JOHNSON PROPERTY GROUP

TRINITY POINT MARINA
BREAKWATER DESIGN STUDY

FINAL

Issue No. 3
DECEMBER 2007

Patterson Britton
& Partners Pty Ltd
consulting engineers

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1 INTRODUCTION

Johnson Property Group *(JPG)* has engaged Patterson Britton & Partners (*PatBrit*) to design the marina for the Trinity Point development. As a preliminary step in the marina design PatBrit have undertaken a number of studies including:

- Trinity Point Maritime Structures, Coastal Processes Study (*PatBrit, 2007a*);
- Trinity Point Marina, Berth Demand Study (*PatBrit, 2007b*); and
- Trinity Point Marina, Breakwater Design Study (*PatBrit, 2007*).

This report comprises the Breakwater Design Study which documents the methodology followed in developing a conceptual breakwater design to protect the marina from excessive wave action. In developing the preferred conceptual design this report considers the following factors:

- metocean design parameters;
- design considerations of the marina; and
- breakwater design options/alternatives

The site location of the marina is shown in **Figure 1**. The conceptual layout of the marina and breakwaters is shown in **Figure 2**.

The structure of this report is as follows:

Section 1: Introduction.
Section 2: Design Parameters – outlining the design parameters for the breakwaters including wave climate, water levels, wind loads and current loads.
Section 3: Breakwater Design Options – outlining the breakwater design options including floating structures, rubble mound structures and fixed vertical structures, and providing a comparative evaluation of these options and a preferred option for the Trinity Point Marina.
Section 4: Design Considerations – providing a discussion of the specific breakwater design issues at the Trinity Point site including wave attenuation, wave reflection, flushing/circulation and shoreline erosion/scour.
Section 5: Concluding Remarks.

2 DESIGN CONDITIONS

Designing a breakwater structure for a marina necessitates consideration of both:

- structural stability/integrity (*extreme conditions*); and
- functional performance (*operational conditions*).

This involves the definition of differing average return interval *(ARI)* periods for design events that relate to each of these situations. For structural considerations of the breakwater an ARI design event of 1 in 200 years is considered appropriate for the extreme condition. Within all marinas *(including those protected by breakwaters)* the ARI design events of 1 in 50 years, and 1 in 1 year, are appropriate for consideration in terms of operational conditions (*in accordance with AS 3692-2001*).

It is noted that the design of the two arms of the breakwater may differ given the different design conditions. For the purposes of discussion the two arms of the breakwater structure will be referred to in this report as the eastern breakwater and the southern breakwater (*as shown in **Figure 2**).*

2.1 WAVE CLIMATE

The incident wave climate at the Trinity Point site is a direct consequence of the local wind climate; no ocean wave penetration occurs through the Swansea Channel into the Lake. In the absence of recorded wave data at the site, or any nearby suitable location, design wave parameters have been estimated from regional wind data using hindcasting methods based on depth/fetch limited wave algorithms.

As discussed in Coastal Processes Study (*PatBrit, 2007a*), Sydney Airport wind data has been adopted for use in wave hindcasting, in relation to operational conditions, for the Trinity Point marina site. Sydney and Newcastle have been found to experience very similar wind conditions and the Sydney data set is longer and more reliable. For structural design considerations, design site wind speeds from AS/NZS 1170.2-2002 have been used to predict waves for extreme events.

The practice of using depth and fetch limited wave algorithms to hindcast or predict wave heights from wind data *(choosing a representative depth and fetch in each directional sector)* is considered to be conservative and appropriate for the conceptual stage of the marina and breakwater design. For subsequent detailed design stages the use of more complex numerical modelling techniques to predict wave heights should be considered. Numerical wave models can represent complex physical processes and wave interactions such as:

- complex real bathymetry;
- refraction;
- shoaling;
- non-linear wave-wave interaction;

- full directional wave spectral description;
- bed friction;
- white capping;
- currents; and
- wave breaking.

More accurate representation of the wave climate using numerical modelling techniques, including spatial variations at the site, can assist in optimising the design during the detailed design process. However, for conceptual design purposes the more conservative outcomes presented below are considered appropriate.

2.1.1 Structural Design Requirements

As discussed above, consideration of extreme conditions is required for the structural design of the breakwater. An event of 200 years ARI was adopted as the design event representing the extreme conditions. This design event ARI was chosen following consideration of the:

- design life *(specified as 25 years)*;
- consequences of structural failure; and hence
- acceptable risk of failure for the breakwater structure.

AS/NZS 1170.2 *(2002)* describes design site wind speeds for extreme wind events. The design site wind speeds for the region including the Trinity Point site have been used to predict extreme wave events using algorithms based on those developed in Coastal Engineering Manual *(2003)* for deep-water, fetch-limited waves.

Table 2.1 summarises design wave events from all the directional sectors that the Trinity Point site is exposed to. The configuration of breakwater structures in the design may influence which directional event is critical as the incident wave direction influences the forces applied to the structures.

Table 2-1: Predicted 200 year ARI wave conditions

Direction (degrees)	Fetch length (m)	Average Depth (m)	200yr ARI	
			H_s (m)	T_p (s)
0	948	3.5	0.50	2.5
45	732	4.5	0.40	2.3
80	4260	6.0	1.00	3.6
90	3920	7.0	0.90	3.6
135	2024	6.0	0.90	3.2
158	6330	5.0	1.45	4.1
180	3000	5.0	1.00	3.5

In the analysis of maritime structures, different wave parameters from the design wave climate should be considered for different applications, or expected failure modes. For progressive failure modes (*failure due to the impact of many waves*) or failure due to fatigue or cyclical loading H_s[1]

[1] H_s is the average of the highest 1/3 of waves, termed “significant wave height”

should be the design parameter adopted. In the case of sudden failure due to single wave loading a larger wave parameter should be adopted *(e.g.* H_1[2] *or* H_{max}[3]*).*

2.1.2 Operational Requirements

The wave climate for the Trinity Point site was established in the Coastal Processes Study (*PatBrit, 2007a*) and is summarised below in **Table 2.2**.

