Tuggerah Lakes–
The Entrance
Morphodynamic Modelling

Entrance Beach Management Investigations

Prepared for
NSW Office of Environment and Heritage

Cardno
Shaping the Future

Document Information

Prepared for	NSW Office of Environment and Heritage
Project Name	Tuggerah Lakes–The Entrance Morphodynamic Modelling
File Reference	Rep2791
Job Reference	LJ2985
Date	23 October 2013

Contact Information

Cardno (NSW/ACT) Pty Ltd
ABN 95 001 145 035
Level 9, The Forum, 203 Pacific Highway
St Leonards NSW 2065
PO Box 19 St Leonards NSW 1590
Telephone: 02 9496 7700
Facsimile: 02 9439 5170
International: +61 2 9496 7700
sydney@cardno.com.au
www.cardno.com

Executive Summary

This report has been prepared for the Office of Environment and Heritage (OEH) by Cardno to describe the outcomes of investigations undertaken to study a range of Management Options for The Entrance Beaches. It follows-on from previous investigations and reporting undertaken for the 'Development of a Combined Hydrodynamic and Morphodynamic Numerical Model of the Tuggerah Lakes, its Entrance Channels and the Adjacent Ocean Beaches', Cardno (2013).

The primary purpose of this investigation was to assess whether or not a range of feasible beach management options, such as beach nourishment, groynes and/or training walls would provide significant benefit in terms of amenity, or indeed, possible reduction in erosion hazard to North and South Entrance Beaches.

A suite of beach management options of varying scope and cost were assessed, shown in **Table ES1** below.

Table ES1 - Assessed Management Options

Option	Structure (s)	South Entrance Beach Nourishment Program
1	None	10,000m^3 per 5yrs
2	Short Groyne at South Entrance Beach	10,000m^3 per 7-10yrs
3	Long Groyne at South Entrance Beach	15,000m^3 per 7-10yrs
4	Northern Entrance Training Wall and Northern Revetment Wall	10,000m^3 per 5yrs
5	Fully Trained Entrance	15,000m^3 Initially

A description of these options, together with their projected costs, pros and cons describes presented in this report, and summarised below. In order to allow for a comprehensive comparison of the aforementioned options, a 50 years life cycle period assessment of the cost of each option has been made. The costs account for the fact that sand nourishment on South Entrance Beach will be required less often with the groyne and fully trained entrance options (though Option 4 will not affect South Entrance Beach, and as such it will still require a nourishment program in line with Option 1).

Annual maintenance costs on the structures have been estimated as a percentage of the capital investment (see **Table 8-3**). Approximate 50 years costs are calculated in terms of Net Present Value using a discount rate of 7%.

<u>Option 1 – Periodic South Entrance Beach Nourishment</u>

This option would consist of periodic sand nourishment (10,000m^3) on South Entrance Beach – performed in conjunction with Councils dredging program. It is anticipated that this nourishment would be required approximately every 5 years – depending on local storm activity. This sand nourishment has been done (circa 2005) in the past with satisfactory results – the previous occasion provided beach amenity for a number of years.

Cost:	Nourishment: $256,000 per 5 years (approx.).
Projected NPV Cost over 50 years:	$0.9 million
Pros	• Would provide enhanced beach amenity in front of the surf club and other areas of the beach.
Cons	• Requires periodic replenishment and approvals.

<u>Option 2 – Short Groyne at South Entrance Beach + Periodic South Entrance Beach Nourishment</u>

This option would consist of a short 100m long rock groyne located just south of the SLSC tower. It is estimated that this rock groyne would increase the length of time that sand is retained on South Entrance

Beach post nourishment by 2-5 years. This option would still need to be accompanied by periodic sand re-nourishment, but it would not be required as often is in Option 1.

Cost:	Construction: $2,000,000 Nourishment: $256,000 per 7-10 years (approx.).
Projected NPV Cost over 50 years:	$2.9 million
Pros	• Would provide enhanced beach amenity in front of the surf club and other areas of the beach. • Would increase the length of time that sand is retained on South Entrance Beach post nourishment by 2-5 years. • Semi-Permanent.
Cons	• Visual Impact of the structure • Construction would require 500 Truck and Dog movements → consequent road damage, congestion and social impacts • Some pedestrian obstruction and loss of beach amenity in proximity of groyne

Option 3 – Long Groyne at South Entrance Beach + Periodic South Entrance Beach Nourishment

This option would consist of 130m long rock groyne located approximately 400m north of the SLSC. It is estimated that this rock groyne would increase the length of time that sand is retained on South Entrance Beach post nourishment by 2-5 years. This option would still need to be accompanied by periodic sand re-nourishment, and this nourishment volume would need to be larger to account for the greater length of beach it would need to be placed over. This option may also result in some long term sand accumulation on South Entrance Beach because it would gradually trap sand on its southern side after each significant flood.

Cost:	Construction: $2,540,000 Nourishment: $385,000 per 7-10 years (approx.).
Projected NPV Cost over 50 years:	$3.8 million
Pros	• Would provide enhanced beach amenity in front of the surf club and other areas of the beach. • Would increase the length of time that sand is retained on South Entrance Beach post nourishment by 2-5 years. • Would result in a longer beach area than the short groyne option • Modelling shows that the long groyne would accumulate sand on its southern side in the long term.
Cons	• Visual Impact • Construction would require 600 Truck and Dog movements → consequent road damage, congestion and social impacts • Some pedestrian obstruction and loss of beach amenity in proximity of groyne

Option 4 – Northern Entrance Training Wall and Northern Revetment Wall + Periodic South Entrance Beach Nourishment

The training wall would be built to a high crest level and be of substantial design. Its intent would be to very gradually trap sand on its northern side after each significant flood. In order to prevent short circuiting or a breakout of the entrance channel through Karagi Point north of the northern training wall due to a severe flood, the northern training wall structure includes a revetment along the shoreline up to Karagi Park and then to the Entrance Bridge. As this would have minimal impact upon South Entrance Beach, nourishment would still need to be conducted there – the same nourishment program as Option 1.

Cost:	Construction of Northern Training Wall: $23,440,000

Construction of Northern Revetment Wall: $7,230,000
Nourishment: $256,000 per 5 years (approx.).

Projected NPV Cost over 50 years: $33.7 million

Pros
- Would very gradually accumulate sand on its northern side without sand nourishment (although this would be very localised to the proximity of the wall).
- Revetment would prevent erosion / shoreline recession inside the Entrance at Karagi Park.
- Would prevent dredged sand placed near Hutt Road from re-entering The Entrance.
- The South Entrance Beach nourishment would provide enhanced beach amenity in front of the surf club and other areas of the beach.

Cons
- Significant costs involved - both initial and ongoing maintenance.
- Visual Impact
- Construction would require 8,000 Truck and Dog movements → consequent road damage, congestion and social impacts
- Zone of sand accumulation would be very localised - there would be no reduction in shoreline recession and erosion hazards as far north as Hutton Road for many decades.
- Would have negative impact on the Little Tern habitat near Karagi Point.

Option 5 – Fully Trained Entrance + Initial South Entrance Beach Nourishment

The fully trained entrance would consist of the northern training wall and northern revetment wall on the northern side of the Entrance Channel, in addition to a southern training wall on the south side of the Entrance Channel. Cardno (2013) showed that training walls would not increase flood levels or flood durations in Lake Tuggerah provided that the walls were spaced 150m apart or wider. Additionally Cardno (2013) showed that the training walls would not impact upon the flushing of the lake system, and thus would not be expected to affect water quality within the lake. The training walls would be of substantial design, as they would be required to withstand considerable wave action and flood currents.

Apart from formalizing the entrance area, the training walls would be intended to very gradually trap some sand south and north of the southern and northern training walls respectively after severe (rare) lake flood events, as sand is transported back onshore by swell wave activity. This alternative could be accompanied by 15,000m^3 of initial nourishment sand to bring forward the expected long term beach amenity improvement on South Entrance Beach.

Cost:
Construction of Southern Training Wall: $12,830,000
Construction of Northern Training Wall: $23,440,000
Construction of Northern Revetment Wall: $7,230,000
Nourishment: $385,000 (initially).

Projected NPV Cost over 50 years: $46.9 million

Pros
- Would very gradually accumulate sand on its northern side without sand nourishment (although this would be very localised to the proximity of the wall). Sand dredged from The Entrance and placed on North Entrance Beach would not return to the entrance.
- Revetment would prevent erosion / shoreline recession inside the Entrance at Karagi Park.
- Would prevent dredged sand placed near Hutt Road from re-entering The Entrance.
- Modelling shows that the southern training wall would accumulate sand on its

southern side in the long term.

- Would increase the length of time that sand is retained on South Entrance Beach post nourishment by 5-10 years.

Cons

- Significant costs involved - both initial and ongoing maintenance.
- Visual Impact
- Construction would require 15,400 Truck and Dog movements → consequent road damage, congestion and social impacts.
- Zone of sand accumulation north of the northern training wall would be very localised - there would be no reduction in shoreline recession and erosion hazards as far north as Hutton Road for many decades.
- Would have negative impact on the Little Tern habitat near Karagi Point.
- Loss of beach width (long term) along the southern bank of the entrance channel (inside the walls along Marine Parade).

Glossary

Australian Height Datum (AHD)		A common national plane of level corresponding approximately to mean sea level.
Amenity		Those features of an estuary/beach that foster its use for various purposes, e.g. Clear water and sandy beaches make beach-side recreation attractive.
ARI		Average Recurrence Interval
Bed Load		That portion of the total sediment load that flowing water moves along the bed by the rolling or saltating of sediment particles.
Calibration		The process by which the results of a computer model are brought to agreement with observed data.
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.
CD		Chart Datum, common datum for navigation charts - 0.92m below AHD in the Sydney coastal region. Typically Lowest Astronomical Tide.
Discharge		The rate of flow of water measured in terms of volume per unit 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 flowing.
Diurnal		A daily variation, as in day and night.
Ebb Tide		The outgoing tidal movement of water within an estuary.
Eddies		Large, approximately circular, swirling movements of water, often metres or tens of metres across. Eddies are caused by shear between the flow and a boundary or by flow separation from a boundary.
Estuarine Processes		Those 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.
Foreshore		The area of shore between low and high tide marks and land adjacent thereto.
Geomorphology		The study of the origin, characteristics and development of land forms.
H_s (Significant Wave Height)		H_s may be defined as the average of the highest 1/3 of wave heights in a wave record ($H_{1/3}$), or from the zeroth spectral moment (H_{mo}), though there is a difference of about 5 to 8%.
Intertidal		Pertaining to those areas of land covered by water at high tide, but exposed at low tide, e.g. intertidal habitat.
Littoral Zone		An area of the coastline in which sediment movement by wave, current and wind action is prevalent.
Littoral Drift Processes		Wave, current and wind processes that facilitate the transport of water and sediments along a shoreline.
Marine Sediments		Sediments in sea and estuarine areas that have a marine origin.
Mathematical/ Computer Models		The mathematical representation of the physical processes involved in runoff, stream flow and estuarine/sea flows. 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 wave and current processes.
MHL		Manly Hydraulics Laboratory
MSL		Mean Sea Level

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).
NSW		New South Wales
Numerical Model		A mathematical representation of a physical, chemical or biological process of interest. Computers are often required to solve the underlying equations.
Phase Lag		Difference in time between the occurrence between high (or low water) and maximum flood (or ebb) velocity at some point in an estuary or sea area.
Salinity		The total mass of dissolved salts per unit mass of water. Seawater has a salinity of about 35g/kg or 35 parts per thousand.
Saltation		The movement of sediment particles along the bed of a water body in a series of 'hops' or 'jumps'. Turbulent fluctuations near the bed lift sediment particles off the bed and into the flow where they are carried a short distance before falling back to the bed.
Sediment Load		The quantity of sediment moved past a particular cross-section in a specified time by estuarine flow.
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.
Slack Water		The period of still water before the flood tide begins to ebb (high water slack) or the ebb tide begins to flood (low water slack).
SLSC		Surf Life Saving Club
Spring Tides		Tides with the greatest range in a monthly cycle, which occur when the sun, moon and earth are in alignment (the gravitational effects of the moon and sun act in concert on the ocean)
SS		Suspended Solids
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 and piling it up against the coast.
Suspended Sediment Load		That portion of the total sediment load held in suspension by turbulent velocity fluctuations and transported by flowing water.
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 Lag		The delay between the state of the tide at the estuary mouth (e.g. high water slack) and the same state of tide at an upstream location.
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 Propagation		The movement of the tidal wave into and out of an estuary.
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.
Training Walls		Walls constructed at the entrances of estuaries to improve navigability by providing a persistently open entrance.
Turbidity		A measure of the ability of water to absorb light.

T_z (Zero Crossing Period)		The average period of waves in a train of waves observed at a location.
Velocity Shear		The differential movement of neighbouring parcels of water brought about by frictional resistance within the flow, or at a boundary. Velocity shear causes dispersive mixing, the greater the shear (velocity gradient), the greater the mixing.
Wind Shear		The stress exerted on the water's surface by wind blowing over the water. Wind shear causes the water to pile up against downwind shores and generates secondary currents.

* A number of definitions have been derived from the Estuary Management Manual (1992).

Table of Contents

Appendices

Figures

1 Introduction

1.1 Background

This report has been prepared for the Office of Environment and Heritage (OEH) by Cardno to describe studies undertaken to investigate a range of Management Options for The Entrance Beaches. It follows-on from previous investigations and reporting undertaken for the 'Development of a Combined Hydrodynamic and Morphodynamic Numerical Model of the Tuggerah Lakes, its Entrance Channels and the Adjacent Ocean Beaches', Cardno (2013). It describes the background, data, study approach and outcomes of this supplementary investigation. The primary purpose of this investigation was to assess whether or not a range of feasible beach management options, such as beach nourishment, groynes and/or the training walls investigated in Cardno (2013) would provide significant benefit in terms of amenity, or indeed, possible reduction in erosion hazard. The study has also addressed the matter of safe navigation at The Entrance.

The Tuggerah Lakes system on the NSW Central Coast comprises a series of three inter-connected shallow coastal lagoons (Tuggerah, Budgewoi and Munmorah) that have a weak and intermittent connection to the ocean at The Entrance, see **Figure 1.1**.

The management of the entrance and adjacent beaches has been a locally controversial issue for decades. Some community members consider that training walls would improve water quality and resolve widespread aesthetic issues. For this reason, the option of entrance training was thoroughly considered at all stages of the preparation of the Tuggerah Lakes Estuary Management Plan (EMP) prepared by WSC (2010).

More recently, Council received a report entitled "Entrance Dynamics and Beach Conditions at The Entrance and North Entrance Beaches" (Umwelt, 2011), including coastal processes investigations undertaken by SMEC. The study was commissioned to examine the sediment budget linkages between the entrance channels and the adjacent beaches and to identify options to manage the sedimentary processes to minimise coastal erosion hazards. Umwelt studied the interaction between entrance management strategies and the condition of the adjacent beaches and the estuary. It cautioned that training walls "... are more likely to have a detrimental transformational impact on the hydrodynamics and fragile ecology of the Tuggerah Lakes." It did, however, recommend that Council invest in a 3D hydrodynamic and sediment transport model of the entrance area to test a range of management strategies under present and sea level rise scenarios. That work is reported in Cardno (2013).

Cardno (2013) advise that sand transported from The Entrance during severe flood events would be carried seaward of the ends of the training walls investigated in that study. Following storm abatement, sand is transported shoreward by swell waves. Whereas in the present case it would all eventually end up in the entrance dynamics, some of it would now end up on the beach south of the southern training wall and some of it would end up north of the proposed northern training wall. Hence over some decades, each flood event and the associated post-flood onshore sediment transport would gradually 'ratchet' sand from the entrance onto the South and North Entrance Beaches. Although this process has a negligible effect on water quality and flood levels in the lakes (because of the very large volumes of sand held in the entrance area west to the drop-over), it may have some benefit on the beaches nearby. That outcome prompted this investigation, which was widened to consider other options, their costs and an assessment of navigation issues.

1.2 Entrance Beaches

The Entrance Beaches form a dynamic region at the junction of Tuggerah Lake with the Tasman Sea. They provide valuable recreational amenity as well as some protection from the coastal hazards of erosion and inundation.

During severe catchment floods in the lakes, the northern and southern ends of South and North Entrance Beaches, respectively, may be eroded. Entrance and shoreline sand is transported seaward. Following storm abatement, onshore propagating swell transports this sand shoreward – much into The Entrance itself, but some also onto the two beach areas.

Cardno (2013) describes these processes and investigated the likely effect of a range of training wall options on flood levels and tidal exchange in the lakes. Those results demonstrated that although training walls would not change tidal flushing; the walls themselves might gradually trap some onshore propagating post-flood sand on the two beach areas. Over time there would be a gradual reduction of sand within the lower estuarine entrance and a gradual accumulation of sand on the beaches. These outcomes were perceived to be advantageous to:-

- South Entrance Beach because it could gradually improve the amenity of that beach, notably at The Entrance Surf Life Saving Club (SLSC) building, where the sandy beach area is often quite narrow, exposing bed-rock, thereby discouraging recreational activities.
- North Entrance Beach, by gradually widening it and thereby reducing the erosion hazard at its southern end. Previous studies (Patterson Britton (1988) and Cardno (2013)) have shown that sediment transport on the southern end of North Entrance Beach is typically southward towards The Entrance. Beyond a null-point further north, sediment transport is northward. This sediment transport structure currently transports onshore moving post-flood sand back towards The Entrance where it gradually accumulates in a flood-tide delta. Construction of a northern training wall would trap most of this southward moving sand against the northern side of this training wall. Over time this accumulation of sand from the entrance would gradually widen North Entrance Beach, thereby reducing the erosion hazard over an extended period of time.

These outcomes have prompted OEH to engage Cardno to investigate these entrance beach processes further, together with more general beach amenity investigations.

1.3 Conventions

Standard directional conventions have been adopted, that is:-

- Winds and waves – coming from
- Currents and sediment transport – flowing towards

All levels are to AHD, unless specified otherwise.

2 General Scope of Work

As well as the coastal processes investigations introduced above, Cardno were requested to undertake costing of a range of works that might be considered viable, as well as a navigation study; which was undertaken by Captain Charles Weston. That work is presented in Weston (2013) – included as **Appendix G**. Matters to be addressed for the two beach areas are outlined concisely below.

South Entrance Beach

Define the deficiencies of the present sand/rock shoreline:- history of storm-caused denudation/natural restoration – use aerial photography; club records.

Evaluate possible solutions to ensure sufficient sand reserves for surf lifesaving activities and South Entrance Beach amenity, such as:-

- Sand Nourishment (examining storm bite effects).
- Groyne(s) and nourishment/no nourishment
- Southern training wall (as part of a fully trained entrance)

North Entrance Beach

Investigate the:-

- Northern training wall and its long term effect on North Entrance Beach amenity.

3 Model Systems

A range of model systems was applied to the entrance beach investigations at The Entrance. They are described below.

3.1 General

Investigations of water levels, currents and morphological processes to describe current fields and sediment transport processes required the application of a high level model capable of simulating a range of processes – wave and tidal forcing and then morphological modelling, with some confidence.

These simulations were undertaken using the Delft3D modelling system. The post-processing capabilities of this model system have been applied to stored model results from Cardno (2013) to determine changes in sediment volume north and south of the modelled training walls for the range of flood events investigated in that study. Additionally, wave driven current and sediment transport vectors on North Entrance Beach were prepared from that stored data.

The Delft3D modelling system has been applied to current and wave investigations at many international locations, as well as within Australia by Cardno Coastal and Ocean – Port Botany (Sydney), Cairns Navy Base (Queensland), Gulf of Papua, Pittwater 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.

The model domain developed for The Tuggerah lakes investigation is shown in **Figure 3.1**. The model extends offshore to a depth of approximately 70m AHD and water level and wave boundaries offshore. Sub-model areas have been developed for wave and sediment analyses in The Entrance area itself to better resolve the hydrodynamics and sediment transport processes, which are quite intense there.

The model has a curvilinear grid with variable resolution. Offshore areas have a grid resolution in the order of 100m x 100m, while areas near the entrance itself, where steep hydrodynamic gradients exist, have horizontal grid cells in the order of 8m x 8m.

3.2 Hydrodynamic Numerical Scheme

The Delft3D FLOW module is based on the robust numerical finite-difference scheme developed by G. S. Stelling (1984) at the Delft Technical University in The Netherlands. Since its inception the Stelling Scheme has undergone considerable development and review by Stelling and others.

The Delft3D Stelling Scheme arranges modelled variables on a horizontal staggered Arakawa C-grid. The water level points (pressure points) are designated in the centre of a continuity cell and the velocity components are perpendicular to the grid cell faces. Finite difference staggered grids have several advantages including: -

- Boundary conditions can be implemented in the scheme easily
- It is possible to use a smaller number of discrete state variables in comparison with discretisations on non-staggered grids to obtain the same accuracy
- Staggered grids minimise spatial oscillations in the water levels.

Delft3D can be operated in 2D (vertically averaged) or 3D mode. In 3D mode, the model uses the σ-coordinate system first introduced by N Phillips in 1957 for atmospheric models. The σ-coordinate system is a variable layer-thickness modelling system, meaning that over the entire computational area, irrespective of the local water depth, the number of layers is constant. As a result, a smooth representation of the bathymetry is obtained. Also, as opposed to fixed vertical grid size 3D models, the full definition of the 3D layering system is maintained into shallow water and until the computational point is dried.

Horizontal solution is undertaken using the Alternating Direction Implicit (ADI) method of Leendertse for shallow water equations. In the vertical direction (in 3D mode) a fully implicit time integration method is also applied.

Vertical turbulence closure in Delft3D is based on the eddy viscosity concept.

The model was applied in 2D mode for these investigations because non-cohesive sediment transport algorithms are based on depth averaged currents.

3.2.1 Wetting and Drying of Intertidal Flats

Many near shore areas include shallow inter-tidal regions; consequently Delft3D includes a robust and efficient wetting and drying algorithm to handle this process.

3.2.2 Conservation of Mass

Problems with conservation of mass, such as a ‘leaking mesh’, do not occur within the Delft3D system.

However, whilst the Delft3D scheme is unconditionally stable, inexperienced use of Delft3D, as with most modelling packages, can result in potential mass imbalances.

Potential causes of mass imbalance and other inaccuracies include: -

- Inappropriately large setting of the wet/dry algorithm and unrefined inter-tidal grid definition
- Inappropriate bathymetric and boundary definition causing steep gradients
- Inappropriate time step selection (i.e. lack of observation of the scheme’s allowable Courant Number condition) for simulation

3.3 Sediment Transport Model

The past ten years have seen the development of a hydrodynamic module that is capable of simultaneous sediment transport modelling and morphological updating while performing the hydrodynamic simulation. This “online sediment version” works by treating sediment as another constituent (in addition to salinity and heat) allowing it to be calculated in two and three dimensions, and subsequently feeding the density effects of the sediment back into the flow simulation. This version includes linked hydrodynamics, wave processes, sediment transport and morphological changes.

Interaction with the bed for sand fractions is computed and is based upon the sediment pick-up functions of Leo van Rijn; bed-load transport is included. For mud fractions the widely recognised sediment flux expressions of Partheniades and Krone are used (van Rijn, 1993).

This version also retains the ability to include fixed, non-erodible areas. To bridge the gap between morphological and hydrodynamic time-scales a morphological acceleration factor can be used. The inclusion of improved formulations that describe the effects of waves on the three-dimensional flow pattern is significant. These features make the version particularly suitable for modelling morphological changes in areas with complex three-dimensional flow patterns, such as river bends, dredged trenches, coastal regions and estuaries, including lateral erosion. In sandy areas, 2D modelling is applied because the algorithms have been developed in terms of depth averaged flow.

It is often necessary to undertake a range of siltation simulations for a range of combined wave and current conditions. Generally, the boundaries of morphological models are set where there is likely to be no seabed change.

3.4 Wave Model System

Wave modelling for this study was based on the SWAN wave model, which is integrated into the Delft3D modelling system. SWAN was developed at the Delft Technical University and includes wind input, (local sea cases), combined sea and swell, offshore wave parameters (swell cases), refraction, shoaling, non-linear wave-wave interaction, a full directional spectral description of wave propagation, bed friction, white capping, currents and wave breaking.

Wave modelling was undertaken for two purposes, namely: -

- To describe the propagation of ocean waves into the estuarine entrance where they affect sediment transport and deposition
- To select sites along North and South Entrance Beaches where longshore transport is an important process. The transformed wave data forms an input to the LITPACK model.

The model layouts prepared for this study, **Figure 3.1c**, ensured that the development of wave conditions arising from the complex bathymetry and variable wind cases were included in the wave parameter descriptions. Thirteen years of recorded directional Waverider buoy data from the Manly Hydraulics Laboratory Long Reef site were transferred from offshore to inshore sites, mainly South Entrance Beach. That transferred wave data was then applied in a range of coastal process modelling tasks.

3.5 LITPACK Coastal Processes Modelling System

This modelling system has been developed by the Danish Hydraulics Institute. It is used internationally for assessment of coastal processes.

LITPACK includes a number of modules. One of these, LITDRIFT, computes longshore sediment transport from a time-series of wave parameters. Natural beach profiles, graded sediments, currents, wind and local roughness are included. Generally the highest transport rate occurs in the breaking wave zone. LITDRIFT was applied at The Entrance beaches with time-series of near shore ocean wave parameters generated using the SWAN model. LITDRIFT output includes the shore normal variation of longshore transport magnitude.

LITLINE is another module of LITPACK and is used to determine changes to a shoreline over a period of time using spatially and temporally varying longshore transport. It includes coastal structures such as groynes and revetments (seawalls). In this case proposed groynes were included in a model of the South Entrance Beach shoreline. Note that groynes in LITLINE can be specified to have an apparent length; that is, sand bypassing can commence before sand builds-up fully on the updrift side to the full length of a groyne. This process is important because it more closely describes what happens naturally than the alternative of delayed bypassing. The extent of bypassing depends on this apparent length and the shore normal profile of longshore sediment transport in different wave conditions. The length of a groyne is also dynamic in terms of the shore normal profile and reduces as a beach builds-out against the up-drift side of it. LITLINE includes realistic shore normal beach profiles, the active depth limitation and dune height in computation of shoreline changes.

LITDRIFT and LITLINE use the basic Engelund and Fredsoe (1976) transport formulation which includes combined wave and current motion as well as bed and suspended sediment loads. It takes account of the threshold shear stress for initiation of sediment transport through the Shields Parameter.

LITDRIFT and LITLINE were applied to analyses of annual longshore transport variation along South Entrance Beach. LITLINE was applied as one method of analysing the long term shoreline effects of groyne construction.

3.6 SBEACH

SBEACH was developed by the U.S. Army Corps of Engineers (USACE) to investigate storm induced profile response on fine to medium grain sand beaches. It is an empirically based model that includes wave shoaling, refraction, breaking, set-up and run-up. The model can simulate a temporally varying wave breaking-point, which produces offshore bar migration. The model has been widely applied at sites all over the world and has demonstrated reasonable levels of calibration. A feature of SBEACH is that underlying rock layers can be specified in the model. SBEACH has been used to describe the changes in beach amenity (width at 1m AHD in this report) arising from different beach nourishment volumes on South Entrance Beach and ocean storms.

4 Supporting Data

A range of data items was required to set up, calibrate and operate the models applied to this investigation as well as investigations of historical beach changes. They are described below.

4.1 Bathymetric Data

Following/during the inception meeting on 13 December 2011, Cardno have received the following data from OEH.

- 2011 Entrance Hydrosurvey
- 2011 Bathymetric LiDAR data of the Entrance and Open Coast
- 2008 Bathymetric LiDAR data of the Entrance and Open Coast
- 1979 and 1975 Tuggerah Lakes Surveys
- 1995 Single Beam offshore bathymetric data to 60m.

Bathymetric data describing the lakes, The Entrance area and the near shore and offshore seabed areas were required for model set up. Additionally, indicative spatially varying seabed and sub-surface bed rock information for the entrance area was required because entrance scour during flood events is limited in depth by that natural rock structure.

Bed rock contour information was obtained from Public Works Department data presented in Patterson Britton Partners (1988). Cardno received the bathymetric data described above from OEH. A digital elevation model (DEM) of the lake and shorelines was prepared by combining these bathymetric data sets, with the most recent data taking precedence. The adopted DEM vertical datum was AHD.

This information was applied mainly in Cardno (2013), but was used in this study also to assess volume changes following simulated flood events. The purpose was to calculate the volumes of sand that would be transported onshore to the North and South Entrance Beaches following a significant flood event.

4.2 Wave Data

Wave data (height, period and direction parameters) for the period from 1992 to 2011 from the offshore Long Reef directional Waverider buoy was provided by Manly Hydraulics Laboratory (MHL). These data were transferred inshore to South and North Entrance Beaches.

4.3 Water Level Data

MHL also provided recorded water level data for Long Jetty and Toukley in Tuggerah Lake. Additionally, time-series water level data from MHL's Middle Harbour tide gauge was provided and used as one basis for forcing the ocean boundary of the hydrodynamic model.

This data was used to calibrate the model systems, see Cardno (2013).

Recorded water level data (from MHL) at Middle Harbour, NSW was used to describe the offshore wave data period 1992 to 2013.

4.4 Sediment Data

Sediment data was taken by Cardno (two samples, a previous study) at a site on the North Entrance shoreline of The Entrance, demonstrating a D_{50} particle size of 0.35mm. Particle sizes of 0.25 and

0.35mm were tested in the entrance morphological modelling to test the sensitivity of those model results to particle size.

5 Beach Characteristics

Although both beaches are connected via estuarine hydro-sedimentological processes to The Entrance, and hence have some inter-action, they are sufficiently different for them to be addressed separately from the points of view of amenity and coastal hazard, as well as the details of beach sediment transport on them and the potential works that might produce a beneficial outcome.

5.1 South Entrance beach

5.1.1 Site Visit and Aerial Photography Analysis

A site visit was undertaken by OEH and Cardno on 9 April 2013 at 1400, mainly to The Entrance itself and to South Entrance Beach. The back-beach area is protected by a rock revetment that extends from, and including, the club-house, north along the shoreline into The Entrance – indicated on **Figure 5.1**. The area behind this revetment is well vegetated and steep. There is a large stormwater drain that discharges to the beach about 25m north of the SLSC, and which cuts a gully across the beach during periods of heavy rain. A large rock headwall is exposed also.

Discussions were held with Mr Glenn Clarke, President of The Entrance SLSC. He advised that a few years ago, circa 2004, Wyong Shire Council had discharged some sand, dredged as part of their routine entrance dredging work within The Entrance, onto the beach near/in front of the SLSC building. It is understood that about 30,000m^3 were placed at that time, (Worley Parsons, 2009), and that some benefit was achieved that lasted, with diminishing benefit, for about 3 to 4 years. There are no details of placement location on the beach profile, but **Figure 5.2a**, June 2005, shows that at the time of the aerial photograph, the sand was well distributed across the whole beach and provided an historically good amenity. Sand slurry was discharged to the beach from a dredge pipeline. A recent request from the SLSC for some sand from the dredging campaign undertaken in late 2012 by Council to be placed on South Entrance Beach was refused by Council, understood to be on the basis of cost.

Mr Clarke advised that the volume of sand on the beach ‘comes and goes’, but the beach is not suitable as a surfing amenity when it is in an eroded state because the underlying bed rock becomes exposed, thereby leading to possible injuries during water sport activities. There is very little sand on the shoreline to the south of the SLSC building and a long pipe crosses the beach, acting as a potential groyne. However, there is very little sand accumulated on its up-drift (southern) side. Hence this beach area is affected by offshore transport caused by storms and the post-storm beach re-building process, rather than longshore transport. This is supported by the fact that it has remained generally where it is for at least many decades and longshore transport has not caused any permanent loss of sand. The substantial cross-beach pipeline that lies to the south of the SLSC would act as a significant longshore transport interceptor, if that process were significant. On the other hand, history has shown that a large volume of re-nourishment sand cannot be maintained on this beach. This is one reason that Council is reluctant to undertake re-nourishment of this beach as part of the periodic entrance dredging program.

Figures 5.2a-e show aerial photographs of the beach on the five dates provided by Google Earth. They are ortho-rectified and to the same scale, but of different quality. They show the beach in various stages of amenity. The most interesting feature on **Figure 5.2d**, which is very clear, is that the plan alignments of the low, mid and high tide lines are quite consistent with bearings of about 126,117,112° True North (TN) and parallel with the near shore swell wave crests.

5.1.2 Photogrammetric Data Analysis

OEH also provided several years of beach profile data, based on photogrammetric analyses of historical aerial photographs, at selected sections along the beach, see **Appendix A** for the locations of these profiles. That data has been analysed to describe the plan locations of the 1m AHD contour on South Entrance Beach for the dates of the aerial photography. This contour was selected because it describes beach width at very high tide.

These lines are shown on **Figure 5.3** and were extracted from photogrammetric profiles in Blocks 1 and 2 as displayed in **Appendix A. Figure 5.3** also illustrates the major shoreline erosion caused by the May-June storms of 1974 and hence the need for the structures to butt up against the back-beach revetment. **Figure 5.7** provides weighted mean wave directions determined from transformation of thirteen years of Long Reef wave data to selected near shore locations. It is clear that the beach plan alignment is basically at 90° to these wave directions. **Appendix B** describes the computation of these wave parameters. **Figure 5.2d** also shows that the plan alignment of the beach is generally parallel with incident wave crests at a range of levels on the beach face.

This beach alignment could not be maintained if there was no effective obstruction to longshore transport at The Entrance. This obstruction is most likely caused by the combined effects of the bed rock structure and the near shore seabed contours that refract the waves that pass close by to the north and south of this rock outcrop. The rock reef that lies seaward and south of The Entrance has formed a dynamic tombolo in its lee that prevents sand from being lost from South Entrance Beach by longshore transport. However, its effect as an offshore breakwater is limited by the level of the rock, its areal extent and its seaward location, and is different at high and low tide. Hence there is a limit to the volume of sand that can be kept on this beach, on average. It will vary as a result of storms that cause offshore transport and temporary greater exposure of the bedrock. It is likely that if the beach were to be widened through beach re-nourishment alone, then much of that sand would move from the beach into The Entrance. The northern side of this tombolo structure will be affected also by tidal and flood flows from The Entrance.

Figure 5.3 shows that the greatest shoreline recession was caused by the storm of May 1974, with perhaps some influence from the June 1974 storm that followed. The plan form of the tombolo moved into the entrance, but maintained much of its shape. Very little rain occurred during that storm, yet the water level in Tuggerah Lake rose to 1.2m AHD (Lawson and Treloar, 1994). This was caused by wave radiation stresses that caused a continual inflow of seawater that also transported some of the sand from the tombolo into the entrance. It has since reformed to its ‘normal’ configuration.

Photogrammetric profiles from Block 1 were also used to generate an ensemble-average shore-normal beach profile for South Entrance Beach. Photogrammetric data (dry area) was then combined with the LiDAR survey data (wet area) to create a profile covering elevations from -10 to 10 m AHD. **Figure 5.4** shows the final profile used as “Existing Profile” in the longshore transport and storm bite modelling. This profile includes ‘rock’ and sand areas, the break-point between the rock and sand regions on the natural beach being estimated from longshore sediment transport investigations.

5.1.3 Wave Modelling

SWAN was used to transfer Long Reef offshore wave data to the inshore locations along the coast that are shown on **Figure 5.4**. A total of 972 SWAN runs (4 wave heights * 9 wave periods * 9 wave directions * 3 water levels) was modelled to generate the interpolation tables required for an accurate estimate of the inshore wave parameters. This complex modelling setup was required to overcome the various shallow bathymetric features that greatly influence wave breaking, bed friction, refraction and also diffraction at this site.

Wave maps were prepared to show the influence of water level on wave directions along the southern coast, and, more particularly in the tombolo area. **Figures 5.5a** and **5.5b** show the wave pattern for an offshore wave direction of 135°TN, peak period of 11.2s, significant height of 5m and a high water level of 0.9m AHD. **Figure 5.5a** shows the waves turning towards the shoreline direction except, where some important breaking and diffraction patterns are observed around 500m offshore of South Entrance Beach. This pattern is caused by a bomborah that is most likely constituted of rocks. **Figure 5.5b** does not show any significant wave turning in the tombolo area at high water level. **Figures 5.6a** and **5.6b** show wave propagation directions for similar offshore wave conditions, but at a low water level of -0.5m AHD. **Figure 5.6a** shows a more significant breaking/diffraction pattern at low water than at high water. **Figure 5.6b** presents the wave pattern of a tombolo structure created by the rocks that are just offshore of the 0mA HD contour (in grey) at the southern point of The Entrance. Waves are breaking and diffracting around the rocks causing the sand to accumulate behind and south of the rocks and form the typical shape of South Entrance Beach.

The accuracy of the inshore wave heights/period/direction is important for use in longshore transport modelling. Wave parameter time-series were also analysed to evaluate the effective wave heights and weighted mean wave directions at the output locations along the -4m and -10m AHD bathymetric contours. The computation of these weighted mean wave directions is described in **Appendix B**. **Figure 5.8** describes an important change in direction between the wave directions at -4m and -10m AHD that is hard to replicate in a 1D longshore sediment transport model.

5.2 North Entrance beach

North Entrance Beach is different in many ways from South Entrance Beach. Its southern extent (Karagi Point) is affected more by catchment flooding and not by major rock formations to the same extent, and is a very long beach. The beach at North Entrance is always sufficiently wide to provide amenity, but is subject to severe storm erosion that exposes properties to shoreline recession and potential erosion hazards, especially with projected sea level rises.

5.2.1 Sediment Transport Structure

It is known that there is a ‘null point’ in the southern region of North Entrance beach, south of which longshore transport is southward, and north of which it is northward. This null point is not fixed spatially, but varies with offshore wave direction and the plan alignment of the southern end of North Entrance Beach.

Cardno (2013) conducted a series of coupled hydrodynamic and wave simulations for various catchment storm events and training wall scenarios. **Figures 5.8a-d** show sediment transport vectors from modelling scenarios undertaken for an existing and trained (150m wide) entrance case under ambient catchment and wave conditions. These plots show that while the exact location of the null point moves depending on offshore met-ocean conditions, it is generally in the region depicted in the figures in red, to the south of Hutton Road (additionally see **Figure 5.9**).

The reason for the formation of this null point is probably threefold, as described below:-

- The rock reef that causes the tombolo formation at the northern end of South Entrance Beach will also cause some refraction of waves that propagate across its northern shoulder. That process will cause a southward deflection to waves for some distance north, after which there will only be ‘normal’ seabed refraction. Any southward deflection of wave propagation direction will have a tendency to reduce the rate of northward sediment transport, or cause southward transport. This refractive process will also cause a reduction in wave heights on southern North Entrance Beach, see below.
- The aforementioned rock reef results in reduced wave energy at the southern end of Karagi Point, when compared with the shoreline slightly to the north, for example, north of Hutton Road. This reduction in wave energy results in smaller wave heights and less wave set-up near the southern end of Karagi Point – as demonstrated in the SWAN model wave height map of **Figure 5.6a**. This model can only show wave height and wave set-up, but not the resulting current structure. This reduction in wave set-up near Karagi Point and the resulting localised water level gradient causes a net southerly transport, regardless of the offshore wave direction. The current vectors and sediment transport vectors developed by the coupled wave, hydrodynamic and morphological modelling demonstrate this process.
- Flood tide flows transport sand from the southernmost tip of Karagi Point into the entrance. Following a flood event this process begins some 100m to 200m north of the common southern position of Karagi Point. Hence it causes the plan alignment of the beach to be rotated clock-wise as the process develops, which, given the mean direction of ocean wave propagation, causes southward sediment transport that feeds the estuarine tidal sand influx process. The rate of estuarine sand influx reduces as the entrance fills with sand and current speeds reduce.

Figure 5.7 describes the weighted mean wave direction at the -4m AHD depth contour along North Entrance Beach. The computation of this parameter is based on transferred wave parameter time-series prepared from the Long Reef directional Waverider buoy using the SWAN wave model. The computation of this statistical descriptor is described in **Appendix B**. The rate and direction of longshore sediment transport depends on the difference between this parameter and the shore normal direction, in principle. However, because it is a mean direction, there will be times when sediment transport occurs in the other direction. Within enclosed embayments, the plan beach alignment tends to be normal to the weighted mean wave direction.

5.2.2 Entrance Dredging Works

Wyong Shire Council undertakes routine dredging of channels upstream of the entrance sill to The Entrance Bridge. This work is undertaken using Council's own dredge, with the spoil being placed most commonly on North Entrance Beach; near to and south of Hutton Road (see **Figure 5.9**). A system of pipes is used to transport the dredged sand as slurry and has been installed in the region shown on **Figure 5.9**. This discharge point is generally placed north of the null point region. Hence this dredged sand will be distributed potentially along the whole of the shoreline to Norah Head. However, it is not separated from the long term entrance morphological processes at present. **Appendix C** provides a description of the dredging plan for The Entrance (Worley Parsons 2009).

The locations of the spoil placement were observed by Cardno on a site visit on 6 June 2013. Photographs from that site visit are shown in **Appendix D**. These photographs show clear sand placement on North Entrance beach up to Hutton Road, as well as inside the entrance adjacent to Karagi Park.

6 Proposed Coastal Structures and Nourishment Programs

Given the beach characteristics described in **Section 5** above, several different coastal structures have been proposed to provide solutions to the issues facing South and North Entrance Beaches. These are (see **Figure 6.1**):-

- Small groyne of 100m length at 2m AHD crest level
- Long groyne of 130m length at 2m AHD crest level
- A Northern Training Wall, including a revetment wall running north to The Entrance Bridge.
- A fully trained entrance. Training walls of approx. 230m long at 4.5 to 5m AHD crest level. Spacing would need to be more then 150m apart as per Cardno (2013).

The groynes and fully trained entrance options would provide some blocking of sand drift into The Entrance following a re-nourishment program on South Entrance Beach, or post-flood onshore transport onto the northern end of the beach. All lengths are measured at 0m AHD from the existing back-beach revetment. These structures need to butt up to this revetment in order to prevent future out-flanking, as might occur in a storm similar to the May/June storms of 1974. They would also help to increase beach width by reducing the length of the South Entrance Beach over which a specific volume of re-nourishment sand would be redistributed. Only the long groyne and trained entrance options would be likely to accumulate sand without nourishment, mainly from onshore transport following a severe (rare) flood event. The length of the short groyne could be increased but there is a risk that it could affect surfing in this area of South Entrance Beach. The long groyne and training wall options are in a more rocky area that would be less likely to impede upon local surfing amenity.

6.1 South Beach Short Groyne Structure

This option would consist of a 100m long groyne located just to the south of the SLSC tower. The landward end of the structure would begin at the existing revetment wall, and from there it would extend seaward out to approximately 0.6mAHD (the approximate mean low water spring level).

The intent of the short groyne would be to increase the length of time that sand is retained on South Entrance Beach post beach nourishment by several years, meaning that sand re-nourishment would be required less often than would be the case without such a structure. Essentially, it would result in a wider beach for longer period of time post nourishment. As the crest level of the structure would be 2mAHD, the landward end of the structure would be buried in the back beach dune system, limiting its impediment upon pedestrian traffic in the back beach region.

The cons of such a structure include the impact of its construction on the community (see **Section 9**), as well as the visual impact of the structure itself. Additionally, it is unlikely that the short groyne would accumulate sand in the long term, and so would still require periodic sand re-nourishment (albeit less often than would be required with no structure in place).

6.2 South Beach Long Groyne Structure

This long groyne structure would consist of a 130m long groyne located approx. 400m to the North of the SLSC. The landward end of the structure would begin at the existing revetment wall, and from there it would extend seaward out to the existing rock sill that is visible at low tide. As the crest level of the structure would be 2mAHD, the landward end of the structure would be buried in the back beach dune system, limiting its impediment upon pedestrian traffic in the back beach region.

The intent of the long groyne would be twofold. Firstly to increase the length of time that sand is retained on South Entrance Beach post beach nourishment (by several years), and secondly to gradually trap some sand scoured from the entrance by severe (rare) lake flooding.

The difference between this option and the short groyne is that the rate of required re-nourishment would slowly decrease over time, as the beach would slowly build after severe flood events. However, it would be best re-nourished rather than waiting for, perhaps, some years for floods to provide the beach building circumstances. Because the length of wider beach would be greater than for the short groyne, a larger volume of sand would be required to create the same beach width improvement at the SLSC.

The cons of such a structure include the impact of its construction on the community (see **Section 9**), as well as the visual impact of the structure itself. Whist the cost of such a structure would be greater than for the short groyne (due to both construction, and the greater volume of periodic re-nourishment); it would result on a longer beach than that provided by the short groyne.

6.3 Training Wall Structures – Fully Trained Entrance

The fully trained entrance would consist of the northern training wall and northern revetment wall on the northern side of the Entrance Channel, in addition to a southern training wall on the south side of the Entrance Channel. Cardno (2013) showed that training walls would not increase flood levels or flood durations in Lake Tuggerah provided that the walls were spaced 150m apart or wider. Additionally Cardno (2013) showed that the training walls would not impact upon the flushing of the lake system, and thus would not be expected to affect water quality within the lake.

The training walls would be of substantial design, as they would be required to withstand considerable wave action and flood currents (see **Section 9**).

6.3.1 Impact on South Entrance Beach

The impact of the fully trained entrance on South Entrance Beach would predominantly arise from the effects of the southern training wall. The main differences between the southern training wall and the long groyne would be in its intent and structural design. The southern training wall would be built to a higher crest level, be wider and of far more substantial design, see **Section 9**.

Apart from formalizing the entrance area, the training walls would be intended to very gradually trap some sand on its southern side after severe (rare) lake flood events, as sand is transported back onshore by swell wave activity; the volume depending on the flood ARI. As shown in **Section 6.3.2**, the total volume of scoured sand that deposits seaward of the training walls depends on flood ARI, and the percentage of those volumes that is transported to South Entrance Beach varies also with ARI. This alternative could be accompanied by 15,000m^3 of initial nourishment sand to bring forward the expected long term beach amenity improvement.

Because of the plan alignment of this beach, the resulting beach width increase at the SLSC would not be much more than would be achieved by the short groyne for many years (most likely decades). Because the length of wider beach would be greater than for the short groyne, a larger volume of sand would be required to create the same beach width. In the longer term, the beaches south of both structures would gradually fill to a dynamic equilibrium condition, leaving a wider beach for more of the time than occurs now.

The cons of a fully trained entrance in relation to South Beach include:-

- The significant costs involved - see **Section 8**.
- Significant Construction impacts – see **Section 8**.
- The high crest level of such a structure (4.5-5.0mAHD) would inhibit pedestrian access along South Entrance Beach.
- Loss of beach width (long term) along the southern bank of the entrance channel (inside the walls along Marine Parade).
- The visual impact of such walls.

It should be noted that none of these structures would protect against storm induced erosion (see **Section 6.1.5**), and may require re-nourishment after such events, should post-storm onshore transport not cause full recovery.

