**SUGARLOAF CREEK
FLOOD STUDY**

VOLUME 1 – REPORT

FINAL REPORT

AUGUST 2010

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 Sugarloaf Creek Flood Study is jointly funded by Willoughby City Council and the NSW Government, via the Department of Environment, Climate Change and Water. The Flood Study constitutes the first stage of the Floodplain Risk Management process for this area and has been prepared for Willoughby City Council to define flood behaviour under current conditions.

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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 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
0.5	200
1	100
5	20
20	5

The report also refers 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

ARI   Average Recurrence Interval (years)

ARR   Australian Rainfall and Runoff, 2001 Edition

BOM   Bureau of Meteorology

DECCW   Department of Environment, Climate Change and Water

SES   State Emergency Service

WCC   Willoughby City Council

S1   SYNOPSIS

The study objective was to define flood behaviour on Sugarloaf Creek in terms of water levels, flows and velocities for design floods ranging between 5 and 100 year ARI, as well as for the Probable Maximum Flood (PMF). **Figure 1.1** shows the Sugarloaf Creek catchment and its stormwater system. The flood study investigation involved the following activities:

➢ The collection of flood related data. A Community Newsletter/Questionnaire introducing the study objectives and seeking information on historic flood patterns was forwarded to residents in the floodplain. Rainfall data at several daily gauges were also collected. A previous flood study of the Sugarloaf Creek catchment by Lyall and Macoun Consulting Engineers (LMCE, 1988) also provided information on historic flooding. Three significant storms which had occurred over the past 25 years were identified (5 August 1986, 30 April 1988 and 10 April 1998) and their rainfalls used to test the flood models developed for the study.

➢ The hydrologic modelling of the catchment of Sugarloaf Creek to determine discharge hydrographs. All reaches of the piped drainage system of diameter 450 mm or larger were modelled.

➢ Application of the discharge hydrographs to a hydraulic model of the main arm of the creek and its overland flow paths. The model extended from the headwaters of the catchment (to the west of Bales Park) to its outfall to Sugarloaf Bay downstream of the Eastern Valley Way.

➢ Presentation of study results as water surface profiles, as well as diagrams showing indicative extents 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 uncertainties in model parameters such as hydraulic roughness of the floodplain, the effects of partial blockage of the piped drainage system and the effects on flooding patterns resulting from future climate change.

The hydrologic modelling approach was based on the DRAINS rainfall-runoff software. DRAINS derived discharge hydrographs resulting from historic storms for each model sub-catchment area, which were then applied to the hydraulic model to demonstrate that the models reproduced observed flood behaviour. The TUFLOW two-dimensional modelling system was adopted for the hydraulic analysis. **Appendix A** describes the results of testing the models.

Both the DRAINS and TUFLOW models included the piped drainage system and routed the flows to the catchment outlet. Hence both models were able to provide independent estimates of the relative magnitudes of piped and overland flows. However, TUFLOW being primarily a hydraulic model (as opposed to DRAINS which is hydrologically based) was used to route flows over the land surface and determine peak flood levels and flow velocities, as well as indicative extents and depths of inundation.

After testing the models for the historic floods, design storm rainfalls ranging between 5 and 100 year ARI were derived using procedures set out in Australian Rainfall and Runoff (ARR, 2001) and applied to the DRAINS model 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**.

1   INTRODUCTION

1.1   Study Background

This report presents the results of a detailed technical investigation of flooding in the Sugarloaf Creek catchment and has been jointly sponsored by Willoughby City Council (WCC) and the NSW Government, via the Department of Environment, Climate Change and Water (DECCW). **Figure 1.1** shows the location of the catchment, which drains residential and commercial areas in the suburbs of Willoughby and Castlecrag before discharging to Sugarloaf Bay in Middle Harbour. The investigation defined flooding as far as the outfall of the culvert which runs beneath the Eastern Valley Way and continues for about 300 m to the head of the waterfall adjacent to 29 Sunnyside Crescent in Castlecrag.

Mathematical models of the catchment and the floodplain were developed using Council's Airborne Laser Scanning (ALS) survey of the catchment. Field surveys were also undertaken to provide data on the dimensions of the 80 m long open channel section of the creek between Fourth Avenue and the Eastern Valley Way and to refine the ALS data at key road crossings. The model results were interpreted to present a detailed picture of flooding under present day conditions. The study objective was to define flood behaviour in terms of flows, water levels and flooding patterns for floods ranging between 5 and 100 year ARI, as well as for the PMF.

The investigation involved hydrologic modelling of the catchment and drainage system to assess flows in Sugarloaf Creek between its headwaters in the Bales Park area and the estuary of Sugarloaf Bay. These flows were applied to a hydraulic model of the main arm of Sugarloaf Creek and its overland flow paths to assess peak water levels and flow patterns.

1.2   Historic Flooding

The Sugarloaf Creek catchment is drained by a conventional urban system comprising street gutters and minor pipelines beneath the streets, leading to larger pipes and box culverts which follow the former creek line from Stanley Street at the western boundary of the catchment, to the eastern side of the Eastern Valley Way. Apart from the short section of open channel between Fourth Avenue and the culvert beneath the Eastern Valley Way, all of the main arm has been piped. A description of the main drainage system and its standard in hydrologic terms is given in **Chapter 2** of the report. The "hydrologic standard" represents the highest return period of the flows which may be conveyed within the piped system without surcharging and was estimated from the results of hydraulic modelling of both design and historic storms described in **Chapter 6** and **Appendix A** respectively.

The underground drainage system is of limited capacity and has been surcharged to varying degrees during several storms experienced over the past 25 years. On the basis of rainfall intensities recorded over the 1 to 2 hour durations likely to maximise flows on the main arm, the return periods of these historic storms ranged between 1 in 2 and 1 in 50 years.

The most recent severe storm occurred on 10 April 1998. Flooding on the main arm was reported in several commercial properties in Penshurst Street, in residential allotments on the upstream side of High Street, in allotments bordering the streets between High Street and the Eastern Valley Way and as overland flooding through allotments between that road and Sugarloaf Bay. This storm was a 50 year ARI event in terms of rainfall intensities recorded at the Chatswood Bowling Club pluviometer, which is located on the Pacific Highway in the adjacent Scotts Creek

catchment about 1.5 km north-west of the centroid of the Sugarloaf Creek catchment (see **Figure A2.1** of **Appendix A**). No rainfall intensity data were available for Sugarloaf Creek. However, the analysis of daily rainfall data at several adjacent rain gauges, the residents' reports of flooding and the results of model testing in **Appendix A** indicate that the Chatswood pluviographic record was representative of rainfalls actually experienced on that catchment.

Other instances of flooding occurred in the late 1980's and are reported by LMCE, 1988 in a previous flood study of Sugarloaf Creek. In the storm of April 1988 (a 2 year ARI event), overland flows at Penshurst Street flowed into the underground car park of a residential unit block on the eastern side of that street. Residences on the western side of High Street were also flooded and overland flooding occurred in Ann Street and through the Willoughby Bus Depot.

