hr	0.55	32.22
1083	1083	6.06	6hr	0.43	32.21
1095	1095	6.06	6hr	0.31	32.21
1104	1104	6.03	6hr	0.70	32.21
1115	1115	5.91	6hr	1.33	32.21
1132	1132	5.76	6hr	1.64	32.21
1153	1153	5.67	6hr	1.58	32.21
1173	1173	5.58	6hr	1.54	32.20
1192	1192	5.46	6hr	1.61	32.20
1212	1212	5.33	6hr	1.73	32.20
1226	1226	5.28	6hr	1.33	32.20
1241	1241	5.23	6hr	1.19	32.20
1257	1257	5.15	6hr	1.31	32.20
1271	1271	5.13	6hr	0.64	32.20
1288	1288	5.10	6hr	0.69	32.20
1310	1310	5.07	6hr	1.04	32.21
1350	1350	4.58	6hr	2.09	33.77
1400	1400	3.31	6hr	2.36	33.77
1460	1460	2.89	9hr	0.58	33.75
1500	1500	2.89	9hr	0.44	33.73
1536	1536	2.83	9hr	1.09	33.72
1583	1583	2.77	9hr	0.98	33.67
1640	1640	2.77	9hr	0.29	34.58
1700	1700	2.77	9hr	0.07	34.39
1750	1750	2.77	9hr	0.08	36.28
1800	1800	2.77	9hr	0.09	36.04
1850	1850	2.77	9hr	0.08	36.04
1896C	1896	2.76	9hr	0.01	1.73
1896L	1896	2.77	9hr	0.06	14.81
1896R	1896	2.77	9hr	0.09	22.09
1950C	1950	2.76	9hr	0.01	1.72
1950L	1950	2.77	9hr	0.13	15.02
1950R	1950	2.77	9hr	0.22	23.56
1995C	1995	2.76	9hr	0.02	4.09
1995L	1995	2.77	9hr	0.14	15.15
1995R	1995	2.77	9hr	0.18	23.53
2040C	2040	2.76	9hr	0.03	6.06
2040L	2040	2.77	9hr	0.14	15.27
2040R	2040	2.77	9hr	0.17	23.50
2100C	2100	2.76	9hr	0.23	43.18

APPENDIX J - SUMMARY OF MODEL RESULTS
FULLY DEVELOPED / FINAL CATCHMENT CONDITION

10% ARI					
Cross Section	Chainage	Max WL (mAHD)	Critical Duration Event	Max Velocity (m/sec)	Max Discharge (m^3/sec)
2100L	2100	2.77	9hr	0.23	23.98
2100R	2100	2.77	9hr	0.32	23.47
CS88	2159	2.71	9hr	0.46	38.32
CS89	2390	2.70	9hr	0.40	36.57
CS90	2463	2.69	9hr	0.23	35.36
CS91	2718	2.66	9hr	0.71	35.45
CS94	2905	2.64	9hr	0.27	33.84
CS95	3071	2.64	9hr	0.40	33.19
CS96	3189	2.62	9hr	0.56	32.73

APPENDIX J - SUMMARY OF MODEL RESULTS
FULLY DEVELOPED / FINAL CATCHMENT CONDITION

20% ARI					
Cross Section	**Chainage**	**Max WL (mAHD)**	**Critical Duration Event**	**Max Velocity (m/sec)**	**Max Discharge (m^3/sec)**
Bensons Creek					
CS3	51	21.40	2hr	3.76	21.02
CS4	128	19.33	2hr	3.68	22.55
CS5	235	18.18	2hr	2.88	22.57
CS5A	275	18.10	1hr	2.57	22.31
CS7	355	16.59	1hr	2.61	21.66
CS8	480	15.00	1hr	2.06	25.25
CS9	644	13.91	1hr	2.35	25.05
CS10	842	12.98	2hr	1.57	24.66
CS12	880	11.61	1hr	1.82	24.87
CS13	995	11.36	1hr	1.14	26.51
CS14	1050	11.27	1hr	1.00	18.68
Bensons Tributary					
CS50	0	12.84	2hr	1.40	22.32
CS52A	147	10.65	2hr	1.83	14.39
CS53	220	10.27	2hr	1.94	19.48
CS54	297	10.16	2hr	1.25	20.30
CS55	341	10.10	2hr	1.45	20.07
CS58	411	8.52	2hr	2.71	15.93
CS59	484	7.97	2hr	3.02	15.93
CS60	736	5.97	2hr	4.32	16.89
CS61	850	5.45	2hr	4.10	16.88
Bensons - Overland					
CS14A	-	11.23	1hr	0.70	18.62
CS15	-	10.73	1hr	0.81	11.42
CS16	-	10.28	1hr	1.12	6.40
CS17	-	9.90	2hr	1.24	6.36
CS18	-	9.25	2hr	1.73	6.35
CS19	-	8.71	1hr	1.95	6.41
CS19A	-	8.33	2hr	0.07	0.00
CS20	-	8.97	30min	0.00	0.00
CS21	-	8.36	2hr	0.02	0.00
CS22	-	8.55	2hr	1.09	5.30
CS23	-	8.56	2hr	0.44	0.10
CS24	-	10.37	2hr	1.00	4.57
CS25	-	9.40	1hr	1.89	4.60
CS26	-	8.54	2hr	1.04	5.36
CS27	-	7.65	2hr	2.75	4.31
CS28	-	7.67	2hr	0.96	4.31
CS29A	-	7.86	2hr	0.85	0.91
CS30	-	7.18	1hr	1.06	0.91
CS31	-	7.74	30min	0.00	0.00
CS32	-	7.66	2hr	0.13	0.01
CS33	-	7.53	2hr	1.28	4.27
CS34	-	7.37	1hr	0.10	0.28
CS35	-	7.64	1hr	0.12	0.06
CS36	-	6.97	1hr	1.60	3.81
CS37	-	6.15	1hr	1.69	3.62
CS38	-	7.12	2hr	1.17	3.00
CS39	-	6.73	2hr	1.36	2.99

20% ARI					
Cross Section	Chainage	Max WL (mAHD)	Critical Duration Event	Max Velocity (m/sec)	Max Discharge (m^3/sec)
CS41	-	6.21	2hr	1.27	1.44
CS42	-	5.80	2hr	0.21	1.00
Davidson-50	-	8.59	2hr	0.72	1.10
Davidson-250	-	8.59	2hr	0.38	0.47
King	-	12.80	2hr	0.11	0.38
King-0	-	12.83	2hr	0.15	0.56
King-100	-	13.10	30min	0.00	0.00
King-205	-	12.74	30min	0.00	0.00
King-236	-	12.57	30min	0.00	0.00
Lake-ent-0	-	12.87	30min	0.00	0.00
Lake-ent-500	-	9.25	30min	0.00	0.00
Lake-ent-900	-	8.59	2hr	0.04	0.03
Lake-ent-1150	-	8.00	2hr	1.24	3.98
Lake-ent-1400	-	7.42	2hr	1.24	3.08
Lake-ent-1800	-	4.56	2hr	1.25	3.11
Lewarra-0	-	9.95	2hr	1.18	4.16
Lewarra-62	-	9.21	2hr	1.72	4.16
Lewarra-94	-	8.91	2hr	1.57	4.16
Pleasant-0	-	12.72	2hr	0.88	5.26
Pleasant-95	-	12.06	2hr	1.58	5.26
Plsnt	-	11.96	2hr	0.24	5.25
Oakley Creek					
CS74J	-215	3.12	2hr	1.48	12.37
CS74	-152	3.03	2hr	2.12	17.22
CS75	-110	3.04	2hr	1.65	15.81
CS76	-50	3.03	2hr	0.70	13.24
CS77	0	3.02	2hr	1.70	13.70
CS78	33	3.03	2hr	1.12	13.19
CS80 us	59	3.03	2hr	-	-
CS80 ds	72	2.04	2hr	-	-
CS81	111	2.04	2hr	-	-
Oakley - Overland					
CS70	-	5.09	1hr	1.46	7.68
CS71	-	3.29	1hr	2.05	6.89
CS72	-	3.06	2hr	0.86	2.16
CS73	-	3.16	2hr	0.48	2.65
CS79	-	2.08	2hr	1.17	2.12
Tongarra Creek					
-100	-100	12.19	2hr	2.31	40.93
-50	-50	11.63	2hr	2.48	42.15
0	0	11.63	2hr	2.09	40.34
50	50	11.63	2hr	0.84	47.19
100	100	11.64	2hr	1.48	42.71
150	150	11.64	2hr	0.65	33.49
200	200	11.64	2hr	0.44	28.33
250	250	11.63	2hr	0.63	26.23
scc-12	297	9.22	2hr	2.67	25.69
scc-11	337	9.13	2hr	0.65	25.70
scc-10	408	8.94	2hr	1.47	25.69
scc-9	480	8.21	2hr	2.17	25.68

APPENDIX J - SUMMARY OF MODEL RESULTS
FULLY DEVELOPED / FINAL CATCHMENT CONDITION

20% ARI					
Cross Section	Chainage	Max WL (mAHD)	Critical Duration Event	Max Velocity (m/sec)	Max Discharge (m^3/sec)
scc-8	568	7.83	2hr	0.79	25.71
scc-7	650	7.62	2hr	1.42	28.54
scc-6	715	7.35	2hr	1.09	28.37
scc-5	846	7.02	6hr	0.89	28.59
scc-4	928	7.02	6hr	0.79	30.64
scc-3	940	7.01	6hr	0.87	30.47
scc-2	963	6.98	6hr	1.23	30.26
990	990	5.93	6hr	4.02	29.62
1010	1010	5.94	6hr	2.80	29.62
1030	1030	6.01	6hr	1.79	29.60
1050	1050	6.01	6hr	0.66	29.59
1067	1067	6.01	6hr	0.56	29.58
1083	1083	6.00	6hr	0.52	29.58
1095	1095	6.00	6hr	0.29	29.58
1104	1104	5.97	6hr	0.68	29.59
1115	1115	5.86	6hr	1.30	29.59
1132	1132	5.72	6hr	1.60	29.59
1153	1153	5.63	6hr	1.55	29.59
1173	1173	5.53	6hr	1.51	29.59
1192	1192	5.42	6hr	1.56	29.59
1212	1212	5.30	6hr	1.66	29.59
1226	1226	5.25	6hr	1.27	29.59
1241	1241	5.21	6hr	1.16	29.59
1257	1257	5.12	6hr	1.30	29.58
1271	1271	5.09	6hr	0.64	29.58
1288	1288	5.07	6hr	0.69	29.59
1310	1310	5.03	6hr	1.04	29.59
1350	1350	4.53	6hr	2.09	30.94
1400	1400	3.28	6hr	2.28	30.94
1460	1460	2.81	9hr	0.55	30.94
1500	1500	2.81	9hr	0.41	30.94
1536	1536	2.74	9hr	1.13	30.95
1583	1583	2.65	9hr	1.01	31.03
1640	1640	2.65	9hr	0.28	31.72
1700	1700	2.65	9hr	0.07	31.87
1750	1750	2.65	9hr	0.07	33.27
1800	1800	2.65	9hr	0.08	33.50
1850	1850	2.65	9hr	0.07	33.61
1896C	1896	2.63	9hr	0.00	0.08
1896L	1896	2.65	9hr	0.05	14.09
1896R	1896	2.65	9hr	0.09	19.94
1950C	1950	2.63	9hr	0.00	0.37
1950L	1950	2.65	9hr	0.12	14.24
1950R	1950	2.65	9hr	0.20	20.80
1995C	1995	2.63	9hr	0.01	0.67
1995L	1995	2.65	9hr	0.13	14.28
1995R	1995	2.65	9hr	0.16	20.77
2040C	2040	2.63	9hr	0.01	0.92
2040L	2040	2.65	9hr	0.13	14.32
2040R	2040	2.65	9hr	0.15	20.74
2100C	2100	2.63	9hr	0.22	38.88

APPENDIX J - SUMMARY OF MODEL RESULTS
FULLY DEVELOPED / FINAL CATCHMENT CONDITION

20% ARI					
Cross Section	Chainage	Max WL (mAHD)	Critical Duration Event	Max Velocity (m/sec)	Max Discharge (m^3/sec)
2100L	2100	2.65	9hr	0.21	21.13
2100R	2100	2.65	9hr	0.29	20.74
CS88	2159	2.58	9hr	0.45	34.88
CS89	2390	2.57	9hr	0.40	33.17
CS90	2463	2.57	9hr	0.22	32.17
CS91	2718	2.54	9hr	0.68	32.27
CS94	2905	2.52	9hr	0.27	30.35
CS95	3071	2.51	9hr	0.46	29.59
CS96	3189	2.49	9hr	0.58	29.20

