SWAINES CREEK
FLOOD STUDY

VOLUME 1 – REPORT

FINAL REPORT

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FOREWORD

The State Government's Flood Policy is directed at providing solutions to existing flooding problems in developed areas and to 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 liable land remains the responsibility of local government. The State subsidises flood mitigation works to alleviate existing problems and provides specialist technical advice to assist councils in the discharge of their floodplain management responsibilities.

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

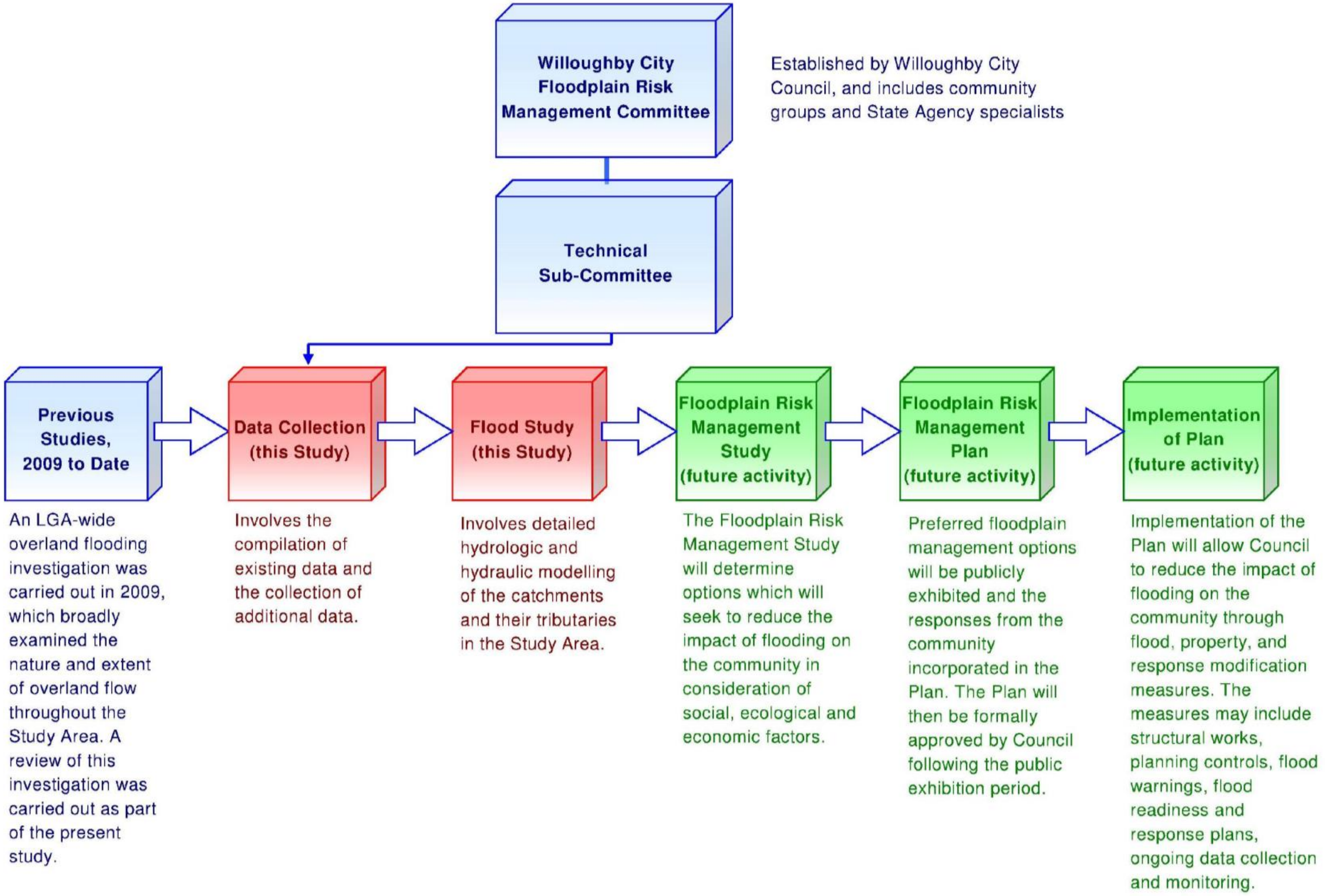
1.	Flood Study	Determines the nature and extent of flooding.
2.	Floodplain Risk Management Study	Evaluates management options for the floodplain in respect of both existing and proposed development.
3.	Floodplain Risk Management Plan	Involves formal adoption by Council of a plan of management for the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

The Swaines Creek Flood Study constitutes the first stage of the Floodplain Risk Management process (refer over) for this area and has been prepared for Willoughby City Council to define flood behaviour under current conditions.

ACKNOWLEDGEMENT

Willoughby City Council has prepared this document with financial assistance from the NSW Government through its Floodplain Management Program. This document does not necessarily represent the opinions of the NSW Government or the Office of Environment and Heritage.

FLOODPLAIN RISK MANAGEMENT PROCESS



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NOTE ON FLOOD FREQUENCY

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

ANNUAL EXCEEDANCE PROBABILITY (AEP) %	AVERAGE RECURRENCE INTERVAL (ARI) YEARS
0.2	500
0.5	200
1	100
2	50
5	20
10	10
20	5
50	2
100	1

In this report floods are referred to in terms of their ARI. Reference is also made in the report to the Probable Maximum Flood (PMF). This flood occurs as a result of the Probable Maximum Precipitation (PMP). The PMP is the result of the optimum combination of the available moisture in the atmosphere and the efficiency of the storm mechanism as regards rainfall production. The PMP is used to estimate PMF discharges using a model which simulates the conversion of rainfall to runoff. The PMF is defined as the limiting value of floods that could reasonably be expected to occur. It is an extremely rare flood, generally considered to have a return period greater than 1 in 10^5 years.

ABBREVIATIONS

AEP	Annual Exceedance Probability (%)
AHD	Australian Height Datum
ALS	Airborne Laser Scanning
AMC	Antecedent Moisture Condition
ARF	Areal Reduction Factor
ARI	Average Recurrence Interval (years)
ARR	Australian Rainfall and Runoff (IEAust, 1998)
BOM	Bureau of Meteorology
CL	Continuing Loss
DTM	Digital Terrain Model
FDM	Floodplain Development Manual (NSW Government, 2005)
FPA	Flood Planning Area
FPL	Flood Planning Level
FRMS	Floodplain Risk Management Study
HHWSS	Highest High Water Solstice Spring (tidal event)
IFD	Intensity-Frequency-Duration
IL	Initial Loss
LGA	Local Government Area
OEH	Office of Environment and Heritage, Department of Premier and Cabinet (formerly Department of Environment, Climate Change and Water [DECCW])
PMF	Probable Maximum Flood
PMP	Probable Maximum Precipitation
RCP	Reinforced Concrete Pipe
RL	Reduced Level
WCC	Willoughby City Council

Chapter 8 of the report contains definitions of flood-related terms used in the study.

S1 SUMMARY

The study objective was to define flood behaviour in the Swaines Creek catchment and along the adjacent reach of the Lane Cove River in terms of water levels, flows and flooding patterns for design floods ranging between 1 and 100 year Average Recurrence Interval (ARI), as well as for the Probable Maximum Flood (PMF). **Figure 1.1** shows the Swaines Creek catchment and its stormwater system. The flood study involved the following activities:

- The collection of flood related data. A Community Newsletter and Questionnaire introducing the study objectives and seeking information on historic flood behaviour was forwarded to residents in the Swaines Creek catchment. Respondents reported flooding problems dating back to the mid-1980s, with the event identified most frequently by residents being that of 10 April 1998. Rainfall data recorded by a pluviometer at Chatswood Bowling Club were used to test the flood models developed for the study by way of comparison with reported local catchment flooding patterns.

  Flood marks for historic flood events which occurred in November 1984 and August 1986 were also identified along the Lane Cove River. Rainfall data recorded at a number of locations across the Lane Cove River catchment were available for these events and used to test the ability of the flood models to reproduce historic flooding patterns.

- The hydrologic modelling of Swaines Creek and Lane Cove River catchments to determine discharge hydrographs. The hydrologic modelling was based on the RORB (for the Lane Cover River catchment) and DRAINS (for the Swaines Creek catchment) rainfall-runoff software. These software derived discharge hydrographs resulting from historic and design storms.
- Application of the discharge hydrographs to a hydraulic model of the Lane Cove River, the main arm of Swaines Creek, its major tributaries and overland flow paths. The hydraulic model extended from the headwaters of the Swaines Creek catchment near the Pacific Highway to its various outfalls into the Lane Cove River. The TUFLOW two-dimensional modelling system was adopted for the hydraulic analysis. Along the river, the model extended from approximately 800 m upstream of Fullers Bridge to a location approximately 2.2 km downstream of the Epping Road bridge.
- Presentation of study results as water surface profiles, as well as diagrams showing indicative extents and depths of inundation, provisional flood hazard and the hydraulic categorisation of the floodplain into floodway and flood fringe areas.
- Sensitivity studies to assess the effects on model results resulting from variations in model parameters such as hydraulic roughness of the floodplain, the effects of partial blockage of the piped drainage system, elevated tailwater levels in Sydney Harbour, and the effects on flooding patterns resulting from future climate change.

After testing the models for the historic floods, design storm rainfalls ranging between 1 and 100 year ARI were derived using procedures set out in *Australian Rainfall and Runoff* (IEAust, 1998) (ARR) and applied to the hydrologic models to determine discharge hydrographs. The PMF was also modelled. Flooding patterns derived by TUFLOW for the design flood events are described in **Chapter 6** of the report, with exhibits presented in **Volume 2**.

Design water surface profiles along the main arms of Swaines Creek and its tributaries, as well as the Lane Cove River are shown on **Figures 6.1 to 6.4**. Discharge and stage hydrographs derived by TUFLOW at key locations are shown on **Figure 6.5**. **Figures 6.6** to **6.13** show the indicative extents of inundation.

Diagrams showing the *provisional flood hazard* and the *hydraulic categorisation* of the floodplain for the 100 year ARI flood and the PMF are shown on **Figures 6.14** to **6.17**.

Several runs of the TUFLOW hydraulic model were carried out to test the sensitivity of flood behaviour to changes in hydraulic roughness of the main stream and floodplain, as well as partial blockage of the piped stormwater system. The impact on flood behaviour of increases in rainfall intensities and sea levels due to future climate change was also assessed. The results of these sensitivity analyses are shown on **Figures 6.18** to **6.25**. The analyses showed that increases in peak 100 year ARI flood levels would lie within the 500 mm freeboard allowance which is usually applied to 100 year ARI peak flood levels for setting minimum floor levels for future development.

The *Interim Flood Planning Area (FPA)* and *Interim Flood Planning Levels (FPL's)* for main stream flooding along the Lane Cove River and main arm of Swaines Creek and its tributaries are shown on **Figure 6.26**. The FPA represents the area which will be subject to flood related development controls for main stream flooding and comprises the area lying within the extent of the 100 year ARI flood plus an allowance of 500 mm for freeboard.

Council placed the Draft Flood Study Report of August 2013 on public exhibition over the period 18 November 2013 to 14 February 2014. Four written submissions were subsequently received (refer **Section 2.3** for a summary of the main issues that were raised). This report of March 2014 incorporates several minor amendments to the August 2013 document and is the Final Report for the project.

The models developed for this flood study could be used in the future *Floodplain Risk Management Study (FRMS)* for the catchment which would enable Council to comprehensively manage the flood risk. In addition to finalising the Interim FPA and FPL's, and setting appropriate controls over future development in flood prone areas, the FRMS would include an assessment of available management options including:

- Property Modification measures such as: flood related controls over future development, voluntary purchase of residential property in high hazard areas and raising of floor levels of residences located in low hazard areas.
- Response Modification measures including: improvements to flood warning and emergency management procedures, improvements to the community's awareness of flooding.
- Flood Modification measures such as: levees, detention basins and improvements to hydraulic capacity of channels and floodways.

1 INTRODUCTION

1.1 Study Background

This report presents the findings of an investigation of flooding in the Swaines Creek catchment and has been jointly sponsored by Willoughby City Council (WCC) and the NSW Government, via the Office of Environment and Heritage (OEH). **Figure 1.1** shows the location of the catchment, which drains residential and commercial areas in the suburbs of Chatswood, Chatswood West and Lane Cove North before discharging to the Lane Cove River.

The study objective was to define flood behaviour in terms of flows, water levels and flooding patterns for floods ranging between 1 and 100 year ARI, as well as for the PMF. The investigation involved rainfall-runoff hydrologic modelling of the catchments and drainage systems to assess flows in the Lane Cove River and Swaines Creek, and application of these flows to a hydraulic model of the river and the Swaines Creek drainage system to assess peak water levels and flow patterns. The model results were interpreted to present a detailed picture of flooding under present day conditions.

The scope of the study included investigation of both main stream flood behaviour along the main arm of Swaines Creek and its major tributaries, as well as overland flooding which occurs either as a result of surcharges of the piped drainage system or upstream of the commencement of the formal drainage system. The study also involved investigation of flooding in the lower reaches of the Swaines Creek catchment as a result of flooding along the Lane Cove River.

The study forms the first step in the floodplain risk management process for the Swaines Creek catchment (refer process diagram presented in the Foreword), and is a precursor of the future FRMS sponsored by WCC which will consider the impacts of flooding on existing and future urban development, as well as potential flood mitigation and management measures.

The results of the present study supersede those presented in the *Overland Flooding Investigation* undertaken for the whole of the Willoughby City Local Government Area (LGA) (L&A, 2009). The work undertaken in that study is summarised in **Chapter 2**.

1.2 Approach to Flood Modelling

1.2.1. Hydrologic and Hydraulic Modelling

Flood behaviour was defined using a two-staged approach to flood modelling involving the running in series of:

1. The hydrologic models of the catchments of the Lane Cove River and Swaines Creek, based on the RORB and DRAINS rainfall-runoff software, respectively.
2. The hydraulic model of the Swaines Creek catchment drainage system and adjacent reach of the Lane Cove River based on the TUFLOW software.

The RORB and DRAINS models computed discharge hydrographs, which were then applied to the TUFLOW hydraulic model at relevant sub-catchment outlets.

Within the Swaines Creek catchment, the TUFLOW model used a two-dimensional (in plan), grid-based representation of the natural surface based on an Airborne Laser Scanning (ALS) survey of the catchment, as well as piped drainage data supplied by WCC. Field surveys provided additional data on ground surface levels and piped drainage details. Field survey was also used to derive cross sections (normal to the direction of flow) along a number of the watercourses, including sections of Swaines Creek and Coolaroo Creek, which were used to model their channels.

In the adjacent reach of Lane Cove River, the TUFLOW model comprised a one-dimensional, cross sectionally based representation of the river channel and its right (western) floodplain. Bathymetric and other detailed field surveys of the river derived from a previous study (L&A, 2002), together with 1:2000 scale ortho-photomaps and ALS survey data, were used to compile the cross sections. Discharge hydrographs derived by the RORB model of the Lane Cove River catchment were routed along the river by TUFLOW.

The TUFLOW model was also configured to allow the impact on flood behaviour of river levels ranging from normal tidal conditions to elevated storm-driven water levels in Sydney Harbour.