A more severe event occurred in August 1986 (20 year storm), when overland flows crossing Penshurst Street extended as far as Rosewall Street, heading eastwards along that street to High Street. Overflows of the piped drainage system downstream of High Street also occurred, with overland flow being conveyed eastwards through residential allotments and across the local cross streets to the Eastern Valley Way. Flows surcharged the culvert at the Eastern Valley Way, with overland flows continuing through the backyards of residential properties to the open channel leading to Sugarloaf Bay. The frequent flooding in the 1980's led to Council's purchasing a residential property in the sag on the western side of High Street and converting the allotment to open space.

As far as could be ascertained during the data collection and community consultation phases of the study, the main drainage system functioned at its potential capacity, with no known instances of blockage. The Sugarloaf Creek main drainage system is less susceptible to blockage than systems in less 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 section between Fourth Avenue and the culvert beneath the Eastern Valley Way. In 1990 Council improved the inlet of that culvert to maximise its hydraulic capacity and provided a trash rack at the head of the channel on the eastern side of Fourth Avenue to reduce the likelihood of blockage. These measures appear to have been quite successful and have reduced the chances of the culvert surcharging, with consequent overland flow through residential allotments on the eastern (downstream) side of the Eastern Valley Way.

1.3 Approach to Flood Modelling

1.3.1. Hydrologic and Hydraulic Modelling

Flood behaviour was defined using a computer based hydrologic model of the catchment and its drainage system based on the DRAINS rainfall-runoff software and a hydraulic model based on the TUFLOW modelling system.

DRAINS was used to determine discharge hydrographs for the various sub-areas of the catchment. The DRAINS model also incorporated the piped drainage system of the catchment and therefore provided an estimate of flows (separate to TUFLOW) conveyed as piped and overland flow along the line of the main arm of Sugarloaf Creek and its tributaries.

TUFLOW routed discharge hydrographs determined by DRAINS through the drainage system to the catchment outlet.   The TUFLOW model incorporated a two–dimensional, grid–based representation of the floodplain, where the natural surface levels were estimated by Council's

ALS survey of the Willoughby City LGA. TUFLOW modelled both the main arm of Sugarloaf Creek and its floodplain, as well as the overland flow paths of its tributary streams.

It is worth noting that TUFLOW has the capacity to convert rain falling directly on the modelled surface to runoff and to route that runoff along the overland flow paths and through the main drainage system. Hence in theory, it would have been possible to *eliminate* the hydrologic modelling component of the investigation (i.e. eliminate using the DRAINS approach to determine discharge hydrographs) and base the analysis solely on TUFLOW. However, the accuracy with which TUFLOW converts rainfall to flows may not be as great as using DRAINS as a first stage hydrologic model.

The sensitivity analyses undertaken in **Chapter 6** of the report showed that the magnitudes of peak overland flows generated by TUFLOW were very sensitive to the values of hydraulic roughness adopted for the natural surfaces. High values of roughness tended to retard the flow. Consequently, with a single TUFLOW approach to the modelling there is the risk of underestimating overland flow discharges in ungauged catchments such as Sugarloaf Creek, where there is no opportunity to compare the flows generated by the hydraulic model with actual historic flood flows. The traditional two phase modelling approach adopted for this study allows flows generated by the rainfall–runoff hydrologic model (DRAINS) to be compared with those estimated from the Rational Method, which is itself a rainfall–runoff approach to hydrologic modelling (see **Chapter 3**).

1.3.2. Model Testing

There are no stream flow data available on the Sugarloaf Creek catchment. Consequently it was not possible to "calibrate" either of the models to reproduce recorded discharges. Unfortunately, no quantitative information on flood levels was identified as a result of the distribution of the Community Newsletter. Therefore it was not possible to calibrate TUFLOW to reproduce historic overland flooding levels. The approach adopted was to test the ability of the two models in combination to reproduce observed flood behaviour, using "best estimates" of model parameters and to reduce uncertainty in the results for the design floods by sensitivity analysis.

The DRAINS model was used to generate flows for the three historic storms which had caused flooding problems in the catchment and for which pluviographic data were available (April 1988, August 1986 and April 1998). These flows were applied to the TUFLOW model to generate water surface levels and flow patterns. The procedure is summarised in **Chapters 3** and **5**, with further details of the model testing and the responses to the Community Newsletter presented in **Appendix A**.

1.4 Design Flood Estimation

Design storms were derived from Australian Rainfall and Runoff (ARR), 2001 and then applied to the DRAINS model to generate discharge hydrographs within the study area. These hydrographs constituted the upstream boundary and sub-catchment inflows to the TUFLOW hydraulic model.

1.5   Layout of Report

Chapter 2 contains background information including a description of the catchment and the main drainage system and an estimate of its capacity.

Chapters 3 and **4** deal with the hydrology of the Sugarloaf Creek catchment and the results of the DRAINS modelling undertaken to assess flood flows on the catchment. These Chapters describe the set up and testing of the model, the determination of design storm rainfall depths over the catchments for a range of storm durations and conversion of the rainfall hyetographs to discharge hydrographs.

Chapter 5 deals with the development and testing of the TUFLOW hydraulic model. This chapter also includes a detailed investigation of the hydraulics of the floodplain and overland flow paths, which control flood levels in areas bordering the trunk drainage system.

Chapter 6 details the results of the hydraulic modelling of the design floods using TUFLOW. Results are presented as water surface profiles and plans showing indicative extents of inundation for the design flood events. A provisional assessment of flood hazard and hydraulic categorisation is also presented. (The assessment of flood hazard according to hydraulic criteria such as velocity and depth of floodwaters is necessarily *"provisional"*, pending a more detailed assessment of other flood related criteria which would be undertaken during the future *Floodplain Risk Management Study* for the catchment.)

The flood study investigation also included the results of various sensitivity studies of the TUFLOW model, including the effects of changes in hydraulic roughness and partial blockage of the piped stormwater system. The effects on flooding patterns of increases in rainfall intensities due to future climate change are also assessed.

Chapter 7 contains a list of references.

Appendix A summarises responses to the Community Newsletter and describes the testing of the hydrologic and hydraulic models

2   SUGARLOAF CREEK CATCHMENT AND ITS DRAINAGE SYSTEM

2.1   Catchment Description

The valley drained by Sugarloaf Creek has a total catchment area of about 1.9 km^2 and extends eastwards through the suburbs of Willoughby and Castlecrag, before discharging to Sugarloaf Bay in Middle Harbour.   The catchment is bounded on its southern side by Mowbray and Edinburgh Roads and extends as far north as MacMahon Street.

The catchment is urbanised and its natural drainage characteristics have been completely altered by residential and commercial development. From the catchment headwaters near Bales Park to downstream of the Eastern Valley Way, stormwater is conveyed solely by street gutters and pipes. Little evidence remains of the original creek apart from the 80 m section of natural channel downstream of Fourth Avenue. The main drain which is laid at a shallow depth in the valley of the original creek comprises sections of reinforced concrete pipes, oval shaped conduits and box culverts.

Most of the piped system upstream of the Eastern Valley Way was constructed in the 1930's and has become increasingly under-capacity as flows increased due to development of the catchment in more recent years. The system is presently surcharged by even minor flood events with overland flow being conveyed through residential allotments and across the local road system.

Downstream of Eastern Valley Way, flows are conveyed in a box culvert which runs for several hundred metres along the rear of properties in Sugarloaf Crescent. That system was constructed in the 1950's and has a larger hydrologic capacity than the remainder of the drainage system. However in the event of major floods, flows surcharging the Eastern Valley Way would flow through the backyards of downstream residential properties.

