Newport Beach Flood Study

Report Prepared For

Pittwater Council

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FOREWORD

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

Under the policy, the management of flood prone land is the responsibility of Local Government. The State Government 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 State Government through the following sequential stages:

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

The Newport Beach Flood Study is the first stage of the management process for the Newport Beach catchment. This study has been prepared for Pittwater Council by Lawson & Treloar Pty Ltd to define flood behaviour under current catchment conditions.

EXECUTIVE SUMMARY

A flood study of the Newport Beach catchment has been undertaken to define the nature and extent of flooding in the area for a range of design rainfall events. The study has been carried out for the existing catchment conditions. The study was first completed in November 1999. This revision to the Newport Beach Flood Study incorporates new topographic survey data for the purpose of more accurately representing the flow regimes in critical areas of the catchment. Refinements to the modelling systems utilised include the revision of the North, Robertson Road and Barrenjoey Road branches of the hydraulic model, as well as re-schematisation of the North branch, Barrenjoey Road and West branch confluence. Additional flow paths were also incorporated for Howell Close, Foamcrest Avenue and Coles Parade. The hydraulic model (MIKE11) was also transferred and upgraded to a newer release of software.

The Newport Beach catchment has an area of 1.79 square kilometres. The catchment is fully urbanised with residential and commercial land uses. Floodwaters from the catchment accumulate near the commercial area and discharge into the ocean.

In the past many major flooding events have occurred, which have caused extensive damage to public and private property, restricted access and have been a general inconvenience to the residents. Above-floor flooding in the lower reaches of the catchment caused extensive damage during the events of March 1977, October 1987 and May 1988.

Estimation of flooding behaviour was undertaken by developing two mathematical models to simulate the hydrologic and hydraulic aspects of flooding. The hydrological modelling package RAFTS (1994) was utilised to determine catchment runoff and for routing flows downstream through the catchment. Predicted hydrographs from RAFTS were then input to the hydraulic model MIKE-11 (2000) for the determination of peak flood levels, velocities and flood extents for various design rainfall events. These events included the 1%, 2%, 5%, 20% annual exceedance probability events (AEP) together with the Probable Maximum Flood (PMF). Both surface and trunk drainage pipe flows were modelled.

Three historical flood events were selected for calibration and validation. The flooding events were chosen on the basis of flood level information obtained from the resident survey and the available rainfall data.

Design rainfall intensities and temporal patterns for the required rainfall events were obtained from Australian Rainfall and Runoff (1987).

The model results reflect the observed flooding regime of the area. The area adjacent to the main concrete lined drain in the lower portion of the catchment is severely affected. This area includes commercial areas on Barrenjoey Road, residential areas in Ross Street and Council's car park near Bramley Avenue. Barrenjoey Road is also flooded where it intersects the main drain near The Boulevard for design storm events with a frequency of 20% AEP and rarer.

The impact of ocean water levels on flooding was considered. The results indicate that catchment flooding is insensitive to the ocean water levels. Only during rare and very severe floods is ocean water level likely to impact on drainage capacity and cause a small increase in flood level. However, it is important to note that flooding is also possible as a result of ocean inundation by wave overtopping of the frontal dune. This flooding mechanism is not considered in this report.

Limits of predicted flooding extents for the PMF, 1% AEP, 2% AEP, 5% AEP and 20% AEP events and the 1% AEP + 0.5m extent are provided on a cadastral plan. Tabulated modelling results, which include predicted flood levels, velocities and flows at a number of locations in the floodplain, are also provided.

This study provides the management tool in the form of a hydraulic model for assessing floodplain management options in the study area under the next stage of the Floodplain Management Process - the Floodplain Risk Management Study.

GLOSSARY OF TERMS*

Annual Exceedence Probability (AEP)	Refers to the probability or risk of a flood of a given size occurring or being exceeded in any given year. A 90% AEP flood has a high probability of occurring or being exceeded; it would occur quite often and would be relatively small. A 1% AEP flood has a low probability of occurrence or being exceeded; it would be fairly rare but it would be relatively large.
Australian Height Datum (AHD)	A common national plane of level corresponding approximately to mean sea level.
cadastre, cadastral base	Information in map or digital form showing the extent and usage of land, including streets, lot boundaries, water courses etc.
Catchment	The area draining to a site. It always relates to a particular location and may include the catchments of tributary streams as well as the main stream.
Design Flood	A significant event to be considered in the design process; Various works within the floodplain may have different design events; some roads may be designed to be overtopped in the annual flood event.
Development	The erection of a building or the carrying out of work; or the use of land or of a building or work; or the subdivision of land.
Discharge	The rate of flow of water measured in terms of volume over time. It is to be distinguished from the speed or velocity of flow, which is a measure of how fast the water is moving rather than how much is moving.
Flash Flooding	Flooding which is sudden and often unexpected because it is caused by sudden local heavy rainfall in another area. Often defined as flooding which occurs within 6 hours of the rain, which causes it.
Flood	Relatively high stream flow which overtops the natural or artificial banks in any part of a stream , river or drainage system.
Flood Hazard	Potential risk to life and limb caused by flooding.
Flood Prone Land	Land susceptible to inundation by the probable maximum flood (PMF) event, i.e. The maximum extent of flood liable land. Floodplain management plans may encompass all flood prone land, rather than being restricted to land subject to designated flood events.
Floodplain	Area of a river valley adjacent to the river channel, which is subject to inundation by the probable maximum flood event.
Floodplain Management Measures	The full range of techniques available to floodplain managers.
Floodplain Management Options	The measures which might be feasible for the management of a particular area.

Flood Storages	Those parts of the floodplain that are important for the temporary storage of floodwaters during the passage of a flood.
Floodway Areas	Those areas of the floodplain where a significant discharge of water occurs during floods. They are often aligned with naturally defined channels. Floodways are areas which, even if only partially blocked, would cause a significant redistribution of flood flow, or significant increase in flood levels. Floodways are often, but not necessarily, areas of deeper flow or areas where higher velocities occur. As for flood storage areas, the extent and behaviour of floodways may change with flood severity. Areas that are benign for small floods may cater for much greater and more hazardous flows during larger floods. Hence, it is necessary to investigate a range of flood sizes before adopting a design flood event to define floodway areas.
Geographical Information Systems (GIS)	A system of software and procedures designed to support the management, manipulation, analysis and display of spatially referenced data.
Hydraulics	The term given to the study of water flow in a river, channel or pipe, in particular, the evaluation of flow parameters such as stage and velocity.
Hydrograph	A graph that shows how the discharge changes with time at any particular location.
Hydrology	The term given to the study of the rainfall and runoff process as it relates to the derivation of hydrographs for given floods.
Integrated Survey Grid (ISG)	ISG is a global co-ordinate system based on a Transverse Mercator Projection. The globe is divided into a number of zones, with the true origin at the intersection of the Central Meridian and the Equator.
Mainstream Flooding	Inundation of normally dry land occurring when water overflows the natural or artificial banks of the principal watercourses in a catchment. Mainstream flooding generally excludes watercourses constructed with pipes or artificial channels considered as stormwater channels.
Management Plan	A document including, as appropriate, both written and diagrammatic information describing how a particular area of land is to be used and managed to achieve defined objectives. It may also include description and discussion of various issues, problems, special features and values of the area, the specific management measures which are to apply and the means and timing by which the plan will be implemented.
Mathematical/ Computer models	The mathematical representation of the physical processes involved in runoff and stream flow. These models are often run on computers due to the complexity of the mathematical relationships. In this report, the models referred to are mainly involved with rainfall, runoff, pipe and overland stream flow.
Met-ocean	Derived from meteorological and oceanographic phenomenon
Peak Discharge	The maximum discharge occurring during a flood event.