1.2.2. Model Testing

There are no streamflow data available for either the Swaines Creek or Lane Cove River catchments. Consequently it was not possible to formally “calibrate” the hydrologic models to reproduce recorded discharges. The approach adopted was therefore to test the ability of the hydrologic and hydraulic models in combination to reproduce observed flooding patterns.

Flood marks along the Lane Cove River were available for the November 1984 and August 1986 floods, recorded at the Lane Cove Boat Shed (refer **Figure 4.1** for location). However, no quantitative information relating to historic flood levels was identified within the Swaines Creek catchment. Information was limited to isolated observations of flooding patterns reported during the community consultation process.

For the Lane Cove River, rainfalls recorded during the November 1984 and August 1986 storms were applied to the RORB model to derive discharge hydrographs which were then applied to the TUFLOW model to derive water surface profiles for comparison with the recorded flood marks. The model parameters were varied until flows were derived which, when hydraulically modelled, gave a reasonable correspondence between recorded and derived flood levels.

In the case of flooding in the Swaines Creek catchment, the approach adopted was to test the ability of the DRAINS and TUFLOW models in combination to reproduce observed flooding patterns for historic storms occurring in April 1998 and April 2012. In this case, “best estimates” of model parameters were used based on experience and engineering judgement.

The model testing procedure is summarised in **Chapters 3** and **4**, with further details contained in **Appendix A**.

1.2.3. Design Flood Estimation

Design storms were derived using procedures set out in ARR and then applied to the RORB and DRAINS models to generate discharge hydrographs. These hydrographs constituted input to the TUFLOW hydraulic model.

An “envelope” approach was adopted for defining design water surface elevations and flow patterns throughout the study area. The procedure involved running the model for a range of scenarios, for both catchment-driven flooding and flooding in the lower Lane Cove River as a result of tidal levels in Sydney Harbour, to define the upper limit (i.e. the envelope) of expected flooding for each design flood frequency.

1.3 Layout of Report

Chapter 2 contains background information including a brief description of the study catchment and its drainage system, details of previous flooding investigations, a summary of community consultation undertaken as part of this present study (refer **Appendix A** for details), and a brief history of flooding within the catchment.

Chapter 3 deals with the hydrology of the Lane Cove River and Swaines Creek catchments, and describes the development of the RORB and DRAINS hydrologic models which were used to generate discharge hydrographs for input to the hydraulic model.

Chapter 4 deals with the development of the TUFLOW hydraulic model which was used to analyse flood behaviour in the study area.

Chapter 5 deals with the derivation of design discharge hydrographs, which involved the determination of design storm rainfall depths over the catchments for a range of storm durations and conversion of the rainfalls to discharge hydrographs.

Chapter 6 details the results of the hydraulic modelling of the design floods. Results are presented as water surface profiles and plans showing indicative extents of inundation for a range of design flood events up to and including the PMF. A provisional assessment of flood hazard and hydraulic categorisation is also presented. (The assessment of flood hazard according to velocity and depth of floodwaters is necessarily *“provisional”*, pending a more detailed assessment which includes other flood related criteria, to be undertaken during the future FRMS.) The results of various sensitivity studies undertaken using the TUFLOW model are also presented, including the effects of changes in hydraulic roughness, partial blockage of the piped stormwater system, and potential increases in rainfall intensities and sea levels due to future climate change. This chapter also deals with the selection of Interim FPL’s for the study area.

Chapter 7 contains a list of references.

Chapter 8 contains a list of flood-related terminology that is relevant to the scope of the study.

Appendix A provides details of the collection of historic flood data and describes the testing of the hydrologic and hydraulic models.

Figures referred to in both the main report and the appendices are bound in a separate volume of the report (refer **Volume 2**).

2 BACKGROUND INFORMATION

2.1 Catchment Description

2.1.1. Catchment Overview

Swaines Creek is a tributary of the Lane Cove River. **Figure 2.1** shows the extent of the Lane Cove River catchment upstream of Swaines Creek, which extends north as far as Wahroonga and west as far as Carlingford and Pennant Hills. The river flows generally to the south-east, with the main arm of Swaines Creek joining on its left (eastern) bank a short distance downstream (south) of Fullers Bridge.

The valley drained by Swaines Creek has a total catchment area of about 2.9 km^2 and extends westwards through the suburbs of Chatswood, Chatswood West and Lane Cove North before discharging to the Lane Cove River. **Figure 2.2** shows the extent of the Swaines Creek catchment study area, as well as details of the existing stormwater drainage system.

The Swaines Creek catchment has its headwaters in the vicinity of the Pacific Highway in Chatswood and is bounded on its northern and southern sides by Fullers Road and Mowbray Road West, respectively. The upper part of the catchment is predominantly low density residential in nature, with higher density residential and commercial development located along the western side of the Pacific Highway. Two schools, Chatswood High School and Mowbray Public School, are also located within the upper catchment. The lower part of the catchment and areas bordering the major creek lines are generally reserved for bushland parks (e.g. Ferndale Park) and recreational usage (e.g. Chatswood Golf Club and Rotary Athletic Field).

2.1.2. Main Arm and Tributaries

The three major watercourses within the Swaines Creek catchment comprise the main arm of Swaines Creek and two tributary arms: the Northern Tributary and a southern tributary arm known as Coolaroo Creek.

Main Arm of Swaines Creek

From the catchment headwaters near Mowbray Road West to Dalrymple Avenue, stormwater is conveyed by street gutters and a piped drainage system. In the event of major floods, stormwater would flow through the residential allotments in this area.

The main arm of Swaines Creek commences downstream (north) of Dalrymple Avenue within Ferndale Park, where the piped drainage system discharges into a rocky creek line. Approximately 120 m further downstream, the creek is piped under a footpath linking Dalrymple Avenue and Beresford Avenue via a single 600 mm diameter reinforced concrete pipe (RCP). From this point, Swaines Creek continues as a natural drainage line through Ferndale Park and Chatswood Golf Club to the Lane Cove River.

Lateral piped drainage lines connect into the main arm along its length, the size of which ranges between 300 mm and 1350 mm in diameter. The largest of these lateral drainage lines follows a WCC-owned reserve located along the natural depression that runs at the rear of residential properties fronting Beresford Avenue and Eddy Road.

The Northern Tributary joins the main arm on its right (northern) bank near the western end of Centennial Avenue, while Coolaroo Creek joins the main arm on its left (southern) bank along the northern boundary of Chatswood Golf Club.

<u>Northern Tributary of Swaines Creek</u>

The headwaters of the catchment draining to the Northern Tributary are located near the Pacific Highway and Fullers Road, with the terrain falling steeply to the west into the valley of the catchment. The piped drainage system and the local road network will convey minor flows in this area. However, stormwater would flow through residential allotments in the event of major floods.

At the sag in Edgar Street near its intersection with Western Way, two overland flow paths converge and continue to the west following the line of the trunk drainage system. Approximately 180 m downstream (west) of Edgar Street the piped drainage system, comprising an 825 RCP at this point, discharges into a short length of open channel located at the rear of residential properties. Approximately 70 m further west, the piped drainage system recommences and continues to the downstream (western) side of Park Avenue as a 1050 RCP. Runoff ponding behind the road embankment formed by Park Avenue has the potential to affect a number of residential properties in this area.

Downstream of Park Avenue, the Northern Tributary continues as a natural creek through bushland and falls steeply toward the main arm of Swaines Creek. The confluence of the two watercourses is located about 300 m to the south of Park Avenue.

<u>Coolaroo Creek</u>

The headwaters of the catchment draining to Coolaroo Creek are located along Mowbray Road West, with the terrain falling steeply to the north toward Dalrymple Avenue.

Several short piped drainage lines discharge into Coolaroo Creek on the downstream (western) side of Dalrymple Avenue. From this point the creek flows to the west through bushland between Moola Parade and Coolaroo Road, and is generally located well below residential properties in this area. A number of lateral piped drainage lines discharge into Coolaroo Creek from the surrounding road network, the size of which ranges between diameters of 300 mm and 600 mm.

Coolaroo Creek enters a 900 RCP where it crosses the eastern boundary of Chatswood Golf Club, and is piped under the golf course to a large, centrally located irrigation dam. The piped drainage system continues on the northern side of the dam to its junction with the main arm of Swaines Creek. Runoff which surcharges the piped drainage system traversing the golf course generally follows the same route as the piped drainage along the valley of the catchment.

2.1.3. Drainage Lines Discharging to the Lane Cove River

To the north and south of the confluence of Swaines Creek with the Lane Cove River, the study area comprises relatively steep-sloping areas of Chatswood West and Lane Cove North that drain directly into the river. The upper slopes in these areas are generally developed, with bushland (e.g. Mowbray Park) and recreational areas (e.g. Chatswood Golf Club and Rotary Athletic Field) dominating the lower areas that lie adjacent to the river.

2.1.4. Drainage Lines Discharging Toward Stringybark Creek

Along the southern boundary of the study area, several minor piped drainage lines discharge to the south across Mowbray Road West and across the Willoughby LGA boundary. These drainage lines continue to the south through residential areas within the Lane Cove LGA toward Stringybark Creek.

2.2 Previous Investigations

2.2.1. Overland Flooding Investigation – Willoughby City Area (L&A, 2009)

WCC commissioned a city-wide "screening" study to broadly define flooding patterns and identify properties potentially at risk of flooding from a 100 year ARI flood, including the Swaines Creek catchment (L&A, 2009).

That study used two-dimensional hydraulic modelling of the channel and floodplain, based on the TUFLOW software. Flows generated by a rainfall-runoff model of the catchment based on the DRAINS software were applied to a TUFLOW hydraulic model which routed the floodwave through the drainage system and assessed flooding patterns and indicative extents of inundation.

The results of the overland flooding investigation provided WCC with initial information on flooding throughout the LGA pending the completion of a formal flood study undertaken according to the procedure set out in the NSW Government's *Floodplain Development Manual, 2005* (*FDM*); that is, this present study.

In the L&A, 2009 study, properties in flood prone areas of the various catchments were assessed as being subject to *"Main Stream Flooding"* or *"Local Overland Flooding"* depending on the dominant flood producing mechanism. In broad terms, *Main Stream Flooding* occurs when the trunk drainage systems surcharge and flows extend on to the surrounding floodplain, forming continuous flow paths for the conveyance of floodwaters. *Local Overland Flooding* results from runoff which travels as shallow sheet flow over grassed and paved surfaces in individual allotments or along roads en route to the trunk drainage system (i.e. in areas upstream of the formal drainage system), or which surcharges the minor piped drainage systems in the catchment headwaters and the lateral sub-catchments bordering the trunk drainage system.

Local Overland Flooding was further differentiated into *"Local Drainage"* and *"Major Drainage"* classifications, based on the severity of flooding involved. Areas subject to *Local Drainage* problems typically involved depths of overland flow up to 300 mm, while for *Major Drainage* overland flow depths typically exceeded that value.

These flood classifications are currently being used by WCC to apply flood-related development controls in flood prone areas of the LGA. *(Further discussion relating to flood producing mechanisms and characteristic flood behaviour used for property classification purposes is provided in L&A, 2009.)*

The results of the present study supersede flooding patterns of L&A (2009) and may be used to review the classifications of flood affected property undertaken as part of the earlier investigation.

2.2.2. Lane Cove River Flooding (L&A, 2002 and L&A, 2006)

Investigation of flood behaviour along the reach of the Lane Cove River along the western boundary of the Swaines Creek and adjacent local catchments was previously undertaken as part of flood studies for the Parramatta Rail Link (L&A, 2002) and Lane Cove Tunnel (L&A, 2006). These studies investigated main stream flood behaviour for design events ranging between 5 and 100 year ARI, as well as for the PMF. The focus of the earlier study was at the location of the (then) proposed rail crossing of the river approximately 100 m upstream of Fullers Bridge, while the later work investigated flood behaviour in the vicinity of Epping Road.

Flood behaviour was defined using a rainfall-runoff model of the Lane Cove River catchment based on the RORB software, and a one-dimensional HEC-RAS hydraulic model of the river channel and floodplain. The later study extended the HEC-RAS model to ultimately cover more than 5 km of the river, commencing 800 m upstream of Fullers Bridge to a location about 2.2 km downstream of Epping Road.

The results of those investigations provided flood information for the design of temporary works associated with construction of a cut and cover tunnel under the river for the railway line, and for the design of the Lane Cove Tunnel and associated road works along Epping Road.

2.3 Community Consultation

To assist with data collection and promotion of the study to the Swaines Creek catchment community, the Consultants prepared a Community Newsletter and Questionnaire which was distributed by WCC in May 2012 inviting residents to provide information on historic flooding.

WCC advised that approximately 3,300 Newsletter/Questionnaires were distributed, with a total of 299 responses received (a response rate of around 9 per cent). Of those that responded, 34 noted that they had observed flooding in or adjacent to their property. **Appendix A** provides details of responses to the Newsletter/Questionnaire.

Relevant information obtained from the responses assisted with “ground-truthing” the results of hydraulic modelling (refer **Appendix A**).

The Draft Flood Study Report of August 2013 was placed on public exhibition over the period 18 November 2013 to 14 February 2014. A community information session was also held at Council Chambers on the evening of Tuesday 26 November 2013, which involved presentation of the study methodology and findings after which representatives of WCC and L&A were available to field questions from the floor.

Four written submissions were received by WCC, with the main issues raised noted below (with responses provided in *italics*):

- Several respondents were concerned that the exhibited extents and depths of inundation within specific properties were either not consistent with observed patterns of overland flow, were overstated (e.g. in the footprint of individual houses), or did not appear to account for the presence of local drainage or topographic features that may influence localised flow patterns.

The structure of the hydraulic model that has been developed is considered to adequately represent the key features that control overland flow behaviour for the purposes of a catchment-wide investigation, noting that it is not practical to incorporate internal property drainage systems and other local topographic features (e.g. raised gardens beds, retaining walls, boundary fences, etc.) within the scope of the present investigation.

The definition of overland flow patterns at an individual allotment level would require detailed property survey which is outside the scope of the present investigation.

In several areas it was considered that the approach adopted to represent buildings in the hydraulic model produced artificially high depths of inundation. To reduce the risk of the study findings being misinterpreted, the hydraulic model results were trimmed such that building footprints are shown free of inundation (refer ***Section 6.1.1*** *for further discussion of this issue).*

- Several respondents questioned the current flooding classifications applied to their property by WCC, and queried how the current classifications would be impacted by the present investigation.

  Current classifications will be reviewed by WCC once the present investigation is finalised and adopted for use.

- One respondent considered that the existing stormwater drainage system was not being adequately maintained in certain areas.

  The areas nominated have been brought to the attention of WCC.

In addition to the above-mentioned trimming of the hydraulic model results, this report of March 2014 incorporates several minor amendments to the Draft Flood Study Report, and is the Final Report for the project.

2.4 Historic Flooding in the Study Area

2.4.1. Swaines Creek

The piped drainage system in the Swaines Creek catchment has surcharged during several storms experienced over the past 25 years. There are, however, very little historic flood data or reported observations of flood behaviour over this time to assist the investigation.

There are no rain gauges located within the Swaines Creek catchment. Based on experiences in the nearby Sugarloaf and Sailors Bay Creek catchments, and supported by resident responses to the Newsletter/Questionnaire, the most recent major storm to have affected the Willoughby area occurred on 10 April 1998. Rainfall intensities recorded at the pluviometer at Chatswood Bowling Club during this event exceeded 100 year ARI values for durations ranging between 30 minutes and 1 hour. This gauge is located only a short distance from the eastern boundary of the Swaines Creek catchment and about 1 km east of the catchment centroid (refer **Figure A2.1** in **Appendix A** for gauge location).

