Great Lakes
COUNCIL

Wallis Lake Floodplain Management Study

TABLE OF CONTENTS

1. INTRODUCTION

2. STUDY APPROACH

2.1 General
2.2 Categorisation of Sites
2.3 Wave Climate
2.3.1 Influence of Oblique Waves
2.3.2 Design Criteria for Wind
2.4 Design Still Water Levels
2.5 Coincidence of Wind Wave Effects and Rain
2.6 Sensitivity Analysis

3. DESCRIPTION OF TYPICAL CROSS-SECTIONS AND METHOD OF ESTIMATION OF FORESHORE FLOOD LEVELS

3.1 General
3.2 Classification of Cross-sections
3.2.1 Type 1 – Gently Sloping Embankment – Wave Breaks Offshore
3.2.2 Type 2 – Sloping Embankment or Sea Wall– Wave Breaking Close to Shoreline Resulting in No Overtopping
3.2.3 Type 3 – Sloping Embankment or Sea Wall – Wave Break Close to Shoreline Resulting in Overtopping
3.2.4 Type 4 – Sloping Embankment or Sea Wall – Design Water Level Higher than Top of the Sea Wall Wave Breaking Offshore and Reforming
3.2.5 Type 5 – Gently Sloping Sandy Embankment – Wave Breaks Offshore
3.2.6 Type 6 – Sudden Change in Bathymetry – Wave Breaks Close to Structure
3.2.7 Type 7 – Sloping Embankment or Sea Wall – Wave Breaking Offshore and Reformed Wave Resulting in Overtopping of the Embankment

4. APPLICATION OF STUDY DATA TO SPECIFIC SITES

5. GEENHOUSE SEA LEVEL RISE

6. DISCUSSION AND RECOMMENDATIONS

6.1 Assumptions and Implications
6.2 Recommendations and Guidelines

7. REFERENCES

8. GLOSSARY

List of Tables

1.1 Selected Sites
2.1 Wind Data
2.2 Wallis Lake Fetch Lengths (km) for Each Site and Wind Direction
2.3 Design Still Water Levels for Existing Conditions
2.4 Adopted Wallis Lake Design Still Water Levels
4.1 Design Significant Wave Heights , Hs, Due to Locally-generated Wind in Wallis Lake
4.2 Design Flood Levels at the Selected Sites
5.1 Projected Global Mean Seal Level Rise

List of Figures

1.1 Location of Study Area
1.2 Selected Sites with Recommended Design Wind-elevated Flood Levels
2.1 Location of Cross Sections from DLWC (1992) MIKE-11 Hydrodynamic Model
3.1 Typical Cross-section Type 1 and Type 5
3.2 Typical Cross-section Type 2
3.3 Typical Cross-section Type 3
3.4 Typical Cross-section Type 3 Overwash Profile vs. Time
3.5 Typical Cross-section Type 3 Overwash Equivalent Depth
3.6 Typical Cross-section Type 4
3.7 Typical Cross-section Type 6
3.8 Typical Cross-section Type 7
3.9 Typical Cross-section Type 1 Photograph
3.10 Typical Cross-section Type 2 Photograph
3.11 Typical Cross-section Type 6 Photograph
4.1 Wind-elevated Flood Levels for Selected Sites at Wallis Lake

Appendix A Surveyed Sections

A.1 Site 1 Tonys Point Island

A.2 Site 2 Pipers Bay North
A.3 Site 3 Pipers Bay South
A.4 Site 4 Sailing Club
A.5 Site 5 Deepwater Point
A.6 Site 6 Duck Creek
A.7 Site 7 Yarrick Swamp
A.8 Site 8 Brushy Point
A.9 Site 9 Whoots Point
A.10 Site 10 Whoota Point
A.11 Site 11 Hurps Gap Foreshore
A.12 Site 12 Little Flat Point
A.13 Site 13 Coomba Bay
A.14 Site 14 Black Rocks Point
A.15 Site 15 Paling Fence Bay
A.16 Site 16 Attunga
A.17 Site 17 McIntosh Point
A.18 Site 18 McClymonts Creek
A.19 Site 19 McLemonts Islands
A.20 Site 20 Harmony Hill Jetty
A.21 Site 21 Wallis Island West
A.22 Site 22 Mosquito Point
A.23 Site 23 Wallis Island North
A.24 Site 24 Dogo Island
A.25 Site 25 Flat Island
A.26 Site 26 Green Point
A.27 Site 27 Elim
A.28 Site 28 Tiona
A.29 Site 29 Bowling Club
A.30 Site 30 Coomba Coolangatta Street
A.31 Site 31 Coomba Road East
A.32 Site 32 Coomba Jetties
A.33 Site 33 Point Road Tuncurry

Appendix A List of Tables

A.1	Wind elevated Flood Levels, Site 1
A.2	Wind elevated Flood Levels, Site 2
A.3	Wind elevated Flood Levels, Site 3
A.4a	Wind elevated Flood Levels, Site 4
A.4b	Wind elevated Flood Levels, Site 4
A.5	Wind elevated Flood Levels, Site 5
A.6	Wind elevated Flood Levels, Site 6
A.7	Wind elevated Flood Levels, Site 7
A.8	Wind elevated Flood Levels, Site 8
A.9	Wind elevated Flood Levels, Site 9
A.10	Wind elevated Flood Levels, Site 10
A.10	Wind elevated Flood Levels, Site 11
A.12	Wind elevated Flood Levels, Site 12
A.13	Wind elevated Flood Levels, Site 13
A.14	Wind elevated Flood Levels, Site 14
A.15	Wind elevated Flood Levels, Site 15
A.16	Wind elevated Flood Levels, Site 16
A.17	Wind elevated Flood Levels, Site 17
A.18	Wind elevated Flood Levels, Site 18
A.19	Wind elevated Flood Levels, Site 19
A.20	Wind elevated Flood Levels, Site 20
A.21	Wind elevated Flood Levels, Site 21
A.22	Wind elevated Flood Levels, Site 22
A.23	Wind elevated Flood Levels, Site 23
A.24	Wind elevated Flood Levels, Site 24
A.25	Wind elevated Flood Levels, Site 25
A.26a	Wind elevated Flood Levels, Site 26
A.26b	Wind elevated Flood Levels, Site 26 (vertical wall)
A.27	Wind elevated Flood Levels, Site 27
A.28a	Wind elevated Flood Levels, Site 28
A.28b	Wind elevated Flood Levels, Site 28 (vertical wall)
A.29a	Wind elevated Flood Levels, Site 29
A.29b	Wind elevated Flood Levels, Site 29 (vertical wall)
A.30	Wind elevated Flood Levels, Site 30
A.31	Wind elevated Flood Levels, Site 31
A.32	Wind elevated Flood Levels, Site 32
A.33	Wind elevated Flood Levels, Site 33

Appendix A List of Figures

A.0 Legend
A.1 Tonys Point Island
A.2 Pipers Bay North
A.3 Pipers Bay South
A.4 Sailing Club
A.5 Deepwater Point
A.6 Duck Creek
A.7 Yarrick Swamp
A.8 Brushy Point
A.9 Whoots Swamp
A.10 Whoota Point
A.11 Hurps Gap Foreshore
A.12 Little Flat Point
A.13 Coomba Bay
A.14 Black Rocks Point
A.15 Paling Fence Bay
A.16 Attunga
A.17 McIntosh Point
A.18 McClymonts Creek
A.19 McLemonts Islands
A.20 Harmony Hill Jetty
A.21 Wallis Island West
A.22 Mosquito Point
A.23 Wallis Island North
A.24 Dogo Island
A.25 Flat Island
A.26 Green Point
A.27 Elim
A.28 Tiona
A.29 Bowling Club
A.30 Coolangatta Street Coomba
A.31 Coomba Road East
A.32 Coomba Jetties
A.33 Point Road Tuncurry

Appendix B Wind Data

1. Introduction

Wallis Lake is located on the mid-north coast of NSW, about 120 km north of Newcastle (refer Figure 1.1). The twin towns of Forster and Tuncurry at the mouth of Wallis Lake form the urban centre of the region. There are also scattered commercial, tourist and residential developments along the extensive foreshores of the lake.

Wallis Lake is a large tidal estuary with a waterway area of some 78 km². The lake has a catchment of some 1,200 km², which rises to its western boundary in the foothills of the Great Dividing Range. Flows entering the lake from surrounding river catchments (Wallamba River, Wang Wauk River, Coolongolook River and Wallingat River) discharge to the ocean through a permanent entrance channel.

Inundation around the foreshores can be influenced by tidal conditions, river flooding, local weather patterns and potential sea level rise due to the postulated global warming. As the wind waves in Wallis Lake approach the shoreline they will combine with the water level, local bathymetry and any foreshore structures to result in wave runup on the foreshore. It is this runup that determines the final flood level.

Therefore, the wind-elevated flood level at any location within Wallis Lake is influenced by some combination of the following six factors:

1. Wallis Lake still water level (PWD 1989), which is influenced by:
   - ocean level, as a function of:
     - astronomical tide levels, and to a lesser extent
     - ocean storm surge (oceanic wind setup and barometric effects).
   - Local wind setup within the lake.
   - Catchment runoff from rainfall.
   - Rain falling onto Wallis Lake directly.

2. Wave setup from local wind waves.

3. Wave runup from local wind waves.

4. The bathymetry of Wallis Lake.

5. The presence of any foreshore structures.

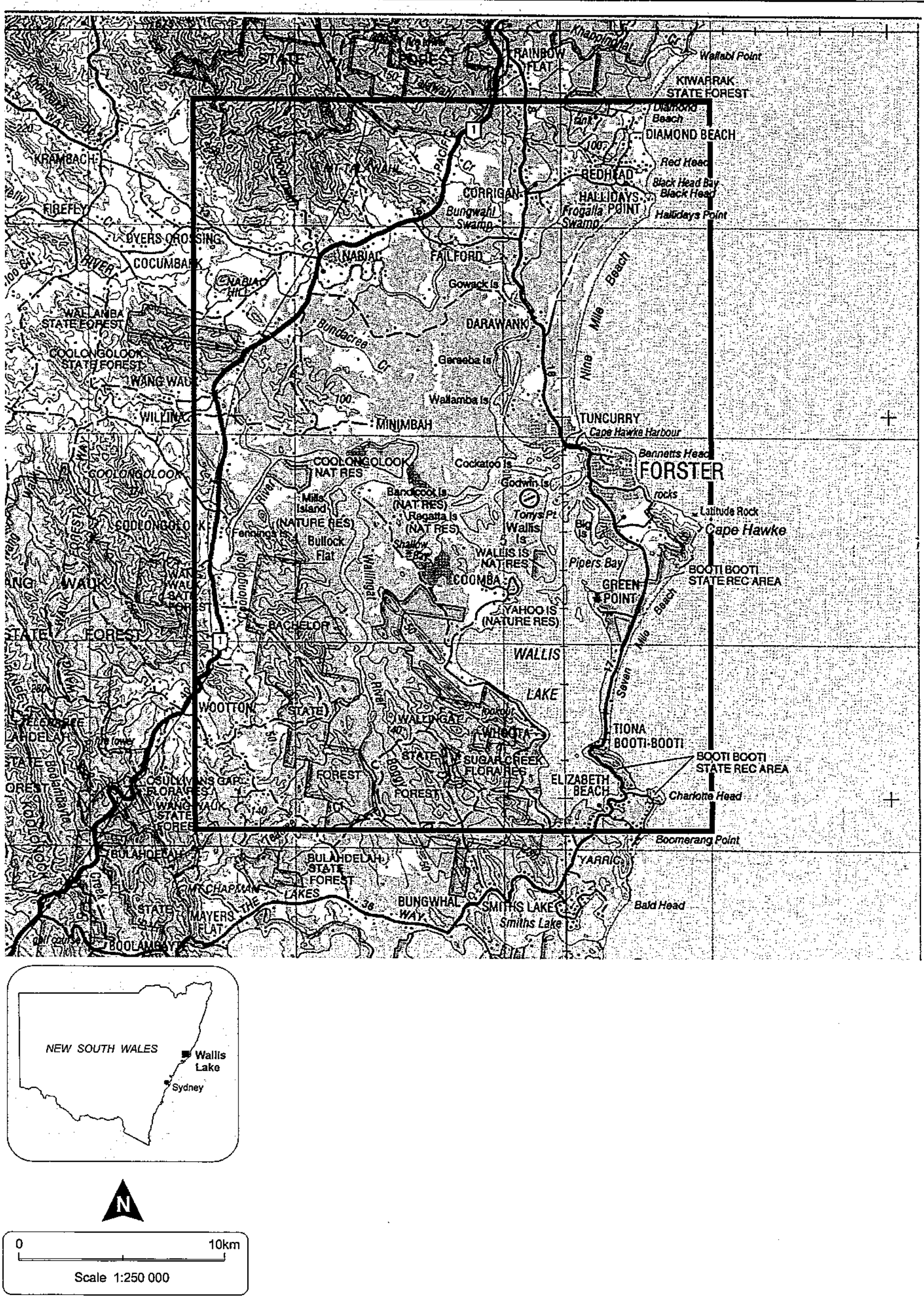
6. Sea level rise due to Greenhouse effects.