Table 2-2: Significant wave height and peak wave period hindcast for the Trinity Point Marina site

Direction (degrees)	Fetch length (m)	Average Depth (m)	1 yr ARI		50yr ARI	
			Hs (m)	Tp (s)	Hs (m)	Tp (s)
0	948	3.5	0.3	1.6	0.51	1.9
45	732	4.5	0.35	1.6	0.51	1.8
80	4260	6	0.66	2.7	1.05	3.1
90	3920	7	0.61	2.5	1.00	3.0
135	2024	6	0.60	2.3	0.96	2.7
158	6330	5	0.86	3.1	1.27	3.6
180	3000	5	0.75	2.6	1.1	3.0

Given the proposed fan layout of the marina (*refer* ***Figure 2***) with the main walkways extending radially from the shore, it is difficult to assign a single direction for 'head' and 'beam' seas approaching the vessels in the marina, as each row is oriented differently. For the purposes of conceptual design the critical 'head' seas that have been adopted are those approaching approximately normal to the southern breakwater i.e. at a bearing of 158 degrees. These waves will be 'head' seas to the first row of vessels and will approach the vessels in the western rows at a slightly oblique angle. For the purposes of conceptual design the critical 'beam' seas that have been adopted are those approaching approximately normal to the eastern most section of the eastern breakwater i.e. at a bearing of 80 degrees. These waves will be 'beam' seas to the first row of vessels and will approach the vessels in the western rows at a slightly oblique angle. This is considered a reasonable approach for the conceptual design stage though further detailed investigation of wave heights in the marina would need to be undertaken as part of the detailed design.

The critical design wave conditions for the southern and eastern breakwaters are summarised below in **Table 2.3.**

Table 2-3: Design wave conditions for the southern and eastern breakwaters

Breakwater	Direction (degrees)	Fetch length (m)	1 yr ARI		50yr ARI	
			Hs (m)	Tp (s)	Hs (m)	Tp (s)
Southern	158	6330	0.86	3.1	1.27	3.6
Eastern	80	4260	0.66	2.7	1.05	3.1

[2] H_1 is the average of the highest 1% of waves
[3] H_{max} is the expected highest wave in a period of time

Table 2.4 (*taken from AS 3962-2001*) provides a recommended 'good' wave climate criteria for small vessel marinas.

Table 2-4: Criteria for 'good' wave climate in small craft harbours, AS 3962(2001)

Direction and peak period of design harbour wave	Significant wave height (Hs)	
	Wave event exceeded once in 50 years	Wave event exceeded once a year
Head seas less than 2s	Conditions not likely to occur during this event	Less than 0.3m wave height
Head seas greater than 2s	Less than 0.6m wave height	Less than 0.3m wave height
Oblique seas grater than 2s	Less than 0.4m	Less than 0.3m wave height
Beam seas less than 2s	Conditions not likely to occur during this event	Less than 0.3m wave height
Beam seas greater than 2s	Less than 0.25m wave height	Less than 0.15m wave height

The predicted un-attenuated wave climate for Trinity Point and the criteria for a 'good' wave climate (*in accordance with AS 3962-2001*), are summarised in **Table 2.5**.

Table 2-5: Predicted (*un-attenuated*) wave climate and design wave climate criteria

Direction	1 year ARI			50 year ARI		
	Predicted Hs (m)	Predicted Tp (s)	Design Criteria Hs (m)	Predicted Hs (m)	Predicted Tp (s)	Design Criteria Hs (m)
Head Sea > 2s	0.86	3.1	0.3	1.27	3.6	0.6
Beam seas >2s	0.66	2.7	0.15	1.05	3.1	0.25

It is therefore evident that wave attenuation will be required to achieve the design wave climate criteria inside the marina. Wave attenuation is discussed further in **Section 4.1.**

2.2 WATER LEVEL

The following design still water levels were established in the Coastal Processes Report (*PatBrit, 2007a*) for adoption in the design of the maritime structures associated with the Trinity Point development.

Table 2-6: Lake Macquarie Design Still Water Levels (m AHD)

Extreme	1% AEP	2% AEP	5% AEP	100% AEP
2.63	1.38	1.24	0.97	0.40

2.3 WINDS LOADS

Wind loading of structures related to the breakwater structure should be determined in accordance with AS 3962 *(2001)* taking design site wind speeds from AS/NZS 1170.2 *(2002)*. Wind loads are not critical for the design of the breakwater structure and are therefore not considered in this conceptual design. Wind loads are however critical for the marina design and will be further analysed during detailed design of the marina.

2.4 CURRENT LOADS

Current velocities in the vicinity of the proposed Trinity Point marina, as reported in the Coastal Processes Report (*PatBrit, 2007a*), are dominated by wind driven processes and as a result would generally be less than 0.5m/s even in extreme cases. However, AS 3962 *(2001)* recommends that a minimum design velocity of 1.0m/s should be adopted even in cases where flows can be demonstrated to be substantially less. Current loads on structures will be determined in accordance with AS 3962 *(2001)* and will be further analysed during detailed design stages.