Probable Maximum Flood	The flood calculated to be the maximum that is likely to occur.
Probability	A statistical measure of the expected frequency or occurrence of flooding. For a fuller explanation see Annual Exceedence Probability.
Runoff	The amount of rainfall that actually ends up as stream or pipe flow, also known as rainfall excess.
Stage	Equivalent to 'water level'. Both are measured with reference to a specified datum.
Stage Hydrograph	A graph that shows how the water level changes with time. It must be referenced to a particular location and datum.
Stormwater Flooding	Inundation by local runoff. Stormwater flooding can be caused by local runoff exceeding the capacity of an urban stormwater drainage system or by the backwater effects of mainstream flooding causing the urban stormwater drainage system to overflow.
Wave Set-up	Increase in water level landward of breaker line caused by conservation of wave momentum flux.
Topography	A surface which defines the ground level of a chosen area.

* Many of the terms have been adopted from the NSW Floodplain Management Manual (2001)

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1. INTRODUCTION

Newport is a fully urbanised suburb within the Pittwater Council Local Government area. The catchment, which drains into the Tasman Sea at Newport Beach, has an area of 1.79 square kilometres. Extents of the catchment, together with the location of study area in relation to neighbouring suburbs, are provided in Figure 1.1.

In the past, flooding in the catchment has caused property damage and poses a severe hazard to the residents in the low lying areas near the ocean. This has prompted Pittwater Council, through the Pittwater Coast, Estuary and Floodplain Management Committee, to follow the stages in the preparation of a comprehensive Floodplain Risk Management Plan for the Newport Beach Floodplain. This is part of the State Government's program to manage major flood impacts and hazards in the floodplains, in accordance with the State Government's Floodplain Management Manual, 2001.

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

This version of the Newport Beach Flood Study incorporates a revision of the hydraulic model used to determine the existing flood conditions. The revisions to the hydraulic model are based on additional survey data for the purpose of more accurately representing the flow regimes in critical areas of the catchment. Changes to the model include the revision of the Northern, Robertson Road and Barrenjoey Road branches of the hydraulic model, as well as re-schematisation of the North branch, Barrenjoey Road and West branch confluence. Additionally new flow paths were incorporated for Howell Close, Foamcrest Avenue and Coles Parade. The hydraulic model was also transferred to a newer release of software (MIKE11).

This version of the flood study also assesses the likelihood and implications of culvert blockages in the catchment. Sensitivity analysis was undertaken to examine the impacts that culvert blockage would have on a critical reach of North branch in the vicinity of Howell Close. This new work is incorporated in Section 7.3.

Various components of the flood study can be grouped together in three stages. First, a review and compilation of available data. This included the collection of available historical rainfall and flood level data. Historical flood data was gathered through an extensive resident survey and perusal of past issues of the local newspaper, *The Manly Daily*. Second, a full hydrologic investigation was carried out for the catchment. Third, a hydraulic model of the major natural and constructed drains in the catchment was established and calibrated using the historical flood level data. The hydraulic model was then used with design rainfall conditions to simulate flood behaviour in the catchment.

The hydraulic model developed in this study simulates flooding which may occur under existing catchment conditions. The model may be used to investigate various management and flood mitigation options and can assist in defining the long-term Floodplain Risk Management Strategies.

2. STUDY METHODOLOGY

The objectives of the Flood Study were to:

- Identify all the flood related data by searching all relevant data sources (Section 3).
- Determine the likely extent and nature of historical flooding by carrying out a survey of residents in the catchment (Section 3).
- Define existing flood behaviour for the mainstream flooding in the catchment (Sections 4, 5, 6 and 7).
- Define design flood levels, velocities and flow distributions for the catchment (Section 8).
- Map the extent of flooding for the design flood events considered including the 1% AEP, 2% AEP, 5% AEP, 20% AEP, Probable Maximum Flood (PMF) and 1% AEP + 0.5 m (flood planning level) (Figures 8.1 to 8.6).

Three numerical modelling tools were developed:

- A hydrologic model to convert rainfall on the catchment into runoff (Section 4). The hydrologic model combines rainfall information with local catchment characteristics to estimate a runoff hydrograph. Figures 2.1 and 2.2 describe the sub-catchments used in this analysis.
- A hydraulic model to convert runoff into water levels and velocities throughout the stormwater system (Section 5). The model simulates the hydraulic behaviour of the water within the study area both for surface flow and underground pipe flow. It relies on boundary conditions which include the runoff hydrographs produced by the hydrologic model and the tail water levels in the Tasman Sea.
- Numerical wave propagation and surf zone models were developed to describe wave propagation to the site, together with wave set-up and wave overtopping potential at the site (Section 6).

Historical rainfall and flood record data are described, and are used in the established hydrologic and hydraulic models.

Peak water levels, discharges and velocities together with their critical duration are presented in Appendix A.

Plots of flood profiles in critical locations are included as Figures 7.1 to 7.6.

3. DATA COMPILATON AND REVIEW

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

3.1. HISTORICAL CATCHMENT LAND USE

Historically, the lower part of the catchment near the shops and Council car park was a ponding area called Farrels Lagoon (Jennings, 1987). As a result, flooding is known to be severe in this part of the catchment. Properties in surrounding streets including Ross Street, The Boulevard and Bramley Avenue are also affected by flooding. Changes to watercourses and floodpaths over time mean that in the event of major flooding, Barrenjoey Road is overtopped by floodwaters near The Boulevard and significant overland flooding occurs along Robertson Road.

In response to past flood events, the stormwater drainage system has been improved over the years with major improvements being an increased ocean outfall capacity and the amplification of the Robertson Road drainage system.

3.2. COUNCIL RECORDS

Pittwater Council provided records of design for various components of the stormwater drainage system and associated modifications over time. Using these records, drainage system conditions were established for various historical storm events. The information was used in setting up the models for calibration purposes.

The records also contained design methodology for flood level determination for previous investigations. This information was used for comparison with the approach adopted in the present study.

3.3. ADDITIONAL GROUND SURVEY

A ground level survey for floodplain areas of the catchment was commissioned to develop the hydraulic model. Information was obtained to define the flow channels, significant overland flood flow paths and floodplain storage areas. Various flow control structures such as culverts and the beach dune were also surveyed. In addition, necessary data was obtained to describe the stormwater pipe network along the main tributaries of the catchment. This data included pipe sizes and inverts, pit sizes and pit inverts. This survey was undertaken by Mepstead and Associates.

In addition to the original survey commissioned for the flood study, a second survey was commissioned in December 2001 to more accurately define some flowpaths. These flowpaths included Barrenjoey Road, Robertson Road, Foamcrest Avenue, Coles Parade, Bramley Lane and Ocean Avenue. Additionally, detail was obtained for two private footbridges constructed over the Northern Channel for properties in Ismona Avenue.

Along with survey information collected specifically for this study, cross sections of the Northern Channel were obtained from SMEC Australia. These cross sections supplemented the detail of the northern flow path from above Howell Close and to the apartment block between Foamcrest Avenue and Barrenjoey Road.

3.4. HISTORICAL AND DESIGN STORM RAINFALL DATA

Historical storm events to be used for model calibration in the study were identified by cross referencing historical rainfall records for the Bureau of Meteorology daily rainfall station at Newport Bowling Club and the pluviometer station at the Warriewood Sewage Treatment Plan (STP) with flooding dates reported in the returned resident questionnaires (Section 3.5) and the local newspaper, *The Manly Daily* (Appendix C).

Table 3.1 lists the dates of the reported events.