Figure 1.1 shows the layout of the drainage system, as provided by WCC. In addition to the main west to east running system, there are several piped tributaries which drain the northern side of the catchment. The most important of these tributaries crosses McClelland Street and joins the main arm on the eastern side of the Willoughby Bus Depot.

Table 2.1 shows details of the piped drainage system and its estimated hydraulic and hydrologic capacity which is generally equivalent to the 1 in 2 year return period, apart from the box culvert section downstream of the Eastern Valley Way, which has a 1 in 10 year capacity.

TABLE 2.1
DETAILS OFSUGARLOAF CREEK
MAIN DRAINAGE SYSTEM

Chainage (m)	Location	Contributing Catchment Area (km^2)	Type of Drain	Approx. Hydraulic Capacity (m^3/s)	Approx. Hydrologic Capacity ARI (years)
-250	Stanley Street		0.75 m dia RCP	1 –2	2
00	Sydney Street		1.82 m Oval Pipe 2.1 m x 1.16 m Box	2.7 – 6.5	2
400	Penshurst Street	0.57	1.2 m dia RCP 1.82 m x 0.9 m Oval Pipe	2.8 – 4	<2
700	d/s High Street	0.93	2.13 m x 1.2 m Oval Pipe	6 – 7	<2
860	Ann Street	0.95	2 x 1.35 m dia RCP	10 – 11	<2
980	First Street	1.12	2.1 m x 2.1 m Box	6 – 7	<2
1160	Second Avenue	1.31	2.74 m x 1.52 m Oval Pipe	12 – 13	<2
1262	Third Avenue	1.35	(1262-1332) 2.74 m x 1.52 m Oval Pipe (1332-1361) 2 m x 0.9 m Oval Pipe (1361-1402) 2.74 m x 1.52 m Box	11 – 13	<2
1403	Fourth Avenue	1.47	Open Channel	–	<2
1494	Eastern Valley Way (d/s side)	1.6	4.5 m x 1.2 m Box	23 – 25	10

3   DEVELOPMENT AND TESTING OF DRAINS HYDROLOGIC MODEL

3.1   Selection of Hydrologic Model

DRAINS is designed to model urban catchments drained by piped drainage systems. In DRAINS, rainfall on each sub-catchment is adjusted to allow for infiltration and other losses. The resulting sub-area rainfall-excess is converted into a discharge hydrograph and assumed to enter the drainage system, subject to constraints imposed by the entrance and conveyance capacity of the system. There, it is added to any existing flow in the system and the combined flow is routed through the system to the outlet.

DRAINS allows for features which control the capacity of the piped system such as pit entry capacity and localised storage areas, assesses the capacity of the piped system using a Hydraulic Grade Line analysis, models gutter flows and routes overland flows along the street system to downstream areas via defined flow paths.

The ability of the DRAINS software to separate the piped flow from the overland flow surcharging the piped system was important in the study. It allowed a comparison to be made with the flows generated by TUFLOW which also modelled the piped drainage system, although DRAINS does not specifically model the overland flow across the natural surface. On Sugarloaf Creek, a minor portion of the flow is piped during major storm events, with most of the flow being conveyed in the street system and across residential allotments as overland flow.

3.2   Model Setup and Layout

The DRAINS model layout, shown on **Figure 3.1**, was developed from details of the existing drainage system supplied by WCC. The pipe data contained in Council's asset database were adequate for the investigation, although in isolated instances pipes had adverse slopes on the basis of their invert levels at gully pits. In such cases the database was considered to be in error and the inverts were adjusted so that pipe gradients conformed with the general overland flow slope. Manipulation of the data did not significantly the results of the flood modelling as the piped system is of limited capacity, as noted above.

Percentages of impervious area were assessed using aerial photos and cadastral boundary data. The sub-catchment areas, pits, conduits and overland flow paths were used to develop the DRAINS model representing the existing drainage system. Apart from the upgrading of the inlet to the culvert at the Eastern Valley Way in 1990, there have been no significant drainage works undertaken in the main system in recent years, so that only one model was used to simulate conditions for both the historic storms and design storms.

3.3   DRAINS Model Parameters

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: initial loss arising from water which is held in depressions which must be filled before runoff commences, and a continuing loss rate which depends on the type of soil and the duration of the storm event.

As mentioned, there are no stream flow data available on Sugarloaf Creek and therefore it was not possible to "calibrate" the model to historic flood flows. The qualitative approach adopted was

to use best estimates of model parameters to simulate flows and levels from historic floods and to compare the models' (i.e. DRAINS and TUFLOW) responses with observed flood behaviour. The results are presented in detail in **Appendix A** and summarised in the flowing sections.

The best estimate of DRAINS model parameters are as follows:

Rainfall Losses

Soil Type = 3.0   (this is an assessment of a soil's rate of infiltration).

AMC   = 3.0   (Antecedent Moisture Condition – assessment of a catchment's wetness at the start of storm event).

Paved area depression storage   = 2.0 mm.

Grassed area depression storage   = 10.0 mm.

Pipe and Pit Data

In addition, the hydraulic roughness for the pipes was assumed to be 0.012, as recommended in ARR, 2001.

Values of pit loss coefficients were assigned in accordance with the Missouri Charts, the DRAINS Manual, various technical papers and in accordance with observed behaviour during historic flooding. The model incorporates a detailed representation of the stormwater drainage system, as supplied by WCC.

Travel Times

Information contained in ARR, 2001 suggests that for large commercial and industrial buildings, which are typical of the commercial areas in the catchment, the response time of the allotments to rainfall would be in the range 5 to 15 minutes. For design purposes, DRAINS modelling was carried out with a minimum response time in the commercial and residential sub-catchments of 5 minutes.

In addition, the path of travel of overland flow was adjusted to follow the pattern of the street system. The resulting flow length and slope was then used by DRAINS to assess the travel time of the floodwave.

3.4   Model Testing Procedure and Results

3.4.1. Comparison with Rational Method

Table 3.1 shows the comparison of peak flows estimated by DRAINS with flows derived from the Rational Method of flood estimation described in Chapter 14 of ARR, 2001. These data are shown at three important road crossings: Penshurst Street, High Street and the Eastern Valley Way. The peak flows shown on the table represent the total discharge passing the three road crossings (i.e. piped flows plus overland flows). DRAINS also provides separate estimates of these two components, as shown on **Figure 4.1**.

The peak flows estimated by DRAINS agree well with the Rational Method estimates.

TABLE 3.1
COMPARISON OF PEAK DISCHARGES
RATIONAL METHOD vs DRAINS MODEL
(values in m³/s)

Location	Rational Method			DRAINS		
	5 yr	20yr	100 yr	5 yr	20 yr	100yr
Penshurst Street	10	16	26	10	17	24
High Street	14	22	36	17	27	38
Eastern Valley Way	21	33	54	28	40	59

3.4.2. Historic Storms

The hydrologic and hydraulic models were tested for the storms of 5 August 1986, 30 April 1988 and 10 April 1998. Rainfalls for the three storms recorded at the Chatswood Bowling Club pluviometer, located near the north-west boundary of the Sugarloaf Creek catchment, were applied to the DRAINS model using the "realistic" values of parameters to estimate flows. The resulting flows were applied to the TUFLOW model and the computed flooding patterns compared with reported flood behaviour.