APPENDIX J - SUMMARY OF MODEL RESULTS
FULLY DEVELOPED / FINAL CATCHMENT CONDITION

50% ARI					
Cross Section	**Chainage**	**Max WL (mAHD)**	**Critical Duration Event**	**Max Velocity (m/sec)**	**Max Discharge (m^3/sec)**
Bensons Creek					
CS3	51	21.19	2hr	3.41	15.89
CS4	128	19.17	2hr	3.36	17.07
CS5	235	17.83	2hr	2.79	17.05
CS5A	275	17.46	2hr	2.54	16.64
CS7	355	16.36	2hr	2.35	15.33
CS8	480	14.70	2hr	1.85	17.70
CS9	644	13.65	2hr	2.10	17.86
CS10	842	12.71	2hr	1.55	17.56
CS12	880	11.40	2hr	1.56	16.93
CS13	995	11.18	2hr	0.94	18.00
CS14	1050	11.12	2hr	0.63	10.06
Bensons Tributary					
CS50	0	12.50	2hr	1.36	15.80
CS52A	147	10.54	2hr	1.81	13.03
CS53	220	10.04	2hr	1.71	13.00
CS54	297	9.89	2hr	1.19	13.67
CS55	341	9.81	2hr	1.37	13.60
CS58	411	8.44	2hr	2.57	13.32
CS59	484	7.86	2hr	2.88	13.31
CS60	736	5.88	2hr	4.08	14.14
CS61	850	5.00	2hr	3.82	14.13
Bensons - Overland					
CS14A	-	11.09	2hr	0.55	10.00
CS15	-	10.60	2hr	0.52	3.43
CS16	-	10.15	2hr	0.84	1.98
CS17	-	9.77	2hr	0.87	1.99
CS18	-	9.11	2hr	1.26	1.98
CS19	-	8.56	2hr	1.37	1.96
CS19A	-	8.25	2hr	0.06	0.00
CS20	-	8.97	30min	0.00	0.00
CS21	-	8.27	2hr	0.01	0.00
CS22	-	8.41	2hr	0.79	2.24
CS23	-	8.45	30min	0.00	0.00
CS24	-	10.22	2hr	0.68	1.49
CS25	-	9.29	2hr	1.42	1.48
CS26	-	8.41	2hr	0.76	2.35
CS27	-	7.56	2hr	2.35	1.81
CS28	-	7.59	2hr	0.61	1.81
CS29A	-	7.70	2hr	0.20	0.00
CS30	-	7.14	2hr	0.01	0.01
CS31	-	7.74	30min	0.00	0.00
CS32	-	7.61	30min	0.00	0.00
CS33	-	7.47	2hr	0.96	1.85
CS34	-	7.32	1hr	0.27	0.13
CS35	-	7.56	2hr	0.12	0.03
CS36	-	6.92	2hr	1.42	2.43
CS37	-	6.10	2hr	1.45	2.37
CS38	-	6.93	2hr	0.60	0.29
CS39	-	6.55	2hr	0.76	0.30

APPENDIX J - SUMMARY OF MODEL RESULTS
FULLY DEVELOPED / FINAL CATCHMENT CONDITION

50% ARI					
Cross Section	**Chainage**	**Max WL (mAHD)**	**Critical Duration Event**	**Max Velocity (m/sec)**	**Max Discharge (m^3/sec)**
CS41	-	6.14	2hr	0.95	0.50
CS42	-	5.59	2hr	0.55	0.40
Davidson-50	-	8.30	2hr	0.48	0.17
Davidson-250	-	8.23	2hr	0.23	0.07
King	-	12.64	30min	0.00	0.00
King-0	-	12.41	30min	0.00	0.00
King-100	-	13.10	30min	0.00	0.00
King-205	-	12.74	30min	0.00	0.00
King-236	-	12.57	30min	0.00	0.00
Lake-ent-0	-	12.87	30min	0.00	0.00
Lake-ent-500	-	9.25	30min	0.00	0.00
Lake-ent-900	-	8.31	2hr	0.15	0.00
Lake-ent-1150	-	7.75	2hr	0.36	0.02
Lake-ent-1400	-	7.19	2hr	0.39	0.01
Lake-ent-1800	-	4.31	2hr	0.35	0.01
Lewarra-0	-	9.74	2hr	0.56	0.19
Lewarra-62	-	9.01	2hr	0.91	0.18
Lewarra-94	-	8.73	2hr	0.82	0.19
Pleasant-0	-	12.47	2hr	0.30	0.36
Pleasant-95	-	11.86	2hr	0.73	0.38
Plsnt	-	11.82	2hr	0.06	0.38
Oakley Creek					
CS74J	-215	3.00	2hr	1.18	7.05
CS74	-152	2.58	2hr	1.85	11.77
CS75	-110	2.57	2hr	1.76	11.18
CS76	-50	2.58	2hr	0.69	9.26
CS77	0	2.57	2hr	1.68	11.02
CS78	33	2.59	2hr	1.08	10.80
CS80 us	59	2.59	2hr	-	-
CS80 ds	72	1.93	2hr	-	-
CS81	111	1.93	2hr	-	-
Oakley - Overland					
CS70	-	4.96	1hr	1.05	2.43
CS71	-	3.19	2hr	1.56	2.07
CS72	-	2.97	2hr	0.58	0.67
CS73	-	3.04	2hr	0.22	0.74
CS79	-	2.02	2hr	0.91	0.70
Tongarra Creek					
-100	-100	12.08	2hr	2.08	30.01
-50	-50	10.95	2hr	2.36	31.21
0	0	10.94	2hr	1.97	30.36
50	50	10.95	2hr	0.71	34.72
100	100	10.95	2hr	1.41	31.72
150	150	10.95	2hr	0.94	25.00
200	200	10.95	2hr	0.44	21.42
250	250	10.94	2hr	0.44	20.05
scc-12	297	9.12	2hr	2.46	20.06
scc-11	337	9.00	2hr	0.57	20.06
scc-10	408	8.82	2hr	1.38	20.04
scc-9	480	8.10	2hr	2.05	20.03

APPENDIX J - SUMMARY OF MODEL RESULTS
FULLY DEVELOPED / FINAL CATCHMENT CONDITION

50% ARI					
Cross Section	**Chainage**	**Max WL (mAHD)**	**Critical Duration Event**	**Max Velocity (m/sec)**	**Max Discharge (m^3/sec)**
scc-8	568	7.69	2hr	0.70	20.03
scc-7	650	7.52	2hr	1.28	21.72
scc-6	715	7.28	2hr	1.02	21.73
scc-5	846	6.71	6hr	0.91	22.37
scc-4	928	6.68	6hr	0.80	24.44
scc-3	940	6.68	6hr	0.87	24.37
scc-2	963	6.58	6hr	1.14	24.33
990	990	5.84	6hr	3.58	24.15
1010	1010	5.83	6hr	2.54	24.15
1030	1030	5.87	6hr	1.47	24.13
1050	1050	5.87	6hr	0.61	24.11
1067	1067	5.87	6hr	0.38	24.09
1083	1083	5.87	6hr	0.36	24.07
1095	1095	5.87	6hr	0.26	24.05
1104	1104	5.84	6hr	0.62	24.03
1115	1115	5.74	6hr	1.22	24.04
1132	1132	5.61	6hr	1.49	24.04
1153	1153	5.52	6hr	1.47	24.04
1173	1173	5.42	6hr	1.43	24.04
1192	1192	5.32	6hr	1.46	24.05
1212	1212	5.22	6hr	1.51	24.05
1226	1226	5.18	6hr	1.15	24.05
1241	1241	5.13	6hr	1.15	24.05
1257	1257	5.04	6hr	1.30	24.05
1271	1271	5.00	6hr	0.66	24.05
1288	1288	4.98	6hr	0.69	24.06
1310	1310	4.92	6hr	1.04	24.06
1350	1350	4.42	6hr	2.06	25.19
1400	1400	3.23	6hr	2.11	25.19
1460	1460	2.70	6hr	0.48	25.17
1500	1500	2.69	6hr	0.36	25.14
1536	1536	2.62	9hr	1.16	25.10
1583	1583	2.44	9hr	1.01	25.03
1640	1640	2.45	9hr	0.24	25.34
1700	1700	2.45	9hr	0.06	25.07
1750	1750	2.45	9hr	0.06	25.79
1800	1800	2.45	9hr	0.07	25.80
1850	1850	2.45	9hr	0.06	25.87
1896C	1896	2.37	9hr	0.00	0.10
1896L	1896	2.45	9hr	0.04	11.19
1896R	1896	2.45	9hr	0.07	15.78
1950C	1950	2.37	9hr	0.00	0.36
1950L	1950	2.45	9hr	0.10	11.30
1950R	1950	2.45	9hr	0.16	16.19
1995C	1995	2.37	9hr	0.01	0.59
1995L	1995	2.45	9hr	0.11	11.36
1995R	1995	2.45	9hr	0.13	16.16
2040C	2040	2.37	9hr	0.01	0.81
2040L	2040	2.45	9hr	0.10	11.41
2040R	2040	2.45	9hr	0.12	16.18
2100C	2100	2.37	9hr	0.18	30.62

APPENDIX J - SUMMARY OF MODEL RESULTS
FULLY DEVELOPED / FINAL CATCHMENT CONDITION

50% ARI					
Cross Section	**Chainage**	**Max WL (mAHD)**	**Critical Duration Event**	**Max Velocity (m/sec)**	**Max Discharge (m^3/sec)**
2100L	2100	2.45	9hr	0.17	16.45
2100R	2100	2.44	9hr	0.23	16.22
CS88	2159	2.35	9hr	0.41	28.26
CS89	2390	2.33	9hr	0.38	27.32
CS90	2463	2.33	9hr	0.19	26.63
CS91	2718	2.30	9hr	0.60	26.48
CS94	2905	2.29	9hr	0.26	24.80
CS95	3071	2.27	9hr	0.50	23.13
CS96	3189	2.24	9hr	0.62	22.05