Other instances of intense rainfall in the Willoughby LGA occurred in the late 1980's and are reported in previous flood studies for Sugarloaf Creek (e.g. LMCE, 1988). These include storms in August 1986 and April 1988, previously assessed at around 20 year ARI and 2 year ARI, events respectively.

The experiences of respondents to the Newsletter/Questionnaire mainly relate to instances of "flash flooding" resulting from surcharging of internal property drainage systems and some elements of WCC's lateral piped drainage system, causing flows along streets and down private driveways and leading to inundation of garages and yard areas. Reported instances of property damage appear to be the result of shallow overland flows approaching from the direction of adjacent property or roads. There were no reported instances of significant property affectation as a result of flows surcharging the main arm of Swaines Creek or its major tributaries, including Coolaroo Creek.

Flood experiences of respondents relate primarily to the 10 April 1998 event, as well as a series of relatively minor storm events that have occurred since October 2009, but which had recurrence intervals of less than 2 years. The time that has elapsed since the occurrence of other large storms which affected the LGA in the mid-1980's is likely to be a contributing factor to the lack of quantitative data in the responses.

As far as could be ascertained, the trunk drainage system of the Swaines Creek catchment generally functioned at its potential capacity. Instances of reported blockage were limited to the piped crossing of Swaines Creek under the footpath linking Dalrymple Avenue and Beresford Road. The trunk drainage system is less susceptible to blockage than systems in other semi-urbanised catchments, due to the presence of grates at the inlet pits in the street system and the absence of open channels, apart from the main arm of Swaines and Coolaroo creeks.

More than 50 respondents to the Newsletter/Questionnaire noted observations of drain blockages that relate to elements of the minor pit and piped drainage system throughout the catchment.

2.4.2. Lane Cove River Flooding

L&A, 2002 and 2006 identified flood marks along the river for the significant flood events which occurred in November 1984 and August 1986. The flood marks for these events are identified by brass plates on the wall of the Lane Cove Boat Shed which is located approximately 800 m upstream of Fullers Bridge (refer **Figure 4.1** for location), and were levelled as part of the 2002 investigation. The recorded peak flood levels for the two events were as follows:

- November 1984 – 5.07 m AHD
- August 1986 – 3.80 m AHD

There are no other historic flood data or reported observations of flooding to assist in understanding historic flooding along the Lane Cove River between Fullers Bridge and Epping Road.

3 HYDROLOGIC MODEL DEVELOPMENT AND TESTING

3.1 Hydrologic Modelling Approach

The present investigation required the use of a hydrologic model which is capable of representing the rainfall-runoff processes that occur within the Swaines Creek catchment, as well as the larger Lane Cove River catchment.

The RORB model of the Lane Cove River catchment developed for the previous flood studies of flood behaviour along the river (refer **Section 2.2.2**) was adopted for the purpose of this present study and was used to generate discharge hydrographs from relevant sub-catchments of the Lane Cove River (except for the Swaines Creek catchment). These hydrographs were applied to the TUFLOW hydraulic model as point inflows at appropriate locations along the river.

The hydrologic response of the Swaines Creek catchment was simulated using the DRAINS software, which has been developed primarily for modelling the passage of a flood wave through urban catchments and is therefore well suited to this present investigation. Discharge hydrographs generated by DRAINS were applied to the TUFLOW hydraulic model of the Swaines Creek drainage system.

3.2 RORB Model Layout

Figure 2.1 shows the layout of the RORB model, reproduced from similar figures contained in L&A, 2002 and L&A, 2006. The total catchment area at Fullers Bridge, adjacent to the upstream extent of the study area, is approximately 71 km^2.

The structure of the RORB model, including sub-catchment discretisation and assessed imperviousness, was reviewed and found to be suitable for application to this present investigation with no adjustment.

3.3 DRAINS Model Layout

Figure 2.2 shows the layout of the various sub-catchments which comprise the DRAINS hydrologic model for the Swaines Creek catchment.

As the primary function of the DRAINS model was to generate discharge hydrographs for input to the TUFLOW hydraulic model (which routed the flows through the drainage system), piped reaches and overland flow paths linking the various sub-catchments were not incorporated in the model.

Careful consideration was given to the definition of the sub-catchments which comprise the hydrologic model to ensure peak flows throughout the drainage system would be properly routed through the TUFLOW model. In addition to using the ALS-based contour data, the location of surface inlet pits was also taken into consideration when deriving the boundaries of the various sub-catchments.

Percentages of impervious area were assessed using WCC's aerial photography and cadastral boundary data. Sub-catchment slopes used for input to the DRAINS model were derived from average slope values computed by terrain analysis of the ALS survey data.

3.4 Hydrologic Model Testing – Lane Cove River

3.4.1. General

In the case of Lane Cove River flooding, rainfall data and flood marks were available for the November 1984 and August 1986 historic floods. Rainfalls for those events recorded at several gauges across the catchment (refer **Figure A2.1** in **Appendix A** for locations) were applied to the RORB model to obtain discharge hydrographs which were then used in conjunction with the TUFLOW model to derive water surface profiles for comparison with the recorded flood marks.

3.4.2. RORB Model Parameters

The empirical routing coefficients kc and m are the principal parameters of the RORB model. The values of initial loss (IL) and continuing loss (CL), which are subtracted from the storm rainfalls to determine the rainfall excess, are also important parameters (refer **Appendix A** for overview of the RORB software). In the model testing these parameters were varied until flows were derived which, when hydraulically modelled, gave reasonable correspondence between recorded and derived flood levels.

The parameters found to provide the best overall correspondence with the recorded flood marks were as follows:

- kc = 8.0
- m = 0.8
- IL = 10 mm
- CL = 2.5 mm/hr

3.4.3. Results of Model Testing

The discharge hydrographs generated by RORB, when applied to the TUFLOW hydraulic model, were found to provide a good match to the historic flood marks on the Lane Cove River for the November 1984 and August 1986 flood events. The RORB model parameters set out above were adopted for the design flood estimation described in **Chapter 5**. Model testing for Lane Cove River flooding is discussed in more detail in **Section 4.4** and **Appendix A**.

3.5 Hydrologic Model Testing – Swaines Creek Catchment

3.5.1. General

In the case of flooding on the Swaines Creek catchment, the only quantitative data available to assist in model testing for the storms of April 1998 and April 2012 were rainfall data. Other information was limited to isolated observations of flooding patterns. As a consequence, the experience of the investigators largely governed the choice of model parameters for both the hydrologic and hydraulic analyses.

Rainfalls for the two storms recorded at the Chatswood Bowling Club pluviometer were applied to the DRAINS model to estimate flows. The resulting flows were applied to the TUFLOW model and the computed flooding patterns compared with observed flood behaviour.

3.5.2. DRAINS Model Parameters

As described in **Appendix A**, DRAINS requires information on the soil type, losses to be applied to storm rainfall to determine the depth of runoff, as well as information on the piped drainage system and the time of travel of the flood wave through the catchment. Infiltration losses are of two types: IL arising from water which is held in depressions which must be filled before runoff commences, and a CL rate which depends on the type of soil and the duration of the storm event.

Model testing was undertaken with the following parameters:

- Soil Type = 3.0
- AMC = 3.0
- Paved area depression storage = 2.0 mm
- Grassed area depression storage = 10.0 mm
- Paved flow path roughness = 0.02
- Grassed flow path roughness = 0.07

These parameters have been adopted previously in a number of similar urban flood study investigations for other catchments within the Willoughby City LGA.

3.5.3. Results of Model Testing

The discharge hydrographs generated by DRAINS, when applied to the TUFLOW hydraulic model, gave reasonable correspondence with observed flood behaviour. The DRAINS model parameters set out above were therefore adopted for the design flood estimation described in **Chapter 5**. The model testing for Swaines Creek catchment is discussed in **Section 4.4** and **Appendix A**.

4 HYDRAULIC MODEL DEVELOPMENT AND TESTING

4.1 The TUFLOW Modelling Approach

TUFLOW is a true two-dimensional hydraulic model which does not rely on a prior knowledge of the pattern of flood flows in order to set up the various fluvial and weir type linkages which describe the passage of a flood wave through the system.

The basic equations of TUFLOW involve all of the terms of the equations of unsteady flow. Consequently the model is "fully dynamic" and once tuned will provide an accurate representation of the passage of the floodwave through the drainage system (both surface and piped) in terms of extent, depth, velocity and distribution of flow.

TUFLOW solves the equations of flow at each point of a rectangular grid system which represent overland flow on the floodplain and along streets. The choice of grid point spacing depends on the need to accurately represent features on the floodplain which influence hydraulic behaviour and flow patterns (e.g. buildings, streets, changes in floodplain dimensions and hydraulic roughness, etc).

River, channel and piped drainage systems can be modelled as one-dimensional elements embedded in the larger two-dimensional domain, which typically represents the wider floodplain. Flows are able to move between the one and two-dimensional elements of the model, depending on the capacity characteristics of the drainage system being modelled.

The TUFLOW model developed for the Swaines Creek catchment allows for the assessment of potential flood management measures, such as detention storage, increased channel and floodway dimensions, augmentation of culverts and bridge crossing dimensions, diversion banks and levee systems. All of these measures will need to be considered in the future FRMS of the catchment.

4.2 TUFLOW Model Setup

4.2.1. Model Structure

The layout of the TUFLOW model is shown on **Figure 4.1**. Within the Swaines Creek catchment, the model comprises the pit and pipe drainage system, sections of creek and open channel which are represented by cross sections normal to the direction of flow, as well as overland flow which is modelled by the rectangular grid. The TUFLOW model also incorporates the adjacent reach of the Lane Cove River, the in-bank and right (western) overbank of which is represented by cross sections normal to the direction of flow. The lower reaches of both Blue Gum and Little Blue Gum creeks are also incorporated in the TUFLOW model.

The following sections provide further details of the model development.

<u>**Two-Dimensional Model Domain**</u>

An important consideration of two-dimensional modelling is how best to represent the roads, fences, buildings and other features which influence the passage of flow over the natural surface. Two-dimensional modelling is very computationally intensive and it is not practicable to use a mesh of very fine elements without excessive times to complete the simulation, particularly for long duration flood events. The requirement for a reasonable simulation time influences the way in which these features are represented in the model.

A grid spacing of 2 m was found to provide an appropriate balance between the need to define features on the floodplain versus model run times, and was adopted for the investigation. Ground surface elevations for model grid points were initially assigned using a digital terrain model (DTM) derived from ALS survey data, and updated using ground survey data where such data were available.

Figure 4.2 shows the location and extent of survey data incorporated in the TUFLOW model. Details of previous ground survey undertaken in 2010 along the Northern Tributary of Swaines Creek between Edgar Street and Park Avenue, Chatswood were provided by WCC in the form of a DTM.

Additional ground and cross section survey was also obtained along selected drainage lines where it was considered that ALS survey data were not adequate to define ground levels in the vicinity of existing residential development (refer **Figure 4.2** for the location of additional survey by Byrne and Associates in 2012).

Ridge and gully lines were added to the TUFLOW model where the grid spacing was considered too coarse to accurately represent important topographic features which influence the passage of overland flow. The elevations for these ridge and gully lines were determined from survey data where available, or otherwise from inspection of ALS survey or site-based measurements.

Gully lines were also used to represent various sections of creek remote from residential development in the lower parts of the catchment where it was not necessary to precisely represent the conveyance capacity of these watercourses. The use of gully lines ensured that positive drainage was achieved along the full length of these watercourses, and thus avoided creation of artificial ponding areas as artefacts of the 'bumpy' nature of the underlying ALS survey data.

The footprints of a large number of individual buildings located in the two-dimensional model domain were digitised and assigned a high hydraulic roughness value relative to the more hydraulically efficient roads and flow paths through allotments. This accounted for their blocking effect on flow while maintaining a correct estimate of floodplain storage in the model.

It was not practicable to model the individual fences surrounding the many allotments in the study area. They comprised many varieties (brick, paling, colorbond, etc) of various degrees of permeability and resistance to flow. It was assumed that there would be sufficient openings in the fences to allow water to enter the properties, whether as flow under or through fences and via openings at driveways. Individual allotments where development is present were digitised and assigned a high hydraulic roughness value (although not as high as for individual buildings) to account for the reduction in conveyance capacity which will result from fences and other obstructions stored on these properties.

<u>One-Dimensional Model Elements</u>

All of the piped elements contained in WCC's asset database and which influence the passage of flow were included in the TUFLOW model (approximately 700 pipes and 6 box culverts), with the smallest conduit size measuring 90 mm. A small number of enclosed oviform sections within the piped system were modelled as circular conduits with an equivalent diameter determined from drawings or other relevant information provided by WCC. Selected pipe and culvert details were also available as part of previous survey undertaken for WCC in 2010, and this information was used to supplement the asset database as appropriate.

Limited information was available on pipe invert levels, therefore an assumed cover of 700 mm was adopted for those drainage elements where invert levels or depth measurements were not available. Adjustments were made to the assumed invert levels where this approach resulted in a negatively graded reach of pipe or culvert.

Several types of pits are identified on **Figure 4.1**, including junction pits which have a closed lid and inlet pits which are capable of accepting overland flow. WCC's asset database contained only limited information in regard to inlet pit types and dimensions. Therefore it was not possible to define inlet capacity relationships for incorporation in the TUFLOW model. The capacity of the piped drainage system is therefore based on the hydraulic capacity of the pipes as determined by the model.

Pit losses in the various piped drainage networks were modelled using the approach whereby energy loss coefficients at pipe junctions are re-calculated at each timestep of the simulation. The losses are based on a range of variables including the inlet/outlet flow distribution, the depth of water within the pit, expansion and contraction of flow through the pit, the horizontal deflection angle between inlet and outlet pipes, and the vertical drop across the pit.

A total of 28 cross sections derived from both field survey and ALS survey data were used to define the in-bank waterway area of major creek lines draining the Swaines Creek catchment. These comprise open channel reaches along the main arm of Swaines Creek and its northern tributary, as well as Coolaroo Creek between Dalrymple Avenue and Chatswood Golf Club. The locations of open channel reaches and cross sections are shown on **Figure 4.1**.

An additional 38 cross sections were used to define the in-bank and right (western) overbank waterway area of the Lane Cove River, as well as the full waterway area along Little Blue Gum Creek (refer **Figure 4.1** for location). Cross sectional data for these waterways was derived from previous flood studies along the Lane Cove River (refer **Section 2.2.2**).

Cross sectional data along the river were supplemented with ALS survey data and 2 m ortho-photomap contour data in several locations to ensure the full extent of the floodplain was represented in the model.

4.2.2. Model Parameters

The main physical parameter for TUFLOW is the hydraulic roughness. Hydraulic roughness is required for each of the various types of surfaces comprising the overland flow paths, as well as for the cross sections representing the geometric characteristics of the various river and creek channels. In addition to the energy lost by bed friction, obstructions to flow also dissipate energy by forcing water to change direction and velocity and by forming eddies. Hydraulic modelling traditionally represents all of these effects via the surface roughness parameter known as "Manning's n". Flow in the piped system also requires an estimate of hydraulic roughness.