The models reproduced observed flood behaviour and were therefore considered to be suitable for design flood estimation. The DRAINS model parameters set out in **Section 3.3** were adopted for the design flood estimation described in **Chapter 4**. Further information on the testing the models for the historic storms is presented in **Section 5.3** and **Appendix A**.

4   DERIVATION OF DESIGN FLOOD HYDROGRAPHS

4.1   Rainfall intensity

The procedures used to obtain temporally and spatially accurate and consistent intensity-frequency-duration (IFD) design rainfall curves for the Sugarloaf Creek catchment are presented in Chapter 2 of ARR, 2001. Design storms for frequencies of 5, 10, 20 and 100 year ARI were derived for storm durations ranging between 1 hr and 6 hrs. The procedure adopted was to generate IFD data for each catchment by using the relevant charts in Volume 2 of ARR, 2001. These charts included design rainfall isopleths, regional skewness and geographical factors.

4.1.1.   Areal Reduction Factors

The rainfalls derived using the processes outlined in ARR, 2001 are applicable strictly to a point. In the case of a large catchment of over tens of square kilometres, it would not be realistic to assume that the same rainfall intensity can be maintained over a large area, an areal reduction factor is typically applied to obtain an intensity that is applicable over the entire area.

However, as the area of the Sugarloaf Creek catchment is only 1.9 km^2, the reduction in rainfall intensities would be quite small. Accordingly, the assumption of no reduction in point rainfalls was made for this study.

4.1.2.   Temporal Patterns

Temporal patterns for various zones in Australia are presented in ARR, 2001. These patterns are used in the conversion of a design rainfall depth with a specific ARI into a design flood of the same frequency. Patterns of average variability are assumed to provide the desired conversion. The patterns may be used for ARIs up to 500 years where the design rainfall data is extrapolated to this ARI.

The derivation of temporal patterns for design storms is discussed in Chapter 3 of ARR, 2001 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 ARIs up to 100 years, and to 500 years in those cases where the design rainfall data in Chapter 2 of ARR, 2001 are extrapolated to this ARI.

4.2   Design Discharges

The DRAINS model was run with the parameters given in **Section 3.3** to obtain design hydrographs for input to the hydraulic model. Discharge hydrographs for 5, 20 and 100 year ARI storms at the three road crossings: Penshurst Street, High Street and Eastern Valley Way are plotted on **Figure 4.1**. The 1 hour storm was found to be critical for each recurrence interval. This diagram shows the estimated flow conveyed by the piped system as well as flows conveyed over the roadway.

4.3   Probable Maximum Flood

Estimates of probable maximum precipitation were made using the Generalised Short Duration Method (GSDM) 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 1000 km^2 in area and storm durations up to 6 hours.

The steps involved in assessing PMP for the Sugarloaf Creek catchment 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.

Discharge hydrographs for the critical storm of 15 minutes duration at key locations are shown on **Figure 4.2**. The peak flow ranges between 4.2 times the magnitude of the 100 year ARI peak in the upper reaches of the catchment to 3.3 times in the lower reaches at the Eastern Valley Way. These values are at the lower limit of expected values.

5   DEVELOPMENT AND TESTING OF TUFLOW HYDRAULIC MODEL

5.1   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).

Pipe drainage and channel 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 Sugarloaf Creek 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 *Floodplain Risk Management Study* of the catchment.

5.2   TUFLOW Model Layout

The layout of the Sugarloaf Creek TUFLOW model is shown on **Figure 5.1**. The model comprises the piped system, sections of open channel which are modelled by cross sections normal to the direction of flow, as well as overland flow which is modelled by the rectangular grid.

All of the pipes which influence the passage of flow were included in the TUFLOW model, including pipes less than 600 mm diameter where appropriate. Several types of pits are identified on **Figure 5.1**, including junction pits which have a closed lid and inlet pits which are capable of accepting overland flow. Sub-catchment discharge hydrographs as estimated by DRAINS were applied as "point source hydrographs" at selected inlet pits which are also identified in the figure. Discharge hydrographs from the DRAINS sub-catchments upstream of the TUFLOW model domain were added as distributed flow at the model boundaries.

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 incurring very long 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.

After initial model testing, a 2 m grid spacing was found to provide the appropriate balance between the need to define features on the floodplain versus model run times. Grid elevations were based on WCC's Digital Terrain Model of the catchment. Ridge and gully lines were added to the model where the grid spacing was considered too coarse to accurately represent important topographic features which influence the passage of overland flow, such as road centrelines and footpaths. It was important that the model recognised the ability of roads to capture overland flow and act as floodways.

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 whilst 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.

5.3   Model Roughness

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 between Fourth Avenue and the Eastern Valley Way. 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 "Mannings n". Flow in the piped system also requires an estimate of hydraulic roughness.

There are no historic flood level data available to assist with the tuning of the model for roughness. Assessment of Mannings n values for the short open section 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.

The process of ascribing roughness to the various types of surfaces encountered on the two-dimensional floodplain, where flow was generally shallow and of low velocity, was more difficult. Initial experiments showed that peak flows were quite sensitive to the adopted value of Mannings n. Increasing n resulted in the retarding and storage of water on the upper reaches of the floodplain, with a reduction in downstream flood peaks.

Adoption of high values of n had the potential to over-attenuate the downstream flow, resulting in flood levels that were on the low side. These effects emphasised the need for undertaking sensitivity studies prior to final selection of values for design (see **Section 6.3**) and also confirmed the appropriateness of the two stage hydrologic-hydraulic modelling approach adopted for this study. (As mentioned, that approach allowed the comparison of peak flows derived from the DRAINS hydrologic model with those of the Rational Method -**Table 3.1**).

Table 5.1 presents the "best estimate" of hydraulic roughness values adopted for design purposes. These values gave reasonable correspondence with observed flood behaviour (see **Appendix A**). The adoption of a value of 0.02 for the surfaces of roads, along with an adequate description of their widths and centreline and kerb elevations, allowed an accurate assessment of

their conveyance capacity to be made. Similarly the high value of roughness adopted for buildings recognised that they completely blocked the flow but were capable of storing water when flooded.

Figure 5.2 is a typical example of flow patterns derived from those values. This example applies for the 100 year ARI design flood and shows overland flows on the main arm in the vicinity of the Willoughby Bus Depot.

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 floodwaters when inundated and therefore correctly accounts for flood storage. The flow is conveyed along the paved surface of the bus depot, as well as along the roads and through the open parts of the allotments. Similar information to that shown on **Figure 5.2** may be presented at any location within the model domain (which is shown on **Figure 5.1**) and will be of assistance to Council in assessing individual flooding problems in the floodplain .

