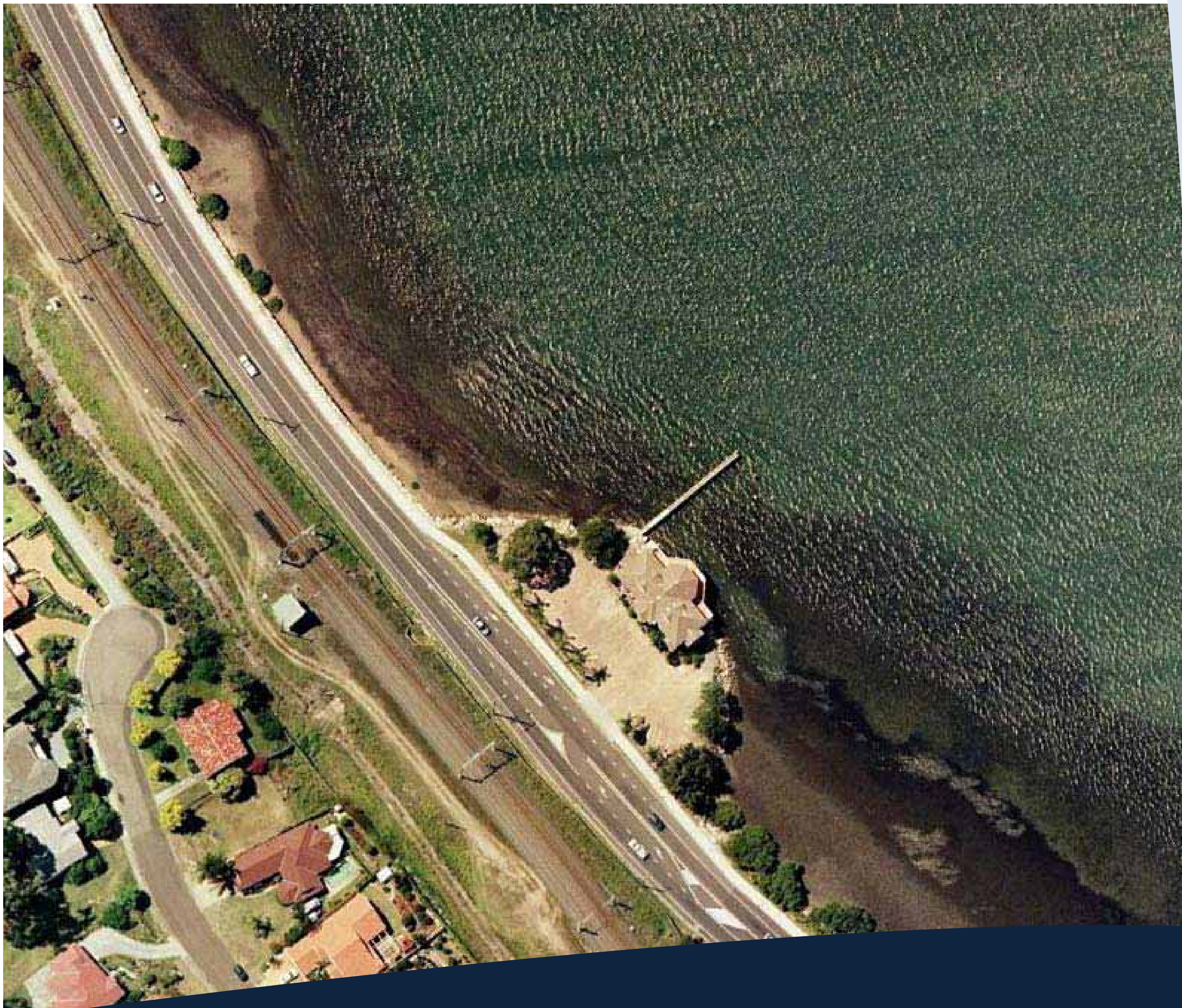

Proposed Koolewong Marina Development

Coastal Processes Investigations

EL1011059/R2718

Prepared for ADW Johnson

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EXECUTIVE SUMMARY

Cardno Coastal and Ocean have been engaged by ADW Johnson to undertake investigations of estuarine hydraulic processes as part of an environmental assessment of the proposed upgrade of the Koolewong Marina, Murphy's Bay, Brisbane Water. The proposed marina site is located on the Koolewong foreshore in Brisbane Water approximately 500m north of the Woy Woy rail bridge (**Figure 1.1**). The proposed re-development includes an extension to the existing jetty footprint, through the addition of new marina arms which would accommodate 50 new berths. It is understood that there is to be no dredging and there are no changes to the shoreline.

As part of this assessment, numerous factors must be investigated, as specified by The Director General's, Requirements (DGR's) for the assessment of this development proposal. The estuarine investigations undertaken by Cardno aimed to address the key issues of the DGR's outlined in Section 6: *Hazard Management and Mitigation* and involves the assessment of a range of coastal and estuarine processes, as outlined in the Coastline Management Manual. The necessary issues which have been addressed in this report are summarised in **Table E.1** below.

Table E.1: Summary of Director General's, Requirements (DGR's) Related to Coastal and Estuarine Processes

Issue	Investigation / Assessment Approach	Outcome
Impacts Associated with Wind and Wave Action	Wave climate analyses conducted	The marina site is *not* classified as having a 'good' wave climate as described by AS3962, but the berths will be satisfactory with appropriately designed pontoons.
Impacts Associated with Coastal Erosion	Not necessary as protection from erosion is provided in the form of a rock revetment.	Development is not expected to impact on shoreline processes. No further investigation required/
Climate Change and Sea Level Rise	Hydrodynamic Modeling conducted incorporating NSW State Government sea level rise projections.	Effect of climate change on flood levels assessed and flood levels provided for a range of scenarios in accordance with NSW policy and guidelines. No further investigation required
Assess Flushing	Hydrodynamic modeling conducted on turnover and flushing time at the site.	Flushing time found to be satisfactory. No further investigation required. The proposed development does not result in an enclosed waterbody.
Provide Hydrodynamic Survey	Cardno has not assessed site survey data.	To be provided by developer.
Provide an Assessment of Flood Risk	Hydrodynamic Modeling conducted incorporating a full range if ARI's for both ocean storm and catchment flooding.	Severe ocean storms cause the highest flood levels rather than catchment floods of the same ARI. Flood levels provided for a range of scenarios in accordance with NSW policy and guidelines. No further investigation required.

Following the range of estuarine and coastal process investigations undertaken for this proposal, it is considered that the expansion of the marina will not have a significant effect on the existing coastal processes. The investigations have not identified any likely changes to the hydrodynamic, morphological and water quality conditions resulting from the proposal that would pose a risk to the built environment and/or members of the public using the marina.

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GLOSSARY AND ABBREVIATIONS

Australian Height Datum (AHD)	A common national plane of level corresponding approximately to mean sea level.
ALS	Aerial Laser Survey
ARI	Average Recurrence Interval; relates to the probability of occurrence of a design event.
BoM	Bureau of Meteorology.
Calibration	The process by which the results of a computer model are brought to agreement with observed data.
CD	Chart Datum, common datum for navigation charts – 0.92m below AHD in the Sydney coastal region. Typically Lowest Astronomical Tide.
Coastal Inundation	Flooding of coastal land due to inundation by ocean waters.
DCP	Development Control Plan
DECC	NSW Department of Environment and Climate Change (now NSW Office of Environment and Heritage).
Discharge	The rate of flow of water measured in terms of volume per unit time. It is to be distinguished from speed or velocity of flow, which is a measure of how fast the water is moving rather than how much is flowing.
Diurnal	A daily variation, e.g. one high water per day.
DoP	NSW Department of Planning (now DP&I).
DP&I	NSW Department of Planning and Infrastructure.
DTM	Digital Terrain Model
Ebb Tide	The outgoing tidal movement of water within an estuary.
Eddies	Large, approximately circular, swirling movements of water, often metres of tens of metres across. Eddies are caused by shear between the flow and a boundary or by flow separation from a boundary.
Estuarine Processes	The processes that affect the physical, chemical and biological behaviour of an estuary, e.g. Predation, water movement, sediment movement, water quality etc.
Estuary	An enclosed or semi-enclosed body of water having an open or intermittently open connection to coastal waters and in which water levels vary in a periodic fashion in response to ocean tides.
Flood Tide	The incoming tidal movement of water within an estuary.
FPL	Flood Planning Level
Foreshore	The area of shore between low and high tide marks and land adjacent thereto.
GCC	Gosford City Council.
HAT	Highest Astronomical Tide.
HHW	High High Water.
Intertidal	Pertaining to those areas of land covered by water at high tide, but exposed at low tide, e.g. Intertidal habitat.
LAT	Lowest Astronomical Tide.

Longshore Transport	The movement of sand along the coastline caused by waves and a wave-caused current running parallel to the beach.
Neap Tides	Tides with the smallest range in a monthly cycle. Neap tides occur when the sun and moon lie at right angles relative to the earth (the gravitational effects of the moon and sun act in opposition on the ocean).
Numerical Model	A mathematical representation of a physical, chemical or biological process of interest. Computers are often required to solve the underlying equations.
PMF	Probable Maximum Flood.
Semi-diurnal	A twice-daily variation, e.g. two high waters per day.
Shoals	Shallow areas in an estuary created by the deposition and build-up of sediments.
Shoreline Recession	The long-term (decadal plus) net landward movement of the shoreline/mean water line. Occasionally referred to as long-term erosion.
SLR	Sea Level Rise.
Spring Tides	Tides with the greatest range in a monthly cycle, which occurs when the sun, moon and earth are in alignment (the gravitational effects of the moon and sun act in concert on the ocean).
Storm surge	The increase in coastal water levels caused by the barometric and wind set-up effects of storms. Barometric set-up refers to the increase in coastal water levels associated with the lower atmospheric pressures characteristic of storms. Wind set-up refers to the increase in coastal water levels caused by an onshore wind driving water shoreward's and piling it up against the coast.
Tidal Exchange	The proportion of the tidal prism that is flushed away and replaced with 'fresh' coastal water each tide cycle.
Tidal Excursion	The distance travelled by a water particle from low water slack to high water slack and vice versa.
Tidal Limit	The most upstream location where a tidal rise and fall of water levels is discernible. The location of the tidal limit changes with freshwater inflows and tidal range.
Tidal Planes	A series of water levels that define standard tides, e.g. 'Mean High Water Spring' (MHWS) refers to the average high water level of Spring Tides.
Tidal Prism	The total volume of water moving past a fixed point in an estuary during each flood tide or ebb tide.
Tidal Range	The difference between successive high water and low water levels. Tidal range is maximum during Spring Tides and minimum during Neap Tides.
Tides	The regular rise and fall in sea level in response to the gravitational attraction of the Sun, Moon and Earth.
WL	Water Level
Wave Height	The height of the tsunami wave crest above the still water level. As opposed to the coastal engineering definition of wave height, being the height between a wave crest and trough. When referring to a particular tsunami event, the wave height refers to the largest wave height in the tsunami wave train.
Wave Length	The distance between two wave crests.
Wave Period	The time it takes for two successive wave crests to pass a given point.

Wave Run-up	The vertical distance between the maximum height that a wave runs up the beach (or a coastal structure) and the still water level, comprising tide and storm surge.
Wave Set-up	Wave set-up in included implicitly in wave run-up calculations.
WMO	World Meteorological Organisation

1 INTRODUCTION

Cardno have been engaged by ADW Johnson to undertake the estuarine hydraulics phases of an environmental assessment of the proposed upgrade of the Koolewong Marina, Murphy's Bay, within Brisbane Water. The proposed marina site is located on the Koolewong foreshore; 500m north of the Woy Woy rail bridge (see **Figure 1.1**). The proposed re-development includes:-

- Construction of a 50 berth marina that would extend approximately 100m into Brisbane Water (**Figure 1.2**);
- Upgrade of the existing jetty (**Figure 1.3**); and
- Upgrade and reconfiguration of the existing car park from 33 spaces to 44 spaces.

Figure 1.2 shows the proposed marina configuration. It is understood that there is to be no dredging and there are no changes to the shoreline. The existing jetty is shown in **Figure 1.3**. The marina extension will be formed of pontoons held in place by piers. These pontoons will also provide some protection from wind waves in Murphy's Bay.

As part of the requirements of the assessment, numerous factors must be investigated, as specified by the Director General, Department of Planning and Infrastructure (DP&I), NSW. The scope of works defined under this engagement include:-

- Wave climate investigations, including assessment of the marina against *AS3962 - Guidelines for Design of Marinas*.
- Hydrodynamic Investigations, including morphological, bed and shoreline stability and flushing assessments.
- Estuarine Flooding Assessments, including providing an assessment of the estuarine flood hazard in the area, as well as assessing the potential impacts that climate change may have on the flood regime.

This report describes the study approach, data, model systems and results of these investigations.

Normal conventions are followed, namely that:-

- Wind and wave directions are coming from
- Currents are flowing towards.

Unless specified otherwise all depths are specified to Chart Datum (CD), which is lowest astronomical tide (LAT) at Koolewong.

2 PHYSICAL PROCESSES

The physical processes that are important to this development proposal at Koolewong are described in **Appendix A**.

3 DATA

A range of data items were required to describe the wave climate, hydrodynamic and water quality setting of the Koolewong marina area within Brisbane Water, and to set up the numerical models applied to local wind-wave and hydrodynamic modelling.

3.1 Bathymetry

Bathymetric data was obtained from a number of sources:

- Chart AUS 204;
- Survey on north-east side of St Huberts Island, August, 2004, Hydrographic Surveys;
- 1992 Public Works hydrosurvey extending from Wagstaff Point to Gosford. This is the most recent overall hydro-graphic survey of the estuary;
- Chart 83042 – Broken Bay. 1989 seabed survey undertaken by NSW Public Works.