APPENDIX K

COMPARISON OF MODEL RESULTS FOR EXISTING & DEVELOPMENT CATCHMENT CONDITIONS

X Section	Chainage	PMF			1%			2%			5%			10%			20%			50%		
		existing	developed	difference	existing	developed	difference	existing	developed	difference	existing	developed	difference	existing	developed	difference	existing	developed	difference	existing	developed	difference
Bensons Creek																						
CS3	51	22.59	22.59	0.00	21.98	21.98	0.00	21.93	21.93	0.00	21.87	21.87	0.00	21.78	21.78	0.00	21.40	21.40	0.00	21.19	21.19	0.00
CS4	128	21.01	21.01	0.00	19.86	19.86	0.00	19.75	19.75	0.00	19.62	19.62	0.00	19.41	19.41	0.00	19.33	19.33	0.00	19.17	19.17	0.00
CS5	235	19.39	19.39	0.00	18.56	18.56	0.00	18.49	18.49	0.00	18.40	18.40	0.00	18.28	18.28	0.00	18.18	18.18	0.00	17.83	17.83	0.00
CS5A	275	19.21	19.21	0.00	18.45	18.45	0.00	18.38	18.38	0.00	18.30	18.30	0.00	18.20	18.20	0.00	18.10	18.10	0.00	17.46	17.46	0.00
CS7	355	18.28	18.28	0.00	17.08	17.08	0.00	16.96	16.96	0.00	16.84	16.84	0.00	16.70	16.70	0.00	16.58	16.59	0.01	16.36	16.36	0.00
CS8	480	16.80	16.80	0.00	15.44	15.44	0.00	15.36	15.36	0.00	15.24	15.24	0.00	15.13	15.13	0.00	14.97	15.00	0.03	14.70	14.70	0.00
CS9	644	15.04	15.04	0.00	14.50	14.50	0.00	14.42	14.42	0.00	14.23	14.23	0.00	14.03	14.03	0.00	13.88	13.91	0.03	13.65	13.65	0.00
CS10	842	14.13	14.13	0.00	13.35	13.35	0.00	13.24	13.24	0.00	13.15	13.15	0.00	13.06	13.06	0.00	12.98	12.98	0.00	12.71	12.71	0.00
CS12	880	12.99	12.99	0.00	12.05	12.05	0.00	11.96	11.96	0.00	11.83	11.83	0.00	11.71	11.71	0.00	11.61	11.61	0.00	11.40	11.40	0.00
CS13	995	12.57	12.57	0.00	11.72	11.72	0.00	11.65	11.65	0.00	11.54	11.54	0.00	11.44	11.44	0.00	11.35	11.36	0.01	11.18	11.18	0.00
CS14	1050	12.29	12.29	0.00	11.54	11.54	0.00	11.48	11.48	0.00	11.40	11.40	0.00	11.33	11.33	0.00	11.26	11.27	0.01	11.12	11.12	0.00
Bensons Tributary																						
CS50	0	14.23	14.23	0.00	13.21	13.21	0.00	13.13	13.13	0.00	13.04	13.04	0.00	12.93	12.93	0.00	12.84	12.84	0.00	12.50	12.50	0.00
CS52A	147	11.37	11.37	0.00	10.83	10.83	0.00	10.80	10.80	0.00	10.76	10.76	0.00	10.70	10.70	0.00	10.65	10.65	0.00	10.54	10.54	0.00
CS53	220	11.37	11.37	0.00	10.65	10.65	0.00	10.60	10.60	0.00	10.52	10.52	0.00	10.36	10.36	0.00	10.27	10.27	0.00	10.04	10.04	0.00
CS54	297	11.35	11.35	0.00	10.47	10.47	0.00	10.41	10.41	0.00	10.34	10.34	0.00	10.25	10.25	0.00	10.16	10.16	0.00	9.89	9.89	0.00
CS55	341	11.23	11.23	0.00	10.39	10.39	0.00	10.33	10.33	0.00	10.26	10.26	0.00	10.18	10.18	0.00	10.10	10.10	0.00	9.81	9.81	0.00
CS58	411	9.37	9.37	0.00	8.84	8.84	0.00	8.74	8.74	0.00	8.65	8.65	0.00	8.57	8.57	0.00	8.52	8.52	0.00	8.44	8.44	0.00
CS59	484	8.96	8.96	0.00	8.40	8.40	0.00	8.25	8.25	0.00	8.14	8.14	0.00	8.04	8.04	0.00	7.97	7.97	0.00	7.86	7.86	0.00
CS60	736	8.28	8.28	0.00	6.26	6.26	0.00	6.19	6.19	0.00	6.10	6.10	0.00	6.02	6.02	0.00	5.97	5.97	0.00	5.88	5.88	0.00
CS61	850	6.49	6.49	0.00	6.12	6.12	0.00	6.05	6.05	0.00	5.90	5.90	0.00	5.65	5.65	0.00	5.45	5.45	0.00	5.00	5.00	0.00
Bensons - Overland																						
CS14A	-	12.07	12.07	0.00	11.49	11.49	0.00	11.44	11.44	0.00	11.36	11.36	0.00	11.30	11.30	0.00	11.23	11.23	0.00	11.09	11.09	0.00
CS15	-	11.77	11.77	0.00	11.00	11.00	0.00	10.96	10.96	0.00	10.90	10.90	0.00	10.81	10.81	0.00	10.72	10.73	0.01	10.60	10.60	0.00
CS16	-	11.69	11.69	0.00	10.58	10.58	0.00	10.51	10.51	0.00	10.46	10.46	0.00	10.37	10.37	0.00	10.28	10.28	0.00	10.15	10.15	0.00
CS17	-	11.32	11.32	0.00	10.21	10.21	0.00	10.14	10.14	0.00	10.08	10.08	0.00	9.99	9.99	0.00	9.90	9.90	0.00	9.77	9.77	0.00
CS18	-	10.86	10.86	0.00	9.63	9.63	0.00	9.55	9.55	0.00	9.46	9.46	0.00	9.35	9.35	0.00	9.25	9.25	0.00	9.11	9.11	0.00
CS19	-	10.14	10.14	0.00	9.08	9.08	0.00	9.00	9.00	0.00	8.91	8.91	0.00	8.81	8.81	0.00	8.71	8.71	0.00	8.56	8.56	0.00
CS19A	-	9.56	9.56	0.00	8.76	8.76	0.00	8.69	8.69	0.00	8.60	8.60	0.00	8.49	8.49	0.00	8.33	8.33	0.00	8.25	8.25	0.00
CS20	-	9.60	9.60	0.00	8.97	8.97	0.00	8.97	8.97	0.00	8.97	8.97	0.00	8.97	8.97	0.00	8.97	8.97	0.00	8.97	8.97	0.00
CS21	-	9.23	9.23	0.00	8.73	8.73	0.00	8.68	8.68	0.00	8.60	8.60	0.00	8.49	8.49	0.00	8.36	8.36	0.00	8.27	8.27	0.00
CS22	-	9.32	9.32	0.00	8.80	8.80	0.00	8.76	8.76	0.00	8.70	8.70	0.00	8.63	8.63	0.00	8.55	8.55	0.00	8.41	8.41	0.00
CS23	-	9.25	9.25	0.00	8.79	8.79	0.00	8.76	8.76	0.00	8.70	8.70	0.00	8.63	8.63	0.00	8.56	8.56	0.00	8.45	8.45	0.00
CS24	-	11.42	11.42	0.00	10.69	10.69	0.00	10.64	10.64	0.00	10.56	10.56	0.00	10.45	10.45	0.00	10.37	10.37	0.00	10.22	10.22	0.00
CS25	-	9.99	9.99	0.00	9.58	9.58	0.00	9.56	9.56	0.00	9.51	9.51	0.00	9.46	9.46	0.00	9.40	9.40	0.00	9.29	9.29	0.00
CS26	-	9.02	9.02	0.00	8.67	8.67	0.00	8.66	8.66	0.00	8.65	8.65	0.00	8.61	8.61	0.00	8.54	8.54	0.00	8.41	8.41	0.00

X Section	Chainage	PMF			1%			2%			5%			10%			20%			50%		
		existing	developed	difference	existing	developed	difference	existing	developed	difference	existing	developed	difference	existing	developed	difference	existing	developed	difference	existing	developed	difference
CS27	-	8.64	8.64	0.00	7.99	7.99	0.00	7.93	7.93	0.00	7.84	7.84	0.00	7.73	7.73	0.00	7.65	7.65	0.00	7.56	7.56	0.00
CS28	-	8.41	8.41	0.00	7.90	7.90	0.00	7.86	7.86	0.00	7.80	7.80	0.00	7.74	7.74	0.00	7.67	7.67	0.00	7.59	7.59	0.00
CS29A	-	8.23	8.23	0.00	7.97	7.97	0.00	7.96	7.96	0.00	7.93	7.93	0.00	7.90	7.90	0.00	7.86	7.86	0.00	7.70	7.70	0.00
CS30	-	8.12	8.12	0.00	7.37	7.38	0.01	7.30	7.30	0.00	7.23	7.23	0.00	7.21	7.21	0.00	7.18	7.18	0.00	7.14	7.14	0.00
CS31	-	8.52	8.52	0.00	7.78	7.78	0.00	7.74	7.74	0.00	7.74	7.74	0.00	7.74	7.74	0.00	7.74	7.74	0.00	7.74	7.74	0.00
CS32	-	8.75	8.75	0.00	8.03	8.03	0.00	7.96	7.96	0.00	7.87	7.87	0.00	7.75	7.75	0.00	7.66	7.66	0.00	7.61	7.61	0.00
CS33	-	8.49	8.49	0.00	7.83	7.83	0.00	7.77	7.77	0.00	7.69	7.69	0.00	7.59	7.59	0.00	7.53	7.53	0.00	7.47	7.47	0.00
CS34	-	8.22	8.22	0.00	7.50	7.50	0.00	7.44	7.44	0.00	7.42	7.42	0.00	7.39	7.39	0.00	7.37	7.37	0.00	7.32	7.32	0.00
CS35	-	8.51	8.51	0.00	7.76	7.76	0.00	7.71	7.71	0.00	7.69	7.69	0.00	7.67	7.67	0.00	7.64	7.64	0.00	7.56	7.56	0.00
CS36	-	7.59	7.59	0.00	7.07	7.07	0.00	7.02	7.02	0.00	7.00	7.00	0.00	6.99	6.99	0.00	6.97	6.97	0.00	6.92	6.92	0.00
CS37	-	6.83	6.83	0.00	6.26	6.26	0.00	6.20	6.20	0.00	6.18	6.18	0.00	6.16	6.16	0.00	6.14	6.15	0.01	6.10	6.10	0.00
CS38	-	8.07	8.07	0.00	7.48	7.48	0.00	7.43	7.43	0.00	7.35	7.35	0.00	7.22	7.22	0.00	7.12	7.12	0.00	6.93	6.93	0.00
CS39	-	7.38	7.38	0.00	6.99	6.99	0.00	6.95	6.95	0.00	6.90	6.90	0.00	6.81	6.81	0.00	6.73	6.73	0.00	6.55	6.55	0.00
CS41	-	7.01	7.01	0.00	6.43	6.43	0.00	6.36	6.36	0.00	6.28	6.28	0.00	6.24	6.24	0.00	6.21	6.21	0.00	6.14	6.14	0.00
CS42	-	6.43	6.43	0.00	6.14	6.14	0.00	6.09	6.09	0.00	6.01	6.01	0.00	5.90	5.90	0.00	5.79	5.80	0.01	5.59	5.59	0.00
Davidson-50	-	9.16	9.16	0.00	8.73	8.73	0.00	8.70	8.70	0.00	8.67	8.67	0.00	8.63	8.63	0.00	8.59	8.59	0.00	8.30	8.30	0.00
Davidson-250	-	9.22	9.22	0.00	8.73	8.73	0.00	8.70	8.70	0.00	8.67	8.67	0.00	8.63	8.63	0.00	8.59	8.59	0.00	8.23	8.23	0.00
King	-	13.59	13.59	0.00	13.09	13.09	0.00	13.04	13.04	0.00	12.97	12.97	0.00	12.89	12.89	0.00	12.80	12.80	0.00	12.64	12.64	0.00
King-0	-	13.75	13.75	0.00	13.15	13.16	0.01	13.10	13.10	0.00	13.02	13.02	0.00	12.93	12.93	0.00	12.83	12.83	0.00	12.41	12.41	0.00
King-100	-	13.57	13.57	0.00	13.14	13.14	0.00	13.10	13.10	0.00	13.10	13.10	0.00	13.10	13.10	0.00	13.10	13.10	0.00	13.10	13.10	0.00
King-205	-	13.21	13.21	0.00	12.78	12.78	0.00	12.74	12.74	0.00	12.74	12.74	0.00	12.74	12.74	0.00	12.74	12.74	0.00	12.74	12.74	0.00
King-236	-	13.23	13.23	0.00	12.69	12.69	0.00	12.57	12.57	0.00	12.57	12.57	0.00	12.57	12.57	0.00	12.57	12.57	0.00	12.57	12.57	0.00
Lake-ent-0	-	13.21	13.21	0.00	12.87	12.87	0.00	12.87	12.87	0.00	12.87	12.87	0.00	12.87	12.87	0.00	12.87	12.87	0.00	12.87	12.87	0.00
Lake-ent-500	-	9.55	9.55	0.00	9.25	9.25	0.00	9.25	9.25	0.00	9.25	9.25	0.00	9.25	9.25	0.00	9.25	9.25	0.00	9.25	9.25	0.00
Lake-ent-900	-	9.16	9.16	0.00	8.73	8.73	0.00	8.70	8.70	0.00	8.67	8.67	0.00	8.63	8.63	0.00	8.59	8.59	0.00	8.31	8.31	0.00
Lake-ent-1150	-	8.49	8.49	0.00	8.13	8.13	0.00	8.11	8.11	0.00	8.08	8.08	0.00	8.04	8.04	0.00	8.00	8.00	0.00	7.75	7.75	0.00
Lake-ent-1400	-	7.84	7.84	0.00	7.54	7.54	0.00	7.52	7.52	0.00	7.49	7.49	0.00	7.46	7.46	0.00	7.42	7.42	0.00	7.19	7.19	0.00
Lake-ent-1800	-	5.59	5.59	0.00	5.02	5.02	0.00	4.97	4.97	0.00	4.88	4.89	0.01	4.71	4.71	0.00	4.56	4.56	0.00	4.31	4.31	0.00
Lewarra-0	-	10.53	10.53	0.00	10.11	10.11	0.00	10.08	10.08	0.00	10.04	10.04	0.00	10.00	10.00	0.00	9.95	9.95	0.00	9.74	9.74	0.00
Lewarra-62	-	9.66	9.66	0.00	9.34	9.34	0.00	9.31	9.31	0.00	9.28	9.28	0.00	9.25	9.25	0.00	9.21	9.21	0.00	9.01	9.01	0.00
Lewarra-94	-	9.37	9.37	0.00	9.04	9.04	0.00	9.02	9.02	0.00	8.99	8.99	0.00	8.95	8.95	0.00	8.91	8.91	0.00	8.73	8.73	0.00
Pleasant-0	-	13.70	13.70	0.00	12.98	12.98	0.00	12.93	12.93	0.00	12.86	12.86	0.00	12.79	12.79	0.00	12.72	12.72	0.00	12.47	12.47	0.00
Pleasant-95	-	12.72	12.72	0.00	12.24	12.24	0.00	12.21	12.21	0.00	12.16	12.16	0.00	12.11	12.11	0.00	12.06	12.06	0.00	11.86	11.86	0.00
Plsnt	-	12.65	12.65	0.00	12.14	12.14	0.00	12.10	12.10	0.00	12.06	12.06	0.00	12.01	12.01	0.00	11.96	11.96	0.00	11.82	11.82	0.00
Oakley Creek																						
CS74J	-215	4.79	4.79	0.00	3.69	3.69	0.00	3.54	3.54	0.00	3.40	3.40	0.00	3.23	3.23	0.00	3.12	3.12	0.00	3.00	3.00	0.00
CS74	-152	4.82	4.82	0.00	3.68	3.68	0.00	3.54	3.54	0.00	3.39	3.39	0.00	3.20	3.20	0.00	3.03	3.03	0.00	2.58	2.58	0.00