6.3.2 Impact on North Entrance Beach

In theory, on an infinitely long, plane beach there is spatially constant transport everywhere and the beach alignment doesn't change. If a structure such as a groyne or training wall blocks this sediment transport there will be an up-drift accumulation of sand and a down-drift loss, in general. However, at North Entrance Beach the outcome is complicated by the tidal entrance and the null point, as discussed in principle below:-

- If the northern training wall were placed south of the null point (as the null point was located at the time of construction), one would expect southward transport of sand to cause gradual accumulation of sand against the northern side of the training wall. This process would continue until the beach on the northern side was realigned and the potential for transport was annulled. On the southern side there would be a continuing southward transport to 'feed' the influx of sediment to the entrance, thereby reducing beach width at Karagi Point. This process would continue until net sediment influx ceased and ocean-side shoals developed as shown on **Figure 6.2**. Depending upon the form of southern Karagi Point at the time of construction, that feature might disappear as a beach. However, if the null point formation is more of a result of entrance hydraulics, once the shoreline on the northern side of the training wall was re-aligned to be consistent with the wave directions in that area, that is, it became isolated from the entrance processes, there would be no net southerly transport immediately north of the training wall following any future floods, however, should post flood onshore transport deliver sand to this beach area, then that sand would theoretically be distributed along the whole of the beach, with the development of a small fillet on the northern side of the training wall. This sand would remain on North Entrance Beach causing a long term reduction of sand within the entrance system. On the other hand, should the null point location be driven by wave direction, affected by the rock formation, then there would be some tendency for continuing southward sediment transport on the northern side of the null point in future post flood scenarios. Nevertheless, that process would not be long-lived because the accumulated sand would realign the beach thereby reducing this transport to zero with only re-distribution along the whole beach.
- If the northern training wall were placed at or north of the null point (as the null point was located at the time of construction), then any sand transported onto North Entrance Beach by post-flood waves would be distributed along the whole of the beach, leading to only a small widening of that beach, as there would be no special tendency for sand to be moved south against the wall. However, sand from the entrance would stay on the beach causing a long term reduction of sand within the entrance system. South of the training wall processes would continue as they do now, but with the volume of sand in the entrance area gradually reducing over a long period (decades). **Figure 6.1** shows that a practical northern training wall would lie south of the null point.

An approximate quantification of the post-flood onshore sediment transport structure at The Entrance can be seen in **Table 6-1**. These tables show the results of morphological modelling conducted by Cardno (2013), in terms of the approximate volumes of sediment removed from the entrance and deposited offshore during 1, 20 and 100-years ARI catchment flood events. These tables also show what volumes of material have been transported north, back into and south of the entrance, in the two months after those events. It should be noted that these values are for the modelled scenarios, and should considered indicative only, because the specifics of individual catchment flood events, such as entrance channel velocity, offshore wave height and direction and tides will vary.

These results indicate that for the existing, untrained entrance condition, in the short term most of the sediment is transported north after these catchment events. A much smaller percentage is transported straight back into The Entrance and an even smaller percentage is transported to South Entrance Beach. It should be noted that the proportion of sediment transport moving back into the entrance is significantly higher for 1 years-ARI events as opposed to 20 and 100-years ARI events. This can most likely be attributed to the lower entrance channel velocities of 1-year ARI events, which do not transport sand as far offshore - allowing a higher proportion of sand to be transported back into The Entrance in the short term.

Table 6-2 shows the same results but for the scenarios with 150m-wide training walls. These results indicate that the addition of the training walls results in a greater proportion of the deposited sand ending up on South Entrance Beach, because the southern training wall has prevented this sand from moving back into the entrance.

Table 6-1 Approximate Post-Flood Sediment Transport Structure – No Training Walls

ARI Event	Approx. Volume of Eroded Material (m^3)	% Material Transported North	% Material Transported Back Into Entrance	% Material Transported South
100	250,000	80%	17%	3%
20	150,000	78%	19%	3%
1	45,000	57%	41%	2%

Table 6-2 Approximate Post-Flood Sediment Transport Structure – 150m Wide Training Walls

ARI Event	Approx. Volume of Eroded Material (m^3)	% Material Transported North	% Material Transported Back Into Entrance	% Material Transported South
100	240,000	76%	14%	10%
20	145,000	77%	12%	11%
1	45,000	55%	41%	4%

As can be seen in **Figures 5.9a-d**, some of the sand that is transported north of the training walls, of Pelnard-Considere, calculations have been made about the expected accumulation of beach width on the northern side of the northern training wall post catchment flood event. The results shown in **Table 6-3** indicate that a 100-years ARI storm event is likely to result in a 4.5m accretion of beach width at the training wall, whilst a 1-year ARI catchment flood event is likely to result in an increase of 1.8m. These figures indicate the increase in beach width at the training wall, and will diminish with increasing distance north, away from the wall.

Table 6-3 Increase in Beach Width at Northern Training Wall Post-Catchment Flood Event

ARI Event (Years)	Increase in Beach Width at Training Wall (m)
100	4.5
20	3.7
1	1.8

Note, however, that after 20 years, there will not be 20 x 1.8m + 3.7m increase in beach width because the widening process is not linear in terms of accumulated sand volume.

The cons of a fully trained entrance in relation to North Beach include:

- The significant costs involved - see **Section 8**.
- Significant Construction impacts – see **Section 8**.
- The visual impact of such walls.
- Negative impact upon a habitat of Little Terns near Karagi Point.

6.4 Single Northern Training Wall and Revetment Wall

It is likely that the single northern training wall and revetment wall would have minimal impact upon South Entrance Beach. As such, the impact of a northern training wall structure alone on North Entrance beach would be the same as for the fully trained entrance– mentioned in **Section 6.3.2** above.

6.5 Longshore Transport Modelling of Structures

Noting that the South Entrance Beach shoreline shape is conserved over the years (**Figures 5.2**), but that a shifting in the offshore-inshore direction occurs, the longshore transport modelling can only be used to evaluate the influence of a new structure and/or nourishment. That is, there is no net longshore transport – **Figures 5.2** also shows that sand does not accumulate on the southern side of the cross-shore pipeline that would act as a groyne at the southern end of South Entrance Beach.

The longshore transport modelling first concentrated on the 'calibration' of the inshore wave data in combination with the 1D profile cross section profile prepared for this investigation. Although a 1D LITDRIFT model can simulate the refraction of waves along the profile, it was observed that the refraction in the 1D model was far smaller than in the 2D wave model that takes better account of the inshore complex bathymetric features.

Therefore, a 13 years wave time-series was generated by the combination of wave heights and periods from Location 23 (-10m AHD) and wave directions from Location 53 (-4m AHD) to overcome this issue. **Figure 6.3** presents the wave time-series and also the jointly occurring predicted water levels.

The profile presented on **Figure 6.4** was also 'calibrated' by adjusting the roughness and the zones of rock and sand. Rocks are introduced up to -0.25mAHD because most interest lies in the transport occurring around the MSL water line and above where the sand from nourishment would be distributed predominantly. Furthermore, below that level, the form of the seabed is sufficiently irregular to unsustainable as a sandy seabed.

Figures 5.2 show the different training wall, short and long groyne options that have been considered for concept design investigations. The location and length of the training wall are those adopted in Cardno (2013). The alignments have been set to be about normal to the near shore incident wave crests. Due to the presence of the existing natural soft groyne (tombolo), the training wall and long groyne designs would only have a small effect in terms of holding a wider beach. Their main effect would be to prevent the loss of sand to The Entrance at high water during a major storm event coming from the east-to-south sector. But, as seen in the photogrammetric data, even though the sand was pushed into The Entrance in the storms of 1974, the sand slowly and naturally took its original position behind the tombolo. Hence, further modelling was only undertaken for the small groyne option (just south of the SLSC tower). Outcomes for the longer groyne and training wall options were inferred from those results.

Figures 6.5a, **6.5b** and **6.5c** present, respectively, three different nourishment options n1, n2 and n3. Nourishment option n1 (as displayed in blue dotted lines in **Figures 5.2**) would require 1,000m^3 of sand where option n2 would need 1,500m^3 and n3 around 10,000m^3. Note that about 30,000m^3 were placed on this beach in 2004 (Worley Parsons, 2009) and it is likely that virtually none of that sand has remained on South Entrance Beach.

The 1D LITLINE (longitudinal direction) modelling system was used to model the different nourishment options with the short groyne by incorporating the calibrated profile and inshore wave time-series from LITDRIFT. Even though more than 13 years of wave time-series were used for modelling, results tended to converge after a time period of only 4 years. **Figure 6.6a** shows the estimated yearly coastline evolution for the three tested nourishment options. The tombolo was integrated in this first set of analyses as a groyne and positioned at chainage = 0m. Please note that the SLSC building position would be centred in the middle of the nourishment, around chainage = 210m. Obviously, the more nourishment sand, the longer the time required for the waves to distribute it along the beach. During individual wave events, the further the wave direction from the shore normal direction the faster is the dispersion of the re-nourishment formation. The different options n1, n2 and n3 present respectively a maximum widening of the beach around 6, 8 and 17m in front of the SLSC building without a groyne after 4 years of wave induced longshore transport.

These results are consistent with those observed in **Figure 5.2a** for 2005 following beach re-nourishment by Council in 2004 of about 30,000m^3. That result shows about 50m of beach widening, which is about 3 x the 17m outcome for re-nourishment Option n3 (10,000m^3).

Figure 6.6b presents the results with the addition of a groyne (short groyne south of SLSC tower). The groyne blocks the transportation of sand towards the tombolo and then allows the sand to be distributed over a shorter length of beach. The different options n1, n2 and n3 present respectively maximum widening of the beach of about 8, 10 and 21m in front of the SLSC building with this groyne. Option n3 would provide a good improvement in beach amenity at the club-house, and would likely be semi-permanent, the new 0m AHD shoreline being set about 20m landward from the seaward end of this groyne. However, closure depth is well beyond the seaward end of this groyne and potentially not all of the sand transported seaward during a storm would be likely transported back onto this beach area following storm abatement. Hence some of the nourishment sand would likely ratchet into The Entrance over time.

Hence, it seems reasonable to advise that beach re-nourishment (10,000m^3) would benefit the dry beach width, with the groyne effects on beach widening (versus no groyne) seeming minor (about 4m). However, the groyne would increase the longevity of the work. It is a concern that the groyne could have a negative effect on the beach between the tombolo and groyne that could result in a reduction in beach width there. However, another beach compartment would form between this short groyne and the natural tombolo structure, noting that there is no obvious negative effect on the beach at the groyne (cross-beach pipeline) at the southern end of South Entrance Beach, see **Figures 5.2**.

6.6 Storm Bite Modelling

The beach width in front of the SLSC building tends to vary over the years (see **Figures 5.2**) through the alternation of storm erosion resulting in recession and calm periods leading to rebuilding of the beach.

Figure 6.7 describes the design 5-years ARI storm wave conditions coupled with a spring tide water level variation. An extreme value analysis (with a peak over threshold of 0.7) of the inshore wave time series (at location 23) was undertaken to estimate a 5-years ARI significant wave height of 5.6m at 10m AHD depth.

The 5-years ARI storm time-series was then used in SBEACH to evaluate the storm bite in the different existing and nourished bathymetric profiles.

Figure 6.8a incorporate the results for the existing profile. A recession of around 12m at elevation 1m AHD is observed, but the results seem to be influenced by some dune avalanching processes due to the lack of the existing revetment. A hard bottom revetment (black line) is used to simulate the natural rock seabed and also the wall revetment described on **Figure 5.1**.

Figure 6.8b shows the results for the n1 profile (equivalent to n2 profile in beach width increase). The extra volume of sand added to the existing profile is spread across the profile (between -1m AHD and 4m AHD), leading to a slightly wider post-storm beach. As for the existing case, most of the sand is shifted seaward between -1m and 0m AHD. This sand is expected to be slowly brought back to the dry beach in calm periods. Some of the sand covering the revetment wall is lost during the storm.

Figure 6.8c displays a width loss of about 12m at 1m AHD between pre and post-storm cases, but no sand has been lost along the revetment. The beach width after the storm is still 21m larger than the existing beach after storm erosion. About half of the nourishment sand has been spread and temporarily shifted between -1m and 0m AHD. This sand is also expected to be slowly and partially brought back onto the dry beach in calm periods.

The fact that most of the eroded sand is transported no further offshore than about -1.5m AHD indicates that the possible gradual loss of sand from the re-nourished short groyne beach may be a slower process than suggested above.

7 Design of Coastal Structures

The range of beach amenity and entrance works discussed above has been costed on the basis of concept design details, developed design water level, wave and current parameters and profile details adopted for training walls, groynes and beach nourishment programs at other sites.

7.1 Design Water Levels

Design water levels vary from a lake flood level of 2.2m AHD (Lawson and Treloar, 1994) to an ocean level of 1.44m AHD (Watson and Lord, 2008) for the present day at 100-years ARI. Including 0.9m of projected sea level rise, one has design levels of 3.1m and 2.3m AHD. In the ocean area this leaves aside wave set-up, which may be 0.5m to 1m at the seaward ends of the training walls and at the landward ends, respectively and at the groynes, respectively. Hence, one has design water levels:-

- Upstream of the sill = 3.1m AHD
- At the seaward ends of the training walls – 2.8m AHD
- Between the shoreline and the near shore rocks – 3.3m AHD, The Entrance South

These water levels, together with design seabed levels of -1m AHD in the back-beach area, or bed rock where it lies above this level, and surveyed seabed levels seaward of the near shore rock structure, define design water depths and base levels for structures. Design wave heights will be breaking wave heights.

7.2 Rock Levels and Geotextile Fabric

For final design it will be necessary to establish reliable bed rock levels from the back-beach areas to the offshore extents of proposed structures. It will be preferable to build the structures on top of the natural bedrock as a base.

No geotextile fabric is proposed for the groynes and training walls because of the difficulty of placing it under those structures on uneven rock and in wave affected areas. A geotextile fabric will be required on the North Entrance Karagi Park revetment wall area from the landward end of the North Entrance training wall to The Entrance Bridge.

7.3 Design Wave Parameters

Design wave parameters include wave height and period, wave direction not being particularly important for structural design. However, all structures are aligned to be generally normal to incident wave crests at their seaward ends in order to minimise effects on beaches, other than prevent longshore drift.

7.4 Stability Assessment of Coastal Structures

Preliminary assessments have been conducted for each of the critical design sections of the proposed training walls and groynes. **Figure 6.1** shows the different sections of the training walls. The training walls have been subdivided into different design sections based on their crest level and required armour rock sizing.

For the entrance training walls and Karagi Park revetment, required rock armouring was determined for both 100-years ARI design waves and currents. It should be noted that all calculations have incorporated a sea level rise component of 0.9m in their design water levels. The details of these calculations are provided in **Appendix E**, and the results summarised in **Table 7-1** below.

Table 7-1 shows that the seaward ends of the Northern and Southern Training walls would require approximately 8 tonne M_{50} rock armouring, and to be built to a crest level of +5.0mAHD. These rock sizing's are quite large and are driven by the design wave parameters and water levels at their seaward ends. Rocks sizes could be reduced to approximately 4.0 to 4.5 tonnes for Sections B and E, where the crest level would lower to +4.5mAHD,

The rock sizes for the long revetment wall inside the lake (Sections C and D) are smaller as there is minimal wave action in these regions and design rock sizes are defined by flood currents. The seaward Karagi Point revetment would require a crest level of +3.6mAHD and a M_{50} rock sizing of 300kg. The lake ward revetment extending from Karagi Park to The Entrance Bridge would experience much smaller flood currents, and as such would require a M50 rock sizing of 80kg.

The South Entrance Beach short groyne option (Section H), would require a crest level of +2.0m AHD, and would require approx. 6 tonne M_{50} rock armouring. Alternatively the long groyne (Section G) would require approx. 4.6 tonne M_{50} rock armouring, similar to Section F. This rock size difference follows from the fact that the long groyne would be more protected by the presence of the seabed rock and its level than the short groyne that would be constructed on more erodible sand and be in a greater design water depth.

Information from Cardno (2013) shows that 4m of scour may occur near the prospective training walls. Hence, the toe widths have been increased from 6m on the outside of the walls to 18m on the inside.

Appendix E shows that rock armour sizing calculations were also performed for a damage factor of 2, and for structure side slopes of 1V:1.5H, however the resulting rock sizing's proved to be impractically large. As such, a damage factor of 5 will need be accepted in order to use the resulting rock sizes in **Table 7-1**.

Two separate cross-sections for the revetment region from Karagi Park to The Entrance Bridge have been prepared.

Rock volumes for the outer sections of the prospective northern and southern walls have been based on assumed settlements of 0.5m during construction.

M_{50} values have been scaled up by a 10% factor of safety – USACE (2002).

Preliminary design profiles were developed for the following proposed structures as set out above:

- Short Groyne
- Long Groyne
- Southern Training Wall
- Northern Training Wall

Table 7-1 Proposed Coastal Structures – Required Rock Armouring

Structure	Section	Crest Level (mAHD)	Design Wave Height H_{100} (m)	Design Current Speed V_{100} (m/s)	Side Slope (cotα)	Damage Coefficient (SD)	Adopted M_{50} (kg)	Adopted D_{50} (m)
Northern Training Wall	A	5.0	3.2	5.0	2	5	**8,400**	**1.5**
Northern Training Wall	B	5.0	2.6	4.7	2	5	**4,600**	**1.2**
Karagi Point Revetment	C	4.5	0.8	3.6	2	5	**300**	**0.5**
Karagi Park to Entrance Bridge Revetment	D	3.6	0.5	2.9	2	5	**80**	**0.3**
Southern Training Wall	E	5.0	3.1	3.2	2	5	**8,000**	**1.5**
Southern Training Wall	F	4.5	2.6	4.0	2	5	**4,600**	**1.2**
South Entrance Beach - Long Groyne	G	2.0	2.6	-	2	5	**4,500**	**1.2**
South Entrance Beach - Short Groyne	H	2.0	2.9	-	2	5	**6,000**	**1.3**

Notes:

1. H_{100} – Wave Height exceeded 6 hours every 100 years
2. V_{100} – Structure adjacent current resulting from 100-years ARI catchment flood

8 Costing Estimates

Cost estimates have been prepared based on these preliminary design details and are presented in **Table 8-1**. Inherent Risk and Contingent Risk are varied to arrive at the P50 and P90 cost estimates (see notes below).

Locations of the proposed walls and groynes are shown in **Figure 6.1**. Long-sections and cross sections of the preliminary designs upon which the estimates are based are given in **Appendix G**.

Table 8-1 Cost Estimates

Option	P50 Cost Estimate	P90 Cost Estimate
Sand nourishment (15,000 m^3 at 5y intervals with pipe laid through sand)	$295,000	$385,000
Additional cost to that above for a permanent pipeline for sand nourishment (400m long through rock)	$880,000	$1,140,000
Short Groyne (100m long, 5000t of rock)	$1,540,000	$2,000,000
Long Groyne (130m long, 6000t of rock)	$1,960,000	$2,540,000
Southern Training Wall (230m long; 52,000t of rock)	$9,920,000	$12,830,000
Northern Training Wall (260m long; 80,000t of rock)	$18,130,000	$23,440,000
Northern Revetment (850m long; 90,000t of rock)	$5,600,000	$7,230,000

NOTES:

- The P50 and P90 estimates represent the average expected cost (50%) and the cost for which there is a 90% confidence the cost will not be exceeded (given the assumptions used in the estimation process).
- Base costs for materials and labour have been increased to account for a number of items: site establishment; risk to the builder of storm damage; builder's margin; internal costs for project management; design costs; Inherent risk (changes in project scope or rates assumed); Contingent risk (costs due to unknowns such as rock level); and Escalation (changes in scope during planning of a project).
- Detailed geotechnical investigation to determine the levels of bedrock combined with design using three-dimensional modelling would refine the estimates of the volumes of rock required for the walls and provide an improved estimate of the costs.
- No assessment of wear to local roads from transport of rock by truck (e.g., 10,800 vehicle trips for the northern wall and revetment). No assessment of cash flow has been included in the estimates.
- While every effort has been undertaken to provide a reasonable estimate of the costs, these estimates are preliminary only and are not expected to accurately reflect the final costs of the various options. Detailed design and documentation would be required followed by a full tender process to determine the actual costs for the projects.

No assessment of cash flow was included in these estimates. Basic details adopted for the development of these costs are presented in **Appendix G**.

Factors that may have an influence on the cost estimates are as follows:-

- Ocean/Storm Hazard: The builder will need to carry a degree of risk related to potential ocean/storm damage. Such impacts may occur during construction where a portion of the constructed works is damaged and requires replacement. Cardno has included an amount for this risk (Builder's Risk Premium).

- Inherent Risk: Variability in the scope of work and in rates and quantities used in the estimate. This includes changes to costs of materials and variation in construction methods used that impact on costs.
- Contingent Risk: Risk due to unmeasured items outside the base estimate (for example, design development, owner or user requirements, etc.). A significant example is the unknown ground conditions - the presence or absence of rock under the sands along the proposed structure alignments around The Entrance would influence the costs of the Training Walls. This may occur because there would be reduced need for rock and toe armour if the structures could be founded on existing rock strata. The carrying out of a geotechnical Investigation including establishing where rock strata lie would reduce risk in the construction estimate. While it is possible that costs could be decreased or increased once this information is obtained, Cardno has included an additional amount for Contingent Risk in the cost estimate.
- Escalation—Escalation relates to the changes in the scope of a project that may occur during planning and development of the design. Escalation has been determined based on 15% for Identification and scoping, 12% for Development and 8% for Delivery (total 35%).

8.1 Sand Nourishment

Sand nourishment is expected to be required at approximately 5 yearly intervals (depending on storm erosion activity on the South Entrance Beach).

The cost of sand dredging and pumping is essentially the additional cost of pumping the sand from the dredging point within The Entrance channel approximately 800m to the beach and spreading it across the beach as needed. The cost estimate above includes an allowance for increase in the size of dredging equipment to pump the sand the required distance. Also included is an allowance for machinery needed to spread sand across the beach from the pipe outlet.

The estimate makes no allowance for savings made due to the sand not being pumped onto the North Entrance Beach.

8.2 Permanent Pipeline for Sand Nourishment

A permanently installed pipe to enable the regular replenishment of sand on the beach has been costed as a separate item. The estimate includes 400m of 250mm diameter polyethylene pipe (OD) drilled through rock to provide a permanent pipeline from the southern side of The Entrance Channel to the location on the beach where sand is to be placed (see **Figure 8.1**). This pipe would be well protected from storm damage unlike the temporary pipe laid in sand assumed for the first estimate.

This would be a “one-off” investment as the pipe would be re-used each time nourishment of the South Entrance Beach was required.

8.3 Rock Size and Source for Walls/Groynes

The rock for the walls and groynes is in a range of sizes from the primary armour (largest rock) at 1.5m diameter, the secondary armour at 0.7m diameter and core material ranging from 0.25m down to 0.05m diameter. The actual rock sizes required changes for each application (refer to the sketches for the sizes for the various training walls, revetments and groynes). Some rock larger than 1.5m would be required for the heads of the Training Walls where potential wave action is greatest.

Due to the marine environment, the rocks would be subject to repeated wetting and drying with salts building up in any cracks or pores in the rock. In this environment, the rock would need to be igneous as sandstone would break down relatively quickly.

The proposed source of the armour rock for the walls would be from the Seaham Quarries (either Boral or Hanson near Raymond Terrace) with a road distance of approximately 100 km via the F3, Sparks Road and Wilfred Barrett Drive. This is the closest source for hard igneous rock in large sizes.

The smaller sizes (such as the core materials) can be sourced from the much closer Peats Ridge or Kulnura Quarries (approximately 40 km via the F3, Enterprise Drive and The Entrance Road).

8.4 Life Cycle Cost Estimates

As certain structural options would still require initial and/or periodic sand re-nourishment on South Entrance Beach, the costs of such nourishment programs need to be factored in when the life cycle costs of the various proposed structures. As such, a series of management options have been proposed that consist of different combinations of structural options and nourishment programs. These options are outlined in **Table 8-2** below.

Table 8-2 Assessed Management Options

Option	Structure (s)	South Entrance Beach Nourishment Program
1	None	10,000m^3 per 5yrs
1A	Permanent pipeline for South Beach nourishment	10,000m^3 per 5yrs
2	Short Groyne at South Entrance Beach	10,000m^3 per 7-10yrs
3	Long Groyne at South Entrance Beach	15,000m^3 per 7-10yrs
4	Northern Entrance Training Wall and Northern Revetment Wall	10,000m^3 per 5yrs
5	Fully Trained Entrance	15,000m^3 Initially

In order to allow for a comprehensive comparison of the aforementioned options, a 50 years life cycle period assessment of the cost of each option has been made - see **Table 8-3** below. The costing's account for the fact that sand nourishment on South Entrance Beach will be required less often with the groyne and fully trained entrance options (though Option 4 will not affect South Entrance Beach, as such will still require a nourishment program in line with Option 1).

Annual maintenance costs on the structures have been estimated as a percentage of the capital investment (see **Table 8-3**). Approximate 50-years costs are calculated in terms of Net Present Value using a discount rate of 7%.

Table 8-3 Life Cycle Costs of Management Options

Option	Capital Investment	Cost Per Nourishment	Approximate Frequency of Nourishment (years)	Structural Maintenance Costs (p.a)	Approximate 50 years NPV Cost
1	$-	$256,000	5	N/A	$870,000
1A	$1,140,000	$246,000	5	0.1%	$1,976,000
2	$2,000,000	$256,000	8	1.0%	$2,875,000
3	$2,540,000	$385,000	8	1.0%	$3,788,000
4	$30,670,000	$256,000	5	0.5%	$33,657,000
5	$43,500,000	$385,000	Only Initial	0.5%	$46,886,000

9 Construction Issues

9.1 General Discussion

Of the 7 options listed in the Cost Estimates Table, the sand nourishment is the cheapest option, even considering the need to re-nourish the beach at 5 yearly intervals (on average, depending on storm activity).

Provision of a permanent pipeline (the second option) would simply provide for easier pumping arrangements and reduce the risk of the loss or vandalism of a shallow pipe buried in the sand.

The Short Groyne and Long Groyne are separate options that assist the retention of sand on the South Entrance Beach.

The remaining 3 options would be carried out together so that the Southern Training Wall, Northern Training Wall and the Northern Revetment are required to be constructed together. This would be necessary to ensure the hydraulic performance of The Entrance Channel was achieved as designed while assisting in retaining sand on the South Entrance Beach.

Construction of the Training Walls and Revetment would involve, as a general guide:

- 185,000 tonnes of rock comprising:
  - 130,000 t sized 0.5m up to 1.5m or larger
  - 55,000 t smaller sizes
- Approximate costs of rock:
  - $60 / tonne for sizes 0.5m up to 1.5m (delivered)
  - $45 / tonne for smaller sizes (delivered)

9.2 Space for Construction

The construction of the structures on the southern side of the channel (Southern Training Wall and Groynes) could be carried out via access from Marine Parade. The sand area at Karagi Point adjacent to Marine Parade could provide a staging area for materials storage and equipment.

For the northern structures, a staging area could be provided at or adjacent to the car park at the end of Hutton Road. In addition, a road would need to be constructed along the western side of the sand spit (out to Dunleith Point). The road would follow the line of the revetment and wall to provide access for trucks and equipment. Disturbance of the sand spit might include space for manoeuvring of equipment and turning of trucks and trailers.

9.3 Construction Program

Construction of the Groynes could be expected to extend for several weeks or months. Larger armour rock would need to be placed one rock at a time due to their size and the need to position them in layers, well packed together. Placement would be with the use of an excavator with a long arm or by crane and grab.

The Training Walls however, require many thousands of tonnes of rock. This would require an extended construction period. A minimum of 18 months could be expected, although a faster construction program could be achieved at increased cost by employing more equipment and manpower. The practical limit on the construction process may be the rate at which the rock can be extracted from the quarry or by the rate at which rock can be physically placed on the walls.

9.4 Rock Transport

The numbers of truck movements (full in, empty out) estimated for the proposals considered are listed below. Each load represents two truck and dog trailer movements as the truck arrives loaded and leaves empty, passing twice along the streets.

- Short Groyne — 500 (truck and dog trailer movements)
- Long Groyne — 600
- Southern Training Wall — 4,600
- Northern Training Wall — 8,000
- Northern Revetment — 2,800

For the Northern Revetment and Training Wall the total is 10,800 vehicle movements over an approximate construction period of 18 months for approximately 15 loads delivered per day (30 vehicle movements).

Trucks loaded with smaller size rock can carry approximately 32 tonnes for truck and dog (core size materials). For the larger rocks approaching 1.5m in diameter, each truck may only be able to transport a couple of rocks of such size in any one load. Therefore the loads could be less than 20 tonnes in each truck and dog trailer.

The Contractor may be left to determine the most efficient method of delivery of the rock to the site.

For example, barges could be used to deliver rock to the off-shore portion of the Northern Training Wall from a loading site in Newcastle. This would not be practical for the Groynes or the Southern Training Wall as they either do not extend appreciably past the low tide mark or are surrounded by rock reefs. The off-shore portion of Northern Training Wall could be accessible via barge but only during calm weather. Approximately 50,000t would be required for this portion of the Wall which is approximately 30% of the rock required for the Northern Training Wall and its Revetment.

Barges would not have access through The Entrance Channel due to the shallow rock bar (reef) that exists across the channel at the beach outlet. Consideration may be given to removal of this rock barrier to allow such access for the delivery of rock by barge. This would reduce the volume of rock required to be delivered by truck from Seaham (a 100km drive one way).

Barges are also an option for delivery of rock within the channel and behind the sand spit from a loading site somewhere on the Lakes foreshore. Rock could be delivered to a temporary loading site by truck (for example, at the end of Highview Avenue or Emu Drive, San Remo, or Wyong wharf at River Road). The barges would then travel across the Lakes to the work site. Rock could be lifted into position directly from the barges. Some dredging of sand may be required inside the channel for the barges to access the construction site.

9.5 Other Construction Considerations

In considering the proposals above, the following issues are noted:

- Bedrock levels. The determination of detailed bedrock levels along the alignments of the structures is needed to allow more accurate assessment of the volumes of rock required. The presence of bedrock at levels suitable for founding of the rock walls and revetments would remove the need to protect the structures from undermining due to scour when high flow events occur. This could potentially reduce the rock volumes by 30% from those assumed in this Report by allowing removal of the toe portions (see Sketches of cross-sections attached).
- The numbers of truck movements. Heavy vehicle movement imposes wear and tear on the road system, represents a traffic hazard for local residents, and creates noise and air pollution.
- Use of public land for stockpiles, staging areas, sheds and equipment storage. For the northern training wall and revetment, the car parking area at the southern end of Hutton Road could be occupied during construction (approximately 18 months) for use as a construction compound. This

use of public land would result in inconvenience for the public, reduced access to the sand spit and reduced access to the North Entrance Beach.

- Disturbance of the sand spit ecosystem (e.g. Little Tern nesting sites). Construction of a road way along the sand spit and a revetment and wall at the water's edge along 1km of the foreshore would remove the sloping beach on that side of the channel. Sandy beach foreshore could be potential habitat for various species including the Little Tern. An investigation of the environmental impact (EIS) would be required to determine what species may be currently present. The provision of the open rock structures would provide a potential new habitat for species that inhabit such environments.
- Loss of amenity for beach goers (tourism). The inside of the sand spit is currently used as a beach providing access to The Entrance waters by small boats, kayaks, boards and other water activities. Placement of a revetment along this portion of the North Entrance foreshore will deprive beach users of the easy sand-beach access.
- Deepening of the Channel for Barges. The Entrance Channel is blocked by a rock bar (reef) across the channel near the beach front. Use of barges would reduce the need to transport approximately 200,000t of rock (via 17,000 truck movements on the F3 and through the Toukley suburbs). The rock bar may be deepened sufficiently to allow barge access by blasting or other methods to remove the bedrock. Provision of suitable access for barge delivery of rock during construction may not provide suitable navigable access for general boating purposes.
- Navigation. Potential opening of The Entrance Channel for navigation by water craft by deepening of the channel bed would require removal of the bedrock. There may be a potential economic benefit to the Tuggerah Lakes system if this were undertaken.

10 Concluding Remarks

This report describes the data and methods adopted to investigate a range of possible options for beach amenity improvements on South and North Entrance Beaches. This work follows previous investigations (Cardno, 2013) undertaken for OEH in terms of the effects of proposed training wall options at The Entrance for the Tuggerah Lakes system.

The work has led to a range of options for South Entrance Beach and a quantification of the likely rate of accumulation of sand on North Entrance Beach over an extended period caused by post-flood event onshore sand transport. The form of South Entrance Beach is controlled by the rocky headland and pipeline/groyne at its southern end (small effect) and the bedrock structure at its northern end, which controls the tombolo feature that provides a soft boundary at its northern end (major effect)

<u>The options addressed on South Entrance Beach are:-</u>

- 10,000m^3 of periodic sand nourishment from The Entrance – with no new structures in place.
- A short groyne south of the rocks with 10,000m^3 of periodic sand nourishment from The Entrance
- A long groyne further north at the rocks with 10,000 or 15,000m^3 of periodic sand nourishment obtained from Wyong Council's dredging campaigns.
- A southern entrance training wall (as part of a fully trained entrance) that extends seaward beyond the rocks to a depth of about 2.3m at datum AHD. This would be a more substantial structure than the long groyne. It would require 10,000 to 15,000m^3 of initial sand nourishment.

All three structures would need to butt up to the existing revetment structures that have built along the southern shoreline of The Entrance in order to prevent out-flanking by a future very severe storm of character similar to the May 1974 storm.

<u>The options addressed for South Entrance Beach are:-</u>

- A northern entrance training wall and northern revetment wall (as a standalone structure). As this would not address South Entrance Beach issues it would require 10,000m^3 of periodic sand nourishment on that beach.
- A northern entrance training wall and northern revetment wall plus a southern training wall (as part of a fully trained entrance). This would require 15,000m^3 of initial sand nourishment.

Sediment transport on the southern end of North Entrance Beach includes a null point that is caused mainly by the northward increasing wave height gradient that arises from the offshore and near shore bed rock structures. Numerical wave, hydrodynamic and morphological modelling reported in Cardno (2013) was used to investigate this phenomenon and showed that it is a region rather than a unique location. A northern training wall would be built generally south of the null point zone. Although there would be a long term accumulation of sand on the northern side of this training wall, caused by post-flood onshore sand transport, and the beach would very gradually widen, there would be no reduction in shoreline recession and erosion hazards at Hutton Road for many decades. That process could only be assessed by long term monitoring – photogrammetry. In order to prevent short-circuiting by a major flood forming a channel north of the northern training wall, this structure includes major works along the northern shoreline up to Karagi Park and then to The Entrance Bridge as a revetment to prevent erosion of that shoreline.

This report also includes costing of concept designs. That task first needed definition of met-ocean design parameters, leading to dimensional requirements. Preliminary P90 costs of the options are:-

- Sand nourishment (15,000m^3 – with pipeline installed) - $385,000.
- Additional cost for a permanent pipeline for sand nourishment - $1,140,000.
- Short groyne - $2,000,000.
- Long groyne - $2,540,000.

- Southern training wall - $12,830,000.
- Northern training wall - $23,440,000.
- Northern revetment wall - $7,230,000.

Final design of groyne or training wall structures will need detailed bed rock definition to determine base levels and toe width parameters – limit of scouring where it may occur.

11 References

Cardno (2013): Tuggerah Lakes, The Entrance Morphodynamic Modelling. Report (LJ2985/R2773) Prepared for the NSW Office of Environment and Heritage.

DECCW (2010): Tuggerah Lakes Flood Study, latest Tuggerah Lakes Floodplain Risk Management Study 2011(in prep).

DECCW (2010): Tuggerah Lakes Estuary Modelling.

Deltares (2011): Delft3D-FLOW – User Manual. Version 4.00, Revision 12556. Deltares, Rotterdam.

Engelund, F and Fredsoe, J. (1976): A Sediment transport Model for Straight Alluvial Channels. Nordic Hydrology, Vol. 7, pp293-306.

Henderson, F M (1966): Open Channel Flow. Macmillan Series in Civil Engineering.

Lawson and Treloar (1994): Tuggerah Lakes Flood Study.

Lawson and Treloar (1999): Recalibration of Tuggerah Lakes model and Evaluation of The Entrance Dredging Impacts.

Manly Hydraulics Laboratory (1975): Tuggerah lakes Investigation – Tidal Data Collection 8.10.75, Report MHL344.

Manly Hydraulics Laboratory (1995) Tuggerah Lakes Entrance Tidal Survey January - December 1993.

Manly Hydraulics Laboratory (2010): Water Levels at Tuggerah Lakes.

NSW Department of Environment, Climate Change and Water (2011): Draft Report "Tuggerah Lakes Estuary Modelling" Prepared by K. Brennan, A. Ferguson, B. Sanderson, T. Weber and P. Scanes.

Office of Environment and Heritage (2011): Report "Box Model Simulation of Tuggerah Lakes with Re-suspension and Settling and Ecosystem Dynamics; Author B.G. Sanderson.

Office of Environment and Heritage (2011): Tuggerah Lakes Ecological Response Model Project – Water and Coastal Science Section, Coastal Waters Unit.

Patterson Britton and Partners (1994): Tuggerah Lakes, Entrance Training Walls: Technical Discussion.

Patterson Britton and Partners (1988): Tuggerah Entrance Improvements - Entrance Restraining Walls - Concept Design Report.

Patterson Britton & Partners (1988): Tuggerah Entrance Improvements. Entrance Restraining Wall – Concept Design. Report Prepared for Public Works Department, NSW and Wyong Shire Council.

Patterson Britton & Partners (2005): Lake Illawarra Entrance Works Project (Stage 2). Northern Training Wall, Southern Spur Wall and Channel Dredging. EIS Prepared for Lake Illawarra Authority.

Public Works Department NSW (1987): Jet Pump Systems for Maintaining Tidal Entrances , Report No. PWD 87051.

U.S. Army Corps of Engineers. (2002): Coastal Engineering Manual. Engineer Manual 1110-2-1100, U.S. Army Corps of Engineers, Washington, D.C. (in 6 volumes)

Umwelt, (2011): Entrance Dynamics and Beach Conditions at The Entrance and North Entrance Beaches.

Watson, P.J. & Lord, D.B. (2008): Fort Denison Sea Level Rise Vulnerability Study. Report Prepared for the Coastal Unit, Department of Environment and Climate Change.

Webb McKeown and Associates (1992): Sensitivity Analysis, The Entrance Channel, Tuggerah Lake - Final Report.

Weston, Captain C (2013): Report on the Safety of Navigation should Training Walls be Established at the Bar Way Entry to The Entrance in New South Wales.

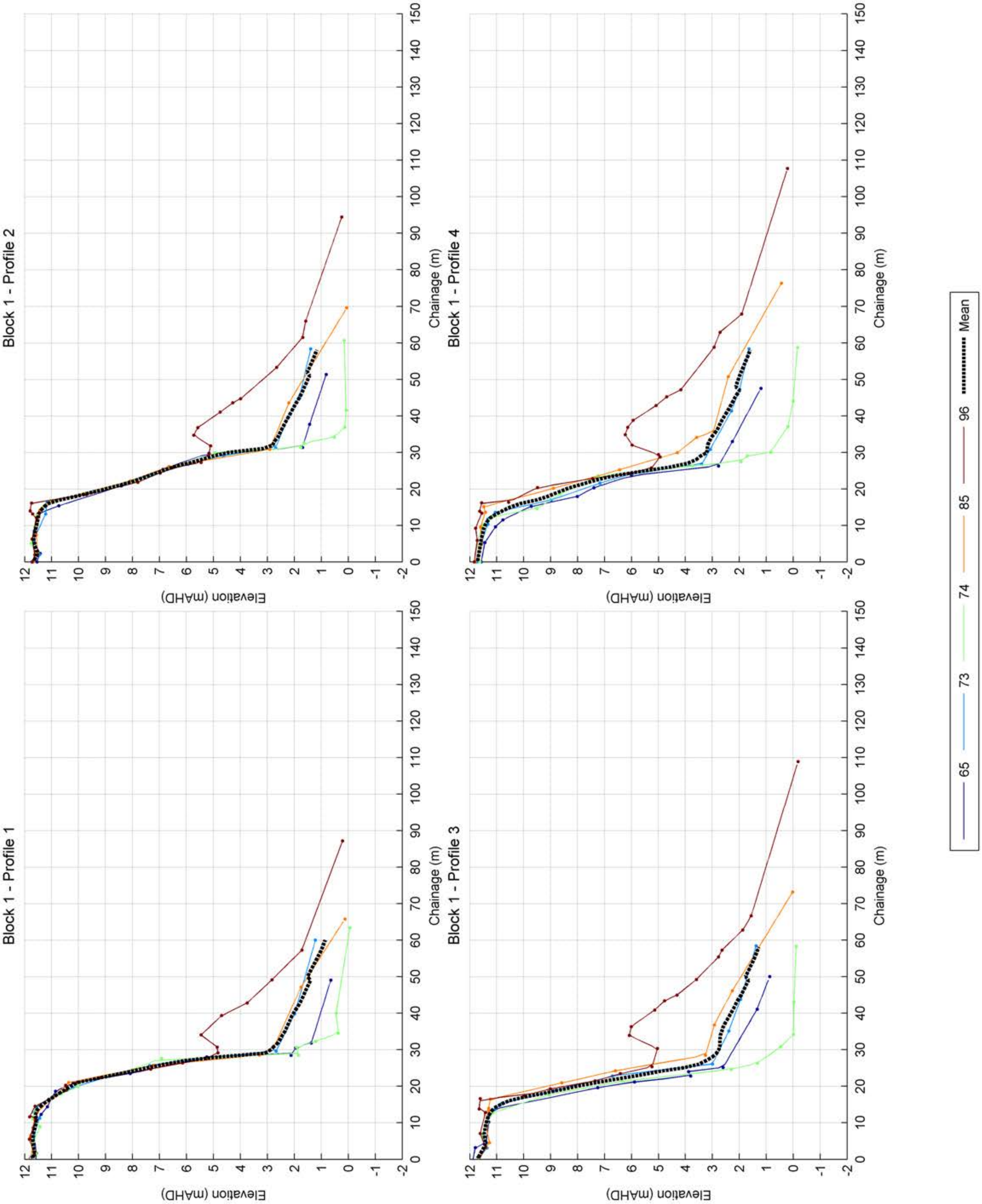
Wyong Shire Council (2010): Tuggerah Lakes Estuary Management Plan and Study.

Worley Parsons (2009): The Entrance Dredging Project, Review of Environmental Factors. Report rp7291oam091112 Prepared for Wyong Shire Council.