This data was digitised to provide a digital terrain model (DTM) extending from the 200m depth contour offshore to the Gosford shoreline and throughout the estuary.

Some site level data is provided in **Appendix B**. This data shows that the deck level of the existing jetty is 1.25m AHD. The survey includes the whole of the proposed berth area and the car park areas.

3.2 Wind Data

Wind data has been recorded at Sydney Airport since 1939 (Moneypenny *et al*., 1997). Winds in the Gosford region will be different from those at Sydney Airport on an event-by event basis. However, the two locations will be statistically similar on a longer term basis.

Over the years the location of the weather station, along with airport development, has resulted in some changes to the wind data since records began. From 1939 to 16 August, 1994, a Dines anemometer was used to record 10-minute averages of wind speed and direction. Since the early 1960’s, at least, this anemometer was located on a 10m mast near the intersection of the east-west and north-south runways. Recommended World Meteorological Organisation (WMO) clearances from buildings and other obstructions were maintained. During its period of service, the Dines anemometer was well maintained.

Since 16 August, 1994, wind data at the airport has been recorded using a Synchrotec anemometer installed on a 10m mast near the threshold of the main north-south runway, and is more exposed than the previous Dines anemometer site.

Analyses of these wind records (Monypenny and Middleton, 1997) showed that there had been a gradual error (reduction) in wind speed recorded by the Dines anemometer. This reduction amounted to 2.6m/s by August (1994). Monypenny and Middleton (1997) advise that a simplified linear adjustment be made to Sydney airport wind speeds up to 16 August, 1994 and this adjustment was undertaken for this study. Data to 31 December (2000) was obtained from the Bureau of Meteorology.

The wind data was analysed in terms of the standard directional octants to provide peak storm wind speeds at selected average recurrence intervals (ARIs), as shown in **Table 3.1**. These analyses were undertaken by ranking recorded wind speeds in each octant and then undertaking an extremal analysis using the maximum likelihood method, fitting to a Weibull distribution.

Table 3.1: Wind Speeds (m/s) by Octant

Octant	Gust Speeds		10 min Average Speeds		3 hour Average Speeds	
	100 yr ARI	20 yr ARI	100 yr ARI	20 yr ARI	20 yr ARI	100 yr ARI
N	28.4	26.1	19.3	17.8	18.5	17.0
NE	23.8	22.9	18.3	17.6	17.6	16.9
E	25.7	22.8	19.8	17.5	19.0	16.8
SE	28.2	25.6	21.7	19.7	20.8	18.9
S	42.1	38.3	31.7	28.8	30.4	27.6
SW	35.1	31.9	25.6	23.3	24.6	22.4
W	38.3	35.0	26.6	24.3	25.5	23.3
NW	33.9	31.3	23.5	21.7	22.5	20.8

Table 3.1 also includes wind gust speed - generally 2-second gusts. These results are general and do not include any shoreline terrain correction factors. Additional information is presented in **Table 3.2**.

Table 3.2: Wind Speeds (m/s) at Selected ARI - 10-Minute Average Speeds

Octant	Average Recurrence Interval (years)					
	5	10	20	50	100	1000
N	16.4	17.1	17.8	18.7	19.3	23.5
NE	17.0	17.3	17.6	18.0	18.3	20.3
E	15.4	16.5	17.5	18.8	19.8	26.3
SE	17.9	18.8	19.7	20.8	21.7	27.4
S	26.2	27.5	28.8	30.5	31.7	39.9
SW	21.2	22.3	23.3	24.6	25.6	32.1
W	22.2	23.3	24.3	25.6	26.6	33.1
NW	20.1	20.9	21.7	20.9	23.5	24.6

3.3 Water Level Data

This data came in a range of forms and from a range of sources. Principal amongst these was the extensive gauging study undertaken in Brisbane Water by Manly Hydraulics Laboratory, (MHL, 2004). Water level data was collected from various sites within the estuary for a period of approximately twelve weeks, from 5 February to 29 April 2004.

This water level data has been used for two aspects. The first was to provide time series data for numerical hydrodynamic model calibration. That data was provided by MHL in digital form. The second was to provide descriptions of the spatial variations of tidal ranges and tidal planes.

Predicted tidal levels and recorded water levels at hourly intervals for Fort Denison, Sydney, was provided by the National Tidal Centre for the May 1974 storm event. Records of indicative peak water levels that were observed in Brisbane Water for that event, other than in isolated locations such as Fagans Bay, remain as the highest recorded water levels in Brisbane Water – Public Works, 1976. These levels have been converted from Standard Datum to AHD.

A second, ocean-water level event that lead to unusually high water levels in Brisbane Water occurred in April 1990 (Public Works, 1991). It arose from a very large low pressure system over the Tasman Sea, but neither significant rainfall nor wave action occurred at that time in the Broken Bay region. A level of 1.4m AHD was recorded in Sydney Harbour, being the third highest recorded water level since 1914. The effects propagated well into Brisbane Water with tidal anomalies up to 0.4m occurring. Peak observed water levels were:-

- Ettalong 1.20m AHD at 2200 27/02/1990;
- Koolewong 1.09m AHD at 0000 28/04/1990;
- Wharf St, Gosford 1.02m AHD at 0000 28/04/1990.

These higher water levels would have caused a temporary increase of influx of ocean water and subsequent outflow. Generally, the highest water levels in Brisbane Water itself, leaving aside local flooding issues in creeks and stormwater flow areas, are caused by ocean storm systems, with little accompanying rainfall.

Water level data was collected at Koolewong by MHL on behalf of Gosford City Council between 1993 and 2003 at 15 minute intervals. It has been analysed and plotted in terms of probability of exceedance. Water level data for a number of sites around Brisbane Water was recorded between February and April 2004, including Koolewong and Ettalong (MHL, 2004). This data was analysed also on a probability of exceedance basis.

Water level records from Koolewong and Ettalong were provided by MHL in digital form - up to ten years at Koolewong, but only three months at Ettalong (February to April, 2004).

Tidal constant data for the offshore region of the Brisbane Water hydrodynamic model was taken from Australian National Tide Tables (2006).

3.4 Wave Data

Both local sea and swell waves are important in different regions of Brisbane Water. Local sea is the more widely spread of the two and occurs throughout the estuary, being most important in the wide expanse of the Gosford Broadwater, where the Koolewong Marina is located.

4 WAVE CLIMATE

No measured wave data was available for the Koolewong Marina site; hence numerical wave modelling was required to develop a wave climate for marina analysis. The wave climate was derived from long-term measured wind data at Sydney Airport, (**Section 3.2)**, and hence includes extreme wind events. The proposed marina layout is shown in **Appendix C**.

4.1 SWAN Wave Model

The wave model system applied to this investigation was the SWAN model developed at the Delft University of Technology (2000). The model can provide full spectral solutions to third generation and includes wind input, refraction, diffraction, shoaling, bed friction, white capping, wave breaking, the effect of currents and non-linear wave-wave interaction The model has the capability of resolving curvilinear grid as well as nested grids that allow for large areas to be modelled whilst providing fine resolution in areas where seabed depths have high spatial variability.

It can be applied as a steady-state model for local sea developed from spatially and temporally constant winds and provides a very reliable basis for generating local sea. The model has been well verified by its authors and is considered to be one of the most reliable systems available at present. It is incorporated into the Delft3D modelling system developed by Deltares. Wave breaking and white capping processes were 'turned-on' for this study.

4.2 Model Setup

Due to the complexity and extent of the Brisbane Water Estuary system, separate model domains were developed to allow local sea and swell to be investigated. A Delft3D model grid of the whole Brisbane Water Estuary (**Figure 4.1**) was used to propagate swell from deep water into the study area. Swell wave conditions typically affected coastal areas and locations near Ettalong, but did not propagate past the point of "The Rip", and so there was no effect of swell waves on the Koolewong site.

In addition to this, a high-resolution grid was utilised to investigate local sea inside Brisbane Water, specifically around Koolewong. **Figure 4.2** shows the extent of the local sea model domain.

Wave propagation was undertaken at a water level of 1.6m AHD, this being analogous to a high storm-tide water level within Brisbane Water, even though this level would be more likely to occur during a severe east coast low ocean storm with onshore winds rather than with northerly winds, for example. Wave generation is sensitive to water level in the more shallow areas of Brisbane Water because of the relatively large increase in water depth, and hence wave heights that high water levels cause.

Several locations in the vicinity of the Koolewong marina were selected for the wave model output. Shoreline locations were generally in a depth of 1m CD, typically. Because local sea periods are relatively short, typically 1 to 3 seconds (T_z), bed friction, does not affect wave propagation to these locations, significantly.

4.2.1 SWAN Model Calibration

Wave model calibration provides confidence that the model system applied to this investigation will reproduce wave conditions in Brisbane Water reliably. The model has been calibrated for local sea in Botany Bay using the same Sydney airport wind as that used for this study (see Lawson and Treloar, 2003). No site specific characteristics required changing and so the SWAN model can be used at this site also with confidence

4.3 Local Sea Wave Climate

The SWAN wave model was used to develop the wind wave 'climate' at the Koolewong foreshore area. A range of wind speeds from 2.5 to 25m/s were included, leading to 176 wave modelling cases at a water level of +1.6m AHD – a rare high water level, but not unknown and in the order of the 100-years storm tide level described in **Section 6.2**. The results of this wave modelling provided a basis for developing 59 years of time series of wind wave data at the Koolewong marina site from the observed wind data at Mascot. This model output provided a long-term time series of wave parameters at the foreshore in terms of H_s, T_z and direction, together with wind speed and direction.

These time series results were then examined to identify peak storm wave heights, which were then analysed using the Extreme Value Type 1 distribution and applying the method of moments to the top 50 wave height results in each directional sector. This provided extremal wave conditions for selected average recurrence intervals (ARI) in each directional sector.

The results are presented below in **Table 4.1** for two of the most exposed directions and for selected average recurrence intervals (ARI) from 1 to 200 years at the marina.

Table 4.1 SWAN Model Results

ARI (years)	NE				ENE			
	H_s (m)	H_{max}(m)	T_z (s)	Max Wave Crest (m)	H_s (m)	H_{max}(m)	T_z (s)	Max Wave Crest (m)
1	0.29	0.52	2.1	0.26	0.16	0.29	1.8	0.15
5	0.41	0.74	2.4	0.37	0.20	0.36	1.9	0.18
10	0.44	0.79	2.4	0.40	0.25	0.45	2.0	0.23
20	0.46	0.83	2.5	0.42	0.28	0.50	2.1	0.25
50	0.49	0.88	2.6	0.44	0.32	0.57	2.2	0.29
100	0.51	0.92	2.6	0.46	0.35	0.63	2.2	0.32
200	0.53	0.95	2.7	0.48	0.40	0.72	2.3	0.36

Due to the orientation of the marina and its location, the most severe wave height cases occurring inside the marina would be caused by north-easterly sector winds. This is due to the large fetch in this direction, which is up to five kilometres long. This long fetch length means that the marina is particularly exposed to waves coming from the north-east despite the presence of oyster farms located 500-700m offshore that offer some protection.

Comparison of the wave results to the *Australian Standards Guidelines for design of Marinas* (AS3962) allows for the qualitative assessment of the marina's wave climate. **Table 4.2** described the conditions that define a 'good' marina wave climate.

Table 4.2 AS3962 Good Wave Climate Conditions

Direction and Peak Period of Design Harbour Wave	Significant Wave Height, H_s (m)	
	Wave Event Exceeded Once in 50 years	**Wave Event Exceeded Once a Year**
Head Seas less than 2s	Conditions not likely to occur during this event	Less than 0.3m wave height
Head seas greater than 2s	Less than 0.6m wave height	Less than 0.3m wave height
Oblique seas greater than 2s	Less than 0.4m	Less than 0.3m wave height
Beam seas less than 2s	Conditions not likely to occur during this event	Less than 0.3m wave height
Beam seas greater than 2s	Less than 0.25m wave height	Less than 0.15m wave height

The largest 'design' waves, as presented in **Table 4.2**, would approach from the north-east and due to the orientation of the berths, some of the moored boats would be affected by what are known as "beam" seas. Additionally peak wave periods are all greater than 2 seconds and thus the conditions required to achieve satisfactory wave conditions at berth are:-

- Less than 0.25m (seas greater than 2sec) for the 50-years ARI condition;
- Less than 0.15m (seas greater than 2sec) for the 1-year ARI condition.

Although the strongest winds come from the south and the west, these fetches are quite short. There will be some unprotected berths on the southern side, but they would generally be subjected only to head seas and the annual wave heights will not exceed 0.3m (H_s) – see **Tables 4.1** and **4.2** – the fetch to the south is in the order of 500m or less. Hence that berth will meet the Marina Design Guidelines. The other berths will be protected by the proposed floating pontoon access-ways, which will act as wave attenuators. These pontoons will need to be designed to attenuate the incident wind waves by about 50% - from about 0.3m to 0.15m; beam seas on occasion with peak periods greater than 2 seconds.

4.4 Boat Waves

This investigation has not addressed the potential impacts of boat waves. Although no ferries are known to pass close by this location, from time to time large vessels may cause boat waves of height in the order of 0.4m with a wave period in the order of 3 to 4 seconds; with potential consequences for moored vessels and the structures.

However, this will be no different from the present position and there are a number of boats moored in this region of Brisbane Water.