Table 3.1: Newport Beach Catchment Flooding Events Reported by Residents

Date	Year
April	1973
March	1977
April	1983
November	1984
April	1984
October	1987
April	1988
January	1989
February	1990
February	1992
April	1997
December	1997
April	1998
August	1998

A review of the full series of daily rainfall records at Newport Bowling Club suggest that a number of heavy rainfall events have occurred in the past, in addition to those listed in Table 3.1. The more recent events were in February 1990, February 1992, April 1997, December 1997 and August 1998. Unfortunately, there were no recorded flood levels for these events and therefore they were not selected to provide calibration data for the study.

Jennings (1987) also reports that the catchment has experienced major storm events in February 1941 and 1943 (apparently ocean inundation rather than catchment flooding).

Whilst a number of floods have occurred, actual flood levels were reported only for the March 1977, April 1988, November 1984 and April 1998 events as part of the resident survey (Section 3.5) and were subsequently surveyed to AHD. However, no rainfall data was available for the April, 1988 flood. Reports of flooding from the local newspapers are included in Appendix C.

The historic events selected for calibration and validation were 1977, 1984 and 1998. These events also coincide with the three major stages in the development of the stormwater drainage system for the catchment. It should be noted that pluviograph data for the March, 1977 event was not available. The temporal pattern for this event was based on anecdotal evidence and information contained in AR&R (1987).

Design rainfall depths and temporal patterns for the 1%, 2%, 5% and 20% AEP events were developed using standard techniques provided in Australian Rainfall and Runoff, Vol. 1 and 2, (1987). Probable Maximum Precipitation (PMP) was estimated using the Generalised Short Duration Method (1994) as recommended by the Bureau of Meteorology. Design storm rainfall intensities for the full range of storm frequencies and durations are presented in Table 3.2.

Table 3.2: Design Rainfall Intensities (mm/h)

Frequency Duration	20%	5%	2%	1%	Probable Maximum Precipitation
10 min	123	157	183	202	*
20 min	91	117	137	151	*
30 min	74	96	113	126	476
1h	51	68	80	89	350
1.5h	40	53	63	69	301
2h	33	44	52	58	265
3h	26	34	41	45	214
6h	17	22	26	29	*

Note: * Rainfalls not calculated.

3.5. RESIDENT SURVEY OF RECORDED FLOOD LEVELS

A questionnaire was hand delivered to the residents living in the flood prone areas of Newport to collect historical flood information. The questionnaire sought information as to whether residents have experienced flooding, the nature and depth of flooding and the timing of such floods. Approximately 400 questionnaires were distributed and 136 residents responded, of which 36 had experienced flooding. A summary of information obtained from these questionnaires is tabulated in Appendix B. Of the residents surveyed, only a few people could recollect, with some degree of confidence, the level to which the floodwaters rose during reported floods. These levels were surveyed and are provided in Table 3.3. The information is also provided on a plan in Figure 3.1.

Table 3.3: Recorded Flood Levels (Based on Resident Interviews)

Flood Event	Locality	Reported Flood Level (m AHD)
March 1977	The Boulevard	3.56
March 1977	Ross Street	3.59
March 1977	Ross Street	3.57
March 1977	Ross Street	3.45
November 1984	Council Car Park (Bramley Ave)	3.22
April 1988	Foamcrest Avenue	4.96
April 1998	Bramley Avenue	3.03

3.6. CADASTRAL AND TOPOGRAPHIC DATA

Pittwater Council provided cadastral base information and contour files for use during the Flood Study in digital format. Data provided in this format was input to the MapInfo geographic information system (GIS).

Sub-catchment definition was undertaken directly using GIS following a site inspection and having regard to the 2m Land Information Centre (LIC) contour data.

Natural surface cross-sections were also located in MapInfo format, so that their locations could be determined for appropriate input to the hydraulic model.

3.7. WAVE DATA AND COASTAL DUNE CONDITIONS

Wave data is required for two purposes. First, during storms wave breaking in the nearshore zone causes wave set-up, which increases the water level above storm tide level at the flood discharge points to the sea. Therefore a time series of offshore wave data (H_s, T_z and direction) were required to enable the computation of nearshore waves and water levels. This data was used during the flood model calibration (Section 6).

Second, wave data is required to describe rare storms at average recurrence intervals (ARI) up to 100 years. These offshore wave parameters were also propagated shoreward to provide wave set-up for flood model design cases. Additionally, this data was used to describe wave set-up at the shoreline during major ocean storms as distinct from flood events. Combined with storm tide, inundation of the area behind the dune may occur because the frontal dune at the seaward end of the floodway has been lowered as part of Council's interim floodplain management measures. Elevated ocean water may flow over this lowered dune and/or wave overtopping may occur. The dune level adopted for this study was approximately 3.2m AHD. This level was derived from the survey of 1998 undertaken by Mepstead & Associates for this study (Section 3.3).

Time series wave data was available from two sources. Other than for 1998, wave data was obtained from the offshore Botany Bay Waverider buoy operated by Sydney Ports Corporation. Record lengths are typically 20 minutes. Until 1980 four records per day were recorded, with occasional more frequent records during storms. Since 1980 data has been recorded on an almost continuous basis. Time series

data is now also available from the Waverider buoy operated by Manly Hydraulics Laboratory at Long Reef (MHL, 1999).

Offshore wave directions for the calibration storms were determined using hindcasting methods applied to synoptic charts available for each event. This is a reasonable approach where measured offshore wave directions are not available.

Offshore design wave climate in terms of average recurrence intervals was developed using offshore Botany Bay wave data for the period 1971 to 1998. Only peak storm significant wave height (H_s) parameters greater than 5m were included in the analysis to ensure that the sample data was drawn from a population of extrema. The analysis was undertaken using the Method of Moments with the Extreme Value Type 1 Distribution. Table 3.4 presents offshore peak storm H_s parameters at selected ARI.

Table 3.4: Peak Storm Wave Height Occurrence Offshore Sydney

Average Recurrence Interval (years)	Peak Storm H_s (m)
5	8.0
20	9.2
50	10.0
100	10.6

Further details on the application of this data and coastal processes and ocean water levels can be found in Section 6.

3.8. PREVIOUS INVESTIGATIONS

Two previous investigations were identified for the Newport area that were relevant to this study:

- Department of Public Works (1992) Coastal Stormwater Planning Studies
- Warringah Shire Council (1987) Newport Amplification of Trunk Drains Investigation.

The 1992 study provided details of the hydrology of the catchment and the capacity of the trunk drainage system but no details of flood levels. The 1987 investigation involved an estimate of hydrology and peak water levels using hand calculations for the lower reaches of the floodplain.

4. CATCHMENT HYDROLOGY

4.1. GENERAL

The Newport Beach catchment is fully urbanised with an area of approximately 1.79 square kilometres. The floodwaters from the catchment flow into the Tasman Sea. The catchment elevation varies from sea level to 150m AHD.

The overland stormwater flow path is modified by the presence of the pipe drainage network. The catchment is fan-type where the travel time of floodwater from the northern, southern and western parts of the catchment to the area near Ross Street and the Council car park near Bramley Avenue is approximately the same. The physical aspects of the catchment and the presence of the control at the beach dune result in severe flooding in the Ross Street and car park areas. The northern parts of the catchment are characterised by steep slopes. The stormwater from these areas drains into a deep gully which carries one of the two major drainage tributaries in the catchment. The other major tributary runs from the flatter western parts of the catchment to the Tasman Sea. Details are shown in Figure 2.1.

Owing to the small area of the catchment, uniform areal distribution of both historical and design storms has been assumed in the hydrologic analysis.

The catchment is characterised by residential and commercial landuse. The commercial area and adjoining residential areas are developed on that part of the catchment which historically was a lagoon (Section 3.1). This lagoon acted as a storage area and also allowed passage for floodwaters towards the ocean.