3 BREAKWATER DESIGN OPTIONS

Breakwater protection could be provided for the Trinity Point marina using a number of different breakwater design options/alternatives. This section provides a discussion of the three main design options, highlighting advantages and limitations of each option before recommending a preferred design option. The design options considered include:

1. Floating breakwater
2. Rubble mound breakwater
3. Fixed vertical structure
   a. Full depth
   b. Partial depth *(skirt)*

Section 3.4 presents a comparative evaluation of the design alternatives discussed for the Trinity Point Marina site.

3.1 FLOATING BREAKWATER

A floating breakwater may be described as a moored structure that interacts with incident wave energy in the upper portion of the water column resulting in a reduction of wave heights on its leeward side. These structures generally comprise a series of floating pontoons fixed together to form a single floating structure which is either anchored to the seabed with chains or supported by a series of piles. These structures are favoured over conventional fixed timber jetty structures and rubble mound structures in areas having a low to moderate wave climate due to their:

- relatively minimal environmental impact;
- relative independence of water depth and seabed geology;
- provision of an access platform at a constant level relative to the water level particularly in extreme tidal range situations;
- flexibility of layout and facility to provide moorings;
- potential for pre-fabrication in a factory environment; and
- relatively economical form of construction.

However, floating breakwaters have a limited performance range particularly with respect to wave length (*or wave period*). For wave periods over approximately 3.0 seconds, floating breakwater performance decreases as the floating structure starts to 'ride' over the top of the wave rather than interact with it. The design wave climate for the Trinity Point Marina includes wave periods over to 3 seconds (*from the south-eastern and north-eastern sectors*) and floating breakwater structures would therefore be operating at the upper limit of their capacity.

Floating breakwaters require a high level of maintenance. Limited operational life results for structures of this type without the provision of regular maintenance. The failure mode for a structure of this type is sudden and can have potentially severe consequences.

3.2 RUBBLE MOUND BREAKWATER

The rubble mound structure is a mound of stones and/or concrete armour units of various sizes and shapes, either dumped at random or placed in courses. Side slopes and armour unit sizes are designed so that the structure will resist the expected wave climate. Rubble mound structures are adaptable to any water depth and to most foundation conditions. The main advantages are as follows:

- structure settling readjusts component units which increases stability;
- failure occurs progressively not catastrophically;
- damage is repairable; and
- rubble absorbs rather than reflects wave energy.

The main disadvantages are as follows:

- large quantities of rock are required, particularly in deep water;
- potential environmental impact as large seabed footprint is covered; and
- potential cost if local source of rock is unavailable.

An alternative to a rubble mound structure would be a similarly shaped structure constructed from geotextile containers *(tubes or bags)* filled with sand. This technique has been successfully used on a number of occasions and eliminates the disadvantages of large quantities of rock being required and the potential cost if a local source of rock is unavailable. It does not however, eliminate the potential environmental impact of covering a large area of the seabed. The source of sand for filling of the geotextile containers can produce issues of it's own with either; high costs apparent for the supply and transport of imported material, or, potential environmental concerns associated with the dredging of a large volume of local material *(if of suitable physical characteristics)*.

For the Trinity Point marina site, the potential environmental impact due to the large area of seabed covered is restrictive for a breakwater of this type given the presence of extensive seagrass meadows in this area.

3.3 FIXED VERTICAL STRUCTURE

There are a wide range of fixed vertical walled structures which can be effectively utilised to provide protection from wave attack in relatively sheltered waterways. These structures generally fall into one of two categories:

a. Full depth solid wall structures - these structures extend to the seabed and essentially reflect 100% of the wave energy with no wave transmitted beyond the structure;
b. Partial depth *(or skirt)* structures – these structures have vertical panels penetrating only a portion of the water depth.

Full depth vertical structures have a similar functional performance to that of a rubble mound breakwater in terms of providing a sheltered wave climate in the lee of the structure. However, differences occur in the way in which the structure achieves this result. For vertical structures the wave energy is fully reflected rather than being partially dissipated or absorbed which can create issues in surrounding environments. Structural loads can be large on vertical structures due to total reflection, and impulsive forces *(instantaneous single wave)* need to be considered. Performance characteristics such as wave run up and overtopping can be also be exacerbated at vertical structures.

The advantages of a full depth vertical structure are:

- they require minimal waterway;
- they have a minimal seabed footprint; and
- a range of alternative construction materials can be considered *(timber, concrete etc)*.

The use of a partial depth structure *(skirt)* can eliminate some of the disadvantages of the use of a vertical structure. The advantages of a skirt breakwater over a rubble mound or full depth structure are:

- they also require minimal waterway;
- they also have a minimal seabed footprint *(less than a full depth structure)*;
- penetration depth can be selected to give required level of protection;
- disturbance of the seabed is minimal;
- by not extending to the seabed, structural loadings are reduced with significant cost savings;
- by not extending to the seabed, water circulation and flow regimes can be maintained; and
- a range of alternative construction materials can be considered also *(timber, concrete etc)*.

The performance of skirt breakwaters in terms of wave transmission is inferior to that of a rubble mound of full depth vertical structure as wave energy can pass beneath the structure. However, if some wave transmission is acceptable *(as is the case for a marina)* and the resultant wave climate satisfies design criteria, the advantages of improving other environmental considerations *(seabed and waterway footprint impacts, retention of some water circulation and flow regime, and wave reflections)* justifies the compromise in wave climate.

Further reductions in wave reflections and improvements in flow or circulation may be possible, through the use of slatted and/or double skirt breakwater configurations.

3.4 COMPARISON OF OPTIONS

Table 3.1 presents a comparative evaluation of the breakwater types described in the preceding section to identify the most appropriate option for the Trinity Point Marina site.