X Section	Chainage	PMF			1%			2%			5%			10%			20%			50%		
		existing	developed	difference	existing	developed	difference	existing	developed	difference	existing	developed	difference	existing	developed	difference	existing	developed	difference	existing	developed	difference
CS75	-110	4.82	4.82	0.00	3.69	3.69	0.00	3.54	3.54	0.00	3.39	3.39	0.00	3.20	3.20	0.00	3.04	3.04	0.00	2.57	2.57	0.00
CS76	-50	4.80	4.80	0.00	3.69	3.69	0.00	3.53	3.53	0.00	3.38	3.38	0.00	3.21	3.21	0.00	3.03	3.03	0.00	2.58	2.58	0.00
CS77	0	4.75	4.75	0.00	3.70	3.70	0.00	3.52	3.52	0.00	3.37	3.37	0.00	3.22	3.22	0.00	3.02	3.02	0.00	2.57	2.57	0.00
CS78	33	4.93	4.93	0.00	3.75	3.75	0.00	3.56	3.56	0.00	3.42	3.42	0.00	3.30	3.30	0.00	3.03	3.03	0.00	2.59	2.59	0.00
CS80 us	59	4.94	4.94	0.00	3.75	3.75	0.00	3.56	3.56	0.00	3.42	3.42	0.00	3.30	3.30	0.00	3.03	3.03	0.00	2.59	2.59	0.00
CS80 ds	72	3.91	3.92	0.01	2.45	2.45	0.00	2.32	2.32	0.00	2.19	2.19	0.00	2.10	2.10	0.00	2.04	2.04	0.00	1.93	1.93	0.00
CS81	111	3.91	3.92	0.01	2.45	2.45	0.00	2.32	2.32	0.00	2.19	2.19	0.00	2.10	2.10	0.00	2.04	2.04	0.00	1.93	1.93	0.00
Oakley - Overland																						
CS70	-	5.79	5.79	0.00	5.27	5.27	0.00	5.23	5.23	0.00	5.19	5.19	0.00	5.14	5.14	0.00	5.08	5.09	0.01	4.96	4.96	0.00
CS71	-	4.88	4.88	0.00	3.58	3.58	0.00	3.48	3.48	0.00	3.37	3.37	0.00	3.33	3.33	0.00	3.29	3.29	0.00	3.19	3.19	0.00
CS72	-	4.23	4.23	0.00	3.41	3.41	0.00	3.30	3.30	0.00	3.21	3.21	0.00	3.11	3.11	0.00	3.06	3.06	0.00	2.97	2.97	0.00
CS73	-	4.67	4.67	0.00	3.64	3.64	0.00	3.50	3.50	0.00	3.37	3.37	0.00	3.23	3.23	0.00	3.16	3.16	0.00	3.04	3.04	0.00
CS79	-	3.91	3.91	0.00	2.45	2.45	0.00	2.32	2.32	0.00	2.19	2.19	0.00	2.11	2.11	0.00	2.08	2.08	0.00	2.02	2.02	0.00
Tongarra Creek																						
-100	-100	16.01	16.02	0.01	13.14	13.20	0.06	12.76	12.82	0.06	12.36	12.40	0.04	12.26	12.26	0.00	12.19	12.19	0.00	12.08	12.08	0.00
-50	-50	16.01	16.02	0.01	13.14	13.20	0.06	12.76	12.82	0.06	12.33	12.39	0.06	11.91	11.97	0.06	11.56	11.63	0.07	10.88	10.95	0.07
0	0	16.02	16.03	0.01	13.14	13.20	0.06	12.76	12.82	0.06	12.33	12.40	0.07	11.92	11.98	0.06	11.56	11.63	0.07	10.87	10.94	0.07
50	50	16.02	16.03	0.01	13.14	13.20	0.06	12.76	12.82	0.06	12.34	12.40	0.06	11.92	11.98	0.06	11.57	11.63	0.06	10.88	10.95	0.07
100	100	16.02	16.03	0.01	13.14	13.20	0.06	12.76	12.82	0.06	12.34	12.40	0.06	11.92	11.98	0.06	11.57	11.64	0.07	10.88	10.95	0.07
150	150	16.02	16.03	0.01	13.14	13.20	0.06	12.76	12.82	0.06	12.34	12.40	0.06	11.92	11.99	0.07	11.57	11.64	0.07	10.88	10.95	0.07
200	200	16.02	16.03	0.01	13.14	13.20	0.06	12.76	12.82	0.06	12.34	12.40	0.06	11.92	11.99	0.07	11.57	11.64	0.07	10.88	10.95	0.07
250	250	16.01	16.03	0.02	13.14	13.20	0.06	12.76	12.82	0.06	12.34	12.40	0.06	11.92	11.98	0.06	11.56	11.63	0.07	10.87	10.94	0.07
scc-12	297	10.44	10.45	0.01	9.39	9.39	0.00	9.35	9.36	0.01	9.31	9.32	0.01	9.25	9.26	0.01	9.22	9.22	0.00	9.11	9.12	0.01
scc-11	337	10.73	10.74	0.01	9.35	9.35	0.00	9.30	9.31	0.01	9.24	9.25	0.01	9.17	9.18	0.01	9.12	9.13	0.01	8.98	9.00	0.02
scc-10	408	10.43	10.45	0.02	9.13	9.14	0.01	9.09	9.10	0.01	9.04	9.05	0.01	8.98	8.99	0.01	8.93	8.94	0.01	8.81	8.82	0.01
scc-9	480	9.68	9.69	0.01	8.40	8.41	0.01	8.36	8.37	0.01	8.31	8.32	0.01	8.25	8.26	0.01	8.21	8.21	0.00	8.09	8.10	0.01
scc-8	568	9.23	9.23	0.00	8.05	8.06	0.01	7.99	8.00	0.01	7.92	7.93	0.01	7.86	7.87	0.01	7.81	7.83	0.02	7.68	7.69	0.01
scc-7	650	9.13	9.13	0.00	7.90	7.92	0.02	7.79	7.81	0.02	7.71	7.72	0.01	7.65	7.66	0.01	7.61	7.62	0.01	7.51	7.52	0.01
scc-6	715	9.13	9.13	0.00	7.82	7.84	0.02	7.64	7.67	0.03	7.47	7.48	0.01	7.37	7.38	0.01	7.34	7.35	0.01	7.27	7.28	0.01
scc-5	846	9.13	9.13	0.00	7.81	7.84	0.03	7.61	7.65	0.04	7.38	7.41	0.03	7.17	7.18	0.01	7.00	7.02	0.02	6.69	6.71	0.02
scc-4	928	9.13	9.13	0.00	7.81	7.84	0.03	7.61	7.65	0.04	7.38	7.40	0.02	7.17	7.18	0.01	6.99	7.02	0.03	6.67	6.68	0.01
scc-3	940	9.13	9.13	0.00	7.80	7.83	0.03	7.61	7.64	0.03	7.38	7.40	0.02	7.16	7.18	0.02	6.99	7.01	0.02	6.66	6.68	0.02
scc-2	963	9.12	9.12	0.00	7.80	7.83	0.03	7.61	7.64	0.03	7.37	7.39	0.02	7.14	7.16	0.02	6.95	6.98	0.03	6.56	6.58	0.02
990	990	8.05	8.05	0.00	6.13	6.14	0.01	6.09	6.10	0.01	6.03	6.04	0.01	5.97	5.98	0.01	5.93	5.93	0.00	5.83	5.84	0.01
1010	1010	8.23	8.23	0.00	6.15	6.16	0.01	6.12	6.12	0.00	6.05	6.05	0.00	5.98	5.99	0.01	5.93	5.94	0.01	5.82	5.83	0.01
1030	1030	8.29	8.29	0.00	6.25	6.25	0.00	6.19	6.20	0.01	6.13	6.13	0.00	6.06	6.06	0.00	6.00	6.01	0.01	5.86	5.87	0.01
1050	1050	8.30	8.30	0.00	6.25	6.26	0.01	6.19	6.20	0.01	6.13	6.13	0.00	6.06	6.06	0.00	6.00	6.01	0.01	5.86	5.87	0.01

X Section	Chainage	PMF			1%			2%			5%			10%			20%			50%		
		existing	developed	difference	existing	developed	difference	existing	developed	difference	existing	developed	difference	existing	developed	difference	existing	developed	difference	existing	developed	difference
1067	1067	8.29	8.29	0.00	6.25	6.25	0.00	6.19	6.20	0.01	6.13	6.13	0.00	6.06	6.06	0.00	6.00	6.01	0.01	5.86	5.87	0.01
1083	1083	8.29	8.29	0.00	6.24	6.25	0.01	6.19	6.20	0.01	6.13	6.13	0.00	6.06	6.06	0.00	6.00	6.00	0.00	5.86	5.87	0.01
1095	1095	8.27	8.27	0.00	6.24	6.25	0.01	6.19	6.20	0.01	6.12	6.13	0.01	6.05	6.06	0.01	5.99	6.00	0.01	5.86	5.87	0.01
1104	1104	8.16	8.16	0.00	6.20	6.21	0.01	6.15	6.16	0.01	6.09	6.09	0.00	6.02	6.03	0.01	5.96	5.97	0.01	5.83	5.84	0.01
1115	1115	7.79	7.79	0.00	6.08	6.09	0.01	6.03	6.04	0.01	5.97	5.98	0.01	5.91	5.91	0.00	5.85	5.86	0.01	5.73	5.74	0.01
1132	1132	7.31	7.31	0.00	5.92	5.92	0.00	5.87	5.88	0.01	5.82	5.82	0.00	5.76	5.76	0.00	5.71	5.72	0.01	5.60	5.61	0.01
1153	1153	7.18	7.18	0.00	5.83	5.83	0.00	5.78	5.79	0.01	5.73	5.73	0.00	5.67	5.67	0.00	5.62	5.63	0.01	5.51	5.52	0.01
1173	1173	7.20	7.20	0.00	5.73	5.73	0.00	5.68	5.69	0.01	5.63	5.63	0.00	5.57	5.58	0.01	5.52	5.53	0.01	5.41	5.42	0.01
1192	1192	7.09	7.09	0.00	5.60	5.61	0.01	5.56	5.57	0.01	5.51	5.51	0.00	5.46	5.46	0.00	5.41	5.42	0.01	5.31	5.32	0.01
1212	1212	7.05	7.05	0.00	5.44	5.45	0.01	5.41	5.41	0.00	5.37	5.37	0.00	5.33	5.33	0.00	5.29	5.30	0.01	5.21	5.22	0.01
1226	1226	7.01	7.01	0.00	5.38	5.38	0.00	5.35	5.36	0.01	5.32	5.32	0.00	5.28	5.28	0.00	5.25	5.25	0.00	5.17	5.18	0.01
1241	1241	6.99	6.99	0.00	5.33	5.34	0.01	5.30	5.31	0.01	5.27	5.27	0.00	5.23	5.23	0.00	5.20	5.21	0.01	5.12	5.13	0.01
1257	1257	7.03	7.03	0.00	5.25	5.25	0.00	5.21	5.22	0.01	5.18	5.18	0.00	5.15	5.15	0.00	5.12	5.12	0.00	5.03	5.04	0.01
1271	1271	7.04	7.04	0.00	5.23	5.23	0.00	5.20	5.20	0.00	5.16	5.16	0.00	5.12	5.13	0.01	5.09	5.09	0.00	4.99	5.00	0.01
1288	1288	7.03	7.03	0.00	5.21	5.21	0.00	5.17	5.18	0.01	5.14	5.14	0.00	5.10	5.10	0.00	5.07	5.07	0.00	4.97	4.98	0.01
1310	1310	7.06	7.06	0.00	5.18	5.19	0.01	5.14	5.15	0.01	5.11	5.11	0.00	5.06	5.07	0.01	5.03	5.03	0.00	4.91	4.92	0.01
1350	1350	6.52	6.52	0.00	4.74	4.75	0.01	4.69	4.70	0.01	4.63	4.64	0.01	4.57	4.58	0.01	4.52	4.53	0.01	4.42	4.42	0.00
1400	1400	4.49	4.49	0.00	3.40	3.41	0.01	3.37	3.37	0.00	3.33	3.34	0.01	3.30	3.31	0.01	3.28	3.28	0.00	3.23	3.23	0.00
1460	1460	4.64	4.64	0.00	3.27	3.27	0.00	3.15	3.15	0.00	3.00	3.01	0.01	2.89	2.89	0.00	2.80	2.81	0.01	2.69	2.70	0.01
1500	1500	4.66	4.66	0.00	3.27	3.27	0.00	3.15	3.15	0.00	3.00	3.01	0.01	2.88	2.89	0.01	2.80	2.81	0.01	2.69	2.69	0.00
1536	1536	4.48	4.48	0.00	3.24	3.24	0.00	3.11	3.11	0.00	2.95	2.96	0.01	2.83	2.83	0.00	2.73	2.74	0.01	2.61	2.62	0.01
1583	1583	4.27	4.27	0.00	3.21	3.21	0.00	3.08	3.08	0.00	2.91	2.91	0.00	2.76	2.77	0.01	2.64	2.65	0.01	2.44	2.44	0.00
1640	1640	4.39	4.40	0.01	3.22	3.22	0.00	3.08	3.09	0.01	2.92	2.92	0.00	2.77	2.77	0.00	2.65	2.65	0.00	2.45	2.45	0.00
1700	1700	4.41	4.42	0.01	3.22	3.22	0.00	3.09	3.09	0.00	2.92	2.92	0.00	2.77	2.77	0.00	2.65	2.65	0.00	2.45	2.45	0.00
1750	1750	4.41	4.42	0.01	3.22	3.22	0.00	3.09	3.09	0.00	2.92	2.92	0.00	2.77	2.77	0.00	2.65	2.65	0.00	2.45	2.45	0.00
1800	1800	4.41	4.41	0.00	3.22	3.22	0.00	3.09	3.09	0.00	2.92	2.92	0.00	2.77	2.77	0.00	2.65	2.65	0.00	2.45	2.45	0.00
1850	1850	4.41	4.41	0.00	3.22	3.22	0.00	3.09	3.09	0.00	2.92	2.92	0.00	2.77	2.77	0.00	2.65	2.65	0.00	2.45	2.45	0.00
1896C	1896	4.39	4.39	0.00	3.21	3.22	0.01	3.08	3.09	0.01	2.91	2.92	0.01	2.76	2.76	0.00	2.62	2.63	0.01	2.36	2.37	0.01
1896L	1896	4.41	4.41	0.00	3.22	3.22	0.00	3.09	3.09	0.00	2.92	2.92	0.00	2.77	2.77	0.00	2.65	2.65	0.00	2.45	2.45	0.00
1896R	1896	4.41	4.41	0.00	3.22	3.22	0.00	3.08	3.09	0.01	2.92	2.92	0.00	2.77	2.77	0.00	2.65	2.65	0.00	2.44	2.45	0.01
1950C	1950	4.39	4.39	0.00	3.21	3.22	0.01	3.08	3.09	0.01	2.91	2.92	0.01	2.76	2.76	0.00	2.62	2.63	0.01	2.36	2.37	0.01
1950L	1950	4.40	4.40	0.00	3.22	3.22	0.00	3.08	3.09	0.01	2.91	2.92	0.01	2.77	2.77	0.00	2.64	2.65	0.01	2.44	2.45	0.01
1950R	1950	4.39	4.39	0.00	3.22	3.22	0.00	3.08	3.09	0.01	2.91	2.92	0.01	2.77	2.77	0.00	2.64	2.65	0.01	2.44	2.45	0.01
1995C	1995	4.38	4.39	0.01	3.21	3.22	0.01	3.08	3.09	0.01	2.91	2.92	0.01	2.76	2.76	0.00	2.62	2.63	0.01	2.36	2.37	0.01
1995L	1995	4.39	4.40	0.01	3.22	3.22	0.00	3.08	3.09	0.01	2.91	2.92	0.01	2.77	2.77	0.00	2.64	2.65	0.01	2.44	2.45	0.01
1995R	1995	4.39	4.39	0.00	3.22	3.22	0.00	3.08	3.09	0.01	2.91	2.92	0.01	2.77	2.77	0.00	2.64	2.65	0.01	2.44	2.45	0.01
2040C	2040	4.38	4.39	0.01	3.21	3.22	0.01	3.08	3.09	0.01	2.91	2.92	0.01	2.76	2.76	0.00	2.62	2.63	0.01	2.36	2.37	0.01
2040L	2040	4.39	4.40	0.01	3.22	3.22	0.00	3.08	3.09	0.01	2.91	2.92	0.01	2.77	2.77	0.00	2.64	2.65	0.01	2.44	2.45	0.01