The Forster/Tuncurry Flood Study (PWD 1989) and the Forster/Tuncurry Floodplain Management Study Hydraulic Analysis of Flood Mitigation and Development Options (DLWC 1992), examined the historical data and utilised numerical models to estimate 1% AEP water levels for Wallis Lake. The effects of ocean levels were included in these flood studies.

Sites selected for wind-elevated flood level determination were provided by Council and are presented in Table 1.1 and Figure 1.2. The foreshore profiles and associated structures adopted for this study at the 33 selected sites may change in the future due to either natural processes or local developments. This may result in changes to flood conditions and therefore the design flood level should be updated to reflect the new conditions.

Table 1.1 Selected Sites

Site Number	Site Name
1	Tonys Point Island
2	Pipers Bay North
3	Pipers Bay South
4	Sailing Club
5	Deepwater Point
6	Duck Creek
7	Yarrick Swamp
8	Brushy Point
9	Whoota Swamp
10	Whoota Point
11	Hurps Gap Foreshore
12	Little Flat Point
13	Coomba Bay
14	Black Rocks Point
15	Paling Fence Bay
16	Attunga
17	McIntosh Point
18	McClymonts Creek
19	McLemonts Islands
20	Harmony Hill Jetty
21	Wallis Island West
22	Mosquito Point
23	Wallis Island North
24	Dogo Island
25	Flat Island
26	Green Point
27	Elim
28	Tiona
29	Bowling Club
30	Coomba Coolangatta Street
31	Coomba Road East
32	Coomba Jetties
33	Point Road Tuncurry



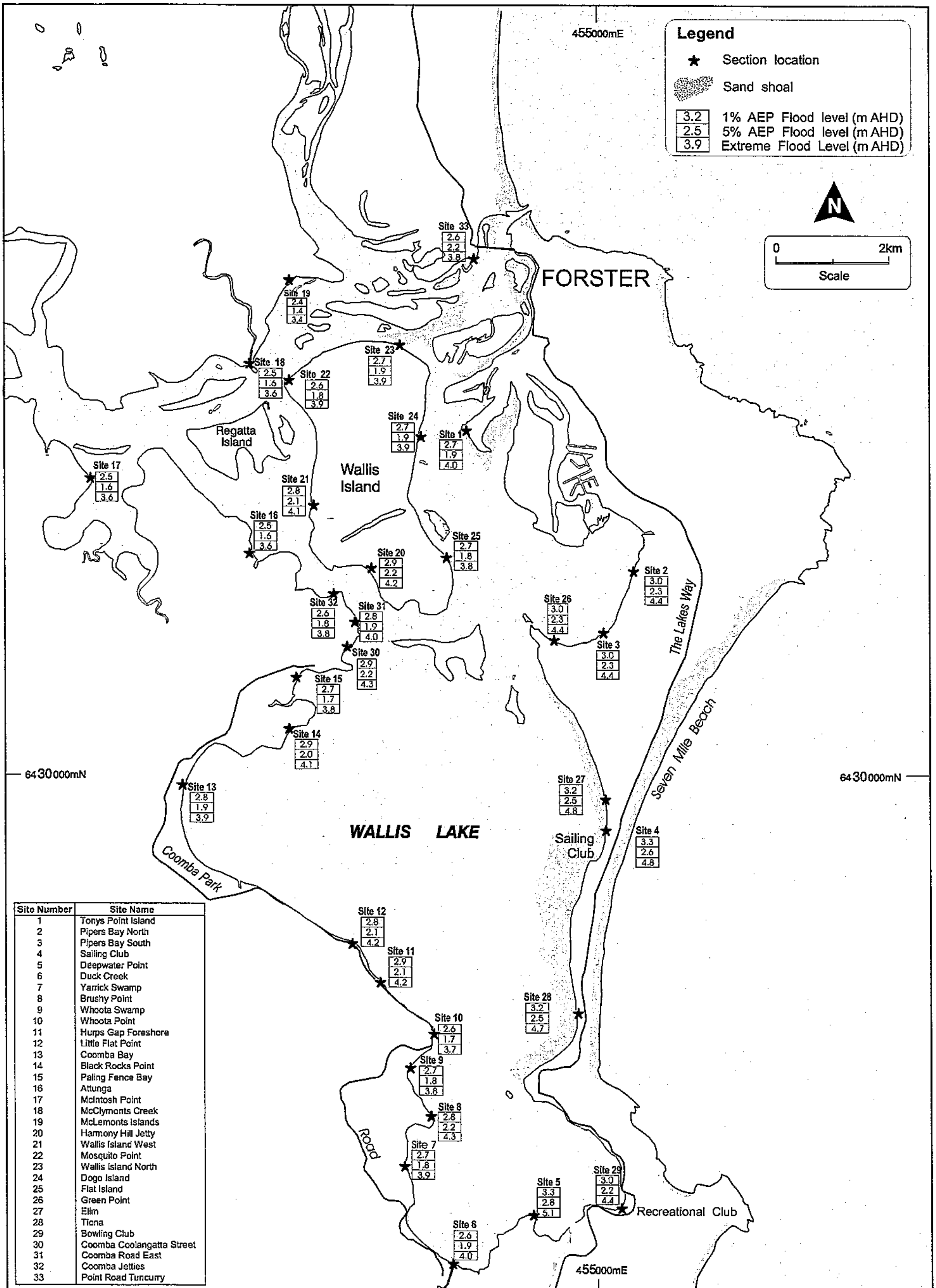
NSW DEPARTMENT OF PUBLIC WORKS AND SERVICES
MANLY HYDRAULICS LABORATORY

WALLIS LAKE FORESHORE FLOODING STUDY
LOCATION OF STUDY AREA

MHL Report 1023

Figure 1.1

DRAWING 1023WL11.CDR



Site Number	Site Name
1	Tonys Point Island
2	Pipers Bay North
3	Pipers Bay South
4	Sailing Club
5	Deepwater Point
6	Duck Creek
7	Yarrick Swamp
8	Brushy Point
9	Whoota Swamp
10	Whoota Point
11	Hurps Gap Foreshore
12	Little Flat Point
13	Coomba Bay
14	Black Rocks Point
15	Paling Fence Bay
16	Attunga
17	McIntosh Point
18	McClymonts Creek
19	McLemonts Islands
20	Harmony Hill Jetty
21	Wallis Island West
22	Mosquito Point
23	Wallis Island North
24	Dogo Island
25	Flat Island
26	Green Point
27	Elim
28	Tiona
29	Bowling Club
30	Coomba Coolangatta Street
31	Coomba Road East
32	Coomba Jetties
33	Point Road Tuncurry

NSW DEPARTMENT OF PUBLIC WORKS AND SERVICES
MANLY HYDRAULICS LABORATORY

WALLIS LAKE FORESHORE FLOODING STUDY
SELECTED SITES WITH RECOMMENDED DESIGN WIND-ELEVATED FLOOD LEVELS

MHL Report 1023

Figure 1.2

DRAWING 1023WL12.CDR

2. Study Approach

2.1 General

At any location around the foreshore of Wallis Lake, the flood level depends on the combination of river flooding (from the Wallingat, Coolongolook and Wallamba rivers), local wind effects and the adjacent ocean water level. Wind-generated wave action at the shoreline, inshore bathymetry and the presence of foreshore structures all impact on the degree of flooding experienced at any one given site. When waves reach the shoreline they break and expend the remaining energy as wave runup. When strong winds coincide with high flood levels the level of wave runup may form a major contribution to the degree of flooding of the foreshore (and any infrastructure located thereon). The aim of this study is to estimate the likely foreshore flood levels at the 33 foreshore locations around the lake (Figure 1.2), by combining the high water levels and wind wave climates.

Cross-sections were surveyed to provide the information required to estimate runup levels due to the combination of design water level and wave action at a particular site. Wave runup levels at each site were calculated using methods outlined in the *Shore Protection Manual* (CERC 1984) or in current research documents (van der Meer et al. 1994 and de Waal & van der Meer 1993).

2.2 Categorisation of Sites

The cross-section information for each of the 33 sites was used to categorise them into one of seven typical cross-sections (described in Section 3). Wave runup may then be calculated using different procedures appropriate to each category. The cross-sections were based on observations of each site during site inspections, together with detailed field survey levels. Wave setup along the foreshore at all locations on Wallis Lake was considered negligible when compared to the wave runup which was calculated as the increase in level above the still water level of the lake (i.e. the design still water level).

2.3 Wave Climate

Waves affecting water levels in foreshore areas around Wallis Lake originate locally from wind blowing across the lake. As the wind waves propagate toward the shore, the breaking and subsequent wave runup can affect foreshore water levels. The size of the wave generated depends on the exposed distance over the water surface (fetch length), depth of water and slope of the bed (bathymetry) and the speed, direction and duration of wind. Wind waves for this study are considered to be those which are generated across Wallis Lake. These waves are generally relatively small in height and are characterised by periods of 2 to 4 seconds between crests.

As a wave moves shoreward its wave length shortens, while its wave height first decreases slightly and then increases. As the wave steepens, it reaches a limiting value, which depends on the relative depth. At this time the wave breaks and a substantial amount of energy is dissipated in the breaking process. Depending on the wave characteristics and bottom slope, the wave may continue to move forward as a broken wave or it may reform into a wave of smaller height which continues to advance shoreward while increasing in steepness. The shallow water wave may become a bore, in which its height decreases as it moves shoreward before finally running up the beach, foreshore or shoreline structure. If the wave steepness fails to reach the limiting value for breaking, the wave simply advances to the shoreline without breaking and runs up the beach or shoreline structure.

Wave behaviour is generally characterised by a range (spectrum) of wave heights, wave periods, wave lengths and directions of propagation. It is therefore appropriate to treat wave parameters in a statistical form because the observed wave behaviour is the result of the interaction between all wave trains arriving at a location.

The water depth at which the wave breaks (d_b), the wave height at breaking (H_b), and the distance from the shoreline at which the wave breaks (x_b) were calculated using linear wave theory as outlined in the *Shore Protection Manual* (CERC 1984).

The foreshore flood levels in this study were estimated using the significant wave height (H_s). The definition of significant wave height states that one-third of all waves are higher than H_s and H_s accounts for design levels (CERC 1984). Hence a small percentage of larger waves are likely to occur at any site and they may produce higher runup or overtopping and/or more severe inundation.

2.3.1 Influence of Oblique Waves

Waves will approach the shoreline at a range of angles but the presence of the shoreline and the bathymetry tend to refract and diffract the approaching waves, resulting in a realignment of wave crests parallel to the shore. As runup is generally at a maximum when the angle of wave approach is at right angles to the shoreline, the analyses carried out for this study assumes that all waves are approaching normal to the shoreline.

2.3.2 Design Criteria for Wind

The design criteria adopted for the development of wind waves and runup have been defined previously (MHL 1998a). Wind conditions are presented in Table 2.1.

For purposes of wave hindcasting a detailed description of the local wind climate is required. Wind recordings are used as the basis for this description. Wind data is essential to design calculations because of the difficulty and expense incurred in routine wave data collection at specific sites. The directional distribution of winds usually presented as a wind rose assists in assigning directional properties to a hindcast wave climate. These directional properties then define the appropriate fetch lengths which are then utilised to estimate flood inundation levels caused by wave runup on shoreline structures.

Wind and wave data was not available for the Wallis Lake area prior to this investigation. The nearest wind monitoring sites are located at Taree and Williamtown. Wind data for Williamtown from 1950 (50 years) and for Taree from 1965 (35 years) was available. Williamtown is situated approximately 60 km south of Wallis Lake and about 5 km from the coast. Taree is situated approximately 40 km north of Wallis Lake and about 15 km from the coast.

An important factor in choosing representative wind data is proximity to the coast. The major fetch lengths on Wallis Lake are roughly 4 km from the coast, similar to the proximity of the Williamtown wind recorder from the coast. As a result, a decision was made to use the Williamtown wind recorder data as opposed to Taree data because of their respective distances from the coast. It is believed that the data from Williamtown is more representative of Wallis Lake than the data from Taree.