TABLE 5.1
"BEST ESTIMATE" OF HYDRAULIC ROUGHNESS VALUES
ADOPTED FOR TUFLOW MODELLING

TUFLOW Identifier	Surface Treatment	Mannings n Value
2	Asphalt or concrete road surface	0.02
3	Well Maintained Grassed Cover e.g. sporting oval	0.03
4	Grass or Lawns	0.045
5	Trees	0.08
8	Creek bank	0.10
9	Creek channel	0.05
10	Rock surface	0.025
13	Allotments (between buildings)	0.05
14	Buildings	10

5.4   Results of Model Testing – Historic Floods

Figures 5.3a and **5.3b** show discharge and stage hydrographs respectively, as modelled by TUFLOW for the three historic floods at the three road crossings. The model was found to reproduce observed flood behaviour. (see **Appendix A** and **Attachments B** to **D** therein for further discussion and exhibits). In particular:

i)   The observed surcharge of the piped drainage system was reproduced in the modelling of the minor storm (2 year ARI) of 30 April 1988. Observed overland flow was modelled at Penshurst and High Streets, including the recorded flooding at the below-ground level car

park at 117 Penshurst Street, as well as the overland flooding in the Willoughby Bus Depot.

ii) The observed overflow of High Street and ponding on the western side of the road, which led Council to voluntary acquire residential property on that side of the road in the late 1980's, was modelled.

iii) The hydraulic modelling of the major 1998 storm correctly modelled the overflows experienced in the local road system including surcharge of the Eastern Valley Way, where the entrance to the culvert had been upgraded by Council in 1990 to increase its hydraulic capacity.

iv) The model correctly reproduced the observed breakouts from the main drainage system into the local road system and the conveyance of flow along the streets, which tend to act as floodways during severe storms.

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 the main arm of Sugarloaf Creek are shown on **Figure 6.1a** for the 20 and 100 year ARI design floods and the PMF. The natural surface profile shown on this figure follows the lowest level above the piped drainage system and is derived from WCC's ALS data. The depressed area between Fourth Avenue and Eastern Valley Way represents the invert of the short section of open channel conveying flows over this reach. The remainder of the main arm is piped. The depressed area upstream of High Street represents the existing open space area on the western side of that street. The roadway is surcharged for floods around the 5 year ARI and acts as a dam with a level pool extending about 100 m upstream.

Discharge hydrographs at the three key road crossings are shown on **Figure 6.1b** and may be compared with the DRAINS discharge hydrographs at the same locations shown on **Figure 4.1**. Whilst the estimates of flows conveyed by the piped drainage are similar, the estimates of peak overland flow discharge are less for TUFLOW than for DRAINS. This difference is attributed to the attenuating effects of the flood storage which is present on the floodplain and which is incorporated into the two-dimensional model domain of TUFLOW, but is not specifically modelled by DRAINS.

Figure 6.1c shows stage hydrographs at the three road crossings. The results confirm the "flash flood" nature of the catchment. Flood levels peak less than one hour after the commencement of rainfall and reach depths of up to 600 mm over the roads for the 100 year ARI flood. Nuisance flooding of around 30 minutes duration would occur at the 5 year ARI for Penshurst Street and High Street. Eastern Valley Way has a higher hydrologic standard and would remain trafficable up to the 10 year ARI level of flooding.

Figures 6.2 to **6.6** show the indicative extents of inundation and flood contours for the 5, 10, 20 and 100 year ARI floods and the PMF. These diagrams show the peak flow conveyed as overland flow (i.e. the flow exceeding the capacity of the piped system). Extents of inundation and flood contour data are also shown for the northern tributary drain, which runs southwards from Glover Street and McClelland Street to the Willoughby Bus Depot; as well as for other overland flow routes including several of the streets which function as floodways.

The extents of inundation of each flood event are indicative only and are based on flood levels derived from Council's ALS data and the locations of residences and commercial buildings bordering the channel as shown on Council's GIS system. Whilst 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.

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 data, which is generally considered to be about plus or minus 150 mm and can often be influenced by the presence of vegetation and small structures in individual allotments.

Given the uncertainties in the ALS data and the definition of features affecting the passage of flow, a threshold depth of 200 mm is considered to be the minimum depth of inundation which allows a realistic representation of flow paths to be made in the areas subject to shallow overland flow approaching the main arm of the creek. Modelled depths of inundation less than 200 mm 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 the 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.

The results of the present investigation will supersede those presented in the recent screening study of flooding in the Willoughby City area (LACE, 2009) which was intended to provide Council with information on flooding, pending the completion of formal flood studies undertaken according to the procedure set out in the NSW Government's *Floodplain Development Manual, 2005*. On Sugarloaf Creek, properties assessed in that earlier study as being subject to main stream flooding or on local overland flow paths approaching the creek are being re-assessed with the benefits of the latest results and in some instances with site survey.

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 errors in the ALS would have a proportionally greater influence on the computed water surface levels than in the deeper flooded mainstream areas.

Pending the future *Floodplain Risk Management Study* for the Sugarloaf Creek catchment minimum floor levels for residential and commercial developments should be based on the 100 year flood level plus 500 mm of freeboard (the Flood Planning Level – FPL) to cater for uncertainties such as wave action, effects of flood debris conveyed in the overland flow stream and precision of modelling. The sensitivity studies of **Section 6.3** below provide guidance on the suitability of the recommended 500 mm allowance for freeboard under present day climatic conditions.

In accordance with DECCW recommendations (DECCW, 2007) sensitivity studies have also been carried out in **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 the FPL. However, final selection of the FPL will also depend on a number of other factors identified in **Section 6.4** and is a matter for more detailed consideration in the *Floodplain Risk Management Study*.

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 Floodplain Development Manual, 2005.

Flood prone areas may be provisionally categorised into *Low Hazard* and *High Hazard* areas depending on the depth of inundation and flow velocity. Flood depths as high as 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 5, 20 and 100 year ARI floods on Sugarloaf Creek based on Diagram L2 of the *Floodplain Development Manual, 2005* are presented in **Figures 6.7** to **6.9**.

For the minor flood events (5 year ARI) there is an isolated area of high hazard in the open space area on the upstream side of High Street, as well as a small area in an allotment on the southern side of Robert Street downstream of First Avenue. The other areas of high hazard downstream of Fourth Avenue and the Eastern Valley Way are situated in sections of open channel. The extents of high hazard increase with increasing flood magnitude. At the 100 year ARI a ribbon of high hazard extends along the overland flow path downstream of Penshurst Street.

There are no new flood paths developed as flood magnitudes increase. Floodwaters continue to follow the line of the original creek, with only a gradual widening of extent and increase of peak levels. Similarly, the likely increase in flood flows associated with future climate change would not result in the development of new flow paths or a sudden increase in hazard in existing flooded areas (see **Section 6.4**).

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

6.2.2. Floodways

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

> ➢ 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 redistribution 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.

Flood storage effects are not significant on Sugarloaf Creek as here is very little storage in the overbank areas. Peak flood levels are primarily determined by the conveyance capacity of the waterway and by definition, most of the conveyance is located within the floodway. For this reason the floodplain was sub-divided into floodway and flood fringe areas only.

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.

The extent of the floodway was assessed by taking into account factors such as the distribution and magnitude of flow velocity vectors and the depths of inundation over the flooded area (see **Figure 5.2** for typical information on the pattern of flow used for this assessment). On this basis the extent of the floodway closely corresponds with the provisional high hazard zone, with some adjustments in areas where the flooding is of a ponding nature and the high hazard categorisation is primarily based on depth.

The assessed 100 year ARI floodway is shown on **Figure 6.10**. It generally follows the line of the creek. Some of the roads approaching the main arm of the creek and several running parallel with it also act as floodways and have been incorporated in the assessment. As noted previously 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.