X Section	Chainage	PMF			1%			2%			5%			10%			20%			50%		
		existing	developed	difference	existing	developed	difference	existing	developed	difference	existing	developed	difference	existing	developed	difference	existing	developed	difference	existing	developed	difference
2040R	2040	4.39	4.39	0.00	3.22	3.22	0.00	3.08	3.09	0.01	2.91	2.92	0.01	2.77	2.77	0.00	2.64	2.65	0.01	2.44	2.45	0.01
2100C	2100	4.38	4.38	0.00	3.21	3.22	0.01	3.08	3.09	0.01	2.91	2.92	0.01	2.76	2.76	0.00	2.62	2.63	0.01	2.36	2.37	0.01
2100L	2100	4.39	4.39	0.00	3.22	3.22	0.00	3.08	3.09	0.01	2.91	2.92	0.01	2.77	2.77	0.00	2.64	2.65	0.01	2.44	2.45	0.01
2100R	2100	4.39	4.39	0.00	3.22	3.22	0.00	3.08	3.09	0.01	2.91	2.92	0.01	2.76	2.77	0.01	2.64	2.65	0.01	2.44	2.44	0.00
CS88	2159	4.32	4.32	0.00	3.19	3.19	0.00	3.02	3.02	0.00	2.84	2.85	0.01	2.70	2.71	0.01	2.58	2.58	0.00	2.34	2.35	0.01
CS89	2390	4.27	4.27	0.00	3.18	3.18	0.00	3.01	3.02	0.01	2.83	2.84	0.01	2.69	2.70	0.01	2.56	2.57	0.01	2.33	2.33	0.00
CS90	2463	4.27	4.27	0.00	3.18	3.18	0.00	3.01	3.02	0.01	2.83	2.84	0.01	2.69	2.69	0.00	2.56	2.57	0.01	2.32	2.33	0.01
CS91	2718	4.24	4.25	0.01	3.17	3.17	0.00	2.99	3.00	0.01	2.81	2.81	0.00	2.66	2.66	0.00	2.53	2.54	0.01	2.29	2.30	0.01
CS94	2905	4.23	4.23	0.00	3.16	3.16	0.00	2.98	2.99	0.01	2.79	2.80	0.01	2.64	2.64	0.00	2.51	2.52	0.01	2.28	2.29	0.01
CS95	3071	4.21	4.21	0.00	3.16	3.16	0.00	2.98	2.99	0.01	2.78	2.79	0.01	2.63	2.64	0.01	2.50	2.51	0.01	2.26	2.27	0.01
CS96	3189	4.18	4.19	0.01	3.15	3.16	0.01	2.97	2.98	0.01	2.77	2.78	0.01	2.61	2.62	0.01	2.48	2.49	0.01	2.23	2.24	0.01

APPENDIX L

SUMMARY OF SENSITIVE ANALYSIS

Appendix L - Summary of Sensitivity Analysis - Change in Water Level for the 1% 2hr AEP Event

Cross Section	Chainage	1%-2hr WL (mAHD)	Flow		D/S Boundary		Roughness		Scour
			+20%	-20%	+20%	-20%	+20%	-20%	
Bensons Creek									
CS3	51	21.98	0.09	-0.11	0.00	0.00	0.08	-0.09	0.00
CS4	128	19.86	0.18	-0.22	0.00	0.00	0.20	-0.23	0.00
CS5	235	18.56	0.12	-0.15	0.00	0.00	0.15	-0.20	0.00
CS5A	275	18.45	0.11	-0.13	0.00	0.00	0.03	-0.03	0.00
CS7	355	17.06	0.18	-0.21	0.00	0.00	0.17	-0.19	0.00
CS8	480	15.43	0.13	-0.15	0.00	0.00	0.17	-0.18	0.00
CS9	644	14.47	0.10	-0.21	0.00	0.00	0.06	-0.08	0.00
CS10	842	13.33	0.12	-0.17	0.00	0.00	0.04	-0.09	0.00
CS12	880	12.04	0.17	-0.18	0.00	0.00	0.18	-0.22	0.00
CS13	995	11.72	0.14	-0.15	0.00	0.00	0.12	-0.16	0.00
CS14	1050	11.53	0.10	-0.11	0.00	0.00	0.09	-0.15	0.00
Bensons Tributary									
CS50	0	13.19	0.12	-0.13	0.00	0.00	0.04	-0.05	0.00
CS52A	147	10.83	0.04	-0.05	0.00	0.00	0.05	-0.10	0.00
CS53	220	10.65	0.07	-0.10	0.00	0.00	0.05	-0.10	0.00
CS54	297	10.47	0.10	-0.11	0.00	0.00	0.04	-0.04	0.00
CS55	341	10.39	0.11	-0.10	0.00	0.00	0.04	-0.03	0.00
CS58	411	8.84	0.09	-0.15	0.00	0.00	0.09	-0.15	0.00
CS59	484	8.40	0.06	-0.22	0.00	0.00	0.03	-0.12	0.00
CS60	736	6.26	0.13	-0.13	0.00	0.00	0.03	-0.06	0.00
CS61	850	6.12	0.09	-0.15	0.00	0.00	0.08	-0.13	0.00
Bensons - Overland									
CS14A	-	11.48	0.09	-0.10	0.00	0.00	0.05	-0.09	0.00
CS15	-	11.00	0.08	-0.08	0.00	0.00	0.04	-0.09	0.00
CS16	-	10.57	0.11	-0.10	0.00	0.00	0.06	-0.08	0.00
CS17	-	10.21	0.13	-0.11	0.00	0.00	0.06	-0.07	0.00
CS18	-	9.63	0.15	-0.14	0.00	0.00	0.08	-0.06	0.00
CS19	-	9.08	0.14	-0.14	0.00	0.00	0.09	-0.08	0.00
CS19A	-	8.76	0.12	-0.13	0.00	0.00	0.09	-0.09	0.00
CS20	-	8.97	0.00	0.00	0.00	0.00	0.00	0.00	0.00
CS21	-	8.73	0.07	-0.10	0.00	0.00	0.07	-0.07	0.00
CS22	-	8.80	0.06	-0.07	0.00	0.00	0.05	-0.05	0.00
CS23	-	8.79	0.05	-0.07	0.00	0.00	0.05	-0.05	0.00
CS24	-	10.68	0.09	-0.10	0.00	0.00	0.03	-0.04	0.00
CS25	-	9.58	0.05	-0.06	0.00	0.00	0.05	-0.05	0.00
CS26	-	8.67	0.03	-0.02	0.00	0.00	0.13	-0.12	0.00
CS27	-	7.99	0.09	-0.12	0.00	0.00	0.07	-0.07	0.00
CS28	-	7.90	0.07	-0.07	0.00	0.00	0.15	-0.12	0.00
CS29A	-	7.97	0.03	-0.03	0.00	0.00	0.02	-0.02	0.00
CS30	-	7.37	0.11	-0.14	0.00	0.00	0.05	-0.07	0.00
CS31	-	7.78	0.12	-0.04	0.00	0.00	0.07	-0.04	0.00
CS32	-	8.03	0.09	-0.13	0.00	0.00	0.04	-0.07	0.00
CS33	-	7.83	0.08	-0.11	0.00	0.00	0.03	-0.04	0.00
CS34	-	7.50	0.10	-0.09	0.00	0.00	0.06	-0.06	0.00
CS35	-	7.76	0.12	-0.08	0.00	0.00	0.07	-0.05	0.00
CS36	-	7.07	0.08	-0.07	0.00	0.00	0.06	-0.06	0.00
CS37	-	6.26	0.10	-0.09	0.00	0.00	0.07	-0.07	0.00
CS38	-	7.48	0.07	-0.10	0.00	0.00	0.03	-0.04	0.00
CS39	-	6.99	0.05	-0.07	0.00	0.00	0.04	-0.05	0.00
CS41	-	6.42	0.09	-0.12	0.00	0.00	0.06	-0.07	0.00
CS42	-	6.14	0.08	-0.11	0.00	0.00	0.05	-0.05	0.00
Davidson-50	-	8.73	0.04	-0.05	0.00	0.00	0.04	-0.04	0.00
Davidson-250	-	8.73	0.04	-0.05	0.00	0.00	0.04	-0.04	0.00
King	-	13.08	0.08	-0.09	0.00	0.00	0.03	-0.03	0.00
King-0	-	13.15	0.09	-0.10	0.00	0.00	0.03	-0.04	0.00
King-100	-	13.13	0.07	-0.03	0.00	0.00	0.03	-0.03	0.00
King-205	-	12.77	0.15	-0.03	0.00	0.00	0.03	-0.03	0.00

Appendix L - Summary of Sensitivity Analysis - Change in Water Level for the 1% 2hr AEP Event