4.5 Wave Crest Level

The development proposal includes widening the existing jetty and a system of access pontoons, see **Appendix C**. An important issue is the design wave crest level because it affects the design of jetties and deck levels, seeking to avoid wave loading on deck structures. The minimum deck level for the existing marina jetty is only 1.25m AHD (Clarke Dowdle & Associates, 2011). The under-side level is likely to be only about 1.05m AHD.

An appropriate planning period for marina facilities exposed to wave and wind action, as well as salt air, is considered to be 50 years – estimated life of pontoon structures. It is noted that the lease of the site is for 25 years.

The estimation of the design wave crest level and the practical implications of this for the design of the jetty are discussed below.

4.5.1 Estimation of Design Wave Crest Level

Based on Cardno (2009) see **Section 6.2** (**Table 6.3**), the 50-years ARI water level is 1.58m AHD. This level was 'observed' in the severe May 1974 ocean storm (approximately). Hence the existing jetty (at 1.25 mAHD) will be submerged from time-to-time.

It is important to note that the wind conditions associated with peak water levels in severe ocean storm events are likely to be from the east to south-west sector. On the other hand, the largest waves are caused by north-easterly to east-north-easterly winds. Hence, it is physically realistic to adopt a more frequent east-north-easterly wave condition (10-years ARI) to jointly occur with the 50-years ARI water level.

Combining the 50-years ARI design water level with the 10-years ARI wave crest height (0.23m) from the wave parameters (H_{max}) of **Table 4.1** and the 2050 projected sea level rise of 0.4m leads to a wave crest level of 2.21m AHD. If a freeboard of 0.2m is allowed then the underside of all jetty structural deck elements should be set at 2.41m AHD, where feasible. Hence, to avoid overtopping of the jetty deck under design conditions the underside of the deck of the jetty would need to be raised to 2.41m AHD to provide all-weather access and no wave loads.

4.5.2 Practical Design Implications for Jetty and Recommended Jetty Level

A design jetty level of 2.41 mAHD (described in **Section 4.5.1**) poses an issue when considering the connection to the landward end of the jetty (which is at 1.9 mAHD). In practical terms, raising the jetty level by 0.5 m to 1.55 mAHD (underside) would be more suitable to address the landward connection limitation, but requires the jetty to be designed for wave loads to cope with periodic overtoppng. It is therefore recommended that the jetty is to be:

- raised by no less than 0.5m from its existing level (to a minimum level of 1.55 mAHD for the underside and approximately 1.75 mAHD for the deck level);
- designed for horizontal and vertical wave loads given the deck will be overtopped; and

- closed by the site manager when there is potential for either waves or the estuary water level to over-top the deck.

5 HYDRODYNAMIC PROCESSES

The dominant water level forcing phenomenon in the Brisbane Water Estuary is the astronomical tide, forced by the ocean semi-diurnal tidal cycle. However, hydrodynamic processes within the estuary may also include swell wave and local sea wave action (see **Section 4**), as well as catchment inflows from local creeks, that may bring with them pollutants associated with storm water runoff. The highest recorded water levels in the estuary occurred during the severe ocean storm of May 1974 (Public Works 1976) with little or no coincident rainfall.

Numerical modelling has been applied to the investigation of local area estuarine flood levels. The model system applied to this investigation is described below. Results of the numerical modelling were extracted from all simulations at the Koolewong foreshore site.

5.1 Hydrodynamic Model Setup

Planning level investigations required application of a high level model capable of simulating a range of processes – wind field, wave and tidal forcing. These simulations were undertaken using the Delft3D modelling system linked with the SWAN wave model.

The Delft3D modelling system has been applied to current and wave investigations at many international locations, as well as within Australia by Cardno including the *Brisbane Water Estuary Process Study* (Cardno, 2008). Other sites include Port Botany (Sydney), Cairns Navy Base (Queensland), New Caledonia and Exmouth Gulf in Western Australia, for example.

The Delft3D modelling system includes wind, pressure, tide and wave forcing, three-dimensional currents, stratification, sediment transport and water quality descriptions and is capable of using irregular rectilinear or curvilinear coordinates.

Delft3D is comprised of several modules that provide the facility to undertake a range of studies. All studies generally begin with the Delft3D-FLOW module. From Delft3D-FLOW, details such as velocities, water levels, density, salinity, vertical eddy viscosity and vertical eddy diffusivity can be provided as inputs to the other modules. The wave and sediment transport modules work interactively with the FLOW module through a common communications file.

For the purposes of Hydrodynamic modelling, a single model domain of Brisbane Water was utilised. The model extended over the entirety of the Brisbane Water Estuary, as well as over low-lying foreshore areas and included topographic data from council's contour information and the ground survey detailed in **Section 3.1**. The model layout can be seen in **Figure 5.1**, which shows that the model extends offshore to a depth of approximately 70m AHD and features water level boundaries offshore and along the boundaries with the Hawkesbury River and Pittwater. A calibrated MIKE-11 model of the Hawkesbury River system, including Brisbane Water and Pittwater, has been used to determine concurrent water level time series estimates for Pittwater and the Hawkesbury River during selected offshore water level conditions.

The model has a curvilinear grid with variable resolution. Offshore areas have a grid resolution in the order of 100m x 100m, while areas inside Brisbane Water, where steep hydrodynamic gradients exist, have horizontal grid cells in the order of 10m x 10m.

5.2 Model Calibration

Model calibration increases confidence that the model system provides a realistic description of the estuarine processes described by it in complex forcing conditions – for example, current speeds (process) caused by a spring tide (forcing condition). Brisbane Water is a complex hydraulic system featuring:-

- Several branches, some of which are interconnected,
- Generally shallow (water depths < 10m),
- Significant mangrove and intertidal areas,
- Mobile sand shoals, and a
- Major hydraulic control at 'The Rip'.

The calibration of the Delft3D hydrodynamic model involved two main stages. The first stage involved calibrating the water levels by adjusting the bed friction factor. The second stage of the calibration was to ensure that the discharges through each section of the model agreed with available flow data. Discharge rate is influenced by two key factors: bed friction and the conveyance, which in turn is affected by bed level and cross-sectional area. The discharge calibration ensures that the correct storage is defined in each branch of the model, thereby ensuring reliable velocity descriptions. It is possible to have a good water level calibration yet a poor discharge calibration if the correct storage is not provided within the model.

Calibration of the model showed that the agreement between modelled and measured water levels is very good and this provides assurance that the model adequately represents the hydrodynamics of the Brisbane Water Estuary for the purposes of this study. Water Level and Discharge calibration plots can be found in Cardno's *Brisbane Water Foreshore Flood Study* (2009).

5.3 Tidal Hydraulics

The hydraulic settings of the estuary play an important role in the forcing of other physical, biochemical and ecological processes within the system. Tidal hydraulics has been established in terms of tidal levels and velocities as described in the following sections.

5.3.1 Tidal Planes

As mentioned in **Section 3**, as part of the MHL data compilation study (2004), water level data was collected at Koolewong for a period of approximately twelve weeks from the 5 February – 29 April 2004. Analysis of this data allowed MHL to determine the local tidal planes at Koolewong. These are shown in **Table 5.1** below. Additionally, the tidal ranges are shown in **Table 5.2**.

Table 5.1 Koolewong Tidal Planes (MHL 2004)

Tidal Planes	m AHD
Higher High Water (Spring Solstices) (HHW)	0.623
Mean High Water Springs (MHWS)	0.384
Mean High Water (MHW)	0.331
Mean High Water Neap (MHWN)	0.278
Mean Tide Level (MTL)	0.076
Mean Low Water Neap (MLWN)	-0.126
Mean Low Water (MLW)	-0.179
Mean Low Water Springs (MLWS)	-0.232
Indian Spring Low Water (ISLW)	-0.403

Table 5.2 Koolewong Tidal Ranges (MHL 2004)

Tidal Planes	m AHD
HHW to ISLW	1.026
Mean Spring	0.616
Mean	0.510
Mean Neap	0.404

As can be seen from **Tables 5.1** and **5.2**, there is significant attenuation of the tides between the ocean and Koolewong. Most of this attenuation occurs at "The Rip", where the narrowness of the channel restricts tidal flows and results in smaller tidal ranges at sites further upstream such as Koolewong.

5.3.2 Tidal Velocities

A 2-week spring-neap tide cycle was run in the calibrated Delft3D hydrodynamic model to investigate the tidal currents in the Koolewong area. Model results for a spring tide period were then assessed. Peak current speeds (depth averaged) at the marina site are in the order of 0.1m/s and likely to occur on the ebb tide. For design purposes, a current jointly occurring with waves, and with a speed of 0.1m/s is advised.

5.4 Tidal Flushing

The concept of tidal flushing refers to the rate of water exchange between waters within and outside the marina area due to tidal flows. Quantitative investigations into flushing can be used to describe the likely character of water quality responses within the area.

A simulation using the Delft3D model was undertaken using a conservative tracer dispersed initially uniformly (concentration 100) over the entire extended marina footprint. The initial tracer concentration outside the investigation area was defined as zero.

The simulation was undertaken for a spring-neap tide cycle (14-days). No catchment flows were supplied to the models; that is, the flushing times determined by these analyses were maxima because catchment flows reduce flushing times by causing a net transport through an area.

Note that the outcome in terms of flushing times depends upon the initial model set up and distribution of tracer. Flushing times have been defined in terms of e-folding times. The e-folding time refers to the time taken for a tracer to reach 1/e or 0.3679 of its initial concentration. At any location in an estuary subject to dynamic equilibrium forcing, the concentration of a particular tracer can be described by **Equation 1**.

$$C_i = C_0 e^{-kt_i} \qquad \text{- (1)}$$

Following-on from **Equation 1**, k can be calculated by **Equation 2**.

$$k = \frac{\ln\left(\frac{C_i}{C_0}\right)}{-t_i} \qquad \text{- (2)}$$

The e-folding time is then the inverse of k, shown in **Equation 3**.

$$efold = \frac{1}{k} \qquad \text{- (3)}$$

An e-folding time distribution has been calculated for the marina basin using **Equations 2** and **3**, and is presented in **Figure 5.2** – a depth averaged result.

Depth averaged output shows that the majority of the marina area displays e-folding times of less than **2** hours, with a maximum e-folding time of 130 minutes. This outcome is to be expected as there are no significant barriers to tidal flows through the marina area. E-folding times in this order demonstrate an area of high water turnover.

5.5 Shoreline Stability

The surrounding shoreline at the site is protected by a rock revetment. There are also mangroves to the north and south, see **Figures 5.3** to **5.5**. The mangroves are indicative of the mild local sea wave climate at the marina site. This shoreline is in no danger of erosion and the proposed works will have no effect on it.

Figures 5.4 and **5.5** also show storm water drains that discharge to Brisbane Water on the northern and southern sides of the site. There is no extensive fan of deposited sediments at either stormwater outlet, though there is some larger debris. The proposed works would not be affected by these drains and the works would not affect discharge from them.

6 ESTUARINE FLOODING

The Brisbane Water Estuary is periodically subject to river/estuarine flooding from both large catchment run-off events, and ocean storm events. Information for this assessment was sought form available sources, most notably the Brisbane Water Foreshore Flood Study (Cardno, 2009). This study was undertaken for Gosford City Council and the Department of Environment and Climate Change (now the Office of Environment and Heritage). It addressed catchment flooding and ocean storm caused inundation. The joint occurrence of catchment floods and elevated water levels caused by ocean storms was considered also. Whilst historical records show that ocean storm events and extreme tidal levels are the predominant cause of foreshore flooding, the Brisbane Water Estuary receives runoff from several large catchments, and so in order to take a comprehensive approach, it was decided that catchment flood events would also be investigated as part of that study.

6.1 Catchment Flood Events

During large catchment storm events, the Brisbane Water Estuary is likely to undergo flooding as a result of large flows entering from major creeks such as Erina, Kincumber and Narara. These creeks usually contain large amounts of runoff from local sub-catchments and thus areas in close proximity to these creeks often undergo some localised flooding. However, the proposed marina site at Koolewong is located on the foreshore of the estuary's broadwater and thus is unlikely to be affected as severely by such catchment flooding.

6.1.1 Hydrology (RAFTS Modelling)

Runoff hydrographs for the flood study were estimated using the RAFTS (WP Software, 2000, version 6.5) rainfall-runoff modelling package. Based on topographic features and land-use patterns, the catchment was divided into 126 sub-catchments. Topographic information for the Brisbane Water catchment was based on digital 2m contour data provided by council. Land-use patterns were based on aerial photography (taken in 2005 and provided by Council) and land-use zoning information supplied by Council. Further information on this modelling can be found in *Brisbane Water Foreshore Flood Study* (Cardno, 2009).

6.1.2 Catchment Flood Modelling

Using the output from the RAFTS hydrological modelling (**Section 6.1.1**), hydrodynamic modelling of the various ARI catchment events was undertaken. Hydraulic modelling of these design catchment storm events were completed using the Delft3D modelling package. As part of the model system, 69 discharge locations were utilised to distribute the individual catchment flows in a spatially realistic manner along the foreshore of the estuary. These locations can be seen in *Brisbane Water Foreshore Flood Study* (Cardno, 2009).

Analysis of recorded Fort Denison water data, in terms of probability of exceedance is provided in **Figure 6.1**. Inspection of this plot shows that the 1% exceedance level is approximately 1m AHD. To this end, an offshore spring tidal signal that peaked at 1m AHD was utilised for the catchment flood simulations. In order to ensure a critical flood level within the estuary, the catchment discharges were input into the Delft3D model so that the peak of the major catchment flows entering the estuary were in phase with the peak of the offshore tidal signal. Each ARI design event was run for the 3, 6 and 9 hours catchment flow durations.