Table 3-1: Design Option Comparison

Design Option	*Advantage*	*Disadvantage*
Floating Breakwater	• minimal environmental impact • independent of seabed bathymetry and geology • access platform at water level • integration with berthing arrangement • economical construction and prefabrication	• high maintenance • limited life *(without maintenance)* • **limited functional performance in relation to design wave climate *(cannot meet required wave transmission criteria)***
Rubble Mound Breakwater	• good functional performance • low reflectivity of wave energy • function includes damage allowance • high stability • good functional performance	• **high environmental impact (*large areas of seagrass covered*)** • Potential high cost • large quantities of rock required
(Sand Filled Geotextile Containers Breakwater)	• inexpensive if local sand source available • high stability • relatively easy construction • good functional performance	• **high environmental impact (*large areas of seagrass covered*)** • **high reflectivity of wave energy** dredging/sand source potential issues • susceptible to vandalism *(compromises stability and performance of structure)*
Fixed Vertical Structure *(Full Depth)*	• minimal seabed and water way footprint • alternative construction materials possible • good functional performance	• **water circulation restriction** • **flow regime restriction** • high structural loads • reflectivity of wave energy *(double skirt design can reduce reflection)*
Fixed Vertical Structure *(Partial Depth - Skirt)*	• minimal seabed and water way footprint • *(relatively)* reduced structural loads • reduced cost • water circulation maintained • flow regime maintained • seabed disturbance minimal • relativity less reflectivity *(than full depth structure)* • wave transmission within required design criteria • alternative construction materials possible	• moderate structural loads

3.5 PREFERRED DESIGN OPTION

3.5.1 Southern Breakwater

In assessing comparative advantages and disadvantages of the breakwater design options, at this particular site, it is apparent that a rubble mound or geofabric bag structure is unlikely to be acceptable in this location due the environmental impact of covering existing seagrass beds. This option is therefore not considered further. It is also apparent that the floating breakwater is not likely to be able to achieve the performance specification in terms of wave attenuation due to the long period of the design wave. The floating breakwater structure is therefore also eliminated as an option. It is therefore evident that the fixed vertical structure is the most appropriate option for the southern breakwater. The partial depth or skirt breakwater has a number of advantages over the full length structure and is therefore the preferred option. While providing the required functional performance this option can also meet auxiliary design criteria *(environmental impact, economic consideration)*. The impacts of features considered inappropriate for this option can be reduced through design modifications. This breakwater design option is discussed further is in **Sections 4 and 5.**

3.5.2 Eastern Breakwater

The rubble mound structure is not considered appropriate for the eastern breakwater for the same reasons as given above. The floating breakwater structure would be operating on the boundary of its wave attenuation capacity in this location and is therefore not considered to be the best option at this stage. As noted for the southern breakwater, the fixed vertical structure *(most likely of partial depth)* is therefore the most appropriate option for the eastern breakwater.

As noted in **Section 3.3**, a skirt breakwater can be constructed in a number of different ways utilising a variety of construction materials. For the Trinity Point Marina it is recommended that the structures be constructed from timber for economic reasons, ease of construction and to provide a suitable design life.

The preferred design would essentially comprise a timber walkway/deck supported on piles with a solid skirt breakwater fixed to the inner row of piles. Further details of the design are discussed in **Section 4**.

4 DESIGN CONSIDERATIONS

This section outlines the various aspects of the breakwater requiring consideration in the design process, including:

- wave attenuation;
- wave reflections;
- flushing and circulation; and
- shoreline scour/erosion.

4.1 WAVE ATTENUATION

The design wave climate for the Trinity Point site was established in the Coastal Processes Study (*PatBrit, 2007a*) and was summarised for the southern and eastern breakwaters in **Table 2.2.**

Design elevated still water levels for the Trinity Point site were also established in the Coastal Processes Study (*PatBrit, 2007a*) and were summarised in **Table 2.4**.

The predicted un-attenuated wave climate for Trinity Point and the design criteria for a 'good' wave climate (*in accordance with AS 3962(2001)*), are summarised in **Table 4.1**. On the basis of these values, the design transmission coefficients for the eastern and southern breakwaters are also set out in **Table 4.1.**

Table 4-1: The design requirements for wave attenuation

Breakwater	Wave Direction	1 year ARI			50 year ARI		
		Predicted H_s (m)	Design Criteria H_s (m)	Transmission coefficient K_t	Predicted H_s (m)	Design Criteria H_s (m)	Transmission coefficient, K_t
Southern breakwater	Head seas > 2s	0.86	0.3	0.3	1.27	0.6	0.4
Eastern breakwater	Beam seas >2s	0.66	0.15	0.2	1.05	0.25	0.2

It is noted that wave transmission will not only occur as a result of wave energy passing beneath the breakwater skirt but also in the event of waves overtopping the structure. The transmission coefficients determined above therefore need to cater for both the transmission due to wave energy passing under the structure as well as wave energy passing over the structure. The two parameters that will control the wave transmission through the breakwater are:

- the skirt crest level; and
- the skirt penetration depth (*or bottom level*).

For the purpose of undertaking calculations it is necessary to fix one of these parameters. The skirt crest level was therefore set at 1.5m AHD. This level is considered to be an appropriate balance between being sufficiently elevated to avoid regular submersion, without being visually inappropriate or unusable.

With the skirt crest fixed, the skirt penetration can then be established and the overtopping transmission checked to ensure the required transmission coefficients as summarised in **Table 4.1** are achieved. The cases considered are as follows:

1 year ARI
Case 1: Water level at ISLW (*say -0.1m AHD*) with 1 year ARI significant wave
Case 2: Water level at MHWS (*say 0.1m AHD*) with 1 year ARI significant wave

50 year ARI
Case 1: 1 year ARI water level (*0.4m AHD*) with 50 year ARI significant wave
Case 2: 50 year ARI water level (*1.24m AHD*) with 1 year ARI significant wave

Overtopping transmission was estimated using techniques set out in the Shore Protection Manual (*Figure 7.38*) (*CERC, 1984*). Transmission beneath the skirt breakwater was estimated using the modified theory by Kriebel and Bollman (*1996*).