The recurrence intervals for the Williamtown wind data was sourced from two letter reports by Lawson and Treloar Pty Ltd (MHL file 128 - 29/9/94 and 5/5/97) (Appendix B) which were produced for the Port Stephens Flood Study (MHL1998b). Long-term data from Williamtown (1942-present) was compared with short-term data from Jimmys Beach (19/1/84 to 6/3/85). The median directional difference between the two sites was 5 degrees and wind speeds showed similar good agreement. Although funnelling effects through the mountains at Wallis Lake could lead to discrepancies, modelling these effects would be beyond the scope of this report.

In order to adopt wave hindcasting procedures used in the *Shore Protection Manual* (CERC 1984) measured wind speeds must be converted to a time-dependent average wind speed. Preliminary calculations using the shallow water forecasting curves (CERC 1984) indicated that for the lengths of fetch and depth over Wallis Lake, durations for equilibrium wave conditions were close to 60 minutes. These values are indicated in Table 2.1. The values for the 16-sector wind climate were obtained by linearly interpolating the values in this table.

Table 2.1 Wind Data

(60-minute averaged)

	Return Period		
Direction	**1-in-1 year (m/s)**	**1-in-20 year (m/s)**	**1-in-100 year (m/s)**
N	5.5	11.5	14
NE	8.5	12	13
E	11.9	15	16
SE	11.9	15	16
S	15.7	19.5	20.5
SW	12.4	20	24
W	21.9	32	35
NW	19.0	28.6	32.4

The fetch for each direction at each site is presented in Table 2.2.

Fetch lengths (Table 2.2) are combined with wind values (Table 2.1) and the corresponding water levels to calculate the significant wave height and period at each site and for each direction. The highest wave height that results from the combinations of wind speed and fetch is chosen as the design wave height. The wind speed for the NNE was obtained by linearly interpolating between the winds from the north and the north-east.

Table 2.2 Wallis Lake Fetch Lengths (km) for Each Site and Wind Direction

Direction	N	NNE	NE	ENE	E	ESE	SE	SSE	S	SSW	SW	WSW	W	WNW	NW	NNW
Site 1								4.0	**3.5**	1.5	1.5	1.0	0.8	1.0	1.0	
Site 2	0.7										1.9	3.0	**3.2**	1.0	1.0	1.1
Site 3	1.8	2.4	1.5												**4.0**	3.7
Site 4											4.7	5.2	**7.5**	6.0	3.7	7.5
Site 5	2.0	1.7	1.7	1.6	1.9	0.7	0.6								2.8	**10.3**
Site 6	**10.5**	3.2	0.4													
Site 7	1.0	**8.0**	4.1	2.9	3.7	2.0	2.0	1.4								
Site 8	1.5	**7.0**	3.5	1.4	2.5	2.2	2.6	2.2	1.0							
Site 9			4.1	3.2	2.0	**4.1**	3.9									
Site 10			3.7	2.7	2.5	1.7	4.0	**3.6**	1.5							
Site 11	4.7	5.2	5.5	4.8	3.8	4.2	**6.6**								4.1	3.6
Site 12	3.2	5.0	5.5	5.8	4.9										**2.8**	3.1
Site 13			0.8	1.0	7.2	7.2	**10.7**	1.9	1.2							
Site 14					5.2	5.8	**7.2**	5.1	3.2							
Site 15					2.0	5.2	**6.0**	0.6	0.6	0.6	0.6	0.6	0.4			
Site 16	1.5	2.4	1.1													**1.5**
Site 17	1.1	1.3	1.1	**2.7**												
Site 18			1.8	0.8	1.0	0.8	1.0	**1.2**								
Site 19					0.3	0.1	0.1	0.2	**0.2**							
Site 20									**7.0**	0.7	0.6	0.6	1.4			
Site 21										1.2	1.5	1.4	**2.0**	0.9	0.7	0.9
Site 22	0.4	0.6	1.1									1.1	0.9	0.9	**1.1**	0.3
Site 23	0.3	0.3	2.4	0.7										**1.1**	0.7	0.3
Site 24		1.7	1.3	0.9	1.1	1.7	**4.5**									
Site 25		2.0	1.5	1.7	**3.4**	3.2	2.0									
Site 26	1.8	2.4	1.5												**4.0**	3.7
Site 27											4.7	5.2	**7.5**	6.0	3.7	7.5
Site 28											4.2	3.2	3.2	**7.4**	6.9	6.3
Site 29													3.9	**4.2**	4.5	
Site 30					1.5	3.5	5.8	**8.0**								
Site 31	1.2	1.2	0.6	0.6	3.6	1.4	**6.0**									
Site 32	0.2	0.2	0.3												**1.0**	0.2
Site 33						0.75	0.5	0.1	0.1	0.1	0.25	**1.0**	0.25			

Note: Fetch lengths in **bold** give the greatest runup for the 1% AEP wind-Elevated flood level.

2.4 Design Still Water Levels

The design wind-elevated water level at any location is influenced by some combination of prevailing local wind setup, catchment runoff from rainfall, rain falling directly onto the lake and the oceanic water levels at the time.

Design still water flood water levels in Wallis Lake, in the absence of wind effects, were estimated previously using mathematical modelling techniques (PWD 1989 and DLWC 1992).

Design flood discharges entering the lake were derived using the design rainfall data and procedures outlined in *Australian Rainfall and Runoff* (IEAust 1997). The Wallingford numerical model was used to identify the hydraulic behaviour of Wallis Lake under a range of conditions (PWD 1989). The Wallingford Model was upgraded to the MIKE-11 model in 1992 'to better define flood behaviour when assessing options' (DLWC 1992). The location of model cross-sections, inflow hydrographs and the ocean boundary condition are shown in Figure 2.1.

The critical duration rainfall was found to be the 36-hour and 48-hour design rainfalls. For this study design flood levels have been based on the results of the 1989 Forster/Tuncurry Flood Study (PWD 1989). The 1% AEP flood levels from the PWD (1989) and the DLWC (1992) flood studies are given in Table 2.3. The DLWC (1992) report states that the flood levels obtained from the MIKE-11 model were generally in good agreement with the flood levels obtained in the 1989 study using the Wallingford model and concluded that the 1989 modelling thus provided an appropriate basis for establishing designated still water flood levels.

Table 2.3 Design Still Water Levels for Existing Conditions

Location	Forster Tuncurry Bridge	Darawank	Wallingat River Junction	Wang Wauk River Junction	Forster Keys
1%AEP design still water level (PWD 1989)	2.28	2.65	2.70	3.51	2.17
Note: Peak 1% flood level for Wallis Lake, including Pipers Bay and Forster Keys, is 2.2 m AHD (PWD 1989)					
Location	**Forster Tuncurry Bridge**	**Darawank**	**Wallingat River Junction**	**Wang Wauk River Junction**	**Forster Keys**
Extreme	3.27	4.51	4.97	6.67	no output
1%AEP design still water level (DLWC 1992)	2.28	2.67	2.69	3.57	2.24
5% AEP design still water level (DLWC 1992)	1.90	2.22	2.18	2.93	no output
Note: Peak 1% flood level for Wallis Lake, including Pipers Bay and Forster Keys, is 2.25 m AHD (DLWC 1992)					

The 100% AEP and the extreme still water flood levels were derived by extrapolation of the flood levels presented in the DLWC (1992) report. All other levels were defined directly by the interpretation of MIKE-11 modelling performed by DLWC (1992). No significant disagreements were found within the interpretation presented and levels have been adopted as presented in Table 2.4.

Table 2.4 Adopted Wallis Lake Design Still Water Levels

	1% AEP Flood	5% AEP Flood	100% AEP Flood	Extreme Flood
Wallis Lake design still water level	**2.28**	**1.90**	**1.26**	**3.27***

* based on the hydrodynamic model results and the model calibration runs (DLWC 1992)

2.5 Coincidence of Wind Wave Effects and Rain

A study on the joint occurrence of wind, rainfall and flood levels in Lake Macquarie (MHL 1997b) concluded that combining the 1% AEP rainfall-generated flood with the 1% AEP-generated wind waves is overly conservative. Based on the results of the Forster/ Tuncurry flood study (PWD 1989), the Lake Macquarie study (MHL 1998a) and the Port Stephens flood study (MHL 1998b) the following combinations of water level and wave climate were adopted to calculate the wind-elevated foreshore flood conditions of Wallis Lake:

Guideline 1 - For the entire area of Wallis Lake foreshores, the 1% AEP (2.28 m AHD) and 5% AEP (1.9 m AHD) still water level be combined with the wind waves generated by 100% AEP winds to estimate the 1% AEP and 5% AEP foreshore flood levels.

Guideline 2 - For the entire area of Wallis Lake, the 100% AEP (1.26 m AHD) still water levels be combined with the 1% AEP and 5% AEP wind waves to determine the 1% AEP and 5% AEP foreshore flood levels.

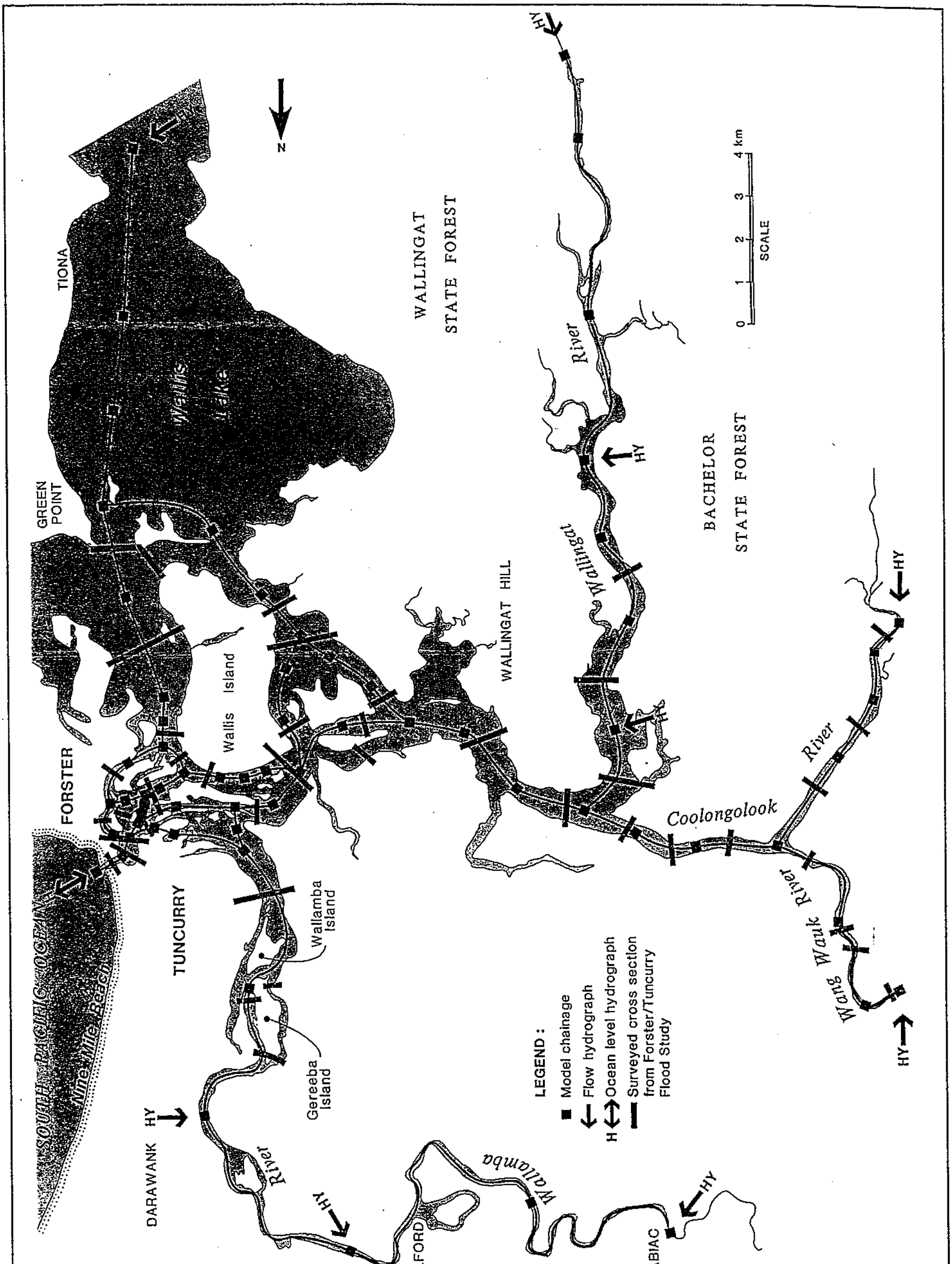
Guideline 3 - The adopted 1% AEP and 5% AEP foreshore flood level be the maximum of Guidelines 1 and 2.