Lane Cove River Hydraulic Roughness

Manning's n values along the main river channel, river banks and immediate overbank areas along the modelled length of the Lane Cove River were varied, with the values in **Table 4.1** providing correspondence between recorded and modelled flood levels.

TABLE 4.1
"TUNED" HYDRAULIC ROUGHNESS VALUES
DERIVED FOR LANE COVE RIVER

Surface Treatment	Manning's n Value
Main river channel	0.04
Vegetated river banks and immediate overbank areas	0.06 – 0.15

Swaines Creek Hydraulic Roughness

There were no historic flood level data available to tune the model for roughness in the Swaines Creek catchment. Assessment of Manning's n values for the open sections of creek was relatively straightforward, as cross sections taken normal to the direction of flow have traditionally been used when modelling one-dimensional waterways. Creek roughness was estimated from site inspection, past experience and values contained in the engineering literature.

Table 4.2 presents the "best estimate" of hydraulic roughness values adopted for model testing. These values gave reasonable correspondence with observed local catchment flood behaviour, and were also adopted for design purposes.

TABLE 4.2
"BEST ESTIMATE" OF HYDRAULIC ROUGHNESS VALUES
ADOPTED FOR TUFLOW MODEL TESTING

Surface Treatment / Model Element	Manning's n Value
Concrete pipes / box culverts	0.015
Asphalt or concrete road surface	0.02
Well-maintained grass cover (e.g. sports field)	0.03
Grass or Lawns	0.045
Trees / Shrubs	0.08
Creek channel	0.05 – 0.08
Creek bank	0.1
Allotments (between buildings)	0.1
Buildings	10

The adoption of a value of 0.02 for the surfaces of roads, along with an adequate description of their widths and centreline/kerb elevations, allowed an accurate assessment of their conveyance capacity to be made. Similarly, the high value of roughness adopted for buildings recognised that these structures will completely block the flow but are capable of storing water when flooded.

Figure 4.3 is a typical example of flow patterns derived from the above roughness values. This example applies for the 100 year ARI design flood and shows overland flows in the vicinity of Jenkins Street, Western Way and Edgar Street, Chatswood.

The left hand side of the figure shows the roads and inter-allotment areas, as well as the outlines of buildings, which have all been individually digitised in the model. The right hand side shows the resulting flow paths in the form of scaled velocity vectors and the depths of inundation. The buildings with their high values of hydraulic roughness block the passage of flow, although the model recognises that they store floodwater when inundated and therefore correctly accounts for flood storage. The flow is conveyed via the road reserves and through the open parts of the allotments. Similar information to that shown on **Figure 4.3** may be presented at any location within the model domain (which is shown on **Figure 4.1**) and will be of assistance to WCC in assessing individual flooding problems in the floodplain.

4.3 Model Boundary Conditions

4.3.1. Inflow Hydrographs

The locations where sub-catchment inflow hydrographs were applied to the TUFLOW model are shown on **Figure 4.1**. These comprise both point-source inflows at selected inlet pits and river reaches (RORB and DRAINS models), and distributed inflows via “Rain Boundaries” (DRAINS model only).

The Rain Boundaries act to “inject” flow into the TUFLOW model, firstly at a point which has the lowest elevation, and then progressively over the extent of the Rain Boundary as the grid in the two-dimensional model domain becomes wet as a result of overland flow. The extent of each Rain Boundary matches the sub-catchment area defined in the DRAINS hydrologic model, resulting in the flows being applied as they would be in the real drainage system.

4.3.2. Downstream Boundary Conditions

The primary downstream boundary of the TUFLOW model comprised a tailwater representing the tidal conditions in the lower Lane Cove River. Due to the relatively short duration of catchment-driven storm events affecting the study area, harbour water levels were applied to the TUFLOW model as a static tailwater.

Several other downstream boundary conditions comprised stage-discharge relationships that were used to model piped and overland flow leaving the study area and entering the Lane Cove LGA to the south of Mowbray Road West.

A static river water level of RL 1.0 m AHD was adopted for modelling the historic storms. A static level of RL 1.0 m AHD was also adopted in the design flood estimation of **Chapter 6** as being representative of tidal conditions in the absence of a storm driven tailwater. **Sections 4.6** and **4.7** describe the various scenarios of concurrent tidal (including storm tides) and catchment flooding adopted for modelling the design floods.

4.4 Hydraulic Model Testing

4.4.1. Lane Cove River

The models were tested for the historic flood events which occurred in November 1984 and August 1986, for which flood marks were identified along the Lane Cove River a short distance upstream of the Swaines Creek catchment.

Based on the findings of the model testing process, the hydrologic and hydraulic models were considered to give satisfactory correspondence with available observed flood behaviour. In particular, the TUFLOW model was found to provide a good match to the historic flood marks on the Lane Cove River for both historic flood events. **Figure A3.1** of **Appendix A** shows water surface profiles derived by TUFLOW along the river.

4.4.2. Swaines Creek

The models were also tested against observed flooding patterns in Swaines Creek for the significant storm which occurred in April 1998, as well as a recent minor storm event which occurred in April 2012.

As far as could be ascertained, there have been no significant drainage works undertaken along the trunk drainage lines within the Swaines Creek catchment in recent years. As a result, it was not necessary to adjust the structure of the TUFLOW model (i.e. from that developed to represent present day conditions) in order to simulate flood behaviour in the lower catchment for these historic storms. While the Epping Road bridge over the Lane Cove River was widened as part of the Lane Cove Tunnel project over the period 2004 – 2007, the increased head loss through the structure is small for the magnitude of the four historic flood events used for model testing purposes. Accordingly, the TUFLOW model was not adjusted to account for this minor change.

Figure A4.1 shows results of TUFLOW modelling for the 10 April 1998 storm. Further details and results of the model testing process are provided in **Appendix A**.

4.5 Design Model Parameters

The hydrologic model parameters set out in **Sections 3.4.2** and **3.5.2**, and the hydraulic roughness values set out in **Tables 4.1** and **4.2** are appropriate for use in defining flood behaviour in the study area over the full range of design flood events and have been adopted for design purposes.

4.6 Design Water Levels in Lane Cove River

4.6.1. Tidal River Water Levels

As mentioned, a static river water level of RL 1.0 m AHD was adopted for simulation of Lane Cove River and local catchment flood events in the absence of any storm-driven tailwater influence. A water level of RL 1.0 m AHD corresponds roughly to the peak water level reached in Sydney Harbour on average once or twice per year during a Highest High Water Solstice Spring (HHWSS) tide. This water level would also be representative of levels in the lower reaches of the Lane Cove River.

4.6.2. Storm-Driven River Water Levels

OEH's "*Flood Risk Management Guide: Incorporating Sea Level Rise Benchmarks in Flood Risk Assessments"* (DECCW, 2010) contains an appendix that deals with modelling the interaction of catchment and coastal flooding for different classes of tidal waterway. The appendix may be used to derive scenarios for coincident flooding from those two sources for both present day conditions and conditions associated with future climate change[1].

For a catchment draining directly to the ocean via trained or otherwise stable entrances, such as is the case for the Lane Cove River, DECCW, 2010 offers the following alternative approaches for selecting storm tidal conditions under present day conditions. In order of increasing complexity they are:

- A default tidal hydrograph which has a peak of RL 2.6 m AHD for the 100 year ARI event; or 2.3 m AHD for the 20 year ARI event. This default option is acknowledged (in DECCW, 2010) as providing a conservatively high estimate of tides for these types of entrances.
- A site-specific analysis of elevated water levels at the downstream boundary location. The analysis should include contributions to the water levels such as tides, storm surge, wind and wave set up. The analysis should also examine the duration of high tidal levels, as well as their potential coincidence with catchment flooding. This approach requires a more detailed consideration of historic tides and the entrance characteristics, but provides information which is more directly relevant to a particular catchment.

The latter approach has been adopted for the purpose of this present investigation. Design still [2] water levels applicable to Sydney Harbour were obtained from Watson & Lord (2008), and are shown in **Table 4.3**.

TABLE 4.3
DESIGN HARBOUR WATER LEVELS

Event	Design Still Water Level [1] (m AHD)	Design Peak Storm Tide Level (m AHD)	Adopted Design Tailwater Level [2] (m AHD)
1 year ARI	1.24	1.74	1.7
2 year ARI	1.28	1.78	1.8
5 year ARI	1.32	1.82	1.8
10 year ARI	1.35	1.85	1.9
20 year ARI	1.38	1.88	1.9
50 year ARI	1.42	1.92	1.9
100 year ARI	1.44	1.94	2.0

(1) Source: Watson & Lord (2008).

(2) Rounded values adopted for modelling of design flood events.

[1] Note that further discussion of the potential impact that future climate change induced sea level rise may have on storm-driven harbour water levels, and the resultant effects on flood behaviour within the study area, is provided in **Section 6.4**.

[2] Still water levels include astronomical tide and storm surge components, but exclude influences from local storm effects such as wind setup and local wave conditions.

An allowance of 0.3 m to account for local storm effects such as wind setup and wave conditions, plus an allowance of 0.2 m to account for minor flood slope that may exist in the lower reaches of the Lane Cove River under catchment-driven flooding conditions, were added to the design still water levels to yield the design peak "storm tide" levels. **Table 4.3** shows the design peak storm tide levels as well as the rounded values adopted for modelling of design flood events.

A flood envelope approach was adopted for defining design water surface elevations and flow velocities throughout the study area. The procedure involved running the model for a range of scenarios, for both catchment-driven flooding and inundation of the lower reaches of the study area as a result of elevated harbour water levels, to define the upper limit of expected flooding for each design flood frequency.

Derivation of design flood envelopes to define the upper limit of expected flooding for each flood frequency (i.e. as a result of both catchment flooding, and storm-driven harbour water levels) is presented in **Section 4.7**. The impact of elevated water levels in the harbour on flood behaviour in the study area is presented in the hydraulic modelling of design floods in **Chapter 6**.

4.7 Derivation of Design Flood Envelopes

The process undertaken for deriving the design flood envelopes for the study area was as follows:

- **Step 1** – Run the hydraulic model for local catchment storms of various return periods and durations in combination with the HHWSS tide level. [The static water level of RL 1.0 m AHD was adopted as the downstream boundary of the hydraulic model for these runs].
- **Step 2** – Combine the results of **Step 1** to create an envelope of maximum catchment flood levels for each return period (i.e. the results of running storms of the same return period but different duration were combined to create a single envelope).
- **Step 3** – Run the hydraulic model for local catchment storms in combination with peak design storm tide levels of various return periods. [The static water levels shown in **Table 4.3** were adopted as the downstream boundary of the hydraulic model for these runs].
- **Step 4** – Prepare a final set of flood envelopes for each return period using a combination of the envelopes derived from **Step 2**, and a corresponding storm tide condition from **Step 3**. **Table 4.4** over sets out the combination of local catchment and storm tide conditions which were used to compile the design flood envelopes for the study area.

The storm durations modelled for assessment of local catchment flooding ranged between 25 minutes and 6 hours. Storms of shorter duration, typically the 25 and 60 minute duration events, were generally critical in terms of maximising peak flood levels within the upper and middle reaches of the Swaines Creek catchment. A storm duration of 6 hours was found to be critical in terms of maximising peak flood levels along the Lane Cove River and adjacent areas of the lower Swaines Creek catchment, following initial assessment of storm durations of up to 12 hours.

TABLE 4.4
DERIVATION OF DESIGN FLOOD LEVEL ENVELOPES

Design Flood Envelope	Local Catchment Flood	Harbour Boundary Condition
1 year ARI	1 year ARI [1]	HHWSS peak tide level (i.e. RL 1.0 m AHD)
	1 year ARI [2]	1 year ARI design tailwater level (i.e. RL 1.7 m AHD)
2 year ARI	2 year ARI [1]	HHWSS peak tide level (i.e. RL 1.0 m AHD)
	2 year ARI [2]	2 year ARI design tailwater level (i.e. RL 1.8 m AHD)
5 year ARI	5 year ARI [1]	HHWSS peak tide level (i.e. RL 1.0 m AHD)
	5 year ARI [2]	5 year ARI design tailwater level (i.e. RL 1.8 m AHD)
10 year ARI	10 year ARI [1]	HHWSS peak tide level (i.e. RL 1.0 m AHD)
	5 year ARI [2]	10 year ARI design tailwater level (i.e. RL 1.9 m AHD)
20 year ARI	20 year ARI [1]	HHWSS peak tide level (i.e. RL 1.0 m AHD)
	5 year ARI [2]	20 year ARI design tailwater level (i.e. RL 1.9 m AHD)
50 year ARI	50 year ARI [1]	HHWSS peak tide level (i.e. RL 1.0 m AHD)
	10 year ARI [2]	50 year ARI design tailwater level (i.e. RL 1.9 m AHD)
100 year ARI	100 year ARI [1]	HHWSS peak tide level (i.e. RL 1.0 m AHD)
	20 year ARI [2]	100 year ARI design tailwater level (i.e. RL 2.0 m AHD)
PMF	PMF [1]	HHWSS peak tide level (i.e. RL 1.0 m AHD)
	100 year ARI [2]	100 year ARI design tailwater level (i.e. RL 2.0 m AHD)

(1) Indicates use of local catchment floods for durations ranging between 25 minutes and 6 hours (for 1 to 100 year ARI), or 15 to 60 minutes (for PMF).

(2) Indicates use of local catchment flood for duration of 6 hours only.

5 DERIVATION OF DESIGN FLOOD HYDROGRAPHS

5.1 Design Storms

5.1.1. Rainfall Intensity

The procedures used to obtain temporally and spatially accurate and consistent intensity-frequency-duration (IFD) design rainfall curves for the Swaines Creek catchment area are presented in Book II of ARR, 1998. Design storms for frequencies of 1, 2, 5, 10, 20, 50, 100, 200 and 500 year ARI were derived for storm durations ranging between 25 minutes and 12 hours. The procedure adopted was to generate an IFD dataset for the catchment by using the relevant charts in Volume 2 of ARR, 1998. These charts included design rainfall isopleths, regional skewness and geographical factors.

A separate IFD dataset that was used as an input to previous RORB modelling of the Lane Cove River catchment was reviewed and found to be suitable for application to this present investigation with no adjustment.

The use of two IFD datasets for the study reflects the spatial variation in design rainfall characteristics across the study area. However, the difference in rainfall intensities is typically less than +/- 5 per cent for the range of storm durations that are relevant to the present study.

5.1.2. Areal Reduction Factors

The rainfalls derived using the processes outlined in ARR, 1998 are applicable strictly to a point. In the case of a large catchment of over tens of square kilometres, as it would not be realistic to assume that the same rainfall intensity can be maintained over a large area, an areal reduction factor (ARF) is typically applied to obtain an intensity that is applicable over the entire area.

For Swaines Creek, point rainfalls were adopted to represent areal values (i.e. ARF = 1.0) due to the small catchment area (less than 3 km^2).

For the Lane Cove River catchment, data in ARR, 1998 indicates that a small reduction in design rainfall intensities of about 3 per cent (i.e. an ARF of about 0.97) is applicable for a catchment area of about 70 km^2 and a storm duration of 6 hours (the critical duration for the catchment). However, as this reduction is quite small, a conservative approach was adopted for design purposes by adopting an ARF = 1.0.

5.1.3. Temporal Patterns

Temporal patterns for various zones in Australia are presented in ARR, 1998. These patterns are used in the conversion of a design rainfall depth with a specific ARI into a design flood of the same frequency. Patterns of average variability are assumed to provide the desired conversion. The patterns may be used for ARI's up to 500 years where the design rainfall data is extrapolated to this ARI.