At this early conceptual design stage it is considered prudent to maintain some conservatism in the design to account for the preliminary nature of the wave hindcasting methods and the transmission calculations. Wave periods are particularly difficult to hindcast confidently using preliminary depth/fetch limited algorithms. For this reason a sensitivity assessment of the required skirt depth was undertaken for slightly longer wave periods for the 50 year ARI Case 1 (*up to 4.0s*) and the 1 year ARI Case 2 (*up to 3.5s*). This resulted in critical bottom of skirt levels for the southern and eastern breakwaters of -3.7m AHD and -4.4m AHD respectively.

The distribution of the resulting transmission coefficients determined are presented in **Table 4.2.**

Table 4-2: The design distribution of the transmission coefficients for wave attenuation

Breakwater	Case	1 year ARI			50 year ARI		
		Total allowable K_t	K_{tu}*	K_{to}**	Total allowable K_t	K_{tu}*	K_{to}**
Southern breakwater	1	0.3	0.27	0	0.4	0.1	0.3
	2	0.3	0.23	0	0.4	0.28	0.12
Eastern breakwater	1	0.2	0.2	0	0.2	0.2	0
	2	0.2	0.2	0	0.2	0.11	0.09

Note: K_{tu} + K_{to} should be less than or equal to the total allowable K_t
*K_{tu} – transmission coefficient under the breakwater skirt
**K_{to} – transmission coefficient for overtopping

Therefore the recommendation for the bottom level of the breakwater skirt is -3.7m AHD for the southern breakwater and -4.4m for the eastern breakwater.

To achieve the adopted skirt crest level of 1.5m it is suggested that a hardwood timber kerb or seat of 0.4m square be fixed along the seaward edge of the deck. The deck level could then be lowered to 1.1m AHD (*refer **Figure 3**)* which would generally be approximately a metre above the lake water level (*refer to the Coastal Processes Study (PatBrit, 2007a) for more detailed information* *regarding tidal levels and extreme water levels*). This would provide a more usable and visually appropriate deck level whilst achieving the required functional performance.

Overtopping with regard to operational safety would need to be considered in the detailed design phase to ensure that pedestrian use of the breakwater decks is safe during regular events. During more severe events it is envisaged that the breakwaters would be closed to pedestrian access.

4.2 WAVE REFLECTIONS

An inherent characteristic of a solid vertical breakwater or wave wall is that a significant proportion of incident wave energy is reflected. Reflected waves can superimpose with incident waves to theoretically double the crest height at the vertical surface. This can have a significant impact in terms of the structural design requirements due to the increase in the wave forces on the structure, as well as operational impacts as water levels at the wall can rise rapidly, affecting the safety and comfort of those on an adjacent deck or landing. A further impact of reflected wave energy is the potential scouring effect the reflected wave energy can have if it impacts with the shoreline. This is discussed further in **Section 4.5.**

Wave reflection off a vertical surface can be reduced by one or a combination of the following techniques:

- Use of a double skirt wall, where the seaward skirt face is porous (*or slatted*) and low wave reflection is achieved through large energy loses due to breaking and turbulence between the two skirts.
- Roughening of the wall surface to promote wave energy dissipation; and
- Angling the wall such that incident wave impacts the wall at an oblique angle.

For the Trinity Point Marina breakwater, the preferred design option of a walkway with a skirt breakwater fixed to the inner side lends itself to the addition of a porous seaward skirt that in combination with a solid inner skirt should reduce reflected wave energy significantly. It is noted that calculations undertaken to determine skirt length to achieve the required wave attenuation were based on a single solid skirt. This will be conservative but is considered a reasonable approach at this conceptual design stage as there are very limited theoretical methods of calculating wave transmission through a double skirt structure. Modelling would be required to achieve more accurate answers during the detailed design phase.

4.3 FLUSHING / CIRCULATION

The Coastal Processes Study (*PatBrit, 2007a*) identified that the dominant currents affecting the Trinity Point Marina site are likely to be wind generated currents moving in a shore parallel direction. Whilst these currents are generally not expected to have significant velocities, (*except in severe events*) it is recommended that the landward end of the southern breakwater be slatted to allow circulation of water through to the marina. A single skirt slatted breakwater structure is likely to be able to achieve the required wave attenuation in this location as the wave heights will be lower due to depth limiting effects as well as the reduced exposure to the south-east. Exactly how far the slatted breakwater could extend would need to be the subject of further investigations at the detailed design stage.

The partial penetration of the skirt breakwater will also allow for some mixing and exchange of water from one side of the breakwater to the other.

4.4 SHORELINE SCOUR/ EROSION

Potential for shoreline scour/erosion has been identified at the landward end of the southern breakwater where reflected wave energy approaching the breakwater at an oblique angle would be reflected towards the shoreline. The recommendation made in **Section 4.3,** to have a single skirt slatted breakwater along this section, would assist in reducing the reflected wave energy nearshore. Further seaward, beyond the slatted portion of the breakwater, the double skirt arrangement would create low wave reflection minimising the potential problem.

4.5 SUMMARY OF DESIGN CONSIDERATIONS AND RECOMMENDATIONS

A summary of the design considerations and the recommendations arising is provided in **Table 4.3** below.