During the Wallis Lake Floodplain Management Study , the extreme wind-elevated water levels will need to be considered for planning purposes. When the extreme flood occurs there are likely to be wind waves active on the lake. How the extreme water level and the wind waves will interact with the foreshore configuration to result in wind-elevated foreshore flooding is very difficult to predict accurately. To be consistent with the approach adopted in the Lake Macquarie study (MHL 1998a) the following guideline was adopted:

Guideline 4 - The extreme foreshore flood level will be the extreme flood level (3.27 m AHD) combined with the 1% AEP wind waves.

2.6 Sensitivity Analysis

A sensitivity analysis was carried out to investigate the influence of error in wind speed estimates on runup values at Wallis Lake. When the wind speed values were increased by 20% the runup values increased by approximately 20%, indicating that for the relatively small fetch values at Wallis Lake a general trend towards a one-to-one relationship exists between wind speed and runup.



SOURCE: DLWC 1992

NSW DEPARTMENT OF PUBLIC WORKS AND SERVICES
MANLY HYDRAULICS LABORATORY

WALLIS LAKE FORESHORE FLOODING STUDY
LOCATION OF CROSS-SECTIONS
FROM DLWC (1992) MIKE-11 HYDRODYNAMIC MODEL

MHL Report 1023

Figure 2.1

DRAWING 1023WL21.CDR

3. Description of Typical Cross-Sections and Method of Estimation of Foreshore Flood Levels

3.1 General

A site inspection was undertaken in May 1999 to determine the characteristics of representative cross-sections at the 33 sites. The cross-sections were based on information gathered during site inspection, together with detailed survey information provided by Great Lakes Council and included in Appendix A.

For calculation purposes, the sites were broadly categorised into seven cross-section types which are shown schematically in Figures 3.1 to 3.8. These categories, and the method of estimating the design foreshore flood level at each typical site, are described below. Figures 3.9 to 3.11 show photographs of typical type cross-sections taken from Wallis Lake.

3.2 Classification of Cross-sections

3.2.1 Type 1 - Gently Sloping Embankment - Wave Breaks Offshore

At this cross-section, characterised by a gently sloping embankment, the design wave breaks offshore from the shoreline (Figure 3.1). During the wave breaking process, considerable wave energy is dissipated and a reformed wave is produced by the wave spilling/breaking process. To determine the design flood level it was assumed that the reformed wave, H_r will raise the water level by $H_b/2$ (AWACS 1991). Therefore the design flood level was given by:

$$\textit{Design Flood Level} = \textit{Design Still Water Level} + H_b/2$$

H_b was calculated using the procedures given in CERC (1984).

An alternative equation is proposed in a paper by De Waal and Van der Meer (1992) to estimate the runup on smooth slopes. The equation is

$$R_1 = 1.6\, H_s\, (L_s/H_s)^{0.5} \tan\beta \quad (R/H_s < 3.2)$$

where β is the angle of the seabed.

This equation is adjusted for roughness, shallow water and oblique wave attack. By adopting appropriate coefficients for Wallis Lake this equation becomes:

(1) Grass embankment $R_1 = 1.2\, H_s\, (L_s/H_s)^{0.5}\, \mathrm{Tan}\, \beta \quad (R/H < 3.2)$

(2) Rock sea wall $R_1 = 0.6\, H_s\, (L_s/H_s)^{0.5}\, \mathrm{Tan}\, \beta \quad (R/H < 3.2)$

where β is the angle of the seabed.

Therefore the design flood level was given by:

$$Design\ Flood\ Level = Design\ Still\ Water\ Level + R_1$$

Estimations of wave runup for this type of cross-section were made using both of these procedures and the larger wave runup value adopted. If the cross-section was characterised by a long even slope with a gradient steeper than 2:1, the methods used for Type 2 or Type 3 profiles were used to estimate the design flood levels.

3.2.2 Type 2 - Sloping Embankment or Sea Wall - Wave Breaking Close to Shoreline Resulting in No Overtopping

The typical cross-section for sites with a sloping embankment where the broken wave does not overtop the structure is shown in Figure 3.2. The runup, R_2, was evaluated using the procedure given by de Waal and Van der Meer (1992) to estimate the runup on smooth slopes applied to this type of cross-section. The following equation was also used to estimate wave runup at these types of cross-sections:

(1) Grass embankment $R_2 = 1.2\ H_s\ (L_s/H_s)^{0.5}\ \text{Tan}\ \beta\ (R/H < 3.2)$
(2) Rock sea wall $R_2 = 0.6\ H_s\ (L_s/H_s)^{0.5}\ \text{Tan}\ \beta\ (R/H < 3.2)$

where β is the angle of the embankment or sea wall.

Therefore the design flood level was given by:

$$Design\ Flood\ Level = Design\ Still\ Water\ Level + R_2$$

3.2.3 Type 3 - Sloping Embankment or Sea Wall - Wave Break Close to Shoreline Resulting in Overtopping

The typical cross-section for sites with a sloping embankment where waves break close to the shoreline and the breaking wave overtops the embankment is shown in Figure 3.3. There is no published information to compute the flood level resulting from this type of overtopping. Studies undertaken to assess foreshore flooding in Pittwater (AWACS 1991) addressed this problem and outlined a solution to the problem.

The following procedure, as used in the Pittwater study (AWACS 1991), was adopted to compute the likely flood level in this situation:

- The design wave height at break, H_b, water depth at break, d_b and distance of wave break, x_b from the sea wall were calculated using the procedure given in CERC (1984).

- The overwash volume would be the primary cause of any flooding behind the foreshore. The overwash from the breaking wave will move as a front landward, changing shape and height until it comes up against a solid barrier or dissipates. A visualisation of the overwash versus time is shown in Figure 3.4. Prediction of the movement of the overwash once it passes the sea wall cannot be determined using any known standard procedures.

However, an estimate of the possible depths of flooding at a set distance from the sea wall can be assessed by assuming the overwash volume is represented by an equivalent depth over the region between the sea wall and some set distance such as the wall of a foreshore house. To estimate the depth of overtopping, the overwash equivalent depth as shown in Figure 3.5 was adopted.

- Because of likely wave runup against the end of the wall containing the overwash, it is estimated that the maximum depth is in the order of 0.2 m greater than the uniform water depth as shown in Figure 3.5.

- The overwash volume was calculated taking into consideration the height of the breaking wave, H_b, the water level of the lake, the height of the foreshore structure and the wave period at the depth of break.

The design flood level is therefore defined as:

Design Flood Level = Height of Sea Wall + Equivalent Overtopping Depth + 0.2

3.2.4 Type 4 - Sloping Embankment or Sea Wall - Design Water Level Higher than the Top of the Sea Wall, Wave Breaking Offshore and Reforming

The typical cross-section for sites with a sloping embankment where waves break offshore of the shoreline and the reformed waves or surge passes over the foreshore structure is shown in Figure 3.6. This type was also used where the wave did not break offshore and actually continued over the top of the foreshore structure. Again, there is no published information to compute the flood level resulting from this type of overtopping. Studies undertaken to assess foreshore flooding in Pittwater (AWACS 1991) addressed this problem and outlined a solution to the problem.

The following procedure was adopted to compute the likely flood level in this situation:

- The design wave height at break, H_b, water depth at break, d_b, and distance of wave break, x_b, from the sea wall were calculated using the procedure given in CERC (1984).

- During the wave breaking process considerable energy is dissipated and the reformed wave from the spilling/breaking process is unlikely to be more than half of the wave height when reaching the shoreline.

- If the wave breaks offshore and reforms then H_r was assumed to be $H_b/2$. If the wave did not break then H_r was assumed to equate to H_b.

The design flood level is therefore defined as:

$$\textit{Design Flood Level} = SWL + H_r$$

3.2.5 *Type 5 - Gently Sloping Sandy Embankment - Wave Breaks Offshore*

At this cross-section, characterised by a gently sloping sandy embankment, the design wave breaks offshore from the shoreline (Figure 3.1). Work by Hanslow and Nielsen (1995) derived the following equation for runup, R_5, on natural beaches:

$$R_5 = 0.9\ H_s\ (L_s/H_s)^{0.5} \tan\beta$$

where β is the angle of seabed.

This equation was adopted for this study, with the foreshore flood level being defined as:

Design Flood Level = Design Still Water Level +R_5

3.2.6 *Type 6 - Sudden Change in Bathymetry - Wave Breaks Close to Structure*

With a sudden change in bathymetry and the wave breaking close to the shoreline, the wave runup on a near-vertical structure is shown in Figure 3.7. This situation would apply to structures located on the foreshore, and the runup would estimate the likely flood level on the side of the structure. The runup, R_6, was calculated using the procedures given in CERC (1984). The flood level would be defined as:

Design Flood Level = Design Still Water Level +R_6

3.2.7 *Type 7 - Sloping Embankment or Sea Wall - Wave Breaking Offshore and Reformed Wave Resulting in Overtopping of the Embankment*

The typical cross-section for sites with a sloping embankment where waves break offshore and reform, with the reformed wave overtopping the embankment, is shown in Figure 3.8. This type is similar to Type 3, however the waves have broken offshore and reformed. Again, there is no published information to compute the flood level resulting from this type of overtopping. Studies undertaken to assess foreshore flooding in Pittwater (AWACS 1991) addressed this problem and outlined a solution to the problem.

The following procedure was adopted to compute the likely flood level in this situation:

- The design wave height at break, H_b, water depth at break, d_b, and distance of wave break, x_b, from the sea wall were calculated using the procedure given in CERC (1984).

- During the wave breaking process considerable energy is dissipated and the reformed wave from the spilling/breaking process is unlikely to be more than half of the wave height when reaching the shoreline. When the wave breaks offshore the reformed wave, H_r, was assumed to be $H_b/2$.

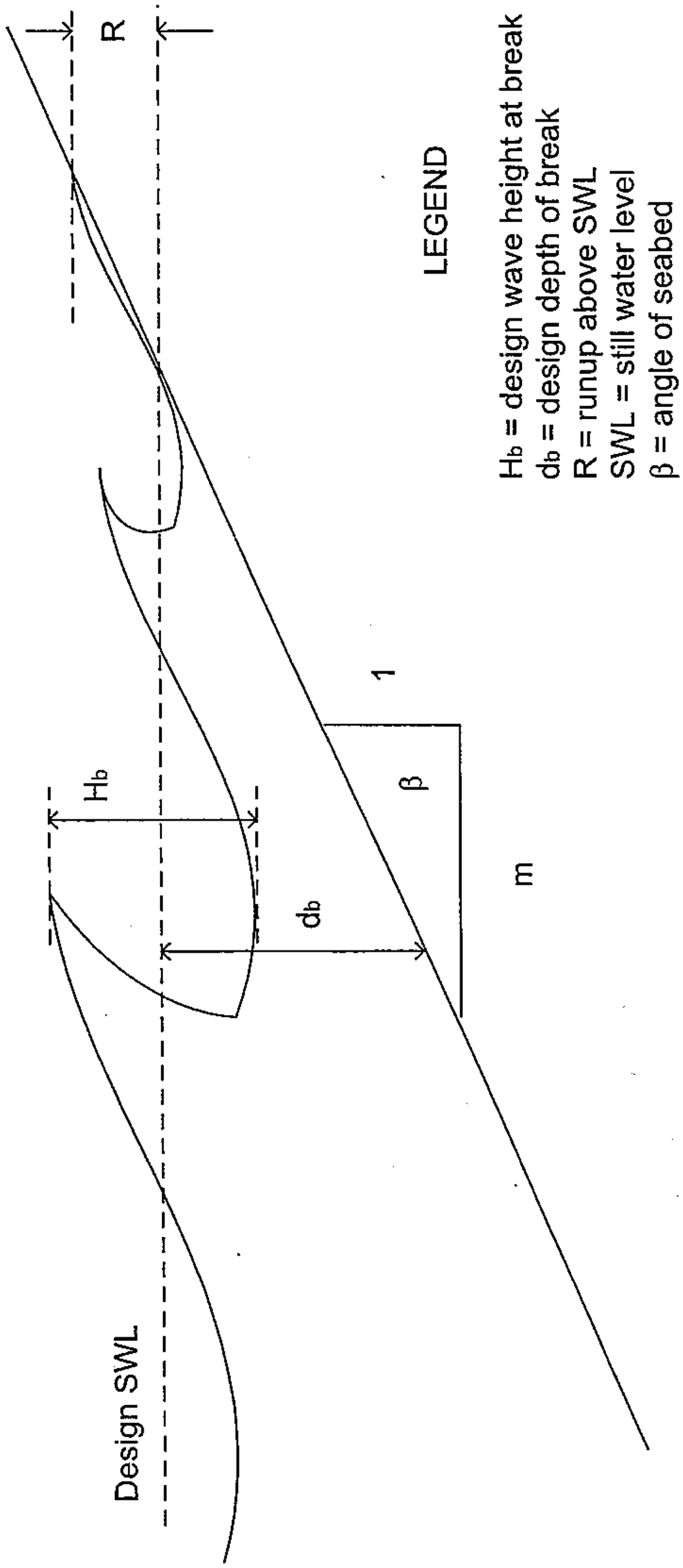
- The overwash volume would be the primary cause of any flooding behind the foreshore. The overwash from the breaking wave will move as a front landward, changing shape and height. A visualisation of the overwash versus time is shown in Figure 3.4. Prediction of the movement of the overwash once it passes the sea wall cannot be determined using standard procedures. However, an estimate of the possible depth of flooding at a set distance from the sea wall can be assessed by assuming the overwash volume is represented by an equivalent depth over the region between the sea wall and the set distance. To estimate the depth of overtopping, the overwash equivalent depth as shown in Figure 3.5 was adopted.

