Stony Creek Flood Study

Report Prepared For

Lake Macquarie City Council

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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 financial assistance to Councils for flood mitigation works to alleviate existing flooding problems. The policy also provides for State Government technical assistance to Councils to ensure that the management of flood prone land is consistent with the flood hazard and that future development does not create or increase flooding problems in flood prone areas.

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 Environmental Planning Instruments to ensure new development is compatible with the flood hazard.

The Stony Creek Flood Study is the first stage of the management process for the Stony Creek Catchment. The study, which has been prepared for Lake Macquarie Council by Cardno Lawson Treloar Pty Ltd, defines flood behaviour for existing catchment conditions.

EXECUTIVE SUMMARY

A flood study of the Stony Creek catchment has been undertaken to define the nature and extent of flooding in the area for a range of design flood events. The flood study has been carried out for the existing catchment conditions only. All of the necessary data for the study was collated from various sources, including Lake Macquarie City Council.

The Stony Creek catchment lies within the wider Lake Macquarie Catchment to the northwest of the lake entrance at Swansea. The headwaters are located in the west of the catchment in the Awaba State Forest. The catchment drains to Lake Macquarie through Edmunds Bay. The catchment has an area of 46.4 km^2 and includes the suburbs of Toronto, Blackalls Park, Awaba and other suburbs to Freemans Waterhole in the west. In the upper reaches of the catchment the land use is rural or bushland. The F3 Freeway passes through the upper reaches of the catchment with a large bridge conveying flow on Palmers Creek and culverts conveying flow for all the minor creek crossings. Other major controls in the upper catchment are the Coal Haul Road and the Great Northern Railway.

In the past, flooding in the Stony Creek catchment has caused property damage and posed a high hazard to residents living close to the creeks in the area. In particular, the storm event of February 1981 caused widespread damage in the catchment. The magnitude of this event was greater than the 200 year ARI design event. Lower parts of the catchment, near Fennell Crescent and area to the north of Awaba Road along Stony Creek, are severely affected in major flood events. The number of creek crossings in the area are likely to exacerbate flooding and potential damages to surrounding properties.

Flood data was obtained from a number of sources. The data included historical storm and rainfall data, ground survey and hydraulic structure data, recorded flood levels, previous flood study reports and cadastral and topographical data. The data was used to undertake the various components of the study as well as to aid in the presentation of the study results.

Estimation of flooding behaviour was undertaken by developing two computer models to simulate the hydrologic and hydraulic aspects of flooding. The hydrologic modelling package RAFTS was utilised for routing flow through the catchment and to determine runoff from various parts of the catchment. Predicted hydrographs from RAFTS were then input to the hydraulic model SOBEK 1D/2D for the determination of peak flood level, velocity and discharge for various design rainfall events. The design events investigated for this study were the 200 year, 100 year, 50 year, 20 year, 10 year and 5 year Average Recurrence Interval (ARI) events together with the Probable Maximum Flood (PMF).

Flood behaviour was assessed using two-dimensional high definition hydraulic modelling for those areas deemed significant. This detailed modelling provides better understanding of flooding processes in the flood affected areas.

Extensive flood level data within the catchment for the storm events of February 1981 was available for calibration of the hydraulic model. The flood data for these events was acquired by Council through resident survey carried out prior to the commencement of flood study. The hydraulic model was calibrated to the historic flood event. The hydrologic model was indirectly validated through hydraulic model calibration.

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

The model results reflect the observed flooding behaviour in Stony Creek catchment. The storm durations of 4.5, 9 and 36 hours were generally found to be critical in the catchment, with 36 hours for majority of the overland flow affected area. For the PMF, the critical duration was generally 4 hours in the upper portion of the catchment, and 5 hours in the low lying areas.

In the upper catchment, the Northern Railway is overtopped during the 10 year ARI design event. The main coal haul road, just downstream of the railway embankment, is only overtopped in the PMF design event. The Sewerage Treatment Plant is overtopped in the PMF event, but not in the 200 year ARI design event.

Flooding in the lower parts of the catchment, upstream of the Railway Parade bridges across Mudd Creek and Stony Creek, is controlled by the general topographic constriction at this location. The majority of the water is forced through the waterway area of these two bridges, which creates a significant hydraulic control.

The lower parts of the catchment, from Fennell Bay to up to Railway Parade are primarily affected by elevated Lake Macquarie levels in addition to catchment flooding. Two design flood conditions were determined; one dominated by the catchment flooding and the other by Lake flooding. The design flood levels were obtained from a peak water level envelope from the two flooding scenarios.

Railway Parade at Stony Creek is not overtopped up to the 200 year ARI event. It is worth noting that it was close to being overtopped in the 1981 event, which is approximately 0.59 m above the 100 year ARI level at this location. However, Railway Parade is overtopped at Mudd Creek starting with a 10 year ARI event.

The limits of predicted flood extents for the 200 year, 100 year, 50 year, 20 year, 10 year and 5 year ARI events together with the PMF are provided in plan form. Tabulated modelling results are also provided for a number of locations in the floodplain.

The flood hazard has been determined for the 200 year, 100 year, 50 year, 20 year, 10 year and 5 year ARI events and 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 200 year, 100 year, 50 year, 20 year, 10 year and 5 year ARI events and PMF.

All the above information has been prepared in a geographic information system (GIS), which is compatible with the Council's system (MapInfo). 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. Detailed results of the analysis are provided and compared with the design flood levels for the existing catchment conditions.

The likely flood damages resulting from the design flood events have been estimated based on the latest advice from DIPNR. Total flood damages to residential commercial and industrial properties along with the total number of properties with above floor flooding are summarised below:

Design Event	**Number of Properties with above floor flooding**	**Total Damages (millions)**
PMF	295	19.50
200 year	87	3.76
100 year	57	2.65
50 year	29	1.59
20 year	10	0.82
10 year	4	0.43
5 year	0	0.14
Feb 1981	135	7.77

Damage calculations for the February 1981 event in the recent dollar terms indicate that this event would cause three times as much damage as the 100 year ARI event with twice the number of properties with above floor flooding. This finding will have a bearing on the determination of Flood Planning Levels in the catchment in the subsequent stages of the Floodplain Risk Management process.

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.

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GLOSSARY

Terms in this Glossary have been derived or adapted from the NSW Government *Floodplain Management Manual*, 2001.

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.
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.
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
AWE	Average Weekly Earnings
BoM	Bureau of Meteorology
CMA	Catchment Management Authority
CPI	Consumer Price Index
DCP	Development Control Plan
DIPNR	Department of Infrastructure, Planning and Natural Resources
FPL	Flood Planning Level
FRMC	Floodplain Risk Management Committee
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
LEP	Local Environment Plan
LGA	Local Government Area
m	metre
m^2	Square metres
m^3	Cubic metres
mAHD	Metres to Australian Height Datum
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
RAFTS	RAFTS proprietary software package
RTA	Roads and Traffic Authority

1. INTRODUCTION

Stony Creek lies within the Lake Macquarie City Council Local Government Area on the western side of the Lake Macquarie catchment. The location of the catchment can be seen in Figure 1.1. The catchment comes under the management of the Lake Macquarie City Council, which is responsible for the floodplain management in the catchment. Management of flooding issues within the Stony Creek Catchment is being undertaken by way of the Floodplain Risk Management Process, which aims to create a Floodplain Risk Management Plan.

The first step in the preparation of a Floodplain Risk Management Plan is to undertake a detailed Flood Study for the catchment. Lake Macquarie City Council commissioned Cardno Lawson Treloar Pty Ltd (Previously Lawson and Treloar Pty Ltd) to undertake this flood study to determine the flood behaviour for the 200 year, 100 year, 50 year, 20 year, 10 year and 5 year ARI floods and the 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. Secondly, a high-definition hydraulic computer model of the major creeks and floodplain was established and calibrated using historical flood level data. The hydraulic model was then used with design rainfall conditions to simulate design flood behaviour in the catchment.

The hydraulic model developed in this study has been used to simulate flooding which may occur under existing catchment conditions. The model may be used to investigate various flood 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 existing catchment condition flood behaviour for mainstream flooding in the catchment with due consideration to the impact of Lake Macquarie levels on flooding characteristics.
- Define design flood levels, velocities and flow distributions for the catchment.
- Define the extent of flooding for the 200 year, 100 year, 20 year, 10 year and 5 year ARI 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 runoff hydrographs.

- A hydraulic model to convert runoff hydrographs into water levels and velocities throughout 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 all the potential overland 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 boundary level from Lake Macquarie.

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. This section also provides details of the survey data used in the study area.

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

Section 6 provides details of the appropriate lake water level boundaries adopted for historic events 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.

Sections 10 and 11 provide details of provisional flood hazard and hydraulic categorisation in accordance with the *Floodplain Management Manual* (NSW Government, 2001).

Section 12 summarises the results of flood damages assessment.

Section 13 summarises the study results and provides discussion on various aspects of the results.

Section 14 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.

3. AVAILABLE 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 Reports

Following reports were obtained from the Council for use in the study:

Manly Hydraulics Laboratory 1998, Lake Macquarie Flood Study, Part 1 – Design Lake Water Levels and Wave Climate, Australia.

Manly Hydraulics Laboratory 1998, Lake Macquarie Flood Study, Part 2 – Foreshore Flooding, Australia.

Weatherex Meteorological Services, *Stony Creek Catchment : Rainfall Study, Storm of 6/7 February 1981*, Australia.

Sinclair Knight & Partners Pty Ltd 1981, *Report on the 7th February 1981 Flood in the Stony Creek Catchment – Stage II : Hydrology*, Australia.

Sinclair Knight & Partners Pty Ltd 1981, *Report on the 7th February 1981 Flood in the Stony Creek Catchment – Stage III : Hydraulics*, Australia.

Sinclair Knight and Partners 1981*, February 1981 Flood and Flood Plain Management Plan Stony Creek, Toronto*, Australia.