The derivation of temporal patterns for design storms is discussed in Book II of ARR, 1998 and separate patterns are presented in Volume 2 for ARI < 30 years and ARI > 30 years. The second pattern is intended for use for rainfalls with ARI's up to 100 years, and to 500 years in those cases where the design rainfall data in Book II of ARR are extrapolated to this ARI.

5.2 Probable Maximum Precipitation

Estimates of PMP were made using the Generalised Short Duration Method as described in the Bureau of Meteorology's update of Bulletin 53 (BOM, 2003). This method is appropriate for estimating extreme rainfall depths for catchments up to 1,000 km^2 in area and storm durations up to 6 hours.

The steps involved in assessing PMP are briefly as follows:

- Calculate PMP for a given duration and catchment area using depth-duration-area envelope curves derived from the highest recorded US and Australian rainfalls.
- Adjust the PMP estimate according to the percentages of the catchment which are meteorologically rough and smooth, and also according to elevation adjustment and moisture adjustment factors.
- Assess the design spatial distribution of rainfall using the distribution for convective storms based on US and world data, but modified in the light of Australian experience.
- Derive storm hyetographs using the temporal distribution contained in Bulletin 53, which is based on pluviographic traces recorded in major Australian storms.

Separate PMP estimates were derived for the Swaines Creek catchment and for the Lane Cove River catchment.

5.3 Derivation of Design Discharges

The RORB and DRAINS hydrologic models were run with the adopted parameters (**Sections 3.4.2** and **3.5.2**) to obtain design hydrographs for ARI's ranging between 1 and 100 years for input to the TUFLOW hydraulic model.

For the PMF, the following adjustments were made to the hydrologic model parameters in accordance with general engineering practice to reduce rainfall losses associated with this event:

- RORB – IL and CL values reduced to zero.
- DRAINS – AMC value increased to 4.

The storm duration of 6 hours was critical in terms of maximising peak discharges along the Lane Cove River for ARI's up to 100 years. For the PMF, the 2 hour duration event was critical. Design peak discharges generated by RORB for the Lane Cove River at Fullers Bridge (i.e. at the upstream extent of the Swaines Creek catchment) are presented in **Table 5.1**. **Figure 5.1** summarises critical storm durations across the catchment for the 100 year ARI.

TABLE 5.1
DESIGN PEAK DISCHARGES
LANE COVE RIVER AT FULLERS BRIDGE
(m^3/s)

5 year ARI [1]	10 year ARI [1]	20 year ARI [1]	50 year ARI [1]	100 year ARI [1]	PMF [2]
380	455	560	670	775	2,470

(1) Peak discharges apply for 6 hour duration storm.

(2) Peak discharge applies for 2 hour duration storm.

From **Table 5.1**, the peak PMF discharge was about 3.2 times the peak 100 year ARI discharge.

For the Swaines Creek catchment, the storm duration of 25 minutes was generally found to be critical for maximising peak flows for individual sub-catchments. Peak PMF flow rates for individual sub-catchments computed by DRAINS for the critical 15 minute PMP storm duration were between 3.7 and 5.0 times the magnitude of peak 100 year ARI flow rates. These values lie within the range of expected multiples for an urban catchment.

Discharge hydrographs derived by the hydraulic model at key locations within the Swaines Creek catchment are presented in **Chapter 6**.

6 HYDRAULIC MODELLING OF DESIGN FLOODS

6.1 Presentation and Discussion of Results

6.1.1. Water Surface Profiles and Extents of Inundation

Water surface profiles along Swaines Creek, its main tributaries and the Lane Cove River are shown on **Figures 6.1** to **6.4** for the 20 and 100 year ARI design floods and the PMF.

Figure 6.5, **Sheets 1** and **2** show discharge and stage hydrographs at key locations in the Lane Cove River and Swaines Creek and its tributaries.

The results confirm the "flash flood" nature of the Swaines Creek catchment, with flood levels generally peaking less than 30 minutes after the commencement of rainfall. On the Lane Cove River flood levels peak around 5 hours after the commencement of rainfall.

Figures 6.6 to **6.13** show the TUFLOW model results for the 1, 2, 5, 10, 20, 50 and 100 year ARI floods and the PMF. These diagrams show the indicative extents of inundation along the main arm of the creek, as well as the overland flow paths and depths of inundation.

In order to create realistic results which remove most of anomalies caused by inaccuracies in the ALS survey (which has a design accuracy such that 68 per cent of the points have an accuracy in level of +/- 150 mm), a filter was applied to remove depths of inundation over the natural surface less than 100 mm. This has the effect of removing the very shallow depths which are more prone to be artifacts of the model, but at the same time giving a reasonable representation of the initiation of the various overland flow paths with increasing flood magnitude.

The relatively high hydraulic roughness that has been applied to building footprints to represent their blocking effect on overland flows, combined with inaccuracies in the ALS which occur across the footprint of buildings, was found to produce artificially high depths of inundation in several properties. This issue was also raised in submissions received by WCC following public exhibition of the Draft Flood Study Report (refer **Section 2.3**), with a number of residents expressing concern that the extent and/or depth of property inundation was overstated. In order to address these concerns, and to reduce the risk of the study findings being misinterpreted, the hydraulic model results were trimmed such that building footprints are shown free of inundation.

The hydrologic capacity of the piped drainage systems in the residential areas bordering the creeks is generally around 2 years ARI. Areas where surcharges occur are evident at the 5 year ARI. However, the resulting overland flows travelling to the main arms of the creeks are shallow, generally not exceeding 200 mm in depth and confined to narrow flow paths. Although both the extents and depths of overland flow increase with the return periods of the storms, flood affectation in the residential areas continues to be relatively minor, with only a few instances of depths of flow greater than 400 mm.

There are several instances of significant ponding upstream of structures on the main arms of the creeks. For example, for floods greater than 20 year ARI, ponding up to 2 m in depth would occur upstream of Park Avenue on the Northern Tributary of Swaines Creek, overtopping the roadway and possibly extending into adjacent allotments bordering the creek. Flood levels upstream of Park Avenue are controlled by the hydraulic capacity of the 1050 mm diameter RCP beneath the road crossing (refer water surface profiles on **Figure 6.1**).

Similarly, on the main arm of Swaines Creek, ponding up to 3.5 m deep could occur upstream of the footpath linking Dalrymple and Beresford Avenues and the footpath would be overtopped by around 0.5 to 0.8 m for floods greater than 20 year ARI. Flood levels are controlled by the 600 mm diameter RCP beneath the crossing. Flooding at both of the above locations would be "flash flooding" with water levels peaking less than 30 minutes after the commencement of heavy rainfall.

As far as flooding in the main arms of the creeks is concerned, the filtering process does not have a significant effect on representation of the areal extent of flooding because of the relatively steep sided channels and floodplains. It is to be noted that while the flood level and velocity data derived from the analyses are consistent throughout the model, the flood extent diagrams should not be used to give a precise determination of depth of flood affectation in individual allotments bordering the main arm.

6.1.2. Accuracy of Hydraulic Modelling

The accuracy of results depends on the precision of the numerical finite difference procedure used to solve the partial differential equations of flow, which is also influenced by the time step used for routing the floodwave through the system and the grid spacing adopted for describing the natural surface levels in the floodplain. Open channels are described by cross-sections normal to the direction of flow, so their spacing also has a bearing on the accuracy of the results. The results are also heavily dependent on the size of the two-dimensional grid, as well as the accuracy of the ALS survey data, which as noted above has a design accuracy based on +/- 150 mm.

Given the uncertainties in the ALS survey data and the definition of features affecting the passage of flow, maintenance of a depth of flow of at least 200 mm is required for the definition of a "continuous" flow path in the areas subject to shallow overland flow approaching the main arm of the creek. Lesser modelled depths of inundation may be influenced by the above factors and therefore may be spurious, especially where that inundation occurs at isolated locations and is not part of a continuous flow path. In areas where the depth of inundation is greater than 200 mm threshold and the flow path is continuous, the likely accuracy of the hydraulic modelling in deriving peak flood levels is considered to be between 100 and 150 mm.

Use of the flood study results when applying flood related controls to development proposals should be undertaken with the above limitations in mind. Proposals should be assessed with the benefit of a site survey to be supplied by applicants, in order to allow any inconsistencies in results to be identified and given consideration. This comment is especially appropriate in the areas subject to shallow overland flow, where the errors in the ALS survey data or obstructions to flow would have a proportionally greater influence on the computed water surface levels than in the deeper flooded main stream areas.

Minimum floor levels for residential and commercial developments should be based on the 100 year ARI flood level plus appropriate freeboard (i.e. the FPL) to cater for uncertainties such as wave action, effects of flood debris conveyed in the overland flow stream and precision of modelling. Selection of Interim FPL's, pending completion of the future FRMS for the catchment, is presented in **Section 6.5**.

The sensitivity studies and discussion presented in **Section 6.3** provide guidance on the suitability of the recommended allowance for freeboard under present day climatic conditions. In accordance with OEH recommendations (DECCW, 2007), sensitivity studies have also been carried out (refer **Section 6.4**) to assess the impacts of future climate change. Increases in flood levels due to future increases in rainfall intensities may influence the selection of FPL's. However, final selection of FPL's is a matter for more detailed consideration in the future FRMS.

6.2 Flood Hazard Zones and Floodways

6.2.1. Provisional Flood Hazard

Flood hazard categories may be assigned to flood affected areas in accordance with the procedures outlined in the FDM. Flood prone areas may be provisionally categorised into *Low Hazard* and *High Hazard* areas depending on the depth of inundation and flow velocity. Flood depths as high as a metre, in the absence of any significant flow velocity, could be considered to represent Low Hazard conditions. Similarly, areas of flow velocities up to 2.0 m/s, but with small flood depths could also represent Low Hazard conditions.

Provisional Hazard diagrams for the 100 year ARI and PMF events in the study area Creek based on Diagram L2 of the FDM are presented on **Figures 6.14** and **6.15**.

For the 100 year ARI, high hazard flooding in the study area is generally confined to the main arms of Swaines Creek and its tributaries as well as a strip along the eastern overbank of the Lane Cove River. Other isolated areas of high hazard, which typically relate to relatively shallow but faster-moving floodwater, relate to flows along and across roadways and down relatively steep sloping areas which fall towards the central threads of the main streams.

For the PMF event, the width of the high hazard zone increases significantly. As expected, the extent of high hazard floodwaters in the various overland flow areas throughout the catchment also increases, both in terms of flow width and connectivity.

The Flood Hazard assessment presented herein is based on considerations of depth and velocity of flow and is *provisional* only. As noted in the FDM, other considerations such as rate of rise of floodwaters and access to high ground for evacuation from the floodplain should also be taken into consideration before a final determination of Flood Hazard can be made. These factors would be taken into account in the future FRMS for the catchment.

6.2.2. Floodways

According to the FDM, the floodplain may be subdivided into the following three hydraulic categories:

- Floodways;
- Flood storage; and
- Flood fringe.

Floodways are those areas of the floodplain where a significant discharge of water occurs during floods. They are often aligned with obvious naturally defined channels. Floodways are the areas that, even if only partially blocked, would cause a significant re-distribution of flow, or a significant increase in flood level which may in turn adversely affect other areas. They are often, but not necessarily, areas with deeper flow of areas where higher velocities occur.

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

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

Floodplain Risk Management Guideline No. 2 Floodway Definition, offers guidance in relation to two alternative procedures for identifying floodways. They are:

- **Approach A**. Using a *qualitative approach* which is based on the judgement of an experienced hydraulic engineer. In assessing whether or not the area under consideration was a floodway, the qualitative approach would need to consider; whether obstruction would divert water to other existing flow paths; or would have a significant impact on upstream flood levels during major flood events; or would adversely re-direct flows towards existing development.
- **Approach B**. Using the hydraulic model, in this case TUFLOW, to define the floodway based on *quantitative experiments* where flows are restricted or the conveyance capacity of the flow path reduced, until there was a significant effect on upstream flood levels and/or a diversion of flows to existing or new flow paths.

One quantitative experimental procedure commonly used is to progressively encroach across either floodplain towards the channel until the designated flood level has increased by a significant amount (for example 0.1 m) above the existing (un-encroached) flood levels. This indicates the limits of the hydraulic floodway since any further encroachment will intrude into that part of the floodplain necessary for the free flow of flood waters – that is, into the floodway.

The *quantitative assessment* associated with **Approach B** is technically difficult to implement. Restricting the flow to achieve the 0.1 m increase in flood levels can result in contradictory results, especially in unsteady flow modelling, with the restriction actually causing reductions in computed levels in some areas due to changes in the distribution of flows along the main drainage line.

Accordingly the *qualitative approach* associated with **Approach A** was adopted, together with consideration of the findings of Howells et al, 2004 who defined the floodway based on velocity of flow and depth. Howells et al suggested the following criteria for defining those areas which operate as a "floodway" in a 100 year ARI event:

- Velocity x Depth greater than 0.25 m^2/s **and** Velocity greater than 0.25 m/s; or
- Velocity greater than 1 m/s.

The portion of the flow path which did not reach the above threshold values would be denoted the "flood fringe".

Flood storage areas would be identified as those areas which do not operate as floodways in a 100 year ARI event but where the depth of inundation exceeded 1 m.

The hydraulic categorisation for the 100 year ARI along both the main arms of the creeks and overland flow paths was assessed in accordance with the Howells et al approach and is shown on **Figure 6.16**. The floodway areas in the Swaines Creek catchment generally encompass the extent of the channels of the main watercourses and a small strip on both sides of the banks. There are no significant "floodway" zones in the residential areas draining to the creeks. These zones are all "flood fringe" areas according to the Howells et al criteria.

The assessed hydraulic categories for the PMF events are shown on **Figure 6.17**. By comparison with the 100 year ARI, the PMF "floodway" is significantly wider along the central threads of the main streams and there are significant "'floodways" in the lateral areas bordering the streams. By inspection of **Figures 6.16** and **6.17**, the Howells et al approach results in a continuous "floodway" being developed along the extents of the main arms of the creeks.

It is also to be noted in the context of defining the "floodway" for the planning flood (100 year ARI) that floods greater than 100 year ARI or increases in peak flows due to climate change will not result in the development of new flow paths along the main arm of the creek and the major overland flow paths.

6.3 Sensitivity Studies

The sensitivity of the hydraulic model was tested to variations in model parameters such as hydraulic roughness, blockage of pipes and the effects of elevated harbour water levels. The main purpose of these studies was to give some guidance on the freeboard to be adopted when setting floor levels of development in flood prone areas, pending the completion of the future FRMS for the catchment. The results are summarised in the following sections.

6.3.1. Sensitivity to Hydraulic Roughness

Figure 6.18 shows the difference in peak flood levels (i.e. the "afflux") for the 100 year ARI 60 minute duration storm resulting from an assumed Manning's n roughness of 0.2 in allotments, compared with the best estimate value of 0.1. This figure also identifies areas where land is rendered flood free, or where additional areas of land are flooded.

Along the main arm and along a number of overland flow paths that follow lateral drainage lines, the higher roughness provides additional resistance to the passage of flow causing the flow to lose momentum. Water is detained in allotments, resulting in a minor increase in peak flood levels which reaches up to 100 mm in isolated locations. Increases in peak flood level are typically accompanied by minor increases to flood extents.