Tuggerah Lakes The Entrance
Morphodynamic Modelling
Entrance Beach Management
Investigations

APPENDIX A
PHOTOGRAMMETRY SECTION LINES

Cardno
Shaping the Future



Tuggerah Lakes − The Entrance Morphodynamic Modelling

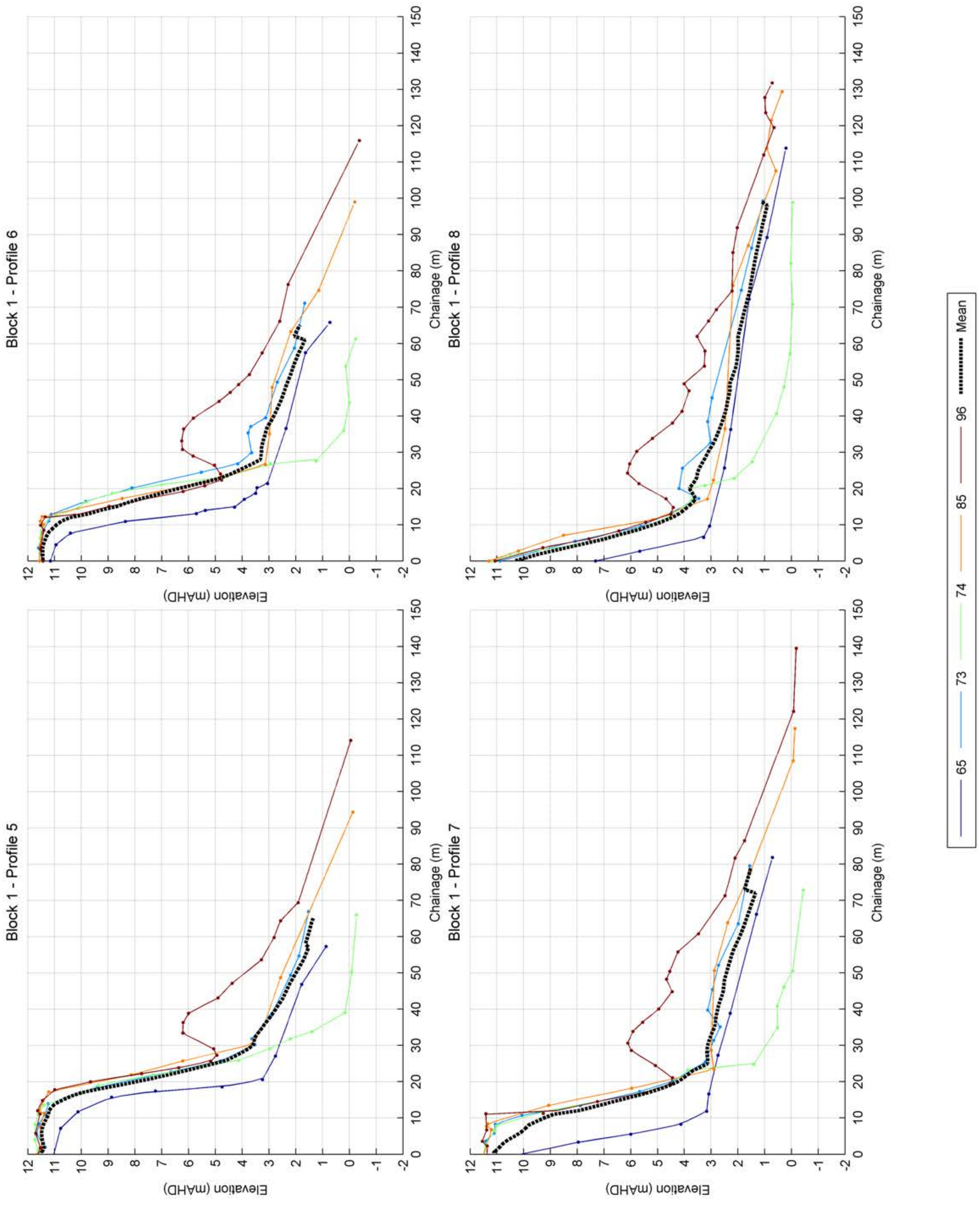
Photogrammetric Profile Plot
Block 1 − Plot 1

Appendix A.1a

Cardno

LJ2985/R2791 Oct 2013

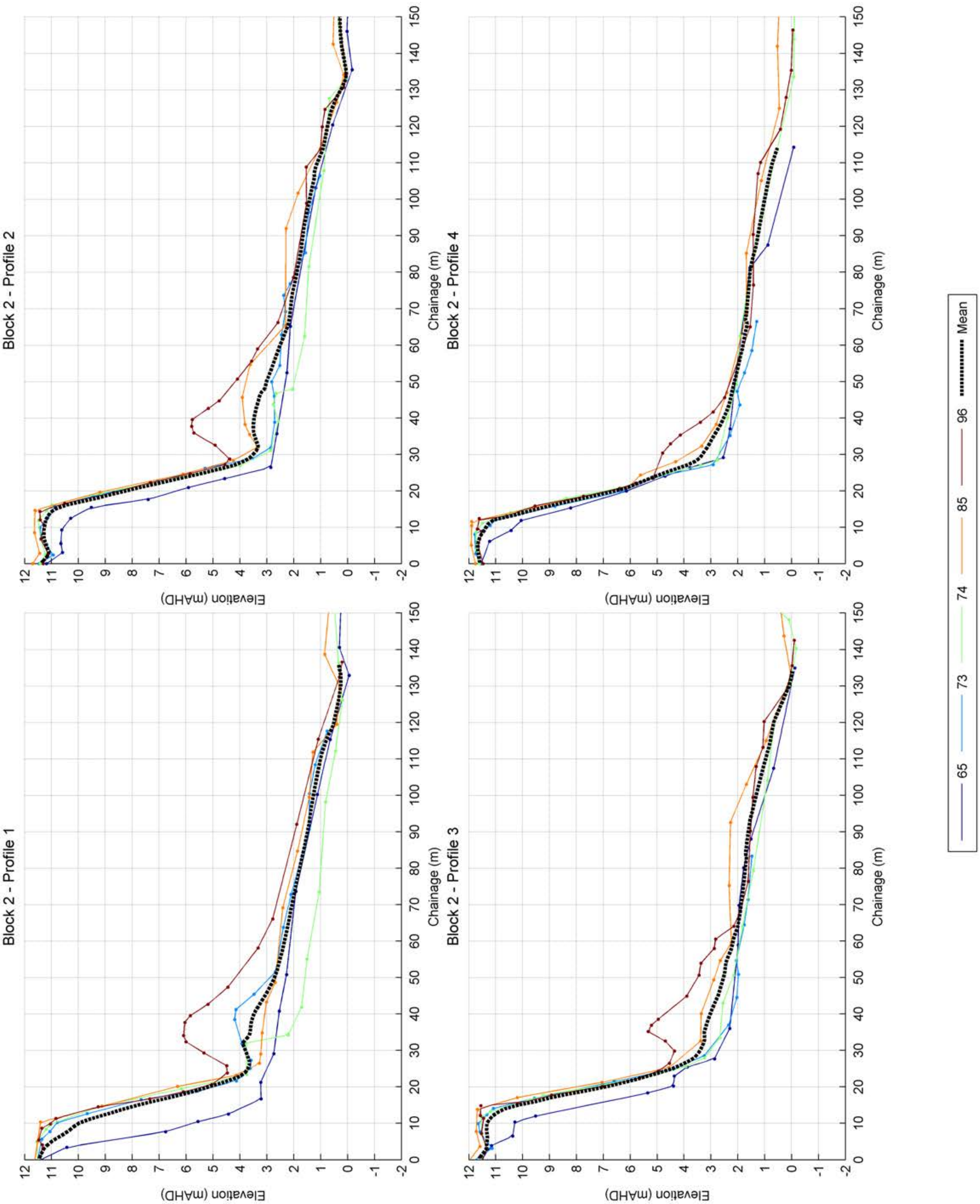
J:\CM\LJ2985−TheEntrance\009−ReportingDocsandFigures\Figures\



Tuggerah Lakes − The Entrance Morphodynamic Modelling

Photogrammetric Profile Plot
Block 1 − Plot 2

Appendix A.1b



Cardno

LJ2985/R2791 Oct 2013

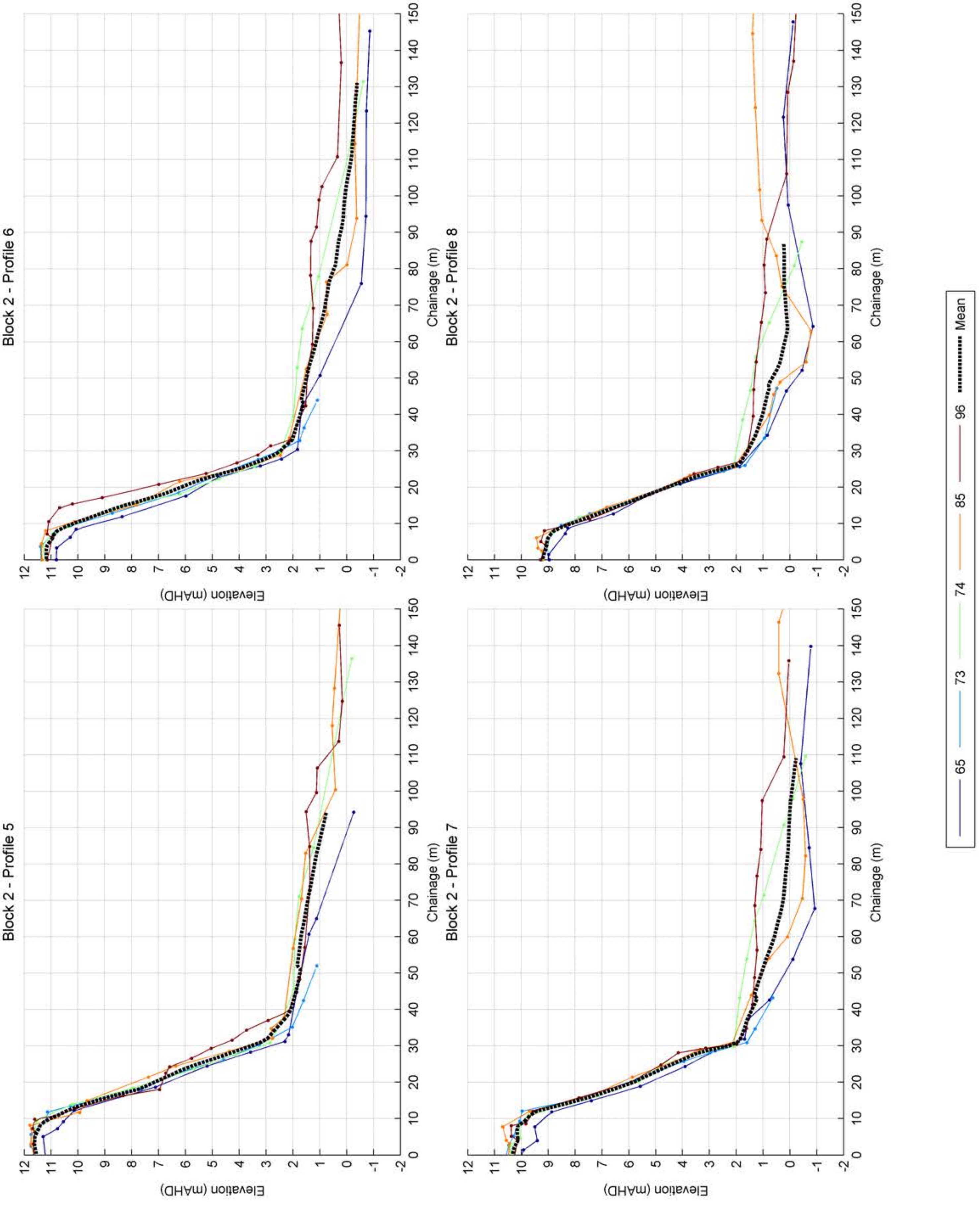
J:\CM\LJ2985−TheEntrance\009−ReportingDocsandFigures\Figures\

Tuggerah Lakes − The Entrance Morphodynamic Modelling

Photogrammetric Profile Plot

Block 2 − Plot 1

Appendix A.2a



Cardno

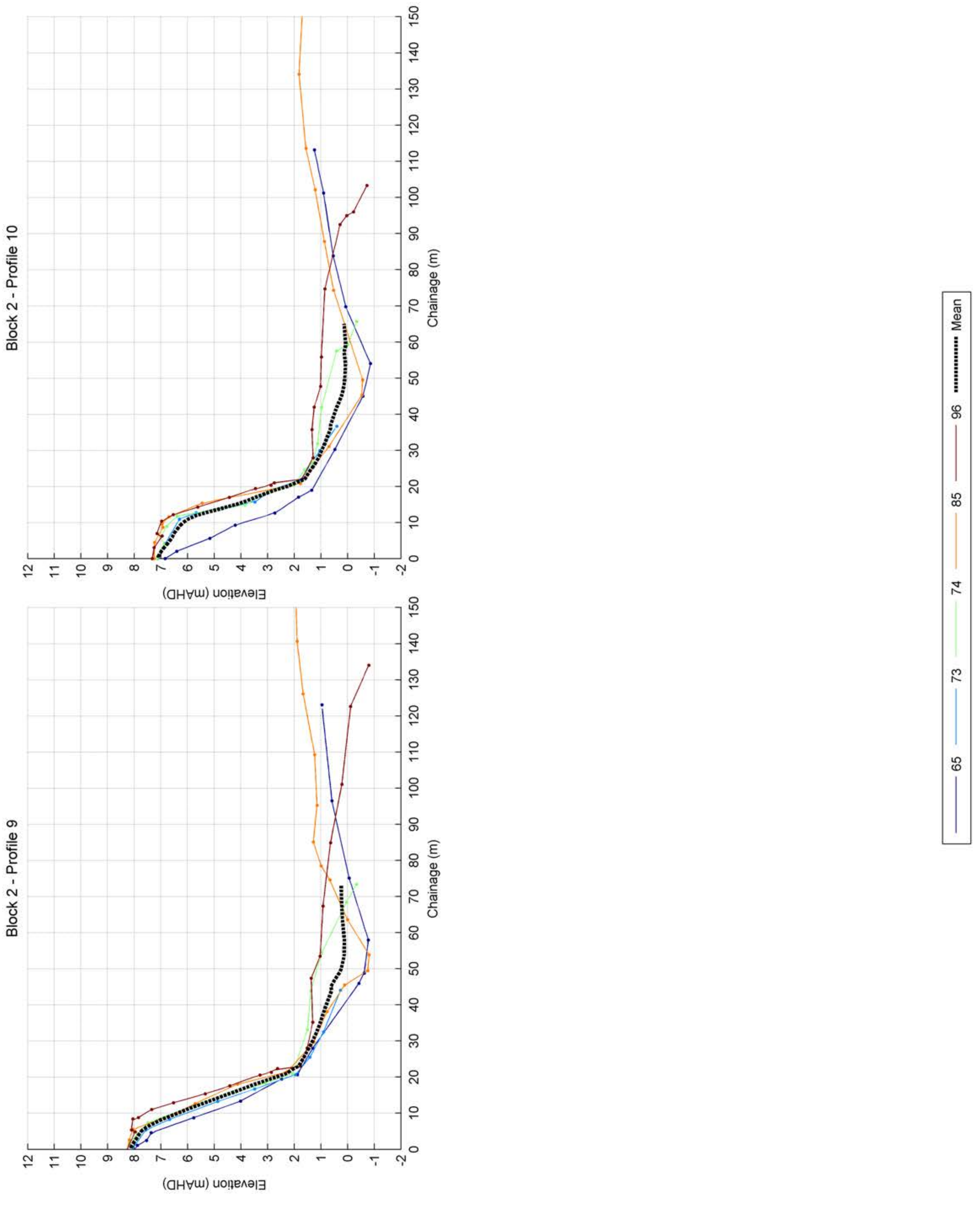
LJ2985/R2791 Oct 2013

J:\CM\LJ2985−TheEntrance\009−ReportingDocsandFigures\Figures\

Tuggerah Lakes − The Entrance Morphodynamic Modelling

Photogrammetric Profile Plot
Block 2 − Plot 2

Appendix A.2b



Tuggerah Lakes − The Entrance Morphodynamic Modelling

Photogrammetric Profile Plot
Block 2 − Plot 3

Appendix A.2c

LJ2985/R2791 Oct 2013
J:\CM\LJ2985−TheEntrance\009−ReportingDocsandFigures\Figures\

Tuggerah Lakes The Entrance Morphodynamic Modelling Entrance Beach Management Investigations

APPENDIX B

COMPUTATION OF MEAN WAVE PARAMETERS

Cardno
Shaping the Future

The quantity of littoral drift along a shoreline is proportional to T x H_e^2 x sin 2β

where T is wave period

H_e is effective wave-height (based on significant wave heights)

β is the angle between the shoreline and breaking wave crests

H_e is a significant or root-mean-square wave-height which must incorporate the description of long term wave occurrence near the shoreline. First, near shore wave heights were computed using the long-term offshore Botany Bay wave climate and computed wave coefficients, (combined K_r, K_s and K_f). At each near shore location the log-normal probability of exceedence distribution describing wave climate was prepared for swell waves. H_e was then calculated from:-

$H_e^2 = \int H^2 \, p(H) \, dH$

where p(H) is the log normal distribution of significant wave heights

with the result that

$H_e = H_{50} \, e^{\sigma y2}$

where H_{50} is the median significant wave-height defined by the log normal distribution = $(H_{10} \times H_{90})^{1/2}$

y = ln(H)

σ_y = standard deviation of y = 1/2.563 ln (H_{10}/H_{90})

Weighting factors E_{ij} for coastal process analyses are defined by the wave energy input

$E_{ij} = P_{ij} \times H_{eij} \times T_j$

where P_{ij} is probability of the occurrence of waves in direction band i period band j

A similar procedure was applied to local sea analyses. In that case P_{ij} relates to wind speed and direction occurrence.

Weighted mean wave direction, ϕ_m, is estimated from:-

$\phi_m = \sum P_{ij} \times H_{ij}^2 \, T_j \, \phi_i \, / \, \sum P_{ij} \times H_{ij}^2 \, T_j$

Tuggerah Lakes The Entrance
Morphodynamic Modelling
Entrance Beach Management
Investigations

APPENDIX C
DREDGING PLAN FOR THE ENTRANCE

Cardno
Shaping the Future

The following description of Wyong Council's dredging operations at The Entrance has been prepared from Worley Parsons (2009) – not verbatim.

Description of the Proposed Works

Proposed Dredging Works

The dredging is generally to be undertaken as per previous dredging campaigns of The Entrance Channel and is predominantly designed to enhance the *ebb* tide flow (out flow) from the estuary. The dredge strategy was developed following trial dredging investigations in 1991 and has been refined following annual maintenance dredging that has been carried out in The Entrance Channel since 1993. The current strategy involves staged dredging by Council using a small (10/8) cutter suction dredger (CSD)**.** The typical arrangement of the dredge footprint covers approximately 2.5km of channels and sumps within The Entrance System**.**

Dredging commences from the upstream end of the channels such that the ebb flows contribute to the dredging efforts. The channels are typically dredged to a width of 50m and to a level of 2.0m below water level except as noted below. Water level in the lake is approximately 0.06m above Australian Height Datum (AHD) in the vicinity of The Entrance which is roughly equivalent to mean sea level. The surveys indicate that much of the proposed dredge footprint will require dredging in the next few years.

Dredging is generally undertaken as follows:-

- creation of a sediment trap (sump) across the main entrance parallel and adjacent to the eastern side of the road bridge. The low velocity environment created by the dredged sediment trap causes deposition of sands migrating with the flood tide, prolonging the timeframe required between maintenance dredging episodes and reducing the need to dredge channels upstream of the bridge. The sump adjacent to the bridge has previously been dredged to approximately 30m in width in the vicinity of Yellawa Island. However, it is proposed to exclude from the dredge footprint that portion of the sump immediately to the west of Yellawa Island to reduce any risk of foreshore erosion to Yellawa Island.
- dredging the main channel to the east of the road bridge on a yearly basis.
- dredging the ebb dominant northern channel (between the road bridge and the caravan park). This section of channel is dredged approximately every two years.
- dredging the ebb dominant northern channel from the caravan park, downstream through the middle of the flood tide shoal to the mouth of the estuary. This channel is dredged to a width of approximately 80m. The southern tip of the sand spit is also dredged. Dredging is undertaken yearly in these areas.
- Additional dredging is also undertaken on an 'as required' basis:
- dredging of Terilbah Channel, from the northern end of Terilbah Island, approximately parallel to Stewart St, downstream to the road bridge. Terilbah Channel has been dredged three times since dredging began in 1993 and was last dredged in 2008 – as at 2009.

Occasional dredging of a sump, perpendicular to and south of the main channel, just to the west of the sand spit.

Dredging of the main channel to the west of the road bridge to a width of approximately 80m. This area was significantly dredged in 1993 and was last dredged in 1995 – as at 2009. The area has progressively become shallower and is likely to require dredging in 2010 (as at 2009) to allow flushing of the ebb tide into The Entrance Channel.

Dredging of a flood dominant southern channel (to 1.0 m below water level) along the southern foreshore of The Entrance Channel.

Production Rates and Quantities

Council's dredge was built to specification based on dredging trials undertaken in March/April 1991. The trials indicated that effective maintenance of The Entrance Channel would require a dredge capable of removing 60,000m3 of material over a 12 weeks period.

Dredge quantities are available from the 2004 campaign. These records indicate that 81,300m3 (132,800t) of material was dredged from The Entrance Channel. Council's dredge crew have indicated that these records are typical of quantities dredged on a yearly basis over approximately a 3 to 4 months dredging campaign.

Dredging production rates of ≈105 m3/hr (170 t/hr) are generally achieved by the CSD. Slower rates are expected during dredging of the sump and in the vicinity of the ebb tide channel between the bridge and the caravan park due to the presence of old bridge supports and old Telecom cables within the channel. Similarly, dredging of the main channel downstream of the caravan park is often slowed due to the presence of fishermen and anchored boats within the channel.

Proposed Beach Nourishment

Dredged sand is beneficially reused to nourish areas where, through visual inspection, it is determined that maximum environmental benefit to the dune system and beach amenity would result. Council aims to nourish beaches and foreshores to:

- re-nourish and protect dunes and foreshore areas and subsequently the ecosystems of these areas;
- protect the recreational value of the beaches as areas of public recreation; and
- retain sand as mobile beach sand circulating within The Entrance sand system and prevent a net reduction of sand from the system over time. This is necessary to maintain the sand spit, The Entrance sand bar and flood tide shoals which are the natural control on lake levels and which provide natural protection of upstream areas from ocean storms.

North Entrance Beach is nourished during each dredging campaign. The beach profile experiences erosion during significant storm events which can undermine the vegetated dunes.

Approximately 50,000m3 of dredged sand is deposited on North Entrance Beach (as indicated by 2004 records). Placement to the south of a null point in the general vicinity of Hargraves St ensures that the sand is reworked back towards The Entrance Channel, thereby retaining sand within The Entrance sand system.

The estuary eastern beach is re-nourished on a regular basis. ‘Recently’, a small sand spur was also placed in the vicinity of the boundary of Karagi Foreshore Park and the Dunleith Caravan Park.

The (South) Entrance Beach is re-nourished on a less frequent basis. Nourishment has been undertaken approximately every five years (1994, 1999, and 2004). Approximately 30,000m3 of dredged sand was placed on The Entrance Beach in 2004. Nourishment generally takes place only following representations from the Surf Club. Council consider that the area is too dynamic for sand to remain in place for any considerable length of time. The nourishment process is often slower than that of adjacent beaches as a result of regular disruption to the floating discharge pipeline during strong flood tides through the throat of The Entrance Channel or due to wave action across the rock platform to the north of The Entrance Beach.

Dredged sand is pumped from the CSD to the nourishment areas along a temporary submerged discharge pipeline. A permanent pipeline is also buried within the dune system and exits onto North Entrance Beach. The maximum pumping distance from the CSD to any nourishment area is 800m. No booster pump is used. Sand dredged from upstream of the road bridge is therefore limited to placement on the estuary eastern beach. Dredged sand from the sump and from the ebb tide channel between the bridge and the caravan park is deposited on the estuary eastern beach, whereas sand dredged further downstream, from the main channel and from the flood dominant southern channel is pumped to North Entrance Beach or occasionally, The (South) Entrance Beach.

To minimise localised erosion at the discharge location, the dredged sand is sprayed upwards to dissipate energy. This is undertaken from an elevated pipeline outlet onto the sub-aerial (above water) profile of the beach, below the edge of the erosion scarp where possible.

The throat is that section of the channel near the southern tip of the sand spit having minimum cross-section dimensions.

Tuggerah Lakes The Entrance Morphodynamic Modelling Entrance Beach Management Investigations

APPENDIX D
SITE INSPECTION PHOTOGRAPHS

Site Inspection Photographs − 6 June 2013
Sand Placement at Karagi Park

Appendix D.1

Cardno

Tuggerah Lakes − The Entrance Morphodynamic Modelling

Site Inspection Photographs − 6 June 2013
Sand Placement at Karagi Park

LJ2985/R2791 Oct 2013
J:\CM\LJ2985−TheEntrance\009−ReportingDocsandFigures\Figures\

Appendix D.2

Tuggerah Lakes − The Entrance Morphodynamic Modelling

Site Inspection Photographs − 6 June 2013
Spoil Disposal Pipeline

Appendix D.3

LJ2985/R2791 Oct 2013
J:\CM\LJ2985−TheEntrance\009−ReportingDocsandFigures\Figures\

Tuggerah Lakes − The Entrance Morphodynamic Modelling

Site Inspection Photographs − 6 June 2013
Sand Placement at North Entrance Beach

Appendix D.4

Site Inspection Photographs − 6 June 2013
Sand Placement at North Entrance Beach

Appendix D.5

Site Inspection Photographs − 6 June 2013
Prevelant Rock Sill at South Entrance Beach

Tuggerah Lakes The Entrance Morphodynamic Modelling Entrance Beach Management Investigations

APPENDIX E

PROPOSED STRUCTURES – STABILITY ASSESSMENTS

Cardno
Shaping the Future

Training Wall Armour Design Calculations

Rock Sizings to Resist Wave Action

Structure: Northern Training Wall

Section: Section A: +5.0mAHD Crest Level

Design Criteria:

Component	Value	Comments
H_{100}	3.15 m	Wave Height Exceeded at toe of structure 6 hours every 100 years
Tz	10.7 s	100 years ARI Tp = 15.0s
d	4.6 m	Depth = -2.3mAHD Bed level at toe of structure + 1.44m Storm Surge + 0.9m SLR
$\cot\alpha$	2	Batter Slope
P	0.1	Porosity of Structure
SD	5	Damage Factor
Wr	2600 kg/m3	Dry Rock Density
Storm Duration	6 hours	Typical Wave Event Duration

Rock Armour Specifications (Van Der Meer (1998)):

		Unfactored		Factored by 10%	
		Diameter (m)	Mass (kg)	Diameter (m)	Mass (kg)
Primary Armour	50% Passing	1.43	7,640	1.48	8,400
	15% Passing	1.28	5,470	1.32	6,020
	85% Passing	1.60	10,680	1.65	11,750
Secondary Armour	50% Passing	0.66	760	0.69	840
	15% Passing	0.59	540	0.61	590
	85% Passing	0.74	1,060	0.77	1,170
Core Details	50% Passing	0.24	38	0.25	42
	15% Passing	0.22	27	0.23	30
	85% Passing	0.27	53	0.28	58

Training Wall Armour Design Calculations

Rock Sizings to Resist Wave Action

Structure: Northern Training Wall

Section: Section A: +5.0mAHD Crest Level

Design Criteria:

Component	Value	Comments
H_{100}	3.15 m	Wave Height Exceeded at toe of structure 6 hours every 100 years
Tz	10.7 s	100 years ARI Tp = 15.0s
d	4.6 m	Depth = -2.3mAHD Bed level at toe of structure + 1.44m Storm Surge + 0.9m SLR
cotα	2	Batter Slope
P	0.2	Porosity of Structure
SD	5	Damage Factor
Wr	2600 kg/m3	Dry Rock Density
Storm Duration	6 hours	Typical Wave Event Duration

Rock Armour Specifications (Van Der Meer (1998)):

		Unfactored		Factored by 10%	
		Diameter (m)	Mass (kg)	Diameter (m)	Mass (kg)
Primary Armour	50% Passing	1.36	6,530	1.40	7,180
	15% Passing	1.22	4,670	1.26	5,140
	85% Passing	1.52	9,130	1.57	10,040
Secondary Armour	50% Passing	0.63	650	0.65	720
	15% Passing	0.56	470	0.58	520
	85% Passing	0.70	910	0.73	1,000
Core Details	50% Passing	0.23	33	0.24	36
	15% Passing	0.16	12	0.17	13
	85% Passing	0.33	93	0.34	102

Training Wall Armour Design Calculations

Rock Sizings to Resist Wave Action

Structure: Northern Training Wall

Section: Section A: +5.0mAHD Crest Level

Design Criteria:

Component	Value	Comments
H_{100}	3.15 m	Wave Height Exceeded at toe of structure 6 hours every 100 years
Tz	10.7 s	100 years ARI Tp = 15.0s
d	4.6 m	Depth = -2.3mAHD Bed level at toe of structure + 1.44m Storm Surge + 0.9m SLR
cotα	2	Batter Slope
P	0.1	Porosity of Structure
SD	2	Damage Factor
Wr	2600 kg/m3	Dry Rock Density
Storm Duration	6 hours	Typical Wave Event Duration

Rock Armour Specifications (Van Der Meer (1998)):

		Unfactored		Factored by 10%	
		Diameter (m)	Mass (kg)	Diameter (m)	Mass (kg)
Primary Armour	50% Passing	1.72	13,230	1.78	14,550
	15% Passing	1.54	9,470	1.59	10,420
	85% Passing	1.92	18,490	1.99	20,340
Secondary Armour	50% Passing	0.80	1,320	0.82	1,450
	15% Passing	0.71	940	0.73	1,030
	85% Passing	0.89	1,840	0.92	2,020
Core Details	50% Passing	0.29	66	0.30	73
	15% Passing	0.26	47	0.27	52
	85% Passing	0.33	92	0.34	101

Training Wall Armour Design Calculations

Rock Sizings to Resist Wave Action

Structure: Northern Training Wall

Section: Section A: +5.0mAHD Crest Level

Design Criteria:

Component	Value	Comments
H_{100}	3.15 m	Wave Height Exceeded at toe of structure 6 hours every 100 years
Tz	10.7 s	100 years ARI Tp = 15.0s
d	4.6 m	Depth = -2.3mAHD Bed level at toe of structure + 1.44m Storm Surge + 0.9m SLR
cotα	2	Batter Slope
P	0.2	Porosity of Structure
SD	2	Damage Factor
Wr	2600 kg/m3	Dry Rock Density
Storm Duration	6 hours	Typical Wave Event Duration

Rock Armour Specifications (Van Der Meer (1998)):

		Unfactored		Factored by 10%	
		Diameter (m)	Mass (kg)	Diameter (m)	Mass (kg)
Primary Armour	50% Passing	1.63	11,310	1.69	12,440
	15% Passing	1.46	8,090	1.51	8,900
	85% Passing	1.83	15,810	1.88	17,390
Secondary Armour	50% Passing	0.76	1,130	0.78	1,240
	15% Passing	0.68	810	0.70	890
	85% Passing	0.85	1,580	0.87	1,740
Core Details	50% Passing	0.28	57	0.29	62
	15% Passing	0.25	40	0.26	44
	85% Passing	0.31	79	0.32	87

Training Wall Armour Design Calculations

Rock Sizings to Resist Wave Action

Structure: Northern Training Wall

Section: Section A: +5.0mAHD Crest Level

Design Criteria:

Component	Value	Comments
H_{100}	3.15 m	Wave Height Exceeded at toe of structure 6 hours every 100 years
Tz	10.7 s	100 years ARI Tp = 15.0s
d	4.6 m	Depth = -2.3mAHD Bed level at toe of structure + 1.44m Storm Surge + 0.9m SLR
$\cot\alpha$	1.5	Batter Slope
P	0.1	Porosity of Structure
SD	5	Damage Factor
Wr	2600 kg/m3	Dry Rock Density
Storm Duration	6 hours	Typical Wave Event Duration

Rock Armour Specifications (Van Der Meer (1998)):

		Unfactored		Factored by 10%	
		Diameter (m)	Mass (kg)	Diameter (m)	Mass (kg)
Primary Armour	50% Passing	1.61	10,780	1.66	11,860
	15% Passing	1.44	7,710	1.48	8,480
	85% Passing	1.80	15,070	1.85	16,580
Secondary Armour	50% Passing	0.75	1,080	0.77	1,190
	15% Passing	0.67	770	0.69	850
	85% Passing	0.83	1,510	0.86	1,660
Core Details	50% Passing	0.27	54	0.28	59
	15% Passing	0.25	39	0.25	43
	85% Passing	0.31	75	0.32	83

Training Wall Armour Design Calculations

Rock Sizings to Resist Wave Action

Structure: Northern Training Wall

Section: Section A: +5.0mAHD Crest Level

Design Criteria:

Component	Value	Comments
H_{100}	3.15 m	Wave Height Exceeded at toe of structure 6 hours every 100 years
Tz	10.7 s	100 years ARI Tp = 15.0s
d	4.6 m	Depth = -2.3mAHD Bed level at toe of structure + 1.44m Storm Surge + 0.9m SLR
$\cot\alpha$	1.5	Batter Slope
P	0.2	Porosity of Structure
SD	5	Damage Factor
Wr	2600 kg/m3	Dry Rock Density
Storm Duration	6 hours	Typical Wave Event Duration

Rock Armour Specifications (Van Der Meer (1998)):

		Unfactored		Factored by 10%	
		Diameter (m)	Mass (kg)	Diameter (m)	Mass (kg)
Primary Armour	50% Passing	1.48	8,460	1.53	9,310
	15% Passing	1.33	6,050	1.37	6,660
	85% Passing	1.66	11,820	1.71	13,000
Secondary Armour	50% Passing	0.69	850	0.71	940
	15% Passing	0.62	610	0.64	670
	85% Passing	0.77	1,190	0.80	1,310
Core Details	50% Passing	0.25	43	0.26	47
	15% Passing	0.23	30	0.23	33
	85% Passing	0.28	59	0.29	65

Training Wall Armour Design Calculations

Rock Sizings to Resist Wave Action

Structure: Northern Training Wall

Section: Section A: +5.0mAHD Crest Level

Design Criteria:

Component	Value	Comments
H_{100}	3.15 m	Wave Height Exceeded at toe of structure 6 hours every 100 years
Tz	10.7 s	100 years ARI Tp = 15.0s
d	4.6 m	Depth = -2.3mAHD Bed level at toe of structure + 1.44m Storm Surge + 0.9m SLR
cotα	1.5	Batter Slope
P	0.1	Porosity of Structure
SD	2	Damage Factor
Wr	2600 kg/m3	Dry Rock Density
Storm Duration	6 hours	Typical Wave Event Duration

Rock Armour Specifications (Van Der Meer (1998)):

		Unfactored		Factored by 10%	
		Diameter (m)	Mass (kg)	Diameter (m)	Mass (kg)
Primary Armour	50% Passing	1.93	18,690	1.99	20,560
	15% Passing	1.73	13,370	1.78	14,710
	85% Passing	2.16	26,120	2.23	28,730
Secondary Armour	50% Passing	0.90	1,870	0.93	2,060
	15% Passing	0.80	1,340	0.83	1,470
	85% Passing	1.00	2,610	1.03	2,870
Core Details	50% Passing	0.33	94	0.34	103
	15% Passing	0.30	67	0.30	74
	85% Passing	0.37	131	0.38	144

Training Wall Armour Design Calculations

Rock Sizings to Resist Wave Action

Structure: Northern Training Wall

Section: Section A: +5.0mAHD Crest Level

Design Criteria:

Component	Value	Comments
H_{100}	3.15 m	Wave Height Exceeded at toe of structure 6 hours every 100 years
Tz	10.7 s	100 years ARI Tp = 15.0s
d	4.6 m	Depth = -2.3mAHD Bed level at toe of structure + 1.44m Storm Surge + 0.9m SLR
cotα	1.5	Batter Slope
P	0.2	Porosity of Structure
SD	2	Damage Factor
Wr	2600 kg/m3	Dry Rock Density
Storm Duration	6 hours	Typical Wave Event Duration

Rock Armour Specifications (Van Der Meer (1998)):

		Unfactored		Factored by 10%	
		Diameter (m)	Mass (kg)	Diameter (m)	Mass (kg)
Primary Armour	50% Passing	1.78	14,660	1.84	16,130
	15% Passing	1.59	10,490	1.64	11,540
	85% Passing	1.99	20,490	2.05	22,540
Secondary Armour	50% Passing	0.83	1,470	0.85	1,620
	15% Passing	0.74	1,050	0.76	1,160
	85% Passing	0.92	2,050	0.95	2,260
Core Details	50% Passing	0.30	74	0.31	81
	15% Passing	0.27	53	0.28	58
	85% Passing	0.34	103	0.35	113

Training Wall Armour Design Calculations

Rock Sizings to Resist Wave Action

Structure: Northern Training Wall

Section: Section B: +5.0mAHD Crest Level

Design Criteria:

Component	Value	Comments
H_{100}	2.60 m	Wave Height Exceeded at toe of structure 6 hours every 100 years
Tz	10.7 s	100 years ARI Tp = 15.0s
d	4.3 m	Depth = -2.0mAHD Bed level at toe of structure + 1.44m Storm Surge + 0.9m SLR
cotα	2	Batter Slope
P	0.1	Porosity of Structure
SD	5	Damage Factor
Wr	2600 kg/m3	Dry Rock Density
Storm Duration	6 hours	Typical Wave Event Duration

Rock Armour Specifications (Van Der Meer (1998)):

		Unfactored		Factored by 10%	
		Diameter (m)	Mass (kg)	Diameter (m)	Mass (kg)
Primary Armour	50% Passing	1.17	4,170	1.21	4,590
	15% Passing	1.05	2,980	1.08	3,280
	85% Passing	1.31	5,830	1.35	6,410
Secondary Armour	50% Passing	0.54	420	0.56	460
	15% Passing	0.49	300	0.50	330
	85% Passing	0.61	590	0.63	650
Core Details	50% Passing	0.20	21	0.21	23
	15% Passing	0.18	15	0.19	17
	85% Passing	0.22	29	0.23	32

Training Wall Armour Design Calculations

Rock Sizings to Resist Wave Action

Structure: Northern Training Wall

Section: Section B: +5.0mAHD Crest Level

Design Criteria:

Component	Value	Comments
H_{100}	2.60 m	Wave Height Exceeded at toe of structure 6 hours every 100 years
Tz	10.7 s	100 years ARI Tp = 15.0s
d	4.3 m	Depth = -2.0mAHD Bed level at toe of structure + 1.44m Storm Surge + 0.9m SLR
cotα	2	Batter Slope
P	0.2	Porosity of Structure
SD	5	Damage Factor
Wr	2600 kg/m3	Dry Rock Density
Storm Duration	6 hours	Typical Wave Event Duration

Rock Armour Specifications (Van Der Meer (1998)):

		Unfactored		Factored by 10%	
		Diameter (m)	Mass (kg)	Diameter (m)	Mass (kg)
Primary Armour	50% Passing	1.10	3,470	1.14	3,820
	15% Passing	0.98	2,480	1.02	2,730
	85% Passing	1.23	4,850	1.27	5,340
Secondary Armour	50% Passing	0.51	350	0.53	390
	15% Passing	0.46	250	0.48	280
	85% Passing	0.57	490	0.59	540
Core Details	50% Passing	0.19	18	0.19	19
	15% Passing	0.17	13	0.17	14
	85% Passing	0.21	24	0.22	27

Training Wall Armour Design Calculations

Rock Sizings to Resist Wave Action

Structure: Northern Training Wall

Section: Section B: +5.0mAHD Crest Level

Design Criteria:

Component	Value	Comments
H_{100}	2.60 m	Wave Height Exceeded at toe of structure 6 hours every 100 years
Tz	10.7 s	100 years ARI Tp = 15.0s
d	4.3 m	Depth = -2.0mAHD Bed level at toe of structure + 1.44m Storm Surge + 0.9m SLR
cotα	2	Batter Slope
P	0.1	Porosity of Structure
SD	2	Damage Factor
Wr	2600 kg/m3	Dry Rock Density
Storm Duration	6 hours	Typical Wave Event Duration

Rock Armour Specifications (Van Der Meer (1998)):

		Unfactored		Factored by 10%	
		Diameter (m)	Mass (kg)	Diameter (m)	Mass (kg)
Primary Armour	50% Passing	1.41	7,230	1.45	7,950
	15% Passing	1.26	5,170	1.30	5,690
	85% Passing	1.57	10,100	1.62	11,110
Secondary Armour	50% Passing	0.65	720	0.67	790
	15% Passing	0.58	520	0.60	570
	85% Passing	0.73	1,010	0.75	1,110
Core Details	50% Passing	0.24	36	0.25	40
	15% Passing	0.21	26	0.22	28
	85% Passing	0.27	50	0.28	55

Training Wall Armour Design Calculations

Rock Sizings to Resist Wave Action

Structure: Northern Training Wall

Section: Section B: +5.0mAHD Crest Level

Design Criteria:

Component	Value	Comments
H_{100}	2.60 m	Wave Height Exceeded at toe of structure 6 hours every 100 years
Tz	10.7 s	100 years ARI Tp = 15.0s
d	4.3 m	Depth = -2.0mAHD Bed level at toe of structure + 1.44m Storm Surge + 0.9m SLR
cotα	2	Batter Slope
P	0.2	Porosity of Structure
SD	2	Damage Factor
Wr	2600 kg/m3	Dry Rock Density
Storm Duration	6 hours	Typical Wave Event Duration

Rock Armour Specifications (Van Der Meer (1998)):

		Unfactored		Factored by 10%	
		Diameter (m)	Mass (kg)	Diameter (m)	Mass (kg)
Primary Armour	50% Passing	1.32	6,010	1.36	6,610
	15% Passing	1.18	4,300	1.22	4,730
	85% Passing	1.48	8,400	1.53	9,240
Secondary Armour	50% Passing	0.61	600	0.63	660
	15% Passing	0.55	430	0.57	470
	85% Passing	0.69	840	0.71	920
Core Details	50% Passing	0.23	30	0.23	33
	15% Passing	0.20	21	0.21	24
	85% Passing	0.25	42	0.26	46

Training Wall Armour Design Calculations

Rock Sizings to Resist Wave Action

Structure: Northern Training Wall

Section: Section B: +5.0mAHD Crest Level

Design Criteria:

Component	Value	Comments
H_{100}	2.60 m	Wave Height Exceeded at toe of structure 6 hours every 100 years
Tz	10.7 s	100 years ARI Tp = 15.0s
d	4.3 m	Depth = -2.0mAHD Bed level at toe of structure + 1.44m Storm Surge + 0.9m SLR
cotα	1.5	Batter Slope
P	0.1	Porosity of Structure
SD	5	Damage Factor
Wr	2600 kg/m3	Dry Rock Density
Storm Duration	6 hours	Typical Wave Event Duration

Rock Armour Specifications (Van Der Meer (1998)):

		Unfactored		Factored by 10%	
		Diameter (m)	Mass (kg)	Diameter (m)	Mass (kg)
Primary Armour	50% Passing	1.31	5,890	1.36	6,480
	15% Passing	1.17	4,210	1.21	4,630
	85% Passing	1.47	8,230	1.52	9,050
Secondary Armour	50% Passing	0.61	590	0.63	650
	15% Passing	0.55	420	0.56	460
	85% Passing	0.68	820	0.70	900
Core Details	50% Passing	0.22	30	0.23	32
	15% Passing	0.20	21	0.21	23
	85% Passing	0.25	41	0.26	45

Training Wall Armour Design Calculations

Rock Sizings to Resist Wave Action

Structure: Northern Training Wall

Section: Section B: +5.0mAHD Crest Level

Design Criteria:

Component	Value	Comments
H_{100}	2.60 m	Wave Height Exceeded at toe of structure 6 hours every 100 years
Tz	10.7 s	100 years ARI Tp = 15.0s
d	4.3 m	Depth = -2.0mAHD Bed level at toe of structure + 1.44m Storm Surge + 0.9m SLR
$\cot\alpha$	1.5	Batter Slope
P	0.2	Porosity of Structure
SD	5	Damage Factor
Wr	2600 kg/m3	Dry Rock Density
Storm Duration	6 hours	Typical Wave Event Duration

Rock Armour Specifications (Van Der Meer (1998)):

		Unfactored		Factored by 10%	
		Diameter (m)	Mass (kg)	Diameter (m)	Mass (kg)
Primary Armour	50% Passing	1.20	4,490	1.24	4,940
	15% Passing	1.07	3,210	1.11	3,530
	85% Passing	1.34	6,270	1.38	6,900
Secondary Armour	50% Passing	0.56	450	0.58	500
	15% Passing	0.50	320	0.51	350
	85% Passing	0.62	630	0.64	690
Core Details	50% Passing	0.21	23	0.21	25
	15% Passing	0.18	16	0.19	18
	85% Passing	0.23	31	0.24	35

Training Wall Armour Design Calculations

Rock Sizings to Resist Wave Action

Structure: Northern Training Wall

Section: Section B: +5.0mAHD Crest Level

Design Criteria:

Component	Value	Comments
H_{100}	2.60 m	Wave Height Exceeded at toe of structure 6 hours every 100 years
Tz	10.7 s	100 years ARI Tp = 15.0s
d	4.3 m	Depth = -2.0mAHD Bed level at toe of structure + 1.44m Storm Surge + 0.9m SLR
cotα	1.5	Batter Slope
P	0.1	Porosity of Structure
SD	2	Damage Factor
Wr	2600 kg/m3	Dry Rock Density
Storm Duration	6 hours	Typical Wave Event Duration

Rock Armour Specifications (Van Der Meer (1998)):

		Unfactored		Factored by 10%	
		Diameter (m)	Mass (kg)	Diameter (m)	Mass (kg)
Primary Armour	50% Passing	1.58	10,210	1.63	11,230
	15% Passing	1.41	7,310	1.46	8,040
	85% Passing	1.76	14,270	1.82	15,700
Secondary Armour	50% Passing	0.73	1,020	0.76	1,120
	15% Passing	0.65	730	0.68	800
	85% Passing	0.82	1,430	0.85	1,570
Core Details	50% Passing	0.27	51	0.28	56
	15% Passing	0.24	36	0.25	40
	85% Passing	0.30	71	0.31	78

Training Wall Armour Design Calculations

Rock Sizings to Resist Wave Action

Structure: Northern Training Wall

Section: Section B: +5.0mAHD Crest Level

Design Criteria:

Component	Value	Comments
H_{100}	2.60 m	Wave Height Exceeded at toe of structure 6 hours every 100 years
Tz	10.7 s	100 years ARI Tp = 15.0s
d	4.3 m	Depth = -2.0mAHD Bed level at toe of structure + 1.44m Storm Surge + 0.9m SLR
cotα	1.5	Batter Slope
P	0.2	Porosity of Structure
SD	2	Damage Factor
Wr	2600 kg/m3	Dry Rock Density
Storm Duration	6 hours	Typical Wave Event Duration

Rock Armour Specifications (Van Der Meer (1998)):

		Unfactored		Factored by 10%	
		Diameter (m)	Mass (kg)	Diameter (m)	Mass (kg)
Primary Armour	50% Passing	1.44	7,780	1.49	8,560
	15% Passing	1.29	5,570	1.33	6,130
	85% Passing	1.61	10,870	1.66	11,960
Secondary Armour	50% Passing	0.67	780	0.69	860
	15% Passing	0.60	560	0.62	620
	85% Passing	0.75	1,090	0.77	1,200
Core Details	50% Passing	0.25	39	0.25	43
	15% Passing	0.22	28	0.23	31
	85% Passing	0.28	55	0.28	60

Training Wall Armour Design Calculations

Rock Sizings to Resist Wave Action

Structure: Revetment Wall - Karagi Point

Section: Section C: +4.5mAHD Crest Level

Design Criteria:

Component	Value	Comments
H_{100}	0.80 m	Wave Height Exceeded at toe of structure 6 hours every 100 years
Tz	10.7 s	100 years ARI Tp = 15.0s
d	4.1 m	Depth = -1.1mAHD Bed level at toe of structure + 2.3m Flood Level + 0.9m SLR
cotα	2	Batter Slope
P	0.1	Porosity of Structure
SD	5	Damage Factor
Wr	2600 kg/m3	Dry Rock Density
Storm Duration	6 hours	Typical Wave Event Duration

Rock Armour Specifications (Van Der Meer (1998)):

		Unfactored		Factored by 10%	
		Diameter (m)	Mass (kg)	Diameter (m)	Mass (kg)
Primary Armour	50% Passing	0.34	100	0.35	110
	15% Passing	0.30	70	0.31	80
	85% Passing	0.38	140	0.39	150
Secondary Armour	50% Passing	0.16	10	0.16	10
	15% Passing	0.14	10	0.16	10
	85% Passing	0.18	10	0.16	10
Core Details	50% Passing	0.06	1	0.06	1
	15% Passing	0.05	0	0.05	0
	85% Passing	0.06	1	0.07	1

Training Wall Armour Design Calculations

Rock Sizings to Resist Wave Action

Structure: Revetment Wall - Karagi Point

Section: Section C: +4.5mAHD Crest Level

Design Criteria:

Component	Value	Comments
H_{100}	0.80 m	Wave Height Exceeded at toe of structure 6 hours every 100 years
Tz	10.7 s	100 years ARI Tp = 15.0s
d	4.1 m	Depth = -1.1mAHD Bed level at toe of structure + 2.3m Flood Level + 0.9m SLR
cotα	2	Batter Slope
P	0.2	Porosity of Structure
SD	5	Damage Factor
Wr	2600 kg/m3	Dry Rock Density
Storm Duration	6 hours	Typical Wave Event Duration

Rock Armour Specifications (Van Der Meer (1998)):

		Unfactored		Factored by 10%	
		Diameter (m)	Mass (kg)	Diameter (m)	Mass (kg)
Primary Armour	50% Passing	0.30	70	0.31	80
	15% Passing	0.27	50	0.28	60
	85% Passing	0.34	100	0.35	110
Secondary Armour	50% Passing	0.16	10	0.16	10
	15% Passing	0.14	10	0.16	10
	85% Passing	0.18	10	0.16	10
Core Details	50% Passing	0.06	1	0.06	1
	15% Passing	0.05	0	0.05	0
	85% Passing	0.06	1	0.07	1

Training Wall Armour Design Calculations

Rock Sizings to Resist Wave Action

Structure: Revetment Wall - Karagi Point

Section: Section C: +4.5mAHD Crest Level

Design Criteria:

Component	Value	Comments
H_{100}	0.80 m	Wave Height Exceeded at toe of structure 6 hours every 100 years
Tz	10.7 s	100 years ARI Tp = 15.0s
d	4.1 m	Depth = -1.1mAHD Bed level at toe of structure + 2.3m Flood Level + 0.9m SLR
$\cot\alpha$	2	Batter Slope
P	0.1	Porosity of Structure
SD	2	Damage Factor
Wr	2600 kg/m3	Dry Rock Density
Storm Duration	6 hours	Typical Wave Event Duration

Rock Armour Specifications (Van Der Meer (1998)):

		Unfactored		Factored by 10%	
		Diameter (m)	Mass (kg)	Diameter (m)	Mass (kg)
Primary Armour	50% Passing	0.41	180	0.43	200
	15% Passing	0.37	130	0.38	140
	85% Passing	0.46	250	0.48	280
Secondary Armour	50% Passing	0.20	20	0.20	20
	15% Passing	0.18	10	0.16	10
	85% Passing	0.22	30	0.23	30
Core Details	50% Passing	0.07	1	0.08	1
	15% Passing	0.07	1	0.07	1
	85% Passing	0.08	1	0.08	2

Training Wall Armour Design Calculations

Rock Sizings to Resist Wave Action

Structure: Revetment Wall - Karagi Point

Section: Section C: +4.5mAHD Crest Level

Design Criteria:

Component	Value	Comments
H_{100}	0.80 m	Wave Height Exceeded at toe of structure 6 hours every 100 years
Tz	10.7 s	100 years ARI Tp = 15.0s
d	4.1 m	Depth = -1.1mAHD Bed level at toe of structure + 2.3m Flood Level + 0.9m SLR
cotα	2	Batter Slope
P	0.2	Porosity of Structure
SD	2	Damage Factor
Wr	2600 kg/m3	Dry Rock Density
Storm Duration	6 hours	Typical Wave Event Duration

Rock Armour Specifications (Van Der Meer (1998)):

		Unfactored		Factored by 10%	
		Diameter (m)	Mass (kg)	Diameter (m)	Mass (kg)
Primary Armour	50% Passing	0.36	120	0.37	130
	15% Passing	0.32	90	0.34	100
	85% Passing	0.40	170	0.42	190
Secondary Armour	50% Passing	0.16	10	0.16	10
	15% Passing	0.14	10	0.16	10
	85% Passing	0.18	10	0.16	10
Core Details	50% Passing	0.06	1	0.06	1
	15% Passing	0.05	0	0.05	0
	85% Passing	0.06	1	0.07	1

Training Wall Armour Design Calculations

Rock Sizings to Resist Wave Action

Structure: Revetment Wall - Karagi Point

Section: Section C: +4.5mAHD Crest Level

Design Criteria:

Component	Value	Comments
H_{100}	0.80 m	Wave Height Exceeded at toe of structure 6 hours every 100 years
Tz	10.7 s	100 years ARI Tp = 15.0s
d	4.1 m	Depth = -1.1mAHD Bed level at toe of structure + 2.3m Flood Level + 0.9m SLR
$\cot\alpha$	1.5	Batter Slope
P	0.1	Porosity of Structure
SD	5	Damage Factor
Wr	2600 kg/m3	Dry Rock Density
Storm Duration	6 hours	Typical Wave Event Duration

Rock Armour Specifications (Van Der Meer (1998)):

		Unfactored		Factored by 10%	
		Diameter (m)	Mass (kg)	Diameter (m)	Mass (kg)
Primary Armour	50% Passing	0.39	150	0.40	170
	15% Passing	0.35	110	0.36	120
	85% Passing	0.43	210	0.45	230
Secondary Armour	50% Passing	0.20	20	0.20	20
	15% Passing	0.18	10	0.16	10
	85% Passing	0.22	30	0.23	30
Core Details	50% Passing	0.07	1	0.08	1
	15% Passing	0.07	1	0.07	1
	85% Passing	0.08	1	0.08	2

Training Wall Armour Design Calculations

Rock Sizings to Resist Wave Action

Structure: Revetment Wall - Karagi Point

Section: Section C: +4.5mAHD Crest Level

Design Criteria:

Component	Value	Comments
H_{100}	0.80 m	Wave Height Exceeded at toe of structure 6 hours every 100 years
Tz	10.7 s	100 years ARI Tp = 15.0s
d	4.1 m	Depth = -1.1mAHD Bed level at toe of structure + 2.3m Flood Level + 0.9m SLR
cotα	1.5	Batter Slope
P	0.2	Porosity of Structure
SD	5	Damage Factor
Wr	2600 kg/m3	Dry Rock Density
Storm Duration	6 hours	Typical Wave Event Duration

Rock Armour Specifications (Van Der Meer (1998)):

		Unfactored		Factored by 10%	
		Diameter (m)	Mass (kg)	Diameter (m)	Mass (kg)
Primary Armour	50% Passing	0.33	90	0.34	100
	15% Passing	0.29	60	0.30	70
	85% Passing	0.36	130	0.38	140
Secondary Armour	50% Passing	0.16	10	0.16	10
	15% Passing	0.14	10	0.16	10
	85% Passing	0.18	10	0.16	10
Core Details	50% Passing	0.06	1	0.06	1
	15% Passing	0.05	0	0.05	0
	85% Passing	0.06	1	0.07	1

Training Wall Armour Design Calculations

Rock Sizings to Resist Wave Action

Structure: Revetment Wall - Karagi Point

Section: Section C: +4.5mAHD Crest Level

Design Criteria:

Component	Value	Comments
H_{100}	0.80 m	Wave Height Exceeded at toe of structure 6 hours every 100 years
Tz	10.7 s	100 years ARI Tp = 15.0s
d	4.1 m	Depth = -1.1mAHD Bed level at toe of structure + 2.3m Flood Level + 0.9m SLR
cotα	1.5	Batter Slope
P	0.1	Porosity of Structure
SD	2	Damage Factor
Wr	2600 kg/m3	Dry Rock Density
Storm Duration	6 hours	Typical Wave Event Duration

Rock Armour Specifications (Van Der Meer (1998)):

		Unfactored		Factored by 10%	
		Diameter (m)	Mass (kg)	Diameter (m)	Mass (kg)
Primary Armour	50% Passing	0.46	250	0.48	280
	15% Passing	0.41	180	0.43	200
	85% Passing	0.51	350	0.53	390
Secondary Armour	50% Passing	0.23	30	0.23	30
	15% Passing	0.20	20	0.20	20
	85% Passing	0.25	40	0.25	40
Core Details	50% Passing	0.08	2	0.09	2
	15% Passing	0.07	1	0.08	1
	85% Passing	0.09	2	0.10	2

Training Wall Armour Design Calculations

Rock Sizings to Resist Wave Action

Structure: Revetment Wall - Karagi Point

Section: Section C: +4.5mAHD Crest Level

Design Criteria:

Component	Value	Comments
H_{100}	0.80 m	Wave Height Exceeded at toe of structure 6 hours every 100 years
Tz	10.7 s	100 years ARI Tp = 15.0s
d	4.1 m	Depth = -1.1mAHD Bed level at toe of structure + 2.3m Flood Level + 0.9m SLR
cotα	1.5	Batter Slope
P	0.2	Porosity of Structure
SD	2	Damage Factor
Wr	2600 kg/m3	Dry Rock Density
Storm Duration	6 hours	Typical Wave Event Duration

Rock Armour Specifications (Van Der Meer (1998)):

		Unfactored		Factored by 10%	
		Diameter (m)	Mass (kg)	Diameter (m)	Mass (kg)
Primary Armour	50% Passing	0.39	160	0.41	180
	15% Passing	0.35	110	0.36	120
	85% Passing	0.44	220	0.45	240
Secondary Armour	50% Passing	0.20	20	0.20	20
	15% Passing	0.18	10	0.16	10
	85% Passing	0.22	30	0.23	30
Core Details	50% Passing	0.07	1	0.08	1
	15% Passing	0.07	1	0.07	1
	85% Passing	0.08	1	0.08	2

Training Wall Armour Design Calculations

Rock Sizings to Resist Wave Action

Structure: Revetment Walll - Karagi Park to Entrance Bridge

Section: Section D: +3.6mAHD Crest Level

Design Criteria:

Component	Value	Comments
H_{100}	0.50 m	Wave Height Exceeded at toe of structure 6 hours every 100 years
Tz	10.7 s	100 years ARI Tp = 15.0s
d	4.1 m	Depth = -1.1mAHD Bed level at toe of structure + 2.3m Flood Level + 0.9m SLR
$\cot\alpha$	2	Batter Slope
P	0.1	Porosity of Structure
SD	5	Damage Factor
Wr	2600 kg/m3	Dry Rock Density
Storm Duration	6 hours	Typical Wave Event Duration

Rock Armour Specifications (Van Der Meer (1998)):

		Unfactored		Factored by 10%	
		Diameter (m)	Mass (kg)	Diameter (m)	Mass (kg)
Primary Armour	50% Passing	0.20	20	0.20	20
	15% Passing	0.18	10	0.16	10
	85% Passing	0.22	30	0.23	30
Secondary Armour	50% Passing	0.00	0	0.00	0
	15% Passing	0.00	0	0.00	0
	85% Passing	0.00	0	0.00	0
Core Details	50% Passing	0.00	0	0.00	0
	15% Passing	0.00	0	0.00	0
	85% Passing	0.00	0	0.00	0

Training Wall Armour Design Calculations

Rock Sizings to Resist Wave Action

Structure: Revetment Walll - Karagi Park to Entrance Bridge

Section: Section D: +3.6mAHD Crest Level

Design Criteria:

Component	Value	Comments
H_{100}	0.50 m	Wave Height Exceeded at toe of structure 6 hours every 100 years
Tz	10.7 s	100 years ARI Tp = 15.0s
d	4.1 m	Depth = -1.1mAHD Bed level at toe of structure + 2.3m Flood Level + 0.9m SLR
cotα	2	Batter Slope
P	0.2	Porosity of Structure
SD	5	Damage Factor
Wr	2600 kg/m3	Dry Rock Density
Storm Duration	6 hours	Typical Wave Event Duration

Rock Armour Specifications (Van Der Meer (1998)):

		Unfactored		Factored by 10%	
		Diameter (m)	Mass (kg)	Diameter (m)	Mass (kg)
Primary Armour	50% Passing	0.20	20	0.20	20
	15% Passing	0.18	10	0.16	10
	85% Passing	0.22	30	0.23	30
Secondary Armour	50% Passing	0.00	0	0.00	0
	15% Passing	0.00	0	0.00	0
	85% Passing	0.00	0	0.00	0
Core Details	50% Passing	0.00	0	0.00	0
	15% Passing	0.00	0	0.00	0
	85% Passing	0.00	0	0.00	0

Training Wall Armour Design Calculations

Rock Sizings to Resist Wave Action

Structure: Revetment Walll - Karagi Park to Entrance Bridge

Section: Section D: +3.6mAHD Crest Level

Design Criteria:

Component	Value	Comments
H_{100}	0.50 m	Wave Height Exceeded at toe of structure 6 hours every 100 years
Tz	10.7 s	100 years ARI Tp = 15.0s
d	4.1 m	Depth = -1.1mAHD Bed level at toe of structure + 2.3m Flood Level + 0.9m SLR
$\cot\alpha$	2	Batter Slope
P	0.1	Porosity of Structure
SD	2	Damage Factor
Wr	2600 kg/m3	Dry Rock Density
Storm Duration	6 hours	Typical Wave Event Duration

Rock Armour Specifications (Van Der Meer (1998)):

		Unfactored		Factored by 10%	
		Diameter (m)	Mass (kg)	Diameter (m)	Mass (kg)
Primary Armour	50% Passing	0.25	40	0.25	40
	15% Passing	0.22	30	0.23	30
	85% Passing	0.28	60	0.30	70
Secondary Armour	50% Passing	0.00	0	0.00	0
	15% Passing	0.00	0	0.00	0
	85% Passing	0.00	0	0.00	0
Core Details	50% Passing	0.00	0	0.00	0
	15% Passing	0.00	0	0.00	0
	85% Passing	0.00	0	0.00	0

Training Wall Armour Design Calculations

Rock Sizings to Resist Wave Action

Structure: Revetment Walll - Karagi Park to Entrance Bridge

Section: Section D: +3.6mAHD Crest Level

Design Criteria:

Component	Value	Comments
H_{100}	0.50 m	Wave Height Exceeded at toe of structure 6 hours every 100 years
Tz	10.7 s	100 years ARI Tp = 15.0s
d	4.1 m	Depth = -1.1mAHD Bed level at toe of structure + 2.3m Flood Level + 0.9m SLR
cotα	2	Batter Slope
P	0.2	Porosity of Structure
SD	2	Damage Factor
Wr	2600 kg/m3	Dry Rock Density
Storm Duration	6 hours	Typical Wave Event Duration

Rock Armour Specifications (Van Der Meer (1998)):

		Unfactored		Factored by 10%	
		Diameter (m)	Mass (kg)	Diameter (m)	Mass (kg)
Primary Armour	50% Passing	0.23	30	0.23	30
	15% Passing	0.20	20	0.20	20
	85% Passing	0.25	40	0.25	40
Secondary Armour	50% Passing	0.00	0	0.00	0
	15% Passing	0.00	0	0.00	0
	85% Passing	0.00	0	0.00	0
Core Details	50% Passing	0.00	0	0.00	0
	15% Passing	0.00	0	0.00	0
	85% Passing	0.00	0	0.00	0

Training Wall Armour Design Calculations

Rock Sizings to Resist Wave Action

Structure: Revetment Walll - Karagi Park to Entrance Bridge

Section: Section D: +3.6mAHD Crest Level

Design Criteria:

Component	Value	Comments
H_{100}	0.50 m	Wave Height Exceeded at toe of structure 6 hours every 100 years
Tz	10.7 s	100 years ARI Tp = 15.0s
d	4.1 m	Depth = -1.1mAHD Bed level at toe of structure + 2.3m Flood Level + 0.9m SLR
cotα	1.5	Batter Slope
P	0.1	Porosity of Structure
SD	5	Damage Factor
Wr	2600 kg/m3	Dry Rock Density
Storm Duration	6 hours	Typical Wave Event Duration

Rock Armour Specifications (Van Der Meer (1998)):

		Unfactored		Factored by 10%	
		Diameter (m)	Mass (kg)	Diameter (m)	Mass (kg)
Primary Armour	50% Passing	0.23	30	0.23	30
	15% Passing	0.20	20	0.20	20
	85% Passing	0.25	40	0.25	40
Secondary Armour	50% Passing	0.00	0	0.00	0
	15% Passing	0.00	0	0.00	0
	85% Passing	0.00	0	0.00	0
Core Details	50% Passing	0.00	0	0.00	0
	15% Passing	0.00	0	0.00	0
	85% Passing	0.00	0	0.00	0

Training Wall Armour Design Calculations

Rock Sizings to Resist Wave Action

Structure: Revetment Walll - Karagi Park to Entrance Bridge

Section: Section D: +3.6mAHD Crest Level

Design Criteria:

Component	Value	Comments
H_{100}	0.50 m	Wave Height Exceeded at toe of structure 6 hours every 100 years
Tz	10.7 s	100 years ARI Tp = 15.0s
d	4.1 m	Depth = -1.1mAHD Bed level at toe of structure + 2.3m Flood Level + 0.9m SLR
cotα	1.5	Batter Slope
P	0.2	Porosity of Structure
SD	5	Damage Factor
Wr	2600 kg/m3	Dry Rock Density
Storm Duration	6 hours	Typical Wave Event Duration

Rock Armour Specifications (Van Der Meer (1998)):

		Unfactored		Factored by 10%	
		Diameter (m)	Mass (kg)	Diameter (m)	Mass (kg)
Primary Armour	50% Passing	0.20	20	0.20	20
	15% Passing	0.18	10	0.16	10
	85% Passing	0.22	30	0.23	30
Secondary Armour	50% Passing	0.00	0	0.00	0
	15% Passing	0.00	0	0.00	0
	85% Passing	0.00	0	0.00	0
Core Details	50% Passing	0.00	0	0.00	0
	15% Passing	0.00	0	0.00	0
	85% Passing	0.00	0	0.00	0

Training Wall Armour Design Calculations

Rock Sizings to Resist Wave Action

Structure: Revetment Walll - Karagi Park to Entrance Bridge

Section: Section D: +3.6mAHD Crest Level

Design Criteria:

Component	Value	Comments
H_{100}	0.50 m	Wave Height Exceeded at toe of structure 6 hours every 100 years
Tz	10.7 s	100 years ARI Tp = 15.0s
d	4.1 m	Depth = -1.1mAHD Bed level at toe of structure + 2.3m Flood Level + 0.9m SLR
cotα	1.5	Batter Slope
P	0.1	Porosity of Structure
SD	2	Damage Factor
Wr	2600 kg/m3	Dry Rock Density
Storm Duration	6 hours	Typical Wave Event Duration

Rock Armour Specifications (Van Der Meer (1998)):

		Unfactored		Factored by 10%	
		Diameter (m)	Mass (kg)	Diameter (m)	Mass (kg)
Primary Armour	50% Passing	0.28	60	0.30	70
	15% Passing	0.25	40	0.25	40
	85% Passing	0.32	80	0.33	90
Secondary Armour	50% Passing	0.16	10	0.16	10
	15% Passing	0.14	10	0.16	10
	85% Passing	0.18	10	0.16	10
Core Details	50% Passing	0.06	1	0.06	1
	15% Passing	0.05	0	0.05	0
	85% Passing	0.06	1	0.07	1

Training Wall Armour Design Calculations

Rock Sizings to Resist Wave Action

Structure: Revetment Walll - Karagi Park to Entrance Bridge

Section: Section D: +3.6mAHD Crest Level

Design Criteria:

Component	Value	Comments
H_{100}	0.50 m	Wave Height Exceeded at toe of structure 6 hours every 100 years
Tz	10.7 s	100 years ARI Tp = 15.0s
d	4.1 m	Depth = -1.1mAHD Bed level at toe of structure + 2.3m Flood Level + 0.9m SLR
cotα	1.5	Batter Slope
P	0.2	Porosity of Structure
SD	2	Damage Factor
Wr	2600 kg/m3	Dry Rock Density
Storm Duration	6 hours	Typical Wave Event Duration

Rock Armour Specifications (Van Der Meer (1998)):

		Unfactored		Factored by 10%	
		Diameter (m)	Mass (kg)	Diameter (m)	Mass (kg)
Primary Armour	50% Passing	0.23	30	0.23	30
	15% Passing	0.20	20	0.20	20
	85% Passing	0.25	40	0.25	40
Secondary Armour	50% Passing	0.00	0	0.00	0
	15% Passing	0.00	0	0.00	0
	85% Passing	0.00	0	0.00	0
Core Details	50% Passing	0.00	0	0.00	0
	15% Passing	0.00	0	0.00	0
	85% Passing	0.00	0	0.00	0

Training Wall Armour Design Calculations

Rock Sizings to Resist Wave Action

Structure: Southern Training Wall

Section: Section E: +5.0mAHD Crest Level

Design Criteria:

Component	Value	Comments
H_{100}	3.10 m	Wave Height Exceeded at toe of structure 6 hours every 100 years
Tz	10.7 s	100 years ARI Tp = 15.0s
d	4.3 m	Depth = -2.0mAHD Bed level at toe of structure + 1.44m Storm Surge + 0.9m SLR
$\cot\alpha$	2	Batter Slope
P	0.1	Porosity of Structure
SD	5	Damage Factor
Wr	2600 kg/m3	Dry Rock Density
Storm Duration	6 hours	Typical Wave Event Duration

Rock Armour Specifications (Van Der Meer (1998)):

		Unfactored		Factored by 10%	
		Diameter (m)	Mass (kg)	Diameter (m)	Mass (kg)
Primary Armour	50% Passing	1.41	7,260	1.45	7,990
	15% Passing	1.26	5,190	1.30	5,710
	85% Passing	1.57	10,150	1.63	11,170
Secondary Armour	50% Passing	0.65	730	0.68	800
	15% Passing	0.59	520	0.60	570
	85% Passing	0.73	1,020	0.76	1,120
Core Details	50% Passing	0.24	37	0.25	40
	15% Passing	0.22	26	0.22	29
	85% Passing	0.27	51	0.28	56

Training Wall Armour Design Calculations

Rock Sizings to Resist Wave Action

Structure: Southern Training Wall

Section: Section E: +5.0mAHD Crest Level

Design Criteria:

Component	Value	Comments
H_{100}	3.10 m	Wave Height Exceeded at toe of structure 6 hours every 100 years
Tz	10.7 s	100 years ARI Tp = 15.0s
d	4.3 m	Depth = -2.0mAHD Bed level at toe of structure + 1.44m Storm Surge + 0.9m SLR
cotα	2	Batter Slope
P	0.2	Porosity of Structure
SD	5	Damage Factor
Wr	2600 kg/m3	Dry Rock Density
Storm Duration	6 hours	Typical Wave Event Duration

Rock Armour Specifications (Van Der Meer (1998)):

		Unfactored		Factored by 10%	
		Diameter (m)	Mass (kg)	Diameter (m)	Mass (kg)
Primary Armour	50% Passing	1.34	6,190	1.38	6,810
	15% Passing	1.19	4,430	1.23	4,870
	85% Passing	1.49	8,650	1.54	9,520
Secondary Armour	50% Passing	0.62	620	0.64	680
	15% Passing	0.55	440	0.57	480
	85% Passing	0.69	870	0.72	960
Core Details	50% Passing	0.23	31	0.24	34
	15% Passing	0.20	22	0.21	24
	85% Passing	0.26	43	0.26	48

Training Wall Armour Design Calculations

Rock Sizings to Resist Wave Action

Structure: Southern Training Wall

Section: Section E: +5.0mAHD Crest Level

Design Criteria:

Component	Value	Comments
H_{100}	3.10 m	Wave Height Exceeded at toe of structure 6 hours every 100 years
Tz	10.7 s	100 years ARI Tp = 15.0s
d	4.3 m	Depth = -2.0mAHD Bed level at toe of structure + 1.44m Storm Surge + 0.9m SLR
cotα	2	Batter Slope
P	0.1	Porosity of Structure
SD	2	Damage Factor
Wr	2600 kg/m3	Dry Rock Density
Storm Duration	6 hours	Typical Wave Event Duration

Rock Armour Specifications (Van Der Meer (1998)):

		Unfactored		Factored by 10%	
		Diameter (m)	Mass (kg)	Diameter (m)	Mass (kg)
Primary Armour	50% Passing	1.69	12,580	1.75	13,840
	15% Passing	1.51	9,000	1.56	9,900
	85% Passing	1.89	17,580	1.95	19,340
Secondary Armour	50% Passing	0.79	1,260	0.81	1,390
	15% Passing	0.70	900	0.72	990
	85% Passing	0.88	1,760	0.91	1,940
Core Details	50% Passing	0.29	63	0.30	69
	15% Passing	0.26	45	0.27	50
	85% Passing	0.32	88	0.33	97

Training Wall Armour Design Calculations

Rock Sizings to Resist Wave Action

Structure: Southern Training Wall

Section: Section E: +5.0mAHD Crest Level

Design Criteria:

Component	Value	Comments
H_{100}	3.10 m	Wave Height Exceeded at toe of structure 6 hours every 100 years
Tz	10.7 s	100 years ARI Tp = 15.0s
d	4.3 m	Depth = -2.0mAHD Bed level at toe of structure + 1.44m Storm Surge + 0.9m SLR
cotα	2	Batter Slope
P	0.2	Porosity of Structure
SD	2	Damage Factor
Wr	2600 kg/m3	Dry Rock Density
Storm Duration	6 hours	Typical Wave Event Duration

Rock Armour Specifications (Van Der Meer (1998)):

		Unfactored		Factored by 10%	
		Diameter (m)	Mass (kg)	Diameter (m)	Mass (kg)
Primary Armour	50% Passing	1.60	10,730	1.66	11,800
	15% Passing	1.43	7,680	1.48	8,450
	85% Passing	1.79	15,000	1.85	16,500
Secondary Armour	50% Passing	0.74	1,070	0.77	1,180
	15% Passing	0.67	770	0.69	850
	85% Passing	0.83	1,500	0.86	1,650
Core Details	50% Passing	0.27	54	0.28	59
	15% Passing	0.25	38	0.25	42
	85% Passing	0.31	75	0.32	82

Training Wall Armour Design Calculations

Rock Sizings to Resist Wave Action

Structure: Southern Training Wall

Section: Section E: +5.0mAHD Crest Level

Design Criteria:

Component	Value	Comments
H_{100}	3.10 m	Wave Height Exceeded at toe of structure 6 hours every 100 years
Tz	10.7 s	100 years ARI Tp = 15.0s
d	4.3 m	Depth = -2.0mAHD Bed level at toe of structure + 1.44m Storm Surge + 0.9m SLR
cotα	1.5	Batter Slope
P	0.1	Porosity of Structure
SD	5	Damage Factor
Wr	2600 kg/m3	Dry Rock Density
Storm Duration	6 hours	Typical Wave Event Duration

Rock Armour Specifications (Van Der Meer (1998)):

		Unfactored		Factored by 10%	
		Diameter (m)	Mass (kg)	Diameter (m)	Mass (kg)
Primary Armour	50% Passing	1.58	10,250	1.63	11,280
	15% Passing	1.41	7,330	1.46	8,060
	85% Passing	1.77	14,320	1.82	15,750
Secondary Armour	50% Passing	0.73	1,030	0.76	1,130
	15% Passing	0.66	740	0.68	810
	85% Passing	0.82	1,440	0.85	1,580
Core Details	50% Passing	0.27	52	0.28	57
	15% Passing	0.24	37	0.25	41
	85% Passing	0.30	72	0.31	79

Training Wall Armour Design Calculations

Rock Sizings to Resist Wave Action

Structure: Southern Training Wall

Section: Section E: +5.0mAHD Crest Level

Design Criteria:

Component	Value	Comments
H_{100}	3.10 m	Wave Height Exceeded at toe of structure 6 hours every 100 years
Tz	10.7 s	100 years ARI Tp = 15.0s
d	4.3 m	Depth = -2.0mAHD Bed level at toe of structure + 1.44m Storm Surge + 0.9m SLR
$\cot\alpha$	1.5	Batter Slope
P	0.2	Porosity of Structure
SD	5	Damage Factor
Wr	2600 kg/m3	Dry Rock Density
Storm Duration	6 hours	Typical Wave Event Duration

Rock Armour Specifications (Van Der Meer (1998)):

		Unfactored		Factored by 10%	
		Diameter (m)	Mass (kg)	Diameter (m)	Mass (kg)
Primary Armour	50% Passing	1.46	8,020	1.50	8,820
	15% Passing	1.30	5,740	1.34	6,310
	85% Passing	1.63	11,210	1.68	12,330
Secondary Armour	50% Passing	0.68	800	0.70	880
	15% Passing	0.60	570	0.62	630
	85% Passing	0.75	1,120	0.78	1,230
Core Details	50% Passing	0.25	40	0.26	44
	15% Passing	0.22	29	0.23	31
	85% Passing	0.28	56	0.29	61

Training Wall Armour Design Calculations

Rock Sizings to Resist Wave Action

Structure: Southern Training Wall

Section: Section E: +5.0mAHD Crest Level

Design Criteria:

Component	Value	Comments
H_{100}	3.10 m	Wave Height Exceeded at toe of structure 6 hours every 100 years
Tz	10.7 s	100 years ARI Tp = 15.0s
d	4.3 m	Depth = -2.0mAHD Bed level at toe of structure + 1.44m Storm Surge + 0.9m SLR
cotα	1.5	Batter Slope
P	0.1	Porosity of Structure
SD	2	Damage Factor
Wr	2600 kg/m3	Dry Rock Density
Storm Duration	6 hours	Typical Wave Event Duration

Rock Armour Specifications (Van Der Meer (1998)):

		Unfactored		Factored by 10%	
		Diameter (m)	Mass (kg)	Diameter (m)	Mass (kg)
Primary Armour	50% Passing	1.90	17,770	1.96	19,550
	15% Passing	1.70	12,720	1.75	13,990
	85% Passing	2.12	24,830	2.19	27,310
Secondary Armour	50% Passing	0.88	1,780	0.91	1,960
	15% Passing	0.79	1,270	0.81	1,400
	85% Passing	0.99	2,490	1.02	2,740
Core Details	50% Passing	0.32	89	0.34	98
	15% Passing	0.29	64	0.30	70
	85% Passing	0.36	124	0.37	137

Training Wall Armour Design Calculations

Rock Sizings to Resist Wave Action

Structure: Southern Training Wall

Section: Section E: +5.0mAHD Crest Level

Design Criteria:

Component	Value	Comments
H_{100}	3.10 m	Wave Height Exceeded at toe of structure 6 hours every 100 years
Tz	10.7 s	100 years ARI Tp = 15.0s
d	4.3 m	Depth = -2.0mAHD Bed level at toe of structure + 1.44m Storm Surge + 0.9m SLR
cotα	1.5	Batter Slope
P	0.2	Porosity of Structure
SD	2	Damage Factor
Wr	2600 kg/m3	Dry Rock Density
Storm Duration	6 hours	Typical Wave Event Duration

Rock Armour Specifications (Van Der Meer (1998)):

		Unfactored		Factored by 10%	
		Diameter (m)	Mass (kg)	Diameter (m)	Mass (kg)
Primary Armour	50% Passing	1.75	13,900	1.81	15,290
	15% Passing	1.56	9,950	1.61	10,950
	85% Passing	1.95	19,430	2.02	21,370
Secondary Armour	50% Passing	0.81	1,390	0.84	1,530
	15% Passing	0.73	990	0.75	1,090
	85% Passing	0.91	1,940	0.94	2,130
Core Details	50% Passing	0.30	70	0.31	76
	15% Passing	0.27	50	0.28	55
	85% Passing	0.33	97	0.35	107

Training Wall Armour Design Calculations

Rock Sizings to Resist Wave Action

Structure: Southern Training Wall

Section: Section F: +5.0mAHD Crest Level

Design Criteria:

Component	Value	Comments
H_{100}	2.60 m	Wave Height Exceeded at toe of structure 6 hours every 100 years
Tz	10.7 s	100 years ARI Tp = 15.0s
d	3.3 m	Depth = -1.0mAHD Bed level at toe of structure + 1.44m Storm Surge + 0.9m SLR
cotα	2	Batter Slope
P	0.1	Porosity of Structure
SD	5	Damage Factor
Wr	2600 kg/m3	Dry Rock Density
Storm Duration	6 hours	Typical Wave Event Duration

Rock Armour Specifications (Van Der Meer (1998)):

		Unfactored		Factored by 10%	
		Diameter (m)	Mass (kg)	Diameter (m)	Mass (kg)
Primary Armour	50% Passing	1.17	4,170	1.21	4,590
	15% Passing	1.05	2,980	1.08	3,280
	85% Passing	1.31	5,830	1.35	6,410
Secondary Armour	50% Passing	0.54	420	0.56	460
	15% Passing	0.49	300	0.50	330
	85% Passing	0.61	590	0.63	650
Core Details	50% Passing	0.20	21	0.21	23
	15% Passing	0.18	15	0.19	17
	85% Passing	0.22	29	0.23	32

Training Wall Armour Design Calculations

Rock Sizings to Resist Wave Action

Structure: Southern Training Wall

Section: Section F: +5.0mAHD Crest Level

Design Criteria:

Component	Value	Comments
H_{100}	2.60 m	Wave Height Exceeded at toe of structure 6 hours every 100 years
Tz	10.7 s	100 years ARI Tp = 15.0s
d	3.3 m	Depth = -1.0mAHD Bed level at toe of structure + 1.44m Storm Surge + 0.9m SLR
cotα	2	Batter Slope
P	0.2	Porosity of Structure
SD	5	Damage Factor
Wr	2600 kg/m3	Dry Rock Density
Storm Duration	6 hours	Typical Wave Event Duration

Rock Armour Specifications (Van Der Meer (1998)):

		Unfactored		Factored by 10%	
		Diameter (m)	Mass (kg)	Diameter (m)	Mass (kg)
Primary Armour	50% Passing	1.10	3,470	1.14	3,820
	15% Passing	0.98	2,480	1.02	2,730
	85% Passing	1.23	4,850	1.27	5,340
Secondary Armour	50% Passing	0.51	350	0.53	390
	15% Passing	0.46	250	0.48	280
	85% Passing	0.57	490	0.59	540
Core Details	50% Passing	0.19	18	0.19	19
	15% Passing	0.17	13	0.17	14
	85% Passing	0.21	24	0.22	27

Training Wall Armour Design Calculations

Rock Sizings to Resist Wave Action

Structure: Southern Training Wall

Section: Section F: +5.0mAHD Crest Level

Design Criteria:

Component	Value	Comments
H_{100}	2.60 m	Wave Height Exceeded at toe of structure 6 hours every 100 years
Tz	10.7 s	100 years ARI Tp = 15.0s
d	3.3 m	Depth = -1.0mAHD Bed level at toe of structure + 1.44m Storm Surge + 0.9m SLR
cotα	2	Batter Slope
P	0.1	Porosity of Structure
SD	2	Damage Factor
Wr	2600 kg/m3	Dry Rock Density
Storm Duration	6 hours	Typical Wave Event Duration

Rock Armour Specifications (Van Der Meer (1998)):

		Unfactored		Factored by 10%	
		Diameter (m)	Mass (kg)	Diameter (m)	Mass (kg)
Primary Armour	50% Passing	1.41	7,230	1.45	7,950
	15% Passing	1.26	5,170	1.30	5,690
	85% Passing	1.57	10,100	1.62	11,110
Secondary Armour	50% Passing	0.65	720	0.67	790
	15% Passing	0.58	520	0.60	570
	85% Passing	0.73	1,010	0.75	1,110
Core Details	50% Passing	0.24	36	0.25	40
	15% Passing	0.21	26	0.22	28
	85% Passing	0.27	50	0.28	55

Training Wall Armour Design Calculations

Rock Sizings to Resist Wave Action

Structure: Southern Training Wall

Section: Section F: +5.0mAHD Crest Level

Design Criteria:

Component	Value	Comments
H_{100}	2.60 m	Wave Height Exceeded at toe of structure 6 hours every 100 years
Tz	10.7 s	100 years ARI Tp = 15.0s
d	3.3 m	Depth = -1.0mAHD Bed level at toe of structure + 1.44m Storm Surge + 0.9m SLR
cotα	2	Batter Slope
P	0.2	Porosity of Structure
SD	2	Damage Factor
Wr	2600 kg/m3	Dry Rock Density
Storm Duration	6 hours	Typical Wave Event Duration

Rock Armour Specifications (Van Der Meer (1998)):

		Unfactored		Factored by 10%	
		Diameter (m)	Mass (kg)	Diameter (m)	Mass (kg)
Primary Armour	50% Passing	1.32	6,010	1.36	6,610
	15% Passing	1.18	4,300	1.22	4,730
	85% Passing	1.48	8,400	1.53	9,240
Secondary Armour	50% Passing	0.61	600	0.63	660
	15% Passing	0.55	430	0.57	470
	85% Passing	0.69	840	0.71	920
Core Details	50% Passing	0.23	30	0.23	33
	15% Passing	0.20	21	0.21	24
	85% Passing	0.25	42	0.26	46

Training Wall Armour Design Calculations

Rock Sizings to Resist Wave Action

Structure: Southern Training Wall

Section: Section F: +5.0mAHD Crest Level

Design Criteria:

Component	Value	Comments
H_{100}	2.60 m	Wave Height Exceeded at toe of structure 6 hours every 100 years
Tz	10.7 s	100 years ARI Tp = 15.0s
d	3.3 m	Depth = -1.0mAHD Bed level at toe of structure + 1.44m Storm Surge + 0.9m SLR
$\cot\alpha$	1.5	Batter Slope
P	0.1	Porosity of Structure
SD	5	Damage Factor
Wr	2600 kg/m3	Dry Rock Density
Storm Duration	6 hours	Typical Wave Event Duration

Rock Armour Specifications (Van Der Meer (1998)):

		Unfactored		Factored by 10%	
		Diameter (m)	Mass (kg)	Diameter (m)	Mass (kg)
Primary Armour	50% Passing	1.31	5,890	1.36	6,480
	15% Passing	1.17	4,210	1.21	4,630
	85% Passing	1.47	8,230	1.52	9,050
Secondary Armour	50% Passing	0.61	590	0.63	650
	15% Passing	0.55	420	0.56	460
	85% Passing	0.68	820	0.70	900
Core Details	50% Passing	0.22	30	0.23	32
	15% Passing	0.20	21	0.21	23
	85% Passing	0.25	41	0.26	45

Training Wall Armour Design Calculations

Rock Sizings to Resist Wave Action

Structure: Southern Training Wall

Section: Section F: +5.0mAHD Crest Level

Design Criteria:

Component	Value	Comments
H_{100}	2.60 m	Wave Height Exceeded at toe of structure 6 hours every 100 years
Tz	10.7 s	100 years ARI Tp = 15.0s
d	3.3 m	Depth = -1.0mAHD Bed level at toe of structure + 1.44m Storm Surge + 0.9m SLR
$\cot\alpha$	1.5	Batter Slope
P	0.2	Porosity of Structure
SD	5	Damage Factor
Wr	2600 kg/m3	Dry Rock Density
Storm Duration	6 hours	Typical Wave Event Duration

Rock Armour Specifications (Van Der Meer (1998)):

		Unfactored		Factored by 10%	
		Diameter (m)	Mass (kg)	Diameter (m)	Mass (kg)
Primary Armour	50% Passing	1.20	4,490	1.24	4,940
	15% Passing	1.07	3,210	1.11	3,530
	85% Passing	1.34	6,270	1.38	6,900
Secondary Armour	50% Passing	0.56	450	0.58	500
	15% Passing	0.50	320	0.51	350
	85% Passing	0.62	630	0.64	690
Core Details	50% Passing	0.21	23	0.21	25
	15% Passing	0.18	16	0.19	18
	85% Passing	0.23	31	0.24	35

Training Wall Armour Design Calculations

Rock Sizings to Resist Wave Action

Structure: Southern Training Wall

Section: Section F: +5.0mAHD Crest Level

Design Criteria:

Component	Value	Comments
H_{100}	2.60 m	Wave Height Exceeded at toe of structure 6 hours every 100 years
Tz	10.7 s	100 years ARI Tp = 15.0s
d	3.3 m	Depth = -1.0mAHD Bed level at toe of structure + 1.44m Storm Surge + 0.9m SLR
cotα	1.5	Batter Slope
P	0.1	Porosity of Structure
SD	2	Damage Factor
Wr	2600 kg/m3	Dry Rock Density
Storm Duration	6 hours	Typical Wave Event Duration

Rock Armour Specifications (Van Der Meer (1998)):

		Unfactored		Factored by 10%	
		Diameter (m)	Mass (kg)	Diameter (m)	Mass (kg)
Primary Armour	50% Passing	1.58	10,210	1.63	11,230
	15% Passing	1.41	7,310	1.46	8,040
	85% Passing	1.76	14,270	1.82	15,700
Secondary Armour	50% Passing	0.73	1,020	0.76	1,120
	15% Passing	0.65	730	0.68	800
	85% Passing	0.82	1,430	0.85	1,570
Core Details	50% Passing	0.27	51	0.28	56
	15% Passing	0.24	36	0.25	40
	85% Passing	0.30	71	0.31	78

Training Wall Armour Design Calculations

Rock Sizings to Resist Wave Action

Structure: Southern Training Wall

Section: Section F: +5.0mAHD Crest Level

Design Criteria:

Component	Value	Comments
H_{100}	2.60 m	Wave Height Exceeded at toe of structure 6 hours every 100 years
Tz	10.7 s	100 years ARI Tp = 15.0s
d	3.3 m	Depth = -1.0mAHD Bed level at toe of structure + 1.44m Storm Surge + 0.9m SLR
cotα	1.5	Batter Slope
P	0.2	Porosity of Structure
SD	2	Damage Factor
Wr	2600 kg/m3	Dry Rock Density
Storm Duration	6 hours	Typical Wave Event Duration

Rock Armour Specifications (Van Der Meer (1998)):

		Unfactored		Factored by 10%	
		Diameter (m)	Mass (kg)	Diameter (m)	Mass (kg)
Primary Armour	50% Passing	1.44	7,780	1.49	8,560
	15% Passing	1.29	5,570	1.33	6,130
	85% Passing	1.61	10,870	1.66	11,960
Secondary Armour	50% Passing	0.67	780	0.69	860
	15% Passing	0.60	560	0.62	620
	85% Passing	0.75	1,090	0.77	1,200
Core Details	50% Passing	0.25	39	0.25	43
	15% Passing	0.22	28	0.23	31
	85% Passing	0.28	55	0.28	60

Training Wall Armour Design Calculations

Rock Sizings to Resist Wave Action

Structure: South Entrance Beach - Long Groyne

Section: Section G: +2mAHD Crest Level

Design Criteria:

Component	Value	Comments
H_{100}	2.60 m	Wave Height Exceeded at toe of structure 6 hours every 100 years
Tz	10.7 s	100 years ARI Tp = 15.0s
d	3.0 m	Depth = -0.7mAHD Bed level at toe of structure + 1.44m Storm Surge + 0.9m SLR
$\cot\alpha$	2	Batter Slope
P	0.1	Porosity of Structure
SD	5	Damage Factor
Wr	2600 kg/m3	Dry Rock Density
Storm Duration	6 hours	Typical Wave Event Duration

Rock Armour Specifications (Van Der Meer (1998)):

		Unfactored		Factored by 10%	
		Diameter (m)	Mass (kg)	Diameter (m)	Mass (kg)
Primary Armour	50% Passing	1.17	4,190	1.21	4,610
	15% Passing	1.05	3,000	1.08	3,300
	85% Passing	1.31	5,860	1.35	6,450
Secondary Armour	50% Passing	0.54	420	0.56	460
	15% Passing	0.49	300	0.50	330
	85% Passing	0.61	590	0.63	650
Core Details	50% Passing	0.20	21	0.21	23
	15% Passing	0.18	15	0.19	17
	85% Passing	0.22	29	0.23	32

Training Wall Armour Design Calculations

Rock Sizings to Resist Wave Action

Structure: South Entrance Beach - Long Groyne

Section: Section G: +2mAHD Crest Level

Design Criteria:

Component	Value	Comments
H_{100}	2.60 m	Wave Height Exceeded at toe of structure 6 hours every 100 years
Tz	10.7 s	100 years ARI Tp = 15.0s
d	3.0 m	Depth = -1.0mAHD Bed level at toe of structure + 1.44m Storm Surge + 0.9m SLR
cotα	2	Batter Slope
P	0.2	Porosity of Structure
SD	5	Damage Factor
Wr	2600 kg/m3	Dry Rock Density
Storm Duration	6 hours	Typical Wave Event Duration

Rock Armour Specifications (Van Der Meer (1998)):

		Unfactored		Factored by 10%	
		Diameter (m)	Mass (kg)	Diameter (m)	Mass (kg)
Primary Armour	50% Passing	1.10	3,480	1.14	3,830
	15% Passing	0.99	2,490	1.02	2,740
	85% Passing	1.23	4,860	1.27	5,350
Secondary Armour	50% Passing	0.51	350	0.53	390
	15% Passing	0.46	250	0.48	280
	85% Passing	0.57	490	0.59	540
Core Details	50% Passing	0.19	18	0.19	19
	15% Passing	0.17	13	0.17	14
	85% Passing	0.21	24	0.22	27

Training Wall Armour Design Calculations

Rock Sizings to Resist Wave Action

Structure: South Entrance Beach - Long Groyne

Section: Section G: +2mAHD Crest Level

Design Criteria:

Component	Value	Comments
H_{100}	2.60 m	Wave Height Exceeded at toe of structure 6 hours every 100 years
Tz	10.7 s	100 years ARI Tp = 15.0s
d	3.0 m	Depth = -1.0mAHD Bed level at toe of structure + 1.44m Storm Surge + 0.9m SLR
cotα	2	Batter Slope
P	0.1	Porosity of Structure
SD	2	Damage Factor
Wr	2600 kg/m3	Dry Rock Density
Storm Duration	6 hours	Typical Wave Event Duration

Rock Armour Specifications (Van Der Meer (1998)):

		Unfactored		Factored by 10%	
		Diameter (m)	Mass (kg)	Diameter (m)	Mass (kg)
Primary Armour	50% Passing	1.41	7,260	1.45	7,990
	15% Passing	1.26	5,190	1.30	5,710
	85% Passing	1.57	10,150	1.63	11,170
Secondary Armour	50% Passing	0.65	730	0.68	800
	15% Passing	0.59	520	0.60	570
	85% Passing	0.73	1,020	0.76	1,120
Core Details	50% Passing	0.24	37	0.25	40
	15% Passing	0.22	26	0.22	29
	85% Passing	0.27	51	0.28	56

Training Wall Armour Design Calculations

Rock Sizings to Resist Wave Action

Structure: South Entrance Beach - Long Groyne

Section: Section G: +2mAHD Crest Level

Design Criteria:

Component	Value	Comments
H_{100}	2.60 m	Wave Height Exceeded at toe of structure 6 hours every 100 years
Tz	10.7 s	100 years ARI Tp = 15.0s
d	3.0 m	Depth = -1.0mAHD Bed level at toe of structure + 1.44m Storm Surge + 0.9m SLR
cotα	2	Batter Slope
P	0.2	Porosity of Structure
SD	2	Damage Factor
Wr	2600 kg/m3	Dry Rock Density
Storm Duration	6 hours	Typical Wave Event Duration

Rock Armour Specifications (Van Der Meer (1998)):

		Unfactored		Factored by 10%	
		Diameter (m)	Mass (kg)	Diameter (m)	Mass (kg)
Primary Armour	50% Passing	1.32	6,040	1.37	6,640
	15% Passing	1.18	4,320	1.22	4,750
	85% Passing	1.48	8,440	1.53	9,280
Secondary Armour	50% Passing	0.61	600	0.63	660
	15% Passing	0.55	430	0.57	470
	85% Passing	0.69	840	0.71	920
Core Details	50% Passing	0.23	30	0.23	33
	15% Passing	0.20	21	0.21	24
	85% Passing	0.25	42	0.26	46

Training Wall Armour Design Calculations

Rock Sizings to Resist Wave Action

Structure: South Entrance Beach - Long Groyne

Section: Section G: +2mAHD Crest Level

Design Criteria:

Component	Value	Comments
H_{100}	2.60 m	Wave Height Exceeded at toe of structure 6 hours every 100 years
Tz	10.7 s	100 years ARI Tp = 15.0s
d	3.0 m	Depth = -1.0mAHD Bed level at toe of structure + 1.44m Storm Surge + 0.9m SLR
cotα	1.5	Batter Slope
P	0.1	Porosity of Structure
SD	5	Damage Factor
Wr	2600 kg/m3	Dry Rock Density
Storm Duration	6 hours	Typical Wave Event Duration

Rock Armour Specifications (Van Der Meer (1998)):