Cross Section	Chainage	1%-2hr WL (mAHD)	Flow		D/S Boundary		Roughness		Scour
			+20%	-20%	+20%	-20%	+20%	-20%	
King-236	-	12.68	0.25	-0.11	0.00	0.00	0.08	-0.09	0.00
Lake-ent-0	-	12.87	0.06	0.00	0.00	0.00	0.00	0.00	0.00
Lake-ent-500	-	9.25	0.05	0.00	0.00	0.00	0.00	0.00	0.00
Lake-ent-900	-	8.73	0.04	-0.05	0.00	0.00	0.04	-0.04	0.00
Lake-ent-1150	-	8.13	0.04	-0.04	0.00	0.00	0.04	-0.04	0.00
Lake-ent-1400	-	7.54	0.03	-0.04	0.00	0.00	0.04	-0.04	0.00
Lake-ent-1800	-	5.02	0.07	-0.10	0.00	0.00	0.02	-0.06	0.00
Lewarra-0	-	10.11	0.06	-0.05	0.00	0.00	0.04	-0.03	0.00
Lewarra-62	-	9.34	0.04	-0.04	0.00	0.00	0.05	-0.04	0.00
Lewarra-94	-	9.04	0.05	-0.04	0.00	0.00	0.03	-0.03	0.00
Pleasant-0	-	12.97	0.08	-0.09	0.00	0.00	0.04	-0.05	0.00
Pleasant-95	-	12.24	0.05	-0.06	0.00	0.00	0.04	-0.05	0.00
Plsnt	-	12.13	0.05	-0.06	0.00	0.00	0.02	-0.02	0.00
Oakley Creek									
CS74J	-215	3.69	0.16	-0.19	0.05	-0.03	0.05	-0.06	0.00
CS74	-152	3.68	0.16	-0.20	0.06	-0.04	0.05	-0.06	0.00
CS75	-110	3.69	0.16	-0.20	0.05	-0.04	0.05	-0.06	0.00
CS76	-50	3.69	0.17	-0.20	0.05	-0.04	0.04	-0.04	0.00
CS77	0	3.70	0.17	-0.21	0.05	-0.03	0.04	-0.03	0.00
CS78	33	3.75	0.17	-0.24	0.04	0.01	0.03	0.02	0.00
CS81	111	2.45	0.10	-0.10	0.20	-0.09	0.03	-0.03	-0.02
Oakley - Overland									
CS70	-	5.25	0.06	-0.07	0.00	0.00	0.03	-0.04	0.00
CS71	-	3.58	0.20	-0.14	0.08	-0.03	0.11	-0.11	0.00
CS72	-	3.41	0.12	-0.14	0.04	-0.02	0.03	-0.02	0.00
CS73	-	3.64	0.15	-0.18	0.05	-0.02	0.05	-0.05	0.00
CS79	-	2.45	0.10	-0.10	0.20	-0.09	0.03	-0.03	-0.02
Tongarra Creek									
-100	-100	13.20	0.65	-0.66	0.00	0.00	0.13	-0.12	0.00
-50	-50	13.20	0.65	-0.66	0.00	0.00	0.12	-0.11	0.00
0	0	13.20	0.65	-0.65	0.00	0.00	0.12	-0.11	0.00
50	50	13.20	0.65	-0.65	0.00	0.00	0.12	-0.11	0.00
100	100	13.20	0.65	-0.65	0.00	0.00	0.13	-0.11	0.00
150	150	13.20	0.65	-0.65	0.00	0.00	0.13	-0.11	0.00
200	200	13.20	0.65	-0.65	0.00	0.00	0.13	-0.11	0.00
250	250	13.20	0.65	-0.65	0.00	0.00	0.13	-0.12	0.00
scc-12	297	9.39	0.05	-0.06	0.00	0.00	0.08	-0.15	0.00
scc-11	337	9.35	0.07	-0.08	0.00	0.00	0.03	-0.03	0.00
scc-10	408	9.14	0.06	-0.07	0.00	0.00	0.03	-0.03	0.00
scc-9	480	8.41	0.07	-0.07	0.00	0.00	0.06	-0.06	0.00
scc-8	568	8.06	0.17	-0.10	0.00	0.00	0.08	-0.06	0.00
scc-7	650	7.92	0.22	-0.17	0.00	0.00	0.11	-0.07	0.00
scc-6	715	7.84	0.27	-0.29	0.00	0.00	0.13	-0.06	0.00
scc-5	846	7.84	0.28	-0.33	0.00	0.00	0.13	-0.05	0.00
scc-4	928	7.84	0.28	-0.33	0.00	0.00	0.13	-0.05	0.00
scc-3	940	7.83	0.28	-0.33	0.00	0.00	0.13	-0.05	0.00
scc-2	963	7.83	0.28	-0.34	0.00	0.00	0.13	-0.05	0.00
990	990	6.14	0.08	-0.08	0.00	0.00	0.12	-0.26	0.00
1010	1010	6.16	0.06	-0.07	0.00	0.00	0.08	-0.30	0.00
1030	1030	6.25	0.09	-0.09	0.00	0.00	0.02	-0.03	0.00
1050	1050	6.26	0.09	-0.09	0.00	0.00	0.02	-0.03	0.00
1067	1067	6.25	0.09	-0.09	0.00	0.00	0.02	-0.03	0.00
1083	1083	6.25	0.09	-0.09	0.00	0.00	0.02	-0.03	0.00
1095	1095	6.25	0.09	-0.09	0.00	0.00	0.02	-0.03	0.00
1104	1104	6.21	0.09	-0.09	0.00	0.00	0.02	-0.04	0.00
1115	1115	6.09	0.08	-0.08	0.00	0.00	0.04	-0.06	0.00
1132	1132	5.92	0.08	-0.08	0.00	0.00	0.06	-0.09	0.00
1153	1153	5.83	0.08	-0.08	0.00	0.00	0.05	-0.09	0.00

Appendix L - Summary of Sensitivity Analysis - Change in Water Level for the 1% 2hr AEP Event

Cross Section	Chainage	1%-2hr WL (mAHD)	Flow		D/S Boundary		Roughness		Scour
			+20%	-20%	+20%	-20%	+20%	-20%	
1173	1173	5.73	0.08	-0.08	0.00	0.00	0.05	-0.08	0.00
1192	1192	5.61	0.08	-0.07	0.00	0.00	0.05	-0.08	0.00
1212	1212	5.45	0.06	-0.06	0.00	0.00	0.06	-0.09	0.00
1226	1226	5.38	0.05	-0.05	0.00	0.00	0.05	-0.08	0.00
1241	1241	5.34	0.05	-0.05	0.00	0.00	0.03	-0.05	0.00
1257	1257	5.25	0.06	-0.05	0.00	0.00	0.02	-0.04	0.00
1271	1271	5.23	0.06	-0.05	0.00	0.00	0.01	-0.02	0.00
1288	1288	5.21	0.06	-0.05	0.00	0.00	0.01	-0.02	0.00
1310	1310	5.18	0.06	-0.06	0.00	0.00	0.00	-0.01	0.00
1350	1350	4.74	0.08	-0.08	0.00	0.00	0.00	-0.01	0.00
1400	1400	3.40	0.05	-0.05	0.00	0.00	0.04	-0.05	0.00
1500	1500	3.16	0.11	-0.13	0.05	-0.03	0.00	-0.01	0.00
1536	1536	3.10	0.11	-0.14	0.05	-0.03	0.01	-0.02	0.00
1583	1583	3.06	0.11	-0.15	0.06	-0.04	-0.01	0.00	0.00
1640	1640	3.07	0.11	-0.15	0.06	-0.04	-0.01	0.01	0.00
1700	1700	3.07	0.11	-0.15	0.06	-0.04	-0.01	0.01	0.00
1750	1750	3.07	0.11	-0.15	0.06	-0.04	-0.01	0.01	0.00
1800	1800	3.07	0.11	-0.15	0.06	-0.04	-0.01	0.01	0.00
1850	1850	3.07	0.11	-0.15	0.06	-0.04	-0.01	0.01	0.00
1896C	1896	3.07	0.11	-0.15	0.06	-0.04	-0.01	0.01	0.00
1896L	1896	3.07	0.11	-0.15	0.06	-0.04	-0.01	0.01	0.00
1896R	1896	3.07	0.11	-0.15	0.06	-0.04	-0.01	0.01	0.00
1950C	1950	3.07	0.11	-0.15	0.06	-0.04	-0.01	0.01	0.00
1950L	1950	3.07	0.11	-0.15	0.06	-0.04	-0.01	0.01	0.00
1950R	1950	3.07	0.11	-0.15	0.06	-0.04	-0.01	0.01	0.00
1995C	1995	3.07	0.11	-0.15	0.06	-0.04	-0.01	0.01	0.00
1995L	1995	3.07	0.11	-0.15	0.06	-0.04	-0.01	0.01	0.00
1995R	1995	3.07	0.11	-0.15	0.06	-0.04	-0.01	0.01	0.00
2040C	2040	3.07	0.11	-0.15	0.06	-0.04	-0.01	0.01	0.00
2040L	2040	3.07	0.11	-0.15	0.06	-0.04	-0.01	0.01	0.00
2040R	2040	3.07	0.11	-0.15	0.06	-0.04	-0.01	0.01	0.00
2100C	2100	3.07	0.11	-0.15	0.06	-0.04	-0.01	0.01	0.00
2100L	2100	3.07	0.11	-0.15	0.06	-0.04	-0.01	0.01	0.00
2100R	2100	3.07	0.11	-0.15	0.06	-0.04	-0.01	0.01	0.00
CS88	2159	2.99	0.14	-0.15	0.09	-0.05	-0.01	0.01	-0.01
CS89	2390	2.98	0.15	-0.15	0.09	-0.05	-0.02	0.01	-0.01
CS90	2463	2.98	0.15	-0.15	0.09	-0.05	-0.02	0.01	-0.01
CS91	2718	2.97	0.15	-0.15	0.10	-0.05	-0.02	0.01	-0.01
CS94	2905	2.96	0.15	-0.16	0.10	-0.06	-0.03	0.02	-0.01
CS95	3071	2.95	0.15	-0.16	0.10	-0.06	-0.03	0.02	-0.01
CS96	3189	2.95	0.15	-0.16	0.10	-0.06	-0.03	0.02	-0.01

Appendix L - Summary of Sensitivity Analysis - Change in Water Level for the 50% 2hr AEP Event

Cross Section	Chainage	1%-2hr WL (mAHD)	Flow		D/S Boundary		Roughness		Scour
			+20%	-20%	+20%	-20%	+20%	-20%	
Bensons Creek									
CS3	51	21.19	0.13	-0.14	0.00	0.00	0.12	-0.12	0.00
CS4	128	19.17	0.11	-0.12	0.00	0.00	0.16	-0.17	0.00
CS5	235	17.83	0.20	-0.15	0.00	0.00	0.15	-0.14	0.00
CS5A	275	17.46	0.43	-0.48	0.00	0.00	0.31	-0.29	0.00
CS7	355	16.36	0.10	-0.13	0.00	0.00	0.11	-0.14	0.00
CS8	480	14.70	0.15	-0.15	0.00	0.00	0.14	-0.18	0.00
CS9	644	13.65	0.14	-0.13	0.00	0.00	0.11	-0.13	0.00
CS10	842	12.71	0.18	-0.30	0.00	0.00	0.11	-0.14	0.00
CS12	880	11.40	0.10	-0.10	0.00	0.00	0.10	-0.13	0.00
CS13	995	11.18	0.09	-0.09	0.00	0.00	0.07	-0.11	0.00
CS14	1050	11.12	0.08	-0.08	0.00	0.00	0.05	-0.09	0.00
Bensons Tributary									
CS50	0	12.50	0.19	-0.43	0.00	0.00	0.15	-0.31	0.00
CS52A	147	10.54	0.06	-0.11	0.00	0.00	0.06	-0.11	0.00
CS53	220	10.04	0.13	-0.15	0.00	0.00	0.11	-0.13	0.00
CS54	297	9.89	0.17	-0.23	0.00	0.00	0.10	-0.10	0.00
CS55	341	9.81	0.18	-0.27	0.00	0.00	0.11	-0.10	0.00
CS58	411	8.44	0.04	-0.06	0.00	0.00	0.04	-0.05	0.00
CS59	484	7.86	0.05	-0.08	0.00	0.00	0.02	-0.03	0.00
CS60	736	5.88	0.04	-0.06	0.00	0.00	0.02	-0.04	0.00
CS61	850	5.00	0.16	-0.29	0.00	0.00	0.16	-0.41	0.00
Bensons - Overland									
CS14A	-	11.09	0.08	-0.07	0.00	0.00	0.05	-0.09	0.00
CS15	-	10.60	0.06	-0.03	0.00	0.00	0.05	-0.05	0.00
CS16	-	10.15	0.07	-0.04	0.00	0.00	0.05	-0.06	0.00
CS17	-	9.77	0.07	-0.04	0.00	0.00	0.05	-0.04	0.00
CS18	-	9.11	0.08	-0.05	0.00	0.00	0.06	-0.05	0.00
CS19	-	8.56	0.08	-0.04	0.00	0.00	0.06	-0.05	0.00
CS19A	-	8.25	0.05	-0.02	0.00	0.00	0.03	0.00	0.00
CS20	-	8.97	0.00	0.00	0.00	0.00	0.00	0.00	0.00
CS21	-	8.27	0.05	-0.02	0.00	0.00	0.03	0.00	0.00
CS22	-	8.41	0.07	-0.03	0.00	0.00	0.04	-0.01	0.00
CS23	-	8.45	0.04	0.00	0.00	0.00	0.01	0.00	0.00
CS24	-	10.22	0.08	-0.05	0.00	0.00	0.05	-0.05	0.00
CS25	-	9.29	0.06	-0.04	0.00	0.00	0.05	-0.04	0.00
CS26	-	8.41	0.07	-0.03	0.00	0.00	0.04	-0.01	0.00
CS27	-	7.56	0.05	-0.01	0.00	0.00	0.04	-0.01	0.00
CS28	-	7.59	0.07	-0.02	0.00	0.00	0.03	-0.01	0.00
CS29A	-	7.70	0.10	-0.02	0.00	0.00	0.05	-0.02	0.00
CS30	-	7.14	0.02	-0.02	0.00	0.00	0.01	-0.01	0.00
CS31	-	7.74	0.09	0.00	0.00	0.00	0.00	0.00	0.00
CS32	-	7.61	0.18	0.10	0.00	0.00	0.00	0.00	0.00
CS33	-	7.47	0.05	-0.01	0.00	0.00	0.02	0.00	0.00
CS34	-	7.32	0.02	-0.03	0.00	0.00	0.01	-0.01	0.00
CS35	-	7.56	0.27	0.18	0.00	0.00	0.01	-0.01	0.00
CS36	-	6.92	0.92	0.83	0.00	0.00	0.02	-0.02	0.00
CS37	-	6.10	1.73	1.64	0.00	0.00	0.02	-0.02	0.00
CS38	-	6.93	0.47	-0.14	0.00	0.00	0.10	-0.10	0.00
CS39	-	6.55	0.85	-0.13	0.00	0.00	0.09	-0.11	0.00
CS41	-	6.14	0.04	-0.13	0.00	0.00	0.04	-0.12	0.00
CS42	-	5.59	0.43	-0.36	0.00	0.00	0.07	-0.26	0.00
Davidson-50	-	8.30	0.21	-0.18	0.00	0.00	0.11	-0.11	0.00
Davidson-250	-	8.23	0.28	-0.45	0.00	0.00	0.18	-0.27	0.00
King	-	12.64	0.01	0.00	0.00	0.00	0.00	0.00	0.00
King-0	-	12.41	0.26	0.00	0.00	0.00	0.19	0.00	0.00
King-100	-	13.10	0.00	0.00	0.00	0.00	0.00	0.00	0.00
King-205	-	12.74	0.00	0.00	0.00	0.00	0.00	0.00	0.00