Wayne Perry & Associates and Webb McKeown & Associates 1992, *Environmental Impact Statement for Dredging and Rehabilitation of Mudd Creek, Blackalls Park, Lake Macquarie, NSW*, Australia.

Webb McKeown & Associates 2000, Lake Macquarie Floodplain Management Study, Australia.

Webb McKeown & Associates 2001, Lake Macquarie Floodplain Management Plan, Australia.

3.2 Survey

Survey is required to define the physical attributes of the floodplain topography including the creek cross sections and the associated floodplain levels. Some survey was available from previous Council studies and other Council work. This data required augmentation with a broad scale survey of the floodplain. This survey was captured using aerial photography and stereo plotting. A detailed review of the available survey and the additional survey captured is provided below.

3.2.1 Existing Survey

Within the study area a reasonable spread of data was available. This data was complied by Lake Macquarie Council Consulting Surveyors (LMC^2) and converted into real world co-ordinates for import into the GIS. The survey comprised creek cross sections, land survey shots and flood level survey captured after the 1981 flood event. In addition to this data a plan of cross sections used to assess the levee around the West Toronto Industrial area was digitised from paper plans provided to Cardno Lawson Treloar by Council. All the survey data provided by the Council is shown in Figure 3.1

In addition to the survey data the 2m LIC contour data was also available. This data was used to supplement the survey data in areas where accurate flood levels are not required, yet the area is required to be represented in the hydraulic model. Examples of this include the northern side of the Fennell Bay.

3.2.2 Additional Survey

Additional survey was collected as a part of this study. Fugro Spatial Solutions Pty Ltd conducted the aerial component of the survey. This aerial survey allowed for a detailed 2-D terrain model to be established as a part of the modelling process. The survey and the associated photography extends from the Main Northern Railway embankment in the west, down to the outlet of Stony Creek into Fennell Bay in the east.

Additional ground survey was also collected by Lake Macquarie Council Consulting Surveyors (LMC^2). This survey included ground control points for the aerial surveyors (used to validate aerial survey) as well as additional hydraulic details, such as bridges and culverts.

This survey data is shown in Figure 3.1.

3.3 Resident Survey and Community Consultation

A survey was carried out to collate information on historical flooding from the local residents. A questionnaire, prepared in consultation with Council, was distributed to all the residents in the floodplain and neighbouring areas to seek information about the flooding behaviour in the catchment. The questionnaire sought information as to whether residents have experienced flooding, the nature and depth of flooding and the timing of such floods. The residents were also asked whether they could identify any historic flood levels. A copy of the questionnaire is provided in Appendix B.

Around 900 questionnaires were distributed and 202 responses were received. The information obtained from the residents was processed and a summary is provided in Appendix B. A few residents, who had either experienced flooding or had a good knowledge of flooding in the area were selected for further interviewing. Phone interviews were carried out and further anecdotal information as well as reported flood levels were collected.

The majority of the information was related to the February 1981 flood event, which was a major event across the Lake Macquarie Catchment including the Stony Creek Catchment.

3.4 Historical Rainfall Data

A number of historical storm events were identified as a part of the resident survey process. The following storm events were identified:

- February 1981
- 1953
- 1983
- 1990

Based on the resident survey responses, as well as other historical sources such as newspapers, the most significant of these events was the 6-7th February 1981 event. Rainfall data for this event was taken from the SKM (1981) study.

3.5 Historic Flood Levels

Historical recorded flood levels were provided by Council. These levels were collected following the 6-7 February 1981 storm event. The recorded flood levels along with their locations in the catchment are provided in Table 3.5.1.

Table 3.5.1 1981 Historic Flood Levels

Street Number	Address	Observed Level (m AHD)
58	Fennell Crescent	2.68
68	Fennell Crescent	2.68
79	Fennell Crescent	2.50
67	Fennell Crescent	2.50
79	Railway Parade	2.32
86A	Fennell Crescent	2.48
82	Fennell Crescent	2.48
74	Fennell Crescent	2.50
70	Fennell Crescent	2.50
52	Fennell Crescent	2.68
42	Fennell Crescent	2.78
48	Fennell Crescent	2.74
50	Fennell Crescent	2.74
46	Fennell Crescent	2.78
44	Fennell Crescent	2.78
80	Fennell Crescent	2.50
86	Fennell Crescent	2.48
78	Fennell Crescent	2.50

Street Number	Address	Observed Level (m AHD)
51	Fennell Crescent	2.65
81	Fennell Crescent	2.50
87	Fennell Crescent	2.48
89	Fennell Crescent	2.48
77	Fennell Crescent	2.50
75	Fennell Crescent	2.50
85	Fennell Crescent	2.48
61	Fennell Crescent	2.70
81	Railway Parade	2.32
76	Fennell Crescent	2.50
72	Fennell Crescent	2.50
84	Fennell Crescent	2.48
67	Fennell Crescent	2.50
69	Fennell Crescent	2.53
83	Fennell Crescent	2.48
53	Fennell Crescent	2.70
62	Fennell Crescent	2.68
60	Fennell Crescent	2.68
68	Fennell Crescent	2.68
52	Fennell Crescent	2.68
40	Fennell Crescent	2.86
88	FennellCrescent	2.48

In addition to the above levels, some other levels were interpreted from the historic flood observations and previous reports.

3.6 Stream Gauging Records

No stream gauging records exist for the Stony Creek catchment.

3.7 Cadastral, Planning and Topographic Data

Lake Macquarie Council provided cadastral base information and contour data for use in the study in digital format. Council also provided information about the land-use in the catchment (Local Environment Plan) and recent aerial photography.

All the data was provided in MapInfo GIS format. The GIS data used in the development of the models and presentation of the results include:

- 2m Department of Lands contour data.
- 10m Department of Lands contour data
- Cadastral Plan.
- Local Environment Plan (LEP)
- Catchment wide ortho-rectified aerial imagery
- Stormwater pit and pipe layout (hard copy only)

In addition to this data Cardno Lawson Treloar holds broad scale orthorectified imagery produced by the Department of Lands.

4. CATCHMENT DESCRIPTION

4.1 General

The Stony Creek catchment lies within the Lake Macquarie Catchment area to the northwest of the lake. The catchment headwaters are located in the west of the catchment in the Awaba State Forest and the catchment outlet is at Fennell Bay in Lake Macquarie. There are several large tributaries of the Stony Creek. Figure 4.1 shows the major creeks within the Catchment. The catchment has an approximate area of 46.35 km^2.

In the upper reaches of the catchment the land use is rural or bushland. The F3 Freeway passes through the upper reaches of the catchment with a large bridge conveying flow on Palmers Creek and culverts conveying flow for all the minor creek crossings. Other major controls in the upper catchment are the Coal Haul Road and the Great Northern Railway.

In the lower reaches, the catchment is developed with low to medium density housing and some industrial and commercial areas.

4.2 Major Creeks

Stony Creek: Its headwaters are in the southwest of the catchment, south of the town of Awaba. The creek flows under the Great Northern Railway Line through culverts and then passes under an old wooden road bridge on Wilton Road. From this bridge the creek flows north parallel to Wilton Road and passes under Awaba Road and the Great Northern Railway Line to end up on the western side of the line. There are three main culverts under the Great northern Railway Line to convey flow towards Toronto, with an additional smaller culvert slightly to the south. Just downstream of the Great Northern Railway Line the Coal Haul Road forms another flow control. Downstream of the Coal Haul Road the creek flows into a broad floodplain and flows past the west Toronto Industrial Area. Just downstream of the industrial area, at the High Street Ford, a weir across the creek forms the tidal limit for the Lake Macquarie waters. Further downstream, the creek flows south of Blackalls Park Sewerage Treatment Plants before entering a wide floodplain, where most of the flood affected properties are located. Past this location a series of controls exist in the form of pipeline bridges, a disused railway bridge, a pedestrian bridge and the Railway Parade Bridge. The urban area in the catchment discharges to Stony Creek via piped conduit or concrete lined channels.

Palmers Creek: This creek is the major tributary of Stony Creek. The headwaters of this creek are in the west of the catchment near Freemans Waterhole. This catchment is largely undeveloped with landuses including bushland and rural pastoral land. Palmers Creek Joins Stony Creek just upstream of the Great Northern Railway Line (second crossing) (Figure 4.1).

Mudd Creek: This creek forms the former drainage path of the Blackalls Park Sewage Treatment Works. The creek forms in the tidal reaches of the urban area and flows parallel to Stony Creek with the two creeks separated by a low floodplain.

Mudd Creek Flows into a water body known as Edmunds Bay, which is a part of Fennell Bay.

Creek at Carleton Street: This nameless creek has its headwaters in the hills south of Toronto in bushland areas. The creek then passes through the new urban development at the end of Forest Lake Way. The creek flows past Biraban Primary School and through the urban area of western Toronto. The creek is constricted and channelled past the swimming Centre, Awaba Road and the pedestrian bridge before opening out past Toronto High School. The creek joins Stony Creek just upstream of the high school.

LT Creek: This creek is not within the floodplain of Stony Creek, however, this creek also flows into Fennell Bay and as such the flow from this creek could impact on water levels along Stony Creek that are affected by Fennell Bay Water levels. This was particularly thought to be the case if the Fennell Bay Bridge was a hydraulic control.

4.3 Soil Types and Drainage Characteristics

The Department of Conservation and Land Management (NSW) soil maps covering the catchment are:

- Gosford – Lake Macquarie
- Cessnock
- Newcastle

These maps were referenced along with the Soil Landscapes of the Gosford – Lake Macquarie 1:100,000 Sheet (Murphy C, 1993).

A plan of the terrain of the catchment is provided in Figure 4.2.

4.4 Flooding Behaviour and History

The Stony Creek catchment has experienced flooding in the past. Most notable of the historic floods is the February 1981 flood. This flood is reported in the Sinclair Knight & Partners (1981) report on this event. The report details the nature and extent of the rainfall for the storm that lead to the flooding. The storm event is reported to have been in excess of the 100 year ARI event in the Stony Creek catchment. The rainfall analysis indicates that it was even greater than the 200 year ARI flood (figure 4.3).