		Unfactored		Factored by 10%	
		Diameter (m)	Mass (kg)	Diameter (m)	Mass (kg)
Primary Armour	50% Passing	1.32	5,920	1.36	6,510
	15% Passing	1.18	4,240	1.21	4,660
	85% Passing	1.47	8,270	1.52	9,100
Secondary Armour	50% Passing	0.61	590	0.63	650
	15% Passing	0.55	420	0.56	460
	85% Passing	0.68	820	0.70	900
Core Details	50% Passing	0.22	30	0.23	32
	15% Passing	0.20	21	0.21	23
	85% Passing	0.25	41	0.26	45

Training Wall Armour Design Calculations

Rock Sizings to Resist Wave Action

Structure: South Entrance Beach - Long Groyne

Section: Section G: +2mAHD Crest Level

Design Criteria:

Component	Value	Comments
H_{100}	2.60 m	Wave Height Exceeded at toe of structure 6 hours every 100 years
Tz	10.7 s	100 years ARI Tp = 15.0s
d	3.0 m	Depth = -1.0mAHD Bed level at toe of structure + 1.44m Storm Surge + 0.9m SLR
cotα	1.5	Batter Slope
P	0.2	Porosity of Structure
SD	5	Damage Factor
Wr	2600 kg/m3	Dry Rock Density
Storm Duration	6 hours	Typical Wave Event Duration

Rock Armour Specifications (Van Der Meer (1998)):

		Unfactored		Factored by 10%	
		Diameter (m)	Mass (kg)	Diameter (m)	Mass (kg)
Primary Armour	50% Passing	1.20	4,510	1.24	4,960
	15% Passing	1.07	3,230	1.11	3,550
	85% Passing	1.34	6,300	1.39	6,930
Secondary Armour	50% Passing	0.56	450	0.58	500
	15% Passing	0.50	320	0.51	350
	85% Passing	0.62	630	0.64	690
Core Details	50% Passing	0.21	23	0.21	25
	15% Passing	0.18	16	0.19	18
	85% Passing	0.23	31	0.24	35

Training Wall Armour Design Calculations

Rock Sizings to Resist Wave Action

Structure: South Entrance Beach - Long Groyne

Section: Section G: +2mAHD Crest Level

Design Criteria:

Component	Value	Comments
H_{100}	2.60 m	Wave Height Exceeded at toe of structure 6 hours every 100 years
Tz	10.7 s	100 years ARI Tp = 15.0s
d	3.0 m	Depth = -1.0mAHD Bed level at toe of structure + 1.44m Storm Surge + 0.9m SLR
$\cot\alpha$	1.5	Batter Slope
P	0.1	Porosity of Structure
SD	2	Damage Factor
Wr	2600 kg/m3	Dry Rock Density
Storm Duration	6 hours	Typical Wave Event Duration

Rock Armour Specifications (Van Der Meer (1998)):

		Unfactored		Factored by 10%	
		Diameter (m)	Mass (kg)	Diameter (m)	Mass (kg)
Primary Armour	50% Passing	1.58	10,260	1.63	11,290
	15% Passing	1.41	7,340	1.46	8,070
	85% Passing	1.77	14,340	1.82	15,770
Secondary Armour	50% Passing	0.73	1,030	0.76	1,130
	15% Passing	0.66	740	0.68	810
	85% Passing	0.82	1,440	0.85	1,580
Core Details	50% Passing	0.27	52	0.28	57
	15% Passing	0.24	37	0.25	41
	85% Passing	0.30	72	0.31	79

Training Wall Armour Design Calculations

Rock Sizings to Resist Wave Action

Structure: South Entrance Beach - Long Groyne

Section: Section G: +2mAHD Crest Level

Design Criteria:

Component	Value	Comments
H_{100}	2.60 m	Wave Height Exceeded at toe of structure 6 hours every 100 years
Tz	10.7 s	100 years ARI Tp = 15.0s
d	3.0 m	Depth = -1.0mAHD Bed level at toe of structure + 1.44m Storm Surge + 0.9m SLR
cotα	1.5	Batter Slope
P	0.2	Porosity of Structure
SD	2	Damage Factor
Wr	2600 kg/m3	Dry Rock Density
Storm Duration	6 hours	Typical Wave Event Duration

Rock Armour Specifications (Van Der Meer (1998)):

		Unfactored		Factored by 10%	
		Diameter (m)	Mass (kg)	Diameter (m)	Mass (kg)
Primary Armour	50% Passing	1.44	7,820	1.49	8,600
	15% Passing	1.29	5,600	1.33	6,160
	85% Passing	1.61	10,930	1.67	12,020
Secondary Armour	50% Passing	0.67	780	0.69	860
	15% Passing	0.60	560	0.62	620
	85% Passing	0.75	1,090	0.77	1,200
Core Details	50% Passing	0.25	39	0.25	43
	15% Passing	0.22	28	0.23	31
	85% Passing	0.28	55	0.28	60

Training Wall Armour Design Calculations

Rock Sizings to Resist Wave Action

Structure: South Entrance Beach - Short Groyne

Section: Section H: +2mAHD Crest Level

Design Criteria:

Component	Value	Comments
H_{100}	2.90 m	Wave Height Exceeded at toe of structure 6 hours every 100 years
Tz	10.7 s	100 years ARI Tp = 15.0s
d	3.3 m	Depth = -1.0mAHD Bed level at toe of structure + 1.44m Storm Surge + 0.9m SLR
cotα	2	Batter Slope
P	0.1	Porosity of Structure
SD	5	Damage Factor
Wr	2600 kg/m3	Dry Rock Density
Storm Duration	6 hours	Typical Wave Event Duration

Rock Armour Specifications (Van Der Meer (1998)):

		Unfactored		Factored by 10%	
		Diameter (m)	Mass (kg)	Diameter (m)	Mass (kg)
Primary Armour	50% Passing	1.31	5,880	1.36	6,470
	15% Passing	1.17	4,210	1.21	4,630
	85% Passing	1.47	8,220	1.51	9,040
Secondary Armour	50% Passing	0.61	590	0.63	650
	15% Passing	0.55	420	0.56	460
	85% Passing	0.68	820	0.70	900
Core Details	50% Passing	0.22	30	0.23	32
	15% Passing	0.20	21	0.21	23
	85% Passing	0.25	41	0.26	45

Training Wall Armour Design Calculations

Rock Sizings to Resist Wave Action

Structure: South Entrance Beach - Short Groyne

Section: Section H: +2mAHD Crest Level

Design Criteria:

Component	Value	Comments
H_{100}	2.90 m	Wave Height Exceeded at toe of structure 6 hours every 100 years
Tz	10.7 s	100 years ARI Tp = 15.0s
d	3.3 m	Depth = -1.0mAHD Bed level at toe of structure + 1.44m Storm Surge + 0.9m SLR
$\cot\alpha$	2	Batter Slope
P	0.2	Porosity of Structure
SD	5	Damage Factor
Wr	2600 kg/m3	Dry Rock Density
Storm Duration	6 hours	Typical Wave Event Duration

Rock Armour Specifications (Van Der Meer (1998)):

		Unfactored		Factored by 10%	
		Diameter (m)	Mass (kg)	Diameter (m)	Mass (kg)
Primary Armour	50% Passing	1.24	4,960	1.28	5,460
	15% Passing	1.11	3,550	1.15	3,910
	85% Passing	1.39	6,930	1.43	7,620
Secondary Armour	50% Passing	0.58	500	0.60	550
	15% Passing	0.52	360	0.54	400
	85% Passing	0.65	700	0.67	770
Core Details	50% Passing	0.21	25	0.22	28
	15% Passing	0.19	18	0.20	20
	85% Passing	0.24	35	0.25	38

Training Wall Armour Design Calculations

Rock Sizings to Resist Wave Action

Structure: South Entrance Beach - Short Groyne

Section: Section H: +2mAHD Crest Level

Design Criteria:

Component	Value	Comments
H_{100}	2.90 m	Wave Height Exceeded at toe of structure 6 hours every 100 years
Tz	10.7 s	100 years ARI Tp = 15.0s
d	3.3 m	Depth = -1.0mAHD Bed level at toe of structure + 1.44m Storm Surge + 0.9m SLR
cotα	2	Batter Slope
P	0.1	Porosity of Structure
SD	2	Damage Factor
Wr	2600 kg/m3	Dry Rock Density
Storm Duration	6 hours	Typical Wave Event Duration

Rock Armour Specifications (Van Der Meer (1998)):

		Unfactored		Factored by 10%	
		Diameter (m)	Mass (kg)	Diameter (m)	Mass (kg)
Primary Armour	50% Passing	1.58	10,190	1.63	11,210
	15% Passing	1.41	7,290	1.46	8,020
	85% Passing	1.76	14,240	1.82	15,660
Secondary Armour	50% Passing	0.73	1,020	0.76	1,120
	15% Passing	0.65	730	0.68	800
	85% Passing	0.82	1,430	0.85	1,570
Core Details	50% Passing	0.27	51	0.28	56
	15% Passing	0.24	36	0.25	40
	85% Passing	0.30	71	0.31	78

Training Wall Armour Design Calculations

Rock Sizings to Resist Wave Action

Structure: South Entrance Beach - Short Groyne

Section: Section H: +2mAHD Crest Level

Design Criteria:

Component	Value	Comments
H_{100}	2.90 m	Wave Height Exceeded at toe of structure 6 hours every 100 years
Tz	10.7 s	100 years ARI Tp = 15.0s
d	3.3 m	Depth = -1.0mAHD Bed level at toe of structure + 1.44m Storm Surge + 0.9m SLR
cotα	2	Batter Slope
P	0.2	Porosity of Structure
SD	2	Damage Factor
Wr	2600 kg/m3	Dry Rock Density
Storm Duration	6 hours	Typical Wave Event Duration

Rock Armour Specifications (Van Der Meer (1998)):

		Unfactored		Factored by 10%	
		Diameter (m)	Mass (kg)	Diameter (m)	Mass (kg)
Primary Armour	50% Passing	1.49	8,600	1.54	9,460
	15% Passing	1.33	6,150	1.38	6,770
	85% Passing	1.67	12,020	1.72	13,220
Secondary Armour	50% Passing	0.69	860	0.71	950
	15% Passing	0.62	620	0.64	680
	85% Passing	0.77	1,200	0.80	1,320
Core Details	50% Passing	0.25	43	0.26	47
	15% Passing	0.23	31	0.24	34
	85% Passing	0.28	60	0.29	66

Training Wall Armour Design Calculations

Rock Sizings to Resist Wave Action

Structure: South Entrance Beach - Short Groyne

Section: Section H: +2mAHD Crest Level

Design Criteria:

Component	Value	Comments
H_{100}	2.90 m	Wave Height Exceeded at toe of structure 6 hours every 100 years
Tz	10.7 s	100 years ARI Tp = 15.0s
d	3.3 m	Depth = -1.0mAHD Bed level at toe of structure + 1.44m Storm Surge + 0.9m SLR
$\cot\alpha$	1.5	Batter Slope
P	0.1	Porosity of Structure
SD	5	Damage Factor
Wr	2600 kg/m3	Dry Rock Density
Storm Duration	6 hours	Typical Wave Event Duration

Rock Armour Specifications (Van Der Meer (1998)):

		Unfactored		Factored by 10%	
		Diameter (m)	Mass (kg)	Diameter (m)	Mass (kg)
Primary Armour	50% Passing	1.47	8,300	1.52	9,130
	15% Passing	1.32	5,940	1.36	6,530
	85% Passing	1.65	11,600	1.70	12,760
Secondary Armour	50% Passing	0.68	830	0.70	910
	15% Passing	0.61	590	0.63	650
	85% Passing	0.76	1,160	0.79	1,280
Core Details	50% Passing	0.25	42	0.26	46
	15% Passing	0.23	30	0.23	33
	85% Passing	0.28	58	0.29	64

Training Wall Armour Design Calculations

Rock Sizings to Resist Wave Action

Structure: South Entrance Beach - Short Groyne

Section: Section H: +2mAHD Crest Level

Design Criteria:

Component	Value	Comments
H_{100}	2.90 m	Wave Height Exceeded at toe of structure 6 hours every 100 years
Tz	10.7 s	100 years ARI Tp = 15.0s
d	3.3 m	Depth = -1.0mAHD Bed level at toe of structure + 1.44m Storm Surge + 0.9m SLR
$\cot\alpha$	1.5	Batter Slope
P	0.2	Porosity of Structure
SD	5	Damage Factor
Wr	2600 kg/m3	Dry Rock Density
Storm Duration	6 hours	Typical Wave Event Duration

Rock Armour Specifications (Van Der Meer (1998)):

		Unfactored		Factored by 10%	
		Diameter (m)	Mass (kg)	Diameter (m)	Mass (kg)
Primary Armour	50% Passing	1.35	6,430	1.40	7,070
	15% Passing	1.21	4,600	1.25	5,060
	85% Passing	1.51	8,990	1.56	9,890
Secondary Armour	50% Passing	0.63	640	0.65	700
	15% Passing	0.56	460	0.58	510
	85% Passing	0.70	890	0.72	980
Core Details	50% Passing	0.23	32	0.24	35
	15% Passing	0.21	23	0.21	25
	85% Passing	0.26	45	0.27	49

Training Wall Armour Design Calculations

Rock Sizings to Resist Wave Action

Structure: South Entrance Beach - Short Groyne

Section: Section H: +2mAHD Crest Level

Design Criteria:

Component	Value	Comments
H_{100}	2.90 m	Wave Height Exceeded at toe of structure 6 hours every 100 years
Tz	10.7 s	100 years ARI Tp = 15.0s
d	3.3 m	Depth = -1.0mAHD Bed level at toe of structure + 1.44m Storm Surge + 0.9m SLR
cotα	1.5	Batter Slope
P	0.1	Porosity of Structure
SD	2	Damage Factor
Wr	2600 kg/m3	Dry Rock Density
Storm Duration	6 hours	Typical Wave Event Duration

Rock Armour Specifications (Van Der Meer (1998)):

		Unfactored		Factored by 10%	
		Diameter (m)	Mass (kg)	Diameter (m)	Mass (kg)
Primary Armour	50% Passing	1.77	14,390	1.83	15,830
	15% Passing	1.58	10,300	1.63	11,330
	85% Passing	1.98	20,110	2.04	22,120
Secondary Armour	50% Passing	0.82	1,440	0.85	1,580
	15% Passing	0.73	1,030	0.76	1,130
	85% Passing	0.92	2,010	0.95	2,210
Core Details	50% Passing	0.30	72	0.31	79
	15% Passing	0.27	52	0.28	57
	85% Passing	0.34	101	0.35	111

Training Wall Armour Design Calculations

Rock Sizings to Resist Wave Action

Structure: South Entrance Beach - Short Groyne

Section: Section H: +2mAHD Crest Level

Design Criteria:

Component	Value	Comments
H_{100}	2.90 m	Wave Height Exceeded at toe of structure 6 hours every 100 years
Tz	10.7 s	100 years ARI Tp = 15.0s
d	3.3 m	Depth = -1.0mAHD Bed level at toe of structure + 1.44m Storm Surge + 0.9m SLR
cotα	1.5	Batter Slope
P	0.2	Porosity of Structure
SD	2	Damage Factor
Wr	2600 kg/m3	Dry Rock Density
Storm Duration	6 hours	Typical Wave Event Duration

Rock Armour Specifications (Van Der Meer (1998)):

		Unfactored		Factored by 10%	
		Diameter (m)	Mass (kg)	Diameter (m)	Mass (kg)
Primary Armour	50% Passing	1.62	11,140	1.68	12,250
	15% Passing	1.45	7,970	1.50	8,770
	85% Passing	1.82	15,570	1.87	17,130
Secondary Armour	50% Passing	0.75	1,110	0.78	1,220
	15% Passing	0.67	790	0.69	870
	85% Passing	0.84	1,550	0.87	1,710
Core Details	50% Passing	0.28	56	0.29	61
	15% Passing	0.25	40	0.26	44
	85% Passing	0.31	78	0.32	85

Training Wall Armour Design Calculations

Rock Sizings to Resist Flood Currents

As per SPM (1984) - Equation 7-142

Northern Training Wall		Unfactored		Factored by 10%	
Section A: +5.0mAHD Height		Diameter (m)	Mass (kg)	Diameter (m)	Mass (kg)
Primary Armour	50% Passing	0.88	1,830	0.92	2,010
	15% Passing	0.79	1,280	0.82	1,410
	85% Passing	0.99	2,510	1.02	2,760
Secondary Armour	50% Passing	0.41	180	0.43	200
	15% Passing	0.37	130	0.38	140
	85% Passing	0.46	250	0.48	280
Core Details	50% Passing	0.15	9	0.16	10
	15% Passing	0.14	6	0.14	7
	85% Passing	0.17	13	0.17	14

Training Wall Armour Design Calculations

Rock Sizings to Resist Flood Currents

As per SPM (1984) - Equation 7-142

Northern Training Wall		Unfactored		Factored by 10%	
Section B: +5.0mAHD Height		Diameter (m)	Mass (kg)	Diameter (m)	Mass (kg)
Primary Armour	50% Passing	0.78	1,260	0.81	1,390
	15% Passing	0.70	880	0.72	970
	85% Passing	0.87	1,730	0.90	1,900
Secondary Armour	50% Passing	0.37	130	0.38	140
	15% Passing	0.33	90	0.34	100
	85% Passing	0.41	180	0.43	200
Core Details	50% Passing	0.14	7	0.14	7
	15% Passing	0.12	5	0.13	5
	85% Passing	0.15	9	0.16	10

Training Wall Armour Design Calculations

Rock Sizings to Resist Flood Currents

As per SPM (1984) - Equation 7-142

Northern Training Wall		Unfactored		Factored by 10%	
Section C: +4.5mAHD Height		Diameter (m)	Mass (kg)	Diameter (m)	Mass (kg)
Primary Armour	50% Passing	0.47	270	0.49	300
	15% Passing	0.42	190	0.43	210
	85% Passing	0.52	370	0.54	410
Secondary Armour	50% Passing	0.23	30	0.23	30
	15% Passing	0.20	20	0.20	20
	85% Passing	0.25	40	0.25	40
Core Details	50% Passing	0.08	2	0.09	2
	15% Passing	0.07	1	0.08	1
	85% Passing	0.09	2	0.10	2

Training Wall Armour Design Calculations

Rock Sizings to Resist Flood Currents

As per SPM (1984) - Equation 7-142

Northern Training Wall		Unfactored		Factored by 10%	
Section D: +3.9mAHD Height		Diameter (m)	Mass (kg)	Diameter (m)	Mass (kg)
Primary Armour	50% Passing	0.30	70	0.31	80
	15% Passing	0.27	50	0.28	60
	85% Passing	0.33	100	0.35	110
Secondary Armour	50% Passing	0.16	10	0.16	10
	15% Passing	0.14	10	0.16	10
	85% Passing	0.18	10	0.16	10
Core Details	50% Passing	0.06	1	0.06	1
	15% Passing	0.05	0	0.05	0
	85% Passing	0.06	1	0.07	1

Training Wall Armour Design Calculations

Rock Sizings to Resist Flood Currents

As per SPM (1984) - Equation 7-142

Northern Training Wall		Unfactored		Factored by 10%	
Section E: +3.9mAHD Height		Diameter (m)	Mass (kg)	Diameter (m)	Mass (kg)
Primary Armour	50% Passing	0.30	70	0.31	80
	15% Passing	0.27	50	0.28	60
	85% Passing	0.33	100	0.35	110
Secondary Armour	50% Passing	0.16	10	0.16	10
	15% Passing	0.14	10	0.16	10
	85% Passing	0.18	10	0.16	10
Core Details	50% Passing	0.06	1	0.06	1
	15% Passing	0.05	0	0.05	0
	85% Passing	0.06	1	0.07	1

Training Wall Armour Design Calculations

Rock Sizings to Resist Flood Currents

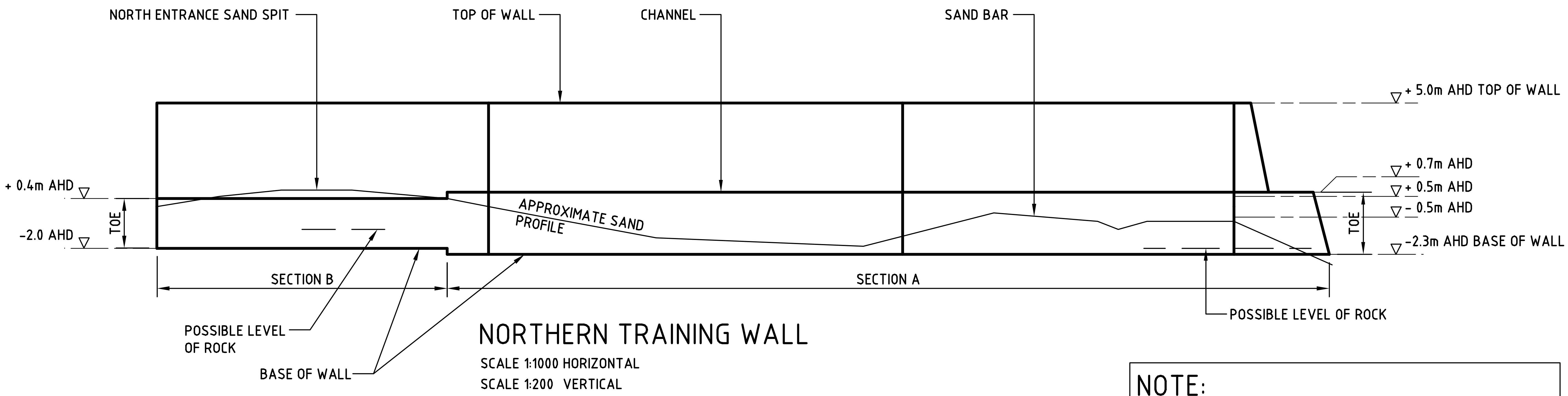
As per SPM (1984) - Equation 7-142

Northern Training Wall		Unfactored		Factored by 10%	
Section F: +4.5mAHD Height		Diameter (m)	Mass (kg)	Diameter (m)	Mass (kg)
Primary Armour	50% Passing	0.72	970	0.74	1,070
	15% Passing	0.64	680	0.66	750
	85% Passing	0.80	1,330	0.83	1,460
Secondary Armour	50% Passing	0.34	100	0.35	110
	15% Passing	0.30	70	0.31	80
	85% Passing	0.38	140	0.39	150
Core Details	50% Passing	0.12	5	0.13	6
	15% Passing	0.11	4	0.11	4
	85% Passing	0.14	7	0.14	8

Tuggerah Lakes The Entrance Morphodynamic Modelling Entrance Beach Management Investigations

APPENDIX F
PROPOSED STRUCTURES – DESIGN SECTIONS

Cardno
Shaping the Future

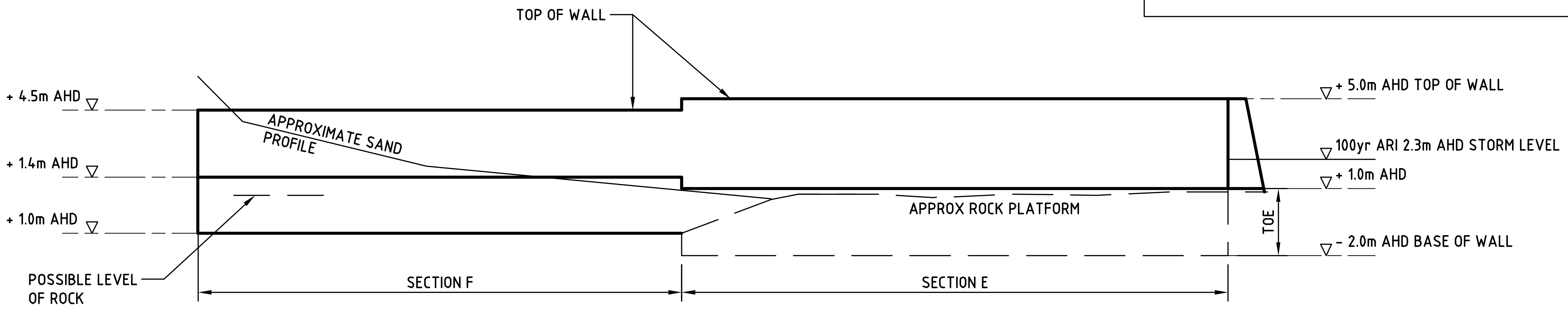


NORTHERN TRAINING WALL

SCALE 1:1000 HORIZONTAL
SCALE 1:200 VERTICAL

NOTE:

ALL WALLS AND REVETMENTS DESIGNED TO BE FOUNDED ON SAND.



SOUTHERN TRAINING WALL

SCALE 1:1000 HORIZONTAL
SCALE 1:200 VERTICAL

SCALE 1:200 (4 3 2 1 0 4 8 12 16 20m)

SCALE 1:1000 (20 15 10 5 0 20 40 60 80 100m)

NOTE:

FOR DETAILS OF WALL REFER SK03 AND SK04.

NOT CONSTRUCTION ISSUE

REV	DATE	REVISIONS	DRN	CHK
1	08/08/13	PRELIMINARY	MK	RW

DESIGNED: R.W.
DRAWN: M.K.
CHECKED:
RECOMMENDED:
APPROVED:
DATUM: AHD
COORD SYSTEM:

Cardno (NSW/ACT) Pty Ltd
ABN: 95 001 145 035
Suite 34, 207 Albany Street North, Gosford, NSW Australia 2250
Phone (+61 2) 4323 2558 Fax (+61 2) 4324 3251
Email: gosford@cardno.com.au Web: www.cardno.com.au

NSW OFFICE OF ENVIRONMENT AND HERITAGE
THE ENTRANCE MODELLING
POSSIBLE TRAINING WALLS

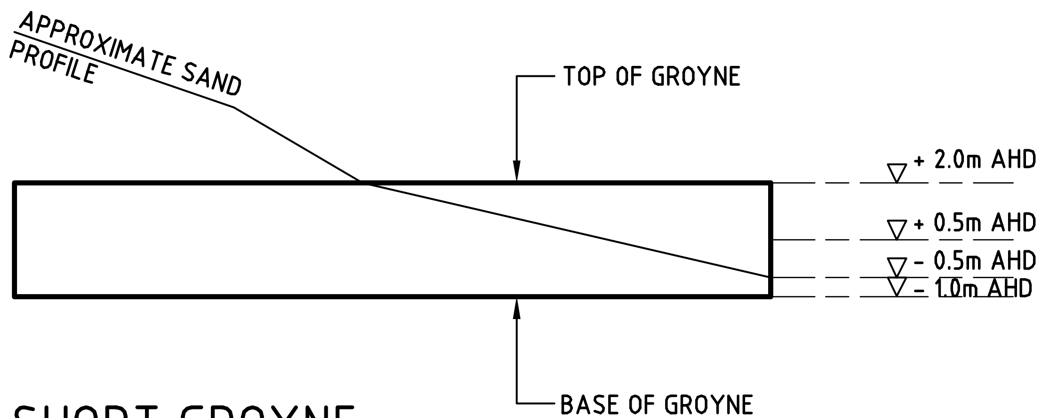
TRAINING WALL LONG SECTIONS

DRAWING STATUS: PRELIMINARY
DRAWING No: LJ2985-SK01
REV 1

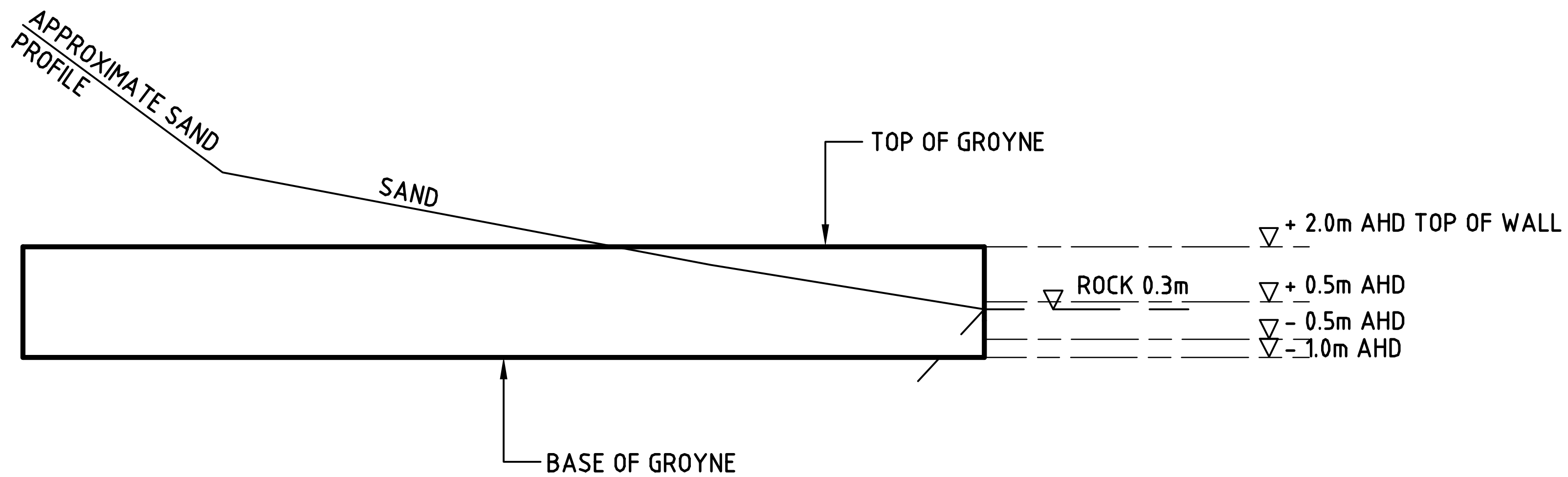
0 10 20 30 40 50 60 70 80 90 100mm A3 ON ORIGINAL

XREF's:
CAD FILE: \\AUGOSCFS01\General\~Richard\LJ2985-003-CentralCoast--TheEntrance-Breakwater\Drawings\Build\Structural\LJ2985-SK01(1).dwg
DATE PLOTTED: 21 August, 2013 - 4:18pm

NOTE:
ALL WALLS AND REVETMENTS DESIGNED TO BE FOUNDED ON SAND.



SHORT GROYNE
SCALE 1:1000 HORIZONTAL
SCALE 1:200 VERTICAL



LONG GROYNE
SCALE 1:1000 HORIZONTAL
SCALE 1:200 VERTICAL

NOTE:
SAND LEVELS WILL DROP DURING STORM EVENTS EXPOSING BURRIED PORTIONS OF THE GROYNES. THE SAND WOULD BUILD UP SLOWLY AFTERWARDS POTENTIALLY COVERING PART OF THE GROYNE.

4 3 2 1 0 4 8 12 16 20m
SCALE 1:200

20 15 10 5 0 20 40 60 80 100m
SCALE 1:1000

NOTE:
FOR DETAILS OF WALL REFER SK05.

NOT CONSTRUCTION ISSUE

REV	DATE	REVISIONS	DRN	CHK
1	08/08/13	PRELIMINARY	MK	RW

DESIGNED: R.W.
DRAWN: M.K.
CHECKED:
RECOMMENDED:
APPROVED:
DATUM: AHD
COORD SYSTEM:

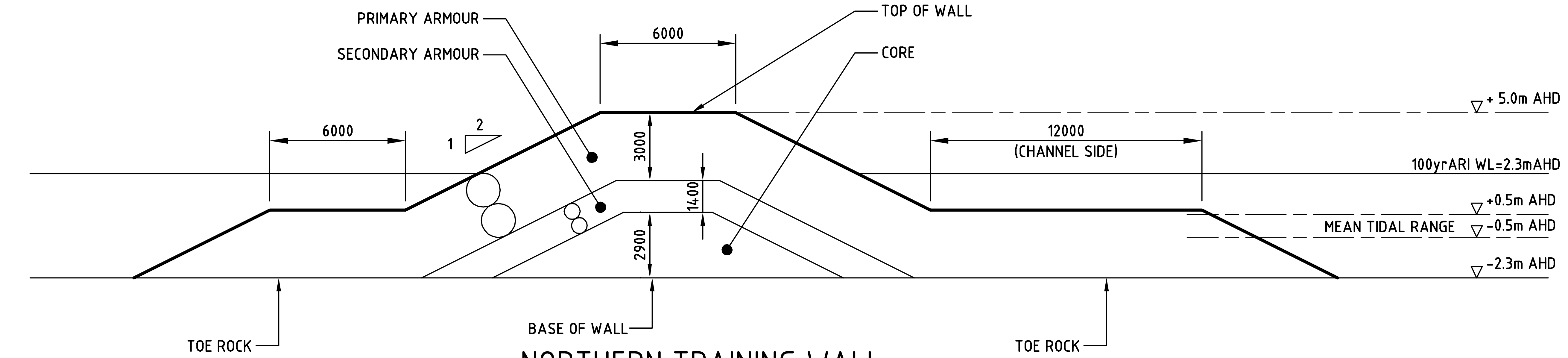
Cardno (NSW/ACT) Pty Ltd
ABN: 95 001 145 035
Suite 34, 207 Albany Street North, Gosford, NSW Australia 2250
Phone (+61 2) 4323 2558 Fax (+61 2) 4324 3251
Email: gosford@cardno.com.au Web: www.cardno.com.au

NSW OFFICE OF ENVIRONMENT AND HERITAGE
THE ENTRANCE MODELLING
POSSIBLE TRAINING WALLS
SOUTHERN ENTRANCE BEACH - GROYNE LONG SECTIONS

DRAWING STATUS: PRELIMINARY
DRAWING No: LJ2985-SK02
REV: 1

0 10 20 30 40 50 60 70 80 90 100mm A3 ON ORIGINAL

XREF's:
CAD FILE: \\AUGOSCFS01\General\~Richard\LJ2985-003-CentralCoast--TheEntrance-Breakwater\Drawings\Build\Structural\LJ2985-SK02(1).dwg
DATE PLOTTED: 21 August, 2013 - 4:19pm

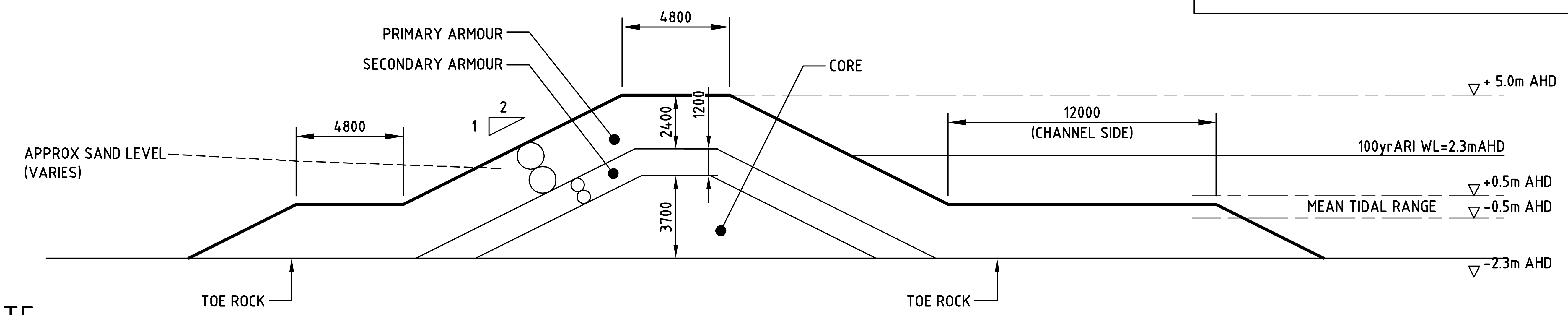


NORTHERN TRAINING WALL

SECTION A -

SCALE 1:200

NOTE:
ALL WALLS AND REVETMENTS DESIGNED TO BE FOUNDED ON SAND.



NORTHERN TRAINING WALL

SECTION B -

SCALE 1:200

NOTE:

- THE TOE ROCK IS INTENDED TO PROTECT THE TRAINING WALL FROM SCOUR DURING EXTREME EVENTS.
- THE TOE ROCK IS EXPECTED TO SETTLE INTO THE SEA BED DURING SCOUR EVENTS AND BE GRADUALLY COVERED WITH SAND AFTERWARDS.
- WHERE THE WALL CAN BE FOUNDED ON BEDROCK AND THEREFORE WILL NOT BE UNDERMINED DURING FLOOD EVENTS, THE TOE MAY BE OMITTED.
- INVESTIGATION TO DETERMINE THE LEVEL OF THE BEDROCK WOULD INDICATE WHERE THE TOE ROCK MAY BE OMITTED.

4 3 2 1 0 4 8 12 16 20m

SCALE 1:200

NORTHERN TRAINING WALL			
	PRIMARY ARMOUR	SECONDARY ARMOUR	CORE
	D50	D50	D50
SECTION A	1.5m	0.7m	0.25m
SECTION B	1.2m	0.6m	0.21m

NOT CONSTRUCTION ISSUE

REV	DATE	REVISIONS	DRN	CHK
1	08/08/13	PRELIMINARY	MK	RW

DESIGNED: R.W.
DRAWN: M.K.
CHECKED:
RECOMMENDED:
APPROVED:
DATUM: AHD
COORD SYSTEM:

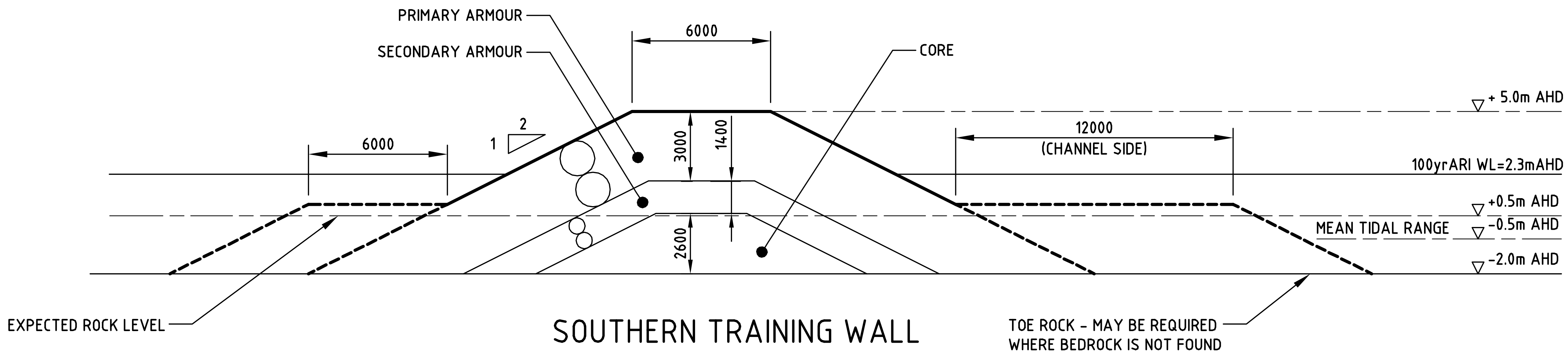
Cardno
Cardno (NSW/ACT) Pty Ltd
ABN: 95 001 145 035
Suite 34, 207 Albany Street North, Gosford, NSW Australia 2250
Phone (+61 2) 4323 2558 Fax (+61 2) 4324 3251
Email: gosford@cardno.com.au Web: www.cardno.com.au

NSW OFFICE OF ENVIRONMENT AND HERITAGE
THE ENTRANCE MODELLING
POSSIBLE TRAINING WALLS
NORTHERN TRAINING WALL CROSS SECTIONS

DRAWING STATUS: PRELIMINARY
DRAWING No: LJ2985-SK03
REV 1

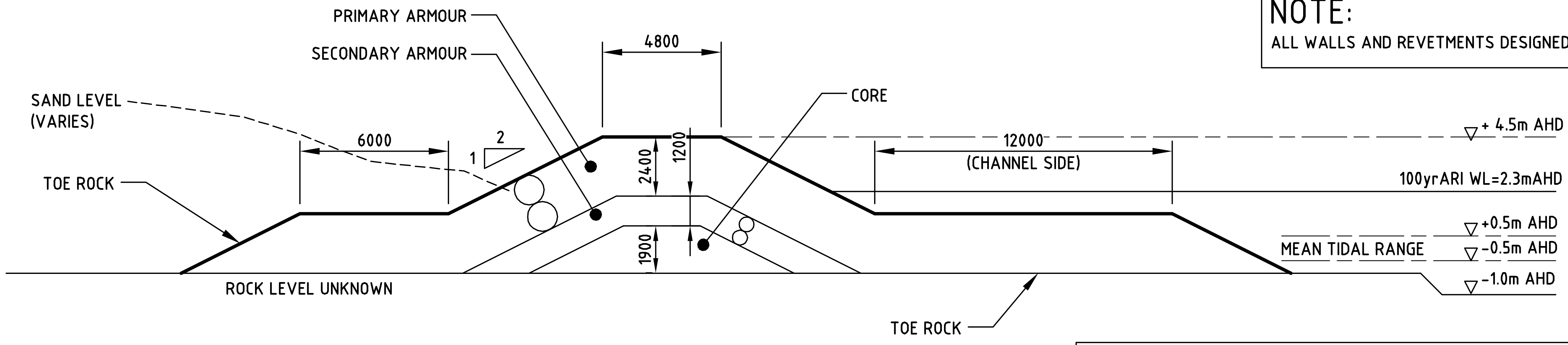
0 10 20 30 40 50 60 70 80 90 100mm A3 ON ORIGINAL

XREF's:
CAD FILE: \\AUGOSCFS01\General\~Richard\LJ2985-003-CentralCoast--TheEntrance-Breakwater\Drawings\Build\Structural\LJ2985-SK03(1).dwg DATE PLOTTED: 21 August, 2013 - 4:20pm



SOUTHERN TRAINING WALL

SECTION E –
SCALE 1:200



SOUTHERN TRAINING WALL

SECTION F –
SCALE 1:200

NOTE:
ALL WALLS AND REVETMENTS DESIGNED TO BE FOUNDED ON SAND.

SOUTHERN TRAINING WALL			
	PRIMARY ARMOUR	SECONDARY ARMOUR	CORE
	D50	D50	D50
SECTION E	1.5m	0.7m	0.25m
SECTION F	1.2m	0.6m	0.21m

0 4 8 12 16 20m
4 3 2 1
SCALE 1:200

NOT CONSTRUCTION ISSUE

REV	DATE	REVISIONS	DRN	CHK
1	08/08/13	PRELIMINARY	MK	RW

DESIGNED: R.W.
DRAWN: M.K.
CHECKED:
RECOMMENDED:
APPROVED:
DATUM: AHD
COORD SYSTEM:

Cardno
Cardno (NSW/ACT) Pty Ltd
ABN: 95 001 145 035
Suite 34, 207 Albany Street North, Gosford, NSW Australia 2250
Phone (+61 2) 4323 2558 Fax (+61 2) 4324 3251
Email: gosford@cardno.com.au Web: www.cardno.com.au

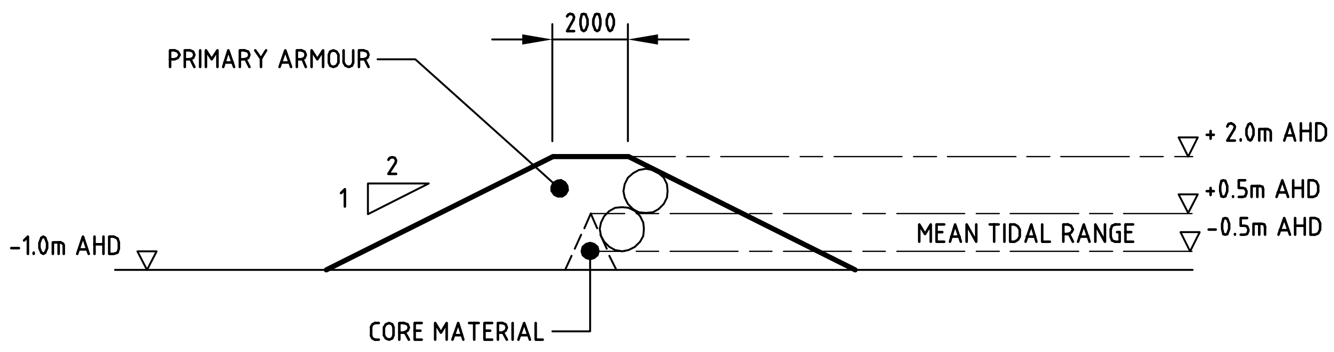
NSW OFFICE OF ENVIRONMENT AND HERITAGE
THE ENTRANCE MODELLING
POSSIBLE TRAINING WALLS

SOUTHERN TRAINING WALL CROSS SECTIONS

DRAWING STATUS: PRELIMINARY
DRAWING No: LJ2985-SK04
REV 1

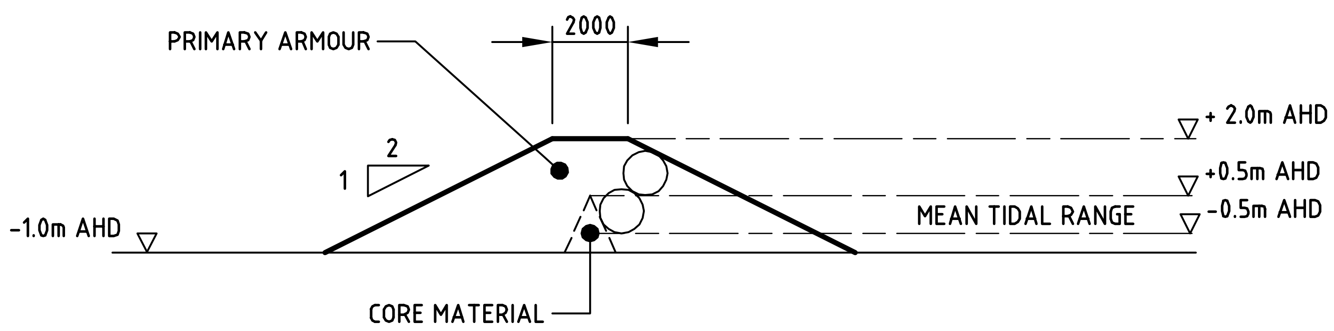
0 10 20 30 40 50 60 70 80 90 100mm A3 ON ORIGINAL

XREF's:
CAD FILE: \\AUGOSCFS01\General\~Richard\LJ2985-003-CentralCoast--TheEntrance-Breakwater\Drawings\Build\Structural\LJ2985-SK04(1).dwg
DATE PLOTTED: 21 August, 2013 - 4:20pm



SOUTH ENTRANCE BEACH – LONG GROYNE

SECTION G –
SCALE 1:200



SOUTH ENTRANCE BEACH – SHORT GROYNE

SECTION H –
SCALE 1:200

NOTE:
ALL WALLS AND REVETMENTS DESIGNED TO BE FOUNDED ON SAND.