Appendix L - Summary of Sensitivity Analysis - Change in Water Level for the 50% 2hr AEP Event

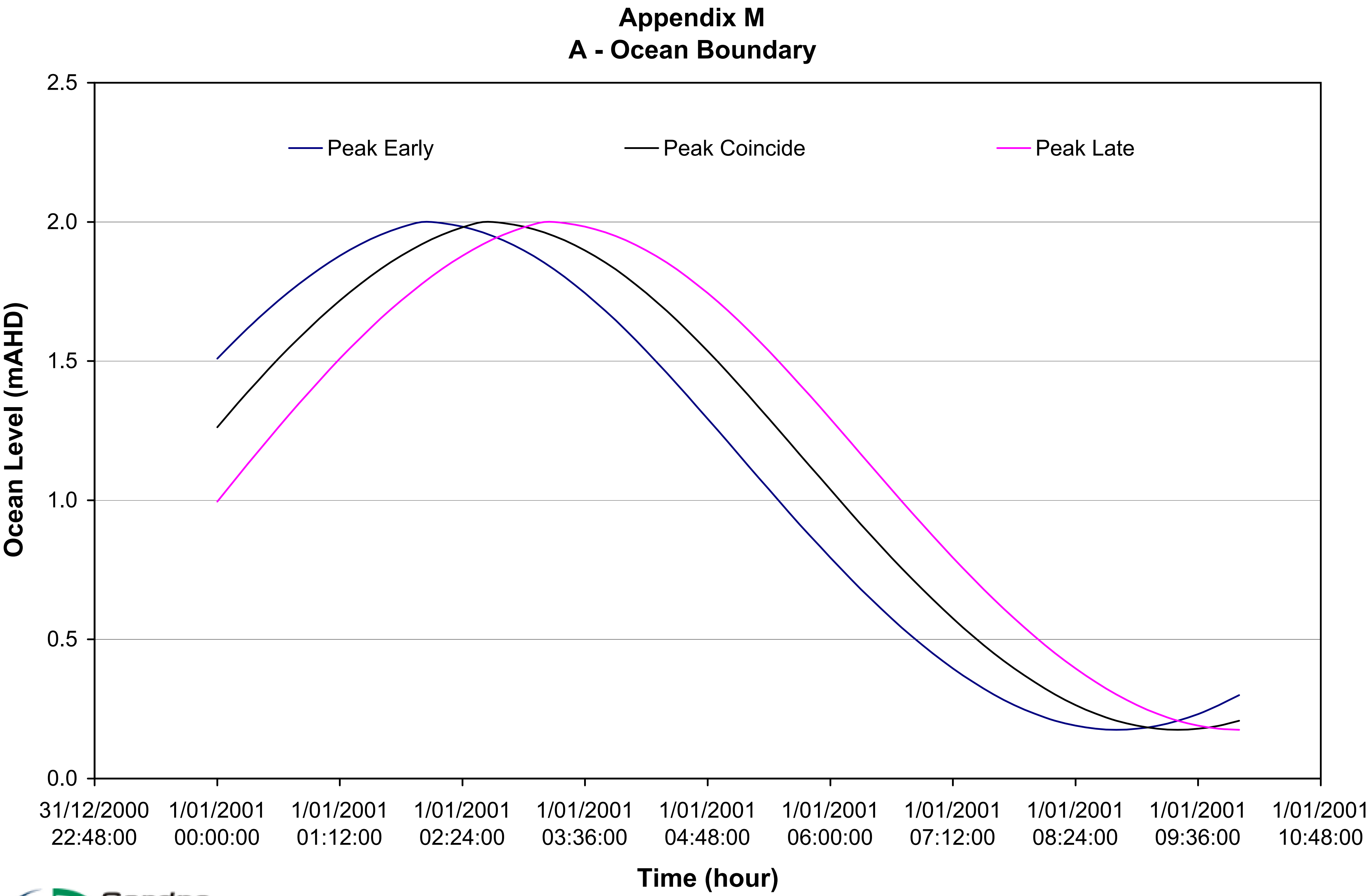
Cross Section	Chainage	1%-2hr WL (mAHD)	Flow		D/S Boundary		Roughness		Scour
			+20%	-20%	+20%	-20%	+20%	-20%	
King-236	-	12.57	0.00	0.00	0.00	0.00	0.00	0.00	0.00
Lake-ent-0	-	12.87	0.00	0.00	0.00	0.00	0.00	0.00	0.00
Lake-ent-500	-	9.25	0.00	0.00	0.00	0.00	0.00	0.00	0.00
Lake-ent-900	-	8.31	0.19	-0.06	0.00	0.00	0.09	-0.06	0.00
Lake-ent-1150	-	7.75	0.18	-0.06	0.00	0.00	0.09	-0.06	0.00
Lake-ent-1400	-	7.19	0.16	-0.05	0.00	0.00	0.08	-0.05	0.00
Lake-ent-1800	-	4.31	0.05	-0.19	0.00	0.00	0.05	-0.07	0.00
Lewarra-0	-	9.74	0.15	-0.11	0.00	0.00	0.08	-0.07	0.00
Lewarra-62	-	9.01	0.14	-0.12	0.00	0.00	0.09	-0.07	0.00
Lewarra-94	-	8.73	0.11	-0.11	0.00	0.00	0.07	-0.07	0.00
Pleasant-0	-	12.47	0.15	-0.16	0.00	0.00	0.13	-0.16	0.00
Pleasant-95	-	11.86	0.12	-0.16	0.00	0.00	0.09	-0.16	0.00
Plsnt	-	11.82	0.07	-0.04	0.00	0.00	0.05	-0.04	0.00
Oakley Creek									
CS74J	-215	3.00	0.06	-0.08	0.00	0.00	0.04	-0.08	0.00
CS74	-152	2.58	0.19	-0.11	0.07	-0.05	0.06	-0.08	0.00
CS75	-110	2.57	0.21	-0.27	0.08	-0.09	0.06	-0.13	0.00
CS76	-50	2.58	0.21	-0.27	0.08	-0.08	0.06	-0.12	0.00
CS77	0	2.57	0.21	-0.27	0.08	-0.08	0.05	-0.12	0.00
CS78	33	2.59	0.24	-0.28	0.08	-0.07	0.05	-0.09	0.00
CS81	111	1.93	0.03	-0.07	0.04	-0.02	0.00	0.02	-0.01
Oakley - Overland									
CS70	-	4.95	0.07	-0.31	0.00	0.00	0.06	-0.31	0.00
CS71	-	3.19	0.06	-0.18	0.00	0.00	0.06	-0.17	0.00
CS72	-	2.97	0.04	-0.06	0.00	0.00	0.03	-0.07	0.00
CS73	-	3.04	0.06	-0.08	0.00	0.00	0.05	-0.09	0.00
CS79	-	2.02	0.02	-0.04	0.00	0.00	0.03	-0.05	0.00
Tongarra Creek									
-100	-100	12.08	0.06	-0.07	0.00	0.00	0.06	-0.06	0.00
-50	-50	10.95	0.34	-0.23	0.00	0.00	0.07	-0.07	0.00
0	0	10.94	0.35	-0.38	0.00	0.00	0.06	-0.05	0.00
50	50	10.95	0.35	-0.38	0.00	0.00	0.06	-0.05	0.00
100	100	10.95	0.35	-0.38	0.00	0.00	0.05	-0.05	0.00
150	150	10.95	0.35	-0.38	0.00	0.00	0.05	-0.05	0.00
200	200	10.95	0.35	-0.38	0.00	0.00	0.05	-0.05	0.00
250	250	10.94	0.35	-0.38	0.00	0.00	0.05	-0.05	0.00
scc-12	297	9.12	0.06	-0.07	0.00	0.00	0.06	-0.11	0.00
scc-11	337	9.00	0.07	-0.09	0.00	0.00	0.04	-0.04	0.00
scc-10	408	8.82	0.06	-0.08	0.00	0.00	0.04	-0.04	0.00
scc-9	480	8.10	0.06	-0.07	0.00	0.00	0.06	-0.07	0.00
scc-8	568	7.69	0.07	-0.08	0.00	0.00	0.05	-0.05	0.00
scc-7	650	7.52	0.05	-0.06	0.00	0.00	0.05	-0.05	0.00
scc-6	715	7.28	0.04	-0.04	0.00	0.00	0.03	-0.04	0.00
scc-5	846	6.70	0.14	-0.11	0.00	0.00	0.10	-0.06	0.00
scc-4	928	6.67	0.16	-0.12	0.00	0.00	0.11	-0.03	0.00
scc-3	940	6.67	0.16	-0.12	0.00	0.00	0.11	-0.03	0.00
scc-2	963	6.57	0.19	-0.12	0.00	0.00	0.14	-0.04	0.00
990	990	5.83	0.05	-0.08	0.00	0.00	0.09	-0.24	0.00
1010	1010	5.82	0.06	-0.09	0.00	0.00	0.06	-0.17	0.00
1030	1030	5.87	0.07	-0.11	0.00	0.00	0.03	-0.06	0.00
1050	1050	5.87	0.07	-0.11	0.00	0.00	0.03	-0.06	0.00
1067	1067	5.87	0.07	-0.11	0.00	0.00	0.03	-0.06	0.00
1083	1083	5.87	0.07	-0.11	0.00	0.00	0.03	-0.06	0.00
1095	1095	5.86	0.07	-0.11	0.00	0.00	0.03	-0.06	0.00
1104	1104	5.84	0.06	-0.11	0.00	0.00	0.03	-0.06	0.00
1115	1115	5.74	0.06	-0.10	0.00	0.00	0.05	-0.08	0.00
1132	1132	5.60	0.06	-0.10	0.00	0.00	0.06	-0.10	0.00
1153	1153	5.51	0.06	-0.10	0.00	0.00	0.06	-0.10	0.00

Appendix L - Summary of Sensitivity Analysis - Change in Water Level for the 50% 2hr AEP Event

Cross Section	Chainage	1%-2hr WL (mAHD)	Flow		D/S Boundary		Roughness		Scour
			+20%	-20%	+20%	-20%	+20%	-20%	
1173	1173	5.42	0.06	-0.10	0.00	0.00	0.06	-0.10	0.00
1192	1192	5.31	0.05	-0.09	0.00	0.00	0.05	-0.10	0.00
1212	1212	5.21	0.04	-0.08	0.00	0.00	0.05	-0.09	0.00
1226	1226	5.17	0.04	-0.09	0.00	0.00	0.04	-0.08	0.00
1241	1241	5.12	0.04	-0.09	0.00	0.00	0.03	-0.07	0.00
1257	1257	5.03	0.04	-0.11	0.00	0.00	0.03	-0.08	0.00
1271	1271	4.99	0.05	-0.12	0.00	0.00	0.03	-0.07	0.00
1288	1288	4.97	0.05	-0.13	0.00	0.00	0.02	-0.07	0.00
1310	1310	4.91	0.07	-0.14	0.00	0.00	0.02	-0.07	0.00
1350	1350	4.41	0.06	-0.08	0.00	0.00	0.02	-0.04	0.00
1400	1400	3.23	0.03	-0.04	0.00	0.00	0.02	-0.04	0.00
1500	1500	2.68	0.05	-0.09	0.01	-0.01	0.01	-0.03	0.00
1536	1536	2.60	0.05	-0.08	0.01	-0.01	0.02	-0.03	0.00
1583	1583	2.40	0.04	-0.06	0.01	-0.02	-0.01	0.00	0.00
1640	1640	2.41	0.04	-0.06	0.01	-0.02	-0.02	0.01	0.00
1700	1700	2.41	0.04	-0.06	0.01	-0.02	-0.02	0.01	0.00
1750	1750	2.41	0.04	-0.06	0.01	-0.02	-0.02	0.01	0.00
1800	1800	2.41	0.04	-0.06	0.01	-0.02	-0.02	0.01	0.00
1850	1850	2.41	0.04	-0.06	0.01	-0.02	-0.02	0.01	0.00
1896C	1896	2.14	0.15	-0.19	0.12	-0.10	0.03	-0.04	-0.01
1896L	1896	2.41	0.04	-0.06	0.01	-0.02	-0.02	0.01	0.00
1896R	1896	2.41	0.04	-0.06	0.01	-0.02	-0.02	0.01	0.00
1950C	1950	2.14	0.15	-0.19	0.12	-0.10	0.03	-0.04	-0.01
1950L	1950	2.41	0.04	-0.06	0.01	-0.02	-0.02	0.01	0.00
1950R	1950	2.41	0.04	-0.06	0.01	-0.02	-0.02	0.01	0.00
1995C	1995	2.14	0.15	-0.19	0.12	-0.10	0.03	-0.04	-0.01
1995L	1995	2.41	0.04	-0.06	0.01	-0.02	-0.02	0.01	0.00
1995R	1995	2.41	0.04	-0.06	0.01	-0.02	-0.02	0.01	0.00
2040C	2040	2.14	0.15	-0.19	0.12	-0.10	0.03	-0.04	-0.01
2040L	2040	2.41	0.04	-0.06	0.01	-0.02	-0.02	0.01	0.00
2040R	2040	2.41	0.04	-0.06	0.01	-0.02	-0.02	0.01	0.00
2100C	2100	2.14	0.15	-0.19	0.12	-0.10	0.03	-0.04	-0.01
2100L	2100	2.41	0.04	-0.06	0.01	-0.02	-0.02	0.01	0.00
2100R	2100	2.40	0.04	-0.06	0.01	-0.02	-0.02	0.01	0.00
CS88	2159	2.12	0.14	-0.18	0.12	-0.10	0.03	-0.04	-0.01
CS89	2390	2.10	0.14	-0.17	0.12	-0.10	0.03	-0.03	-0.01
CS90	2463	2.10	0.14	-0.17	0.12	-0.10	0.03	-0.03	-0.01
CS91	2718	2.07	0.14	-0.17	0.12	-0.10	0.02	-0.02	-0.01
CS94	2905	2.04	0.14	-0.18	0.13	-0.11	0.02	-0.02	-0.02
CS95	3071	2.02	0.15	-0.19	0.14	-0.12	0.01	-0.01	-0.02
CS96	3189	1.97	0.16	-0.19	0.16	-0.12	0.00	0.00	-0.02