The February 1981 storm caused significant flooding within the catchment. The majority of the rainfall fell within approximately 6 hours. The flooding within the study area occurred at night, with the majority of the flooding occurring in the early hours of 7 February 1981.

Resident reports and surveyed flood levels at the time indicate that some of the worst affected areas were the residential properties on Fennell Crescent, positioned between Mudd Creek and Stony Creek. Many of the residents in this area were forced onto their rooves, and were effectively marooned for a number of hours.

The Main Northern Railway, to the west of the catchment, was overtopped and significant damage occurred to the rail lines in that area.

5. HYDROLOGY

5.1 General

The following catchment attributes were considered in the hydrological analysis of the catchment:

- Rainfall intensity-frequency-duration (IFD) relationships
- Slopes and overland flow path, and
- Land use (pervious and impervious areas, catchment roughness).

The state of the catchment development at the time of the capture of the aerial survey was considered to be the existing state of the catchment and adopted for the hydrological analysis. Thus the level of catchment development as at 1 July 2003 is assumed to be the existing catchment condition and has been used in the modelling carried out for this study.

5.2 Establishment of the Hydrological Model

Runoff hydrographs for the flood study were estimated using the RAFTS (WP Software, 2000) rainfall-runoff modelling package. Based on the topographic features (2-metre LPI contour map) and land-use (Councils aerial photography), the catchment was divided into 136 sub-catchments. 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 in RAFTS).

For urban areas, 40% imperviousness was assumed, based upon site inspection and aerial photography. Open residential areas in the rural areas were assumed to have an impervious area of 10%.

The sub-catchment layout is shown in Figure 5.1 and details of these sub-catchments can be found in Appendix D.

Important parameters used in the development of the RAFTS model are provided In Table 5.2.1.

Table 5.2.1: RAFTS Model Parameters

RAFTS Parameter	Forest	Rural areas and open grass land	Urban Impervious Area
Manning's 'n' for subcatchments	0.100	0.050	0.020
Storage delay parameter, B	1.13	1.13	1.13
Hydrograph Routing Lag	Defined for the individual channels using the method described by HEE (1993).		

The hydrograph routing lags were defined for the individual channels using the method described by Hee (1993). Travel times were modified for flows through culverts or pipes in order to represent the lag created due to ponding behind these structures.

5.3 Model Calibration

As there were no flow gauging stations within the catchment, the hydrological model could not be directly calibrated.

In a previous study (SKM, 1981), the RSWM hydrological model (previous version of RAFTS) was developed for the Stony Creek catchment. The model parameters were established by calibrating the model to the adjacent Jigadee Creek catchment, where stream flow data was available. However the calibration process is questionable. (the high rainfall losses, established from the calibration of Jigadee Creek catchment, when applied to Stony Creek catchment produced modelled flood levels too low as compared to the recorded levels) It appears that the rating curve for the stream gauge did not provide accurate flow data for calibration.

The RAFTS model parameters for the current study were therefore derived independently from the SKM (1981) study. The model parameters were indirectly validated through the hydraulic model calibration.

5.4 February 1981 Modelling

The rainfall data was obtained from the Weatherex report (1981) which provides the rainfall temporal pattern for the event. A constant rainfall depth was assumed over the Stony Creek Catchment, based on the Weatherex report (1981), which determined isohyets over the regional area including the Stony Creek Catchment. The rainfall depth of 350mm over a 6 hour period was adopted from the study (Weatherex, 1981) to create the rainfall time series as shown in Figure 4.3.

5.5 Design Rainfall

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 200 year, 100 year, 50 year, 20 year, 10 year and 5 year ARI events were developed using standard techniques provided in AR&R (1999). IFD parameters derived from AR&R (1999) are presented in Table 5.4.1.

Table 5.4.1: Design IFD Parameters for Stony Creek

Parameter	Value
2 Year ARI 1 hour Intensity	33 mm/hr
2 Year ARI 12 hour Intensity	7.5 mm/hr
2 Year ARI 72 hour Intensity	2.5 mm/hr
50 Year ARI 1 hour Intensity	65.0 mm/hr
50 Year ARI 12 hour Intensity	15.0 mm/hr
50 Year ARI 72 hour Intensity	5.4 mm/hr
Skew	0.01
F_2	4.31
F_{50}	16.0
Temporal Pattern Zone	1

The Probable Maximum Precipitation (PMP) was estimated using the publication *Estimation of Probable Maximum Precipitation in Australia: Generalised Short-Duration Method* (Hydrometeorological Advisory Services, June 2003) recommended by the Bureau of Meteorology. The recommended isohyets for spatial distribution of PMP are shown in Figure 5.2. Recommended values used in the calculation of the PMP are provided in Table 5.4.2 below. Table 5.4.3 shows four rainfall intensities for the PMP, which correspond to isohyets A, B, C and D (Figure 5.2).

Table 5.4.2: PMP Calculation Values

Parameter	Value
Moisture Adjustment Factor	0.725
Elevation Adjustment Factor	1.0
Percentage Rough	100%
Area Enclosed A (km^2)	2.6
Area Enclosed B (km^2)	16.0
Area Enclosed C (km^2)	41.9
Area Enclosed D (km^2)	46.3

Estimated design storm rainfall intensities for the full range of storm events and durations are presented in Table 5.4.3.

Table 5.4.3: Design Rainfall Intensities (mm/h)

	5 year ARI	10 year ARI	20 year ARI	50 year ARI	100 year ARI	200 year ARI	PMP			
							A	B	C	D
15 min	89	100	115	134	148	163	680	560	520	480
30 min	63	71	81	95	106	116	480	420	380	360
45 min	51	57	65	76	85	93	413	360	320	307
1 hour	42.9	48.4	56	65	72	79	360	320	290	280
1.5 hour	33.9	38.3	44.0	51	57	63	307	273	247	233
2 hour	28.6	32.3	37.1	43.5	48.3	53	270	240	215	200

	5 year ARI	10 year ARI	20 year ARI	50 year ARI	100 year ARI	200 year ARI	PMP			
							A	B	C	D
2.5 hour	25.0	28.3	32.5	38.1	42.4	45.0	240	212	192	176
3 hour	22.4	25.3	29.2	34.2	38.0	41.9	217	190	173	163
4 hour	18.8	21.3	24.6	28.8	32.0	34.8	188	165	148	135
5 hour	16.5	18.6	21.5	25.2	28.1	30.8	164	144	128	120
6 hour	14.8	16.7	19.3	22.6	25.2	27.8	145	128	115	107
9 hour	11.6	13.1	15.1	17.8	19.8	21.9	*	*	*	*
12 hour	9.73	11.0	12.8	15	16.7	18.5	*	*	*	*
18 hour	7.72	8.80	10.2	12.1	13.5	14.9	*	*	*	*
24 hour	6.54	7.48	8.71	10.3	11.6	12.9	*	*	*	*
30 hour	5.73	6.58	7.67	9.13	10.2	11.4	*	*	*	*
36 hour	5.13	5.91	6.90	8.24	9.26	10.3	*	*	*	*
48 hour	4.29	4.96	5.81	6.96	7.85	8.76	*	*	*	*
72 hour	3.27	3.80	4.48	5.4	6.11	6.85	*	*	*	*

*Not calculated.

5.6 Design Flow

The estimated design rainfalls were applied to the hydrologic model in order to predict design runoff hydrographs. Design rainfall losses were adopted in accordance with the AR&R (1999) guidelines and are provided in Table 5.5.1.

Table 5.5.1: Design Rainfall Losses Used in RAFTS

Catchment Type	Initial Loss (mm)	Continuing Loss (mm/hr)
Pervious	20	2.5
Impervious	1	0

For PMP estimates, an initial loss of 1mm and no rainfall continuing losses were assumed as per the recommendations of AR&R (1999).

Design flows were obtained for the 3hr, 4.5hr, 6hr, 9hr, 12hr, 18hr, 24hr, 36hr, 48hr and 72 hour duration storm events.

6. DOWNSTREAM BOUNDARY CONDITIONS

The flood study carried out by Manly Hydraulics Laboratory (1998) established the Lake Macquarie water levels for various design events. The impact of catchment runoff, elevated ocean water levels, local winds and the condition of the Swansea entrance channel was considered in the assessment of the flooding behaviour of the lake. The study also examined the joint probability of the above factors in generating the lake flooding.

As a compendium to the above study, Manly Hydraulic Laboratory (1998) determined the design flood levels for the lake foreshore area. These design levels were primarily based on the wave run-up process at the foreshore.

6.1 Model Boundary Conditions

The appropriate downstream boundary design flood levels for the hydraulic modelling purposes are those obtained from the lake flooding processes rather than the wave run-up process at the foreshore. The design lake levels are provided in Table 6.1.1 below:

Table 6.1.1 – Lake Macquarie Design Water Levels

Storm Event (ARI)	Downstream Boundary Level (mAHD)
PMF	2.63
100 year	1.38
50 year	1.24
20 year	0.97

The above design levels were adopted for the Stony Creek Flood Study.

Table 6.1.1 does not provide design levels for all design events investigated in the current study. The remaining design levels were sourced from the Lake Macquarie Floodplain Management Study (2000). The complete set of the design lake levels is provided in Table 6.1.2 below:

Table 6.1.2 – Lake Macquarie Design Water Levels

Storm Event (ARI)	Downstream Boundary Level (mAHD)
PMF	2.63
200 year	1.55
100 year	1.38
50 year	1.24
20 year	0.97
10 year	0.80
5 year	0.65
2 year	0.45

6.2 Downstream Boundary for the Design Flood Events

The joint probability of severe catchment flooding from the Stony Creek catchment together with severe lake flooding is small. Hence use of a rare catchment event with a rare lake level as a downstream boundary may not be appropriate. A more likely flooding scenario is where the catchment experiences severe flooding and the lake levels are at a more frequent flooding stage or vice versa. This approach was adopted in setting the downstream boundary for the design flood modelling. Thus two sets of model runs were carried out based on the following catchment and lake flooding conditions.