SOUTH ENTRANCE BEACH GROYNES

	PRIMARY ARMOUR	SECONDARY ARMOUR	CORE
	D50	D50	D50
SECTION G	1.2m	0.6m	0.21m
SECTION H	1.4m	0.6m	0.23m

4 3 2 1 0 4 8 12 16 20m
SCALE 1:200

NOT CONSTRUCTION ISSUE

REV	DATE	REVISIONS	DRN	CHK
1	08/08/13	PRELIMINARY	MK	RW

DESIGNED: R.W.
DRAWN: M.K.
CHECKED:
RECOMMENDED:
APPROVED:
DATUM: AHD
COORD SYSTEM:

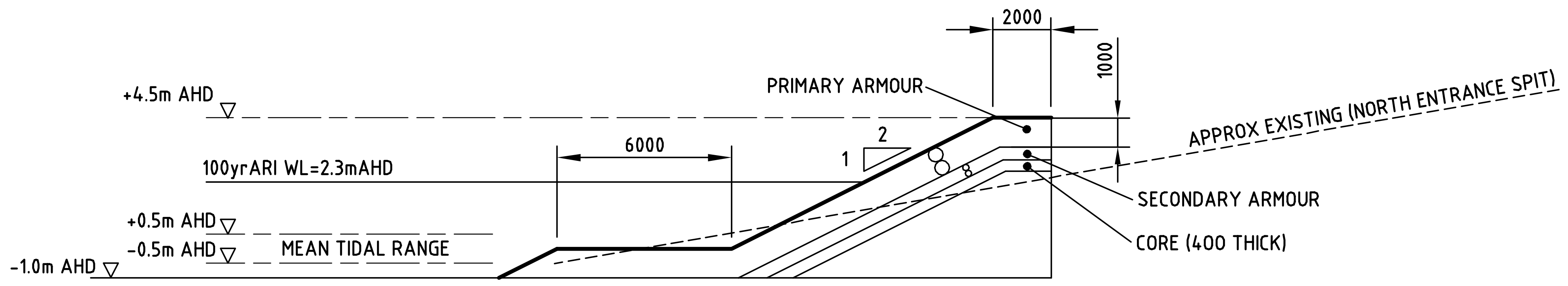
Cardno
Cardno (NSW/ACT) Pty Ltd
ABN: 95 001 145 035
Suite 34, 207 Albany Street North, Gosford, NSW Australia 2250
Phone (+61 2) 4323 2558 Fax (+61 2) 4324 3251
Email: gosford@cardno.com.au Web: www.cardno.com.au

NSW OFFICE OF ENVIRONMENT AND HERTAGE
THE ENTRANCE MODELLING
POSSIBLE TRAINING WALLS
SOUTH ENTRANCE BEACH GROYNE CROSS SECTIONS

DRAWING STATUS: PRELIMINARY
DRAWING No: LJ2985-SK05
REV 1

0 10 20 30 40 50 60 70 80 90 100mm A3 ON ORIGINAL

XREF's:
CAD FILE: \\AUGOSCFS01\General\-Richard\LJ2985-003-CentralCoast--TheEntrance-Breakwater\Drawings\Build\Structural\LJ2985-SK05(1).dwg DATE PLOTTED: 21 August, 2013 - 4:21pm

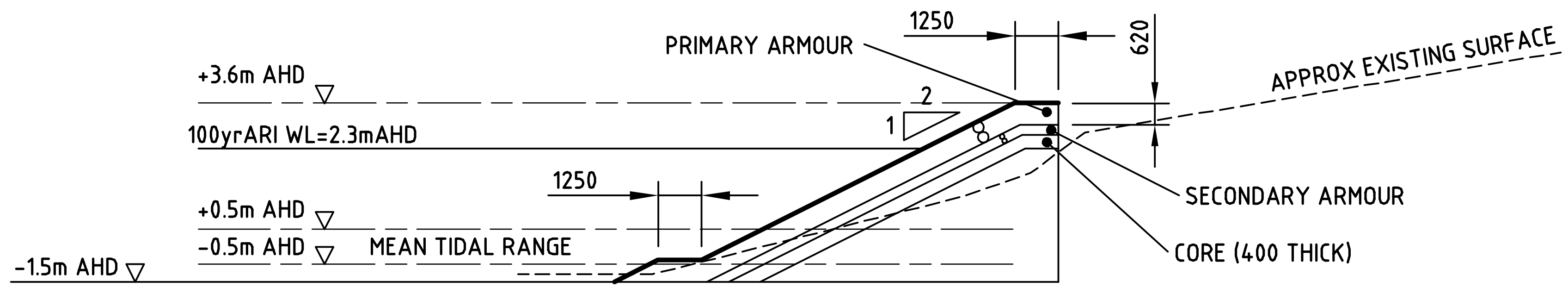


REVETMENT WALL
(NORTHERN TRAINING WALL)

SECTION C
SCALE 1:200

NOTE:

- ALL WALLS AND REVETMENTS DESIGNED TO BE FOUNDED ON SAND.
- SECTION C: PROTECTION FOR NORTH ENTRANCE SAND SPIT.
- SECTION D: PROTECTION FROM SAND SPIT UP TO BRIDGE.



REVETMENT WALL

REVETMENT WALL
(NORTHERN TRAINING WALL)

SECTION D
SCALE 1:200

SOUTHERN TRAINING WALL			
	PRIMARY ARMOUR	SECONDARY ARMOUR	CORE
	D50	D50	D50
SECTION C	0.48m	0.22m	0.08m
SECTION D	0.31m	0.14m	0.05m

NOT CONSTRUCTION ISSUE

REV	DATE	REVISIONS	DRN	CHK
1	08/08/13	PRELIMINARY	MK	RW

DESIGNED: R.W.
DRAWN: M.K.
CHECKED:
RECOMMENDED:
APPROVED:
DATUM: AHD
COORD SYSTEM:

Cardno
Cardno (NSW/ACT) Pty Ltd
ABN: 95 001 145 035
Suite 34, 207 Albany Street North, Gosford, NSW Australia 2250
Phone (+61 2) 4323 2558 Fax (+61 2) 4324 3251
Email: gosford@cardno.com.au Web: www.cardno.com.au

NSW OFFICE OF ENVIRONMENT AND HERITAGE
THE ENTRANCE MODELLING
POSSIBLE TRAINING WALLS

REVETMENT WALL CROSS SECTIONS

DRAWING STATUS: PRELIMINARY
DRAWING No: LJ2985-SK06
REV 1

0 10 20 30 40 50 60 70 80 90 100mm A3 ON ORIGINAL

XREF's:
CAD FILE: \\AUGOSCFS01\General\~Richard\LJ2985-003-CentralCoast--TheEntrance-Breakwater\Drawings\Build\Structural\LJ2985-SK06(1).dwg DATE PLOTTED: 21 August, 2013 - 4:21pm

APPENDIX G
COST DETAILS

A summary of the cost breakdown is given in the Tables below. An explanation of each item is given in the notes below the tables.

Table G-1 Cost Estimate Breakdown – Sand Nourishment

Option:		P50 Est	P90 Est
Sand Nourishment			
Direct costs (preliminary estimate)	$130,000		
Site establishment, indirect costs	$10,000		
Builder's Risk Premium (ocean exposure)	$0		
Builder's Margin (10%)	$14,000		
Project management, internal costs	$25,000		
Design	$10,000		
Base Cost (Total of above)	$189,000		
Inherent Risk		$18,900	$56,700
Contingent Risk		$11,340	$37,800
Escalation		$76,734	$99,225
TOTAL (base + risks + escalation)		**$295,974**	**$382,725**
Permanent pipe installation (not including dredging costs)			
Direct costs (preliminary estimate)	$420,000		
Site establishment, indirect costs	$25,000		
Builder's Risk Premium (ocean exposure)	$0		
Builder's Margin (10%)	$44,500		
Project management, internal costs	$50,000		
Design	$20,000		
Base Cost (Total of above)	$559,500		
Inherent Risk		$55,950	$167,850
Contingent Risk		$33,570	$111,900
Escalation		$227,157	$293,738
TOTAL (base + risks + escalation)		**$876,177**	**$1,132,988**

Table G-2 Cost Estimate Breakdown – Groynes and South Wall

Option:		P50 Est	P90 Est
Short Groyne			
Direct costs (preliminary estimate)	$655,000		
Site establishment, indirect costs	$40,000		
Builder's Risk Premium (ocean exposure)	$100,000		
Builder's Margin (10%)	$79,500		
Project management, internal costs	$50,000		
Design	$60,000		
Base Cost (Total of above)	**$984,500**		
Inherent Risk		$98,450	$295,350
Contingent Risk		$59,070	$196,900
Escalation		$399,707	$516,863
TOTAL (base + risks + escalation)		**$1,541,727**	**$1,993,613**
Long Groyne			
Direct costs (preliminary estimate)	$855,000		
Site establishment, indirect costs	$50,000		
Builder's Risk Premium (ocean exposure)	$120,000		
Builder's Margin (10%)	$102,500		
Project management, internal costs	$50,000		
Design	$75,000		
Base Cost (Total of above)	**$1,252,500**		
Inherent Risk		$125,250	$375,750
Contingent Risk		$75,150	$250,500
Escalation		$508,515	$657,563
TOTAL (base + risks + escalation)		**$1,961,415**	**$2,536,313**
Southern Training Wall			
Direct costs (preliminary estimate)	$5,000,000		
Site establishment, indirect costs	$150,000		
Builder's Risk Premium (ocean exposure)	$400,000		
Builder's Margin (10%)	$555,000		
Project management, internal costs	$100,000		
Design	$130,000		
Base Cost (Total of above)	**$6,335,000**		
Inherent Risk		$633,500	$1,900,500
Contingent Risk		$380,100	$1,267,000
Escalation		$2,572,010	$3,325,875
TOTAL (base + risks + escalation)		**$9,920,610**	**$12,828,375**

Table G-3 Cost Estimate Breakdown – Northern Training Wall & Revetment

Option:		P50 Est	P90 Est
Northern Training Wall			
Direct costs (preliminary estimate)	$8,450,000		
Site establishment, indirect costs	$300,000		
Builder's Risk Premium (ocean exposure)	$1,300,000		
Builder's Margin (10%)	$1,005,000		
Project management, internal costs	$120,000		
Design	$400,000		
Base Cost (Total of above)	**$11,575,000**		
Inherent Risk		$1,157,500	$3,472,500
Contingent Risk		$694,500	$2,315,000
Escalation		$4,699,450	$6,076,875
TOTAL (base + risks + escalation)		**$18,126,450**	**$23,439,375**
Northern Revetment (850m long)			
Direct costs (preliminary estimate)	$2,800,000		
Site establishment, indirect costs	$80,000		
Builder's Risk Premium (ocean exposure)	$100,000		
Builder's Margin (10%)	$298,000		
Project management, internal costs	$70,000		
Design	$220,000		
Base Cost (Total of above)	**$3,568,000**		
Inherent Risk		$356,800	$1,070,400
Contingent Risk		$214,080	$713,600
Escalation		$1,448,608	$1,873,200
TOTAL (base + risks + escalation)		**$5,587,488**	**$7,225,200**

Notes:

P50 and P90 values: The P50 and P90 estimates represent the average expected cost (P50) and the cost for which there is a 90% confidence the estimate will not be exceeded (P90).

Direct costs are calculated on quantities estimated from the sections given on the attached drawings. This includes materials (rock) transport and placement.

Site establishment: Site establishment includes all costs required prior to actual construction under the contract such as site sheds, provision of services, setting up access roads and stockpile areas, etc. For construction of the northern training wall, an 500m long construction road will be required to reach along the sand spit from the car parking area. The construction areas to the south of The Entrance do not require such a road. An allowance for the road on the northern side and other indirect costs for site establishment has been made of $300,000 and apportioned to the training wall and revetment. Similar amounts have been included for the southern training wall and the groynes.

Builder’s Risk due to Ocean/Storm Hazard: The builder will need to carry a degree of risk related to potential ocean/storm damage. Such impacts may occur during construction where a portion of the constructed or incomplete works is washed away and has to be replaced. We have included an amount for this risk (Builder’s Risk Premium).

Project management and Design: These items include costs incurred by the principal for management of the construction work and the fees for preparation of design documentation.

The Base Cost is the sum total of the items above and represents the estimated cost based on the proposal and the quantities expected and this value is used to calculate the P50 and P90 estimates.

To calculate the actual project costs in terms of P50 and P90 values, some assessment of the risk of increased expenses are included for inherent risks, contingent risks and escalation of the project. These are usually assessed on a percentage basis of the Base Cost (see percentages given in the Table below).

Inherent Risk: This is an allowance for variability in the scope of work and in rates and quantities used in the estimate. This includes changes to costs of materials and variation in construction methods used that impact on actual costs at the time of construction.

Contingent Risk: Risk that is contingent on changes in the expected conditions of the job. It includes changes due to unmeasured items outside the base estimate (e.g. design development, owner or user requirements, etc.).

A significant example is the unknown ground conditions—the presence or absence of rock under the sands around The Entrance which could influence the costs of the Training Walls. This may occur because there would be reduced need for rock armour if the structures could be founded on existing rock strata. The carrying out of a Geotechnical Investigation including establishing where rock strata lie would reduce risk in the construction estimate. While it is possible that costs could be decreased or increased once this information is obtained, an additional percentage is included for Contingent Risk in the cost estimate.

Escalation: Escalation relates to the changes in the scope of a project that may occur during planning and development of the design. For example, if refining the design indicates that a better outcome would be achieved with a different extent or location for one of the training walls, the decision may be taken to change the design. Escalation has been estimated based on 15% for Identification and Scoping, 12% for Development and 8% for Delivery (total 35%).

Table G-4 Allowance Percentages for Risk Estimates

Risk assessed	P50 Estimate	P90 Estimate
Inherent Risk	10%	30%
Contingent Risk	6%	20%
Escalation (applied to base + inherent + contingent)	35%	35%

The percentages used are based on typical values used for transport construction projects in Australia.

APPENDIX H
WESTON (2013)

REPORT ON THE SAFETY OF NAVIGATION SHOULD TRAINING WALLS BE ESTABLISHED AT THE BARWAY ENTRY TO THE ENTRANCE IN NEW SOUTH WALES.

Prepared by Captain Charles Weston

24th April 2013

Contents:

SUMMARY:

This report has been prepared at the request of Dr D. Treloar, a Senior Principal, Coastal Engineering, at Cardno Pty Ltd. Cardno have been requested by the NSW Office of Environment, and Heritage (OEH) to conduct a feasibility study into the feasibility of constructing training walls to increase water flow into, and out of, the Tuggerah Lakes system. At the present time, water flow is through the bar-way entry to The Entrance. This bar-way entry is to the south of a sand spit which constantly changes position due, primarily, to catchment flooding and sea conditions. These changes consequently have a direct effect on the condition of the entrance and volume of water flow into and out of the lake system. Wyong Shire Council also undertakes dredging of the entrance waterway in the region between The Entrance Bridge and the entrance bar-way.

The building of two training walls in the vicinity of Karagi Point has been proposed as a method whereby a reasonably predictable flow/volume of water can be facilitated into and out of the lakes. Cardno have investigated a range of training wall opening widths (100m, 150m, and 200m) and 150m would be the most suitable from flooding effects and entrance

The proposed possible training walls superimposed in their approximate positions.

scouring perspectives. None of these cases changes tidal exchange to the lake system.

Cardno have now been engaged by OEH to assess the effects (benefits and deleterious outcomes) of training walls and other options, such as beach nourishment, on South and North entrance beaches. While the prime purpose of the proposed training walls was to facilitate water flow, they may have benefits in terms of shoreline hazard reduction. It is considered by OEH that the navigational aspects of the entry should also be addressed, given the nature and use of the surrounding area during the holiday season and its use by recreational boaters.

This report addresses recommendations for the safety of boaters navigating within the area, with and without training walls.

THE PRESENT SITUATION:

The Bar-way Entry

The sand deposition at the entry is constantly changing due to catchment flooding and sea conditions. This is evident from aerial photos and Google Earth images. The changes may also be due to the use of a dredger, operated by the Wyong Shire Council. This suction dredger, in an effort to increase water flow, removes sand from the western side of the sand spit and deposits the spoil in the region of Dunlieth Point, both on the lagoon side (near the caravan park where shoreline erosion occurs) and the seaward side where storm erosion causes a hazard development. Council has also, in the past, pumped sand to the Surf Club beach which is south of the bar-way.

The rock shelf is evident in the foreground with the outgoing water breaking on it and the bar-way itself in the distance. This photo was taken just before low water and on an almost calm day. (Photo taken at 1415 on 9/4/13)

The base of the entry, inshore of the bar-way, is rock and at low tide has an approximate depth of 0.3m and is virtually non-navigable except by vessels such as jet skis and kayaks. The bar-way itself, the area where the outgoing water meets the sea, is also constantly changing and shallow, and with even a moderate onshore wind and sea/swell, it would be considered dangerous should any attempt be made to navigate the entry.

The same area at high tide. Note that the rock shelf is not evident due to the depth being approximately 1.0 to 1.5m. (Photo taken at 0815 on 10/4/13)

Existing use of the bar-way for navigation

Following discussions with the Senior Boating Safety Officer of NSW Maritime, it would appear that local fishermen have been known to use this entrance and it is also used by persons on jet skis and kayaks. The writer was informed that there was a boating incident some 3 years before, however, this did not involve loss of life.

Safety signage

The writer visited the closest boat ramp to the entry located at Picnic Point and noted that there was no safety signage relating to the dangers of the bar-way. In fact there was no safety signage at all. There is also a boat ramp close to the entry but this has been withdrawn from use. Whilst there is existing signage none of it relates to safe navigation or the bar-way.

The only evidence of warning is published in the NSW Maritime boating map of the area and states as follows; “Caution: Navigation of the Lake Entrance is dangerous and not recommended”.

<u>Alternative boat ramps that are available for boaters to access the sea</u>

To the north is the Cabbage Tree Harbour boat ramp at Norah Head. This ramp is approximately 12 km distant and is reasonably sheltered. To the south is the Terrigal Haven boat ramp, which is sheltered and used by both recreational and commercial boaters. In both instances, these ramps would be a much better alternative for access to the sea and can be used at any state of the tide.

The Cabbage Tree Harbour boat ramp at Norah Head

RECOMMENDATIONS:

Should the Training Walls be Constructed

Available depth

The depth of water between the walls at low water will determine, by its draft, the size of vessel which may safely navigate the entry. Consequently, if the existing rock shelf remains, then the navigational availability of the entrance will essentially remain the same and be severely restricted and could only be used by vessels with small drafts at pre-determined periods either side of high water.

In addition, the seaward entry to the training walls would still be a bar-way and sea and weather conditions may further restrict its use.

Should the depth of the entry between the walls be increased by dredging, this would facilitate its use by larger vessels at all states of the tide.

Management of vessel movements

Given the foregoing, considerable planning would be required to oversee and manage navigation into and out of, the entry. This could be done with the assistance of the existing Volunteer Marine Rescue - Tuggerah Base, if they were willing, and were provided with appropriate closed circuit television views of the entrance and also water depth, wind speed/direction and wave height read outs. The assistance of the base could then be sought from a vessel wishing to transit the entry by use of VHF or UHF radio and also mobile phone.

Appropriate signage would play an important part in the safety management of navigation through the entry. This signage would be placed at either end of the walls and inform boaters of the need to contact the Marine Rescue Base prior to transiting the entrance whether from the sea or the lagoon end. Signage would also carry the usual warnings, regulatory requirements and advice when crossing bar-ways.

Safety information signage should also be erected at the nearby boat ramps to inform boaters, planning to go to sea via the entrance, of the safety requirements and the need to contact Marine Rescue.

Navigation marks

The training walls would require the fitting of appropriate red and green navigation marks and lights at both ends of each wall. In addition, offshore buoyage, possibly a north cardinal mark, should also be installed due to the close proximity of dangerous rocks to the southeast.

Leading marks for use of vessels approaching the training walls from seawards should also be considered. A sectored night/day laser light positioned on the western shore of the lagoon would be simple to set up and less obtrusive than the traditional day and night types of leading marks.

Due to the possibility of sand build-up in the lagoon and to seawards, moveable lateral buoyage marks would assist boaters navigating the area once they are outside the training walls. However, the strong possibility of the sand build-up moving will have to be taken into consideration.

Should the Present Situation Continue

Comments on the existing bar-way entry

In the opinion of the writer, the use of this entry and bar-way by any vessel is considered dangerous and should not be attempted. However, in reality, it is used by some local boaters and in this regard every effort should be made by the Wyong Shire Council and the NSW Maritime to draw the attention of these boaters to the dangers that are involved.

This safety information should be in the form of signage, information pamphlets, boating maps and safety notices posted on the web. At the present time the only information available is the NSW Maritime boating map for the area and verbal information from the Marine Rescue Base.

Signage

While there is a plethora of signage at the various boat ramps visited by the writer, very little addresses safety information and reminders to boaters. There are no signs at The Entrance boat ramp facilities warning of the dangers of the entrance bar-way.

Signage should be placed at the three nearby boat ramps drawing the attention of boaters to the fact that the entrance bar-way is considered dangerous and its use is not recommended. In addition a large sign should be located on the south-western shore of the entrance waterway, upstream of the rock sill, pointing towards the water in order to inform persons in vessels close to the entrance about the dangerous situation should the use of the entrance be contemplated.

Information pamphlets and safety notices posted on the web

Suitable information pamphlets could be drawn up on the dangers of the entry bar-way and made available through such outlets as fishing tackle shops and the tourist information centre and also the Marine Rescue organisation. Similarly, web notices could be posted on the NSW Maritime and Marine Rescue websites.

CONCLUSIONS:

1. The existing entrance and bar-way is dangerous in low tide and on-shore wind and sea conditions, consequently it should not be used by boaters, especially those who are not familiar with the area.

2. Should the building of the training walls eventuate then this may provide a facility where safe access to and from the sea by boaters will be possible. However, it must be remembered that the prime purpose of the walls is to provide an increased water flow to and from the Tuggerah Lake system and its use for navigation will only be of a secondary consideration. Consequently, if the depth of water between the walls is not increased from the present depth in the area, due to the rock bottom, then it will have limited use for navigation by boaters - similar to the existing case.

3. More and more members of the public are buying and using small boats and as such, public authorities such as the Wyong Shire Council and NSW Maritime must take on the responsibility of drawing the attention of these boaters to any potential dangers that may be in the geographical areas under their authority. In this case, the bar-way entry to The Entrance.

4. This report is relatively brief and covers the present situation. If, in the future, the navigational situation changes at The Entrance, then it is recommended that a more comprehensive study, together with recommendations, be undertaken.

5. The safe navigation of any vessel of any size is the responsibility of the person in charge of that vessel. However, that person can be assisted in his or her decision making when good and suitable safety information is made available.

This report is compiled without prejudice.

Capt. Charles Weston

24th April, 2013

ACKNOWLEDGEMENTS:

I would like to thank the following persons, who assisted me with information which I used to compile this report.

Mr Doug Treloar	Cardno Pty Ltd.
Mr Vince Cubis	Cardno Pty Ltd.
Mr Richard Weller	Cardno Pty Ltd.
Mr Peter Evans	NSW Dept. of Environment, Climate Change and Water.
Mr Neil Kelleher	NSW Dept. of Environment, Climate Change and Water.
Mr Darryl Lennox	Senior Boating Safety Officer, NSW Maritime

FIGURES

Cardno
Shaping the Future

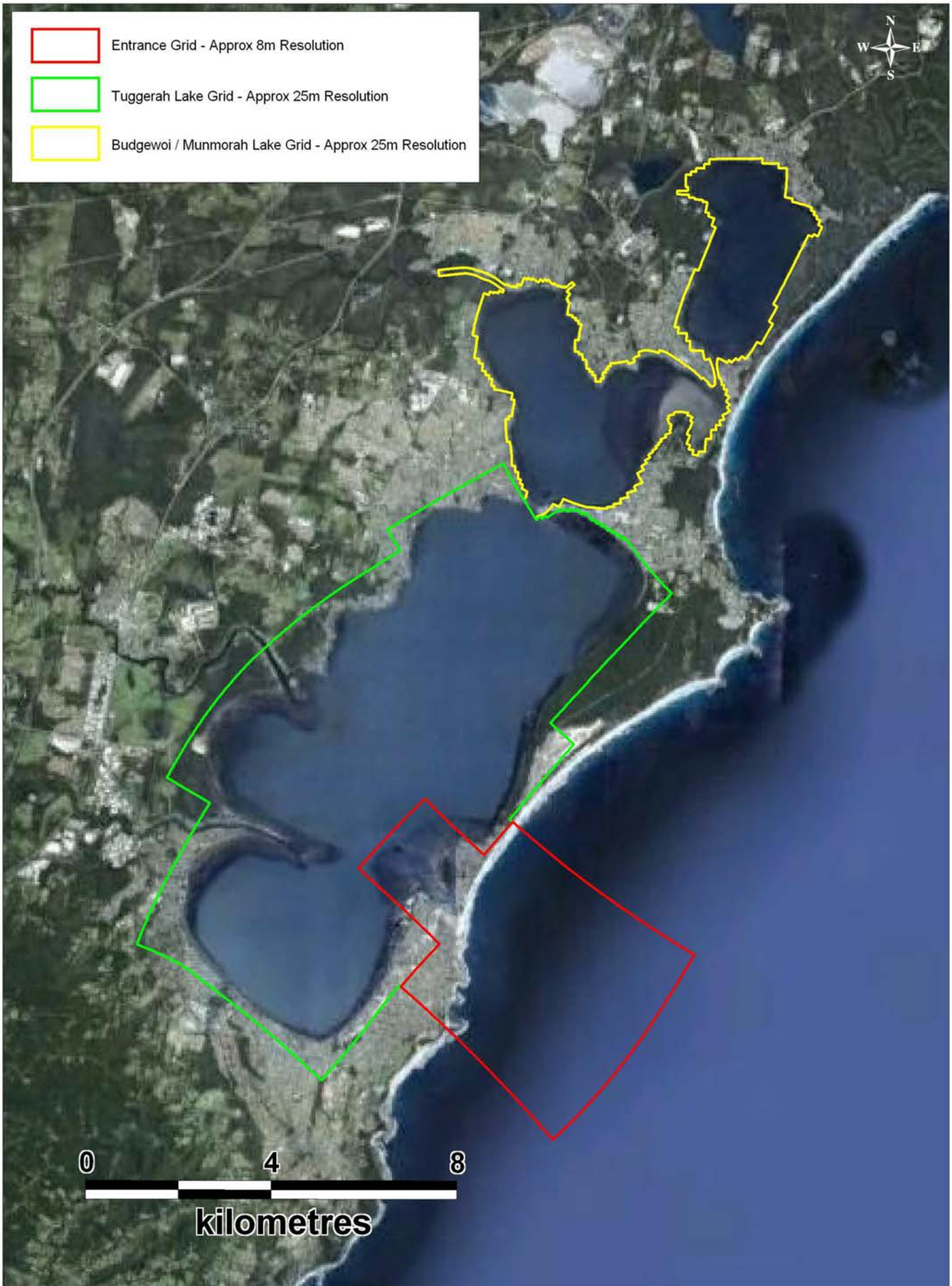
Cardno

Tuggerah Lakes − The Entrance Morphodynamic Modelling

Locality Plan
Tuggerah Lakes & The Entrance

Figure 1.1

LJ2985/R2791 Oct 2013
J:\CM\LJ2985−TheEntrance\009−ReportingDocsandFigures\Figures\



Cardno

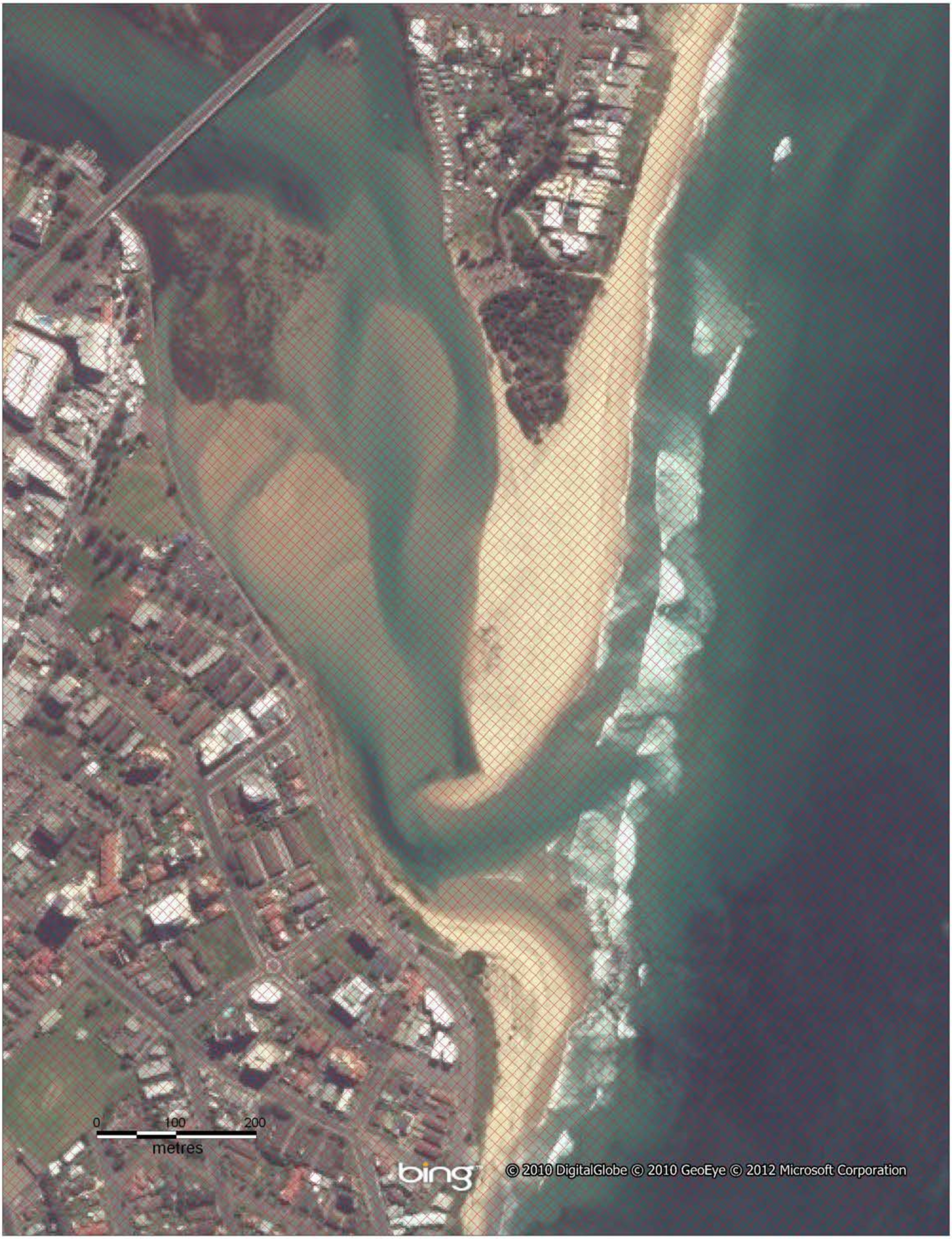
Tuggerah Lakes − The Entrance Morphodynamic Modelling

Grid Set−up
D3D Hydrodynamic Model

Figure 3.1a

LJ2985/R2791 Oct 2013

J:\CM\LJ2985−TheEntrance\009−ReportingDocsandFigures\Figures\2013−06−14−FinalReport\



Cardno

Tuggerah Lakes − The Entrance Morphodynamic Modelling

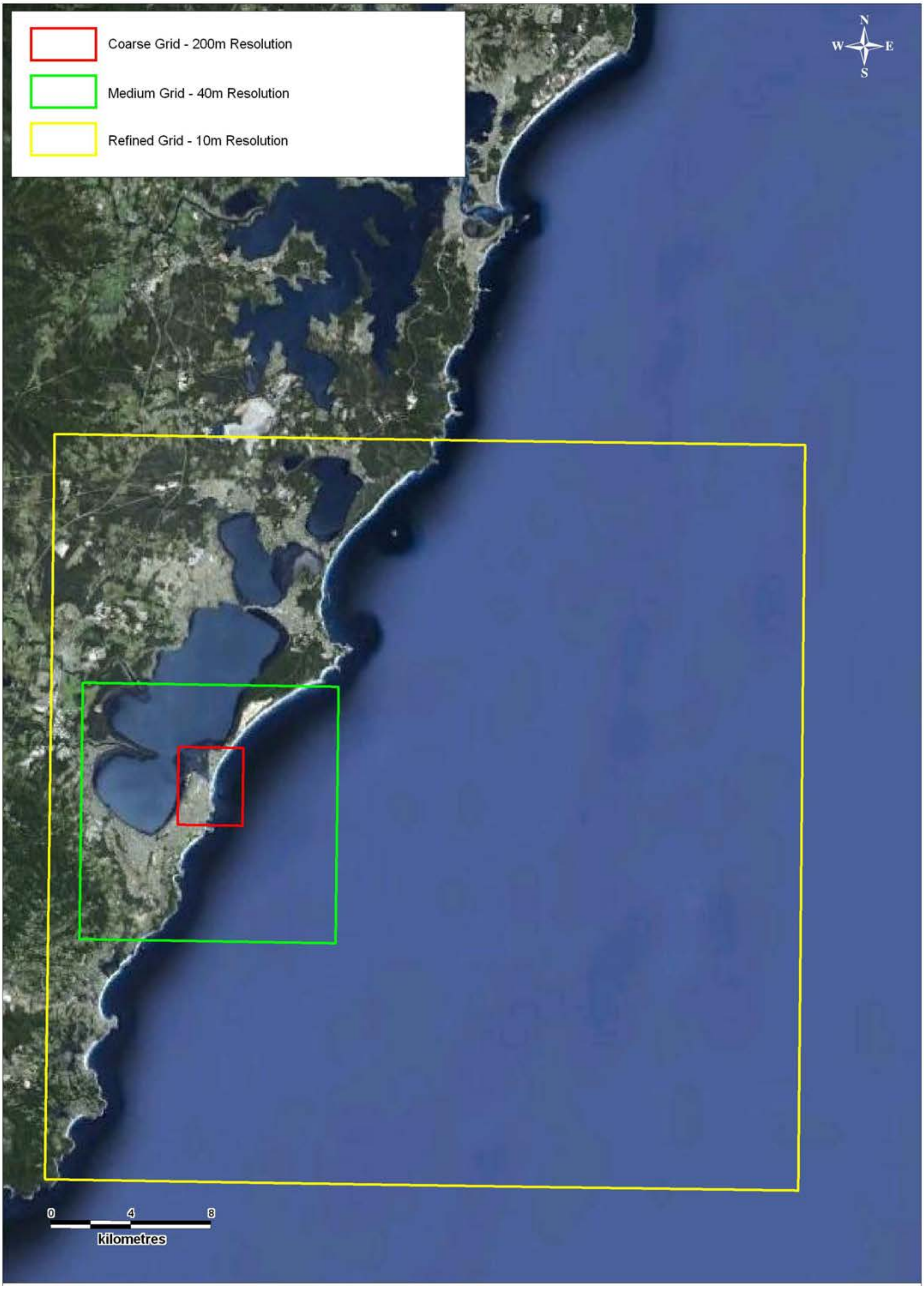
Grid Set−up

D3D Hydrodynamic Model − Entrance Grid

LJ2985/R2791 Oct 2013

J:\CM\LJ2985−TheEntrance\009−ReportingDocsandFigures\Figures\2013−06−14−FinalReport\

Figure 3.1b



Cardno

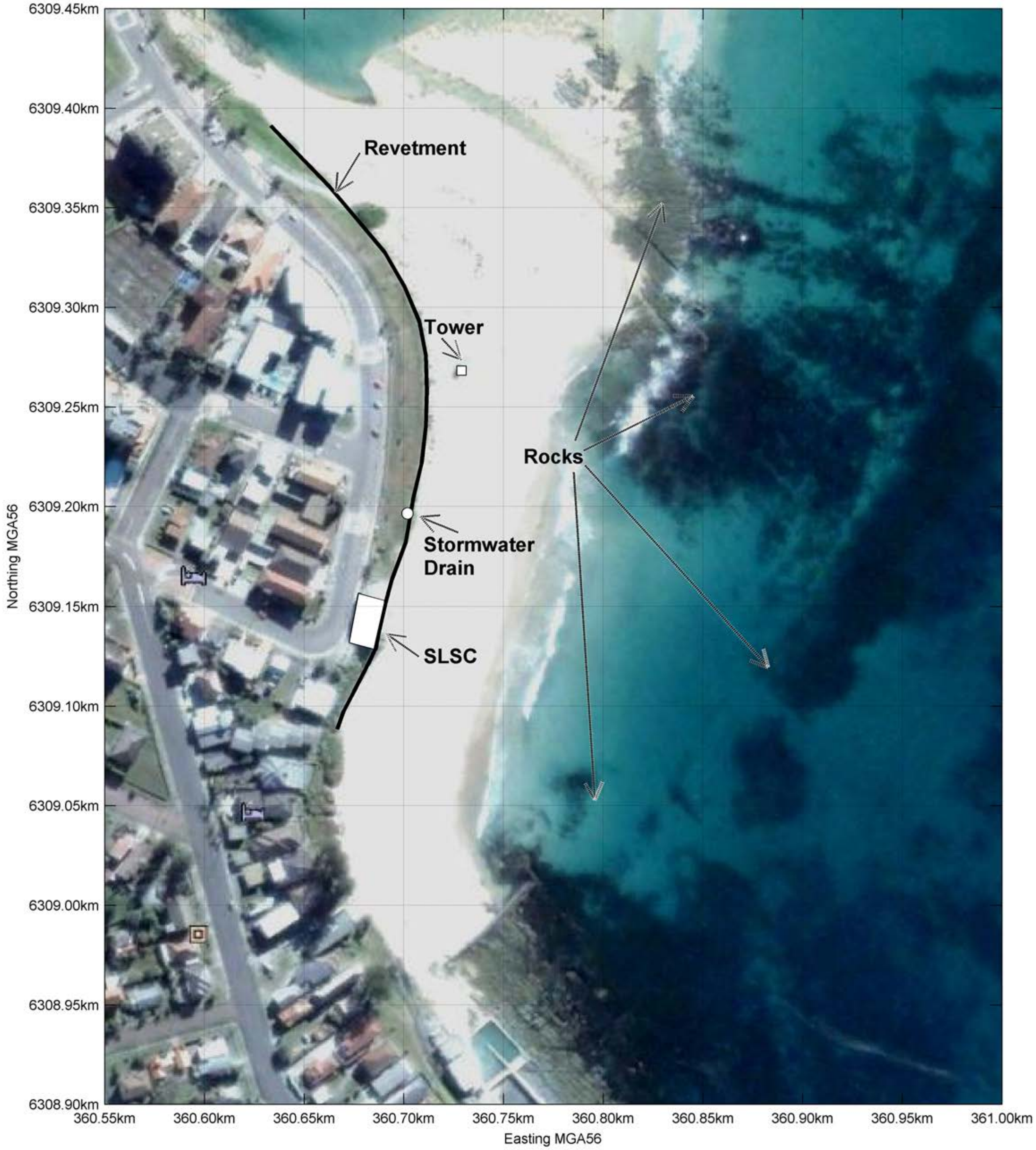
Tuggerah Lakes − The Entrance Morphodynamic Modelling

Grid Set−up
SWAN Wave Model

Figure 3.1c

LJ2985/R2791 Oct 2013

J:\CM\LJ2985−TheEntrance\009−ReportingDocsandFigures\Figures\2013−06−14−FinalReport\



Cardno

LJ2985/R2791 Oct 2013

J:\CM\LJ2985−TheEntrance\009−ReportingDocsandFigures\Figures\

Tuggerah Lakes − The Entrance Morphodynamic Modelling

Locality Plan
South Entrance Beach

Figure 5.1

Tuggerah Lakes − The Entrance Morphodynamic Modelling

Aerial Map
20/06/2005 − Google Earth

Figure 5.2a

LJ2985/R2791 Oct 2013

J:\CM\LJ2985−TheEntrance\009−ReportingDocsandFigures\Figures\

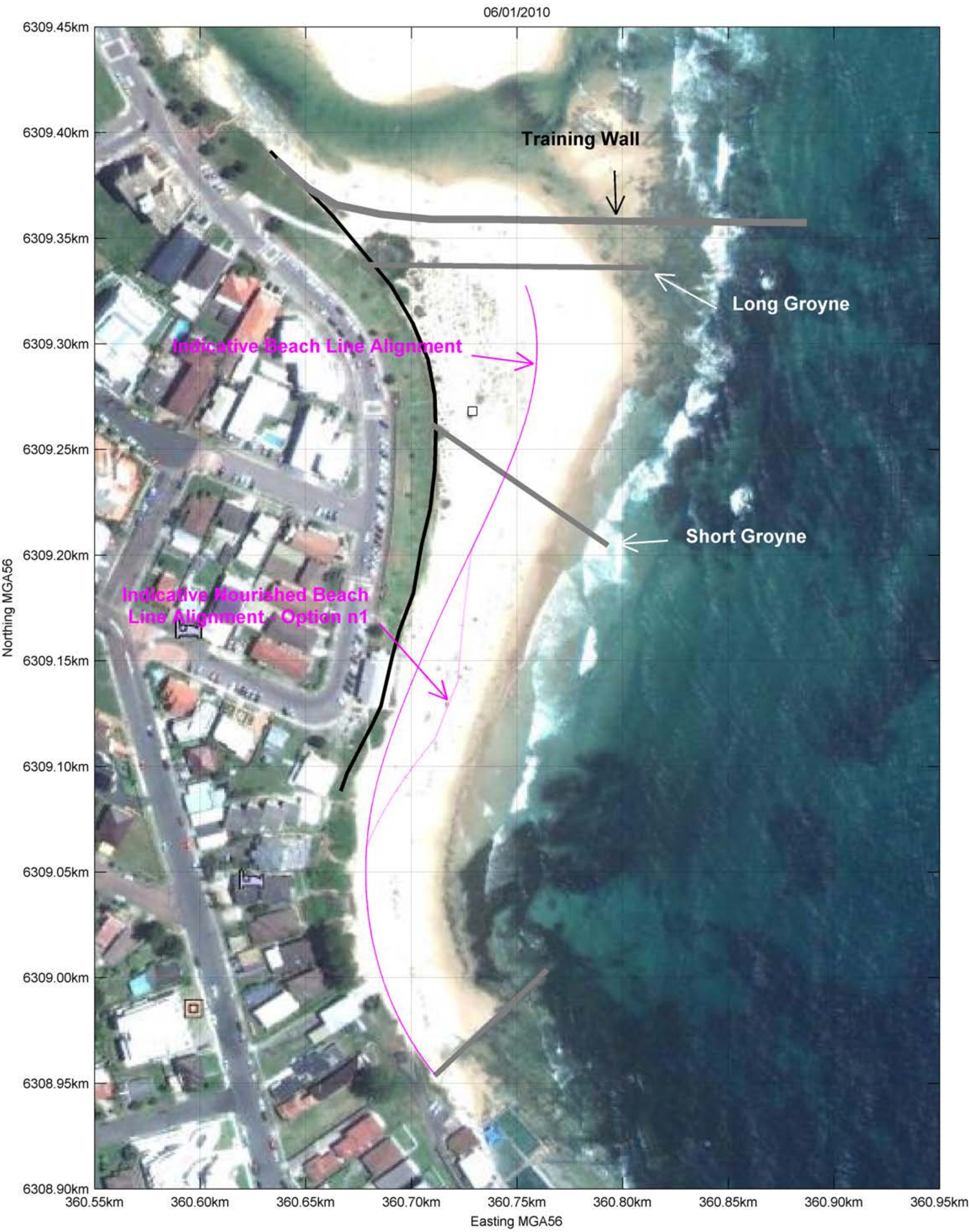
Tuggerah Lakes − The Entrance Morphodynamic Modelling

Aerial Map
11/01/2007 − Google Earth

Figure 5.2b

LJ2985/R2791 Oct 2013

J:\CM\LJ2985−TheEntrance\009−ReportingDocsandFigures\Figures\



Tuggerah Lakes − The Entrance Morphodynamic Modelling

Aerial Map
06/01/2010 − Google Earth

Figure 5.2c

LJ2985/R2791 Oct 2013

J:\CM\LJ2985−TheEntrance\009−ReportingDocsandFigures\Figures\

Cardno

Tuggerah Lakes − The Entrance Morphodynamic Modelling

Aerial Map
12/02/2010 − Google Earth

Figure 5.2d

LJ2985/R2791 Oct 2013
J:\CM\LJ2985−TheEntrance\009−ReportingDocsandFigures\Figures\

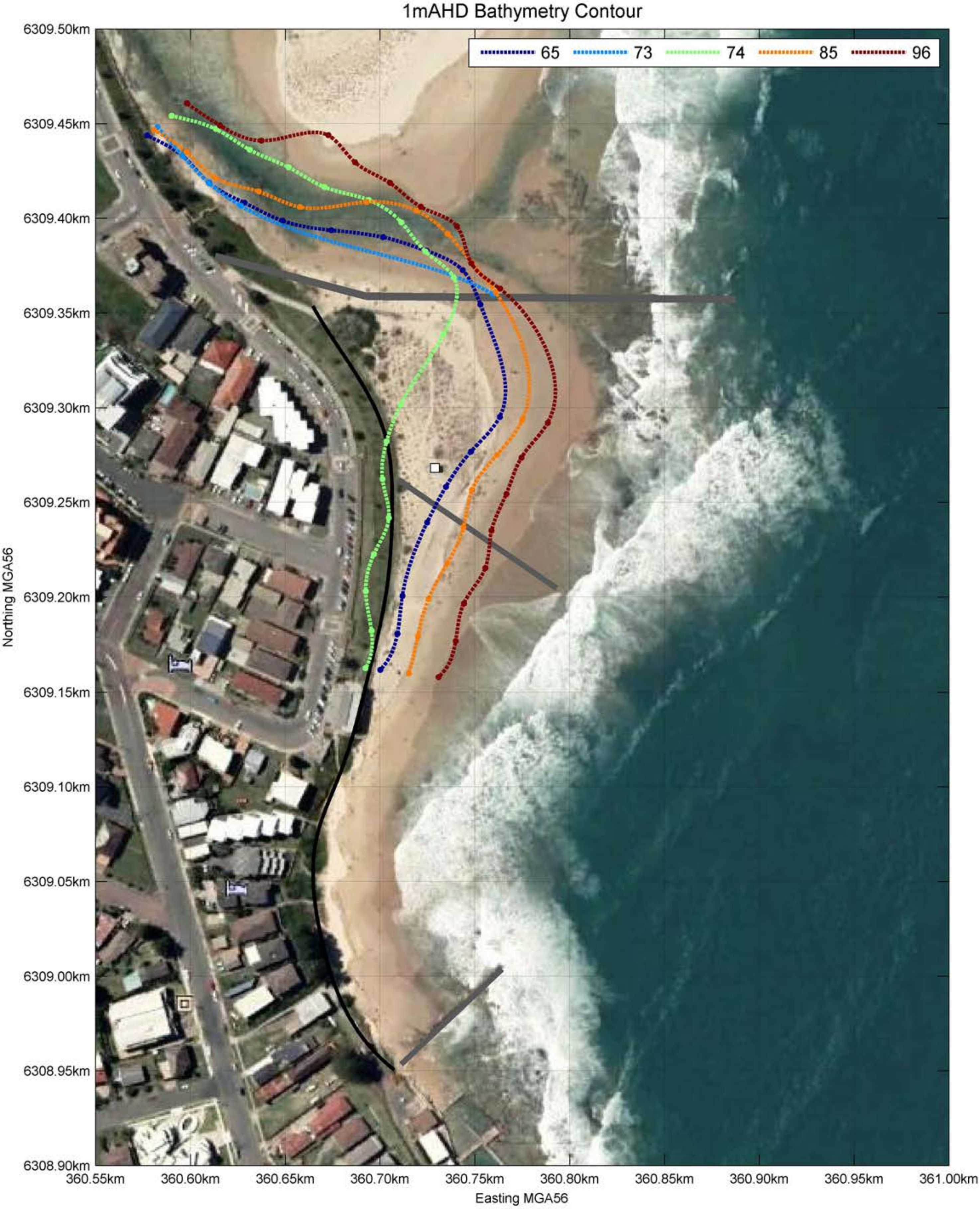
Cardno

Tuggerah Lakes − The Entrance Morphodynamic Modelling

Aerial Map
23/06/2012 − Google Earth

Figure 5.2e

LJ2985/R2791 Oct 2013
J:\CM\LJ2985−TheEntrance\009−ReportingDocsandFigures\Figures\