APPENDIX M

IMPACT OF TIME VARYING OCEAN BOUNDARY – SUMMARY OF RESULTS

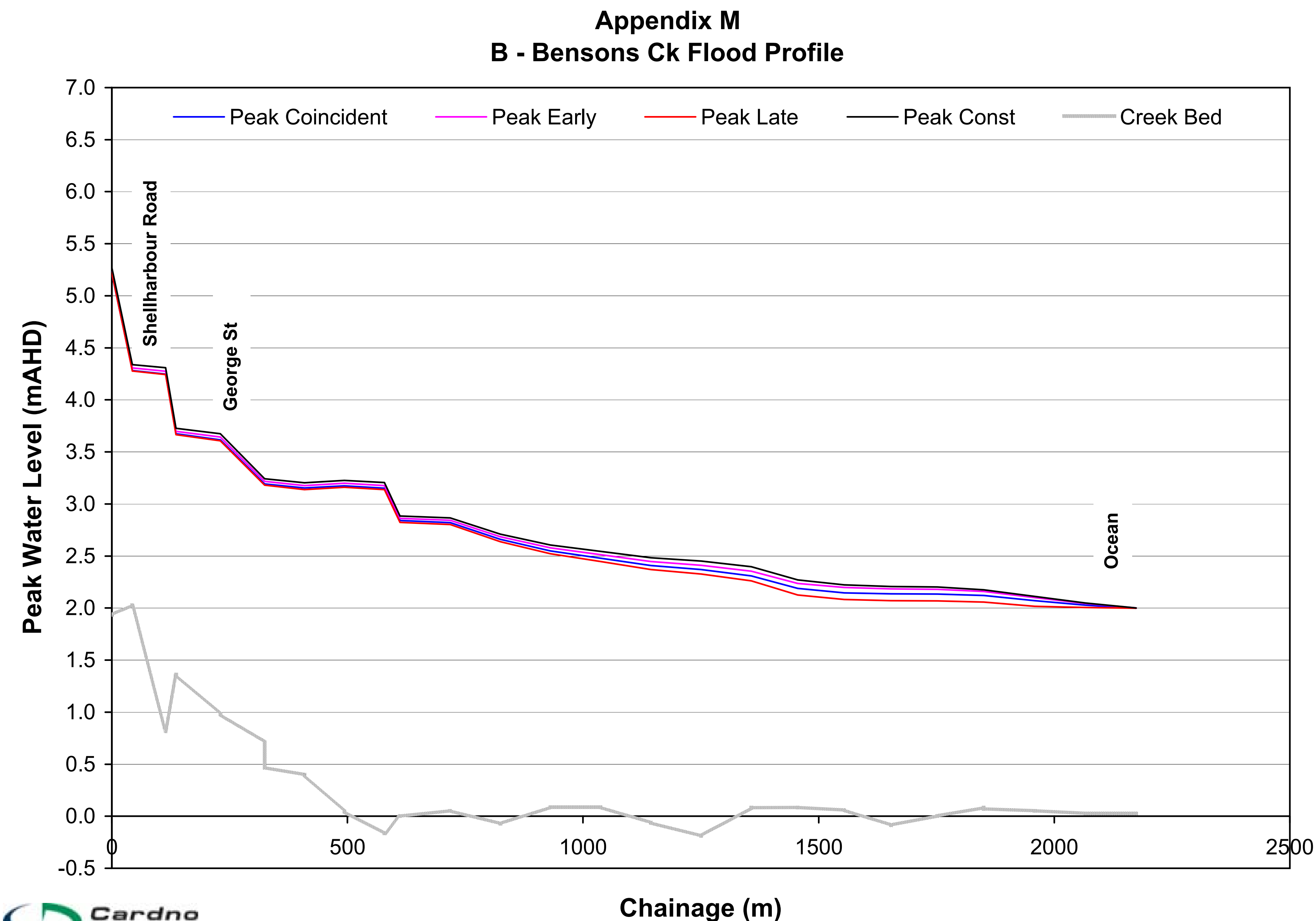


Cardno Lawson Treloar

J1959/R1974/V5
January 2006

Elliot Lake - Little Lake
Flood Study

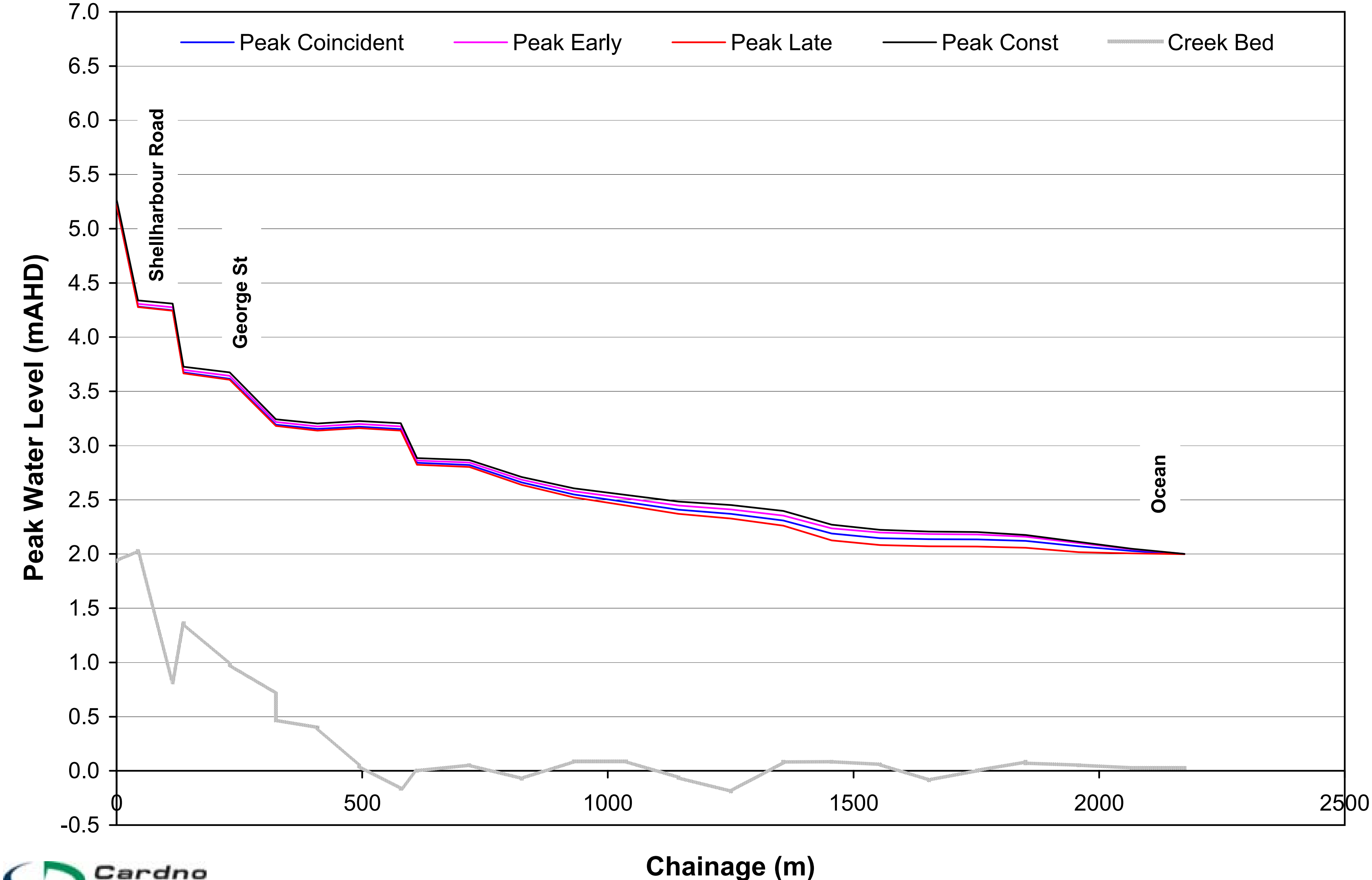
FIGURE M (A)
OCEAN BOUNDARY
J1959\Appendices V5\Appendix M-Ocean-Levels.xls



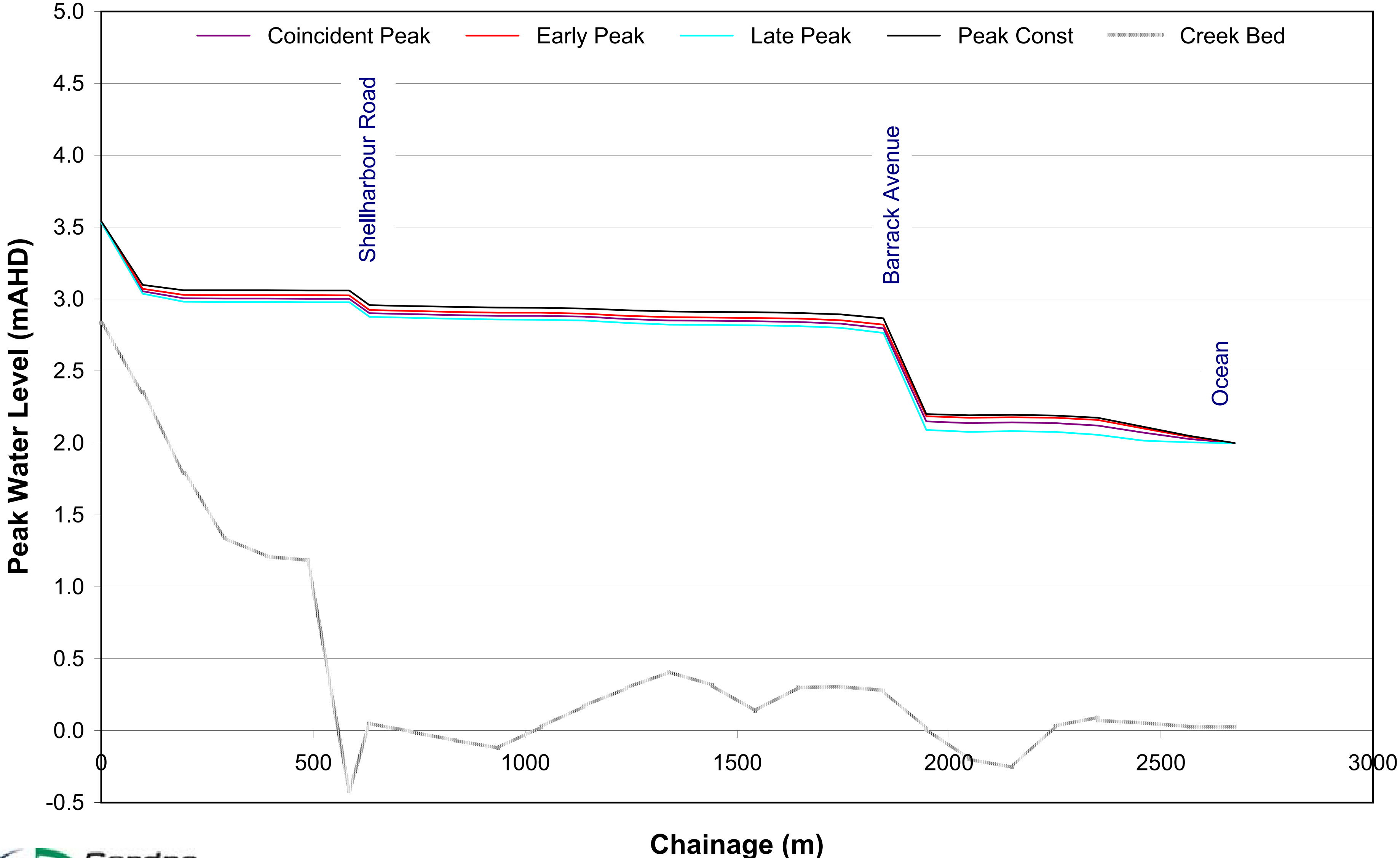
Cardno Lawson Treloar

FIGURE M (B)
BENSONS CK FLOOD PROFILE

Appendix M
B - Bensons Ck Flood Profile



Appendix M
C - Tongarra Ck Flood Profile



Cardno Lawson Treloar