- Various ARI catchment flows combined with a 5 year ARI lake level
- Various ARI lake levels combined with a 5 year ARI catchment flows

Further details of design flood modelling are provided in Section 8.

7. HYDRAULIC MODELLING

7.1 Establishment of Hydraulic Model

The hydraulic modelling system, SOBEK 1D/2D, was used for hydraulic modelling. The model is fully dynamic hydraulic-routing model developed by WL|Delft Hydraulics of the Netherlands, which has been 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.

In the model schematisation for the Stony Creek Flood Study, the creeks were described as typical 1D branches with cross-sections defining the creek geometry. The 1D network is overlaid by a 2D grid of the floodplain. Once the creek capacity is exceeded, flow is able to spill into the 2D grid as an overland flow. During the flood recession, flow is also able to drain from the overland areas back into the defined creeks.

7.2 1D Model Setup

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, was also taken into account. Only main, well defined flowpaths identified in the catchment were included as 1D branches in the model (such as Stony Creek and its major tributaries). Other flowpaths, such as overland flow through properties and along roads, were described in the 2D component. The 1D model layout is shown in Figure 7.1.

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 7.1. The model cross sections are provided in Appendix E.

7.3 2D Model Setup

The major component of a two-dimensional model is the model grid or topographic grid. The model topography was developed from the survey data captured for the study (Section 3). The civil and surveying package 12D (12d Solutions Pty Ltd, 2004) was used to generate a detailed three dimensional surface of the flood study area.

Important hydraulic controls such as road embankments in areas where a road is likely to be overtopped by the floodwaters need to be correctly represented in the topographical grid. Wherever required, the controlling road levels were determined and applied over the width of the road to ensure the generated grid would accurately represent these levels.

SOBEK dynamically links 1D and 2D components of the model with creek cross sections represented within the 1D domain to ensure a high level of detail and accurate conveyance calculations. The floodwaters flow into the 2D topographical grid when the level of the grid cell is reached. To ensure the creek flow is calculated within the 1D domain, the creek must not be represented in the 2D grid. A three dimensional surface of the 2D area was therefore developed with the creek removed by using the top of creek bank breaklines. The 2D grid was then developed from this three dimensional surface.

The topographic grid used in the model had a grid size of 10 metres by 10 metres, which is suitable for providing details of the significant catchment features while allowing a moderate size grid with reasonable model computation time. SOBEK has the ability to accommodate parent/child grids, enabling grids of higher definition to be used in areas of interest. This feature can be utilised to assess any future developments in the 2D area. The grid details are outlined in Table 7.3.1 and can be viewed in Figure 7.2.

Table 7.3.1 – Two-dimensional Grid Parameters

Grid Parameter	Dimension
Origin *	364,340 6,346,350
Grid Size	10m
X-Dimension (east-west)	479
Y-Dimension (north-south)	270
Rotation	0

* MGA94 Co-ordinate System

The buildings in the floodplain were assumed to block the floodwaters completely (i.e. no active flow path through the buildings was considered nor any storage of floodwaters was allowed in the buildings).

7.4 Hydraulic Roughness

The hydraulic roughness for the 1D components was determined from a site inspection and aerial photography. Table 7.4.1 provides roughness values for various creek conditions.

Table 7.4.1 – One-dimensional Cross Section Roughness

Creek Condition	Manning's 'n'
Major Creeks	0.03
Concrete Drainage Channels	0.015

The 2D model component requires a 2D roughness grid to be generated which assigns a Manning's 'n' value to each grid cell. The roughness map was developed from data collected during the site visit and aerial photography. The grid is presented in Figure 7.3 and Table 7.4.2 summarises the classification of roughness with land use.

Table 7.4.2 – Two-dimensional Grid Roughness

Land Description	Manning's 'n'
Light Vegetation	0.04
Medium Vegetation	0.06
Dense Vegetation	0.16
Grassed Areas	0.03
Built up areas	0.04
2D creek areas	0.03
Bay	0.02
Roads	0.015
2D creek (above banks	0.01

7.5 Model Boundaries

The model boundaries are located at the model extremities. The upstream boundaries are defined as discharge boundaries, which are applied directly to the 1D branches of the model. The downstream boundaries are located on Lake Macquarie and are constant water level boundaries adopted from the Lake Macquarie Flood Study (MHL, 1998). The development of the downstream boundary levels is discussed in more detail in Section 6.

7.6 Model Calibration

7.6.1 General

The hydraulic model was calibrated to the February 1981 storm event. A large data set of historic flood levels was available for this purpose. In addition, substantial anecdotal evidence was also available.

The floodwaters overtopped the Northern Railway Line with a depth of flooding noted by the driver of a passing train at 2am to be approximately 300mm (SKM, 1981). At the time, the access road behind the rail embankment was considerably higher than its present position. The minimum level on the access road was 6.95m AHD, while the minimum level of the rail embankment was 7.5m AHD (SKM, 1981). The culverts under this access road were relatively small, which resulted in water overtopping this access road before it could flow through the culverts under the railway embankment. A significant storage behind this embankment was taken up before water could overtop the access roads.

An observation was made by the residents regarding the sudden rise in floodwater, which was described as a “wall of water” travelling down the creek. The residents suspected that the Northern Railway embankment had failed which caused a sudden surge of water. However no evidence could be obtained from the available information that suggested such an occurrence. The only observation available about the embankment is that it was overtopped and a sag was created due to erosion of the top layer of the embankment with a maximum erosion depth of 0.6m. Based on debris levels at the time, the maximum water level overtopping the railway was approximately 8.3 m AHD (SKM, 1981).

A large number of the observations for the calibration were in the lower portions of the catchment. In particular, around Fennell Avenue and near the Toronto Workers Club. Many of the residents on Fennell Avenue were reportedly forced onto their rooves, as the floodwaters rose rapidly. Patrons of the Toronto Workers Club, which closed around 1am, were forced back into the Club as the floodwaters entered the carpark. Many of the vehicles in the carpark were reportedly submerged.

7.6.2 Model Set-up

The topographic details of the catchment at the time of the flood event were not available. The current topography was used as the best available estimate of the topography during 1981.

Land use in the catchment was derived from Belmont U5442-4, Awaba U4542-6, Awaba U4542-6 (1987) orthophoto maps, which was obtained from the Department of Lands. This information was used as a best estimate of the 1981 conditions to define the flood plain and channel roughness at that time.

Details of the Northern Railway Bridge and service road were derived from the SKM (1981) flood study, although complete details were not available and reliance was made on the current topography in the areas surrounding the railway.

The coal haul road, downstream of the railway embankment, was not constructed until after the 1981 calibration event. The culverts and the embankment of the coal haul road were removed from the model by assuming a linear gradient between the levels on each side of the embankment.

The downstream boundary level at Fennell Bay was established from the Lake Macquarie Flood Study (MHL, 1998). This report includes recorded flood levels in the bay during 1981. However, there are no specific dates attached to this data. The three flood levels observed in 1981 are 0.79, 0.81 and 1.05 m AHD.

A constant water level of 0 m AHD was assumed in the main body of Lake Macquarie for the modelling purposes. Note that this is downstream of Fennell Bay Bridge, and higher water levels are observed in Fennell Bay during the model calibration. A sensitivity of this water level was tested as a part of the model calibration process.

7.6.3 Calibration Results

The model was calibrated by modifying the following parameters.

- Inflow Hydrographs
- Catchment / Channel Roughness

The inflow hydrographs were modified by changing the model parameters in the hydrological model. The initial peak flow estimates were too high, which were reduced primarily by increasing the sub-catchment lag time and adjusting the initial and continuing losses.

The initial and continuing losses were adjusted based on the SKM flood study (SKM, 1981) calibration with the Jigadee Creek Catchment, which is adjacent to the Stony Creek Catchment and has a stream gauging station. The losses estimated by SKM (1981) that would have resulted in the discharge hydrographs observed at this stream gauge are extraordinarily high (the estimated rainfall over this catchment was 320mm, the estimated hydrography volume was 180mm). The estimates of losses are excessive and is likely due to the fact that the February 1981 storm was well in excess of a 200 year ARI storm event, which may have exceeded the useful range of the stream gauging location.

Nonetheless, the estimates from this neighbouring catchment indicate that the losses may have been relatively high, perhaps due to a relatively long period of no rainfall in the preceding year (SKM, 1981). The following table shows the assumed losses during the calibration storm. Note that the majority of the pervious area within the catchment is forested or rural.

Table 7.6.1 Calibrated Losses

	Initial Loss (mm)	Continuing Loss (mm/hr)
Impervious Area	2	1
Pervious Area	50	5

The calibration inflow hydrograph at the Northern Railway is provided in Figure 7.4.

The catchment and channel roughness were assessed from the (1987) orthophoto maps. However, the current catchment condition roughness values were used as a best estimate of the roughness during the 1981 storm.

The results of model calibration are presented in the Table 7.6.2 and Table 7.6.3.

Table 7.6.2 provides calibration results for the recorded flood levels whereas Table 7.6.3 provides calibration results for the observed flood behaviour in the catchment.