The drainage system in the catchment has several major pipe and culvert drains which carry stormwater to the major surface drains. The main feature of the stormwater drainage system is an open lined drain which runs from Barrenjoey Road to the Council car park near Bramley Avenue. This drain joins two box culverts under the car park, which in turn join three 1500mm diameter pipes which act as the ocean outfall and discharge to the surf zone in the centre of Newport Beach.

4.2. ESTABLISHMENT OF HYDROLOGICAL MODEL

Runoff was estimated using the RAFTS (1994) rainfall/runoff-modelling package. There are six major subcatchments in the area (Figure 2.1), which were further divided for flow routing. The subcatchment layout as used in the RAFTS system is provided in Figure 2.2.

The catchment is fully urbanised and an imperviousness of 60% was assumed. This is based on analysis of aerial photography of the catchment along with Council's Local Environment Plan. This mapping indicates a significant level of medium to high-density housing along with the low-density housing. These land uses generally have a high level of imperviousness. The adopted imperviousness is a conservative assessment given that studies of urban residential land use by George et al (1998) indicate generally an impervious fraction of 40-50%.

The RAFTS subcatchments were based on the contour information and the physical layout of the streets in the subcatchment areas. The stormwater drainage system modifies the overland flow paths and the impact of this system is taken into account in the RAFTS sub-catchment layout. Using the RAFTS utility, each subcatchment is further divided to account for different initial/continuing rainfall loss rates for pervious and impervious areas of a subcatchment. The rainfall losses assumed in the model are shown in Table 4.1.

Table 4.1: Rainfall Losses Used in RAFTS

Catchment type	Initial Loss (mm)	Continuing Loss (mm/hr)
Pervious	10	1.5
Impervious	5	0

Other important parameters used in the development of RAFTS model are given in Table 4.2.

Table 4.2: RAFTS Model Parameters

RAFTS parameter	Pervious Area	Impervious Area
Catchment Manning's 'n'	0.03	0.02
% Impervious	5	100
Hydrograph Routing Lag	Based on a flow velocity of 1 m/s	

4.3. MODEL CALIBRATION

There are no stream flow gauges in the study area and hence the hydrological model could not be calibrated directly. However, the model results were compared with the Rational Method for urban catchments as described in AR&R (1998). The comparison was made for 1% AEP flows at the ocean outfall and are shown in Table 4.3.

Table 4.3: Comparison of RAFTS Model and Rational Method Results

Model	Critical Storm Duration (min)	Peak Discharge (m^3/s)
RAFTS	60	55
Rational Method	50	45

A RAFTS model for the Newport catchment was also developed by the Department of Public Works as part of the *Coastal Stormwater Planning Study* (1992). That study reported the peak discharge for a 5% AEP storm at the ocean outfall. A comparison of the results from that study with the present one is shown in Table 4.4.

Table 4.4: Comparison of RAFTS Model and PWD (1992) Results

Model	Storm Duration (min)	Peak Discharge (m^3/s)	Discharge Volume (m^3)
RAFTS (PWD, 1992)	60	42	92,000
RAFTS - Present Study	60	40	105,000

These two comparisons show an acceptable agreement, with the results from the present study shown to be conservative with respect to volume but show a good comparison with peak discharge.

4.4. HISTORICAL STORM EVENTS

Daily rainfall data for March, 1977, November, 1984 and April, 1998 storm events was obtained from the gauge at Newport Bowling Club. The data is summarised in Appendix D.

The daily rainfall totals do not provide information about variation of the storm intensity over a time period of less than one day. For accurate definition of the historical storms, there is a need to define the temporal distribution of the daily rainfall totals at Newport Bowling Club. The closest pluviometer data available for this purpose was the Warriewood STP station. The location of the pluviometer is shown in Figure 4.1.

The storm events of November, 1984 and April, 1998 were generated by using daily rainfall totals from the gauge at Newport Bowling Club and temporal pattern information from the pluviometer at the Warriewood STP gauge. A comparison of daily rainfall totals for the two gauges is presented in Table 4.5.

Table 4.5: Comparison of Daily Rainfall Totals (mm)

Storm Event	Newport Bowling Club	Warriewood STP
November 1984	73	88
April 1998	157	144

The rainfall patterns for these events are shown in Figures 4.2 and 4.3.

Pluviometer data for the March 1977 event was not available. However, there is some information available about the temporal distribution of the storm in *The Newport Story* (Jennings, 1987), which narrates the history of Newport.

Given the fact that the recorded flood levels for this event were reasonably well documented (Table 3.3), an attempt was made to utilise this event in the calibration/validation process by developing a temporal pattern from available information. From the evidence contained in Jennings (1987), 146mm fell from 8am to 4pm on March 1, 1977, with 50mm falling in 15 minutes around noon. The total rainfall between 9am on March 1, 1977 and 9am the next day was 171.3mm. Thus 146mm of rain fell in 8 hours duration with a sudden burst midway during this period.

In the search for a suitable temporal pattern for this storm, recourse was made to AR&R (1987). The temporal patterns derived in AR&R are based on the average variability method of Pilgrim et al. (1969) and Pilgrim and Cordery (1975). The method provides an average temporal pattern for a given climate zone. The temporal patterns are, therefore, not likely to be suitable for use with an historical storm. However, to translate the anecdotal evidence of rainfall to a temporal pattern, the AR&R temporal pattern was utilised as a starting point. The selected pattern was for a 9-hour duration event and for an ARI greater than 30 years. The temporal pattern was modified to accommodate the anecdotal evidence as described above and used in the hydrological model with the hydrographs produced being used in the hydraulic model.

The final rainfall temporal pattern used in the validation of the hydrologic model provides peak water level results matching those recorded during the 1977 flood. The final temporal pattern used is consistent with anecdotal records of the storm and remains similar to the 9 hour AR&R temporal pattern from which it was based.

The AR&R and the modified temporal patterns are compared in Figure 4.4.

It should be noted that this approach of developing a historical event for input to the hydrological model is only approximate. The approach was adopted to produce supplementary results for the calibration/validation process and serves to increase the models' accuracy.

4.5. DESIGN RAINFALLS

Design rainfall depths and temporal patterns were developed using standard techniques provided in AR&R. The techniques provided in AR&R are recommended by the Institution of Engineers Australia, and have been used extensively in Australia for the purposes of design rainfall calculation. This method has undergone extensive peer review and is the current best practice for design rainfall determination within the Newport Beach study area. Table 3.2 in Section 3.4 of this report provides a summary of the rainfall intensities used for the hydrological modelling. The design rainfall estimates were applied to the calibrated hydrologic model in order to predict design runoff hydrographs. The critical storm duration for peak water level varies throughout the catchment. Critical storm durations based on peak water level for each design storm are presented in Appendix A.

The Probable Maximum Precipitation (PMP) was estimated using the Generalised Short Duration Method (BoM, 1994). For a small catchment the generalised short Duration Method provides a statistically based determination of the probable maximum precipitation. This method was prepared and is recommended by the Bureau of Meteorology.

5. HYDRAULIC MODELLING

5.1. ESTABLISHMENT OF HYDRAULIC MODEL

A one-dimensional hydraulic model set up in a quasi two-dimensional fashion was developed for the study using MIKE-11 version 2000B (SP1) (DHI, 2000). MIKE-11 is a dynamic hydraulic-routing model developed by the Danish Hydraulic Institute (DHI). MIKE-11 is widely used and has been shown to provide reliable, robust simulation of flood behaviour in urban and tidal areas through a vast number of applications. The model has been tested over a period of many years in Australian rivers, creeks and urban systems. The wide variety of hydraulic structures the model can handle (weirs, roads, levees, culverts, bridges etc) makes it a flexible and adaptable hydraulic analysis tool.