Table 4-3: Summary of design considerations and recommendations arising

Design Issue	Recommendation	
	Southern BW	**Eastern BW**
Wave attenuation	Skirt penetrating 3.7m below AHD with a deck level of 1.1m AHD and a timber seat providing an effective wave barrier to 1.5m AHD.	Skirt penetrating 4.4m below AHD with a deck level of 1.1m AHD and a timber seat providing an effective wave barrier to 1.5m AHD.
Wave reflections	Double skirt breakwater to reduce reflected wave energy. Single slatted skirt at seaward end to reduce reflected wave energy.	Double skirt breakwater to reduce reflected wave energy.
Flushing/Circulation	Partial skirt penetration will allow water circulation to continue. A slatted skirt is recommended for the landward portion of the breakwater to allow flow to continue.	No major issue identified as flow is generally shore-parallel. Partial skirt penetration will allow water circulation to continue.
Shoreline scour	A slatted skirt is along the landward portion of the breakwater will minimise reflected wave energy and thus reduce potential for shoreline scour. A double skirt for seaward portion will also limit reflected energy.	Not applicable.

5 CONCLUDING REMARKS

The proposed conceptual design for the eastern and southern breakwaters providing protection to the Trinity Point Marina comprises skirt breakwaters as presented in **Figure 3**.

As noted throughout the report, there are a number of aspects of the design that could be refined in the detailed design stage through application of hydraulic models to improve estimates of design parameters, such as wave climate. Some design issues such as wave transmission and overtopping could also be more accurately determined using physical modelling techniques.

6 REFERENCES

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FIGURES

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FIGURE 1

6759.04-TRINITY POINT/6759.04 F1.ai

SITE LOCATION PLAN

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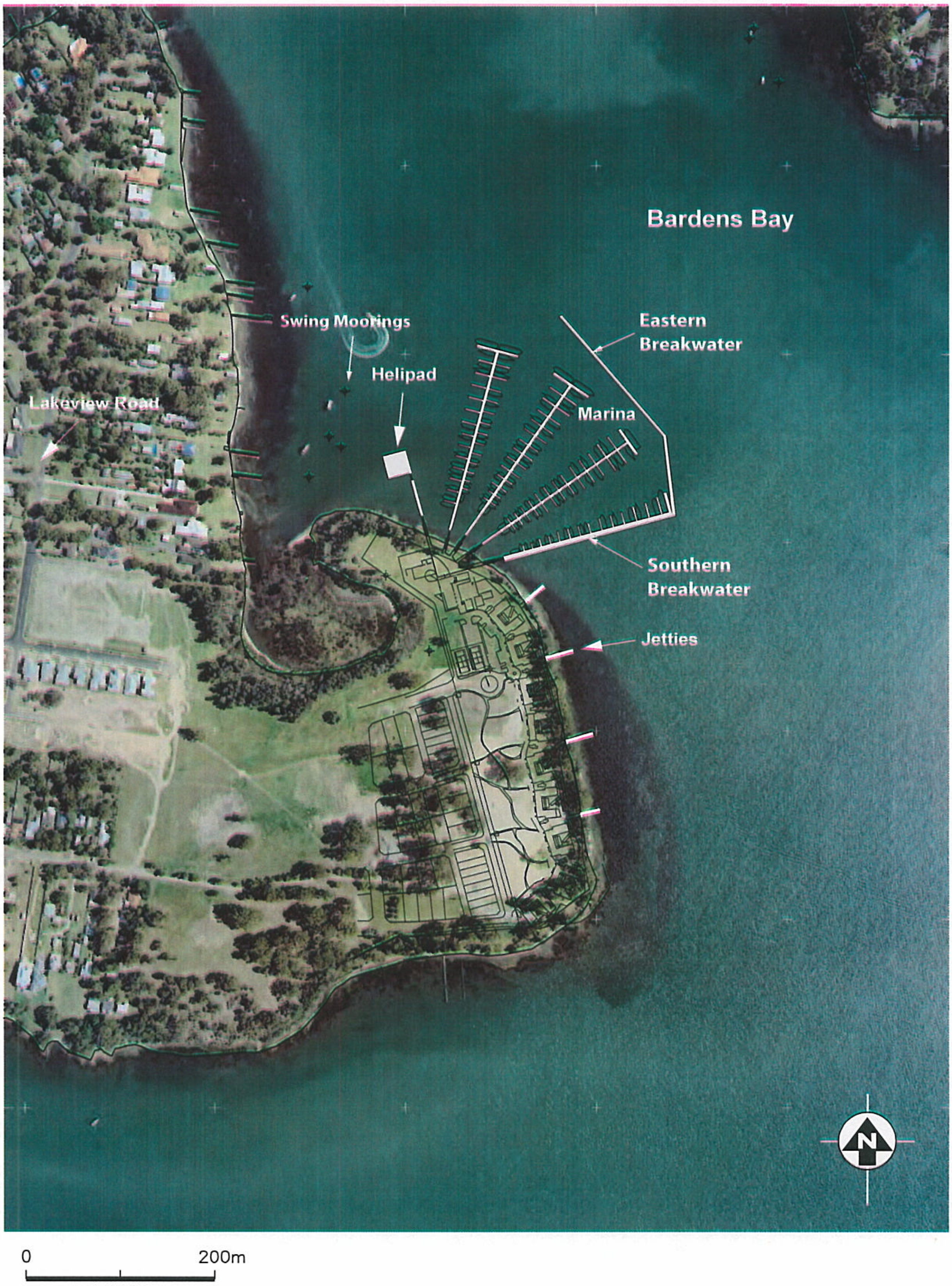
FIGURE 2