- It is estimated that the maximum depth is in the order of 0.2 m greater than the uniform water depth as shown in Figure 3.5.

- The overwash volume was calculated, taking into consideration the height of the breaking wave, H_b, the water level of the lake, the height of the foreshore structure and the wave period at the depth of break.

The design flood level is therefore defined as:

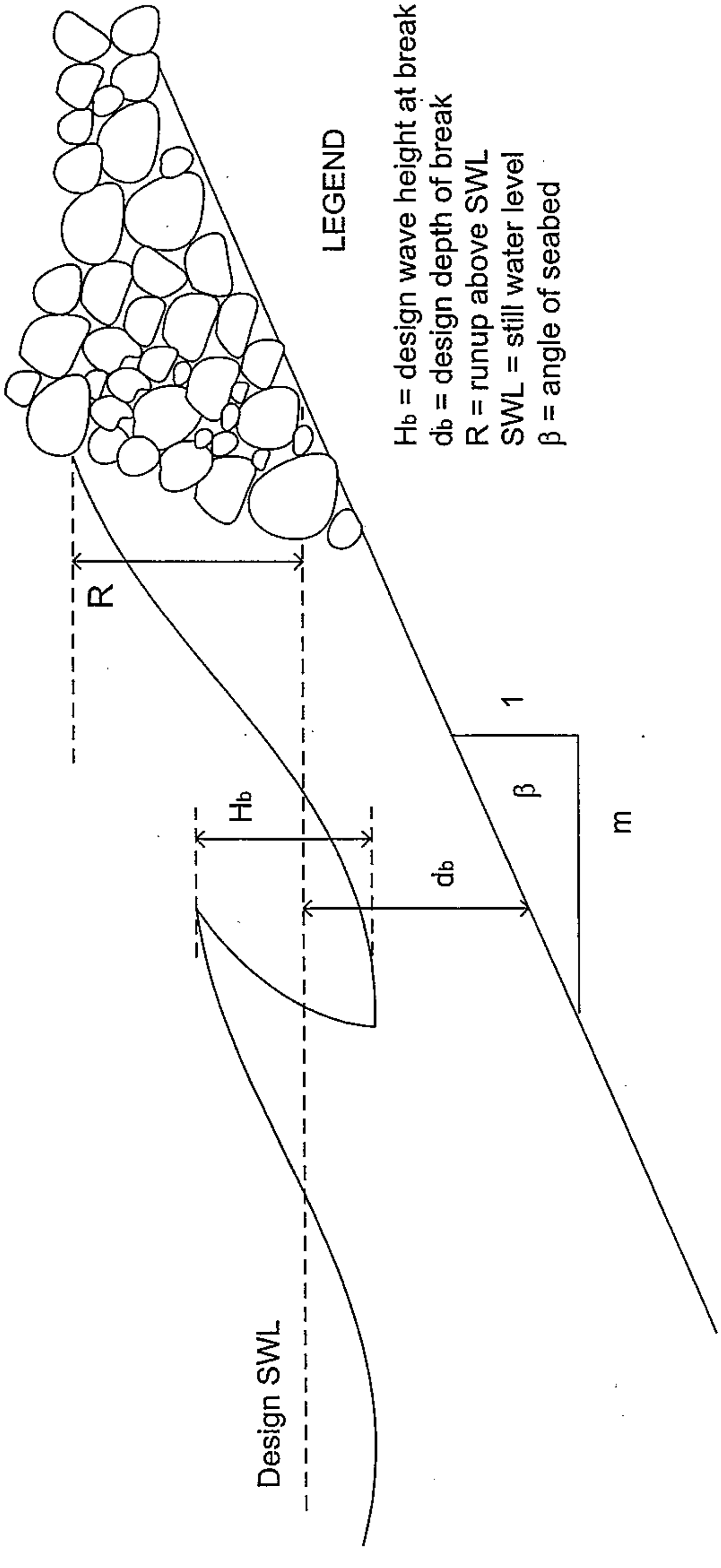
Design Flood Level = Height of Sea Wall + Equivalent Overtopping Depth + 0.2