Table 7.6.2 Comparison of Recorded and Modelled Flood Levels within the Catchment

Location	Observed Level (m AHD)	Modelled Level (m AHD)	Difference (m)
42 Fennell Crescent	2.78	2.81	0.03
48 Fennell Crescent	2.74	2.73	-0.01
50 Fennell Crescent	2.74	2.73	-0.01
46 Fennell Crescent	2.78	2.78	0.00
44 Fennell Crescent	2.78	2.79	0.01
40 Fennell Crescent	2.86	2.81	-0.05
58 Fennell Crescent	2.68	2.72	0.04
68 Fennell Crescent	2.68	2.69	0.01
52 Fennell Crescent	2.68	2.76	0.08
51 Fennell Crescent	2.65	2.75	0.10
61 Fennell Crescent	2.7	2.70	0.00
53 Fennell Crescent	2.7	2.72	0.02
62 Fennell Crescent	2.68	2.69	0.01
60 Fennell Crescent	2.68	2.70	0.02
68 Fennell Crescent	2.68	2.68	0.00
52 Fennell Crescent	2.68	2.74	0.06
79 Fennell Crescent	2.5	2.62	0.12
67 Fennell Crescent	2.5	2.71	0.21
86A Fennell Crescent	2.48	2.61	0.13
82 Fennell Crescent	2.48	2.63	0.15
74 Fennell Crescent	2.5	2.66	0.16
70 Fennell Crescent	2.5	2.66	0.16
80 Fennell Crescent	2.5	2.65	0.15
86 Fennell Crescent	2.48	2.60	0.12
78 Fennell Crescent	2.5	2.65	0.15
81 Fennell Crescent	2.5	2.62	0.12
87 Fennell Crescent	2.48	2.61	0.13
89 Fennell Crescent	2.48	2.61	0.13
77 Fennell Crescent	2.5	2.64	0.14
75 Fennell Crescent	2.5	2.64	0.14
85 Fennell Crescent	2.48	2.60	0.12
76 Fennell Crescent	2.5	2.65	0.15
72 Fennell Crescent	2.5	2.66	0.16
84 Fennell Crescent	2.48	2.62	0.14
67 Fennell Crescent	2.5	2.70	0.20
69 Fennell Crescent	2.53	2.69	0.16
83 Fennell Crescent	2.48	2.62	0.14
88 FennellCrescent	2.48	2.58	0.10
79 Railway Parade	2.32	2.62	0.30
81 Railway Parade	2.32	2.56	0.24

Table 7.6.3 Comparison of Recorded and Modelled Observations within the Catchment

Location	Description	Observation	Model Observation
A	Railway at upstream of model	A train driver went across the tracks at 2:05am and reportedly went through 300mm of water (SKM, 1981). Lowest sag of rail = 7.50m AHD. Level observed is therefore approximately 7.8m AHD.	The model shows 8.03m AHD at 2:05am
A	Railway at upstream of model	Flood debris level observed after the flood (SKM, 1981) approximately 8.3m AHD	The peak flood level is 8.34m AHD, at about 3:40am
B	Fennell Street	SKM report (1981) indicates peak in this area around 4am. Newspaper report says people climbed up on their roofs around 3am. Therefore, peak occurs probably around 3 to 4 am	The peak occurs at about 4:20am
C	Wastewater Treatment Plant	Newspaper reports say that the treatment ponds were overtopped, releasing wastewater into floodwaters	The treatment ponds are overtopped, but only marginally. Note that embankments may have changed since 1981. Local catchments flows may also have affected the Treatment ponds
D	Toronto Workers Club	Newspaper reports that club closed around 1am, and that the carpark was beginning to fill a little after this	Carpark begins to fill around 2:00am, and is completely filled by 2:30am
D	Toronto Workers Club	Newspaper interviews say that cars in the carpark were completely under water at some stage during the flood. Level of carpark is about 1.3m AHD	Depth of around 1.0m
E	2 Farrell Avenue	Resident observed approximately 3ft of water inside of house at 3am	Floor level is 1.86m AHD in 2003. Flood level is 2.61m AHD. Water level in model reaches 1.86m AHD around 2:15am.
F	18 Adam Street	Water up to knees inside lounge room, and water covering fences.	Floor level is 2.29m AHD in 2003. Flood level is 2.81m AHD.
G	Toronto High School	Many of the demountables at the school were flooded and ground floor of D Block. Houses in adjacent streets near school also flooded.	All of these areas are inundated in the model.

Location	Description	Observation	Model Observation
H	64 Railway Parade North	Property surrounded by water, and bottom of block inundated.	Lower portion of the property inundated up to 0.75m depth. Property is effectively "surrounded by water" from three sides. Water does not reach house.
I	226 Awaba Road	Water entered houses.	No water shown in this location, most likely local catchment flows.
J	72 Thorne Street	Cook St, James St, Workers Club all flooded. Netball and Cricket pitch flooded but did not enter this property	Model shows some water in Thorne street, but not on property. Cook St, James St, Workers Club and the Netball & Cricket Pitch all inundated.
K	6 Adam Street	About 3 ft of water inside house	Floor level is 1.85m AHD in 2003. Flood level is 2.75m AHD.
L	James Street	Had a foot of water come through the house	Difficult without a definite location. Water level ranges from 1.1m depth to zero along James Street, near the residential dwellings.
M	66 Railway Parade North	Flooding of backyard	Backyard is flooded. Water does not reach house.
N	322 Awaba Road	Approximately 3ft of water in front yard	No water shown in front yard. Approximately 0.5m depth of water on opposite side of street.
O	3 Galbraith Avenue	Water entered house.	Floor level is 2.20m AHD in 2003. Flood level is 2.61m AHD.
P	9 William Street	Water to height of front verandah below doorway. Also car flooded in carport.	Floor level is 2.60m AHD in 2003. Flood level is 2.60m AHD. Therefore, just below floor level. Garage would be inundated.
Q	Railway crossing of Stony Creek, at downstream of model	Photo shows flooding at some stage during the daytime, but the caption indicates the flood level. Uncertain as to whether this represents the peak flood level. Level reported is 1.99m AHD.	Modelled level of 2.24m AHD.
R	Intersection of James and Cook Street	Photo shows the flooding near intersection of James and Cook Street. Reported flood level is 2.5m AHD.	Modelled level of 2.53m AHD.

Location	Description	Observation	Model Observation
S	Thorne and Cook St	Photo taken near intersection of Thorne and Cook St. Reported Flood level is 2.5m AHD.	Modelled level of 2.54m AHD.

Note: Refer to Figure 7.5 for the location of the flood observations

The above results have been discussed in Section 13.

8. DESIGN FLOOD ESTIMATION

Design flooding behaviour was modelled for the catchment state as at July 2003, the date of photography for the aerial survey. Note, however, that the majority of the cross sectional survey dates back further from July 2003. Design inflow hydrographs were obtained from the RAFTS hydrologic model and applied to the SOBEK 1D-2D hydraulic model, which represents the study area. A range of hydrographs representing different storm durations was applied to the model in order to estimate critical flood levels for different areas of the floodplain.

A peak water level envelope was developed based on two downstream boundary scenarios as discussed in Section 6.

8.1 Results

The model results for all the durations of a design event were compiled into a single result list for each ARI. The reported results are the envelope of results for all durations for each ARI. A summary of peak water levels and critical duration is presented in Appendix F for all design events. Cross section results are also presented in Figures 8.12 to 8.14 for the PMF, 100 year ARI and 20 year ARI design storm events. The 2D flow reporting locations for these results are shown in Figure 8.1.

Flood extents for each ARI have been developed and are presented in Figures 8.2 to 8.8. These figures also include peak water levels at significant locations in the catchment.

Model results for the design events at significant locations in the floodplain are summarised in Table 8.1.1. These locations are shown in Figure 8.9.

Table 8.1.1 : Design Peak Water Levels

Location ID	Location Details	Peak Water Level (m AHD)					
		PMF	200year ARI	100 year ARI	20 year ARI	10 year ARI	5 year ARI
A	Railway Parade (Mudd Creek)	3.61	1.99	1.88	1.83	1.71	N/A
B	Railway Parade (Stony Creek)	3.69	N/A	N/A	N/A	N/A	N/A
C	Fennell Crescent	3.91	2.19	2.08	1.95	1.81	1.58
D	James Street (Toronto Workers Club)	3.83	2.1	1.98	1.85	1.69	1.44
E	Intersection of Farrell Avenue & Galbraith Avenue	3.86	2.16	2.05	1.98	1.98	N/A
I	Upstream of the Main Northern Railway	5.48	3.39	3.28	3.13	2.98	2.82
J	Intersection of Fennell Crescent & Adam Street	11.77	8.35	8.22	8.08	7.94	7.8
K	End of Lake Road	4.31	2.31	2.19	2.05	1.92	1.75
L	Intersection of Sara Street & Day Street	2.64	1.57	1.4	1.26	1.01	0.86
H	Intersection of High Street & Nicholson Street	2.87	1.6	1.45	1.34	1.15	1.05

The results are also provided as longitudinal profiles in Figures 8.10 and 8.11 of the major creek/channel in the floodplain. The flood profiles are provided for:

- Stony Creek
- Mudd Creek.

9. SENSITIVITY ANALYSIS

The sensitivity of the hydraulic model was tested to demonstrate the range of uncertainty in the model results for the 100 year ARI 36 hour design event, for the low downstream boundary conditions (Section 6), except for the downstream boundary sensitivity, where elevated downstream boundary was assumed. The following variables were tested for sensitivity:

- Catchment runoff - increased/decreased by 20%
- Channel roughness - increased/decreased by 20%
- Downstream boundary - increased/decreased by 20%
- Culvert Blockage – 100% blocked.

The culvert blockage scenario was applied to all culverts modelled within the catchment.

The sensitivity results are presented in Figures 9.1 to 9.8 as profiles. Appendix G summarises the model sensitivity results compared to the base conditions at significant locations in the catchment.

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. The hazard caused by a flood varies both in time and place across the floodplain. The Floodplain Management Manual (NSW Government, 2001) 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 (Appendix G, NSW Government, 2001). The Floodplain Management Manual (2001) 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 200, 100, 50, 20, 10 and 5 year ARI floods and the PMF is presented in Figures 10.1 to 10.7 as an extent. 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 ARI's presented.

11. HYDRAULIC CATEGORISATION

11.1 General

Hydraulic categorisation of the floodplain is used in the development of the Floodplain Risk Management Plan. The Floodplain Management Manual (2001) defines flood prone 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 200, 100, 50, 20, 10 and 5 year ARI 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.