6.3   Sensitivity Studies

A number of runs were carried out to test the sensitivity of the hydraulic model results to variations in model parameters such as hydraulic roughness, blockage of pipes and the effects of likely increases in rainfall intensities resulting from future climate change. 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 *Floodplain Risk Management Study* for the catchment. The results are summarised in the following sections.

6.3.1. Sensitivity to Hydraulic Roughness

Figure 6.11 shows 100 year ARI discharge and stage hydrographs in the channel at the entrance to the culvert beneath the Eastern Valley Way for several values of hydraulic roughness for overland flow conveyed through allotments. The "best estimate" of roughness adopted for design purposes was 0.05 and resulted in a total flow of 39.6 m^3/s in the channel, of which 18.8 m^3/s surcharged the road. Increasing the allotment roughness to 0.2 resulted in a considerable attenuation of flow from flood storage effects, with the total flow reducing to 32 m^3/s of which 11.2 m^3/s surcharged the road. These values compare with a total discharge of 54 m^3/s (Rational Method) and 59 m^3/s (DRAINS) and show the attenuating effect on flows of increasing the hydraulic roughness in the model. Increasing the allotment roughness to 0.2 in the areas upstream of the Eastern Valley Way resulted in a reduction of 0.2 m in peak water levels in the channel because of the reduction in peak flows at that location.

Figure 6.12a shows the difference in peak 100 year ARI flood levels (i.e. the "afflux") resulting from an assumed roughness of 0.2 in allotments, compared with the best estimate value of 0.05. Along the main arm the higher roughness provides additional resistance to the passage of flow causing the flow to lose momentum. Water is detained in allotments, resulting in an increase in peak flood levels which averages about 100 to 200 mm, but reaches up to 500 mm in isolated locations. Depths and flows may reduce in downstream open areas due to the reduction in flows resulting from increased roughness upstream (e.g. in Butt Park, in the Bus Depot and in Carlson Park).

On several of the overland flow paths approaching the main arm from the lateral catchments, where the overland flow path runs parallel with the road, a higher roughness in allotments provides resistance to flow spreading perpendicular to the principal direction of flow. This effect can result in a small reduction in flood extents.

Figure 6.12b shows areas of the floodplain where the afflux is 300 mm or greater due to the increase in allotment roughness. This figure also identifies areas where land is rendered flood free, or where additional areas of land are flooded.

There are only very localised areas where the increase in roughness would result in afflux greater than 300 mm. Generally an allowance of 200 to 250 mm would cater for increases in flood levels resulting from uncertainties in hydraulic roughness.

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 the blockage scenario which could reasonably be expected to occur and is presented for illustrative purposes.

Figure 6.13 shows the increase in peak 100 year ARI flood levels (i.e. the "afflux") resulting from a 50 per cent blockage. The average increase from a global blockage would be around 100 mm. Increases of up to 300 mm could result in very isolated areas along the main arm because of the resulting increases in overland flow. On the lateral sub-catchments, the effects of blockage on overland flows approaching the main arm are generally not significant.

A 500 mm freeboard allowance would be sufficient to cater for the effects of blockage plus uncertainties in the estimate of roughness in the floodplain.

6.4 Climate Change Sensitivity Analysis

6.4.1. General

The weight of scientific evidence shows that climate change will have adverse impacts on sea levels and rainfall intensities. The significant 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.

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 *Floodplain Risk Management Study* for Sugarloaf Creek, using site specific data.

CSIRO prepared reports for the NSW Government on the impacts of climate change on rainfall intensities in the major river basins in the state (CSIRO, 2007). In the Sydney Metropolitan catchments, the 40 year ARI, 1 day rainfall was predicted to change by between -3 per cent to +12 per cent by 2030 and by between +1 per cent to +10 per cent by 2070.

DECCW recommends that its guideline *Practical Considerations of Climate Change, 2007* be used as the basis for examining climate change in projects undertaken under the State Floodplain Management program and the *Floodplain Development Manual, 2005*. 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.

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 pattern 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 *Floodplain Risk Management Study* 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 Sugarloaf Creek, it will also be necessary to consider the impact of decisions on the existing streetscape.

Mitigating measures which could be considered in the *Floodplain Risk Management Study* 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. Scope of Investigation

As mentioned, the investigations undertaken at the flood study stage are mainly seen as sensitivity studies pending more detailed consideration in the *Floodplain Risk Management Study*.

For the purposes of the investigation and as recommended by DECCW, design 100 year ARI rainfall intensities for the critical 60 minutes storm were increased by 10 per cent and 30 per cent to test the sensitivity on flood behaviour in the catchment. **Figure 6.14** shows the stage and discharge hydrographs on the upstream side of the Eastern Valley Way for present day conditions and for the two increased rainfalls. Peak flood levels at this location would increase by 100 mm for the 10 per cent increase in rainfalls and by 200 mm for the 30 per cent increase.

Figure 6.15a shows the increased extents of inundation for the two cases, compared with present day conditions. **Figure 6.15b** shows the afflux resulting from an increase of 10 per cent in 100 year ARI rainfall intensities. The maximum increase in flood levels would be 200 mm.

Figure 6.15c shows the afflux for a 30 per cent increase in rainfall intensities. The increase along the main arm of the creek would be between 100 and 300 mm. **Figure 6.15d** shows the very isolated areas along the main arm where the afflux lies within the 300 to 500 mm range.

6.4.3. Effects of Increased Rainfall Intensities

The impact on *flooding patterns* may be summarised as:
- A gradual widening of the extent of inundation along the length of the main arm of Sugarloaf Creek.
- A small increase in flow velocities within the inundated area running along the main arm, but no sudden increase in the provisional flood hazard due to increased flood depths and flow velocities.

> ➢ No islands or new flow paths would be created. Flow would continue to follow its existing course along the valley of the creek.

> ➢ There may be a reduction in the time of rise of the floodwaters. Sugarloaf Creek is flash flooding with little warning time available to residents (there is about one hour 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 Council and SES is required to limit risks to people and property. Further consideration of flood warning arrangements and strategies will be undertaken in the *Floodplain Risk Management Study*.

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APPENDIX A

HISTORIC FLOODS AND MODEL TESTING

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LIST OF FIGURES

ATTACHMENTS

1   INTRODUCTION

This Appendix deals with the following matters:

➢   The results of the Community Consultation process aimed at collecting data on flooding on the Sugarloaf Creek catchment,

➢   The review of historic flood information contained in the previous Sugarloaf Creek flood investigation carried out by Lyall and Macoun Consulting Engineers (LMCE, 1988),

➢   The results of testing the hydrologic and hydraulic models for historic storm events.

Three historic storms were identified when instances of flooding occurred within the catchment: 30 April 1988, 5 August 1986 and 10 April 1998. Significant surcharges of the stormwater drainage system and flooding in adjacent allotments were reported for all of these events. However, there are very limited data available, or recollection of historic flooding by residents, probably because of the extended flood free period since the last major storm on the catchment.

The main sources of flood related information were contained in the LMCE, 1988 study together with recorded rainfall data. Pluviographic data for the three historic storms were recorded at the Chatswood Bowling Club, as well depths at daily-read rain gauges at Northbridge Bowling Club and Rosebridge Avenue in Castle Cove. Recorded rainfalls were analysed and applied to the DRAINS catchment model to estimate discharge hydrographs, which were then applied to the TUFLOW model of the floodplain and overland flow paths.