Peak water level results from each of the design events, being the 2, 5, 10, 20, 50, 100, 200 and 500-years ARI events as well as the PMF event, can be seen in **Table 6.1** for Koolewong. The critical duration for the major catchment flows is approximately 9 hours and hence the hydraulic modelling results show that the peak water levels within the estuary are associated with this critical duration.

Table 6.1 Delft3D Catchment-Only Flood Results

ARI	Peak WL (m AHD)
2	0.77
5	0.91
10	0.95
20	0.99
50	1.04
100	1.10
200	1.19
500	1.24
PMF	1.32

6.1.3 Climate Change Impacts

In addition to this, modelling was conducted to examine the effects of climate change on the Brisbane Water catchment flooding scheme. Included in this assessment were various scenarios covering potential increases in rainfall intensity associated with large catchment events that may occur in the year 2100. Rainfall increases of 10%, 20% and 30% were all considered to examine the sensitivity of the catchment water levels to increases in catchment storm intensity. Additionally, 0.9m SLR by 2100 were incorporated into the hydrodynamic modelling in accordance with the NSW Sea Level Rise Policy Statement (DECCW, 2009). The results can be seen in **Table 6.2** below.

Table 6.2 Delft3D Catchment Flood Results – 100-years ARI Including 2100 Climate Change Scenarios

Rainfall Intensity Increase	Peak WL (m AHD)
0% Increase	1.87
10% Increase	1.89
20% Increase	1.92
30% Increase	1.95

It is important to note that for the climate change scenario of the100yr ARI catchment storm with 0% rainfall increase, the flood level is not merely 0.9m greater than the current 100yr ARI level (1.10m). This is because as the mean water level in the estuary increases, so does its storage. This means that the same volume of water pumped into the estuary as catchment runoff will result in a smaller flood level in 2100 than in 2011 (under NSW government sea level rise projections).

6.2 Ocean Storm Events

As identified by historical records, ocean storm events and extreme tidal levels are the predominant cause of foreshore flooding in this estuary. To quantify the nature of these phenomena both hindcast and hypothesised storm events were simulated using the Delft3D model system.

6.2.1 Design Ocean Storm Events

Simulations for design ARI events conditions were undertaken for 2, 5, 10, 20, 50, 100, 200, 500-years ARI and a PMF event, assumed to be equivalent to a 10,000-years ARI event. The basis for their selection as representing these ARI was the analysed Fort Denison water level data, analyses of long term offshore Botany Bay wave data and Sydney Airport wind data, all in terms of probability of exceedance.

Although these events were selected to represent the abovementioned ARI events, the joint probability of all factors contributing to elevated water levels may result in the return period for the combined selected met-ocean parameters being greater, especially in Brisbane Water; even though there is some correlation between them. For the waves and wind, it was assumed that peak conditions must persist for six hours to ensure that water levels could propagate into the estuary. This is because an elevated ocean water level of short duration will not have time to 'fill' Brisbane Water. Cardno's *Brisbane Water Foreshore Flood Study* (2009) describes the boundary conditions for each simulation and how these boundaries were obtained.

Table 6.3 summarises the peak water levels for each ARI at Koolewong.

Table 6.3 Delft3D Ocean Storm Results

ARI	Peak WL (m AHD)
2	1.23
5	1.34
10	1.41
20	1.48
50	1.58
100	1.66
200	1.73
500	1.84
PMF	2.06

These results demonstrate that ocean storm events are the predominant cause of flooding in the Koolewong Foreshore Area. In fact a 5-years ARI ocean storm results in a higher flood level than a 500-years ARI catchment flood. These results are in keeping with historical records, which showed that ocean storm events are the predominant cause of foreshore flooding in Koolewong.

6.2.2 Climate Change Effects

In addition to this, modelling was conducted to examine the effects that climate change may have on the extent of ocean storm flood levels in Brisbane Water. This involved simply adding 0.9m to the previously analysed ARI ocean storm surge levels. Whilst it would be expected that this would simply raise the flood levels by 0.9m, it was possible that changes in mean water level in the estuary may affect the attenuation of storm surge through the estuary. The results of this modelling can be found in **Table 6.4** below.

Table 6.4 Delft3D Ocean Storm Results Including Climate Change SLR

ARI	Peak WL (m AHD)
2	2.13
5	2.24
10	2.31
20	2.38
50	2.48
100	2.56
200	2.63
500	2.74
PMF	2.96

As expected, these results showed that the attenuation of storm surges through the estuary were not affected and thus flood levels under the 2100 climate change scenario were 0.9m greater than the present day levels.

6.3 Joint Occurrence of Catchment Floods and Ocean Storm Events

Cardno (2009) also addressed the matter of joint occurrence of catchment floods and ocean storm events. It is noted that the highest water levels observed at Koolewong (1.57m AHD, May 1974 and 1.09m AHD, April 1990), occurred at times of very little or no catchment rainfall and were ocean events. Both are much higher than Higher High Water (Spring Solstices) at Koolewong, **Table 5.1**.

Adopting a 2-years ARI catchment flood jointly occurring with the 50-years ARI ocean storm event, consistent with analyses of past event joint occurrence in Brisbane Water, causes a negligible increase to the ocean storm caused water level at Koolewong (<5cm). This is because the volume of catchment runoff is small up to the 5-years ARI catchment flood and large areas of Davistown and Saratoga are flooded in such an ocean event and the effect of the increase in water volume is reduced by the flood plain storage. The most likely situation would be much less catchment rainfall and runoff than a 2-years ARI event.

Design flood levels for the site for these joint occurrences are described in **Section 8.1**.

7 SEDIMENT RE-SUSPENSION

The seabed sediments at the marina site have been sampled by Cardno to provide sediment particle size information, amongst other parameters. Based on twelve sediment samples, the fines content can vary between 15 and 90% - <75µm. Hence there is significant potential for propeller action to re-suspend this fine material, thereby causing plumes of suspended sediment. Given the number of vessels to be accommodated, this might lead to detrimental environmental effects.

Cardno have been advised (pers. comm. Bob Tuckwell – Doug Treloar) that the drafts of likely motor vessels that would moor at the marina range from 0.8m (9m vessel at the inner berths in 3m depth at LAT) to 1.8m (21m vessel at the outer berths in 5m depth at LAT). By draft, in the context of this report, is meant the depth to the bottom of the propeller blades. A sailing vessel of 9m would have a draft of about 1.5m and the biggest sailing vessel that could pass beneath The Rip Bridge is about 12m. Such a vessel would have a draft of about 2m. However, whilst their drafts are greater than those for equivalent length motor vessels, their propellers are not.

Hence at all berths, keeping the larger vessels to the deeper berths, there would be 2m of water column beneath the propellers of the vessels using the marina. Provided that vessel speeds are kept to no more than 2 knots within the marina precinct there is little likelihood that bottom sediments will be re-suspended by the boats that would use this facility.

8 FLOOD RISK MANAGEMENT AND PLANNING

This section of the report responds to Director-General's Requirements 2.4 (emergency access) and 6.6 (flood risk). It provides a discussion on:-

- Flooding and flood hazard on the subject site, including the marina, jetty and car park (**Section 8.1**);
- Planning guidelines and other relevant development controls or standards (**Section 8.2**); and
- Recommendations for managing flood risk (**Section 8.3**).

For the purposes of this assessment, it has been assumed that the Koolewong marina would have a design life of up to 50 years, which is representative of a typical design life for a similar structure. The flood risk management and planning assessment, including the assessment of flood hazard, is therefore based on a 2050 sea level rise planning horizon.

8.1 Flooding and Flood Hazard on the Subject Site

Design Flood Levels

Flood conditions for the Brisbane Water foreshore were considered in the *Foreshore Flood Study* (Cardno, 2009). Flooding of the subject site is primarily due to elevated estuarine water levels. The terrestrial portion of the site is not affected by catchment flooding, however catchment flood flows from other parts of the larger Brisbane Water catchment do contribute a minor increase (<5cm) to estuarine water levels (**Section 6.3**). The coincident catchment and ocean storm flood levels on both the estuarine and terrestrial portions of the site are therefore a function of:-

- Elevated estuarine water levels due to coastal inundation/elevated ocean water levels;
- Minor increases in estuarine water level due to catchment flooding elsewhere in the Brisbane Water catchment; and
- Sea level rise (for climate change scenarios).

Table 8.1 summarises the design estuarine flood levels for the subject site based on these parameters.

Table 8.1: Design Flood Levels

Estuarine Flood Level	Present Day (m AHD)	2050 (+0.4m SLR) (m AHD)
20-year ARI	1.53	1.93
50-year ARI	1.63	2.03
100-year ARI	1.71	2.11

The majority of the land-based portion of the site lies above the 100 year ARI level (present day) of 1.71 mAHD. Portions of the car park for the site would be inundated generally by approximately 0.4 m for the 100 year ARI (2050) scenario of 2.11 mAHD. This is described further below.

In a similar fashion to that discussed in **Section 4.5**, based on consideration of probability of co-occurrence, the 20-year ARI wave conditions (with a crest height of 0.25m) would typically occur during a 100-year ARI estuarine flood event. This would result in inundation to levels of 1.96 mAHD for the present day 100 year ARI and 2.36 mAHD for the 2050 100 year ARI event.

The marina itself would be on pontoons and would rise with the estuarine water levels, such that it is not submerged during the 100-years ARI estuarine flood event.

The duration of inundation for the land based portion of the site is approximately 3 – 4 hours for the 2050 scenario (i.e. the time at which inundation occurs via overtopping of the foreshore).

Flood Impact Assessment

Due to the fact that the site is dominated by estuarine flooding as opposed to catchment flooding, the proposal will not have any impact on flood levels either on the site itself, or on any neighbouring sites.

Provisional Flood Hazard

Flood hazard has been considered for the proposed jetty, marina and land based infrastructure only, and as such **Figure 8.1** does not consider hazard for the estuary waterway itself. Flood hazard has been evaluated by considering the provisional hazard guidelines within the *Floodplain Development Manual* (NSW Government, 2005).

For the present day, the 100-years ARI estuarine flood levels are lower than the crest of the rock wall (which ranges from 1.85 – 2.26 mAHD, except for a small portion near Brisbane Water Drive (which ranges between 1.56 mAHD and 1.72 mAHD). Therefore the terrestrial portion of the site would not be inundated for a flood level of 1.71 mAHD and would have minor inundation for the event with wave breaking on the rock wall (of the order of 0.1 m). Presuming the deck of the jetty is set at 1.75 mAHD (**Section 4.5.2**), the deck of the jetty would be inundated by wave action (up to 1.96 mAHD, ie a depth of 0.21m).

During an estuarine flood event, some waves of up to 20cm in height may also propagate through the site at a velocity of up to 2m/s. These waves are not expected to change the level of hazard on the jetty or in the car park. Small waves may also be generated by any cars passing on Brisbane Water Drive.

For the 2050 planning horizon (i.e. with 0.4m sea level rise), the site would also be subject to low hazard under the 100-years ARI estuarine flood event (**Figure 8.1**).

Flood Risk

Under day-to-day operational conditions, there is a very low risk from coastal or flood hazard on the subject site due to the existing elevation of the site, which sits above the HHW level of 0.6m AHD (**Table 5.1**). The gradual rise in estuarine water levels associated with projected sea level rise in the planning horizon for the proposed marina would progressively reduce this freeboard, however, at 0.4m sea level rise the terrestrial portion of the site would still sit above the approximate HHW level of 1.0m AHD.

There is an existing level of risk from estuarine flooding on the overall site under the 100-years ARI event. The proposal is not expected to substantially increase the level of risk during an estuarine flood event because the proposal would not alter flood levels or extents, and would result in only a minor increase in the number of people that may be present on the site at any one time.

Based on the existing deck levels, the jetty is submerged in a 100-years ARI estuarine flood event and patrons of the marina would have difficulty walking between the marina and the foreshore. At these times, there would be a risk (albeit low given the time available to depart from the pontoons prior to peak water levels occurring), that people on the marina would be unable to get to the shore and would potentially become isolated for a few hours with their vessels until the peak water level begins to drop. This represents a minor increase in the level of risk currently experienced on site, due to the presence of additional people on the marina. Design parameters for the marina pontoons have been recommended in accordance with the requirements of the Australian Standard Guidelines for design of marinas (AS3962), and there is a low level of risk to any vessels (or passengers or crew) berthed at the marina during a storm event.

For the 2050 planning horizon (and based on the existing levels of the jetty and car park), the jetty would be submerged in a 100-years ARI estuarine flood event, as would the low-lying portions of the car park, which would be inundated up to around 0.4m depth. Patrons would be able to take refuge on higher ground on the foreshore (or within the building, where the ground floor level is 2.23 mAHD), but would have significant difficulty transiting the jetty between the floating pontoons and the foreshore. At depths in excess of 0.3m, cars may become buoyant, and there is a risk of damage to vehicles or injury to people. It is also likely that the site would become isolated for 3 -4 hours due to inundation of parts of Brisbane Water Drive, until such time as the tide drops and the estuarine water levels recede.

To manage the flood risks (to persons and property) it is recommended that a flood emergency response plan be prepared for the site, noting that there will be a site manager located at the facility 8am – 8pm (seven days a week) at normal operating times and available 24/7 in the event of a flood. The flood emergency response plan should cover a range of matters including the relocation of vehicles from the car park and the management of boat owners and visitors in the event of a flood.