Previous analysis of flood storage in 1D cross sections 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 (Howells et al, 2003). Thus the criteria used to determine the flood storage is:

- Depth greater then 0.2m
- Not classified as floodway.

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

The hydraulic categories for 200, 100, 50, 20, 10 and 5 year ARI and PMF are provided as plans in Figures 11.1 to 11.7. 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 ARI's presented.

12. ANNUAL AVERAGE DAMAGE

12.1 Background

The economic impact of flooding can be defined by what is commonly referred to as 'flood damages'. Table 12.1 provides classifications of various types of flood damages incurred in a catchment. Direct damage costs are just one component of the entire cost of a flood event. There are also indirect costs. Both direct and indirect costs are referred to as 'tangible' costs. In addition to this there are also 'intangible' costs. The values discussed in this report are the 'total' damages and include an assumed intangible cost of 25% of the tangible cost.

Table 12.1 Types of Flood Damages

Direct	Building contents (internal) Structural (building repair and clean) External items (vehicles, contents of sheds etc)
Indirect	Clean-up (immediate removal of debris) Financial (loss of revenue, extra expenditure) Opportunity (non-provision of public services)
Intangible	Social – increased levels of insecurity, depression, stress General inconvenience in post-flood stage

Flood damages can be assessed by a number of means including the use of programs such as FLDAMAGE or ANUFLOOD or via more generic methods using spreadsheets. For the purposes of this project, generic spreadsheets have been used with assistance from DIPNR on the adoption of damage curves.

12.2 Floor Level and Property Survey

Floor level and property survey was provided by the Lake Macquarie City Council. The survey included details of each property within the known extent of the floodplain at the time of the survey. The property details included a floor level and property type. A representative ground level was estimated from the available ground survey.

A number of categories of property were identified within the floodplain including:

- Residential
- Commercial
- Industrial

The property survey provided by the Council did not have complete information for flood damages assessment and as such assumptions were made as discussed in the following sections.

12.3 Stage - Damage Curves

There are currently no strict guidelines regarding the adoption of damage curves in NSW. DIPNR have recently created a draft methodology for the creation of damage curves (2004), but this does not cover industrial or commercial properties.

Consequently, an approach, using the Draft DIPNR methodology for residential areas with a combination of other published approaches for commercial and industrial areas was used for the assessment.

12.3.1 Residential Damage Curves

The draft DIPNR Floodplain Management Guideline No. 4 *Residential Flood Damage Calculation* (2004) was used for this study. This guideline includes a template spreadsheet program that determines damage curves for three types of residential buildings:

- Single Storey, slab on ground (floor level assumed to be 0.5m above the ground)
- Two Storey, slab on ground (floor level assumed to be 0.5m above the ground)
- Single Storey, ‘high-set’ eg piered structures (floor level assumed to be 1.5m above the ground)

Due to lack of details in the property survey, all properties were assumed to be single slab on ground and the floor level was assumed to be 0.5 above the ground.

The DIPNR curves are derived for late 2001 (base curves). General recommendations by DIPNR are to adjust values in the base residential damage curves by Average Weekly Earnings (AWE), rather than by the inflation rate as measured by the Consumer Price Index (CPI). While not specified, we have assumed that the base curves were derived in November 2001, which allows the use of November 2001 AWE statistics (issued quarterly). November 2001 AWE are shown in Table 12.2. The most recent data for AWE from the Australian Bureau of Statistics at the time of assessment was for May 2004. AWE values were sourced from the Australian Bureau of Statistics (ABS, 2004).

Table 12.2 AWE Statistics from 2001 and 2004

Month	Year	AWE
November	2001	$898.50
May	2004	$997.70
Change	11.04%	

All ordinates in the base residential flood damage curves were therefore converted into May 2004 dollars. In addition, DIPNR recommends that all damage curves include GST and as such GST was included. Consequently, all ordinates on the damage curves are increased by 11.04% and GST has been added.

Damages are generally incurred on a property prior to any over floor flooding. There are two possibilities:

- The flooding overtops the garden but does not necessarily reach the main structure. For these type of properties, the ground survey data collected (Section 12.2) includes a representative ground level for the property. When this representative ground level is exceeded, a nominal flat value of $3,000 (May 2004 dollars) was assigned to represent garden damage.
- The flooding overtops the garden and also reaches the structure. The DIPNR curves allow for a damage of $7,437 (May 2004 dollars) to be incurred when the water level reaches the base of the house (the base of the house is determined by 0.5m below the floor level for slab on ground and 1.5m below the floor level for 'high-set'). This accounts for some garden and structural damage, and includes some damage to cars.

The approach adopted was to use a cost of $3,000 (May 2004 dollars) when only the ground level of the property is overtopped. When the flooding reaches the base of the house, the DIPNR curves, with $7,437 (May 2004 dollars) of external damage (i.e. an additional $4,437 over the garden damage) was adopted.

There are a number of input parameters required for the DIPNR curves, such as floor area and level of flood awareness. The following parameters were adopted:

- Based on property level survey information, the average residential floor area in Stony Creek catchment is approximately 140m^2. A value of 150m^2 was adopted as a conservative estimate of the floor area for residential dwellings for the floodplain. With a floor area of 150m^2, the default contents value is $37,500.
- The Effective Warning Time has been assumed to be zero due to the absence of any flood warning systems in the catchment. A long Effective Warning Time allows residents to prepare for flooding by moving valuable household contents (e.g. the placement of valuables on top of tables and benches).
- Stony Creek catchment is a small part of the regional centres and as such is not likely to cause any post flood inflation. These inflation costs are generally experienced in regional areas, where re-construction resources are limited and large floods can cause a strain on these resources.

The adopted residential damage curves are shown in Figure 12.1.

12.3.2 Commercial Damage Curves

Commercial damage curves were determined based on those included in the FLDamage Manual (Water Studies Pty Ltd, 1992). FLDamage allows for three types of commercial properties:

- Low Value Commercial
- Medium Value Commercial
- High Value Commercial.

The FLDamage curves have a base date of 1990. The Consumer Price Index (CPI) was used to adjust the 1990 data to June 2004 dollars (this data was obtained from the Australian Bureau of Statistics website (ABS, 2004). It was assumed that the FLDamage data was in June 1990 dollars. The CPI data is shown in Table 12.3.

Table 12.3 CPI Statistics from 1990 and 2004

Month	Year	CPI
June	1990	102.50
June	2004	144.80
Change	41.27%	

Consequently, ordinates on the 1990 damage curves have been increased by 41.27% and GST has been included.

In determining the ordinates on the damage curves, it has been assumed that the effective warning time is approximately zero, and the loss of trading days as a result of the flooding has been taken as 10.

The curves are determined based on the floor area of the property. As this information was not available, floor areas of the commercial properties were estimated from aerial photograph. An example of the curve for a property with a floor area of 100m^2 is provided in Figure 12.1.

12.3.3 Industrial Damage Curves

Cardno Lawson Treloar, as a part of the Allans Creek Floodplain Risk Management Study (Lawson and Treloar, 2004) conducted a survey of industrial properties in 1998 for Wollongong City Council. The damage curves derived from this survey were modified for use for this study. These are broken into four categories:

- Low Value Industrial
- Medium Value Industrial
- High Value Industrial (e.g. BHP steelworks in Wollongong).

Within the catchment, there are no properties considered to be representative of ‘high value’ industrial properties, and hence these curves were not used.

The floor areas for the industrial properties were estimated from aerial photograph. To normalise the damages for property size, the curves have been factored to account for floor area, with Figure 12.1 showing the industrial damage curves for a nominal floor area of 100m^2.

The survey conducted only accounts for structural and contents damage to the property. Clean up costs and indirect financial costs were estimated based on FLDamage Manual (Water Studies, 1992). Actual internal damage was estimated, along with potential internal damage, using various factors within FLDamage. Using both the actual and potential internal damages, estimation of both the clean up costs and indirect financial costs were made.

The values were adjusted to June 2004 dollars using the CPI statistics shown in Table 12.4.

Table 12.4 CPI Statistics from 1998 and 2004

Month	Year	CPI
June	1998	121.00
June	2004	144.80
Change	19.67%	

Consequently, all ordinates on the damage curves were adjusted by 19.67% and GST was added.

12.4 Results

Table 12.5 shows the results of the flood damage assessments. Based on the analysis, the average annual damage for the floodplain under existing conditions is approximately $205,716.