Section 2 of this Appendix deals with the collection of data and includes a description of observed flood behaviour on the catchment, taken from the LMCE, 1988 investigation.

Section 3 describes the results of testing the models for the three historic floods and compares the results with observed behaviour.

2 COLLECTION OF HISTORIC FLOOD DATA

2.1 Community Newsletter

A Community Newsletter was prepared and distributed to residents bordering the creek and its floodplain to gain knowledge of flood behaviour in the study area. A total of 500 Newsletters were distributed and 31 responses were received, which are summarised in **Attachment A**.

Some respondents were able to identify dates of flooding. However, there was limited information relating to specific flooding patterns. One resident recalled that in 1998 flooding occurred at the corner of High Street and Rosewall Street; in Penshurst Street near Forsyth Street and at the bus depot on Stan Street. Another resident recalled flooding in residential allotments on the western side of High Street in 1986, prior to the voluntary purchase of Numbers 150 – 152 by Council and the conversion of those allotments to open space.

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 occurred.

Eight of the respondents were able to identify dates of significant flooding. However, only one could recall more than two flood events. This resident ranked the floods of 1998, 1988, 1986, 1984 in order from highest to lowest.

A summary of the number of respondents who could recall specific flooding events is given in **Table A2.1**.

TABLE A2.1
NUMBER OF RESPONSES
RELATING TO HISTORIC FLOODS

1984	1986	1988	1994	1998
2	5	3	1	3

Unfortunately from the information obtained there were no specific recorded flood height data that would be of use in the calibration of the hydrologic/hydraulic models. The main source of information on flooding patterns in the Sugarloaf Creek catchment is the LMCE, 1988 Flood Study which is described in the following section.

2.2 Sugarloaf Creek Flood Study (LMCE, 1988)

The LMCE, 1988 study was commissioned by Council following the occurrence of several storms in the 1980's which had caused significant flooding on the main arm of Sugarloaf Creek. The report was prepared shortly after the occurrence of the April 1988 storm which, although only a 2 year ARI event in terms of rainfall intensities had resulted in flooding along the length of the creek from Penshurst Street to the Eastern Valley Way.

The initial breakout point in April 1988 was in the reach downstream of Mabel Street (refer **Figure 1.1** of the Main Report which shows the layout of the catchment and its drainage system). Overflows ponded in the low point in the road in Penshurst Street and flooded the underground garage of the block of units at number 117-121 on the eastern (downstream) side of that street. Overland flows then followed the line of the creek which runs as an underground system through the backyards of properties located between Rosewall Street and Hollywood Crescent and entered the ponding area on the western side of High Street. Flood levels in this area which was at the time occupied by several residences are controlled by the raised level of High Street.

Several of the worst affected properties were subsequently purchased by Council and the area was converted to open space. Water was reported to have flowed over High Street at a depth of 100 mm and continued through backyards along the former line of the creek to Ann Street. Ponding occurred in the low spot in that road with water eventually flowing into the Willoughby Bus Depot. Floodwaters also entered the Bus Depot from the direction of Stan Street which is located on its northern side. The 750 mm diameter tributary line draining the northern part of the catchment runs beneath Stan Street and joins the main drain within the Bus Depot. It eventually became overloaded and was partly responsible for the flooding in the Bus Depot.

A similar pattern of ponding in the low points of the streets and overland flow through the backyards was maintained downstream of First Avenue. The occurrence of above-floor level inundation was confined to the residence of the western side of the culvert running beneath the Eastern Valley Way. However, the floors of several houses in the sag points of Third and Fourth Avenue were almost flooded. Of those, Number 56 on the western side of Fourth Avenue was worst affected. Considerable scouring of the backyard and damages to fences occurred. The allotment of Number 54 on the western side of Third Avenue also experienced scouring.

At the Eastern Valley Way, the peak flood level was about 100 mm below the level at which water flows over the road. Hydraulic calculations gave an estimated peak discharge of 19 m^3/s, with the road culvert flowing under inlet control. No surcharging of the culvert downstream of the Eastern Valley Way occurred. However several properties in Sunnyside Crescent experienced drainage problems due to overflow of the lateral sub-catchment drainage system.

Following the preparation of the LMCE, 1988 report a physical hydraulic model study of the culvert beneath the Eastern Valley Way was commissioned by Council. That study was carried out at the University of Technology and reported in UTS, 1989. The study led to the detailed design and construction of improvements to the inlet of the culvert aimed at increasing its hydraulic capacity (LMCE, 1990).

The LMCE, 1988 report also presented the results of an analysis of the rainfalls associated with the August 1986 storm which was a considerably larger event than the April 1988 storm. This analysis was repeated for the present investigation, along with analysis of the more recent storm which occurred in April 1998. The results are described in the next section.

2.3 Analysis of Historic Storm Rainfall Data (Present Report)

Australian Water Technologies (AWT) supplied rainfall intensity data for the pluviometer at the Chatswood Bowling Club, which is located on the Pacific Highway about 2 km to the west of the centroid of the Sugarloaf Creek catchment.

The Bureau of Meteorology sponsors a daily rain gauge at the Northbridge Bowling Club, situated to the south of the catchment. Other daily rain gauges are located at Castlecove (Rosebridge Avenue) on the eastern side of Eastern Valley Way to the north of Sugarloaf Creek and at the Gordon Bowling Club (**Figure A2.1**). **Table A2.2** shows depths of rainfall for the 24 hour period encompassing the most intense burst of the storm.

These data were used to assess the areal distributions and temporal patterns of rainfall experienced on the Sugarloaf Creek catchment for the August 1986, April 1988 and April 1998 storms and to test the models for those storms.

Figure A2.2 shows cumulative depths of rainfall recorded at the Chatswood Bowling Club for each of these storm events.

The 10 April 1998 storm had the most intense rainfalls of the three storms, with the most intense burst occurring over the 30 minute period from 11:50 am to 12:20 pm on that day, when 72.5 mm fell. Over the 1 to 2 hours durations which maximise flows in the Sugarloaf Creek catchment, the rainfall intensities approximated a 50 year ARI storm.

The most intense burst of rainfall for the 10 April 1998 storm is incorporated in the daily rainfall for the "rainday" of 11 April 1998 (the 24 hour period ending at 09:00 hours on that day), when the total depth recorded at the Chatswood Bowling Club daily gauge amounted to 222 mm. At Castlecove, the daily rainfall was 279 mm, about 25 percent higher than at Chatswood and at Northbridge, the daily rainfall was 175 mm about 20 percent lower. The daily rainfall at Chatswood is about equal to the average of the depths recorded at the other two stations. On this basis, it is considered that the Chatswood record is representative of rainfall intensities experienced over the Sugarloaf Creek catchment during this storm and could be adopted to assess runoff in the flood models.

The 5 August 1986 storm was a long duration event, with about 300 mm of rainfall being experienced at Chatswood over the 24 hour period from 00 to 2400 hours on that day. Rainfall intensities were less than in the 10 April 1998 storm and were around 20 year ARI for the critical 1 to 2 hours duration on Sugarloaf Creek. The bursts of storm rainfall responsible for this flood were included in the depth recorded for the rainday of 6 August 1986. Daily rainfall at Northbridge was considerably less than at the other two stations. However, according to resident reports (LACE, 1988) this was a significant flood event and consequently Northbridge may not have been representative of rainfalls experienced over the Sugarloaf Creek catchment.