8.2 Relevant Planning Instruments

Gosford City Council (GCC) currently adopts a Flood Planning Level (FPL) of the existing 100-years ARI flood level plus 0.5 metres freeboard for all properties within the 100-years ARI flood extent. An estuarine FPL of 2.45m AHD presently applies for the entire Brisbane Water foreshore. This FPL was based on the estuarine flood level observed during the 1974 severe ocean storm event (1.95m) and incorporates a 0.5m freeboard. Cardno (2009) determined that this storm event is likely to have been closer to the 200-years ARI event, and as such, the 2.45m FPL may be somewhat conservative for the existing conditions. However, given sea level rise predictions, the current FPL may be exceeded in the future, but generally not in the design life of the proposed facility.

Under Section 6.8 Council's draft Development Control Plan (DCP, 2009) for water cycle management, car parking for commercial developments are required to be flood free in the 100-year ARI so as to enable flood free evacuation from the site. There are no flood-related requirements within the draft DCP relating to jetties or marinas. At present the car park is generally set at approximately the 100 year ARI level of 1.71 mAHD (without wave action) with some small isolated areas slightly below this level.

As the site is to have an on-site manager, an alternative to setting the car park at the 100 year ARI level (for both present and 2050 conditions) is to implement a flood emergency response plan ensuring that vehicles are relocated prior to the car park being inundated. It is understood that vehicles are not to be allowed to be parked at the site over night.

The site is not currently under a floodplain risk management study or plan (because it is not subject to catchment flooding), but will be subject to the forthcoming *Brisbane Water Foreshore Coastal Floodplain Risk Inundation Management Study and Plan* currently being prepared by Council.

Other relevant guideline documents include the *Practical Consideration of Climate Change – Floodplain Risk Management Guideline* (DECC, 2007) and the *NSW Coastal Planning Guideline: Adapting to Sea Level Rise* (DoP, 2010). The requirements of DECC (2007) were considered in development of the estuarine flood levels, as can be seen in **Sections 6.1.3** and **6.2.2**.

The *Coastal Planning Guideline* (DoP, 2010) requires that the proposal be assessed against a series of eight planning criteria applicable to coastal risk areas. This assessment is presented in **Table 8.2**.

Table 8.2: Assessment against Planning Criteria for Development in Coastal Risk Areas

Criteria	Assessment
1. Development avoids or minimizes exposure to immediate coastal risks (within the immediate hazard area or floodway).	The development is coastal dependent and may therefore not be located elsewhere, outside of the immediate hazard area. However, a series of recommendations have been made in **Section 8.3** as to how the level of risk may be reduced.
2. Development provides for the safety of residents, workers or other occupants on-site from risks associated with coastal processes.	Design recommendations for the marina have been provided on how to reduce the level of risk to patrons of the marina, and design parameters for the structure provided in accordance with the Australian Standards Guidelines for design of marinas (AS3962).
3. Development does not adversely affect the safety of the public off-site from a change in coastal risks as a result of the development.	The proposal would not result in any impacts on coastal processes that would result in an increase to risk to members of the public off-site.
4. Development does not increase coastal risk to properties adjoining or within the locality of the site.	The proposal is not expected to result in any impacts on longshore sediment transport or patterns of erosion and accretion. It is noted that the shoreline in this part of Brisbane Water is protected by rock walls.
5. Infrastructure, services and utilities on-site maintain their function and achieve their intended design performance.	The location and design of any services or utilities installed as part of the proposal (e.g. drainage works) should be undertaken in accordance with the recommendations provided in **Section 8.3**. No additional impacts on existing on-site services and utilities would occur as a result of the proposal.
6. Development accommodates natural coastal processes including those associated with projected sea level rise.	The marina design incorporates floating pontoons that can accommodate an increase in estuarine water levels under sea level rise conditions.
7. Coastal ecosystems are protected from	Reference is made to the specialist aquatic ecological

Criteria	Assessment
development impacts.	report (Cardno, 2011).
8. Existing public, beach, foreshore or waterfront access and amenity is maintained.	The proposal would not negatively impact on public access or amenity, except during the short term construction phase. Access and amenity would be enhanced by the provision of boating facilities.

A review of the draft *Brisbane Water Estuary Management Plan* (Cardno, 2010) indicates that the proposal in its conceptual form is not inconsistent with the overarching aims or objectives for management of the Brisbane Water estuary, provided the proposal is implemented with appropriate management of any impacts during the construction phase, and with due consideration of the recommendations made in **Section 8.3**.

8.3 Recommendations

Based on a review of the proposal in its conceptual form, a series of recommendations have been provided that reduce the level of risk from flooding through the detailed design process:

- Stormwater drainage should be designed in accordance with Council's *Design Specification for Survey, Road and Drainage Works* (GCC, 2008).
- A Flood Emergency Response Plan should be prepared for the site to address both present and 2050 flood risks for patrons of the marina.
- The pontoons should as a minimum be designed so as to accommodate the 100-years ARI estuarine flood level for the 2050 planning horizon, by which time the structure will have reached the end of its design life.
- The pontoons should be designed so as to attenuate wave activity in accordance with Australian Standard *Guidelines for design of marinas* (AS3962).
- Because of the likely practical jetty level and the design estuarine flood levels, any electrical services in particular, need to be designed with these issues in mind to ensure safety.

9 CONCLUDING REMARKS

This report has summarised the methodology and findings of Coastal Processes and Flood Investigations undertaken as part of proposed Koolewong Marina development. The proposed re-development includes an extension and upgrade to the existing jetty footprint, the construction of a marina in the form of floating pontoons which would accommodate 50 new berths, and reconfiguration of the existing car park. The pontoons will provide some protection from wave activity and will be held in position by piles. No dredging is proposed.

Wave climate investigations have shown that the marina site is not classified as having a 'good' wave climate as described by the Australian Standards Guidelines for design of marinas (AS3962). However, appropriate design of the mooring pontoons and layout will provide satisfactory conditions.

Hydrodynamic and morphological investigations have shown that current speeds in the vicinity of the marina are not strong enough to enable significant sediment transport at the seabed, and as a result maintenance dredging is unlikely to be required to maintain adequate depths at the site. There are no existing shoreline or seabed instabilities at the site and hence the proposed development would not change this state.

It has been found that for the Koolewong area, severe ocean storms cause the highest estuarine flood levels, and that local catchment flooding does not contribute to flood hazard on the site. The investigations were based on extensive data analysis and calibrated modelling systems, with the outcomes showing considerable consistency between the two data types.

Following the range of coastal processes investigations undertaken it is considered that the expansion of the marina will not have a significant effect on the existing coastal processes and the investigations have not identified any likely changes to the hydrodynamic, morphological and water quality conditions at the site.

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Figures

Cardno
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Proposed Expansion of Koolewong Marina

LOCALITY PLAN

EL1011059 March 2011
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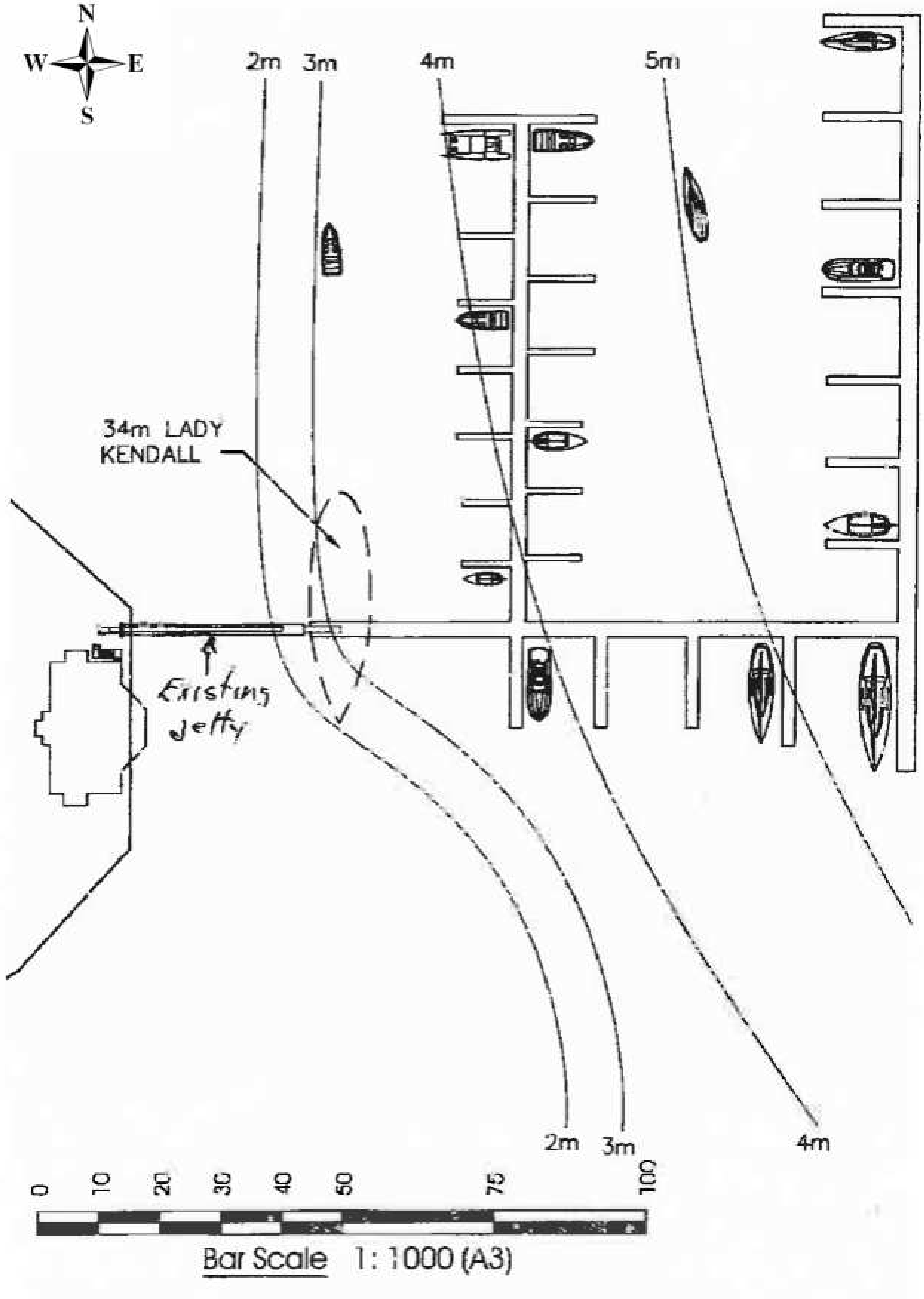
Figure 1.1



Proposed Expansion of Koolewong Marina

PROPOSED MARINA EXPANSION

Figure 1.2

Proposed Expansion of Koolewong Marina

EXISTING JETTY

EL1011059 March 2011
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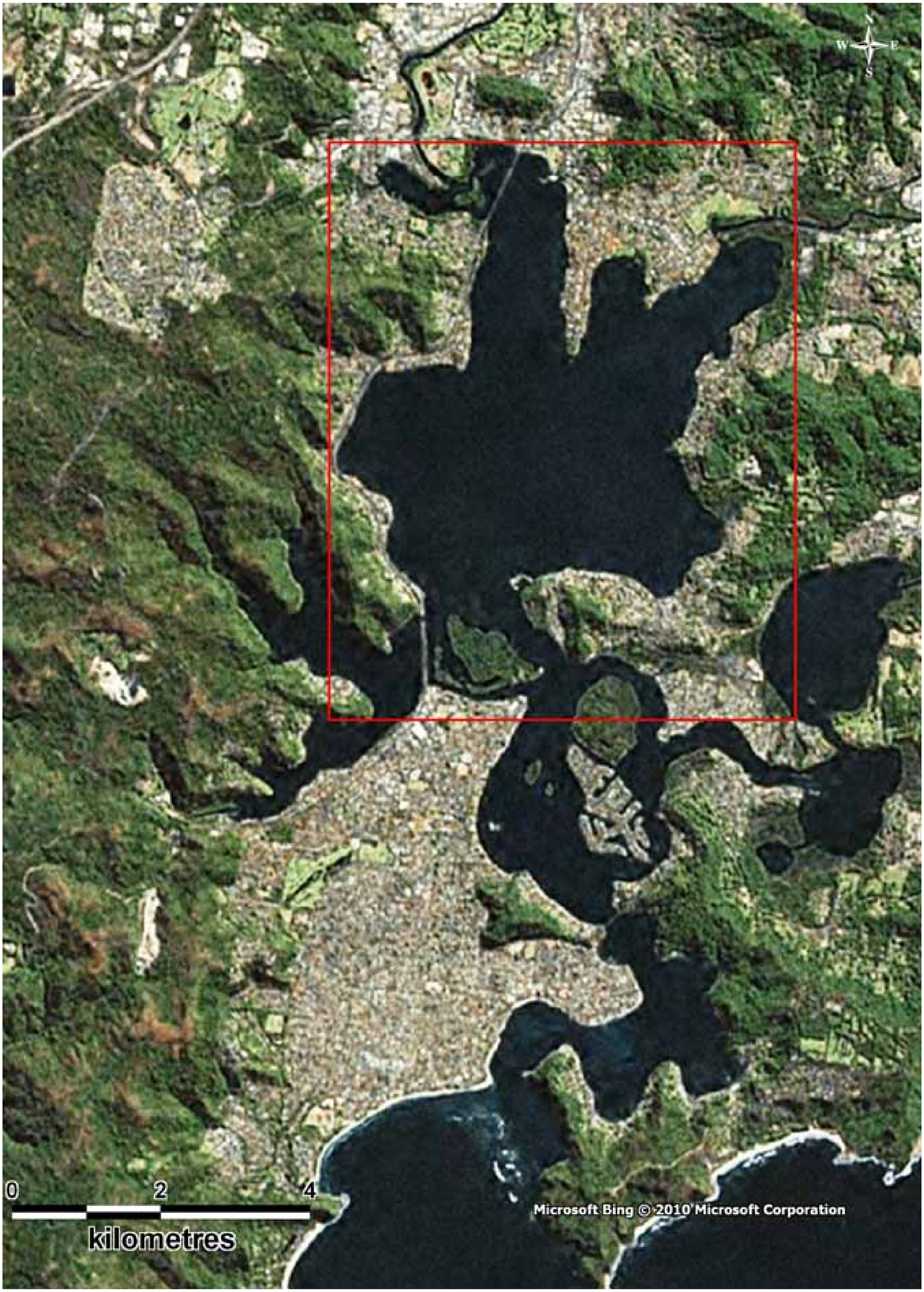
Figure 1.3