SOURCE: AERIAL PHOTO - JPG
6759.04-TRINITY POINT/6759.04 F2

CONCEPTUAL LAYOUT OF TRINITY POINT MARINA, JETTIES AND HELIPAD

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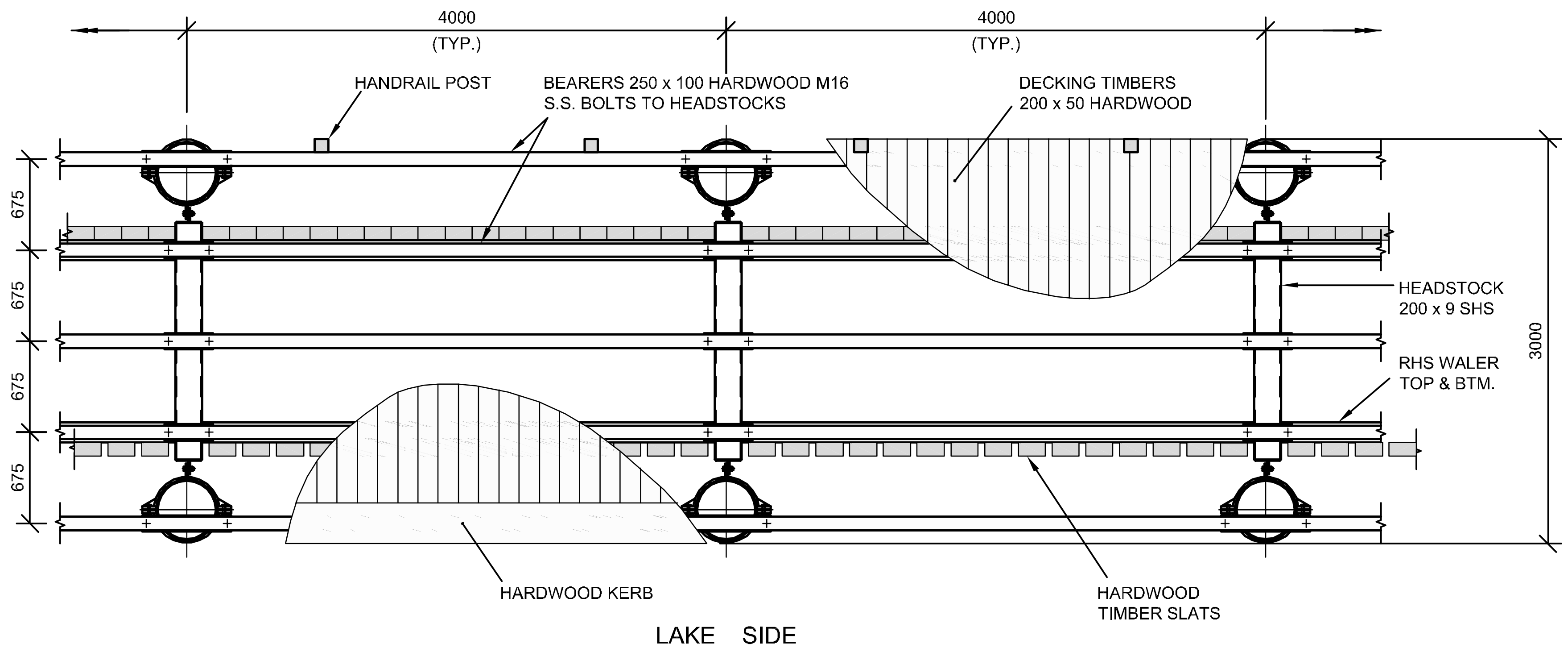