NSW DEPARTMENT OF PUBLIC WORKS AND SERVICES
MANLY HYDRAULICS LABORATORY

TYPICAL CROSS-SECTION
TYPE 1 AND TYPE 5

MHL Report 1023
Figure 3.1
DRAWING 1023wf31.cdr



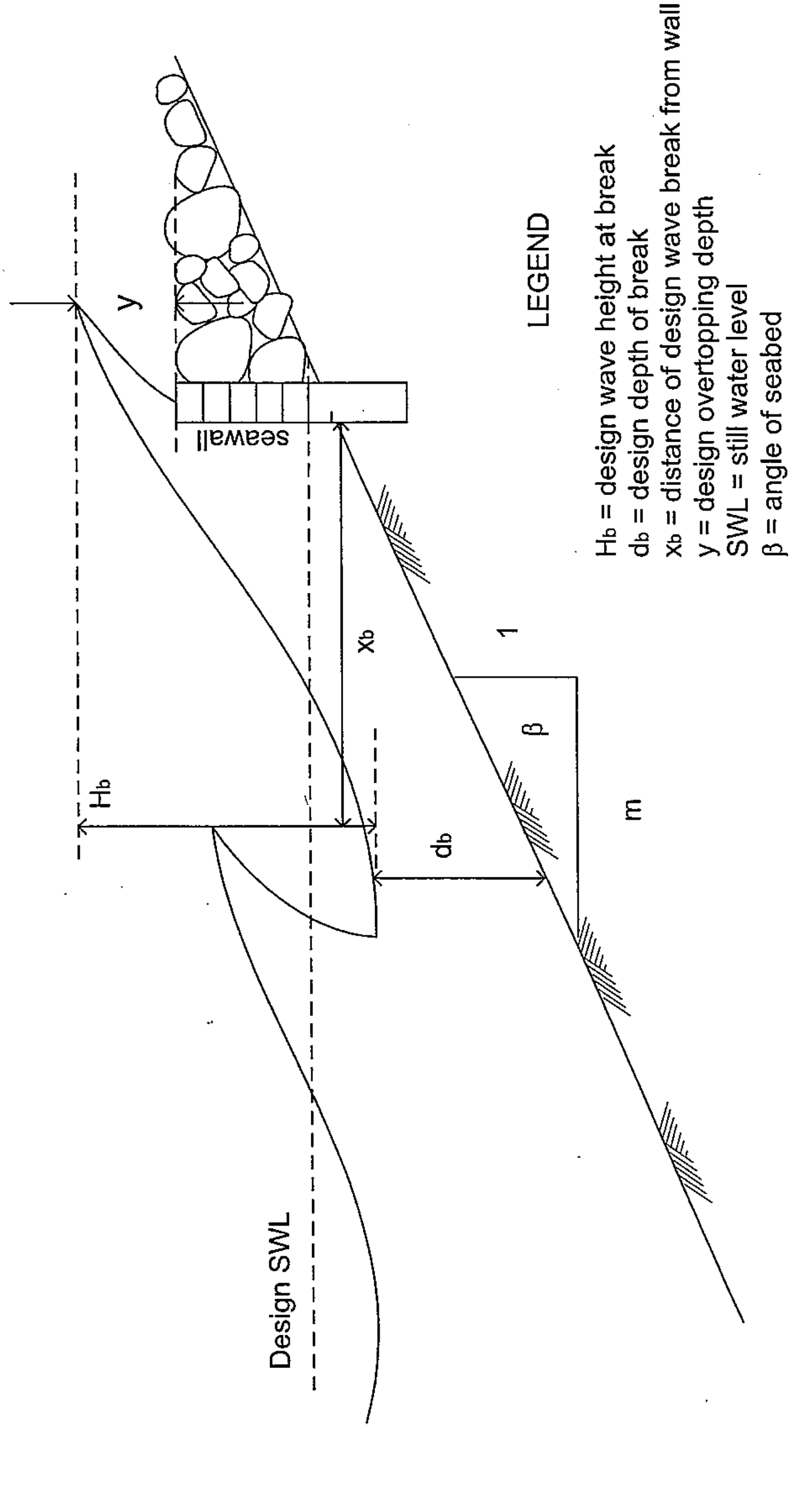
NSW DEPARTMENT OF PUBLIC WORKS AND SERVICES
MANLY HYDRAULICS LABORATORY

TYPICAL CROSS-SECTION TYPE 2

MHL Report 1023

Figure 3.2

DRAWING 1023w132.cdr



NSW DEPARTMENT OF PUBLIC WORKS AND SERVICES

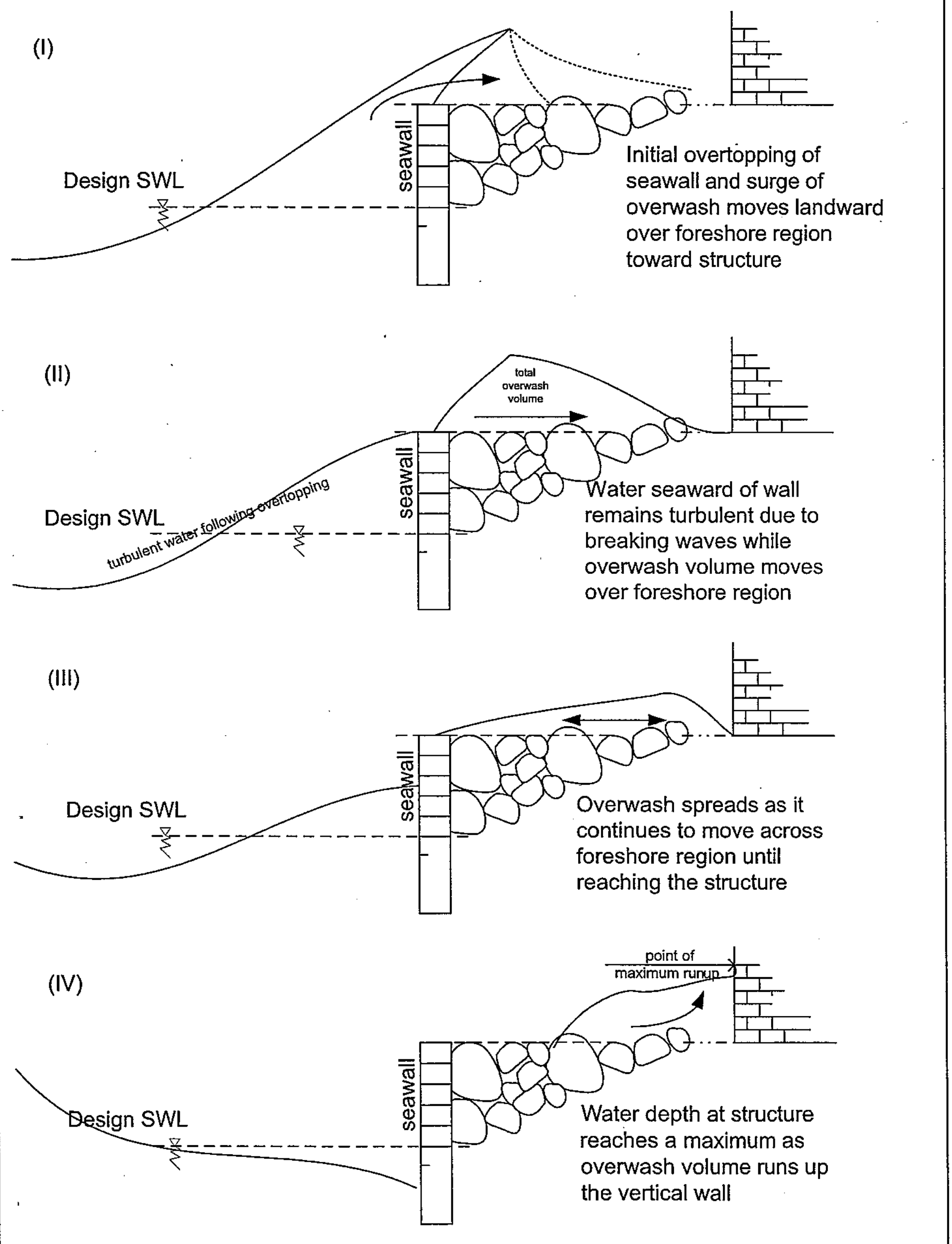
MANLY HYDRAULICS LABORATORY

TYPICAL CROSS-SECTION TYPE 3

MHL Report 1023

Figure 3.3

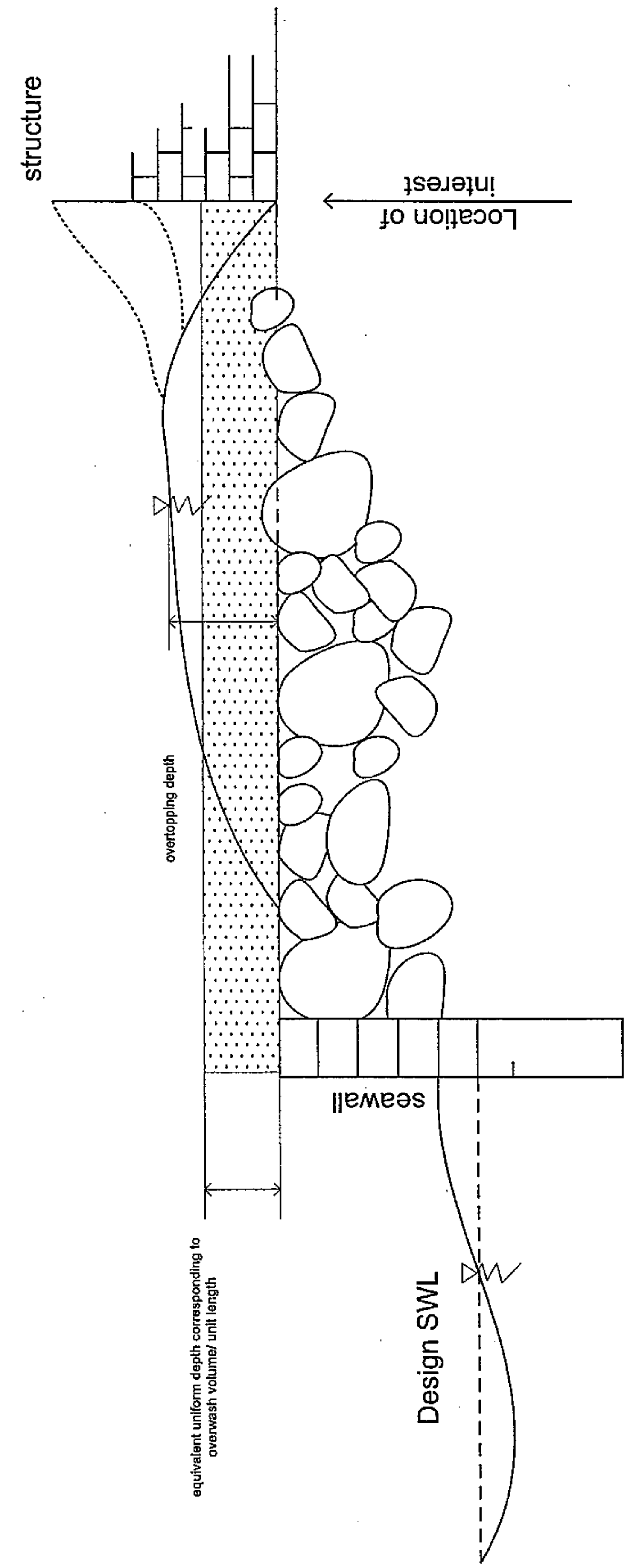
DRAWING 1023wf33.cdr



NSW DEPARTMENT OF PUBLIC WORKS AND SERVICES
MANLY HYDRAULICS LABORATORY

TYPICAL CROSS-SECTION TYPE 3
OVERWASH PROFILE vs. TIME

MHL Report 1023
Figure 3.4
DRAWING 1023wt34.cdr



NSW DEPARTMENT OF PUBLIC WORKS AND SERVICES

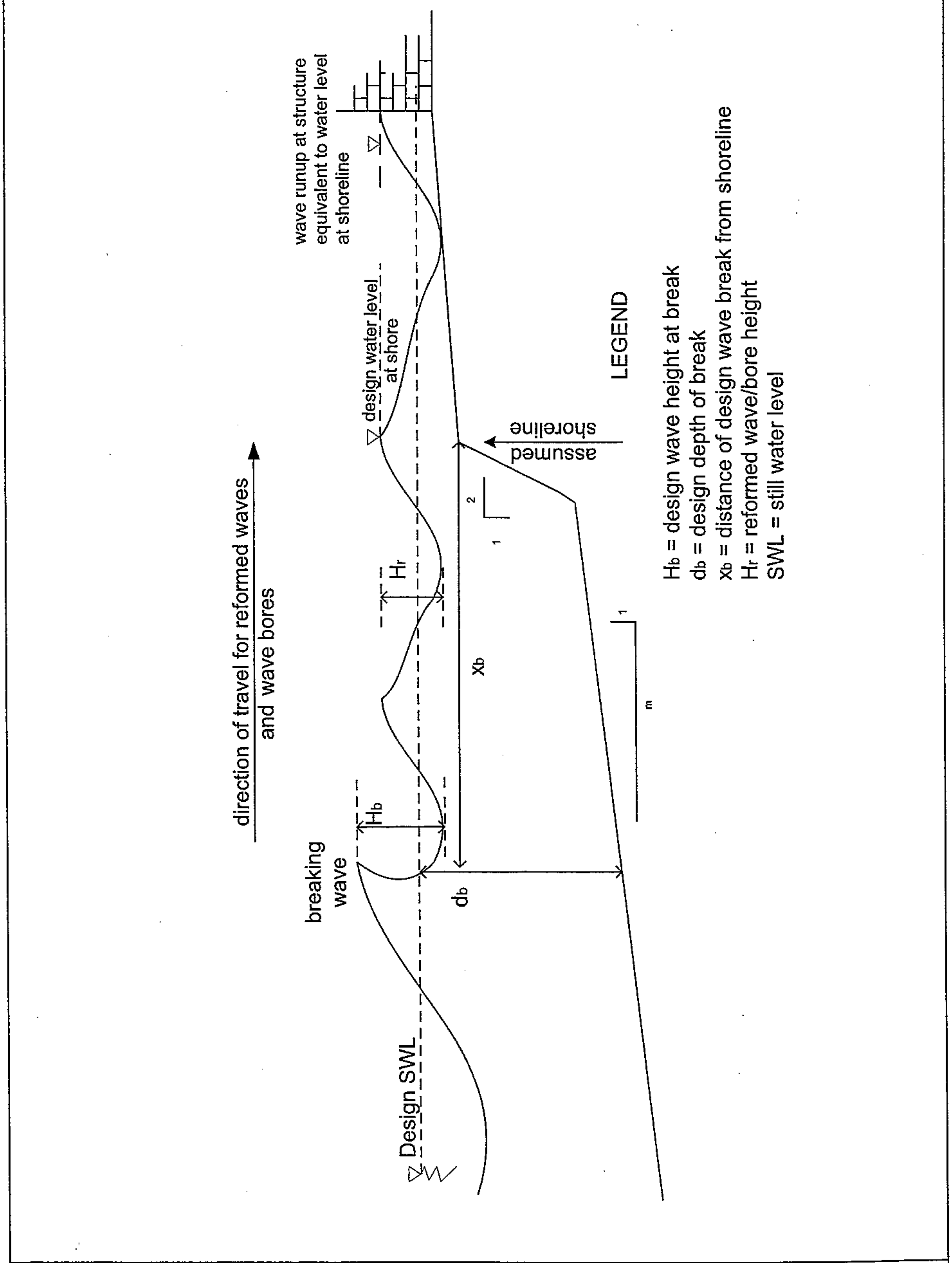
MANLY HYDRAULICS LABORATORY

TYPICAL CROSS-SECTION TYPE 3
OVERWASH EQUIVALENT DEPTH

MHL Report 1023

Figure 3.5

DRAWING 1023wt35.cdr



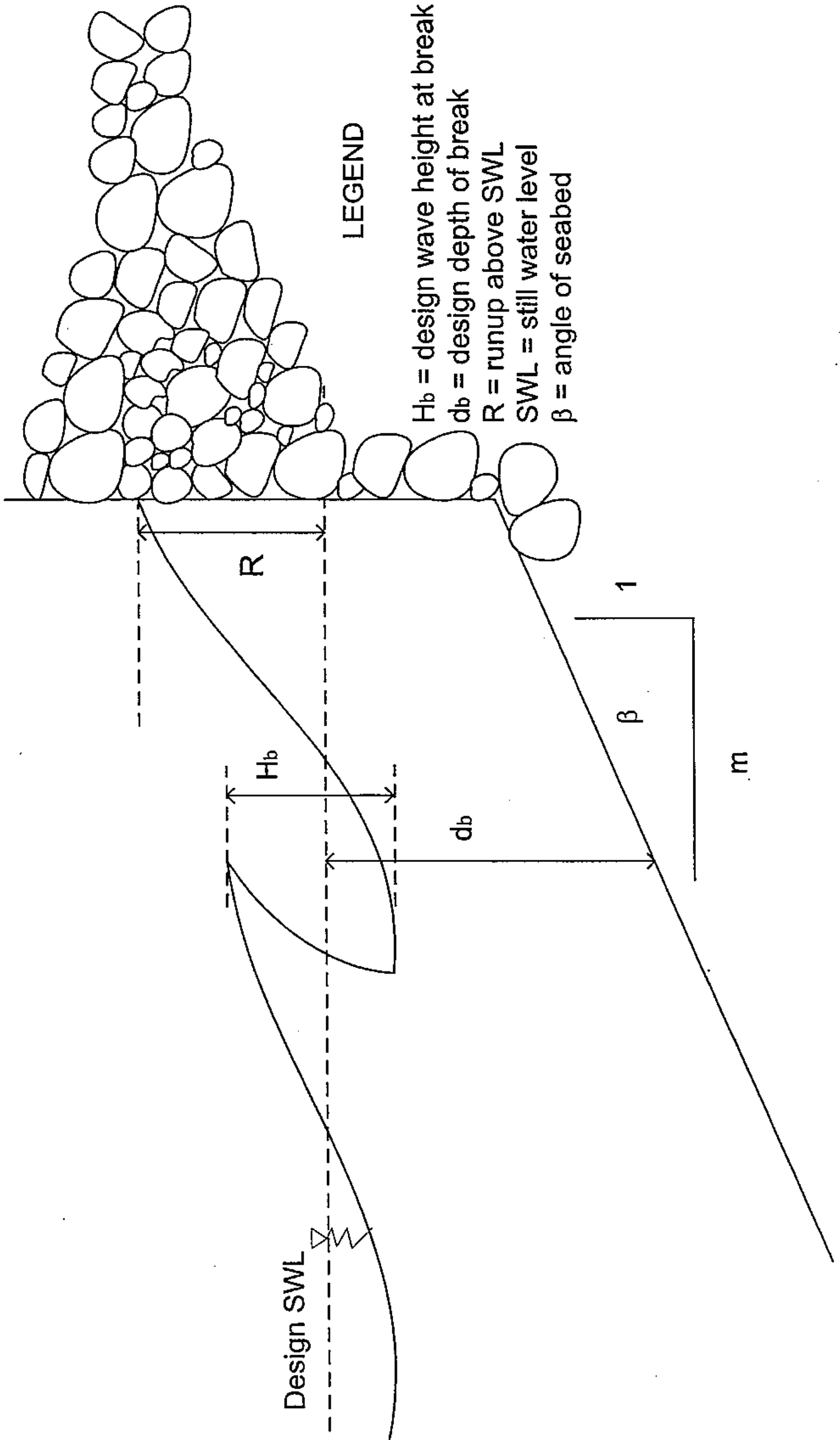
NSW DEPARTMENT OF PUBLIC WORKS AND SERVICES
MANLY HYDRAULICS LABORATORY

TYPICAL CROSS-SECTION TYPE 4

MHL Report 1023

Figure 3.6

DRAWING 1023wt36.cdr



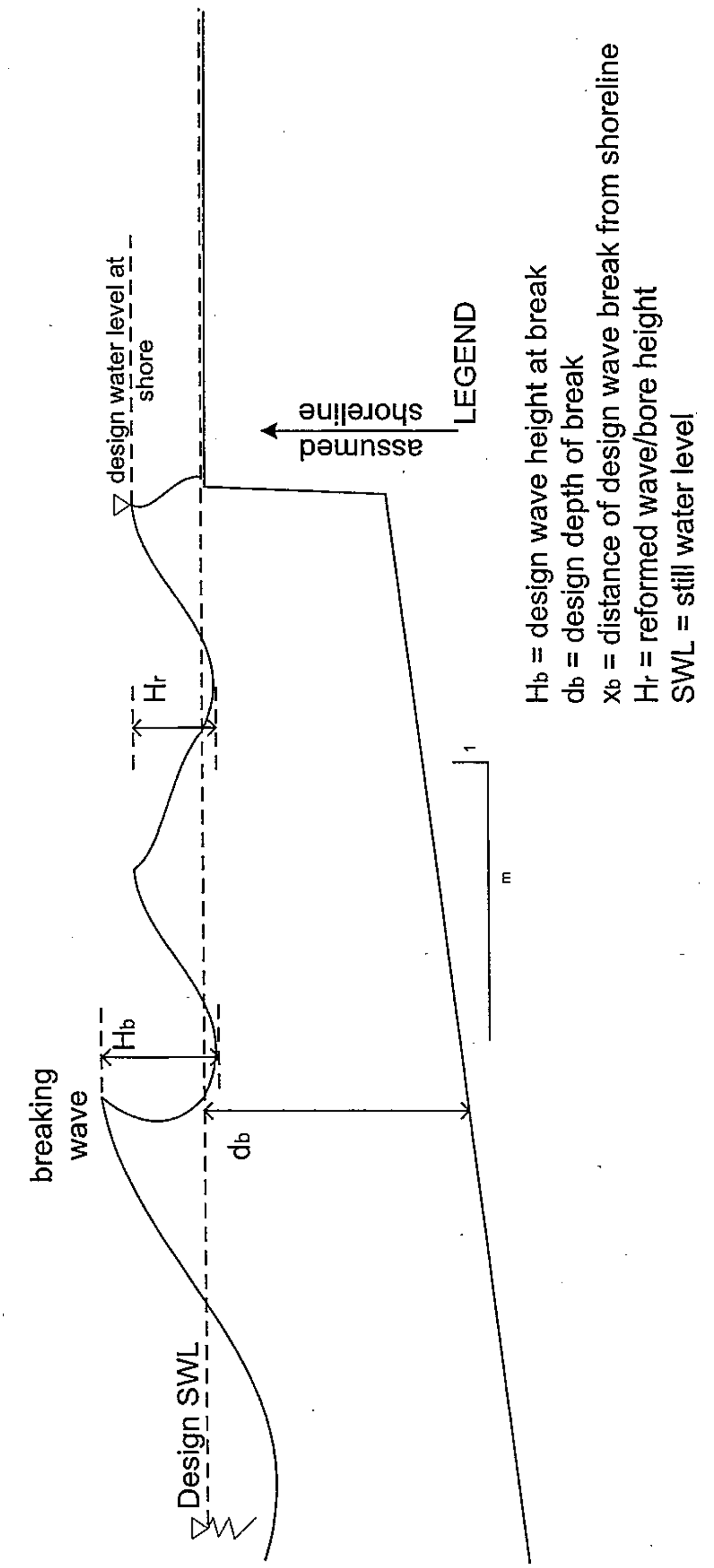
NSW DEPARTMENT OF PUBLIC WORKS AND SERVICES
MANLY HYDRAULICS LABORATORY

TYPICAL CROSS-SECTION TYPE 6

MHL Report 1023

Figure 3.7

DRAWING 1023wl37.cdr



NSW DEPARTMENT OF PUBLIC WORKS AND SERVICES
MANLY HYDRAULICS LABORATORY

TYPICAL CROSS-SECTION TYPE 7

MHL Report 1023

Figure 3.8

DRAWING 1023wl38.cdr

NSW DEPARTMENT
OF PUBLIC WORKS
AND SERVICES
MANLY HYDRAULICS LABORATORY

TYPICAL CROSS-SECTION TYPE 1
PHOTOGRAPH

MHL
Report 1023

Figure
3.9

DRAWING 1023w39.cdr

NSW DEPARTMENT
OF PUBLIC WORKS
AND SERVICES
MANLY HYDRAULICS LABORATORY

TYPICAL CROSS-SECTION TYPE 2
PHOTOGRAPH

MHL
Report 1023

Figure
3.10

DRAWING 1023m310.cdr

NSW DEPARTMENT
OF PUBLIC WORKS
AND SERVICES
MANLY HYDRAULICS LABORATORY

TYPICAL CROSS-SECTION TYPE 6
PHOTOGRAPH

MHL
Report 1023

Figure
3.11

DRAWING 1023wl311.cdr

4. Application of Study Data to Specific Sites

The following section summarises the estimation of the wind-elevated foreshore flood level estimates at each of the 33 sites for the 1% AEP, 5% AEP and extreme floods. Table 4.1 summarises the hindcasted wave heights used for runup calculations. Table 4.2 summarises the results which can be found in Appendix A. Wind-elevated flood levels at each site are shown graphically in Figure 4.1 and at each location in Figure 1.2. Table 2.1 Wind Data shows that the strongest winds occur from the west. As can be seen in Figure 4.1 the sites on the eastern shore that are generally located at the downwind end of the longest fetch are most susceptible to wave effects.