The hydraulic model for the Newport floodplain was schematised so that it could be readily used for investigating flood mitigation works and the development of management options at a later stage.

The model branch layout was developed through consideration of reports of historical floods and available mapping (Section 3). The physical lie of the land, in addition to major controls such as elevated roads and embankments were also taken into account. The model branch layout is shown in Figure 5.1.

For an extreme event such as the PMF it is likely that flood levels would rise to such an extent in the Ross Street area that flow would occur across Myola Road and break through the dune on to the beach area creating a major flow path. A model branch (Myola Road) was introduced to describe this flow path.

The location of cross sections in the model was determined by field inspection. Cross sections were located to be perpendicular to defined flow paths. The floodway cross sections were located so that flow controls on the floodplain could be modelled satisfactorily, with cross sections spaced to adequately represent variations in the topography of the floodplain.

The locations of the cross sections in the model are shown in Figure 5.2. The model cross sections are provided in Appendix H. Due to the quasi-2D schematisation of the hydraulic model, a very small number of cross section results are listed in the design flood levels in Appendix A for completeness for which there is no cross section shown in plan (Figure 5.2). All cross sections required to aid the determination of flood levels for flood planning purposes throughout the catchment are shown in Figure 5.2.

In addition to modelling surface flow, pipe flow was also modelled along the main tributaries of the catchment. A layout of the modelled pipe network is shown in Figure 5.3.

5.2. MODEL CALIBRATION

The storm events of April, 1998 and November, 1984 were selected for calibration and validation purposes. Another event in March, 1977 was also used, to provide supplementary information because only an approximate temporal pattern was available for this event (Section 4.4).

Recorded flood level data used in the calibration was obtained through the resident survey as discussed in Section 3.5. A limited number of flood level data could be identified through the survey. For April, 1998 and November, 1984 only one recorded level was available, which limited the calibration process.

The model topography was developed such that it represented the best estimate of the floodplain features at the time of the flood. Significant modifications in the stormwater drainage system, which were represented in the modelling of the three events, are presented in Table 5.1.

Table 5.1: Significant Modifications in Stormwater Drainage System

Storm Event	Description of the Modified Part of the Stormwater Drainage System
April, 1998 (Existing Conditions)	• 4 x 1500mm diameter pipes as ocean outfall • 2 culverts underneath the Council car park • Embankment along Barrenjoey Road side of the oval • Channel lining on the invert of the open concrete channel
November, 1984	• 3 x 1500 mm diameter pipes as ocean outfall • 2 culverts underneath the Council car park • No embankment along Barrenjoey Road Side of the oval
March, 1977	• 3 x 1500 mm diameter pipes as ocean outfall • 1 culvert underneath the Council car park • No embankment along the Barrenjoey Road Side of the oval

The model was first calibrated using the April, 1998 event. Inflow hydrographs were obtained from the RAFTS catchment model. The model parameters such as channel and pipe roughness were modified and a match was obtained between the recorded and modelled flood level. The result is shown in Figure 5.4.

The calibrated model was then validated using the November, 1984 event. The model was modified to reflect the November, 1984 catchment and drainage system conditions. Inflow hydrographs were obtained from the RAFTS catchment model. The hydraulic model parameters were kept the same as those used in the April, 1998 model. The modelled flood level compared favourably with the recorded flood level. The validation profile is provided in Figure 5.5.

It is important to note that the parameters used to calibrate the model to April, 1998 event were carefully selected and lie in the range expected in an urban setting.

Secondary validation of the model was then carried out using the March, 1977 catchment conditions and drainage system. Inflow hydrographs were estimated based on the temporal pattern derived from AR&R as discussed in Section 4.4 above. The hydraulic model parameters were kept the same as those used in the April, 1998 model. This secondary validation showed good correlation between the recorded peak flood levels and the modelled peak flood levels. Of particular note in this validation is the models reproduction of the backwater caused by the dune, giving reasonably level flood recordings at the locations on West Branch. The validation profile is provided as Figure 5.6.

The confidence levels in the model calibration are reduced due to the single calibration and verification points, their occurrence in West Branch only and the degree of uncertainty in the levels generated by the 1977 temporal pattern. It was therefore imperative that the model parameters were determined carefully, and a sensitivity analysis carried out to determine the range of errors associated with each of the calibration parameters (see Section 7.3).

6. COASTAL BOUNDARY CONDITIONS

6.1. COASTAL PROCESSES OVERVIEW

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

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

These water level components are described in detail in Appendix E. These components have significant implications for catchment flood behaviour. The effects of fresh water flow, eustatic and tectonic changes, Tsunamis, the Greenhouse effect and global changes in meteorological conditions have not been considered in detail as part of this study.

6.2. TIDES, WIND SET-UP AND THE INVERSE BAROMETER EFFECT

Ocean water levels under normal astronomical tidal conditions will not normally affect discharge through the stormwater outfall pipes. However, during very severe ocean storms elevated ocean water levels occur. MHL (1992) describes the extremal analysis of historical peak water levels from the Sydney (Fort Denison) tide gauge. These levels include astronomical tide, the inverse barometer effect and wind set-up. Ocean levels at selected average recurrence intervals (ARI) are presented in Table 6.1. These levels are also applicable for the open ocean region of Newport Beach because Port Jackson tides are very similar to those at Newport Beach.

Table 6.1: Peak Storm Ocean Levels at Sydney (Fort Denison Data 1914-1991)

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

These elevated ocean level events are very unlikely to occur during severe fresh water flood events of short duration in small coastal catchments, but form a component of potential ocean inundation levels near the entrance to Newport Floodplain.

Measured Sydney water level data has also been used to describe the probability of exceedance of astronomical tide plus meteorological effects (Figure 6.1).

For this study combined astronomical tide, inverse barometer and wave set-up were computed to provide ocean boundary water level ocean boundary data for the calibration events. The invert level of the ocean outlet pipe is about 0.3 m AHD and the pipe diameter is 1.5 m. For calibration events wind speeds were low and have been found previously (L&T, Avoca/Wamberal) to provide only a small water level increment unless they are very high (>30 m/s). Ocean water levels were also too low to affect the flood levels for calibration events. Discharge from the pipes effectively discharged down the beach face because the outlets were not drowned and the slope down the beach face is steeper than the pipeline gradient.

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

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

Recent analyses of long term rainfall and measured water levels at Newcastle (Hunter River) showed virtually no correlation. The most likely jointly occurring ocean water level is mean sea level (MSL), with some minor hint of elevation caused by low atmospheric pressure. Therefore, one could adopt 0m AHD as the most likely or expected ocean level. However, it is recommended that a more risk-based approach is to specify the ocean boundary water level to be that level which is only equalled or exceeded for 1% of the time. This level is 1.0m AHD in the Sydney region (Figure 6.1), where AHD is 0.9m above tide datum (LAT). This approach has also been developed for other coastal urban areas (e.g. areas within Newcastle City Council). The level excludes wave set-up.

6.3. WAVE SET-UP

6.3.1 Overview

Wave propagation to the outlet of the catchment at the shoreline may affect nearshore water levels as well as cause run-up and shoreline erosion during a storm. Wave breaking at the shoreline leads to a phenomenon called wave set-up which increases still water levels above storm tide levels (astronomical tide + wind set-up + inverse barometer effect).