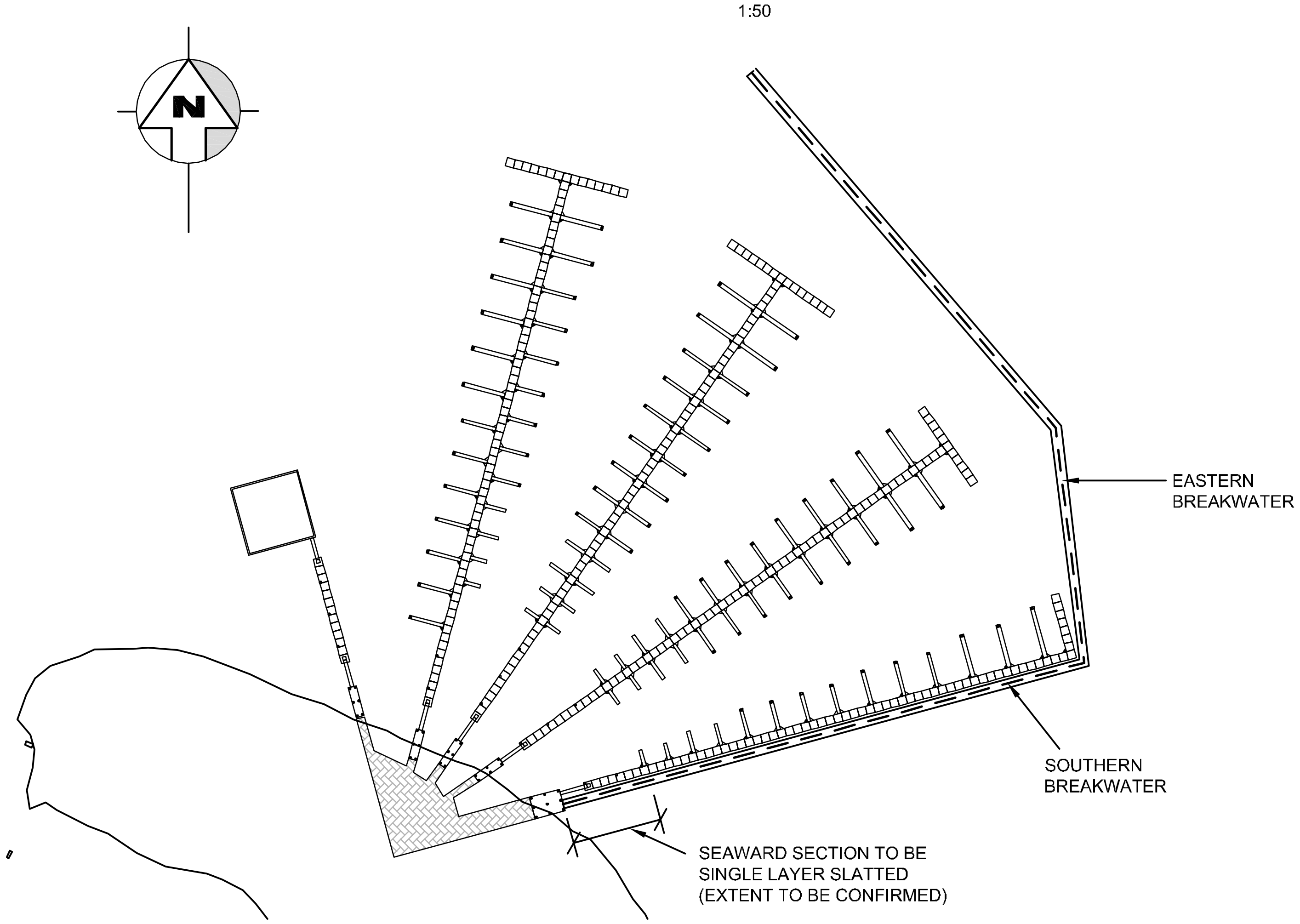
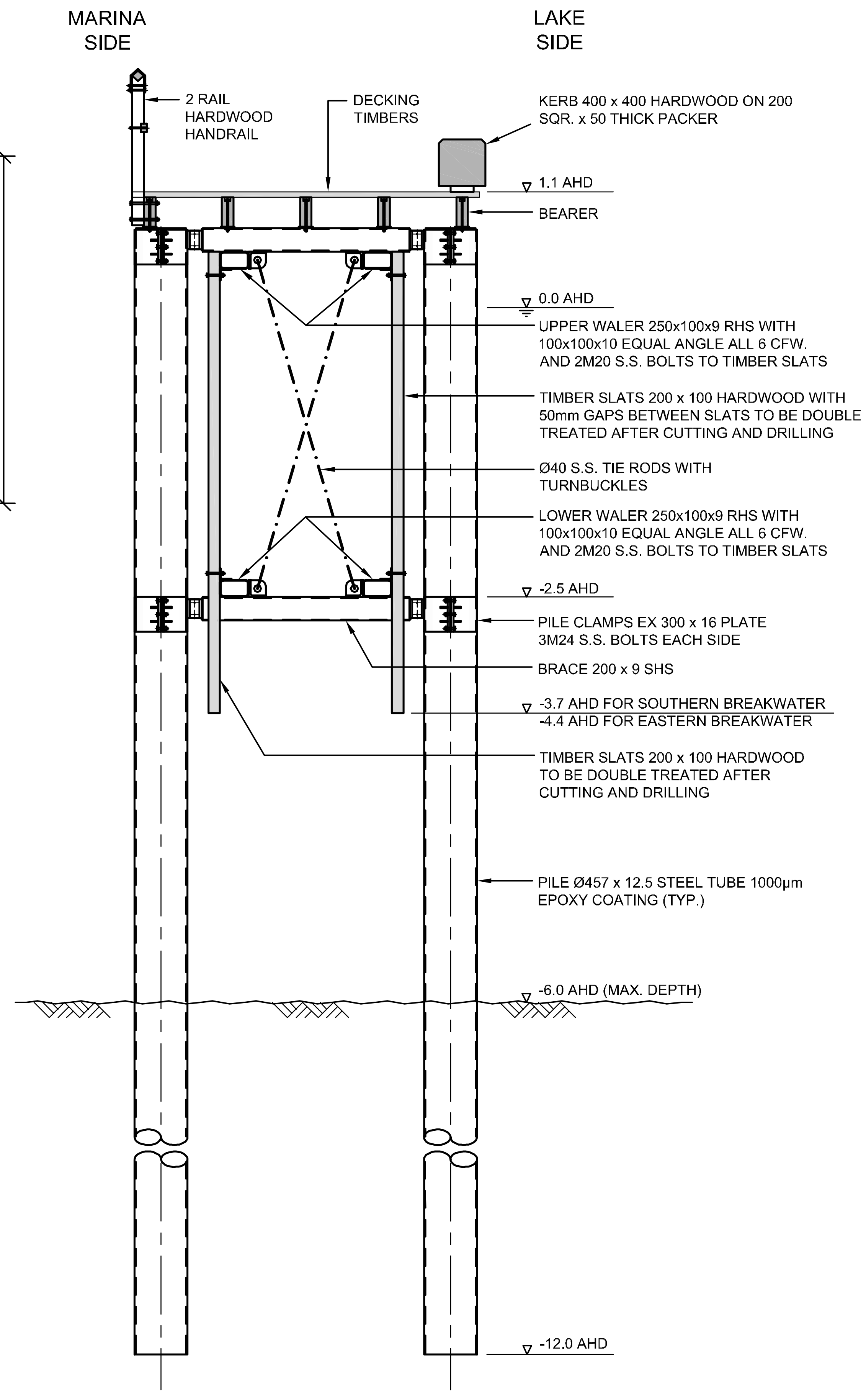
FIGURE 3



PARTIAL PLAN
1:50



MARINA & BREAKWATER PLAN
1:2500



TYPICAL SECTION
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CONCEPT DESIGN
TIMBER SKIRT BREAKWATER