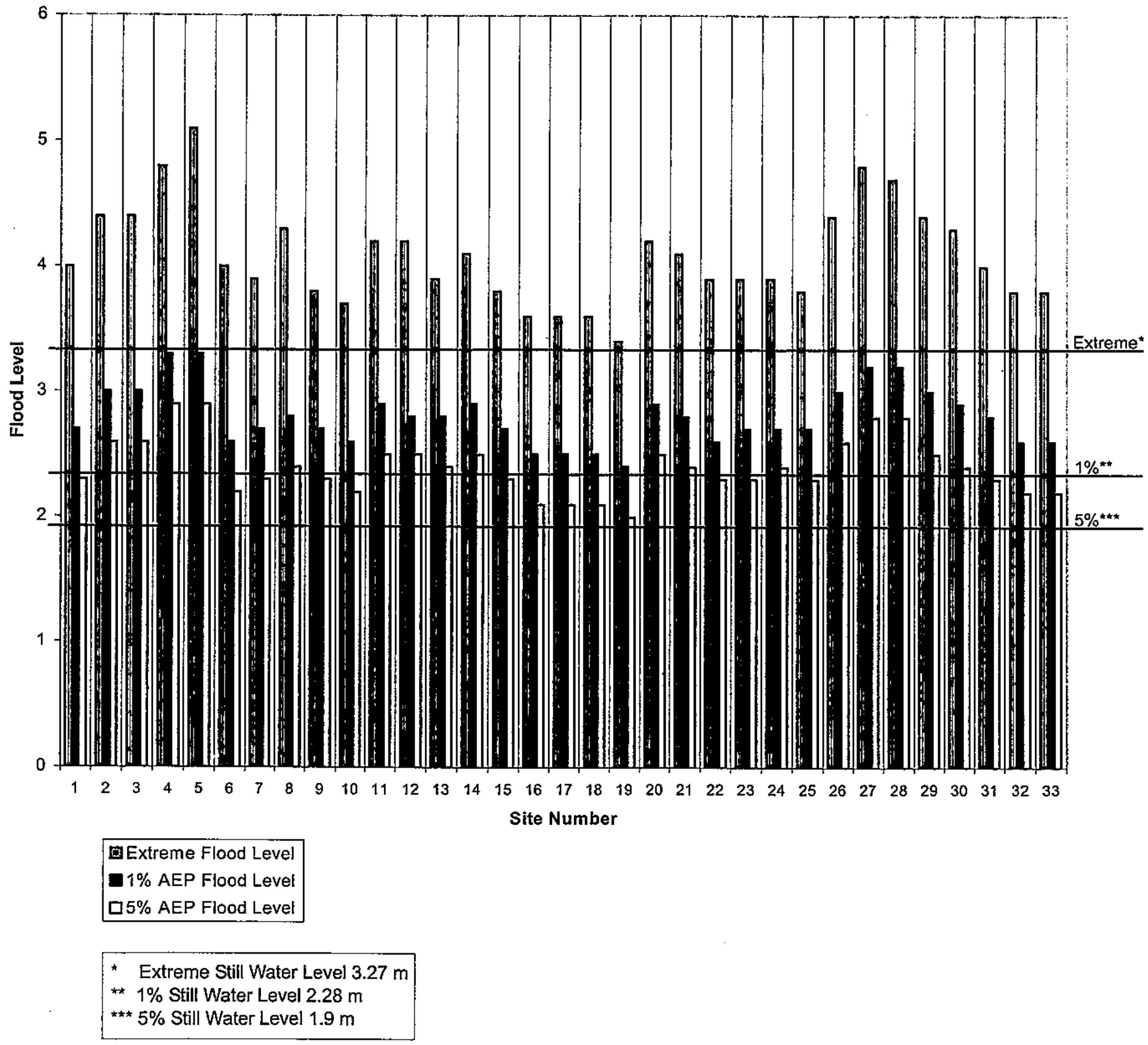
Table 4.1 Design Significant Wave Heights, H_s, Due to Locally-generated Wind in Wallis Lake

	1% AEP Wind on 3.27 m Depth	100% AEP Wind on 2.28 m Depth	1% AEP Wind on 1.26 m Depth	100% AEP Wind on 1.90 m Depth	5% AEP Wind on 1.26 m Depth
Site No.	H_s	H_s	H_s	H_s	H_s
1	0.84	0.46	0.82	0.46	0.75
2	1.45	0.84	1.35	0.83	1.23
3	1.45	0.79	1.35	0.78	1.19
4	1.85	1.11	1.54	1.07	1.43
5	1.26	0.64	1.02	0.63	0.91
6	0.79	0.27	0.65	0.26	0.55
7	0.69	0.43	0.59	0.42	0.52
8	0.65	0.37	0.57	0.36	0.50
9	0.64	0.44	0.57	0.43	0.54
10	0.72	0.45	0.68	0.45	0.64
11	1.42	0.77	1.26	0.76	1.12
12	1.23	0.66	1.13	0.65	0.99
13	0.95	0.67	0.84	0.65	0.79
14	0.83	0.58	0.77	0.57	0.72
15	0.77	0.54	0.71	0.53	0.66
16	0.64	0.30	0.63	0.30	0.54
17	0.49	0.32	0.48	0.32	0.44
18	0.44	0.26	0.44	0.26	0.41
19	0.21	0.12	0.21	0.12	0.20
20	1.11	0.60	1.04	0.60	0.99
21	1.20	0.69	1.16	0.68	1.05
22	0.84	0.47	0.82	0.47	0.74
23	0.88	0.48	0.86	0.48	0.76
24	0.69	0.49	0.67	0.48	0.62
25	0.61	0.43	0.60	0.43	0.55
26	1.47	0.79	1.38	0.79	1.21
27	1.79	1.07	1.43	1.03	1.33
28	1.72	1.00	1.38	0.96	1.27
29	1.44	0.84	1.18	0.81	1.09
30	0.99	0.61	0.88	0.59	0.83
31	0.77	0.54	0.71	0.53	0.66
32	0.80	0.42	0.77	0.42	0.67
33	0.71	0.37	0.70	0.37	0.60

Note: 3.27 m refers to the extreme still water level, 2.28 m refers to 1% AEP still water level, 1.90 m refers to the 5% AEP still water level and 1.26 m refers to the 100% AEP still water level.

Table 4.2 Design Flood Levels at the Selected Sites

Site	1% AEP Flood Level (m AHD)	5% AEP Flood Level (m AHD)	Extreme Flood Level (m AHD)
1	2.7	2.3	4.0
2	3.0	2.6	4.4
3	3.0	2.6	4.4
4	3.3	2.9	4.8
5	3.3	2.9	5.1
6	2.6	2.2	4.0
7	2.7	2.3	3.9
8	2.8	2.4	4.3
9	2.7	2.3	3.8
10	2.6	2.2	3.7
11	2.9	2.5	4.2
12	2.8	2.5	4.2
13	2.8	2.4	3.9
14	2.9	2.5	4.1
15	2.7	2.3	3.8
16	2.5	2.1	3.6
17	2.5	2.1	3.6
18	2.5	2.1	3.6
19	2.4	2.0	3.4
20	2.9	2.5	4.2
21	2.8	2.4	4.1
22	2.6	2.3	3.9
23	2.7	2.3	3.9
24	2.7	2.4	3.9
25	2.7	2.3	3.8
26	3.0	2.6	4.4
27	3.2	2.8	4.8
28	3.2	2.8	4.7
29	3.0	2.5	4.4
30	2.9	2.4	4.3
31	2.8	2.3	4.0
32	2.6	2.2	3.8
33	2.6	2.2	3.8



NSW DEPARTMENT OF PUBLIC WORKS AND SERVICES

MANLY HYDRAULICS LABORATORY

WIND-ELEVATED FLOOD LEVELS FOR SELECTED SITES AT WALLIS LAKE

MHL Report 1023

Figure 4.1

DRAWING 1023WL41.CDR

5. Greenhouse Sea Level Rise

The Greenhouse effect is a predicted global climatic change (global warming) associated with the build-up of certain gases in the atmosphere. Greenhouse gases are essentially transparent to incoming short-wave solar radiation, but they absorb the longer wavelength infra-red radiation (heat) emitted by the earth. Thus heat is trapped in the atmosphere and the global temperature is increased.