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

6.3.2 Calibration Events

The wave coefficients described in Table F1 were used to transform the time series offshore wave data (H_s, T_z and direction) to the nearshore location. For each time step, T_z and direction were used to enter Table F1, (including interpolation), to determine the appropriate wave coefficient. Equation F1 was then used to calculate the equivalent inshore wave height. A surf zone model based on the work of Goda (1987) was then used to calculate wave set-up. Set-up was calculated in a water depth which included tide and scour. A bed slope of 1:15 was used which is considered realistic on a beach undergoing storm erosion, where back-beach sand is being transported offshore. Further landward there would be a steep erosion escarpment on the dune face, but that feature would not affect wave set-up at the discharge location. Depth was determined on the basis that the invert level of the discharge pipes is approximately 0.3 mAHD and general scour to 0 mAHD is likely during a minor storm based on experience and engineering judgement. Greater scour would be conservative because wave set-up decreases in deeper water. This assumption is realistic since scour is likely to be confined to a local area around the pipe exit rather than a more considerable broad area of the surf zone.

The inverse barometer effects on water level were included using barometric data for each event. Wind set-up was very small because wind speeds were low during the calibration events and the nearshore seabed is relatively deep close to the shoreline.

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

The largest offshore waves for the selected calibration events occurred during November, 1988 when peak H_s reached 5.4m. The largest met-ocean water level increment for the assumed storm beach profile, 0.43m, also occurred at that time.

6.4. ADOPTED COASTAL BOUNDARY CONDITIONS FOR DESIGN EVENTS

For design events, the severe storm offshore wave heights described in Table 3.4 were transferred to the near shore location using the weighted average wave coefficient (0.81). The surf zone model was then used to calculate wave set-up in a water depth of 1m, assuming scour to 0m AHD. More scour would be less conservative from a wave set up point of view.

For the PMF flood an offshore H_s of 11m (see Table 3.4 and Section 3.7) with T_p of 15 seconds was adopted. Ocean levels (with all relevant components outlined above) adopted for design flood modelling are presented in Table 6.2.

Table 6.2: Ocean Boundary Water Levels for Design Flood Modelling

Flood Event (% AEP)	Design Ocean Water Level (mAHD)
PMF	2.22

Flood Event (% AEP)	Design Ocean Water Level (mAHD)
1	1.95
2	1.9
5	1.8
20	1.75

These water levels are considered to be conservative. They are based on a storm tide water level of 1.0m AHD (level exceeded for only 1% of the time), plus 0.2m Greenhouse increment in MSL and wave set-up. Nearshore breaking waves in the order of 1m may occur at the overtopping point, depending on dune scour. For the 1% and 5% AEP flood events, water depths in the floodway near the dunes may be in the order of 0.9m. Applying wave overtopping procedures described in the Shore Protection Manual (1984), wave heights in the order of 0.3m may occur in this area and propagate into the car park/commercial area, thereby causing some additional water damage.

6.5. WAVE RUN-UP

One other nearshore coastal flooding scenario is possible - oceanic inundation via wave run-up. This is presented here for reference but does not form a component of the boundary conditions adopted for the flood modelling. Wave set-up and run-up may lead to overtopping of the low dune (crest at 3.2m AHD, approximately) which has been formed at the seaward end of the floodway.

These severe ocean storms may occur during times of little or no rain, but will be associated with the elevated ocean levels described in Table 6.1.

Wave set-up is greater at the shoreline than at the seaward end of the pipes. The Greenhouse related increment in MSL of 0.2m has been included also. Additionally, wave run-up will occur, potentially causing overtopping and flooding of the nearshore floodway area. Wave overtopping has been calculated following Holman (1986).

$R_2 = (5.2\beta+0.2)H_s$

where R_2 is wave run-up height not exceeded for more than 2% of the time
β is beach slope from 6m depth to top of run-up, taken to the 1:10
H_s is in 6m depth (assumed to be the adopted study location)

These results are presented in Table 6.3.

Table 6.3: Peak Ocean Storm Levels at the Still Water Line and Wave Run-up Level at Newport.

Ocean Storm Event ARI (Years)	Peak Storm Level at Shoreline (m AHD)	Run-up Level (R_2 – m AHD)
100	2.5	7.1
50	2.4	6.8
20	2.3	6.4

Wave run-up is calculated assuming the dune continues sufficiently high to accommodate these run-up heights. These results show that significant wave overtopping will occur. Therefore, major ocean storms will cause significant overtopping of the frontal dune. Some ponding will occur, but this will be limited by the storm water grates landward of the dune and discharge back to the sea and the average overtopping discharge rate which is approximately $2m^3/m/s$ (PIANC, 1992) during a period of a group of higher waves over a period of 1 to 2 minutes. This scenario will be repeated a number times near high tide.

7. DESIGN FLOOD ESTIMATION

7.1. GENERAL

The hydraulic model used to calibrate the storm event of April, 1998 was selected for use in design flood estimation. Design flood inflow hydrographs were obtained from the RAFTS model. These hydrographs were applied to the above hydraulic model, which represents the current floodplain conditions. A range of hydrographs with different storm durations were applied to the model in order to estimate the critical storm duration for different areas in the floodplain.

7.2. RESULTS

Model results for predicted flood behaviour at key locations in the floodplain are summarised below in Table 7.1. The results are also provided as longitudinal profiles of the main floodplain flow paths in Figures 7.1 to 7.6.

Table 7.1: Summary of Peak Design Flood Levels

Location	Branch	Peak water Level (mAHD)				
		AEP				
		20%	5%	2%	1%	PMF
Bishop St	West	5.08	5.13	5.15	5.18	5.47
Barrenjoey Rd	West	4.09	4.12	4.14	4.18	5.41
The Boulevard	West	3.51	3.81	3.95	4.07	5.40
Bramley Ave	West	3.49	3.80	3.96	4.08	5.47
Council Car Park nr Bramley Ave.	West	3.47	3.79	3.94	4.07	5.47
Beach dune	West	3.39	3.67	3.82	3.93	5.41
Crown of Newport reserve	North	13.78	13.91	13.95	13.99	14.37
Top of Howell Close	North	11.44	11.57	11.62	11.67	12.37
Seaview Ave/Neptune Rd	North	11.18	11.33	11.38	11.44	12.24
Ocean Ave	North	6.71	6.86	6.91	6.95	7.59
Foamcrest Ave	North	5.62	5.95	5.98	6.04	6.86
Barrenjoey Rd	North	4.59	4.68	4.72	4.73	5.51

The results presented in Table 7.1 and Figures 7.1 to 7.6 represent the peak flood level at each location when all storm durations have been considered. Therefore the flood profiles represent the peak water level envelope. A summary of results for all the branches is provided in Appendix A. It should be noted that the flow velocities presented in Appendix A are average values for each cross section and may vary due to localised effects.

7.3. SENSITIVITY ANALYSIS

Hydraulic model sensitivity was tested for the West and North model branches to demonstrate the range of uncertainty in the model for the 1% AEP design flood event for the critical 2 hour storm duration. The following model parameters were tested for sensitivity:

- Channel roughness – increased/decreased by 20%
- Catchment runoff – increased/decreased by 20%

- Tailwater level – increased/decreased by 0.2m
- Beach dune level – lowered to 2.8 mAHD
  – increased to 3.4 mAHD
  increased to 3.8 mAHD
- Culvert Blockage – Howell Close Culverts Blocked 100%.

The sensitivity of the flood levels to a lowered dune condition was also tested for the 20% AEP design flood event (critical duration of 2 hours and an assumed lowered dune level of 2.8 mAHD).

The sensitivity results are presented in Figures 7.7 to 7.14 and are summarised for selected model branches in Table 7.2.