Cardno

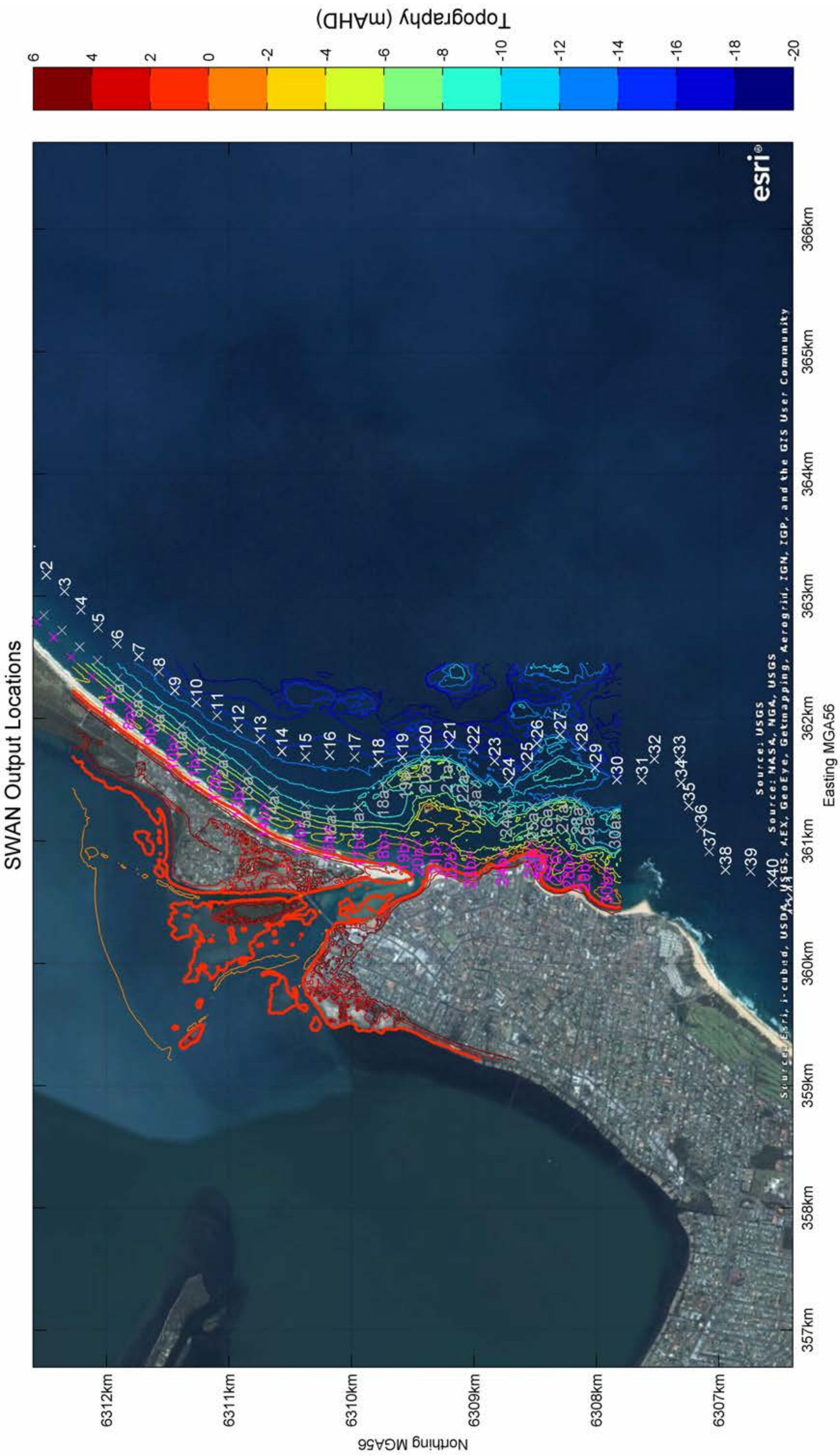
Tuggerah Lakes − The Entrance Morphodynamic Modelling

Coastline Evolution − 1 mAHD Elevation Contour
Years: 1965, 1973,1974,1985 and 1996

Figure 5.3

LJ2985/R2791 Oct 2013

J:\CM\LJ2985−TheEntrance\009−ReportingDocsandFigures\Figures\



Tuggerah Lakes − The Entrance Morphodynamic Modelling

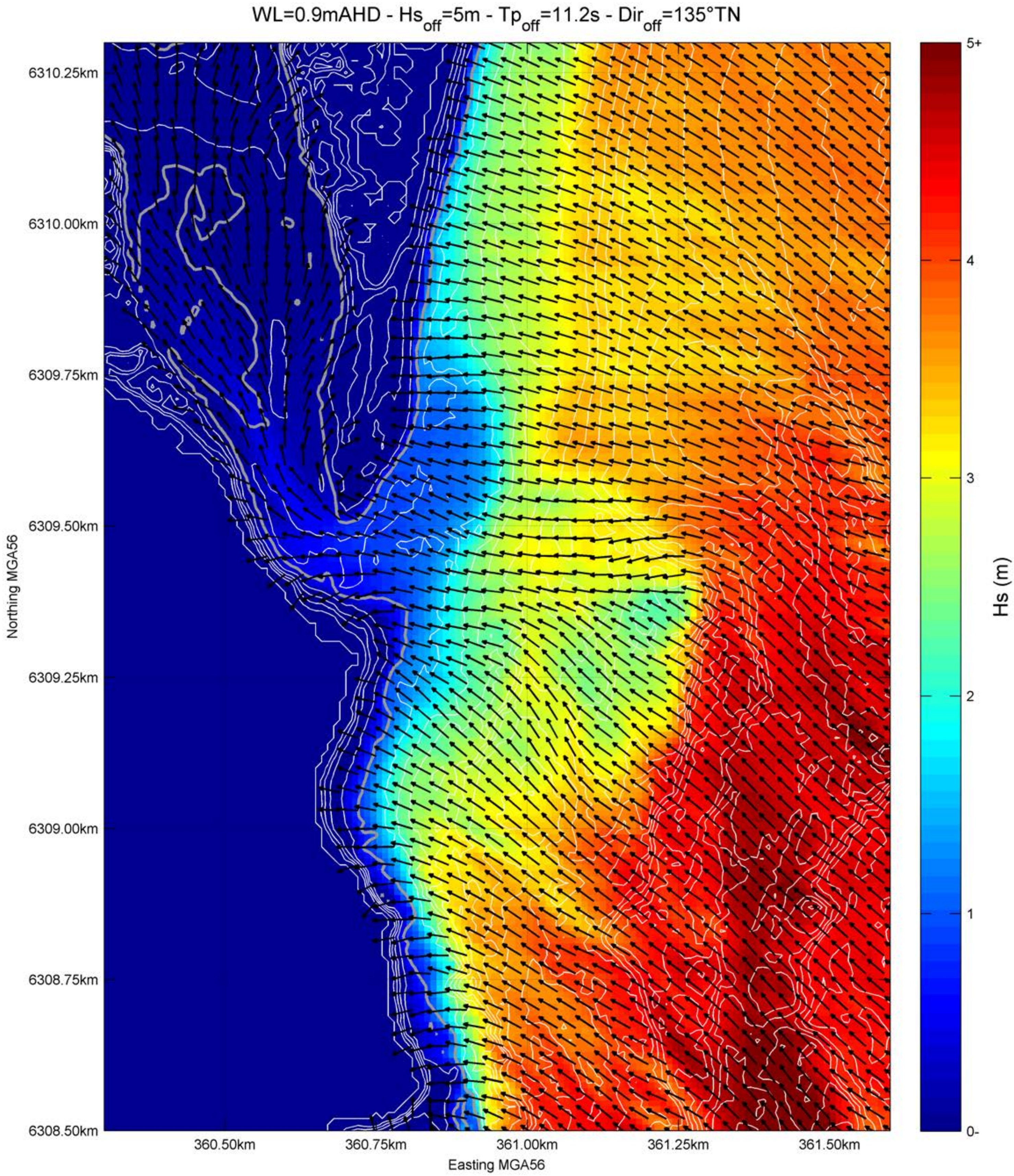
SWAN Output Locations and Modelled Bathymetry

Cardno

LJ2985/R2791 Oct 2013

J:\CM\LJ2985−TheEntrance\009−ReportingDocsandFigures\Figures\

Figure 5.4



Cardno

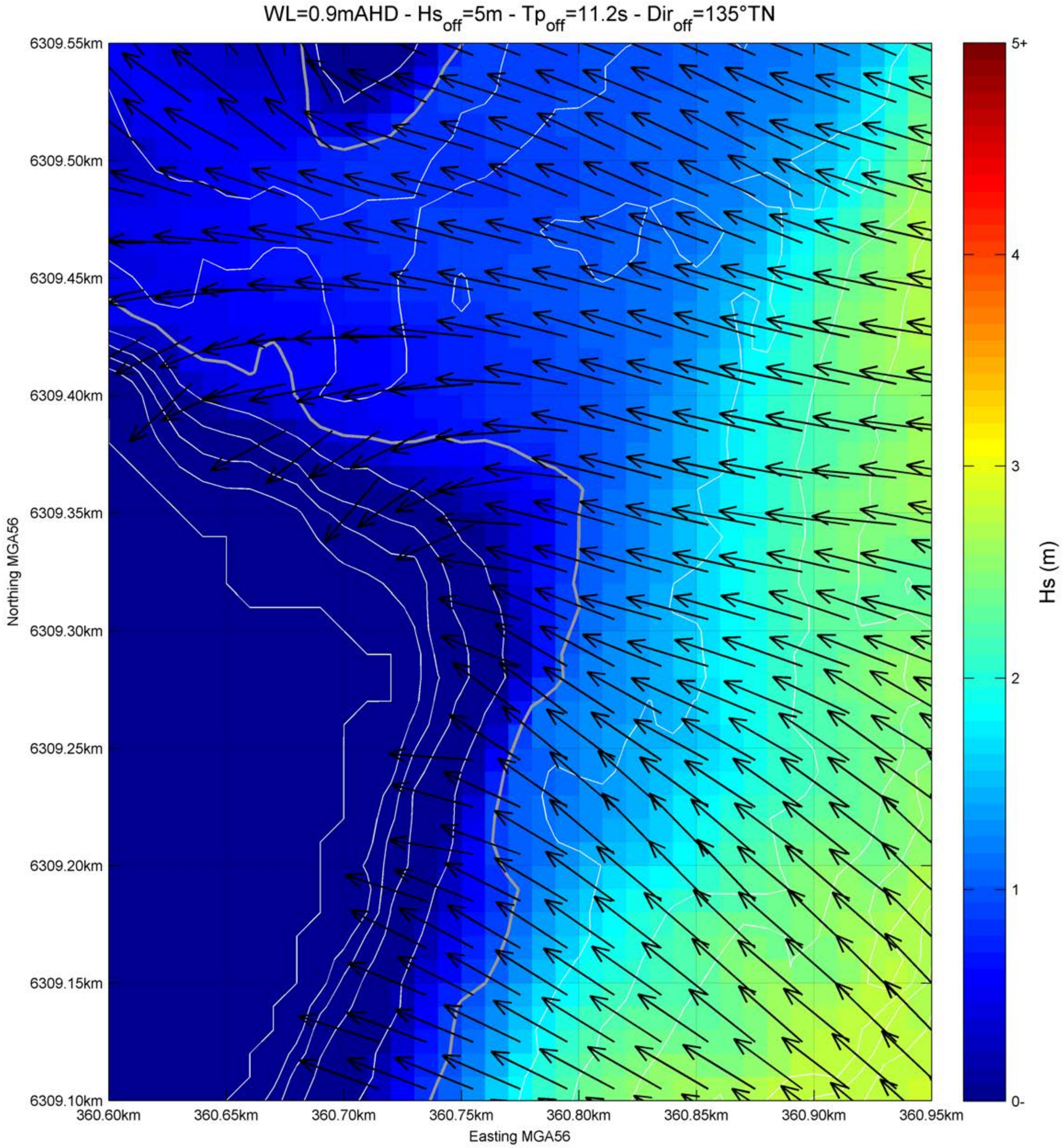
Tuggerah Lakes − The Entrance Morphodynamic Modelling

Wave Map at High Water

LJ2985/R2791 Oct 2013

J:\CM\LJ2985−TheEntrance\009−ReportingDocsandFigures\Figures\

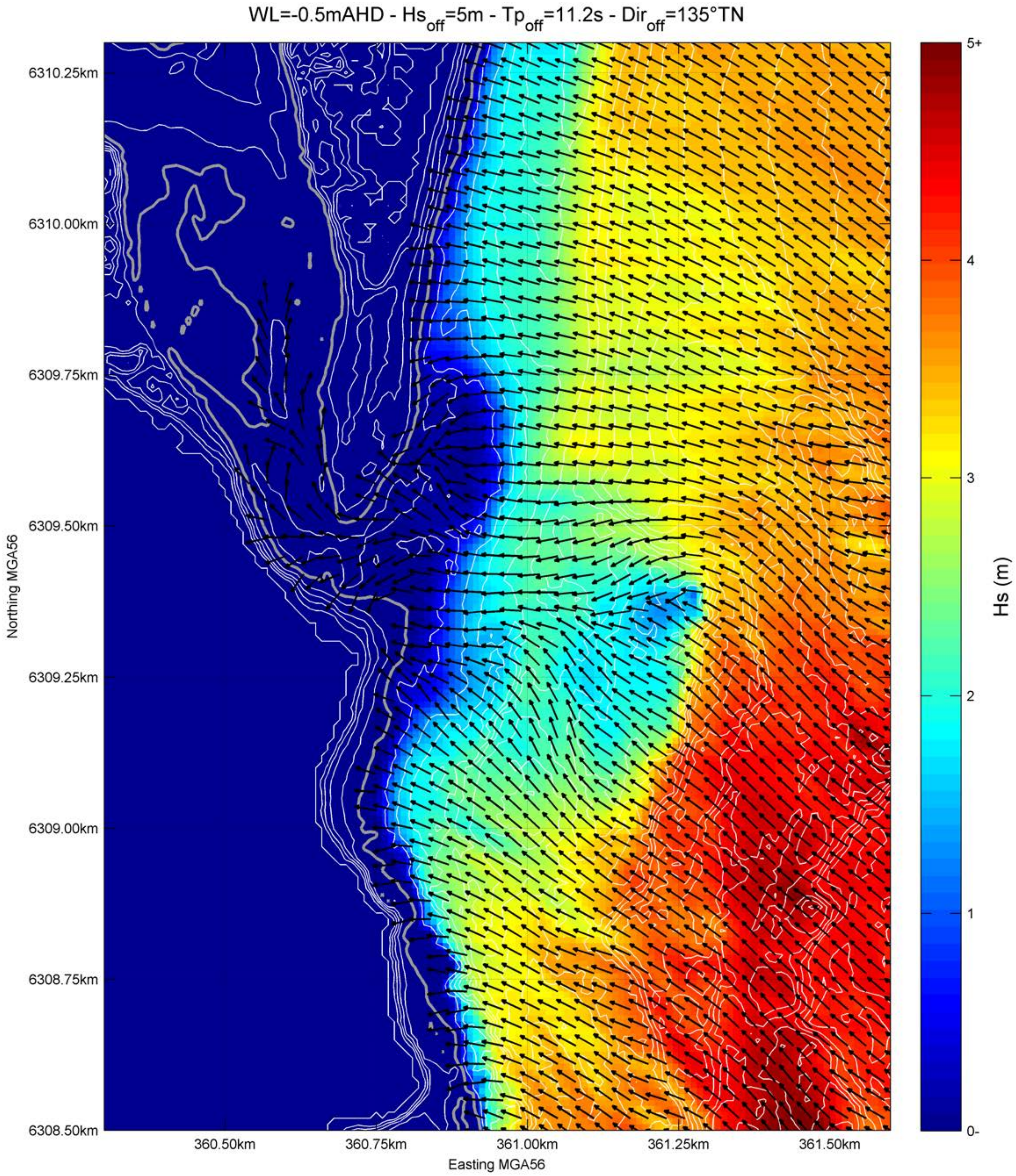
Figure 5.5a



Tuggerah Lakes − The Entrance Morphodynamic Modelling

Wave Map at High Water
Zoom−in at Tombolo Area

Figure 5.5b

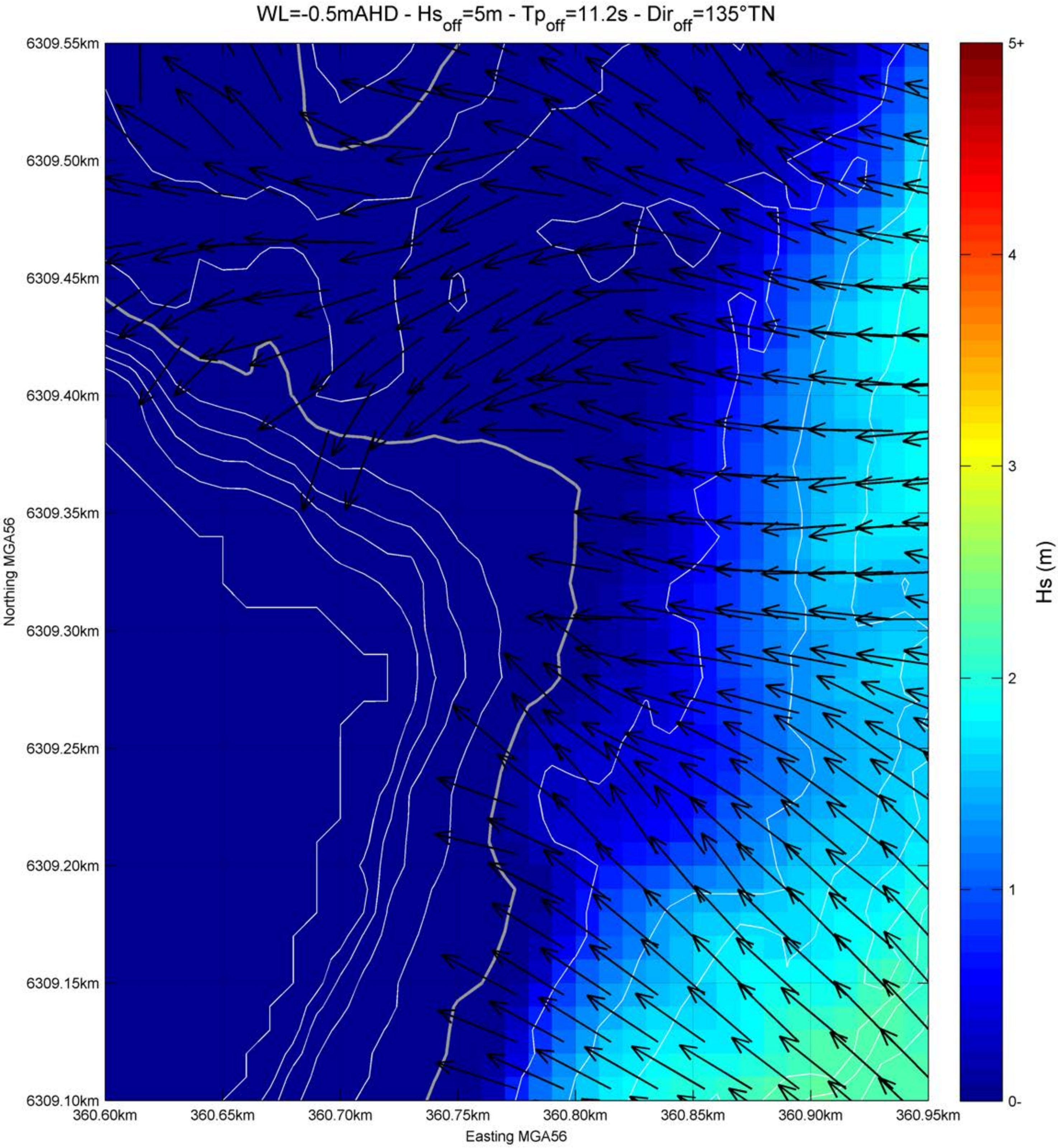


Cardno

Tuggerah Lakes − The Entrance Morphodynamic Modelling
Wave Map at Low Water

LJ2985/R2791 Oct 2013
J:\CM\LJ2985−TheEntrance\009−ReportingDocsandFigures\Figures\

Figure 5.6a



Cardno

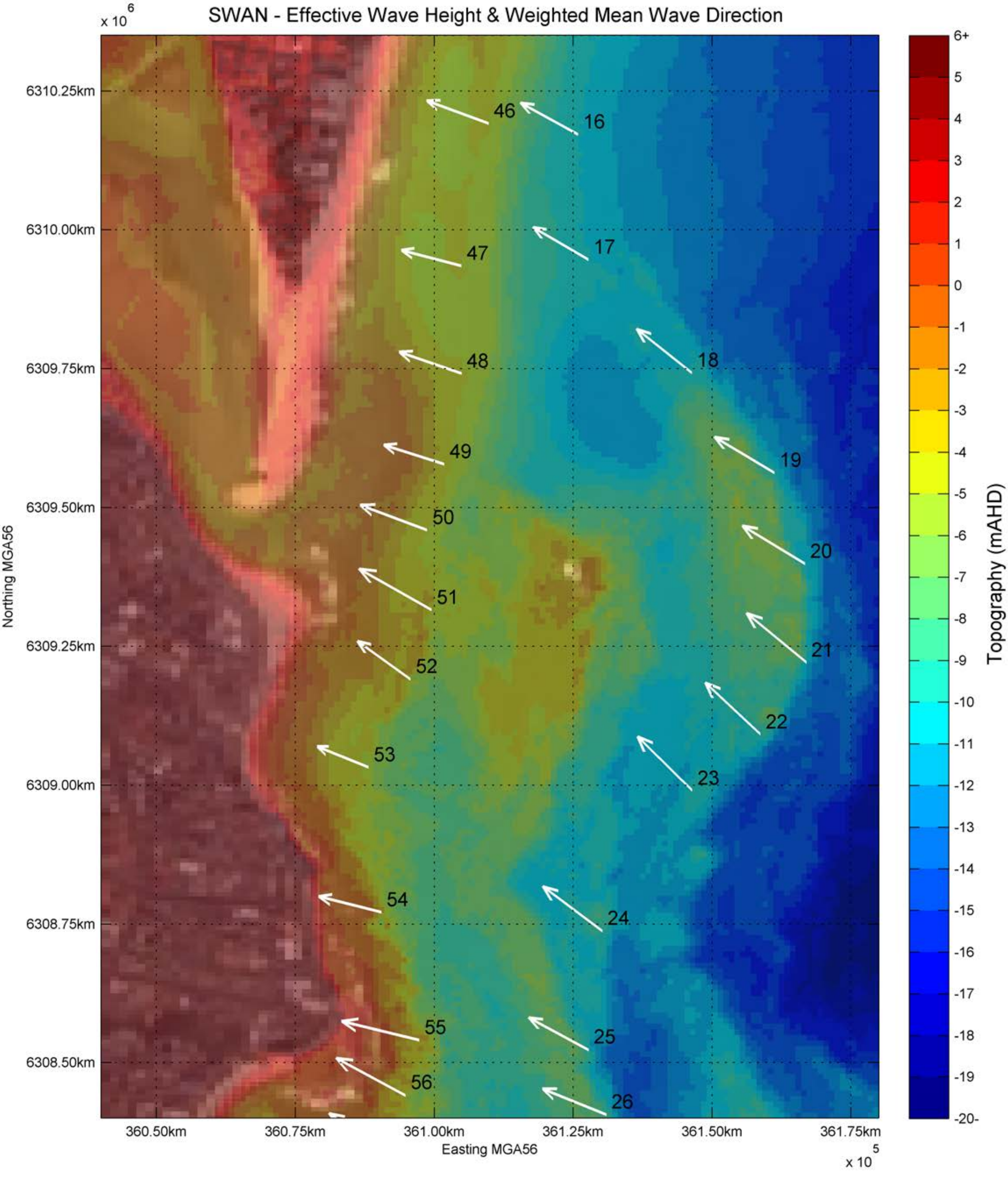
Tuggerah Lakes − The Entrance Morphodynamic Modelling

Wave Map at Low Water
Zoom−in at Tombolo Area

Figure 5.6b

LJ2985/R2791 Oct 2013

J:\CM\LJ2985−TheEntrance\009−ReportingDocsandFigures\Figures\



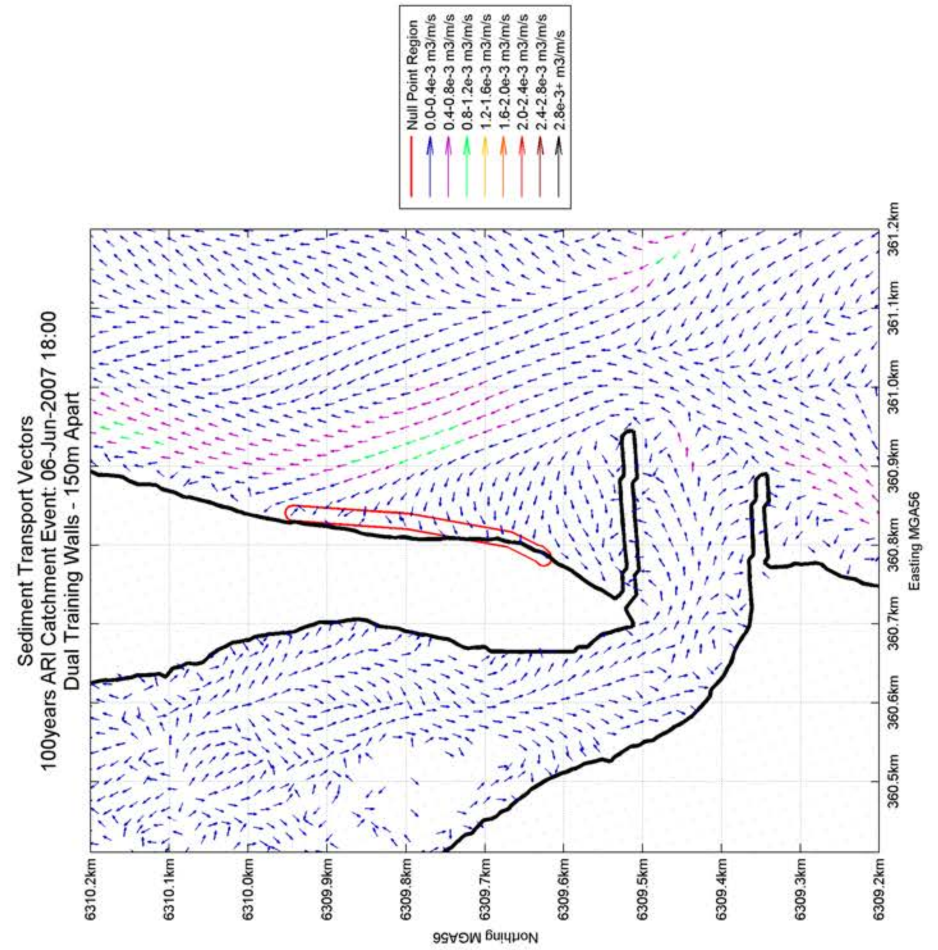
Tuggerah Lakes − The Entrance Morphodynamic Modelling

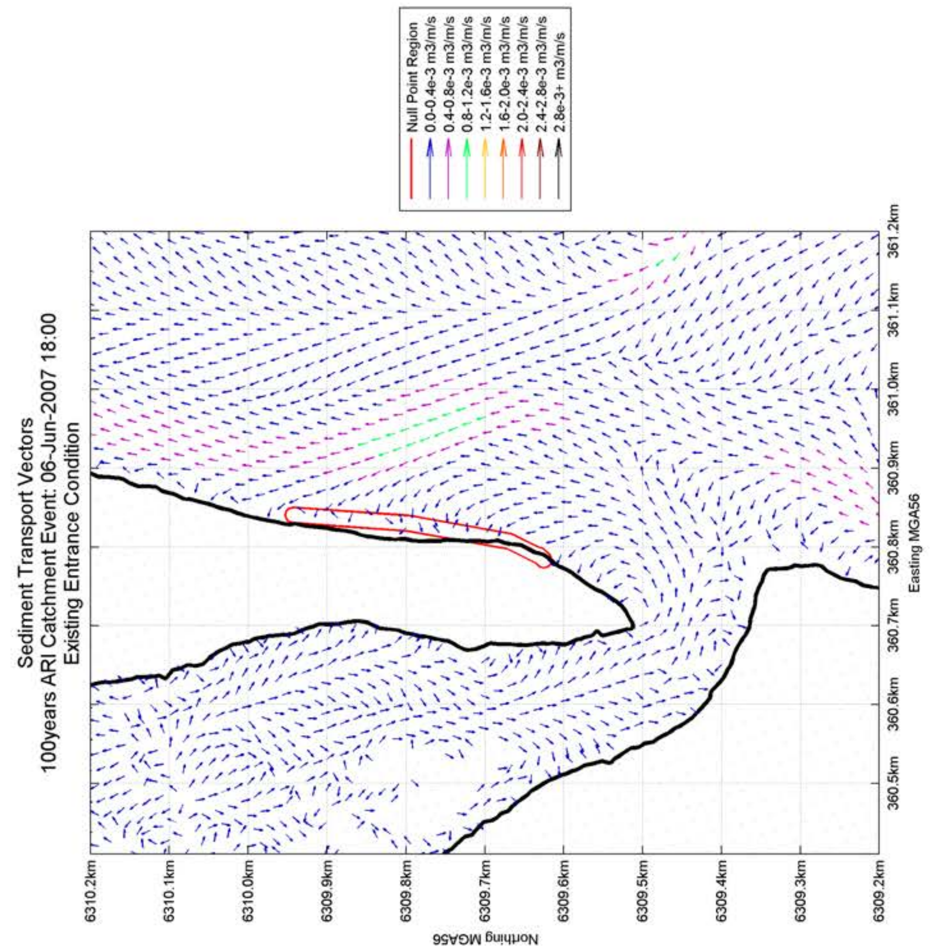
Wave Map of Effective Height and Mean Wave Direction

LJ2985/R2791 Oct 2013

J:\CM\LJ2985−TheEntrance\009−ReportingDocsandFigures\Figures\

Figure 5.7





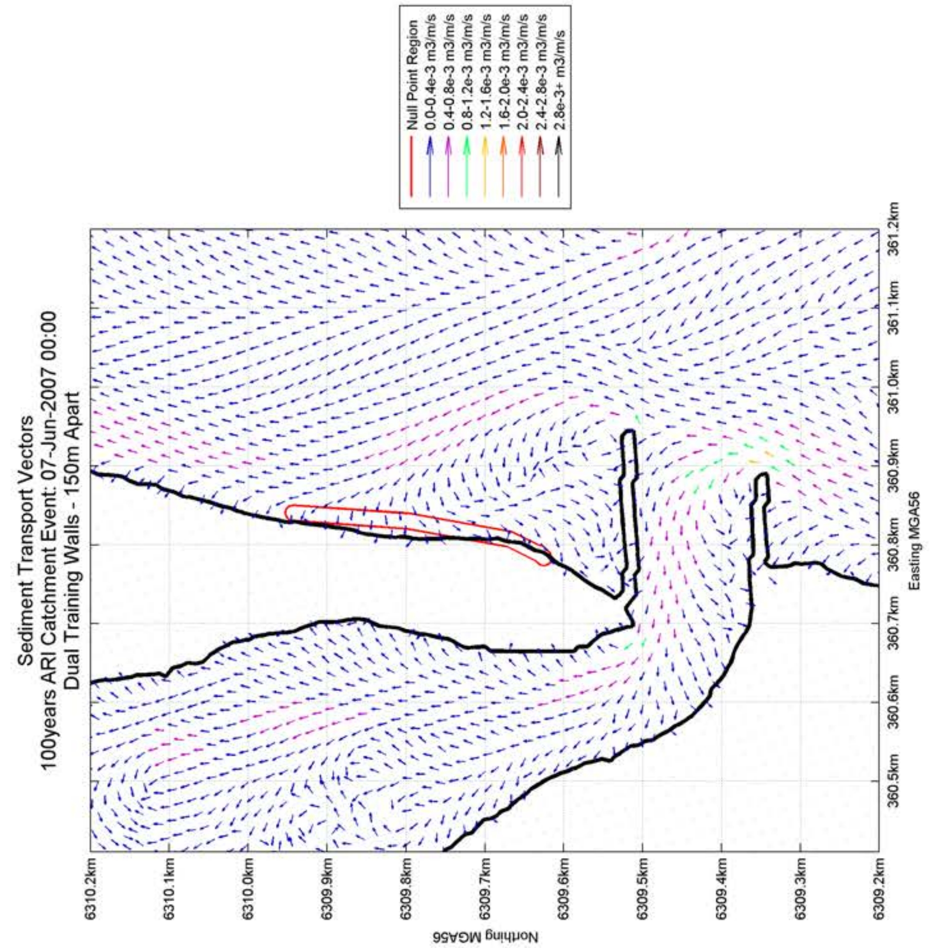
Tuggerah Lakes − The Entrance Morphodynamic Modelling

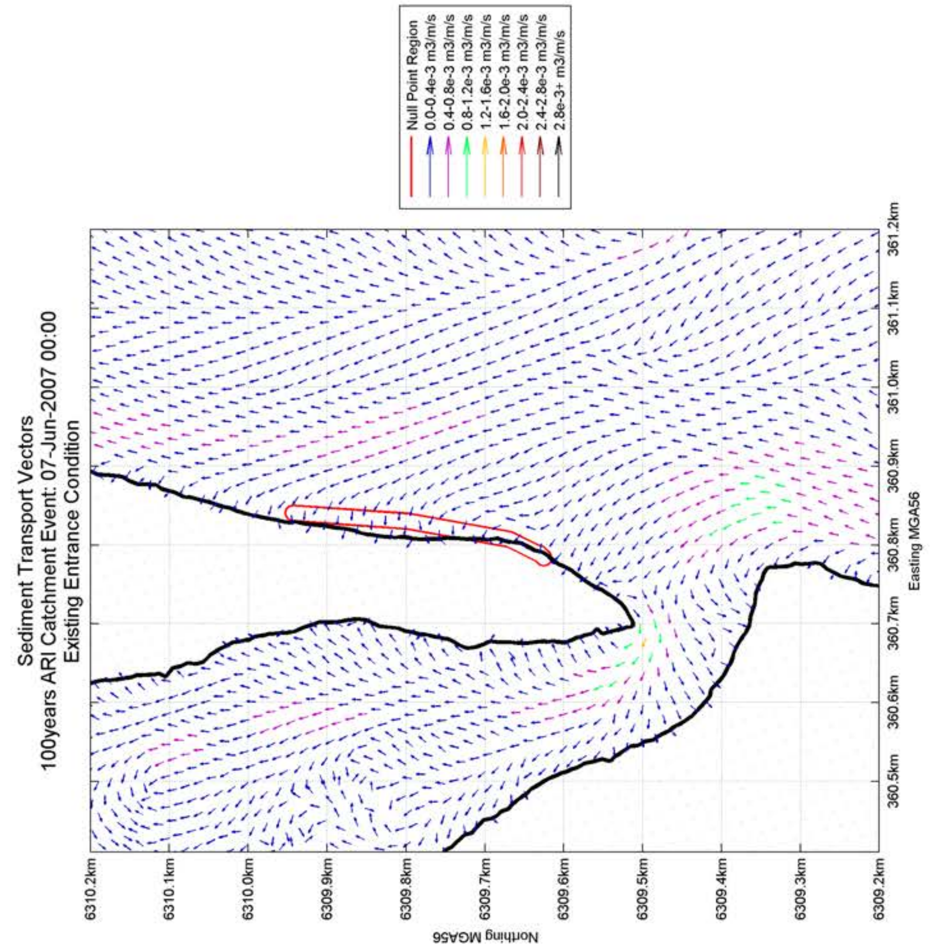
Sediment Transport Vectors
and Approximate Null Point Location

Figure 5.8a

LJ2985/R2791 Oct 2013

J:\CM\LJ2985−TheEntrance\009−ReportingDocsandFigures\Figures\



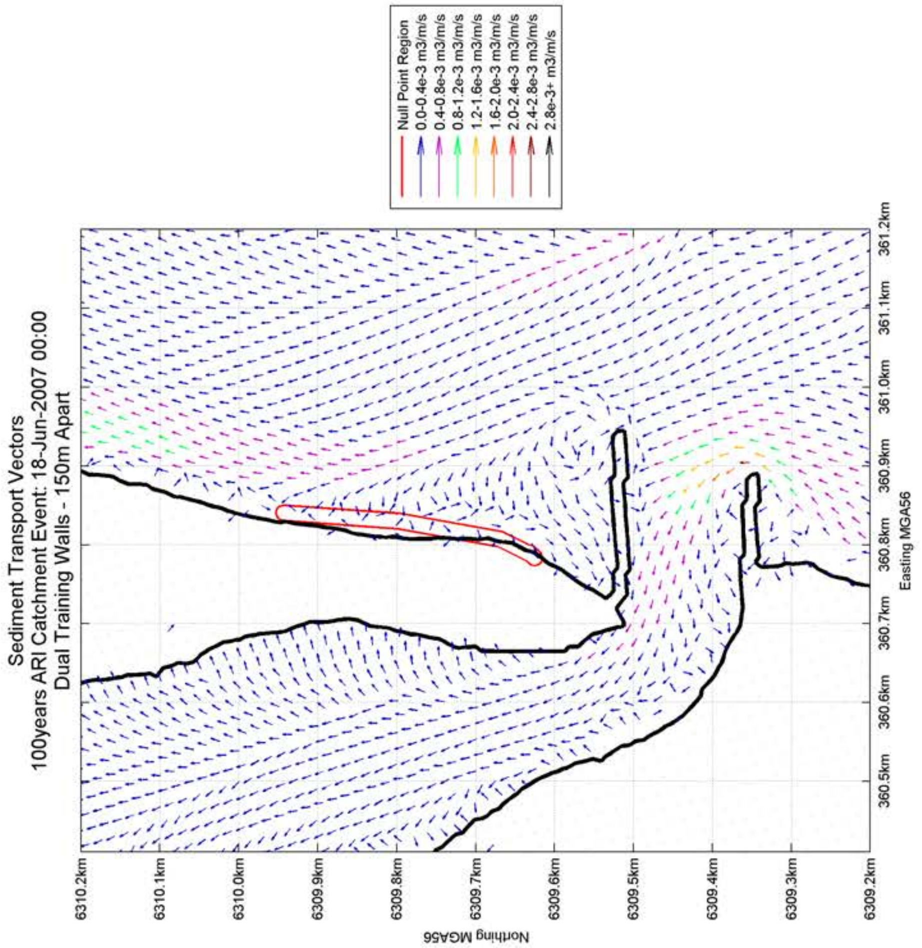


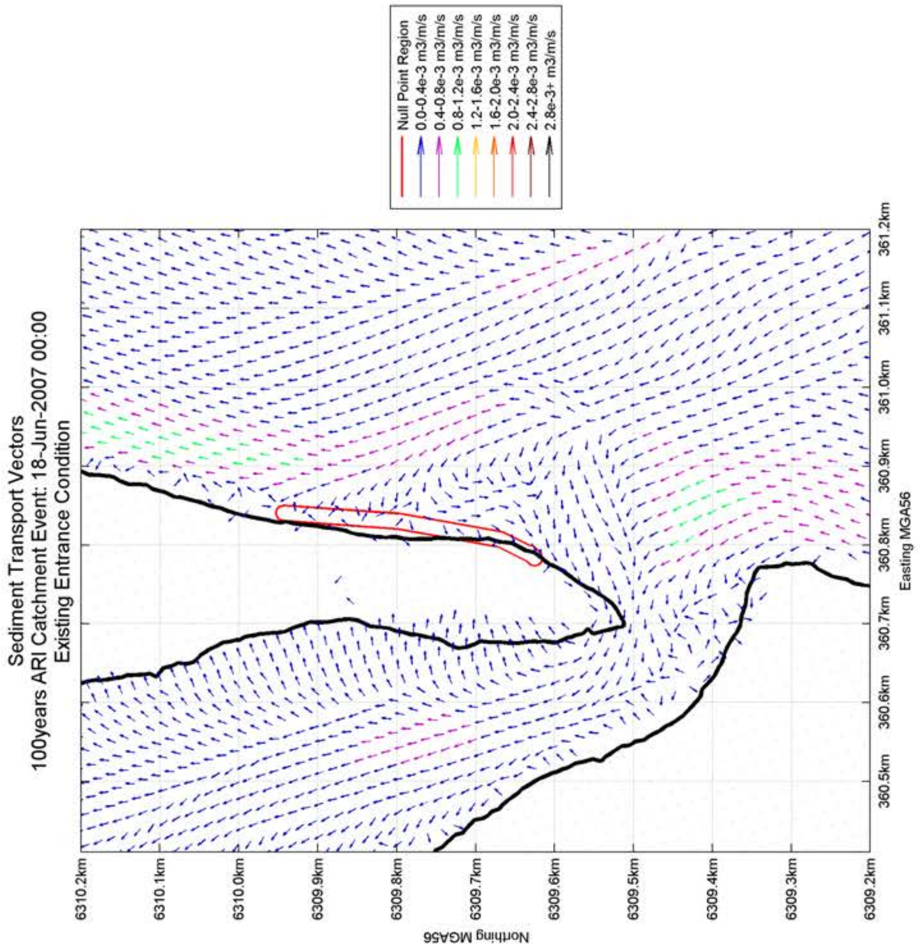
Cardno

Tuggerah Lakes − The Entrance Morphodynamic Modelling

Sediment Transport Vectors
and Approximate Null Point Location

Figure 5.8b





Cardno

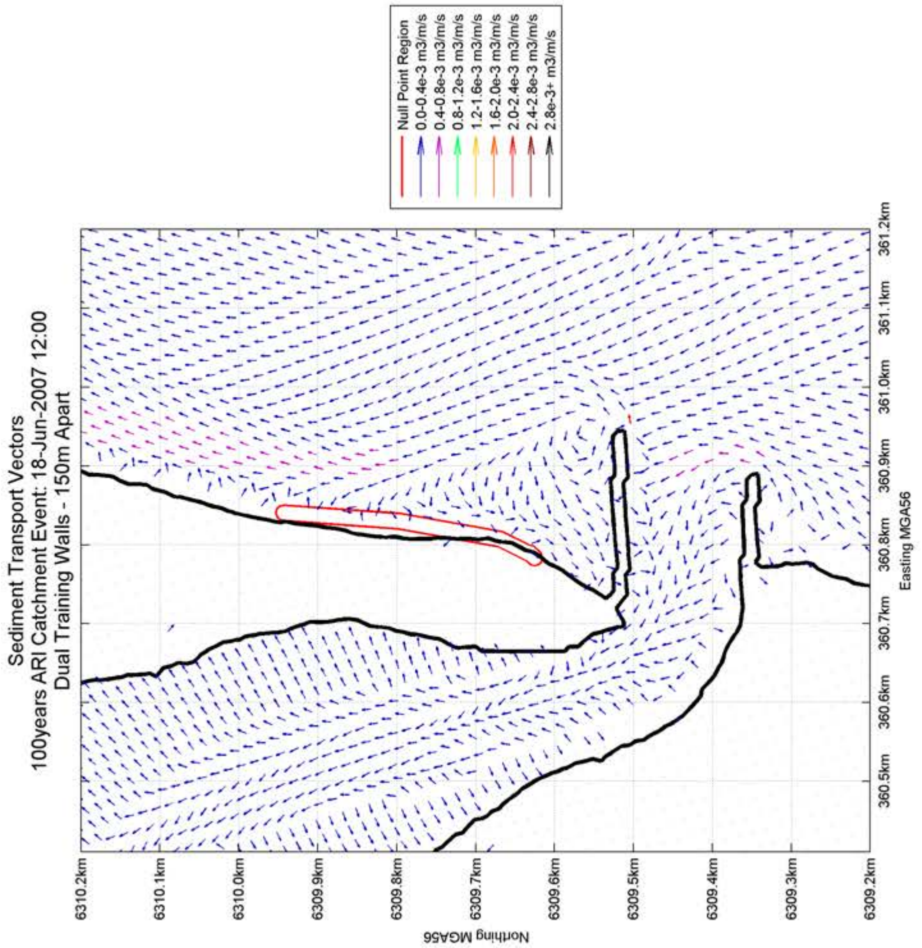
Tuggerah Lakes − The Entrance Morphodynamic Modelling