The scenario of a rising sea level associated with the postulated warming of the earth's atmosphere (the Greenhouse effect) may result in changes to both coastal processes affecting foreshore areas and a change to predominant weather patterns. These changes will affect foreshore alignment and stability, siltation and shoal formation and directly impact on foreshore inundation levels. These potential changes need to be accommodated in planning foreshore development, facilities and services.

Predictions of changes to weather patterns and the impact of these on coastal, estuary and catchment processes are preliminary and not sufficiently reliable for planning purposes. Rather, it is accepted that a flexible and robust approach to decision-making should be employed within which likely variations in the ambient conditions can be accommodated. Predictions of a sea level rise are more realistic and should be incorporated in design and planning for foreshore areas.

The most recent Intergovernmental Panel on Climate Change (IPCC 1995) states: 'sea level rise as a result of thermal expansion of the oceans and melting of glaciers and ice-sheets by 2100 is expected to be between 15 cm and 95 cm, with a 'best guess' of 50 cm. This range is due largely to uncertainty in the amounts of greenhouse gases which nations will emit.'

The most up-to-date estimates of sea level rise are those provided by the International Panel on Climate Change (IPCC 1995). These more recent estimates of the likely impact of climate change on sea level over the next 100 years have been made on the basis of improved models that generate results with increased confidence. These results predict a slightly reduced rate of sea level rise in comparison to earlier estimates but the general scenario is unchanged. Low, mid and high range estimates are tabulated for the 50-year and 100-year planning period in Table 5.1.

Table 5.1 Projected Global Mean Sea Level Rise

Planning Period (Years)	Low (m)	Best Estimate (m)	High (m)
50	0.05	0.20	0.40
100	0.15	0.50	0.95

Source : IPCC (1995)

It is assumed that the ocean level rise due to Greenhouse will cause the same increase in lake levels. Given the entrance conditions for Wallis Lake this is thought to be a reasonable assumption.

6. Discussion and Recommendations

The following sections detail the implications of the assumptions made during this study and recommendations with respect to the use of information contained within this report.

6.1 Assumptions and Implications

Throughout this report there have been a number of assumptions made with respect to the values and methods used to calculate the wind-elevated flood levels. The following assumptions were made:

- The depth over the fetch length is assumed constant:

  The values of Hs derived using this assumption are valid at the downwind end of the idealised basin, that is, in the depth of water assumed. For the subsequent runup calculations the actual water depth near the shoreline is considerably shallower than the assumed average depth across the fetch and hence wave shoaling and breaking effects are likely to lead to reduced wave heights contributing to runup. These effects have not been incorporated in this investigation.

- The still water levels from the previous flood studies are correct. It may also be possible that the previous flood studies inherently included wind-elevated flood levels through calibration of peak events:

  There is no way to determine this within the scope of this investigation.

- The occurrence of winds and floods are not totally independent:

  It is known that the coincidence of wind and flood events is not totally independent, however, the amount of dependence is not known quantitatively. In accordance with the methods included in the Lake Macquarie study (MHL1998a), the same guidelines have been used. If the wind and flood events were totally independent then the values obtained would be conservative. Until more detailed work has been conducted the guidelines used in the Lake Macquarie study (MHL1998a) will be considered best practice.

- The cross-sections obtained are representative of the surrounding area:

  The local bathymetry can have a significant effect on the level of runup.

- In areas where the foreshores are low-lying for some distance inland and no structures are currently in the vicinity, a slope of 10 degrees was assumed. Any development should be sited above this level:

  Since these cases are required for planning purposes, it is considered reasonable to assume building will not be taking place within these areas. If building is to take place in these areas then further investigation would be required.

- The effects of foreshore vegetation on wave dampening have not been considered:

  Wave dampening caused by foreshore vegetation could result in an over-estimation of wave heights and thus the level of runup at the foreshore. Further investigation needs to be made to quantify these effects, however this is beyond the scope of this study.

6.2 Recommendations and Guidelines

The following points and guidelines are made with regard to the wind-elevated design flood levels.

1. The design cross-sections are based on information gathered during the site inspection, together with detailed field survey information. The cross-sections are subject to human activities which may result in subsequent changes to the profiles. The sub-aqueous profiles are also subject to possible changes from various sediment transport processes. Future changes in cross-sections will result in changed design flood levels.

2. The 1% AEP and 5% AEP foreshore flood levels due to wave inundation were calculated based on the assumption that the cross-section was representative of the site. However, as predicted, inundation levels can vary with changes in cross-section at any specific site. It is essential to recalculate foreshore flood levels at cross-sections which are significantly different to the typical profiles that were used for this study.

3. The recommended design wind-elevated water levels for the 33 sites are given in Table 5.1. For other sites around Wallis Lake these results may be viewed as a guide to the likely wind-elevated foreshore flood levels, however, it is strongly recommended that specific assessment be undertaken for each site.

4. If significant development is to be undertaken at any of the sites, it is recommended that the inundation levels be reviewed and possibly re-evaluated for the site specific conditions.

5. At foreshore sites where building structures are located close to the foreshore, design flood levels have been calculated assuming wave action directly impacts on the side of the structure. These foreshore sites result in the highest calculated wave runup. However, the particular arrangement of the building needs to be considered. If there are no openings in the side of the building then water may not enter and flood the building. If the building is structurally sound, designed to withstand wave action and constructed of suitable material then the development may be able to satisfactorily withstand the conditions and may be an acceptable development. Council may need a professional engineering report on certain developments to help in assessing each individual case.

6. The estimated design flood levels behind any future foreshore seawalls can be calculated. As the distance behind the seawall to the structure decreases the flood level theoretically increases and as the distance from the seawall to the structure increases the flood level should theoretically decrease. However, the estimation of these flood levels very much depends on the amount of dispersion and both the porosity of the ground and the available flood storage volume behind the wall. Very sandy areas may reduce the flood impacts and in some circumstances the overwash volume on impervious ground may pond to greater flood levels. Also the impacts of any wave action over the top of a seawall may be reduced by local vegetation or the first line of development from the seawall. Again, Council will need to use judgement and knowledge of local conditions to apply the estimated design flood level to development controls behind seawalls.

7. The design cross-sections are based on information gathered during the site inspection, together with detailed field survey information. This survey is presented in Appendix A. The cross-sections are subject to human activities which may result in subsequent changes to the profiles. The sub-aqueous profiles are also subject to possible changes from various sediment transport processes. Future changes in cross-sections will result in changed design flood levels.

7. References

AWACS 1991, *Pittwater Water Level and Wave Study - Application of Study Data*, Australian Water and Coastal Studies Pty Ltd, September 1991.

CERC 1984, *Shore Protection Manual*, US Army Corps of Engineers, Coastal Engineering Research Centre, Waterways Experiment Station, Vicksburg, USA.

de Waal, J.P and Van der Meer, J.W. 1992, *Wave Runup and Overtopping on Coastal Structures*, ASCE Proc, 23rd ICCE, Venice Italy 1992.

DLWC 1992, *Forster/Tuncurry Floodplain Management Study - Hydraulic Analysis of Flood Mitigation and Development Options*, December 1992 (draft).

Hanslow D.J and Nielsen P. 1995, *Field Measurement of Runup on Natural Beaches*, 12th Australian Coastal and Ocean Engineering Conference, Melbourne.

IEAust 1993, *Guidelines for responding to the effects of Climatic Change in Coastal Engineering Design*, Institution of Engineers Australia.

IEAust 1997, *Australian Rainfall and Runoff, A Guide to Flood Estimation*, D.H. Pilgrim (Editor-in-Chief), The Institution of Engineers, Australia.

IEAust 1998, Coastal Engineering Guidelines, Report Series 14 (draft), The Institution of Engineers, Australia.

IPCC 1995, Climate Change 1995 – Impacts, Adaptations and Mitigation of Climate Change: Scientific-Technical analyses,

Manly Hydraulics Laboratory 1984, *Wave Statistics for Port Kembla*, MHL Report No. 416.

Manly Hydraulics Laboratory 1997a, *NSW Central Coast Wave Data Analysis*, MHL Report No. 754.

Manly Hydraulics Laboratory 1997b, *Coincidence of Rainfall, Ocean Level and Wind for Lake Macquarie*, MHL Report No. 847 (final draft).

Manly Hydraulics Laboratory 1998a, *Lake Macquarie Flood Study – Part 2 Foreshore Flooding*, MHL Report No. 715

Manly Hydraulics Laboratory 1998b, *Port Stephens Flood Study - Stage 3 Foreshore Flooding*, MHL Report No. 880 (final draft).

Public Works Department 1987, *Jimmys Beach Erosion Study,* PWD Report No. 85042 February 1987.

Public Works Department 1989, *Forster/Tuncurry Flood Study,* DPWS Report No. 88036, September 1989.

Van der Meer, J.W. and Janssen, J.P.F.M. 1994, *Wave Runup and Wave Overtopping at Dykes and Revetments.*

8. Glossary

Average Recurrence Interval (ARI)	Refers to the probability or risk of a flood of a given size occurring. A 1-in-100-year ARI has a probability of occurring once in 100 years. The probability AEP is also used to describe the probability of floods. ARI is used generally where data and procedures are based on partial series analysis, and AEP where annual series are used.
Annual Exceedance 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 of being exceeded; it would be fairly rare but it would be relatively large.
Australian Height Datum (AHD)	A common national surface level corresponding approximately to mean sea level.
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.
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.
discharge hydrograph	A graph which shows how the discharge of water changes with time at any particular location.
extreme flood event	An approximation of the flood expected to be the maximum that can occur.
floodplain	The portion of a valley, adjacent to a channel or water body which is covered with water when the water body overflows during floods.
flood storages	Those parts of the floodplain that are important for the temporary storage of floodwaters during the passage of a flood.

floodways	Those areas where a significant volume of water flows during floods. They are often aligned with obvious naturally defined channels. Floodways are areas which, even if only partially blocked, would cause a significant redistribution of flood flow, which may in turn adversely affect other areas. They are often, but not necessarily, the areas of deeper flow or the areas where higher velocities occur.
hydraulic	The term given to the study of water flow in a water body, in particular, the evaluation of flow parameters such as stage and velocity.
hydrology	The term given to the study of the rainfall and runoff process as it relates to the derivation of hydrographs for given floods.
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 numerical representation of the physical processes involved in runoff and streamflow. 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 and stream flow.
peak discharge	The maximum discharge occurring during a flood event.
probability	A statistical measure of the expected frequency or occurrence of flooding.
refraction	The tendency of wave crests to become parallel to bottom contours as waves move into shallower waters. This effect is caused by the shoaling process which slows down waves in shallower waters.
runoff	The amount of rainfall which actually ends up as streamflow, also known as rainfall excess.

shoaling	The influence of the seabed on wave behaviour. Generally manifested as a reduction in wave speed, a shortening in wave length and an increase in wave height.
significant wave height	The average height of the highest one third of waves recorded in a given monitoring period. Also referred to as $H_{1/3}$ or H_s.
stage hydrograph	A graph which shows how the water level changes with time. It must be referenced to a particular location and datum.
still water level	The ocean levels that are likely to occur under the combined influence of an astronomical tide and storm surge and other oceanic conditions excluding wave setup.
storm surge	The increase in coastal water level caused by the effects of storms. Storm surge consists of two components: the increase in water level caused by the reduction in barometric pressure (barometric setup) and the increase in water level caused by the action of wind blowing over the sea surface (wind setup).
swash	The rush of water up onto the beach face following the breaking of a wave.
tailwater levels	Water levels at the downstream end of the area being hydraulically modelled. For this study it is the effective ocean level at the entrance.
tides	The regular rise and fall of sea level in response to the gravitational attraction of the sun, moon and planets. Tides along the New South Wales coastline are semi-diurnal in nature with a period of about 12.5 hours.
wave setup	The increase in water level within the surf zone above mean still water level caused by wave action.
wind setup	The increase in mean sea level caused by the 'piling up' of water on the coastline by the wind.