Table 7.2: Summary of Model Sensitivity Results

Branch	Maximum Variation in Water Levels			
	Catchment Runoff	Channel Roughness	Tailwater level	Culvert Blockage
North	-0.24/+0.18 m	-0.12/+0.1 m	0 m	0 m
West	+/- 0.23 m	-0.12/+0.1 m	+/- 0.05 m	+0.08 m
Barrenjoey Rd	-0.21/+0.17 m	-0.11/+0.09 m	-0.04/+0.05 m	0 m
Robertson Rd	-0.23/+0.18 m	-0.11/+0.08 m	-0.04/+0.05 m	0 m
Foamcrest Ave	-0.09/+0.08 m	-0.08/+0.06 m	0 m	0 m

The sensitivity analysis results for West Branch (Figure 7.7 to 7.9) show the model parameters have little sensitivity for areas upstream of Barrenjoey Road.

The increase in model inflows had the greatest influence on peak water levels. For example, increased runoff results in a water level increase of up to 0.23m downstream of Barrenjoey Road. This is due to the storage effect between Barrenjoey Road and Newport Beach. The increased inflows also resulted in an increased volume flowing into the storage area and caused a consequent rise in the water levels.

The results indicate that the North Branch (Figures 7.10 to 7.12) shows little sensitivity to tailwater levels, but some sensitivity to catchment runoff, roughness and culvert blockages. The change in catchment runoff changed the water levels in North Branch by up to 0.24 m. Variations in the tailwater level do not have an impact on the water levels in North Branch or West Branch. The model shows some sensitivity to culvert blockage in the Howell Close reach of North branch. Blockage of the three 0.9 m diameter pipe culverts results in a 0.08 m increase in the peak water level.

The beach dune near the ocean outfall has the potential to erode to lower levels during a flood event. Sensitivity of the model to dune level was also tested by lowering the dune level to 2.8 mAHD. A higher dune was also tested with levels of 3.4 and 3.8 mAHD. The results are presented in Figures 7.13 and 7.14 and indicate that water level in both the North and West Branch are not sensitive to the dune level for 1% AEP and 20% AEP critical storm durations.

8. DISCUSSION OF RESULTS

8.1. MODEL SETUP TO CALIBRATION SUMMARY

Flooding behaviour of the Newport Beach catchment floodplain was investigated. The floodplain was modelled using the MIKE-11 hydraulic model. The branches, which represented the floodplain are shown in Figure 5.1.

Inflows to the hydraulic model were developed using the RAFTS hydrologic model and applied to the hydraulic model as described in Section 5. The output from RAFTS was compared with the results of Rational Method calculations and those of a previous study in Tables 4.3 and Table 4.4. The comparison shows similar peak flows and volumes and supports the results from the RAFTS model.

The hydraulic model was calibrated to April, 1998 event and validated against the April 1988 and March 1977 events. The model parameters adopted are within the expected range for urban areas.

The calibrated model was used to estimate design flood levels for existing catchment and floodplain conditions. The 2-hour storm duration was found to be critical for the lower reaches of the catchment. Peak flood profile levels are presented in Figures 7.1 to 7.6. Full details of model results are provided in Appendix A. Figures 8.1 to 8.6 indicate areas likely to be inundated for the range of design flood events considered.

8.2. SEVERE FLOODING LOCATIONS

The model results describe the observed flooding regime of the area. They indicate that the most severely affected area of the catchment lies between The Boulevard, Myola Road and Barrenjoey Road. This area includes properties in Ross Street, The Boulevard, shops on Barrenjoey Road and the Council car park near Bramley Avenue.

8.3. OTHER FLOOD AFFECTED LOCATIONS

Flooding in other parts of the floodplain is limited to the adjacent areas of the two main tributaries, the North Branch and the West Branch.

- In the upper reaches of the North Branch, flooding is confined to the drainage easement and adjacent properties along the easement. From Foamcrest Avenue and downstream, the flow passes through the properties between Foamcrest Avenue and Barrenjoey Road. The flow then crosses over Barrenjoey Road and joins West Branch before discharging to the sea. In addition to the surface drainage, there are underground stormwater pipes, which run parallel to the North Branch. These pipes carry a significant amount of floodwater. For the 1% AEP storm, approximately 35% of the peak flow goes through the pipes under Barrenjoey Road.

- On the West Branch, properties between Gladstone Street and Bardo Road, which lie in the flow path of floodwaters, are inundated. The results, plotted in Figure 7.1, represent flow conditions for a channel with uniform roughness. Fine detail local obstructions such as hedges, gate posts, guard rails etc. are not taken into account. After crossing Bishop Street, the floodwaters enter Newport Park. The low relief in the northeastern corner of the Park allows the flow to pass onto Barrenjoey Road. The floodplain between Barrenjoey Road and the Newport Beach dune system near the ocean outfall acts as a detention area. The water surface profile is very flat which creates substantial hydraulic head for water to flow through the culverts under the Council car park near Bramley Avenue finally discharging through the ocean outfall. For the 1% AEP storm, approximately 80% of the peak flow passes through the ocean outfall.

- Flooding along Robertson Road is shown in Figure 7.3. The floodwaters travel down Robertson Road to Barrenjoey Road where most of the flow is diverted south along Barrenjoey Road towards the intersection with The Boulevard. The model is schematised such that downstream of Foamcrest Avenue the flow is contained by the shop buildings on either side of the road. Water depths at 1% AEP in this area vary from 0.3m at Foamcrest Avenue to 0.16m at Barrenjoey Road.

8.4. COMPARISON OF RESULTS WITH PREVIOUS MODELLING

The peak water levels for the 1% AEP storm in the area around Ross Street were compared with previously calculated levels available in Council records (Warringah Council, 1987). A review of the previous modelling methodology was carried out.

It was found that hydraulic grade line calculations were used to establish the flood levels for the 1987 assessment. The water level boundary at the ocean outfalls was lower than that adopted for this study. In addition a very simple approach was adopted to produce the inflow hydrograph to the model. Although the calculated peak of the assumed hydrograph for the 1% AEP storm (47 m^3/s) is close to the peak obtained from the RAFTS model in this study, the inflow volume is only close to one-third of that from RAFTS hydrograph established for this study (refer Section 4.3). Because flooding in the Ross Street area is sensitive to the inflow volume, the previous model gave appreciably lower flood levels compared with the results from this study.

8.5. COMPARISON OF RESULTS WITH NOVEMBER 1999 FLOOD STUDY RESULTS

Refinement of the hydraulic model in this revision of the study has resulted in changes to the design peak flood levels and the design flood extents previously reported (November, 1999). Re-schematisation of the North branch of the model (at Howell Close) has resulted in lower design flood levels than previously reported (November, 1999).

Re-schematisation of the lower end of North branch between Foamcrest Avenue and Barrenjoey Road has also lead to a variation in peak water levels. The previous study had a broader scale cross-section with high roughness to represent

development within the floodplain. The new schematisation depicts individual flow paths through and between apartment buildings in the area.

The changes to the Robertson Road and Barrenjoey Road branches of the model also result in a change in peak flood levels. The new schematisation provides a direct flow path from Robertson Road to Bramley Lane.

8.6. MODEL SENSITIVITY EFFECTS

The level of variability in the model results was tested by carrying out a sensitivity analysis. This analysis assumed additional significance due to the limited number of data points available for calibration (Section 5.2). Validation of the model was also limited to one point on one branch of the model. The sensitivity of various parameters needs to be considered carefully because it provides a guide to the variability of model results. The sensitivity of the model to catchment runoff, channel roughness and the downstream tailwater conditions was investigated as outlined in Section 7.3.

Channel roughness was increased and decreased by 20%, which had a maximum impact in the order of 0.1m on all branches. This impact is small, given the wide range of roughness values (Manning's “n” of 0.016 to 0.1) adopted to physically represent the catchment characteristics.

Similarly an increase and decrease of 20% in the tailwater level had only a small impact. The impact on West branch was limited to the downstream boundary with a change of only 0.05m upstream of the dune. This small change was carried through to all low-lying branches affected by ponding of water behind the dune.