Proposed Expansion of Koolewong Marina

SWAN MODEL GRID

Figure 4.1

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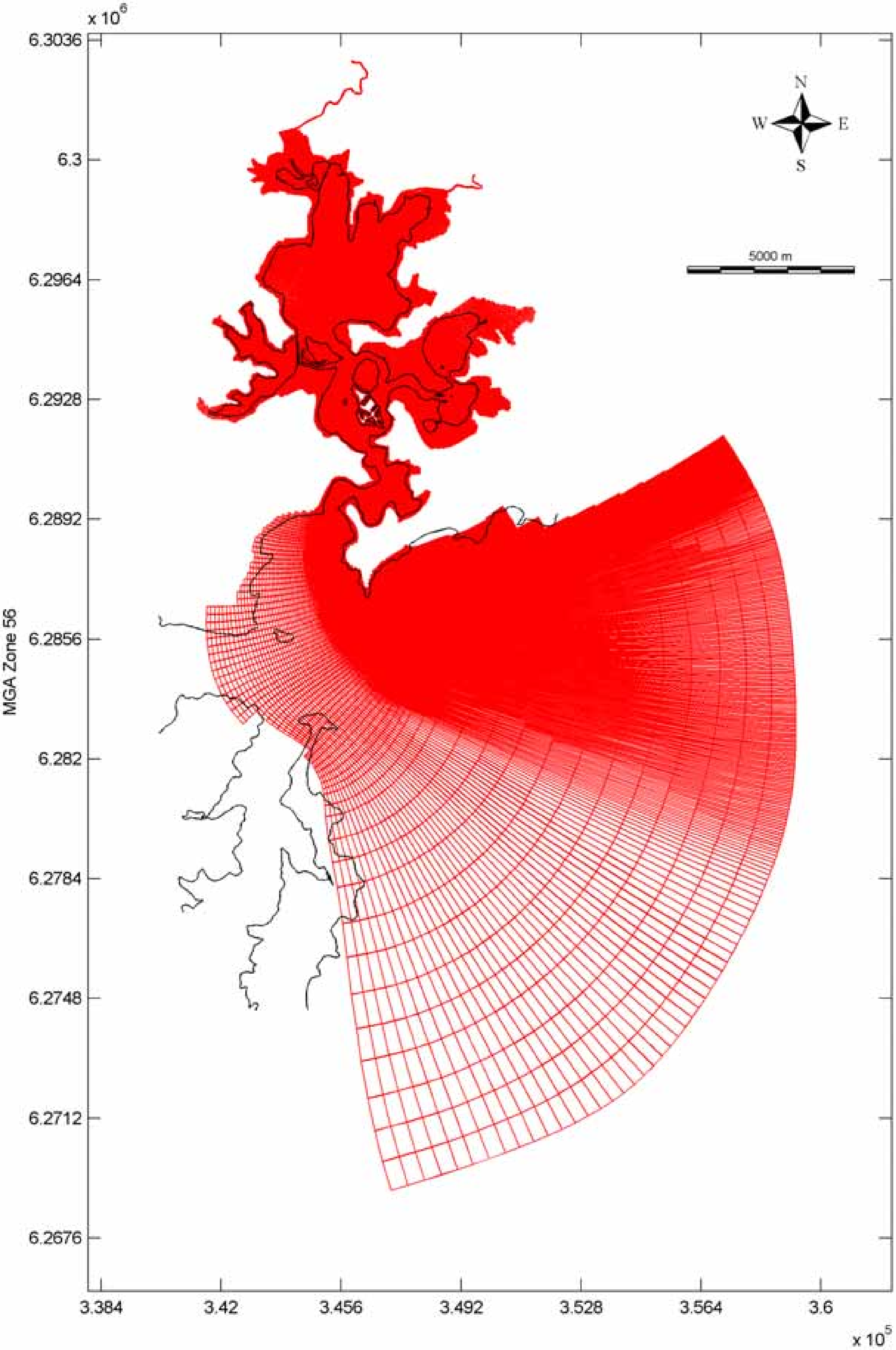
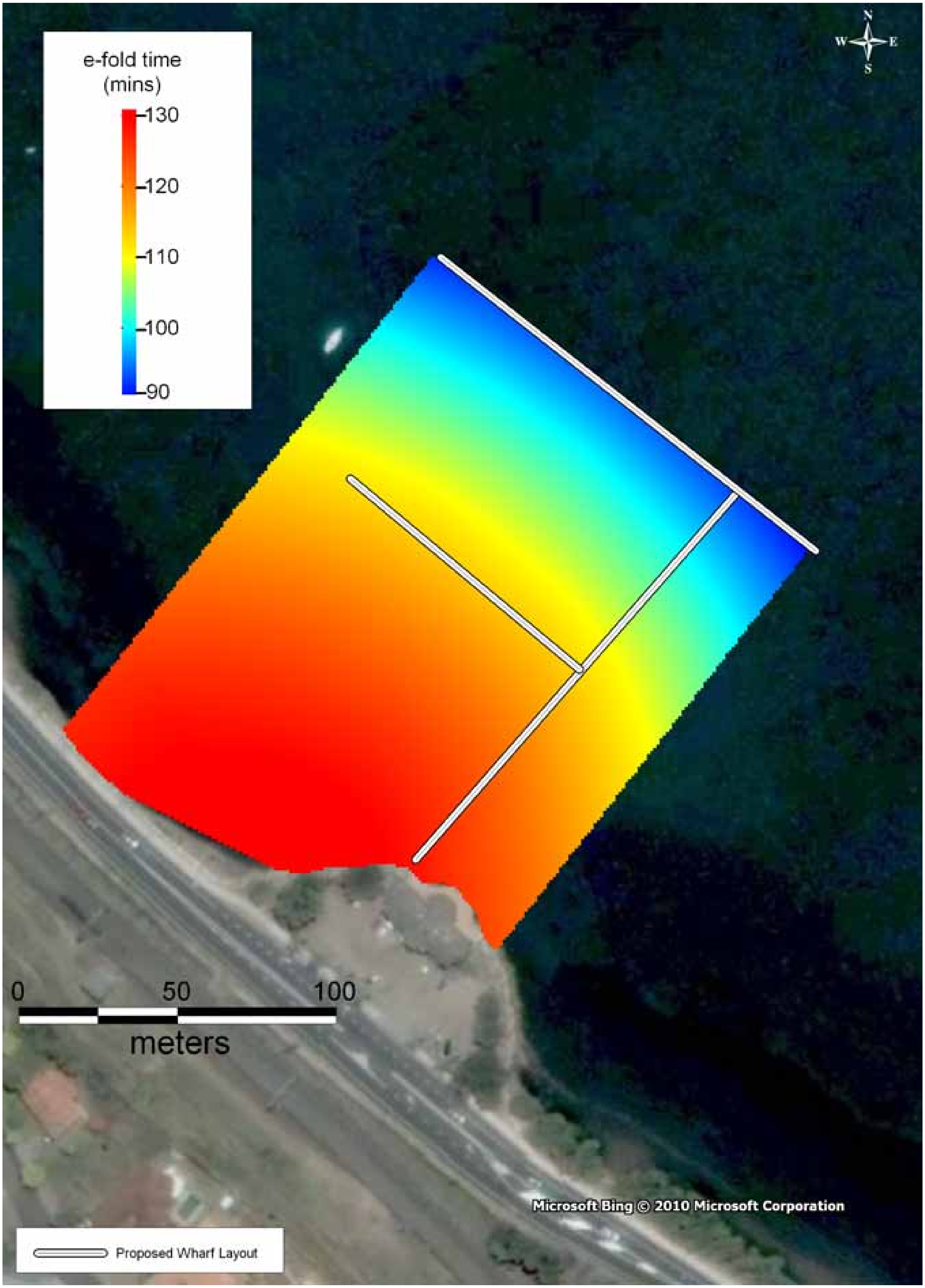


Proposed Expansion of Koolewong Marina

DELFT3D MODEL GRID

Figure 5.1



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Proposed Expansion of Koolewong Marina

MARINA FLUSHING
e−FOLDING TIME
Figure 5.2

Proposed Expansion of Koolewong Marina

ROCK REVETMENT AND ADJACENT MANGROVES

Figure 5.3

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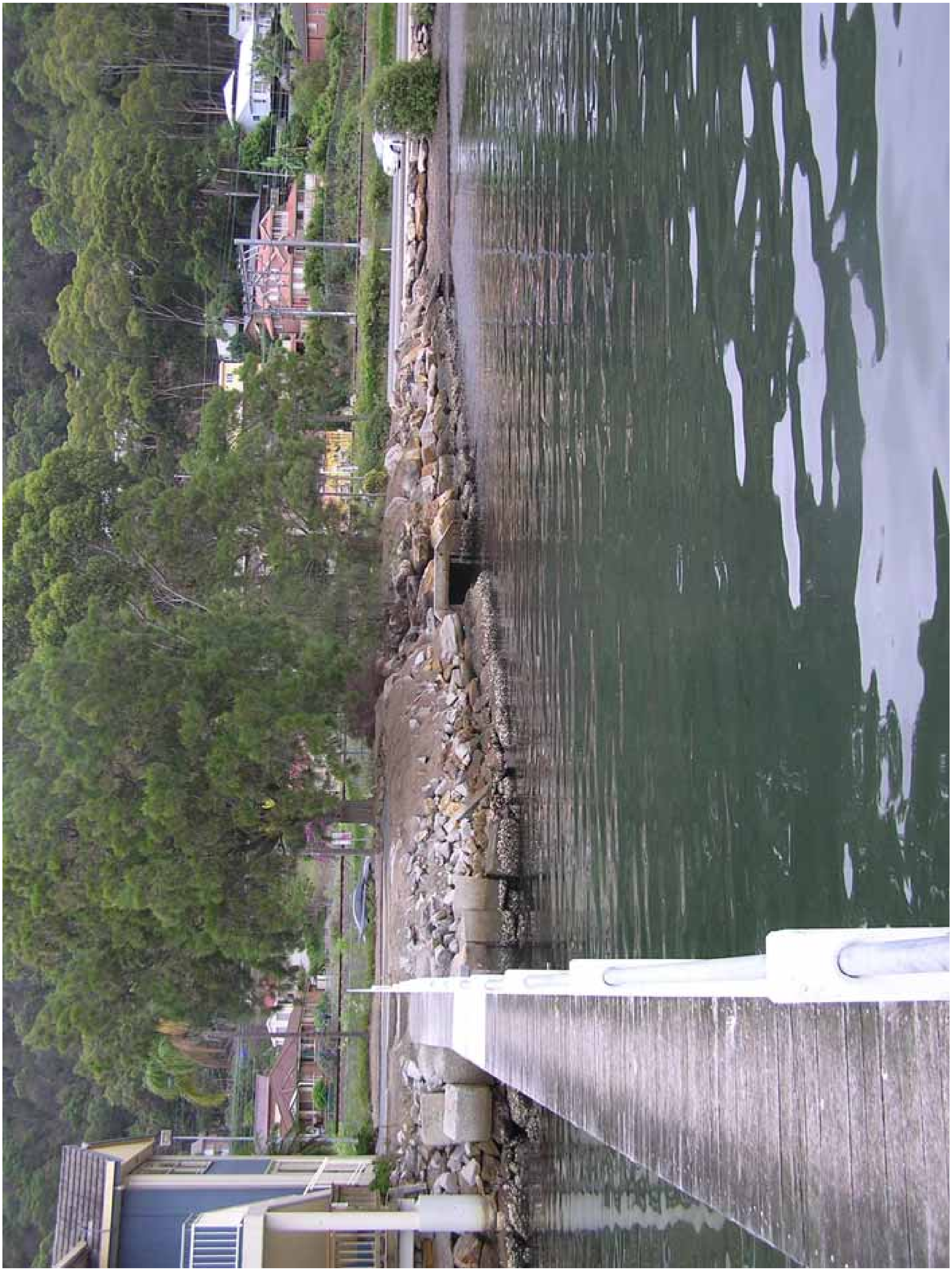