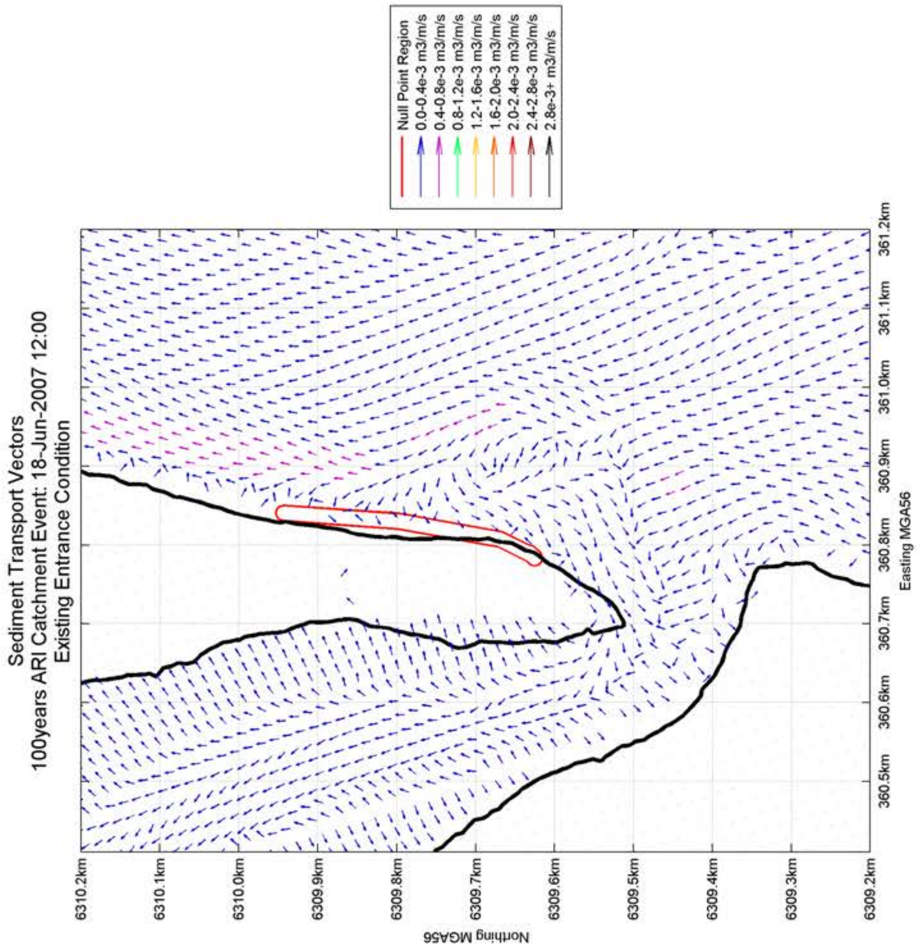
Sediment Transport Vectors
and Approximate Null Point Location

LJ2985/R2791 Oct 2013

J:\CM\LJ2985−TheEntrance\009−ReportingDocsandFigures\Figures\

Figure 5.8c





Cardno

Tuggerah Lakes − The Entrance Morphodynamic Modelling

Sediment Transport Vectors
and Approximate Null Point Location

LJ2985/R2791 Oct 2013
J:\CM\LJ2985−TheEntrance\009−ReportingDocsandFigures\Figures\

Figure 5.8d

Cardno

Tuggerah Lakes − The Entrance Morphodynamic Modelling

Dredge Regime, Spoil Placement and Sediment Transport Null Point Zone

Figure 5.9

LJ2985/R2791 Oct 2013

J:\CM\LJ2985−TheEntrance\009−ReportingDocsandFigures\Figures\

Cardno

Tuggerah Lakes − The Entrance Morphodynamic Modelling
Proposed Coastal Structures

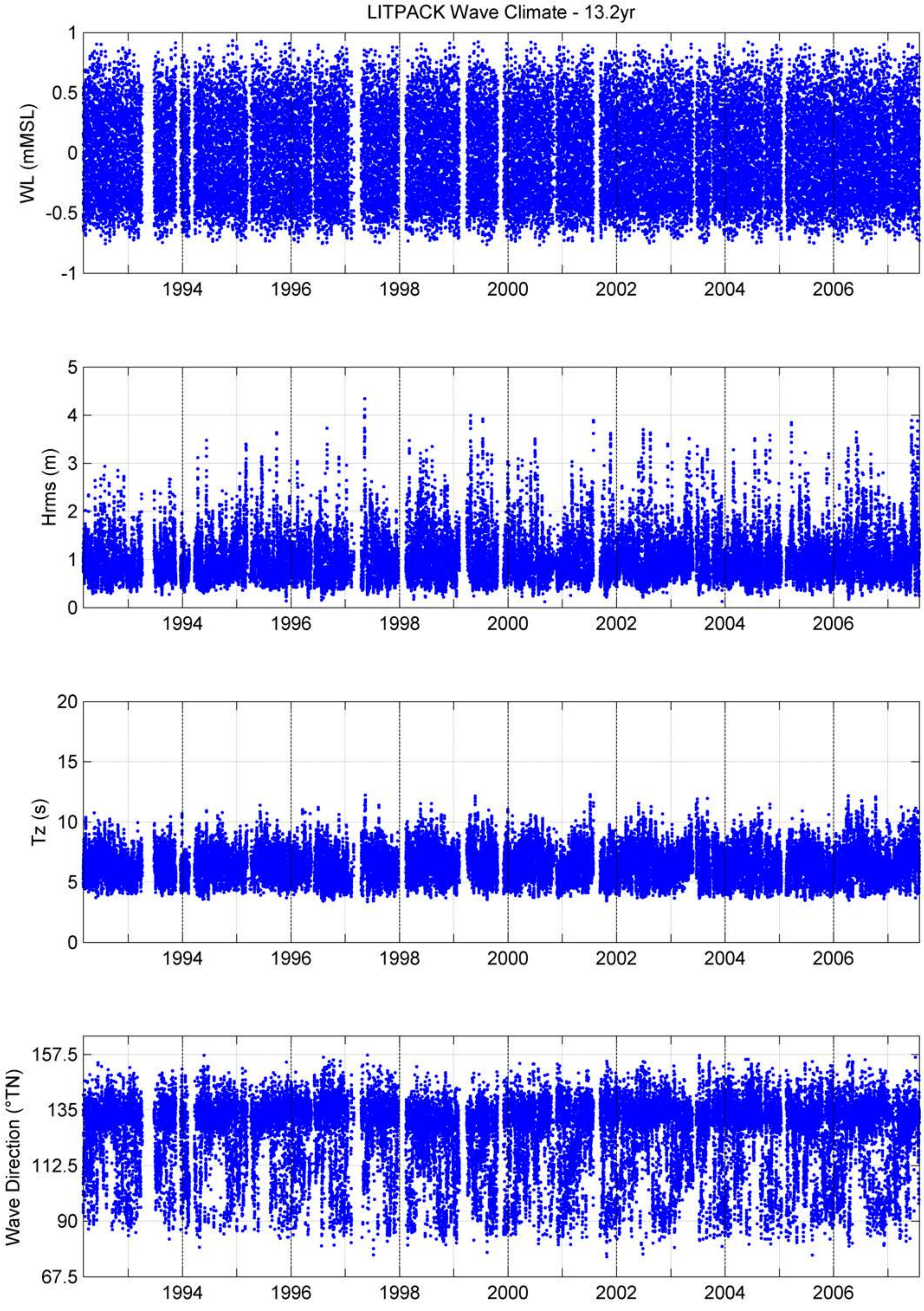
LJ2985/R2791 Oct 2013
J:\CM\LJ2985−TheEntrance\009−ReportingDocsandFigures\Figures\

Figure 6.1

Tuggerah Lakes − The Entrance Morphodynamic Modelling

Near Shore Shoal Structure
2001

Figure 6.2

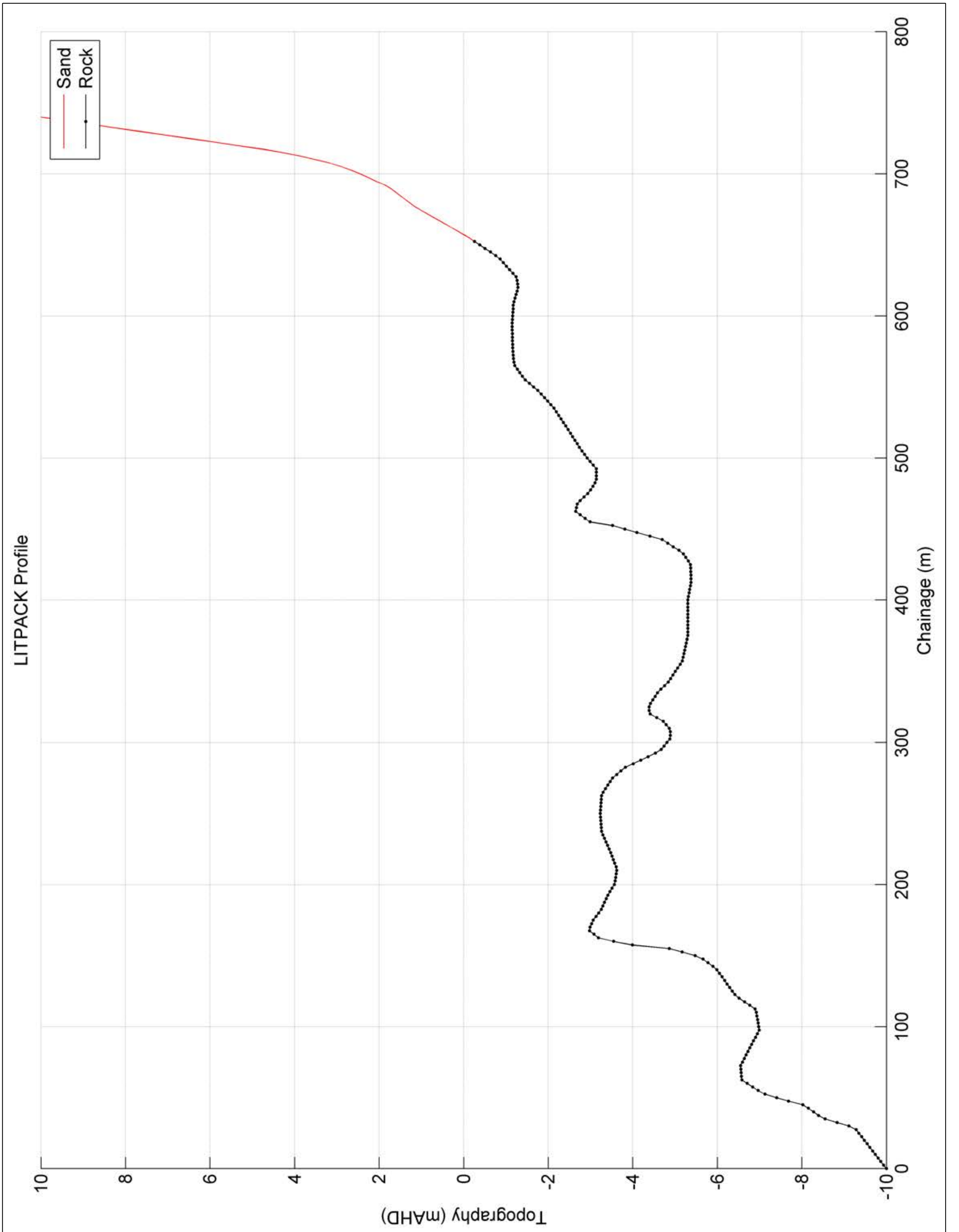


Cardno

Tuggerah Lakes − The Entrance Morphodynamic Modelling

LITPACK − Wave Parameter Time−Series
SWAN Locations 23 (Hs,T) & 53 (Dir)

Figure 6.3

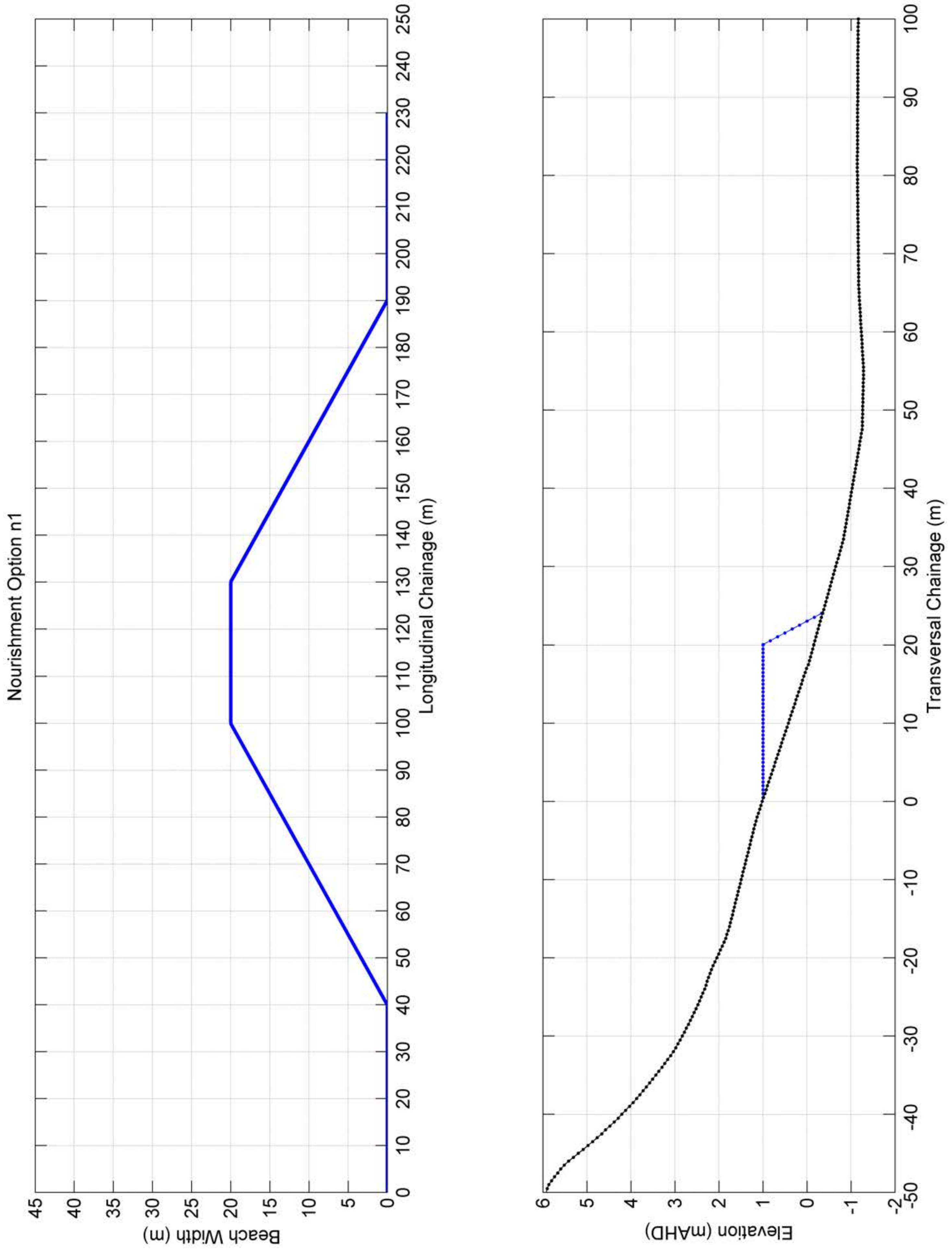


Tuggerah Lakes − The Entrance Morphodynamic Modelling

LITPACK − Bathymetric Profile
Sand & Rock

Figure 6.4

LJ2985/R2791 Oct 2013
J:\CM\LJ2985−TheEntrance\009−ReportingDocsandFigures\Figures\



Tuggerah Lakes − The Entrance Morphodynamic Modelling

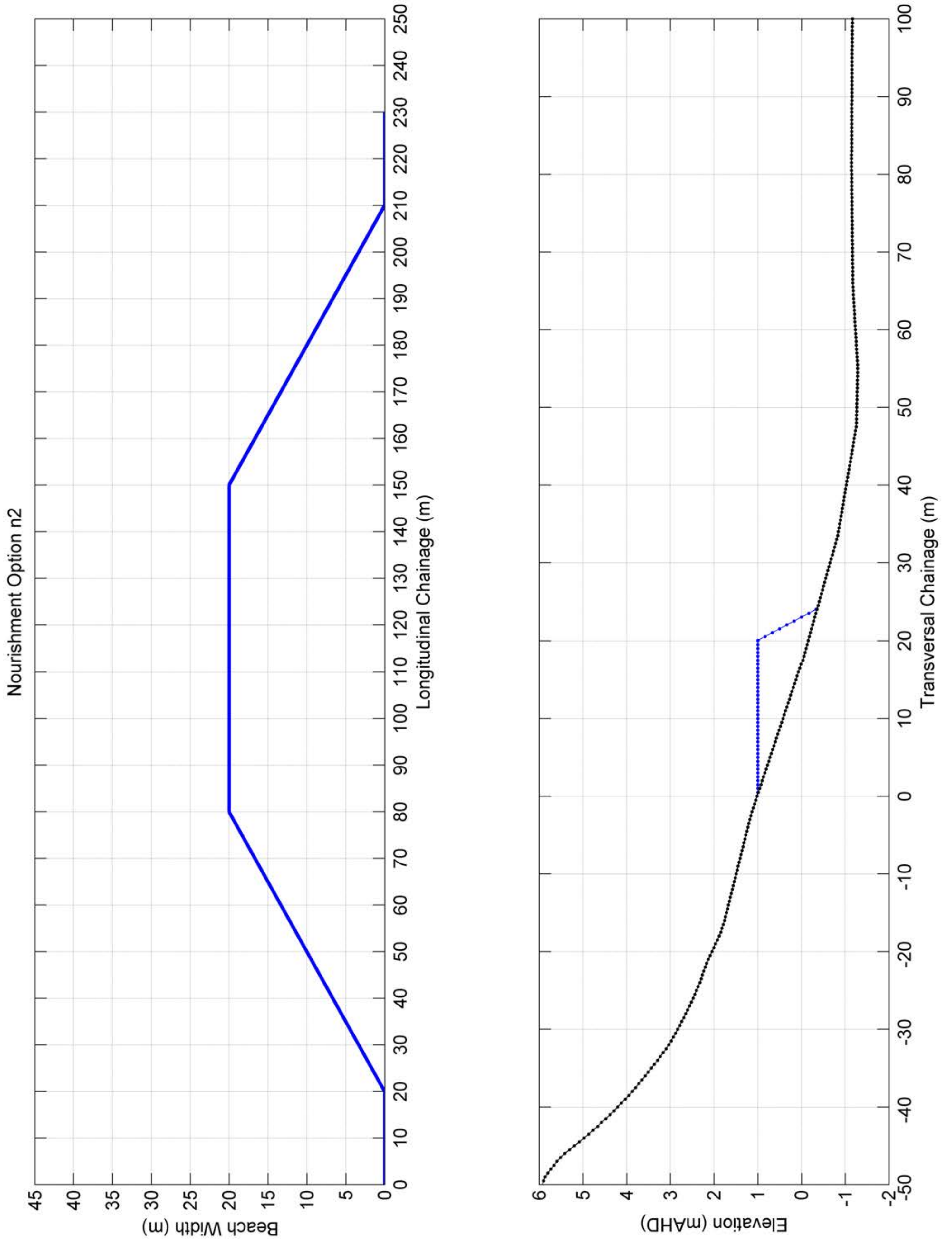
Nourishment Profile and Coastline
Option n1 (1000m3)

Figure 6.5a

Cardno

LJ2985/R2791 Oct 2013

J:\CM\LJ2985−TheEntrance\009−ReportingDocsandFigures\Figures\



Tuggerah Lakes − The Entrance Morphodynamic Modelling

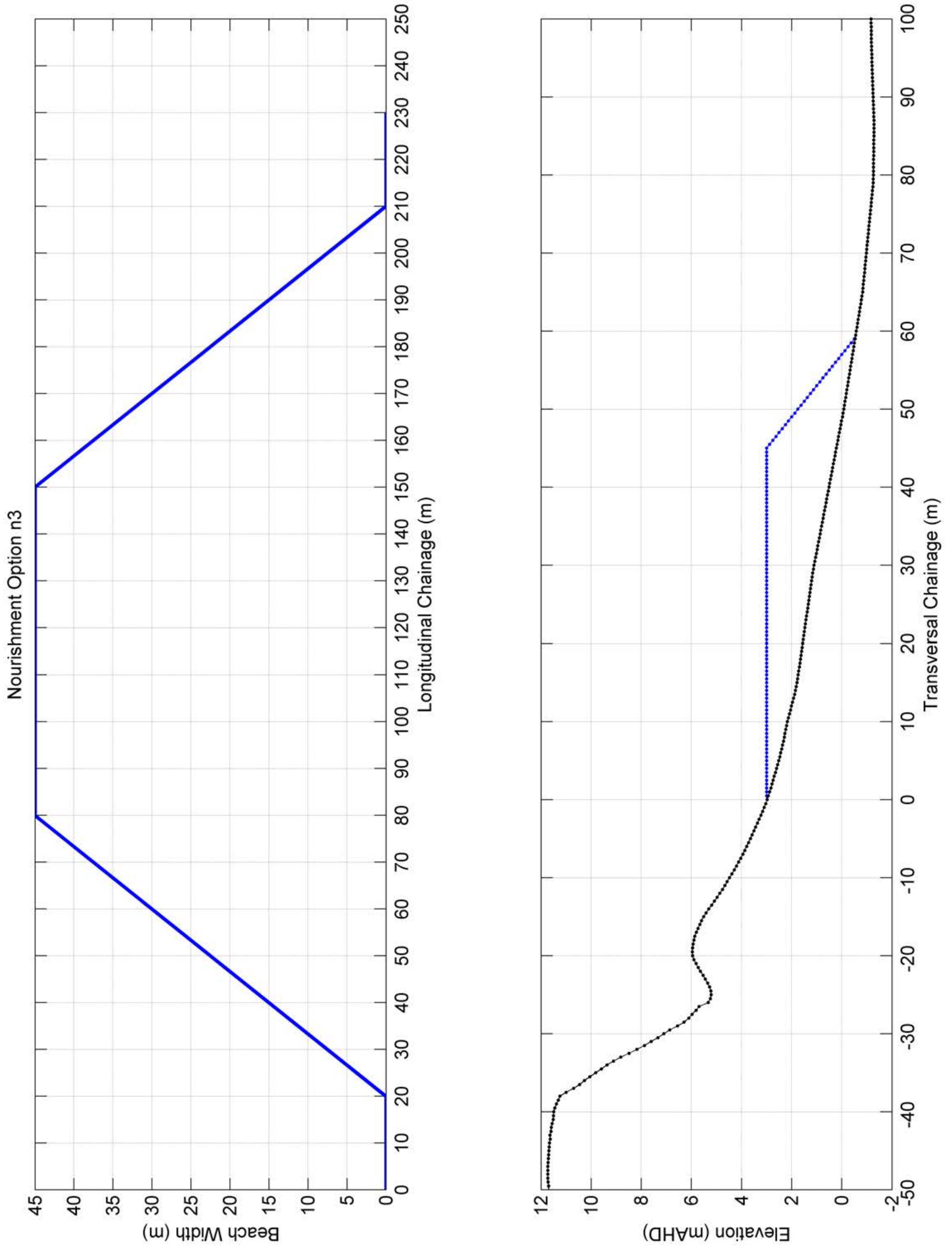
Nourishment Profile and Coastline
Option n2 (1500m3)

Figure 6.5b

Cardno

LJ2985/R2791 Oct 2013

J:\CM\LJ2985−TheEntrance\009−ReportingDocsandFigures\Figures\



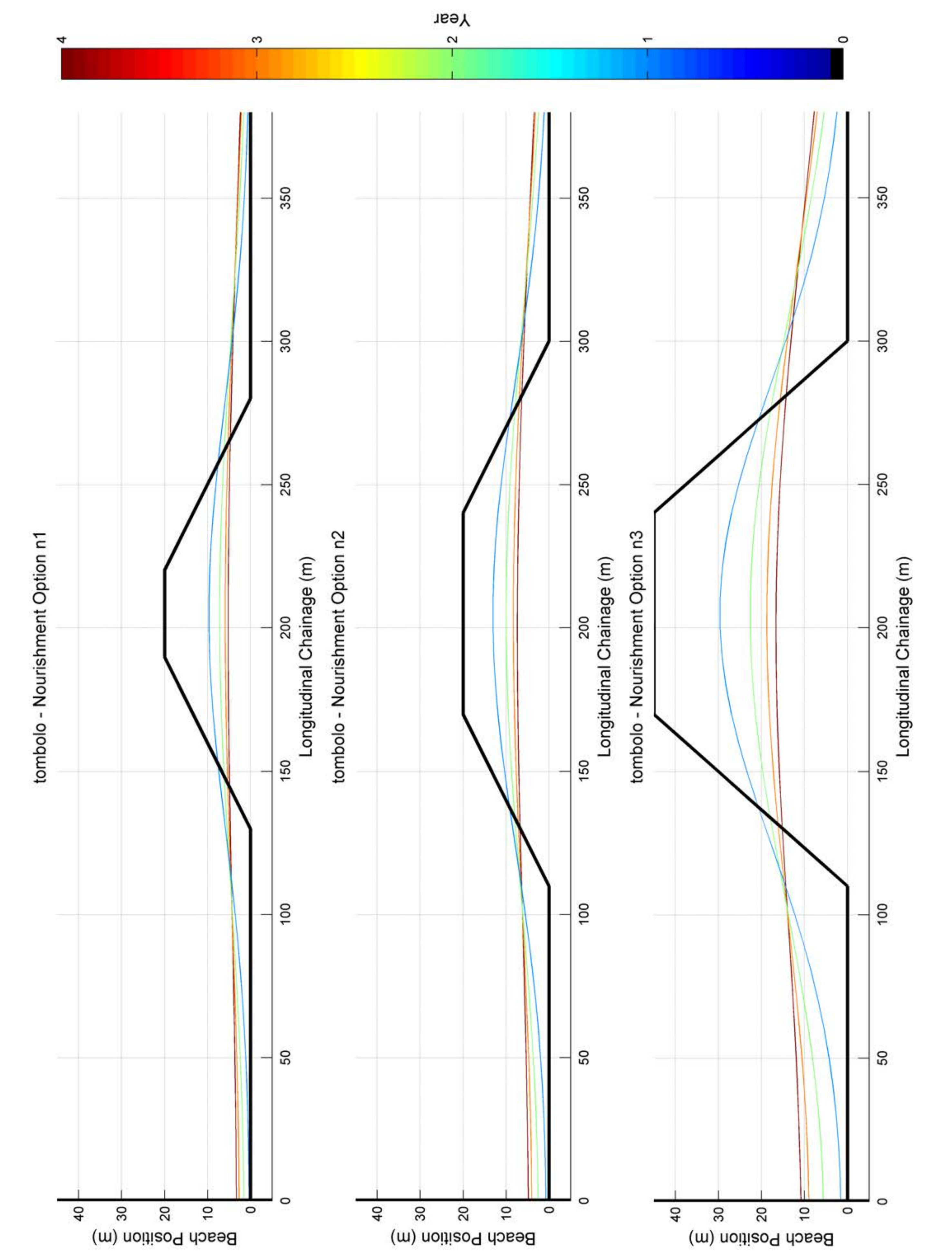
Cardno

Tuggerah Lakes − The Entrance Morphodynamic Modelling

Nourishment Profile and Coastline
Option n3 (10000m3)

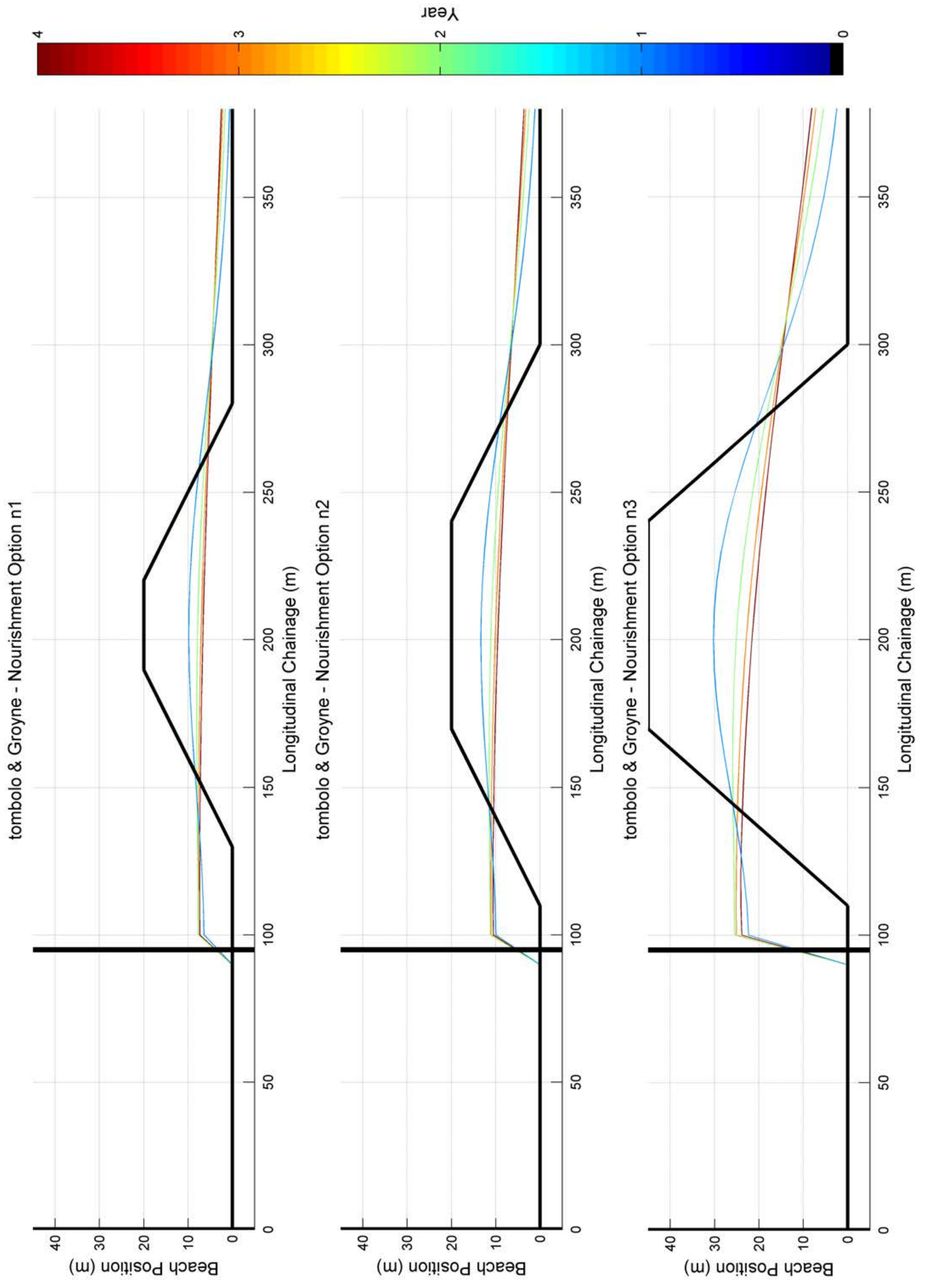
LJ2985/R2791 Oct 2013
J:\CM\LJ2985−TheEntrance\009−ReportingDocsandFigures\Figures\

Figure 6.5c



LITLINE − 4 years Coastline Evolution
Tombolo

Figure 6.6a

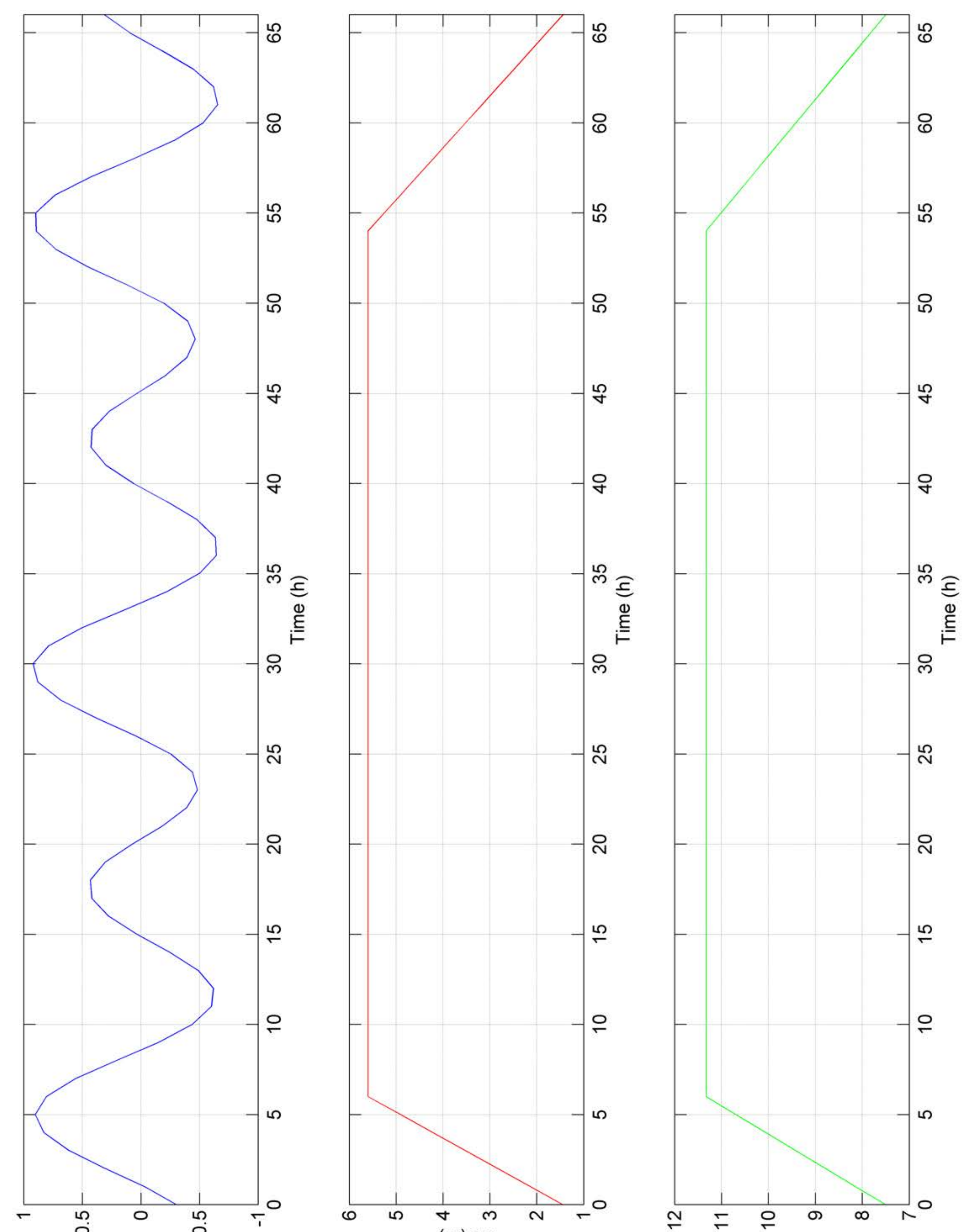


Cardno

Tuggerah Lakes − The Entrance Morphodynamic Modelling

LITLINE − 4 years Coastline Evolution
Tombolo & Groyne

Figure 6.6b



Cardno

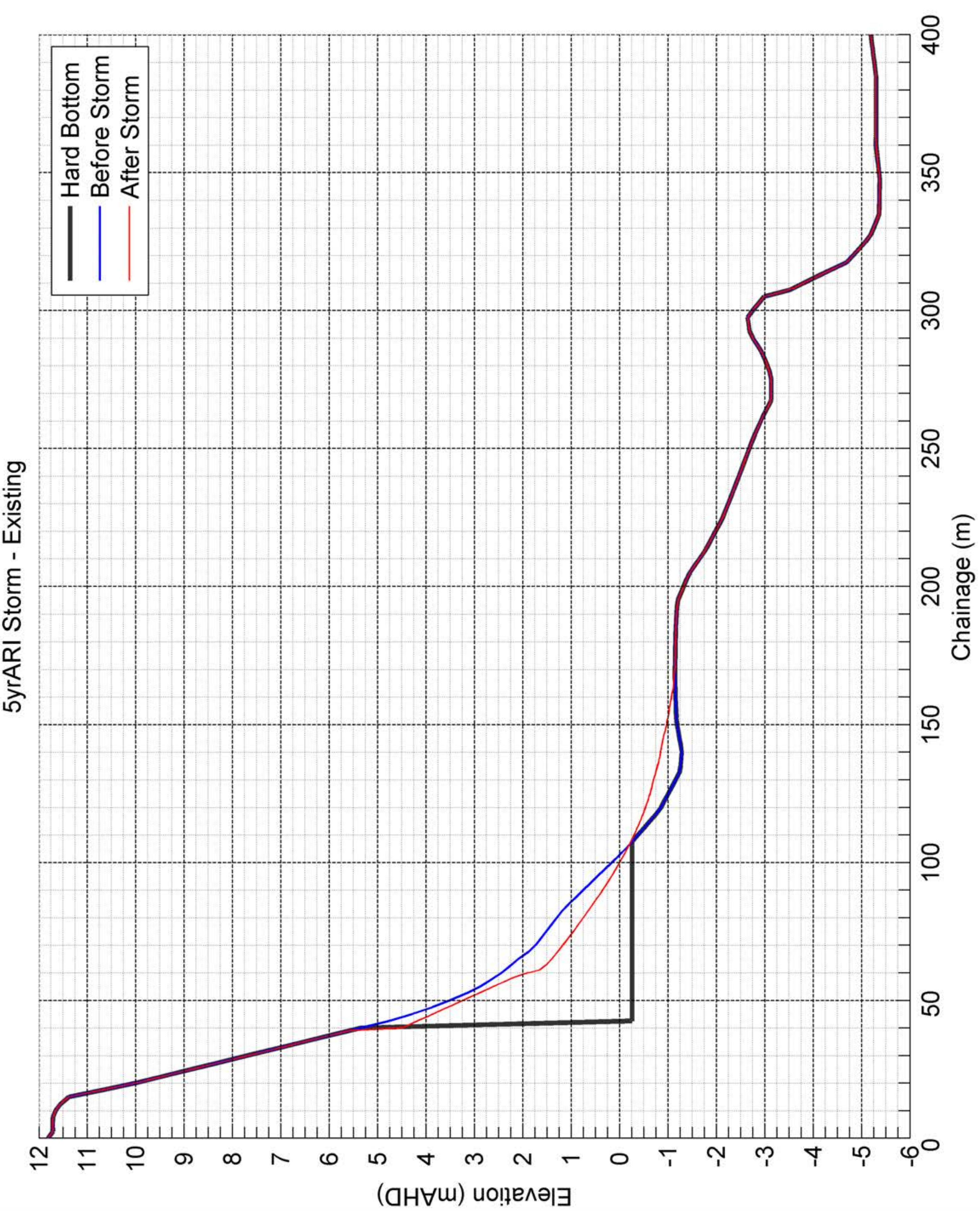
LJ2985/R2791 Oct 2013

J:\CM\LJ2985−TheEntrance\009−ReportingDocsandFigures\Figures\

Tuggerah Lakes − The Entrance Morphodynamic Modelling

SBEACH − Wave Climate Time Series
5 year ARI Artificial Storm

Figure 6.7



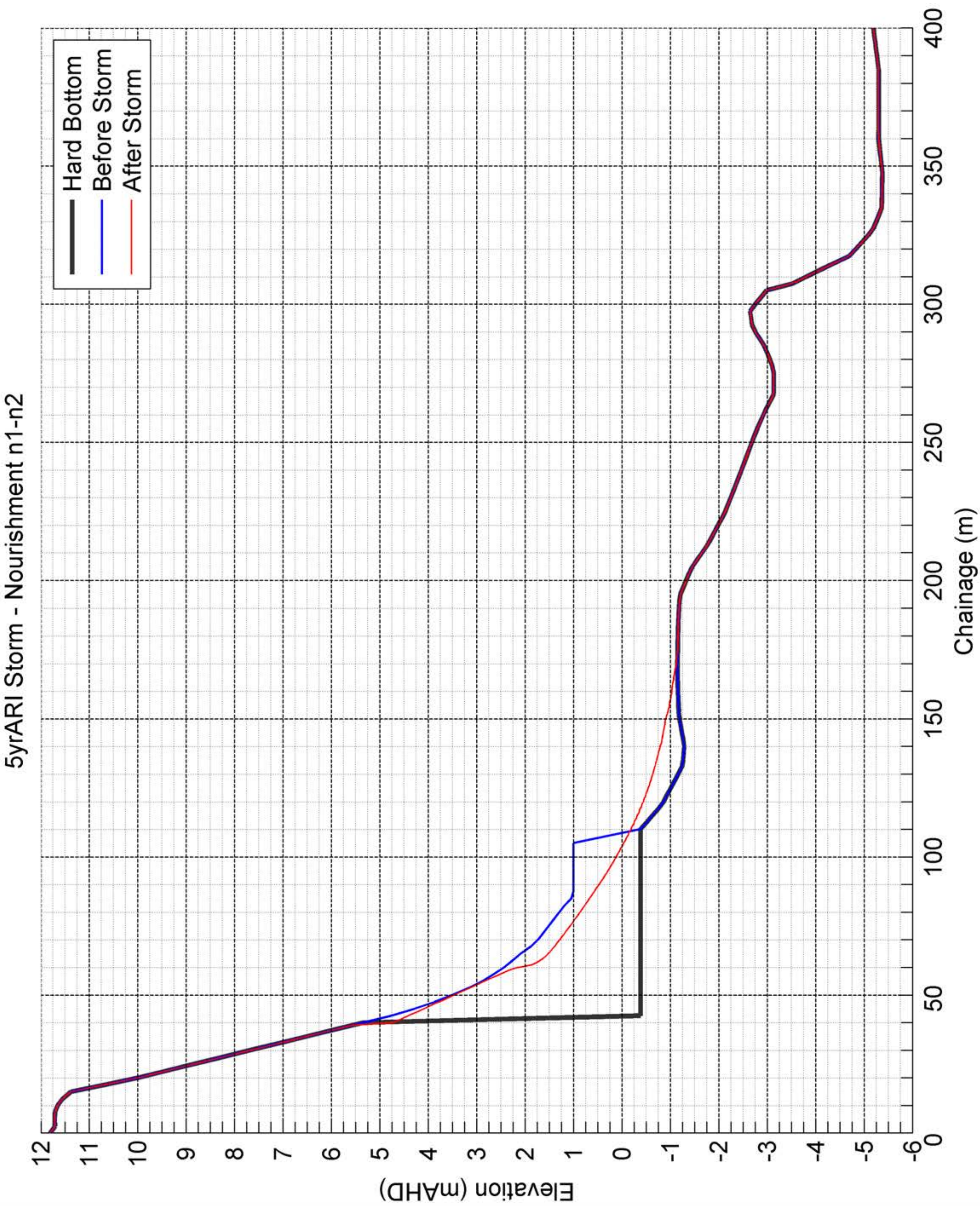
Tuggerah Lakes − The Entrance Morphodynamic Modelling

SBEACH − 5 years ARI Storm Bite
Existing

Figure 6.8a

LJ2985/R2791 Oct 2013

J:\CM\LJ2985−TheEntrance\009−ReportingDocsandFigures\Figures\



Cardno

LJ2985/R2791 Oct 2013

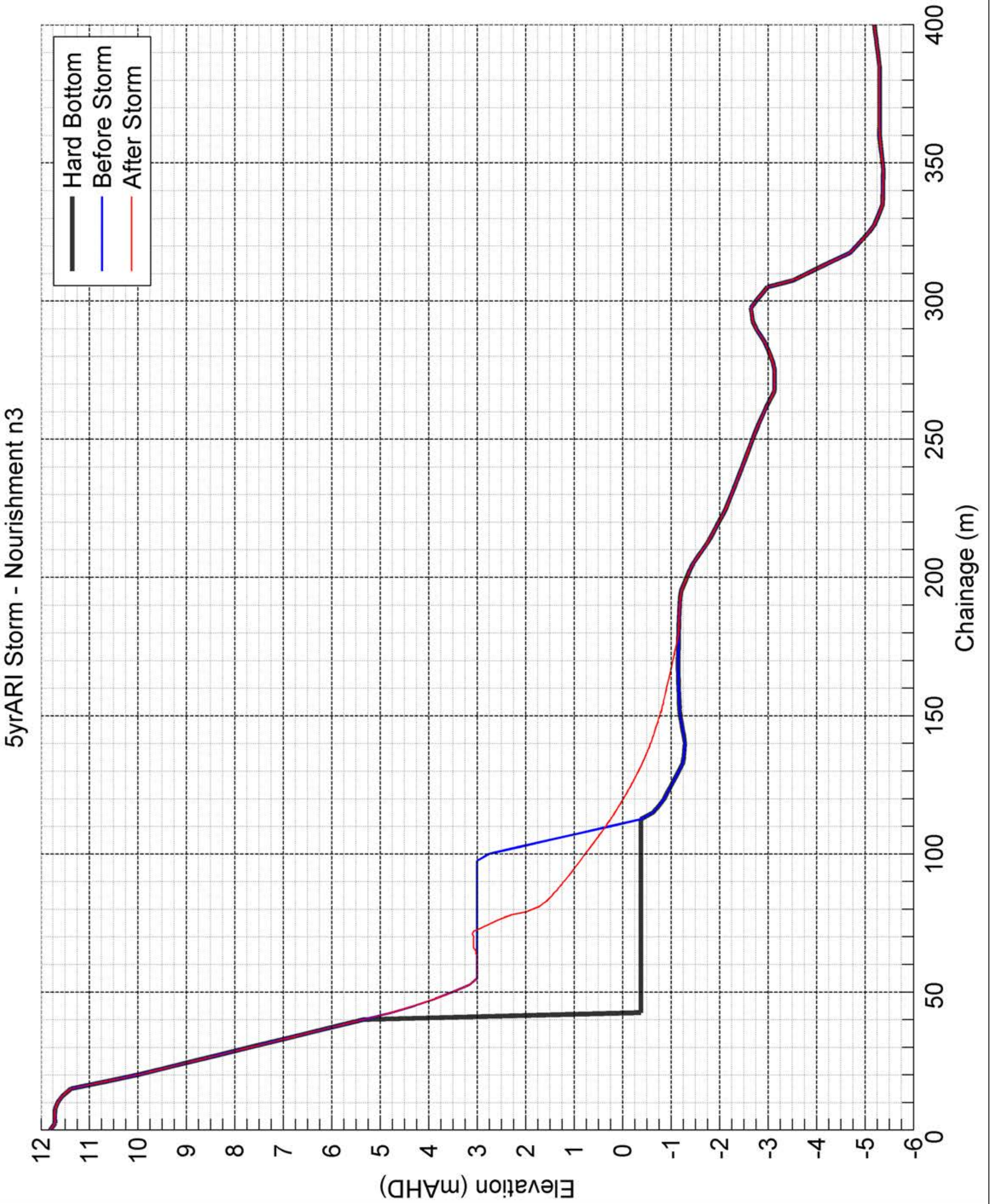
J:\CM\LJ2985−TheEntrance\009−ReportingDocsandFigures\Figures\

Tuggerah Lakes − The Entrance Morphodynamic Modelling

SBEACH − 5 years ARI Storm Bite

Options n1 & n2

Figure 6.8b



Cardno

LJ2985/R2791 Oct 2013

J:\CM\LJ2985−TheEntrance\009−ReportingDocsandFigures\Figures\

Tuggerah Lakes − The Entrance Morphodynamic Modelling

SBEACH − 5 years ARI Storm Bite
Options n3

Figure 6.8c

Cardno

Tuggerah Lakes – The Entrance Morphodynamic Modelling

Proposed Permanent Pipeline

LJ2985/R2791 Oct 2013

J:\CM\LJ2985-TheEntrance\009-ReportingDocsandFigures\Figures\

Figure 8.1