Table 12.5 Flood Damage Assessment Summary

Event/Property Type	Number of Properties with overfloor flooding	Average Overfloor Flooding Depth (m)	Maximum Overfloor Flooding Depth (m)	Number of Properties with overground flooding	Total Damage ($June 2004)
PMF					
Residential	**247**	**1.18**	**2.56**	**262**	**$11,664,251**
Commercial	**11**	**1.73**	**2.66**	**12**	**$ 2,024,725**
Industrial	**37**	**1.98**	**2.81**	**37**	**$ 5,813,436**
PMF Total	**295**			**311**	**$19,502,411**
200 year ARI					
Residential	**66**	**0.68**	**1.18**	**177**	**$ 3,018,408**
Commercial	**3**	**0.51**	**0.52**	**8**	**$ 92,541**
Industrial	**18**	**0.85**	**4.06**	**30**	**$ 648,391**
200 Year ARI Total	**87**			**215**	**$ 3,759,340**
100 year ARI					
Residential	**41**	**0.19**	**0.66**	**165**	**$ 2,233,596**
Commercial	**3**	**0.26**	**0.30**	**6**	**$ 66,426**
Industrial	**13**	**0.15**	**0.67**	**26**	**$ 346,163**
100 Year ARI Total	**57**			**197**	**$ 2,646,186**
50 year ARI					
Residential	**20**	**0.18**	**0.52**	**147**	**$ 1,491,613**
Commercial	**3**	**0.15**	**0.20**	**6**	**$ 27,322**
Industrial	**6**	**0.12**	**0.42**	**21**	**$ 72,856**
50 Year ARI Total	**29**			**174**	**$ 1,591,791**

Event/Property Type	Number of Properties with overfloor flooding	Average Overfloor Flooding Depth (m)	Maximum Overfloor Flooding Depth (m)	Number of Properties with overground flooding	Total Damage ($June 2004)
20 year ARI					
Residential	**7**	**0.21**	**0.37**	**98**	**$ 798,043**
Commercial	**2**	**0.08**	**0.10**	**5**	**$ 16,786**
Industrial	**1**	**0.09**	**0.09**	**3**	**$ 4,075**
20 Year ARI Total	**10**			**106**	**$ 818,904**
10 year ARI					
Residential	**3**	**0.12**	**0.16**	**59**	**$ 426,435**
Commercial	**0**	**-**	**-**	**2**	**$ -**
Industrial	**1**	**0.04**	**0.04**	**2**	**$ 2,533**
10 Year ARI Total	**4**			**63**	**$ 428,968**
5 year ARI					
Residential	**0**	**-**	**-**	**25**	**$ 143,683**
Commercial	**0**	**-**	**-**	**1**	**$ -**
Industrial	**0**	**-**	**-**	**1**	**$ -**
5 Year ARI Total	**0**			**27**	**$ 143,683**
February 1981 Event					
Residential	**89**	**0.50**	**1.23**	**212**	**$ 4,172,487**
Commercial	**9**	**0.53**	**1.30**	**11**	**$ 1,132,280**
Industrial	**37**	**0.55**	**1.18**	**37**	**$ 2,464,812**
February 1981 Total	**135**			**260**	**$ 7,769,579**

13. DISCUSSION OF RESULTS

A flood study for the Stony Creek catchment was undertaken for the existing catchment conditions (July 2003). Flooding behaviour of the catchment was investigated using hydrologic and hydraulic computer models. The floodplain was modelled using a dynamically linked one-dimensional/two-dimensional hydraulic model, SOBEK 1D/2D. The one-dimensional component of the model represented the major creek line and drainage channels, while the two dimensional component of the model was used to represent the overland flow. Hydraulic structures such as culverts were included in the one-dimensional component of the model, while weirs were modelled in the 2D component of the model. The model was calibrated to a single historic event.

Inflows to the hydraulic model were developed using the RAFTS hydrologic model. Significant catchment parameters such as slope, roughness and imperviousness were accounted for in the model. The model was indirectly verified through hydraulic model calibration.

13.1 Model Calibration

The hydraulic model was calibrated to the February 1981 event. The calibration results indicate a reasonable match of historic and modelled flood levels. The model was modified to represent the catchment conditions at the time of the event. However, detailed topographic/culvert/bridge data was not available and as such existing topography and structure data was relied upon in the model set-up.

The calibration results are presented in Section 7 and shown in Figure 13.1 and 13.2. In the upper parts of the catchment, calibration data was only available at the Northern Railway. The model was calibrated well at that location. In the lower parts of the catchment, the observed flood levels are along Fennell Crescent. Four distinct areas can be identified in terms of reported flood levels. Description of calibrated results for each of these areas is provided below:

13.1.1 Area A

In area A (Figure 13.1), which is north of the Fennell Crescent crossing of Mudd Creek, the observed flood levels vary from 2.86m AHD to 2.74m AHD. The modelled levels are within 0.03m of the observed levels except at 40 Fennell Crescent, where the difference is 0.05m. The comparison for this area is presented in Table 13.1.1 below:

Table 13.1.1: Model Calibration – Area A

Location	Observed Flood Level (m AHD)	Modelled Flood Level (m AHD)	Difference (m)
42 Fennell Crescent	2.78	2.81	0.03
48 Fennell Crescent	2.74	2.73	-0.01
50 Fennell Crescent	2.74	2.73	-0.01
46 Fennell Crescent	2.78	2.78	0.00
44 Fennell Crescent	2.78	2.79	0.01
40 Fennell Crescent	2.86	2.81	-0.05

Hence model calibration for this area is satisfactory.

13.1.2 Area B

In area B, which lies downstream of area A, the observed flood level is a constant level of approximately 2.68m AHD. The observed flood levels show a sudden drop of approximately 0.1m from area A to area B, across Mudd Creek. The modelled levels vary from 2.75m AHD to 2.68m AHD. Thus model results show a smooth gradient from area A to B whereas the observed levels show a discontinuity at Mudd Creek. This drop in observed levels can only be explained by the presence of a hydraulic control, for which no information was available.

A comparison of the modelled and observed flood levels is provided in Table 13.1.2 below:

Table 13.1.2: Model Calibration – Area B

Location	Observed Flood Level (m AHD)	Modelled Flood Level (m AHD)	Difference (m)
58 Fennell Crescent	2.68	2.72	0.04
68 Fennell Crescent	2.68	2.69	0.01
52 Fennell Crescent	2.68	2.76	0.08
51 Fennell Crescent	2.65	2.75	0.10
61 Fennell Crescent	2.70	2.70	0.00
53 Fennell Crescent	2.70	2.72	0.02
62 Fennell Crescent	2.68	2.69	0.01
60 Fennell Crescent	2.68	2.70	0.02
68 Fennell Crescent	2.68	2.68	0.00
52 Fennell Crescent	2.68	2.74	0.06

For the majority of the levels, the difference is within 0.03m, with a maximum difference of 0.1m at 51 Fennell Crescent. As discussed above this is due to a sudden drop of observed flood levels across the Mudd Creek.

13.1.3 Area C

In this area the observed flood levels have a constant value around 2.5 m AHD. Thus there is a 0.18m drop from the adjacent area B. This sudden drop can not be explained and can only be attributed to the observation errors. A comparison of the modelled and observed flood levels is provide in Table 13.1.3 below:

Table 13.1.3: Model Calibration – Area C

Location	Observed Flood Level (m AHD)	Modelled Flood Level (m AHD)	Difference (m)
79 Fennell Crescent	2.5	2.62	0.12
67 Fennell Crescent	2.5	2.71	0.21
86A Fennell Crescent	2.48	2.61	0.13
82 Fennell Crescent	2.48	2.63	0.15
74 Fennell Crescent	2.5	2.66	0.16
70 Fennell Crescent	2.5	2.66	0.16
80 Fennell Crescent	2.5	2.65	0.15
86 Fennell Crescent	2.48	2.60	0.12
78 Fennell Crescent	2.5	2.65	0.15
81 Fennell Crescent	2.5	2.62	0.12
87 Fennell Crescent	2.48	2.61	0.13
89 Fennell Crescent	2.48	2.61	0.13
77 Fennell Crescent	2.5	2.64	0.14
75 Fennell Crescent	2.5	2.64	0.14
85 Fennell Crescent	2.48	2.60	0.12
76 Fennell Crescent	2.5	2.65	0.15
72 Fennell Crescent	2.5	2.66	0.16
84 Fennell Crescent	2.48	2.62	0.14
67 Fennell Crescent	2.5	2.70	0.20
69 Fennell Crescent	2.53	2.69	0.16
83 Fennell Crescent	2.48	2.62	0.14
88 Fennell Crescent	2.48	2.58	0.10

The difference varies from 0.21m at 67 Fennell Crescent to 0.1m at 88 Fennell Crescent. The model is therefore not well calibrated in this region. As discussed above the likely reason is the error in the observed data. Another reason could be the hydraulic controls on Mudd Creek at Railway Parade, which are significant controls under the existing conditions. However if the topography of the area in 1981 was such that these controls were less significant, the modelled flood levels would drop in the Fennell Crescent. Unfortunately, no information was available for the 1981 topography and it is likely that the model is not representative of the floodplain conditions at the time of the flood.

13.1.4 Area D

The observed flood level in this area is 2.32m AHD, which shows a drop of 0.18m from the adjacent area C. A comparison of the observed and modelled flood levels is provided in Table 13.1.4.

Table 13.1.4: Model Calibration – Area D

Location	Observed Flood Level (m AHD)	Modelled Flood Level (m AHD)	Difference (m)
79 Railway Parade	2.32	2.62	0.30
81 Railway Parade	2.32	2.56	0.24

The difference in the observed and modelled flood levels is significant and is likely due to the hydraulic controls at Railway Parade, that were either not present or were present to a lesser degree in 1981.

Model parameters were modified within a reasonable range to achieve calibration in areas C and D. However, the model could only be calibrated to the accuracy provided in the above tables. The model parameters were not unreasonably modified to achieve calibration.

13.1.5 Other Observations within the Catchment

Other observations of the February 1981 event were also collated. The comparison of the model results with the observed flood behaviour is provided in Section 7. Some of these observations were sourced from the resident survey, while others were gained from newspaper reports and the a previous report (SKM, 1981).

The results indicate that the modelled results closely match the observed flood behaviour. The important observation of “wall of water” in the creek is substantiated by the model results that show an approximate 1m rise in the flood levels in a 15 minute period above the High Street Ford, near Ada Street. Figure 13.2 provides the water level time series at that location.

13.1.6 Downstream Boundary Sensitivity

The calibration model was analysed assuming a downstream boundary (downstream of Fennell Bay Bridge) of 0m AHD. This results in a level within Fennell Bay (near the outlet of Stony Creek) of 1.05m AHD in the model. This is similar to one of the observed peak flood levels in Fennell Bay (MHL, 1998).

To further test the sensitivity of the calibration results to the downstream boundary, a downstream boundary of 1.05m AHD was used downstream of Fennell Bay Bridge. This level results in an increase in the level at the outlet of Stony Creek, with the new level at 1.38m AHD. However, the impact on the calibration results is small. Table 13.1.6 shows a comparison of the results from the two different models for observed flood levels at the Fennell Crescent properties. The difference in this location due to the increased downstream boundary is small.