Proposed Expansion of Koolewong Marina

NORTHERN STORM WATER DRAIN

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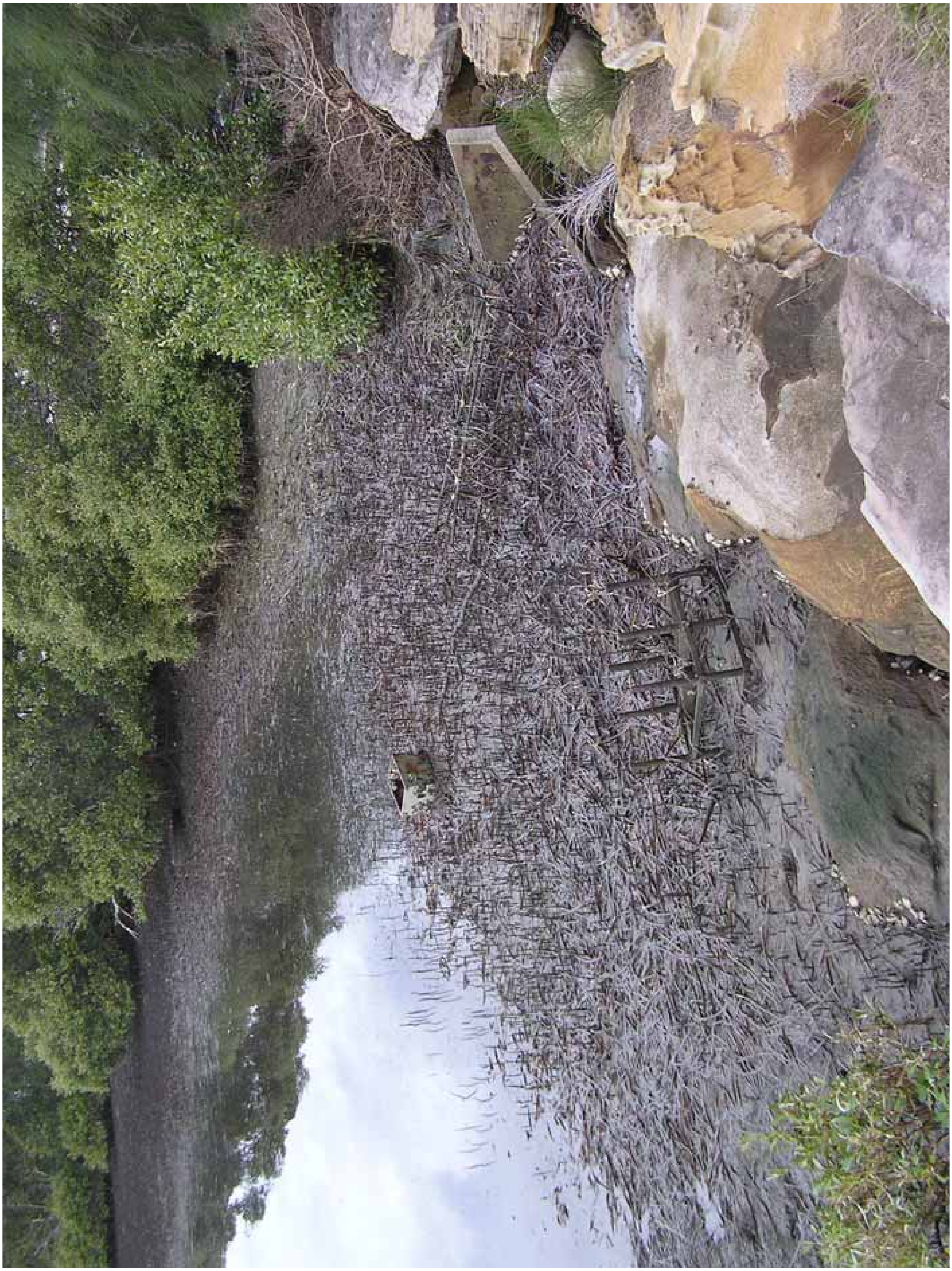
Figure 5.4



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Proposed Expansion of Koolewong Marina

SOUTHERN STORM WATER DRAIN

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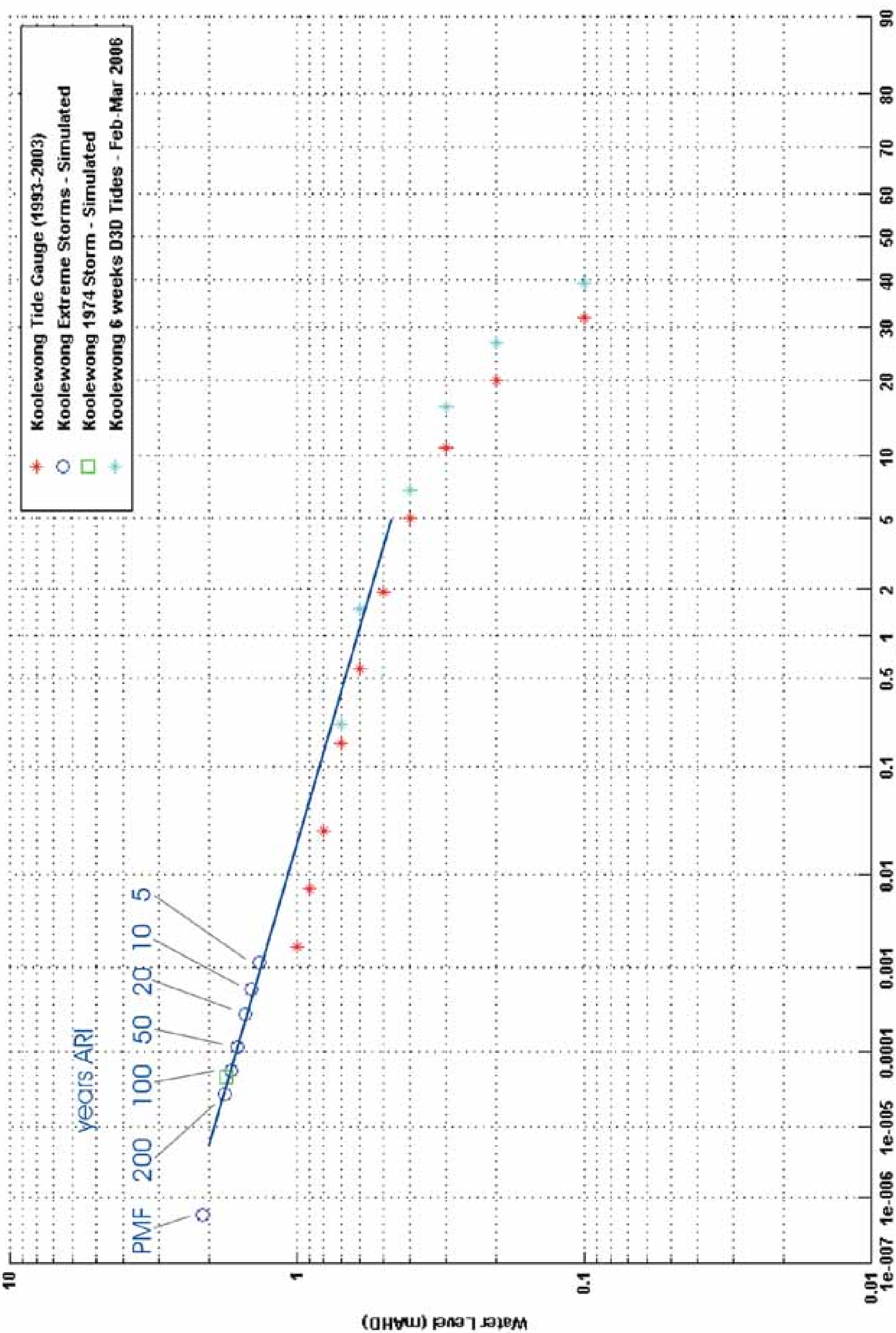
Figure 5.5



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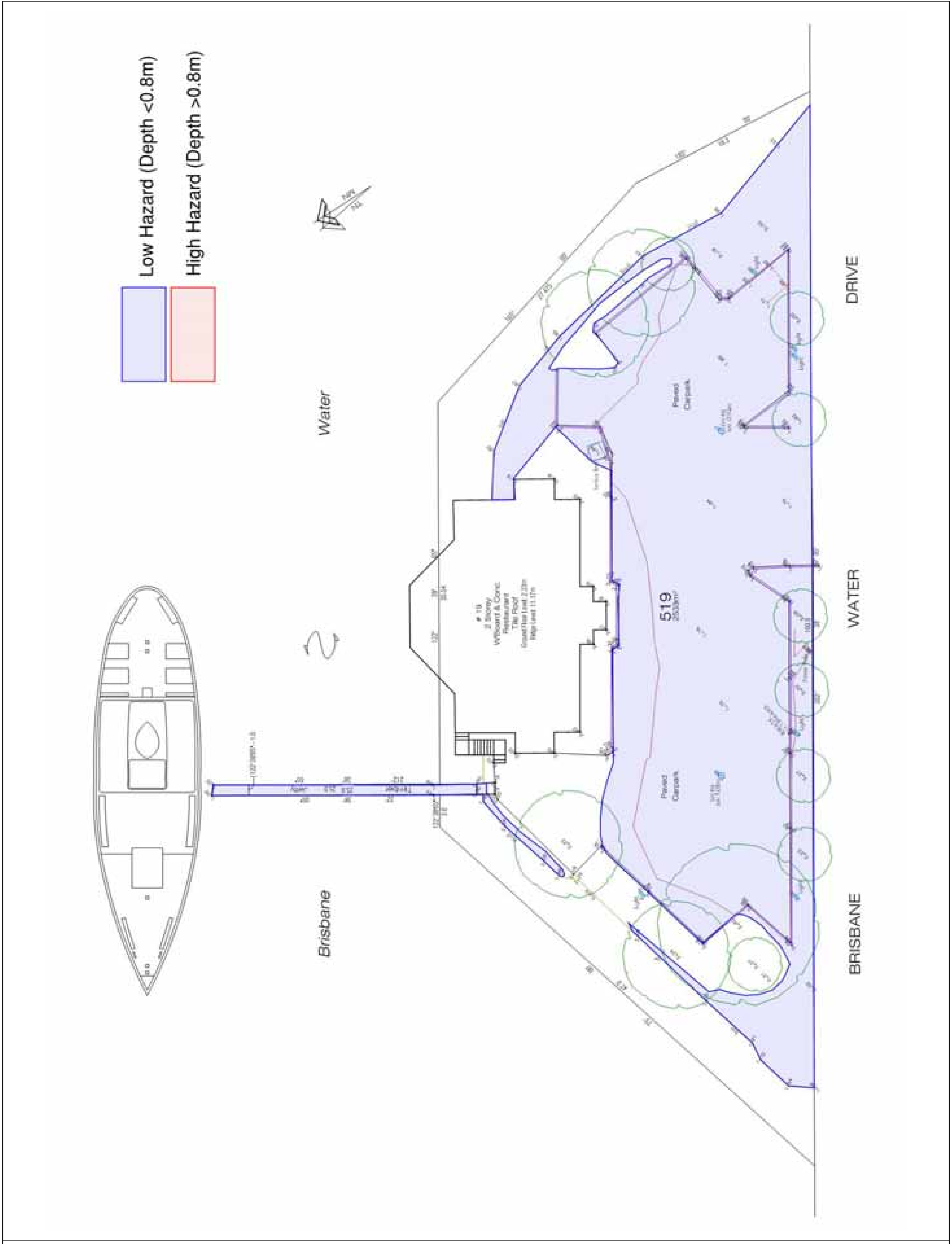
Proposed Expansion of Koolewong Marina

KOOLEWONG WATER LEVEL
PROBABILITY OF EXCEEDANCE CURVES
Figure 6.1



Proposed Expansion of Koolewong Marina

100 YR ARI ESTUARINE FLOOD HAZARD INCORP. 0.4m SLR

Figure 8.1

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EL1011059 August 2011
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Appendix A

Physical Processes

The purpose of this section is to describe the physical processes that are important to the overall physiography of Brisbane Water in the Koolewong area. These processes are: -

- Waves
- Currents
- Water Levels
- Winds

Wave Processes

Waves that occur in estuarine areas may have energy in three distinct frequency bands. These are principally related to the generation and propagation of ocean swell (wave periods 7 to 20 seconds) and local sea (less than 7 seconds, typically even shorter at this site). Large waves generated by an ocean storm are generally categorised as sea because wind energy is still being transferred to the ocean. Long waves (wave periods greater than 25 seconds) occur during storms and are caused by wave grouping. However, long period waves are not important to this study because seabed pipelines are not affected by them. Nor does swell propagate to Koolewong and so only local wind waves, and waves caused by boats from time-to-time, are important.

Natural water 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_s — significant wave height - either H_{mo} or $H_{1/3}$, which is the average of the highest 1/3 of waves in a record
- H_{mo} — significant wave height (H_s) based on mo where mo is the zeroth moment of the wave energy spectrum (rather than the time domain H1/3 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 energy 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.

Water waves also have a dominant direction of wave propagation and directional spread about that direction that 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 that is in the more shallow water has a lower speed than those parts in deeper water. When waves approach a coastline obliquely these speed 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 over a steep sided trench at a small angle to the trench alignment may undergo a spatially rapid refraction process, effectively causing wave reflection.

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. However, because much of the saltwater intake pipeline is in deep water relative to the wave periods for the local sea waves, this process was important only for the ocean storm waves that propagate to the site in severe storms.

A turbulent boundary layer forms above the seabed with associated wave energy losses that are manifested as a continual reduction in wave height in the direction of wave propagation - leaving aside further wind input, refraction, shoaling and wave breaking. The rate of energy dissipation increases with greater wave height and reducing depth.

Wave breaking occurs in shallow water when the wave crest speed becomes greater than the wave phase speed. For irregular waves this breaking occurs in different depths so that there is a breaker zone rather than a breaker line. Seabed slope, wave period and water depth 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. Lower set-up occurs in estuarine entrances, but the momentum flux remains the same. Wave set-up is smaller where waves approach a beach obliquely, but then a longshore current can be developed. Wave grouping and the consequent surf beats also cause fluctuations in the still water level. Wave set-up is also smaller for local sea where the wave periods are relatively short. However, these processes are not important to this study.

Wave diffraction will not be particularly important for this study because there will be no real obstructions near the shoreline.

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, that is 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. Long waves have very irregular spectral forms.