Figure 6.19 shows the afflux for the 100 year ARI 60 minute duration storm resulting from an assumed 20 per cent increase in roughness (compared with best estimate values) along the open channels of the creeks and other heavily vegetated areas throughout the Swaines Creek catchment. The typical increase in peak flood level along the middle to upper reaches of Swaines Creek would be in the range 100 to 200 mm, with lesser values of 50 to 100 mm in the tributaries. The increase in extents of inundation in land bordering the channels would not be significant.

Figure 6.20 shows the afflux for the 100 year ARI 6 hour storm resulting from an assumed 20 per cent increase in roughness along the Lane Cove River. The increase in peak flood levels along the river ranges from 230 mm a short distance downstream (south) of Epping Road to about 500 mm at Fullers Bridge (Millwood Avenue). Increases in peak flood levels in the order of

500 mm to the south of Millwood Avenue are confined to the river and its immediate left (eastern) overbank, and do not affect existing residential development in this area.

6.3.2. Sensitivity to Blockage of Pipes

The mechanism and geometrical characteristics of blockages in the piped system are difficult to quantify and would no doubt be different for each flood event. Realistic scenarios would be limited to one or two pipes becoming partially blocked during a flood event (although it is noted that no instances of blockage were reported to have occurred during historic flooding in the catchment). However, for the purposes of this study, analyses were carried out with the cross sectional areas of all pipes and conduits reduced by 50 per cent of their unobstructed areas. This represents a case which is well beyond a blockage scenario which could reasonably be expected to occur and is presented for illustrative purposes.

Figure 6.21 shows the afflux for the 100 year ARI 60 minute duration storm resulting from a 50 per cent blockage. The average increase in peak flood level from this global blockage would be around 50 to 100 mm. Increases in the extent of inundation are generally minor in nature along the main arm of Swaines Creek, but are more substantial in a number of areas adjacent to the piped drainage system where blockage would result in larger overland flows draining to the creeks.

A 300 mm freeboard allowance would be sufficient to cater for the effects of pipe blockage plus uncertainties in the estimate of roughness in the floodplain.

6.4 Climate Change Sensitivity Analysis

6.4.1. General

Scientific evidence shows that climate change will lead to sea level rise and potentially increase flood producing rainfall intensities. The significance of these effects on flood behaviour will vary depending on geographic location and local topographic conditions. Climate change impacts on flood producing rainfall events show a trend for larger scale storms and resulting depths of rainfall to increase. Future impacts on sea levels are likely to result in a continuation of the rise which has been observed over the last 20 years.

OEH recommends that its guideline *Practical Considerations of Climate Change, 2007* be used as the basis for examining climate change induced increases in rainfall intensities in projects undertaken under the State Floodplain Management Program and the FDM. The guideline recommends that until more work is completed in relation to the climate change impacts on rainfall intensities, sensitivity analyses should be undertaken based on increases in rainfall intensities ranging between 10 and 30 per cent. On current projections the increase in rainfalls within the service life of developments or flood management measures is likely to be around 10 per cent, with the higher value of 30 per cent representing an upper limit. Under present day climatic conditions, increasing the 100 year ARI design rainfall intensities by 10 per cent would produce a 200 year ARI flood; and increasing those rainfalls by 30 per cent would produce a 500 year ARI event.

The NSW Government had previously adopted a Sea Level Rise Policy Statement (NSW Government, 2009) to support adaptation to projected sea level rise impacts. The policy statement included sea level rise planning benchmarks for use in assessing potential impacts of projected sea level rise in coastal areas, including flood risk and coastal hazard assessment. These benchmarks were a projected rise in sea level (relative to 1990 mean sea level) of 0.4 m by 2050 and 0.9 m by 2100, based on work carried out by the Intergovernmental Panel on Climate Change and CSIRO. OEH recommends in its guideline *Flood Risk Management Guide: Incorporating Sea Level Rise Benchmarks in Flood Risk Assessments (DECCW, 2010)* that these benchmark rises should be used to assess the sensitivity of flood behaviour to future sea level rise.

The NSW Government announced its Stage 1 Coastal Management Reforms in September 2012. As part of these reforms, the NSW Government no longer recommends state-wide sea level rise benchmarks, with local councils now having the flexibility to consider local conditions when determining local future hazards. However, WCC considers that the guidance in DECCW, 2010, and in particular the use of the above-mentioned sea level rise benchmarks, remains an appropriate basis for the assessment of potential impacts of sea level rise throughout the LGA.

The impacts of climate change and associated effects on the viability of floodplain risk management options and development decisions may be significant and will need to be taken into account in the future FRMS for the Swaines Creek catchment, using site specific data.

At the present flood study stage, the principal issue regarding climate change is the potential increase in flood levels throughout study area. In addition it is necessary to assess whether the patterns of flow will be altered by new floodways being developed for key design events, or whether the provisional flood hazard will be increased.

In the future FRMS it will be necessary to consider the impact of climate change on flood damages to existing development. Consideration will also be given both to setting floor levels for future development and in the formulation of works and measures aimed at mitigating adverse effects expected within the service life of development. When setting floor levels for future developments in planning policies for a developed catchment like Swaines Creek, it will also be necessary to consider the impact of decisions on the existing streetscape.

Mitigating measures which could be considered in the future FRMS include the implementation of structural works such as levees and channel improvements, improved flood warning and emergency management procedures and education of the population as to the nature of the flood risk.

6.4.2. Sensitivity to Increased Rainfall Intensities

As mentioned, the investigations undertaken at the flood study stage are mainly seen as sensitivity studies pending more detailed consideration in the future FRMS. For the purposes of the investigation, the design rainfalls for 200 and 500 year ARI events were adopted as being analogous to flooding which could be expected should present day 100 year ARI rainfall intensities increase by 10 and 30 per cent, respectively.

Figure 6.22 shows the afflux resulting from an increase of 10 per cent in 100 year ARI rainfall intensities. The average increase in peak flood levels in the channel of the Swaines Creek catchment would be around 100 to 200 mm with lesser values in the tributaries. There are also several residential areas on the slopes adjacent to Swaines and Coolaroo Creeks, where flood levels would be up to 100 mm higher and additional land would be inundated as a result of the increase in flows. The increase in flood levels in the Lane Cove River would be in the range 200 to 500 mm, however the increase in flood affectation would generally be restricted to the immediate overbank, apart from isolated areas on the left bank in the Mowbray Park area.

Figure 6.23 shows the afflux for a 30 per cent increase in 100 year ARI rainfall intensities. The increase in peak flood levels in the channel of the Swaines Creek catchment would be up to 300 mm with lesser values up to 200 mm in the tributaries. There are also residential areas on the slopes adjacent to the creeks, where flood levels would be up to 200 mm higher and an increase in inundation of land. The increase in flood levels in the Lane Cove River would generally be greater than 500 mm upstream of Epping Road. There would be a further increase in flood affectation in the land bordering the left bank in the Mowbray Park area.

The impact of increased rainfall intensities on flooding patterns may be summarised as follows:

- The extent of inundation along the length of the main arm of Swaines Creek and its tributaries does not widen significantly, owing to the relatively steep nature of the surrounding overbank areas.
- While flow would continue to follow its existing course along the valley of the streams, there will be some widening of existing overland flow paths throughout the study area.
- There may be a reduction in the time of rise of the floodwaters. The Swaines Creek catchment is flash flooding with little warning time available to residents (there is typically less than 30 minutes in the time of rise of floodwaters to peak levels after the commencement of heavy rainfall). Therefore effective flood warning may not be achievable even with the benefit of future technical improvements in such systems. Therefore on-going community education via WCC and the NSW State Emergency Service is required to limit risks to people and property. Further consideration of flood warning arrangements and strategies will be undertaken in the future FRMS.

6.4.3. Sensitivity to Rises in Sea Level

For the purposes of the investigation, sensitivity analyses were carried out to assess the impacts of a future 0.4 m (2050 conditions) and 0.9 m (2100 conditions) rise in sea levels on the design 100 year ARI flood envelope for the study area. Adoption of these rises would result in the following design peak 100 year ARI storm tide levels in the Lane Cove River at the downstream end of the model:

- 2050 conditions = 2.4 m AHD (i.e. 2.0 m AHD + 0.4 m)
- 2100 conditions = 2.9 m AHD (i.e. 2.0 m AHD + 0.9 m)

Figures 6.24 and **6.25** show the afflux for design 100 year ARI flood envelope resulting from the above increases in storm tide level, respectively. These figures show that increases in peak flood level are confined to the Lane Cove River below Mowbray Park. Impacts do not propagate into the Swaines Creek catchment.

6.5 Selection of Interim Flood Planning Levels

After consideration of the TUFLOW results and the findings of sensitivity studies outlined in **Section 6.3**, a freeboard allowance of 500 mm was adopted for determination of Interim FPL's for main stream flooding along the Lane Cove River and main arm of Swaines Creek and its tributaries.

Interim FPL contours developed on that basis and the associated Interim FPA are shown on **Figure 6.26**.

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8 FLOOD-RELATED TERMINOLOGY

Note: For an expanded list of flood-related terminology, refer to glossary contained within the Floodplain Development Manual, NSW Government, 2005).

TERM	DEFINITION
Afflux	Increase in water level resulting from a change in conditions. The change may relate to the watercourse, floodplain, flow rate, tailwater level etc.
Annual Exceedance Probability (AEP)	The chance of a flood of a given or larger size occurring in any one year, usually expressed as a percentage. For example, if a peak flood discharge of 500 m^3/s has an AEP of 5%, it means that there is a 5% chance (that is one-in-20 chance) of a 500 m^3/s or larger events occurring in any one year (see average recurrence interval).
Australian Height Datum (AHD)	A common national surface level datum approximately corresponding to mean sea level.
Average Recurrence Interval (ARI)	The average period in years between occurrences of a flood of a particular magnitude or greater. In a long period of say 1,000 years, a flood equivalent to or greater than a 100 year ARI event would occur 10 times. The 100 year ARI flood has a 1% chance (i.e. a one-in-100 chance) of occurrence in any one year (see annual exceedance probability).
Catchment	The land area draining through the main stream, as well as tributary streams, to a particular site. It always relates to an area above a specific location.
Discharge	The rate of flow of water measured in terms of volume per unit time, for example, cubic metres per second (m^3/s). Discharge is different from the speed or velocity of flow, which is a measure of how fast the water is moving (e.g. metres per second [m/s]).
Flood fringe area	The remaining area of flood prone land after floodway and flood storage areas have been defined.
Flood Planning Area (FPA)	The area of land inundated at the Flood Planning Level.
Flood Planning Level (FPL)	A combination of flood level and freeboard selected for planning purposes, as determined in floodplain risk management studies and incorporated in floodplain risk management plans.
Flood prone land	Land susceptible to flooding by the Probable Maximum Flood. Note that the flood prone land is synonymous with flood liable land.
Flood storage area	Those parts of the floodplain that are important for the temporary storage of floodwaters during the passage of a flood. The extent and behaviour of flood storage areas may change with flood severity, and loss of flood storage can increase the severity of flood impacts by reducing natural flood attenuation. Hence, it is necessary to investigate a range of flood sizes before defining flood storage areas.
Floodplain	Area of land which is subject to inundation by floods up to and including the probable maximum flood event (i.e. flood prone land).

TERM	DEFINITION
Floodplain Risk Management Plan	A management plan developed in accordance with the principles and guidelines in the *Floodplain Development Manual, 2005*. Usually includes both written and diagrammatic information describing how particular areas of flood prone land are to be used and managed to achieve defined objectives.
Floodway area	Those areas of the floodplain where a significant discharge of water occurs during floods. They are often aligned with naturally defined channels. Floodways are areas that, even if only partially blocked, would cause a significant redistribution of flood flow, or a significant increase in flood levels.
Freeboard	A factor of safety typically used in relation to the setting of floor levels, levee crest levels, etc. It is usually expressed as the difference in height between the adopted Flood Planning Level and the peak height of the flood used to determine the flood planning level. Freeboard provides a factor of safety to compensate for uncertainties in the estimation of flood levels across the floodplain, such and wave action, localised hydraulic behaviour and impacts that are specific event related, such as levee and embankment settlement, and other effects such as “greenhouse” and climate change. Freeboard is included in the flood planning level.
High hazard	Where land in the event of a 100 year ARI flood is subject to a combination of flood water velocities and depths greater than the following combinations: 2 metres per second with shallow depth of flood water depths greater than 0.8 metres in depth with low velocity. Damage to structures is possible and wading would be unsafe for able bodied adults.
Low hazard	Where land may be affected by floodway or flood storage subject to a combination of floodwater velocities less than 2 metres per second with shallow depth or flood water depths less than 0.8 metres with low velocity. Nuisance damage to structures is possible and able bodied adults would have little difficulty wading.
Mainstream flooding	Inundation of normally dry land occurring when water overflows the natural or artificial banks of a stream, river, estuary, lake or dam.
Mathematical/computer models	The mathematical representation of the physical processes involved in runoff generation and stream flow. These models are often run on computers due to the complexity of the mathematical relationships between runoff, stream flow and the distribution of flows across the floodplain.
Merit approach	The merit approach weighs social, economic, ecological and cultural impacts of land use options for different flood prone areas together with flood damage, hazard and behaviour implications, and environmental protection and well-being of the State’s rivers and floodplains.
Overland flooding	Inundation by local runoff rather than overbank discharge from a stream, river, estuary, lake or dam.
Peak discharge	The maximum discharge occurring during a flood event.

TERM	DEFINITION
Peak flood level	The maximum water level occurring during a flood event.
Probable Maximum Flood (PMF)	The largest flood that could conceivably occur at a particular location, usually estimated from probable maximum precipitation coupled with the worst flood producing catchment conditions. Generally, it is not physically or economically possible to provide complete protection against this event. The PMF defines the extent of flood prone land (i.e. the floodplain). The extent, nature and potential consequences of flooding associated with events up to and including the PMF should be addressed in a floodplain risk management study.
Probability	A statistical measure of the expected chance of flooding (see annual exceedance probability).
Risk	Chance of something happening that will have an impact. It is measured in terms of consequences and likelihood. In the context of the manual it is the likelihood of consequences arising from the interaction of floods, communities and the environment.
Runoff	The amount of rainfall which actually ends up as stream flow, also known as rainfall excess.
Stage	Equivalent to water level (both measured with reference to a specified datum).

APPENDIX A

FLOOD DATA COLLECTION AND MODEL TESTING

TABLE OF CONTENTS

Page No.

ATTACHMENTS

LIST OF FIGURES (BOUND IN VOLUME 2)

A1. INTRODUCTION

This Appendix deals with the following matters:

- The results of the community consultation process undertaken as part of this present investigation, which was aimed at collecting data on flooding on the Swaines Creek catchment.
- Compilation of relevant historic flood data from previous investigations.
- The results of testing the hydrologic and hydraulic model for historic storm events in both the Lane Cove River and Swaines Creek.

A number of historic storms were identified when instances of flooding occurred within the Swaines Creek catchment, dating back as far as the mid-1980's. The event identified most frequently by residents was that of 10 April 1998. A number of more recent wet periods were also identified, generally in the first half of 2012. However, there are very limited available historical flood data, or recollection of historic flooding by residents, probably because of the extended flood free period since the last major storm on the catchment.