The 30 April 1988 storm was also a long duration event, with 220 mm of rainfall recorded over the 17 hour period between 2200 hours on 29 April 1988 and 1700 hours on 30 April 1988**.** The most intense period of rainfall was recorded between 1500 hours and 1600 hours on 30 April (included in the rainday of 1 May 1988), with the average 1 hour intensity over this period approximating a 2 year ARI storm. Daily rainfalls at Northbridge and Chatswood were similar, with a lesser daily rainfall recorded at Castlecove.

TABLE A2.2
DAILY RAINFALLS* - mm

Location	11 Apr 1998^	6 Aug 1986^	1 May 1988^
Chatswood B.C.	222	317	145
Northbridge B.C.	175	248	149
Castlecove (Rosebridge Ave)	279	335	117
Gordon Golf Club	166	253	253

* Rainfall depths are for the 24 hours ending at 09:00 hrs on the day shown.

^ This rainday encompasses the rainfall period responsible for the flood event.

3 TESTING HYDROLOGIC AND HYDRAULIC MODELS

3.1 Procedure Adopted for Testing the Models

The procedure adopted for testing the DRAINS model of Sugarloaf Creek, in situations where historic flood data are available, would involve the collection and analysis of rainfall data to ascertain the temporal and areal distribution of rainfall over the catchment. These rainfalls would then be applied to the model to generate flows within the catchment.

In situations where there was a stream gauging station located on the catchment, the modelled discharge hydrograph would then be compared with historic hydrographs and model parameters varied until a fit was achieved. Similarly, when sufficient data are available on historic flood levels along the channel it is possible to use the known discharges and adjust the parameters of the hydraulic model to achieve a fit between recorded and modelled levels. Thus it would be possible to achieve independent calibration of each of the models (hydrologic and hydraulic) in turn. However, in most situations the streams are not gauged and data is usually limited to some isolated flood marks along the stream plus some recorded rainfall data.

Under those circumstances, independent "calibration" of the models cannot be achieved. The usual procedure adopted is to use realistic values of the hydrologic model parameters, adopted from experience and the engineering literature, in conjunction with recorded rainfall data to estimate flows and to vary the parameters of the hydraulic model to achieve a reasonable agreement with recorded flood levels. Sometimes the recorded flood marks or levels recorded at structures are used in conjunction with uniform flow or culvert formulae to estimate historic flood flows to assist with the selection of model parameters. However, in the absence of recorded stream flow data, the overall process as outlined above can at best be termed "model tuning" or "model testing" rather than calibration.

In the case of Sugarloaf Creek, the only quantitative data were the rainfall depths recorded at the Chatswood Bowling Club. Therefore in the present study, the experience of the investigators dictated the choice of parameters for both the hydrologic and hydraulic modelling phases of the analysis.

3.2 DRAINS Model

Pluviographic data for the three historic storms identified in **Section 2**, as recorded at the Chatswood Bowling Club, were applied to the DRAINS model to generate discharge hydrographs, which were then applied to the TUFLOW model.

3.3 DRAINS Model Parameters

Model testing was undertaken with the following parameters:

- ➢ Soil Type = 3.0 (assessment of a soil's rate of infiltration.)
- ➢ AMC = 3.0 (Antecedent Moisture Condition – assessment of a catchment's wetness at the start of storm event).
- ➢ Paved area depression storage = 2.0 mm.
- ➢ Supplementary area depression storage = 1.0 mm.
- ➢ Grassed area depression storage = 10.0 mm.

In addition, the manning's 'n' roughness value for the pipes was assumed to be 0.012, as recommended in the DRAINS Manual.

Pit loss coefficients were assigned with values adopted in accordance with Missouri Charts, the DRAINS manual and various technical papers. A response time of 5 minutes was adopted in the residential areas. In addition, the overland flow path was adjusted to closely follow the pattern of the street system. The resulting flow length and slope was then used by DRAINS to assess the travel time of the flood wave.

3.4   DRAINS Model Results for Historic Floods

Modelled peak flows are shown on **Table A3.1**. These flows represent the total discharge passing the nominated locations, including both overland and piped flows. Peak flows derived by TUFLOW, which routes the floodwave through both the piped system and overland flow paths, independently of DRAINS are also shown for comparison purposes.

TABLE A3.1
HISTORIC STORMS
MODELLED PEAK FLOWS
(m³/s)

Location	April 1998	August 1986	April 1988
Sydney Street	8.5 (7.7) *	5.6 (5.5)	2.8 (2.5)
Penshurst Street	20.0 (15.7)	13.8 (11.7)	7.1 (6.5)
High Street	32.1 (28.2)	22.6 (20.4)	13.1 (10.1)
Second Ave	44.0 33.8)	30.7 (24.9)	16.7 (13.0)
Eastern Valley Way	54.0 (40.7)	37.5 (29.3)	23.2 (16.0)

* Values in brackets are peak flows at the same location generated by TUFLOW

3.5   TUFLOW Model Results for Historic Floods

3.5.1   Presentation of Results

Indicative flood extents, peak water surface contours and depths of inundation as computed by the TUFLOW model are shown on **Figures A3.1** to **A3.3** for the April 1988, August 1986 and April 1998 floods. Discharge and stage hydrographs for each flood at the three important road crossings on the main arm of Sugarloaf Creek are shown on **Figure 5.3a** and **Figure 5.3b** of the Main Report. These figures separately show piped and overland flows.

3.5.1   Comparison of TUFLOW Results with Observed Flood Behaviour

Attachments B to **D** show details of flood inundation for the three floods at the three important road crossings.

Attachment B – April 1988 Storm

At Penshurst Street the model predicted surcharge of the piped system for all floods. For the April 1988 storm, flooding in the street extended as far as the underground car park at No. 117 – 121. At High Street the ponding on the western side of the road surcharged the road centreline as reported. At the Eastern Valley Way, the hydraulic modelling was carried out with the improved culvert inlet in place. Hence the model results gave a larger freeboard against overtopping the road than was actually experienced at the time. The model predicted that all of the flow would be conveyed via the culvert. The modelled freeboard against overtopping (with the improved inlets) was about 1.5 m, compared with the observed freeboard of 100 mm. The model predicted surcharging of the eastern gutter of Ann Street with overland flow through the Willoughby Bus Depot.

Attachment C – August 1986 Storm

For the August 1986 flood up to 600 mm of surcharge was predicted in the street system at Penshurst Street with overland flows extending into Rosewall Street. About 300 mm of surcharge was predicted over High Street. The Eastern Valley Way would have been flooded even if the improved inlets to the culvert had been in place in 1986. Overland flows would have traversed the residential allotments downstream of the Eastern Valley Way between Sugarloaf and Sunnyside Crescents.

Attachment – D April 1998 Storm

Modelled surcharges of the drainage system became more severe for the April 1998 storm compared with the earlier events.

4 REFERENCES

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

University of Technology, (1989). *"Hydraulic Model Study of Sugarloaf Creek at Eastern Valley Way"*

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