Table 13.1.6 Calibration downstream sensitivity

Location	Modelled Flood Level (m AHD) for ds=0m AHD	Modelled Flood Level (m AHD) for ds=1.05m AHD	Difference (m)
42 Fennell Crescent	2.81	2.82	0.01
48 Fennell Crescent	2.73	2.73	0.00
50 Fennell Crescent	2.73	2.74	0.01
46 Fennell Crescent	2.78	2.79	0.01
44 Fennell Crescent	2.79	2.79	0.00
40 Fennell Crescent	2.81	2.82	0.01
58 Fennell Crescent	2.72	2.73	0.01
68 Fennell Crescent	2.69	2.70	0.01
52 Fennell Crescent	2.76	2.77	0.01
51 Fennell Crescent	2.75	2.76	0.01
61 Fennell Crescent	2.70	2.71	0.01
53 Fennell Crescent	2.72	2.73	0.01
62 Fennell Crescent	2.69	2.70	0.01
60 Fennell Crescent	2.70	2.71	0.01
68 Fennell Crescent	2.68	2.69	0.01
52 Fennell Crescent	2.74	2.75	0.01
79 Fennell Crescent	2.62	2.63	0.01
67 Fennell Crescent	2.71	2.71	0.00
86A Fennell Crescent	2.61	2.62	0.01
82 Fennell Crescent	2.63	2.64	0.01
74 Fennell Crescent	2.66	2.66	0.00
70 Fennell Crescent	2.66	2.67	0.01
80 Fennell Crescent	2.65	2.66	0.01
86 Fennell Crescent	2.60	2.61	0.01
78 Fennell Crescent	2.65	2.66	0.01
81 Fennell Crescent	2.62	2.63	0.01
87 Fennell Crescent	2.61	2.61	0.00
89 Fennell Crescent	2.61	2.61	0.00
77 Fennell Crescent	2.64	2.64	0.00
75 Fennell Crescent	2.64	2.65	0.01
85 Fennell Crescent	2.60	2.61	0.01
76 Fennell Crescent	2.65	2.66	0.01
72 Fennell Crescent	2.66	2.66	0.00
84 Fennell Crescent	2.62	2.63	0.01
67 Fennell Crescent	2.70	2.71	0.01
69 Fennell Crescent	2.69	2.69	0.00
83 Fennell Crescent	2.62	2.63	0.01

Location	Modelled Flood Level (m AHD) for ds=0m AHD	Modelled Flood Level (m AHD) for ds=1.05m AHD	Difference (m)
88 Fennell Crescent	2.58	2.59	0.01
79 Railway Parade	2.62	2.63	0.01
81 Railway Parade	2.56	2.57	0.01

ds= downstream boundary condition

13.1.7 Summary

Comparison at other locations shows that the model calibration is satisfactory and the overall flood behaviour corresponds to what has generally been reported in the area.

The calibration process established enough confidence in the model for the design flood modelling to be undertaken for the catchment.

13.2 Design Flood Estimation

The calibrated model was used to estimate design overland flow depths for the existing catchment and floodplain conditions. The storm durations of 4.5, 9 and 36 hours were generally found to be critical in the catchment, with 36 hours for majority of the overland flow affected area. For the PMF, the critical duration was generally 4 hours in the upper portion of the catchment, and 5 hours in the low lying areas.

In the upper catchment, the Northern Railway is overtopped during the 10 year ARI design event. The lowest level of the railway is 7.83m AHD, resulting in an overtopping depth of 0.14m during a 10 year ARI design event. During the 100 year ARI design event, the water level at this location reaches 8.31m AHD, which corresponds to an overtopping depth of 0.48m.

The main coal haul road, just downstream of the railway embankment, is only overtopped in the PMF design event. The level of this road is approximately 10.01m AHD at this location. The peak water level during the PMF at this location is 11.36 mAHD. This results in an overtopping depth of approximately 1.35 mAHD.

The Sewerage Treatment Plant is overtopped in the PMF event. The ponds are significantly inundated, with active flow proceeding through the treatment plant area. Overtopping is not observed in the 200 year ARI design event.

Flooding in the lower parts of the catchment, upstream of the Railway Parade bridges across Mudd Creek and Stony Creek, is affected by the general topographic constriction at this location. The majority of the water is forced through the area of these two bridges, which creates a constriction.

The lower parts of the catchment, from Fennell Bay to up to Railway Parade are primarily affected by elevated Lake Macquarie levels in addition to the catchment flooding. Two design flood conditions were determined; one dominated by the catchment flooding and the other by Lake flooding. The design flood levels were obtained from a peak water level envelope from the two flooding scenarios.

Railway Parade at Stony Creek is not overtopped up to the 200 year event. However, this road is overtopped at Mudd Creek starting with a 10 year event. It is worth noting that Railway Parade at Stony Creek was close to being overtopped in the 1981 event, which is approximately 0.59 m above the 100 year level at this location.

Design flood profiles of peak water levels for Stony Creek and Mudd creek are presented in Figures 8.8 and 8.10. Detailed model results are presented in Appendix F. Figures 8.2 to 8.8 indicate areas likely to be inundated by the PMF, 200 year, 100 year, 50 year, 20 year, 10 year and 5 year ARI storm events for the existing catchment and floodplain conditions.

13.3 Model Sensitivity

The uncertainty in the model results was tested by carrying out a sensitivity analysis (Section 9). The sensitivity of various parameters needs to be considered carefully because it defines the variability of model results. Sensitivity to catchment runoff, channel roughness, culvert blockage and downstream boundary was investigated for the 100 year and 5 year ARI events of 36hr duration for the existing catchment condition.

13.3.1 Catchment Runoff

The model shows a variation in water levels as a result of changes in the applied inflows to the model (Figures 9.1 and 9.2). The impacts of a decrease in flow appear to have a more significant impact than an increase in flow. A 20% increase in flow generally results in a maximum increase in water level of around 0.16m, while a 20% decrease in flow results in a maximum decrease in water level of around 0.40m in a 100 year ARI design event.

13.3.2 Channel/Flowpath Roughness

The impact of changes in model roughness are not as significant as changes to catchment runoff (Figures 9.3 and 9.4). A 20% increase in model roughness results in a maximum increase in water level of around 0.15m, while a 20% decrease in roughness results in maximum decrease in water level of around 0.14m in a 100 year ARI design event.

13.3.3 Downstream Boundary

Changes in the downstream boundary primarily affect the lower lying areas of the catchment (Figure 9.7 and 9.8). Note that the elevated downstream boundary condition was tested.

13.3.4 Culvert Blockage

The effect of culvert blockage is significant within catchment (Figure 9.5 and 9.6). The blockage of the upper catchment culverts results in a greater amount of water being detained upstream. The blockage of the two significant control points, the Railway Parade crossing of Mudd Creek and Stony Creek, results in significant impacts on water levels. The maximum increase in a 100 year ARI design event is around 0.67m near Railway Parade.

13.4 Provisional Hazard Definition

Hazard categories were defined for the design events of PMF, 100 year, 50 year, 20 year and 5 year ARI events. The definition of these categories was based on the guidelines provided in the Floodplain Management Manual (NSW Government, 2001).

Figures 10.1 to 10.7 provide extents of high and low hazard in the study area. In the lower parts of the catchment, the provisional high hazard extents are dominated by depth limited criteria in the lower portion of the catchment. This arises from the Lake Macquarie flood conditions.

13.5 Hydraulic Categorisation

Hydraulic Categories were defined for the PMF, 200 year, 100 year, 50 year, 20 year and 5 year ARI events. The definition of these categories was based on the guidelines provided in the Floodplain Management Manual (NSW Government, 2001) and Howells et al. (2003).

Figures 11.1 to 11.7 provide the extents of floodway, flood storage and flood fringe in the study area. Except for the PMF event, the majority of the catchment is dominated by flood storage. This is a reflection of the low lying nature of the lower half of the catchment.

13.6 Average Annual Damage

The economic impact of the flooding in Stony Creek catchment was assessed by estimating ‘flood damages’. These estimates were based on the latest advice from DIPNR. The total damages for the residential, commercial and industrial properties for various design events is summarised in Table 13.5.1.

Table 13.5.1: Flood Damages Summary

Design Event	Properties with above floor flooding	Total Damages
PMF	295	$19,502,411
200 year	87	$3,759,340
100 year	57	$2,646,186
50 year	29	$1,591,791
20 year	10	$818,904
10 year	4	$428,968
5 year	0	$143,683
Feb 1981	135	$7,769,579

The table also notes the number of properties with above floor flooding for various design flood events. Majority of these properties are residential in nature.

The Annual Average Damage (AAD) for the Stony Creek catchment is **$205,716**.

Since February 1981 event was a major event in the catchment, flood damages for this event in recent dollar terms have also been calculated. The damage estimates show that if the February 1981 event had occurred in recent times, the resulting flood damages would have been twice as much as the 200 year event and thrice as much as the 100 year event. This is a significant finding that would have a bearing on the Flood Planning Level determination in the later floodplain management studies.

14. REPORT QUALIFICATIONS

This report has been prepared for Lake Macquarie City Council to define the nature and extent of flooding for the study area in the Stony Creek catchment. The report defines the flooding behaviour for the entire floodplain of 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 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 buildings within the floodplain are assumed to completely block the floodwaters. Hence they do not have any active flowpaths nor they provide any storage for floodwaters.
- Design flood levels along the foreshore area of Edmunds Bay should be estimated from the Manly Hydraulics Lake Macquarie Flood Study (1998) that takes into account the flooding due to the wave runup processes within the lake.
- The property floor level data obtained from Council does not necessarily cover all properties inundated up to the PMF. This may impact on the flood damage calculations.

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

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APPENDIX A

Survey Data

APPENDIX B

Residents Questionnaire and Response

APPENDIX C

Historic Data

APPENDIX D

Sub Catchment Details

APPENDIX E

Model Cross Sections

APPENDIX F

Summary of Results

APPENDIX G

Model Sensitivity Results