Flood marks for historic flood events which occurred in November 1984 and August 1986 were identified along the Lane Cove River. These flood marks were documented in previous flood studies dealing with investigation of main stream flood behaviour along the Lane Cove River, undertaken as part of the Parramatta Rail Link (L&A, 2002) and Lane Cove Tunnel (L&A, 2006) projects.

As described in the Main Report, the methodology for assessing flood behaviour within the study area involved:

- The development of hydrologic models of the Lane Cove River catchment and Swaines Creek catchment based on RORB and DRAINS rainfall-runoff software, respectively. The hydrologic models were used to determine the responses to historic and design storms in terms of discharge hydrographs.
- The development of a hydraulic model of the Swaines Creek catchment and adjacent reach of the Lane Cove River based on the TUFLOW two-dimensional (in plan) software. The TUFLOW model was used to route the discharge hydrographs along the river and through the Swaines Creek catchment drainage system and convert the flows to water levels, indicative flood extents and flooding patterns.

Pluviographic rainfall data for the historic storms of November 1984, August 1986, April 1998 and April 2012 were analysed and applied to the RORB and DRAINS catchment models to estimate discharge hydrographs, which were then applied to the TUFLOW hydraulic model of the Lane Cove River and the main arm of Swaines Creek and its overland flow paths.

Section A2 deals with the collection of historic flood data, identification of significant past flood events and analysis of historic storm rainfall data for these events.

Section A3 describes the results of testing the Lane Cove River component of the models for the historic floods and compares the results with observed behaviour.

Section A4 describes the results of testing the local catchment flooding component of the models for the historic floods and compares the results with observed behaviour.

A2. COLLECTION OF HISTORIC FLOOD DATA

A2.1 Previous Investigations

There have been no previous investigations of flooding in the Swaines Creek catchment.

Previous flood studies dealing with main stream flood behaviour along the Lane Cove River (L&A, 2002 and L&A, 2006) identified flood marks for historic flood events which occurred in November 1984 and August 1986. Flood marks for these events are identified by brass plates on the wall of the Lane Cove Boat Shed, which is located approximately 800 m upstream of Fullers Bridge (refer **Figure 4.1** of the Main Report for location), and were levelled as part of the earlier investigation. The recorded peak flood levels for the two events were as follows:

- November 1984 – 5.07 m AHD
- August 1986 – 3.80 m AHD

A2.2 Community Newsletter

A Community Newsletter and Questionnaire was prepared and distributed to residents in the catchment to gain knowledge of historic flood behaviour in the study area (refer **Attachment A**). WCC advised that approximately 3,300 Newsletter/Questionnaires were distributed. A total of 299 responses were received, which represents a response rate of around 9 per cent.

Of those that responded, 56 noted that they had observed flooding in or adjacent to their property. Some respondents were able to identify dates of flooding. However, there was limited information relating to specific flooding patterns or flood levels. On further review, it was determined that 34 respondents had noted identifiable flood behaviour that could be related to dates or periods of historic flooding. *[Note that map showing the location of these respondents, as well as a summary of their comments in relation to historic flood behaviour, was provided separately to WCC.]*

A further 55 respondents noted observations of drain blockages throughout the catchment, but were either not specific about observed flooding at the time or did not provide sufficient information that would enable the flood-affected area to be identified.

The remaining 188 respondents noted that they had not experienced flooding in or adjacent to their property.

However, for flood information to be of direct use in the testing of the hydrologic/hydraulic models, it is necessary to have evidence of the date the flood occurred and the peak flood level that was reached. Unfortunately no such historic flood marks were identified by the consultation process.

Various other separate instances of flooding were identified by respondents, dating back as far as the mid-1980's. However, many reports related only to a year or decade during which flooding occurred, rather than specific events. Where specific events (or at least wet periods) were identified, storms occurring in April 1998 and over the summer of 2011/2012 extending through to April 2012 were identified most frequently by respondents as having caused flooding in or adjacent to their property.

Based on experiences in the nearby Sugarloaf Creek catchment (L&A, 2010), the storm which occurred on 10 April 1998 was a particularly severe event in terms of short duration rainfall intensities. This event was therefore selected for analysis and model testing.

A minor rainfall event which occurred over the two day period 18-19 April 2012 was also selected for analysis and model testing, due to its relatively recent occurrence and availability of photographic records showing overland flow patterns within Chatswood Golf Course in the lower part of the catchment.

A2.3 Historic Storm Rainfall Data

A2.3.1 8 November 1984 Storm

Severe weather during the period 5–9 November 1984 caused extensive damage over the Sydney area, and was the subject of a special meteorological report prepared by BOM, 1985. Sydney's northern suburbs were affected by flash flooding on the morning of Thursday 8 November 1984. BOM, 1985 states that the heaviest daily rainfall was recorded by an unofficial source at Turramurra, where 234 mm of rain was reported to have fallen over the 24 hours ending at 09:00 hours on 8 November 1984. The observer estimated that about 125 mm fell between 07:15 and 08:15 hours on 8 November 1984, which is in excess of the 100 year ARI rainfall for this location.

An isohyetal map presented in BOM, 1985 and partially reproduced in **Figure A2.1** (left hand side), shows that the Turramurra area was the focus of the storm, with the heaviest rainfall occurring in a band that extended in generally a north-south direction roughly through the centroid of the Lane Cove River catchment.

Temporal patterns of rainfall were recorded at several pluviographic sites in and adjacent to the Lane Cove River catchment during the storm event. BOM (1985) notes that these pluviographic sites were not in the centre of the heaviest rainfalls and that, whilst the recorded rainfall intensities were assessed to be generally less than 20 year ARI, rainfall intensities in the centre of the storm would have "far exceeded" the values given in **Table A2.1** over.[1]

By inspection of the recorded values given in **Table A2.2** over for the rain day of 8 November 1984, rainfall depths across much of the Lane Cove River catchment were significantly larger than were experienced at Chatswood.

Whilst there is no rainfall station located within the Swaines Creek catchment, the gauge located at Chatswood Bowling Club is located only a short distance beyond the eastern boundary of the catchment and about 1 km east of the catchment centroid (refer **Figure A2.1** for location). Recorded rainfall intensities over the local catchment approached 5 year ARI for this event for short duration rainfall bursts of between 30 minutes and 1 hour that are generally critical for maximising flows throughout the Swaines Creek catchment (refer **Figure A2.2** for comparison of historic and design rainfall intensity-frequency-duration data).

[1] Note that intense bursts of rain of around 3 hours duration embedded in longer, less intense storm rainfalls are likely to be critical in producing flood discharges in the Lane Cove River.

TABLE A2.1
MAXIMUM RECORDED RAINFALL INTENSITIES
OVER DURATIONS 1-3 HOURS
Rain Day of 8 November 1984
(Values in mm/h)

Location (1)	1 hour	2 hour	3 hour
Ryde Pumping Station	64.5	41.0	29.0
Hornsby Bowling Club	59.5	39.8	27.8
Chatswood Bowling Club	51.0	38.0	27.0
West Epping Bowling Club	35.5	25.8	18.5

(1) Refer **Figure A2.2** for gauge location.

TABLE A2.2
RECORDED RAINFALL TOTALS
Rain Day of 8 November 1984
(Values in mm)

Location (1)	Depth of Rainfall
Hornsby Bowling Club	102
Turramurra	235
Eastwood Bowling Club	149
Ryde Pumping Station	121
Chatswood Bowling Club	104.5
Lane Cove Country Club	82

(1) Refer **Figure A2.2** for gauge location.

A2.3.2 5 August 1986 Storm

The 5 August 1986 storm was a long duration event, with the heaviest falls recorded in the afternoon between 12:00 to 16:00 hours. About 300 mm of rain fell in the Chatswood area over the 24 hour period ending at 09:00 hours on 6 August 1986. **Tables A2.3** and **A2.4** over show details of rainfall intensities and daily falls, respectively whilst **Figure A2.1** (right hand side) shows 24 hour rainfall isohyets for the rain day of 6 August 1986.

Figure A2.2 shows that rainfall intensities over the local Swaines Creek catchment, as recorded at the Chatswood Bowling Club gauge, were generally in the range 10–20 year ARI for storm durations ranging between 30 minutes and 1 hour (i.e. for those storms that are generally critical for maximising flows throughout the Swaines Creek catchment).

TABLE A2.3
MAXIMUM RECORDED RAINFALL INTENSITIES
OVER DURATIONS 1-3 HOURS
Rain Day of 6 August 1986
(Values in mm/h)

Location (1)	1 hour	2 hour	3 hour
Ryde Pumping Station	46.5	43.5	34.3
Chatswood Bowling Club	62.0	39.2	40.3

(1) Refer **Figure A2.2** for gauge location.

TABLE A2.4
RECORDED DAILY RAINFALLS
Rain Day of 6 August 1986
(Values in mm)

Location (1)	Depth of Rainfall
Hornsby Bowling Club	268
Turramurra	258
Eastwood Bowling Club	276
Ryde Pumping Station	337
Chatswood Bowling Club	317
Lane Cove Country Club	242

(1) Refer **Figure A2.2** for gauge location.

A2.3.3 10 April 1998 Storm

Previous investigations (refer LMCE, 1988 and L&A, 2010) assessed the areal distributions and temporal patterns of rainfall associated with the storm of April 1998 and found that the Chatswood Bowling Club record was reasonably representative of recorded depths across other parts of the Willoughby City LGA.

Approximately 222 mm was recorded in the 24 hours to 09:00 on 11 April 1998, with the most intense burst occurring over the 30 minute period from 11:50 to 12:20 hours on 10 April, when 72.5 mm was recorded.

Figure A2.2 shows that the storm of 10 April 1998 exceeded 100 year ARI for storm durations ranging between 30 minutes and 1 hour that are generally critical for maximising flows throughout the Swaines Creek catchment.

A2.3.4 April 2012 Storm

Approximately 131 mm was recorded at Chatswood Bowling Club in the 48 hours to 09:00 on 19 April 2012. The most intense rainfall during this period occurred in two bursts between 07:00-10:30 hours and 17:30-20:30 hours on 18 April 2012.

Figure A2.2 shows that the April 2012 storm was a relatively minor event with an ARI of less than 1 year.

A3. MODEL TESTING – LANE COVE RIVER FLOODING

A3.1 Procedure Adopted for Testing the RORB Model

A3.1.1 General

There was no information available on flood flows in the Lane Cove River to enable a formal calibration of the RORB model. Rainfall data and flood marks were available for the November 1984 and August 1986 historic floods. Rainfalls for those events were applied to the RORB model to obtain an estimate of discharge hydrographs which were then used in conjunction with the TUFLOW model to derive water surface profiles for comparison with the recorded flood marks.

Review of available historic aerial photography indicates that the degree of urbanisation throughout much of the Lane Cove River catchment at the time of the earliest historic event analysed (i.e. November 1984) was broadly consistent with present day conditions. Accordingly, no adjustment to the RORB model sub-catchment boundaries or characteristics (including imperviousness) was made for model testing purposes. Whilst it is appreciated that redevelopment within the catchment since the mid-1980’s may have occurred at a higher density/ intensity than prior development, on-site detention policies have also been in place for much of this time, the aim of which is to offset the impact of such development.

The RORB model parameters were varied until flows were derived which, when hydraulically modelled, gave a reasonable correspondence between recorded and derived flood levels. This process is a “tuning” of the models rather than a formal “calibration”, as the water surface profiles derived from TUFLOW for a given discharge depend on the hydraulic roughness of the river channel and overbank adopted for the analysis. Hydraulic roughness was estimated on the basis of experience and information presented in engineering literature.

A3.1.2 RORB Model Parameters

There are four parameters which have to be estimated when running a RORB model:

Routing parameter m

The parameter m is a measure of the catchment's non-linearity with a value of m=1.0 implying a linear catchment. Most catchments over a range of floods exhibit a non-linear behaviour whereby the peak discharge increases at a proportionally greater rate than the rainfall intensity. This effect is simulated in RORB by adoption of an m value less than 1.0.

For this analysis, a constant m value of 0.8 was used in conformity with recommendations in the RORB manual for flood estimation on ungauged catchments.

Lag parameter kc

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

The value of kc is principally dependent on the catchment area, peak discharge and the parameter m. Various investigators have developed relationships for kc which are based on analysis of the flood record at gauged catchments (IEAust, 1998). They relate kc to catchment area and apply for an m value of 0.8.

<u>Initial loss IL and continuing loss CL</u>

The values of IL and CL, which are subtracted from the storm rainfalls to give the rainfall excess, are also important parameters. Altering the value of these parameters may cause significant changes in the shape and peak of the computed hydrograph. A constant value of 2.5 mm/h for CL was adopted, while the value of IL was varied in sensitivity analyses undertaken during the model tuning process (refer **Section A3.3** for further discussion).

A3.2 Tuning Models to Historic Storms

For the historic storms of November 1984 and August 1986 there was a long duration of low intensity rain prior to the occurrence of the intense burst responsible for the flood peak. Accordingly, peak discharges derived from RORB were not sensitive to variations in assumed IL, with a value of 10 mm adopted for modelling of both historic storms. **Tables A3.1** and **A3.2** show the sensitivity of derived peak flows to variations in kc. Also shown are the recorded flood level at the boatshed and the peak water surface level derived from the TUFLOW model.

TABLE A3.1
RESULTS OF TESTING RORB MODEL
SENSITIVITY OF RESULTS TO VARIATIONS IN Kc
STORM OF 8 NOVEMBER 1984

	Kc=9.0	Kc=8.5	Kc=8.0
Derived Peak Discharge at Fullers Bridge (m^3/s)	638	664	711
Recorded Flood Level at Boatshed (RL m AHD)	5.07	5.07	5.07
Derived Peak Level at Boatshed (RL m AHD)	4.82	4.96	5.11

TABLE A3.2
RESULTS OF TESTING RORB MODEL
SENSITIVITY TO VARIATIONS IN Kc
STORM OF 5 AUGUST 1986

	Kc=9.0	Kc=8.5	Kc=8.0
Derived Peak Discharge at Fullers Bridge (m^3/s)	348	355	361
Recorded Flood Level at Boatshed (RL m AHD)	3.80	3.80	3.80
Derived Peak Level at Boatshed (RL m AHD)	3.44	3.49	3.56

While rainfall intensities and daily rainfall depths experienced over the Swaines Creek catchment were larger for the August 1986 event, maximum point rainfall intensities across much of the Lane Cove River catchment were significantly higher for the 8 November 1984 event. As a result, peak flows in the Lane Cove River adjacent to the Swaines Creek catchment were greater for the November 1984 event, when compared to the August 1886 event.

For the November 1984 flood, the best results were achieved with kc=8.0. This flood was almost 1.3 m higher than the August 1986 event at the boatshed and surcharged the crest of the weir by about 3 m.

For the August 1986 flood, the modelled peak level at the boatshed for kc=8.0 was 240 mm lower than the recorded level. A lesser value of kc would be required to generate a peak discharge sufficiently high to obtain correspondence with the recorded level of 3.80 m AHD for the August 1986 flood. However, adoption of that lesser value of kc for the larger November 1984 flood would have resulted in a computed level substantially higher than 5.07 m AHD.

As the focus of the present investigation is on analysing major design flood events, the kc value of 8.0 found to apply to a major historic flood (i.e. November 1984) was considered more appropriate than a lesser value found to apply to the smaller event.