Catchment runoff was increased and decreased by 20% to model any uncertainty in the results of the hydrologic modelling. Errors in the hydrologic modelling may arise from local variations to the standard practices applied to develop the design storms. This may include differences in the design temporal pattern and the likely local design temporal pattern. An increase in catchment runoff by 20% had only a small impact in the upper reaches of the catchment, but had an appreciable impact in the area around Ross Street where the peak water levels increased by 0.2m. Similarly, a 20% decrease in the catchment runoff reduced peak water levels by 0.2m in the same area.

These results indicate that flooding in the area around Ross Street is sensitive to the inflow volume in addition to other factors. The area between Barrenjoey Road and the beach dune near the ocean outfalls effectively acts as a detention area where the storage volume influences the flood levels.

The sensitivity of the model to catchment runoff also points to the significance of the output from the hydrological model. The inflows to the hydraulic model therefore become the critical factor in determining flooding in the lower reaches of the catchment. The fact that the results of the hydrological model compared well with the Rational Method and with those from a previous study suggests that the hydraulic model consequently produces realistic flood level information.

The beach dune near the ocean outfall can potentially erode under flood flow conditions. Sensitivity testing of the model results indicated that there was no appreciable impact on the peak water levels due to this variation in the dune level.

Culvert blockage in the reach of North branch at Howell Close was examined. The model showed little sensitivity to the blockage of the culverts under the properties on Howell close with a localised increase in water level in North branch only. The peak elevation in design flood levels for the 1% AEP Flood was 0.08 m at Howell Close. A discussion of the implications of this phenomena are discussed in Section 8.9.

The results of sensitivity analysis are presented in Figures 7.7 to 7.14.

8.7. MODEL LIMITATIONS

It is to be noted that MIKE-11 is basically a surface flow modelling package. The latest version of the model used for this study has the additional feature of modelling pressurised pipe flow, but lacks the ability to model pit hydraulics in a direct sense. The new feature has been utilised to model the pipe network in the floodplain with an approximate representation of pit hydraulics. Therefore the results in the upper reaches of the two branches, where most of the pits are situated, should be treated with caution. The area around Ross Street is not expected to be affected by pit discharges and the results are more reliable when compared to the other parts of the catchment.

8.8. DATA LIMITATIONS

Limited data was available for calibration of the hydraulic model and no data was available for the calibration of the hydrological model. However, the results of the hydrological modelling compared well with the Rational Method results and Department of Public Works (1992).

Design flood information produced in this study is also dependent on the accuracy of design data provided in AR&R. As more historical storm information is gathered, the data presented in AR&R is likely also to be modified, which consequently may result in variations to flood design flood levels.

8.9 BLOCKAGES AND DEBRIS

Recent experience in the Wollongong City Council area (Rigby and Silveri, 2001) indicates that the effects of the blockage of culverts and similar structures with debris from the catchment can have a significant impact on flood levels in the vicinity of the structure. No similar reports of widespread culvert blockages have been identified for the Newport Floodplain apart from a localised report for Howell Close. As such inclusion of blockages only consisted of the consideration of the effects of the blockage of culvert in Howell Close.

The assessment of the hydraulic model sensitivity to culvert blockage undertaken in this study indicates that the reach of North branch at Howell Close is not sensitive to culvert blockages. The change in peak water level of 0.08 m is minor in comparison to the sensitivity of the model to hydrological inputs.

Lack of data to support a specific culvert blockage policy (e.g. Rigby and Silveri, 2001) and the insensitivity of the model to culvert blockages in the critical reach of Howell Close provide a reasonable basis to accept the design flood levels without culvert blockage incorporated in the hydraulic model used to develop the design flood levels. It is recommended that a review of this issue be included as part of the Floodplain Risk Management Study.

9. REPORT QUALIFICATIONS

This report has been prepared for Pittwater Council to define the nature and extent of flooding in the Newport Beach catchment floodplain. The report defines the flooding behaviour for the major flow paths in the catchment.

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

There will be an unknown level of uncertainty in the results and this should be borne in mind in their application. However, the three modelling tools used in the preparation of this flood study are suitably accurate for the Flood Study undertaken, and the Floodplain Risk Management Study.

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

- The beach dune is assumed to be maintained by Council in the same shape and profile as surveyed.
- Channel roughness for the flow path between King Street and Bishop Street represents average conditions. Flow blockage due to local obstructions, such as fences, has not been modelled.
- The course nature of pit modelling. Uniform losses were assumed for pits. Actual losses will be different for various pit configurations.
- Adequate inlet capacity is assumed for the underground pipe system.
- Design flood extents are approximate between cross sections of the model. Where surveyed levels are not available, flood extents are based on the 2m Land Information Centre land contour data provided by Council and interpolation of model results.
- The drains and stormwater drainage system are maintained in reasonable condition.

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

10. ACKNOWLEDGEMENTS

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Their assistance is gratefully acknowledged.

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ARAMETRIC OFFSHORE WAVE CLIMATE

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

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

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

σ_y is standard deviation of y : y=lnH.

TABLE 6: Parametric Offshore Wave Climate Description - Botany Bay

APPENDIX G: Parametric Offshore Wave Climate - Botany Bay

Report Prepared For
Pittwater Council

Newport Beach Flood Study

Report J1740/J2056/R1838
March, 2002

APPENDIX A

APPENDIX B

APPENDIX C

APPENDIX D

APPENDIX E
COASTAL PROCESSES

E1 WAVES

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

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

- H_{mo} significant wave height (H_s) based on $4\sqrt{M_o}$ where M_o is the zeroth moment of the wave energy spectrum (rather than the time domain $H_{1/3}$ parameter).

- H_{max} maximum wave height in a specified time period

- T_p wave energy spectral peak period, that is, the wave period related to the highest ordinate in the wave spectrum

- T_z average zero crossing period based on upward zero crossings of the still water line. An alternative definition is based on the zeroth and second spectral moments.

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

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

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

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

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

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

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

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

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

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

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

$$H_{max} = H_s \sqrt{(0.5 \ln Nz)} \qquad \textbf{(E1)}$$

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

This relationship has been found to overestimate H_{max} by about 10% in severe ocean storms. In shallow water the relationship described by equation (E1) is not fulfilled. In very shallow water H_{max} is replaced by the breaking wave height, H_b.

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

E2 WATER LEVELS

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

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

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

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

Wind set-up and the inverse barometer effect are caused by regional meteorological conditions. When the wind blows over an open body of water, drag forces develop between the air and the water surface. These drag forces are proportional to the

square of the wind speed. The result is that a wind drift current is generated. This current may transport water towards the coast upon which it piles up causing wind set-up. Wind set-up is inversely proportional to depth. It is not particularly high in the Central Coast region of NSW because of the relatively deep water close to the shoreline.

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

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

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

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

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

Many scientists believe that global warming of the Earth's atmosphere will lead to a rise in mean sea level. Predictions of global sea level rise due to the Greenhouse effect vary considerably. It is impossible to state conclusively by how much the sea

may rise, and no policy yet exists regarding the appropriate provision which should be made in the design of new coastal developments.

Based on models developed by the American National Academy of Science and the American National Research Council incorporating relevant environmental factors, a guide to future ocean level rises is presented in Table E1. For a fifty years planning period for Newport a MSL rise of 0.2m has been included for preparation of design flood levels.

Table E1: Predicted Greenhouse Related Mean Sea Level Rises.

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

APPENDIX F
OCEAN WATER LEVELS

F.1 WAVE CLIMATE INVESTIGATION

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

$$H_i = H_o K_w \qquad \text{(F1)}$$

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

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

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

APPENDIX G

APPENDIX H

FIGURES