For structural design in the marine environment it is necessary to define the H_{max} parameter related to storms having average recurrence intervals (ARI) of a pre-determined number of years. However, the expected Hmax, relative to Hs in statistically stationary wave conditions, increases as storm/sea state duration increases. Based on the Rayleigh Distribution the usual relationship is:-

$$H_{max}=H_s\sqrt{(0.5\,\ell nNz\ )}$$

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

ℓn is the natural logarithm

This relationship has been found to overestimate H_{max} by about 10% in severe ocean storms. In shallow water the relationship is not fulfilled. However, analyses of wave data sets for coastal Australia shows that a factor of 2.1 can occur reasonably often. 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.

In the Koolewong area current speeds are low and wave propagation will not be affected by currents.

There is no measured wave data at this site. Hence local sea wave conditions were developed using available wind data, see **Section 3.2**, together with a numerical wave model; see **Section 4**.

Currents

Currents within Brisbane Water are caused by a range of phenomena, including: -

- Astronomical Tides
- Winds
- River/Creek Discharges
- Coastal Trapped Waves and Other Tasman Sea Processes
- Nearshore Wave Processes
- Density Flows

The astronomical tides are caused by the relative motions of the Earth, Moon and Sun, see **Section 2.3**. The regular rise and fall of the tide level in the sea causes a periodic inflow (flood tide) and outflow (ebb tide) of oceanic water to the harbour and mixed oceanic and oceanic/river water from the harbour to the sea.

A consequence of this process is the generation of tidal currents. The volume of sea water that enters the Brisbane Water or leaves it on flood and ebb tides, respectively, is termed the tidal prism; which parameter varies due to the inequality between tidal ranges. The tidal prism is affected by changes in inter-tidal areas, such as reclamations, but not by dredged areas below low tide, such as navigation channels and trenches. There would be no change in tidal prism as a result of this proposal.

Wind forcing is applied to the water surface as interfacial shear, the drag coefficient and consequent drag force varying with wind speed. Momentum from the wind is gradually transferred down through the water column by vorticity, the maximum depth of this effect being termed the Ekman depth. At the surface, wind caused currents are in the direction of the wind, but in the southern hemisphere they gradually turn to the left of the wind direction until they flow in the opposite direction at the Ekman depth. Brisbane Water is too shallow for this condition to develop fully and wind driven currents are affected by the seabed boundary layer and seabed form. Wind driven currents diminish with depth. Because wind forcing is applied at the water surface, the relative effect is greater in shallow water where there is less water column volume per unit plan area. Therefore wind driven currents are greater in more shallow areas. Maximum surface current speed is in the order of 1% to 3% of the wind speed, depending on water depth. Where water is piled up against a coastline by wind forcing a reverse flow develops near the seabed.

Density currents may be caused by freshwater inflows, for example, when Narara Creek is in flood. The freshwater is more buoyant and tends to spread across the surface until mixing with the ambient seawater occurs. However, there are no significant currents of this type at this site, other than from storm water drains that carry suspended sediments and contaminants into Brisbane Water.

Coastal Trapped Waves (CTW) are long period wave phenomena that propagate northward along the continental shelf. Their origin is not fully understood, but they are believed to originate from the passage of successive high and low pressure meteorological systems across southern Australia. These systems have inter-arrival times varying from 3 to 7 days, typically, and these are the periods of the observed CTW. These waves are irregular and cause approximate coast parallel currents and variations in water levels. They are trapped on the continental shelf by refraction and the Coriolis force. CTW are known to occur on the continental shelf of NSW and will affect observed water levels in Port Jackson. They cause flow into and out of Brisbane Water, but current speeds will be low.

In terms of this study, wind driven and tidal currents will be most important.

Water Levels

Water level variations in Brisbane Water 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
- Tsunami

- Greenhouse Effect
- Global Changes in Meteorological Conditions

Eustatic sea level changes are long term worldwide 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 in this study. Nevertheless, a minimum rise of 1.5mm per annum is now generally accepted. Tectonic changes are caused by movement of the Earth's crust; they may be vertical and/or horizontal

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.

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.

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 hecta-Pascal (hPa) drop in central pressure below the average sea level atmospheric pressure in the area for the particular time of year, typically about 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 0.05m. 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 or a coastal structure and the still water level, comprising tide plus storm surge. Additionally, run-up level varies with surf-beat, which arises from wave grouping effects. Wave set-up is included implicitly in wave run-up. Neither is directly important to this study, but wave run-up at the shoreline seawall is important to the project; more so when sea level rise is included.

Tsunami 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. The highest tsunami observed in the Sydney region is about 0.8m (crest to trough) recorded at Fort Denison and caused by an earthquake in Chile in 1960. Tsunami penetration to Koolewong is likely to be minor.

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.

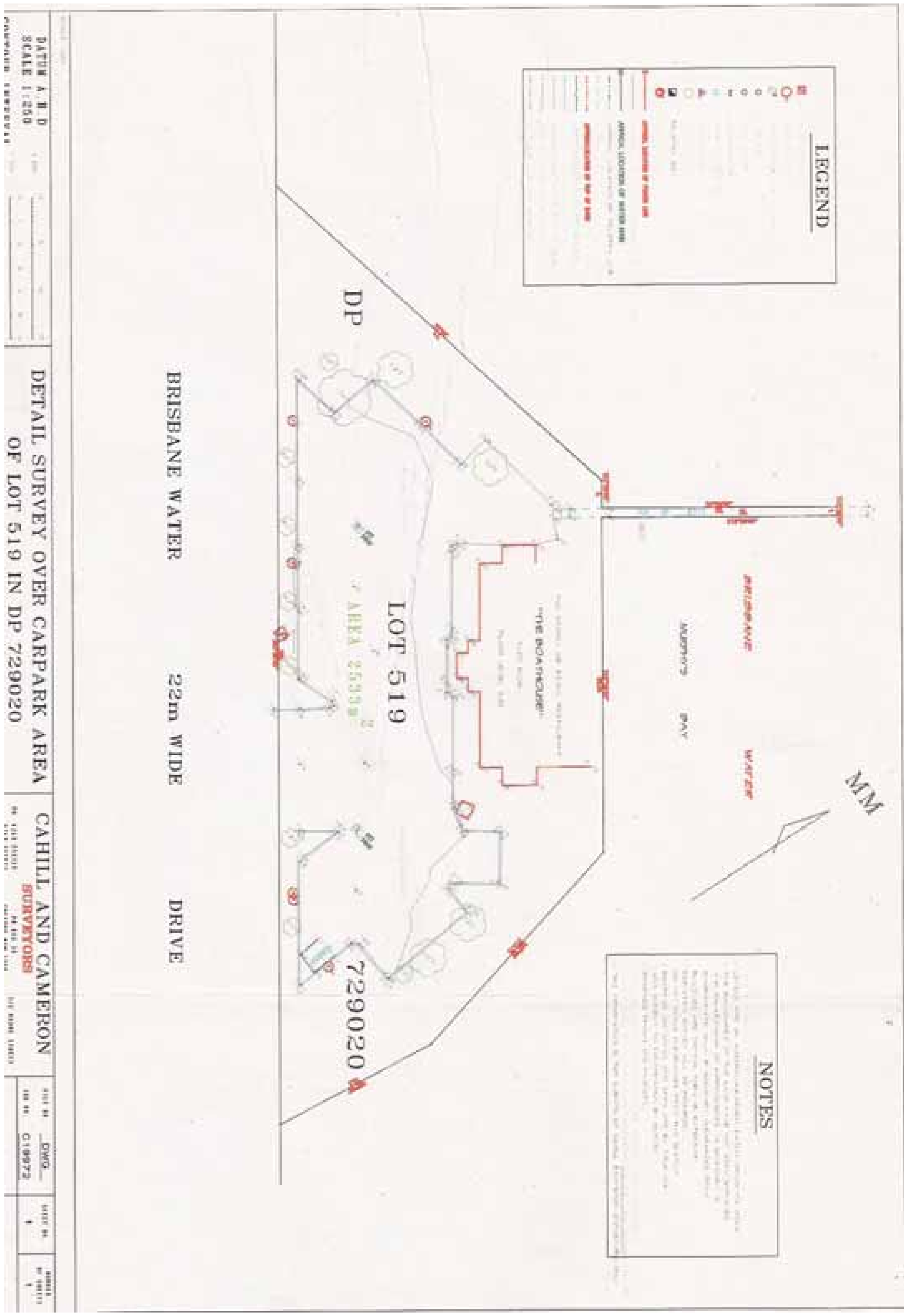
Many scientists believe that global warming of the Earth's atmosphere will lead to a rise in mean sea level. Predictions of global sea level rise due to this Greenhouse effect vary considerably.

Water Levels

Wind affects the wave, current and water level climates in Brisbane Water, as discussed in Sections 2.1 to 2.3 above. Wind data was obtained from a long term (since 1949) site at Sydney Airport. Discussion of the wind climate is provided in Section 3.2. This site provides reliable long-term data appropriate for use in design event assessment. Although this wind data would not describe wind conditions at Koolewong on an event-by-event basis, it provides realistic extremal data for wave climate analyses.

Appendix B

Site Survey


LEGEND
NOTES
BRISBANE WATER
MURPHY'S BAY
"THE BOATHOUSE"
MM
LOT 519
AREA 2539m²
DP 729020
BRISBANE WATER 22m WIDE DRIVE
DETAIL SURVEY OVER CARPARK AREA
OF LOT 519 IN DP 729020
CAHILL AND CAMERON
SURVEYORS
DATUM A.H.D
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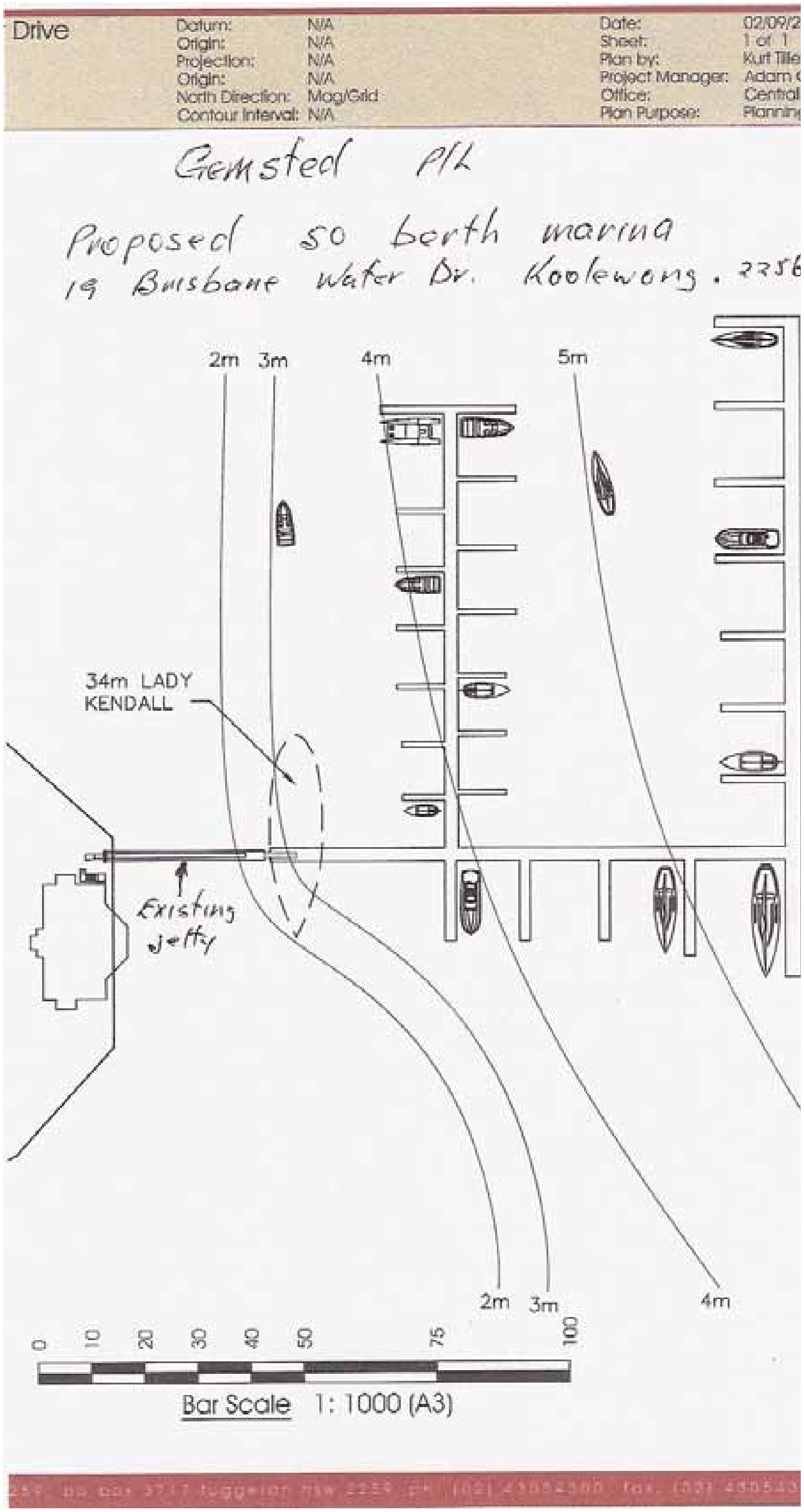
DWG
C19972

Appendix C

Proposed Marina Development Plan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Sheet: 1 of 1
Plan by: Kurt Tille
Project Manager: Adam
Office: Central
Plan Purpose: Plannin
Gemsted P/L
Proposed 50 berth marina
19 Brisbane Water Dr. Koolewong. 2256
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34m LADY KENDALL
Existing jetty
2m 3m 4m
0 10 20 30 40 50 75 100
Bar Scale 1: 1000 (A3)