A3.2.1 Adopted RORB Model Parameters

The RORB model parameters adopted for use in design flood estimation were therefore as follows:

- kc = 8.0
- IL = 10 mm
- m = 0.8
- CL = 2.5 mm/hr

A3.2.2 TUFLOW Model Parameters

The main physical parameter for TUFLOW is the hydraulic roughness. Hydraulic roughness is required for each of the various types of surfaces comprising the overland flow paths, as well as for the cross sections representing the geometric characteristics of the creek channel. In addition to the energy lost by bed friction, obstructions to flow also dissipate energy by forcing water to change direction and velocity and by forming eddies. Hydraulic modelling traditionally represents all of these effects via the surface roughness parameter known as “Manning’s n”.

Manning’s n values along the main river channel, river banks and immediate overbank areas along the modelled length of the Lane Cove River were varied in sensitivity analyses undertaken during the model tuning process. The following values were considered to provide the best correspondence between recorded and modelled flood levels:

- 0.04 Main river channel
- 0.06 – 0.15 Vegetated river banks and immediate overbank areas

A3.3 Modelled Flood Levels in Lane Cove River

Figure A3.1 shows historic water surface profiles along the Lane Cove River derived by TUFLOW for the two flood events.

A4. MODEL TESTING – SWAINES CREEK CATCHMENT FLOODING

A4.1 Procedure Adopted for Testing the Models

In the case of flooding on the Swaines Creek catchment, the only quantitative data available to assist in model testing for the storms of April 1998 and April 2012 were rainfall data, with historic flooding information limited to isolated observations of flooding patterns. The experience of the investigators therefore dictated the choice of parameters for both the hydrologic and hydraulic modelling phases of the analysis.

Due to the lack of Newsletter/Questionnaire responses in relation to the floods of November 1984 and August 1986, these two events were not considered for local catchment model testing purposes.

Flows in the Lane Cove River for the April 1998 and April 2012 storms were derived by running the tuned RORB model. The pluviographic trace recorded at the Chatswood Bowling Club gauge was used to describe the temporal variation of rainfall across the Lane Cove River catchment. The total depth of rainfall at the centroid of each sub-catchment in the model was then adjusted based on daily data recorded at the nearby gauging stations. In the case of the April 1998 storm, this approach yielded a peak flow in the river equivalent to a design flood of about 2 year ARI, even though the rainfall recorded by the Chatswood gauge had an equivalent ARI of about 50 years for the critical duration of 6 hours (refer **Figure A2.2**). The reason for this is attributed to the heavier rain occurring in two distinct bursts separated by several hours, coupled with the fact that the total depth of rain which fell across the majority of the catchment was about half that which was recorded at Chatswood.

A4.1.1 DRAINS Model Parameters

Model testing was undertaken with the following parameters:

- Soil Type = 3.0
- AMC = 3.0
- Paved area depression storage = 2.0 mm
- Grassed area depression storage = 10.0 mm
- Paved flow path roughness = 0.02
- Grassed flow path roughness = 0.07

These parameters have been applied previously in a number of similar urban flood study investigations, including studies for other catchments within the Willoughby City LGA.

A4.1.2 TUFLOW Model Parameters

There are very limited historic flood level data available to assist with the tuning of the TUFLOW model for hydraulic roughness. The process of ascribing roughness to the various types of surfaces encountered on the two-dimensional floodplain of the Swaines Creek catchment was therefore based largely on past experience and values contained in the engineering literature.

Table A4.1 over presents the "best estimate" of hydraulic roughness values within the Swaines Creek catchment that were adopted for model testing.

TABLE A4.1
"BEST ESTIMATE" OF HYDRAULIC ROUGHNESS VALUES
ADOPTED FOR TUFLOW MODEL TESTING

Surface Treatment	Manning's n Value
Asphalt or concrete road surface	0.02
Well-maintained grass cover (e.g. sports field)	0.03
Grass or Lawns	0.045
Trees / Shrubs	0.08
Creek channel	0.05 – 0.08
Creek bank	0.1
Allotments (between buildings)	0.1
Buildings	10

A4.1.3 Presentation of Results

Indicative extents and depths of inundation as computed by the TUFLOW model are shown on **Figure A4.1** for the 10 April 1998 storm.

In order to create realistic results which remove most anomalies caused by inaccuracies in the underlying ALS survey data, a filter was applied to remove depths of inundation over the natural surface less than 100 mm. This has the effect of removing the very shallow depths which are more prone to be artifacts of the model, but at the same time giving a reasonable representation of the various overland flow paths.

Note that TUFLOW model results for the April 2012 storm are not presented due to the minor nature of the event and corresponding minor extents of inundation throughout the study area.

A4.1.4 Comparison of TUFLOW Results with Observed Flood Behaviour

The hydrologic and hydraulic models were considered to provide satisfactory correspondence with available historic flood data, given the limited quality and quantity of such data, with respect to simulation of:

- observed flows along sections of open creek;
- observed overland flow paths and flooding patterns; and
- observed differences in property affectation between storm events of varying magnitude.

While the TUFLOW model generally reproduced observed overland flow behaviour in the Swaines Creek catchment, there were several locations where modelled depths and extents of inundation differed from those identified by respondents to the questionnaire. Based on a review of the information provided by these respondents, it was determined that the observations related to inter-allotment drainage and/or seepage problems, the assessment of which lies beyond the scope of the present investigation. In other areas, the presence of local topographic features (e.g. raised garden beds and boundary fences), details of which cannot practically be incorporated in the TUFLOW model, could also explain the differences which were identified between modelled and observed patterns of flow.

[An assessment of the modelled results against reported observations for each of the 34 respondents to the questionnaire that had noted identifiable flood behaviour was provided separately to WCC.]

A4.2 Adopted Model Parameters

The DRAINS and TUFLOW model parameters adopted for design flood estimation are set out in **Section A4.1** above.

A5. REFERENCES

BOM (Bureau of Meteorology), 1985. *"A Report on the Flash Floods in the Sydney Metropolitan Area over the Period 5 to 9 November 1984"*.

IEAust (The Institution of Engineers Australia), 1998. *"Australian Rainfall and Runoff – A Guide to Flood Estimation"*, Volumes 1 and 2.

L&A (Lyall & Associates Consulting Water Engineers), 2010. *"Sugarloaf Creek Flood Study".*

L&A (Lyall & Associates Consulting Water Engineers), 2006. *"Lane Cove Tunnel and Associated Road Improvements – Flooding at Proposed Epping Road Crossing of Lane Cove River".*

L&A (Lyall & Associates Consulting Water Engineers), 2002. *"Flood Study at Parramatta Rail Link Construction Crossing of Lane Cove River".*

LMCE (Lyall & Macoun Consulting Engineers), 1988. *"Sugarloaf Creek Flood Study".*

ATTACHMENT A

COMMUNITY NEWSLETTER / QUESTIONNAIRE

SWAINES CREEK
FLOOD STUDY

Community Newsletter

Willoughby City Council has engaged consultants to prepare a *Flood Study* for Swaines Creek and those residentially developed areas which drain directly to the adjacent reach of the Lane Cove River. The approximate extent of the study area is shown on the back of this Newsletter. The *Flood Study* will build upon the findings of a recent investigation which was undertaken by Council to identify individual parcels of land which should be subject to flood related development controls. The *Flood Study* is an important step in the Floodplain Management Process for this area and will be managed by Council according to the NSW Government's Flood Prone Lands Policy. The *Flood Study* will define flooding patterns and flood levels in the study area under present day conditions.

The various stages of the *Flood Study* will be as follows:

- Survey along the creek and collection of data on historic flooding.
- Preparation of computer models of the catchments to determine flows for both historic storms and design floods up to the Probable Maximum Flood.
- Preparation of computer based hydraulic models of the creek and floodplain to determine flooding patterns, flood levels and velocities of flow. Flooding in the study area from both the creek and overland flow paths will be evaluated.

The results of the *Flood Study* will provide Council with information on the nature and extent of flooding to assist with planning of development, pending the completion of *Floodplain Risk Management Study*, which will be the next stage of the Floodplain Management Process.

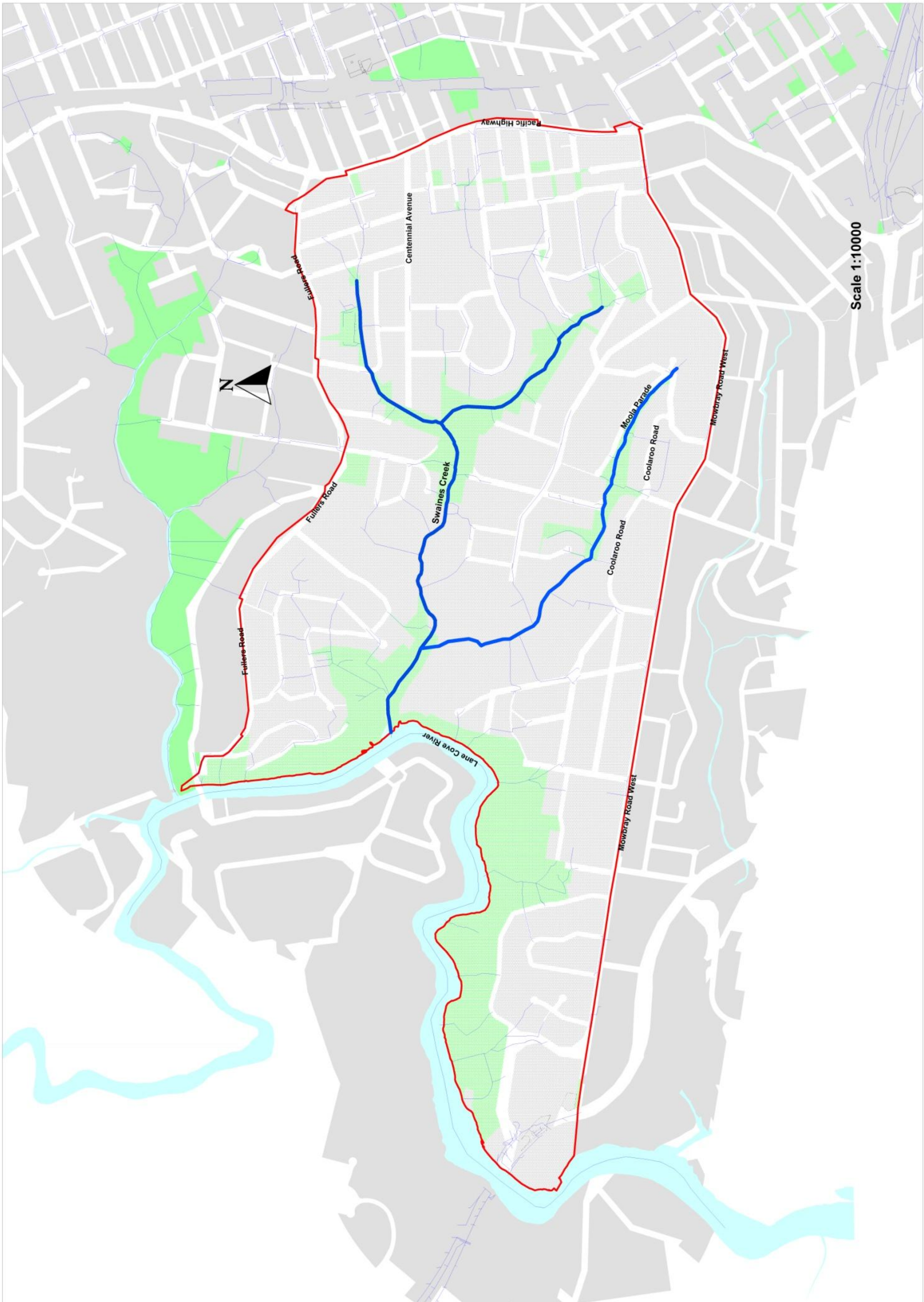
From our initial review of historic rainfall and streamflow data, we have identified the occurrences of several significant flood events in the study area over the past 30 years. These floods are identified below in descending magnitude of severity:

Rank	***Date of Flood***
1	April 1998
2	August 1986
3	November 1984
4	April 1988

We would like information on any of the above events, or other floods which you may have experienced. Several questions relating to flooding in the study area are set out on the attached Questionnaire. Please take a minute or two to read these questions and provide responses where you can. Please return your completed questionnaire in the reply paid envelope provided by **Thursday 31st May 2012.** No postage stamp is required. If you have misplaced the supplied envelope or wish to send an additional submission the address is:

Willoughby City Council
PO Box 57, Chatswood
NSW 2057

Any information you provide will remain confidential and will only be used as statistical data for the *Flood Study.*


Pacific Highway
Centennial Avenue
Fullers Road
Swaines Creek
Moola Parade
Coolaroo Road
Mowbray Road West
Lane Cove River
N
Scale 1:10000

STUDY AREA

SWAINES CREEK
FLOOD STUDY

Community Questionnaire

1. Contact Name: ____

   Address: ____

   Home Phone Number: ____

   Mobile Number: ____

   Email: ____

2. How long have you lived in this location?

   _________ years

3. Has your property ever been inundated by stormwater from the streets or channels in the past?

   [ ] Yes [ ] No

   If yes, when did it occur and where in your property? (Please provide a short description such as: duration of flooding, source of water, flow directions, etc)

	Location	Date/ Time/ Description
[ ]	Driveway	
[ ]	Building (below floor level)	
[ ]	Building (above floor level)	
[ ]	Garage	
[ ]	Front yard	
[ ]	Backyard	
[ ]	Shed	
[ ]	Other (please specify)	

4. If your property has been affected by stormwater flooding in the past, what damages occurred as a result?

____

____

____

5. Are you aware of any other flooding problems in the study area? (Feel free to mark the locations of these problems on the attached map).

____

____

____

6. Please provide dates of historic flooding, even if it is only the year in which the event occurred. Rank the floods from the most severe to the least severe.

1. ___________ 2. ___________ 3. ___________ 4.___________

7. For the floods you have listed, do you have any records of the height the floodwaters reached? For example, a flood mark on a building, shed, fence, light pole, etc.

[ ] Yes [ ] No

If yes, please provide a short description of the location of the flood mark(s), source and or direction of water, etc.

	Location	**Description**
[ ]	Residential	
[ ]	Commercial	
[ ]	Park	
[ ]	Road/ Footpath	
[ ]	Other (please specify)	

8. Do you have any photos, videos or evidence of the watermark on walls or post?

   [ ] Yes   [ ] No

   If yes, could you please provide as much detail as possible, including whether you would be willing to provide Council with electronic copies of any photos/videos? You may wish to email any flood data that you have directly to Council (refer email address below).

   ____

   ____

   ____

   ____

9. Do you have any information on culvert blockage or the inundation of local roads due to water surcharging the local stormwater drainage system?

   [ ] Yes   [ ] No

   If yes, could you please identify the location? Could you also comment on the nature of the blockage and/or the duration and depth of the flooding in the local road network.

   ____

   ____

   ____

   ____

10. If you have any additional information which you believe would assist Council in completing the *Flood Study*, please provide details of such below. (Note that additional space is provided on the back of this page should you need it).

   ____

   ____

   ____

Thank you for your assistance in completing this Questionnaire. Please send the completed Questionnaires using the replied paid envelope.

For any further enquiries, please contact the Council's Design Engineer, Ms Parissa Ghanem on 9777 1000 or email

Parissa.Ghanem@willoughby.nsw.gov.au.

COMMENTS

Please write any further comments you may have here: