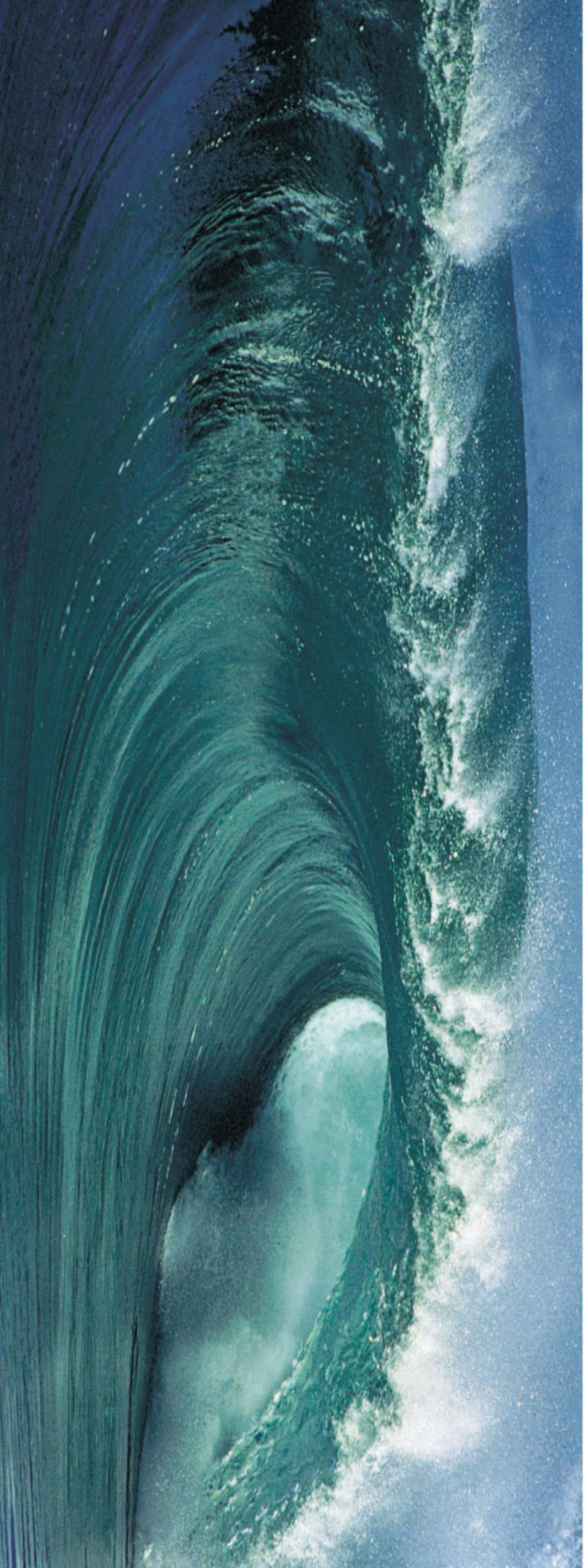

COASTAL ENGINEERING SOLUTIONS

TOWNSVILLE OCEAN TERMINAL PROJECT

COASTAL ENGINEERING STUDIES

SEPTEMBER 2007

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EXECUTIVE SUMMARY

Background

Coastal Engineering Solutions Pty Ltd has been commissioned by City Pacific Limited to undertake the *Coastal Engineering Studies* to support the preparation of an Environmental Impact Statement (EIS) pursuant to the Terms of Reference (TOR) dated March 2007. Particular regard has been made to the TOR Clause 4.7 in relation to local coastal processes.

The studies will also provide technical input to the subsequent detailed engineering design / documentation of maritime elements of the Townsville Ocean Terminal Project.

The *Coastal Engineering Studies* utilise mathematical modelling techniques to address the following aspects with regard to the existing conditions and those that will occur subsequent to the completion of the proposed development:

- the local inshore wave climate (under both cyclonic and ambient conditions); and
- the sediment transport regime on adjacent foreshores.

Preliminary structural designs for the rock armoured breakwaters, seawalls and revetments around the perimeter of the development have also been prepared using the findings of investigations into the extreme wave climate and storm tides.

The storm tide levels used throughout the *Coastal Engineering Studies* for the Townsville Ocean Terminal Project are those determined by the "*Townsville - Thuringowa Storm Tide Study*". That earlier study was completed in April 2007 under the auspices of the Federal Government's Natural Disaster Risk Mitigation Program (NDRMP).

Potential Impacts to Local Wave Climate

Waves arrive in the nearshore waters around the Townsville Ocean Terminal Project as a consequence of several phenomena, namely;

- Swell waves - generated by weather systems in the distant waters of the Coral Sea and Pacific Ocean beyond the Great Barrier Reef. In order to propagate to the mainland foreshores in the vicinity of Townsville, these waves must pass thorough and over the extensive reefs and shoals that constitute the Barrier Reef. There is extensive attenuation of wave energy during this propagation process;

- Distant Sea waves - generated by winds blowing across the open water fetches between the mainland and the outer Great Barrier Reef system (some 70 kms offshore). This includes those fetches to the north of Magnetic Island and south-east of Cape Cleveland (from which waves are then diffracted and refracted as they propagate to the project site);
- Local Sea waves generated by winds blowing across the open waters of Cleveland Bay and the West Channel between Magnetic Island and The Strand / Rowes Bay foreshores.

Because of the complex nature of the wave transformation processes, the studies have utilised mathematical modelling techniques.

Given that the main entrance into the harbour of the Townsville Port is in the vicinity of the proposed Townsville Ocean Terminal Project, the possibility of altered wave conditions at the existing Port entrance were investigated. It was found that the wave energy currently arriving at the existing entrance into the Townsville Port will not be significantly altered by the proposed development.

The results of the investigations indicate that the substantial reclamations associated with the Breakwater Casino and the Townsville Port currently provide varying wave protection to The Strand Beaches.

In particular, the 635m long existing Northern Breakwater that extends to the west from the Port entrance (and which will form part of the new Northern Breakwater for the Townsville Ocean Terminal Project) creates a “wave shadow” on The Strand foreshore. The wave shadow is an area of reduced wave energy. This existing breakwater strongly influences the wave climate on the sandy beaches of The Strand. Consequently it also affects the beaches since it is the prevailing waves which shape this foreshore.

Given that the proposed Townsville Ocean Terminal Project will be extending the existing Northern Breakwater by some 225m in a west-north-west direction, the wave shadow from the lengthened structure will reach further along The Strand foreshore than it does at present. The proposed dredging of a navigable sea access into the new Breakwater Cove waterways will also affect the way in which incoming waves will sweep around the end of this lengthened structure. Both of these aspects have an effect on the beaches, which has been investigated and quantified.

Under existing conditions, the wave shadow significantly affects The Strand foreshore south of about the Gregory Street headland. Further northward, the effect diminishes gradually. However because the existing breakwater is to be lengthened, the wave shadow on The Strand Beaches will extend further north.

As a consequence of the proposed works, the location at which the effects of the wave shadow (ie. the region of lower wave energy) will gradually diminish migrates northward to a location around the Burke Street headland.

The results also indicate that the incoming waves within the new wave shadow (ie. south of around Burke Street headland) will tend to be refracted such that they arrive at The Strand foreshore on a slightly more northerly approach. This is will have implications to the orientation of the beaches along these southern reaches, with a subtle change in the plan alignment of the foreshore in response to the changed wave directions.

There are no identifiable changes to the height and direction of waves arriving at the northern end of The Strand Beaches as a consequence of the proposed development.

Potential Impacts to The Strand Beaches

An expected impact of the proposed development is the reduction and realignment of the wave energy propagating onto the southern reaches of The Strand foreshore. This will manifest itself as some localised realignment of the beach in this area. The extent of this induced change to the plan orientation has been investigated using mathematical modelling techniques.

The results indicate that the beach compartments between the various headlands along central and southern regions of The Strand foreshore will gradually and subtly rotate their plan orientations so as to align themselves with the slightly more northerly wave energy regime.

This realignment of the naturally preferred plan form of the beaches will occur through the transport of sand from the northern end of each beach compartment towards its southern end.

However the changes entail only very minor realignment of the beaches, as follows:

- *Mariners Peninsula to Gregory Street Headland* : A rotation of 0.5º to face more northward (ie. an “anti-clockwise” rotation in its plan alignment). It is anticipated that the beach against the southern side of the Gregory Street headland will gradually migrate to a stable position that is only some 2m inland from its present location.

- *Gregory Street Headland to Burke Street Headland* : A rotation of 0.75º to face more northward (ie. an "anti-clockwise" rotation in its plan alignment). The beach at the northern end of this compartment will recede by approximately 2.5m to attain this stable orientation.
- *Burke Street Headland to Stuart Street Headland* : A rotation of 0.5º to face more northward (ie. an "anti-clockwise" rotation in its plan alignment). The predicted recession at the northern end of this short length of beach is predicted to be less than 1m.
- *Stuart Street Headland to Kissing Point* : No detectable effect on the existing foreshore alignment.

Once these various realignments occur, the new beach plan shape will then become the preferred orientation. This readjustment and movement of sand will manifest itself as a slight narrowing of the beach width in the areas immediately south of the headlands at Gregory Street, Burke Street and Stuart Street - in conjunction with widening of the adjacent beach compartment against their northern sides.

The predicted reorientation of the various beach compartments along The Strand may be perceived by the community as erosion on the southern side of the headlands (which indeed it is). However it will be accompanied by accretion of each beach further south.

The impacts of the proposed Townsville Ocean Terminal Project on the various beach compartments along The Strand foreshore are minor and will not threaten any foreshore infrastructure, nor adversely affect beach amenity.

Rock Armour Works

There are a number of permanent marine structures within the proposed Townsville Ocean Terminal Project which rely on rock armouring to maintain their structural integrity. These being:

- the existing breakwater facing north-east directly out to Cleveland Bay, which will serve as a seawall when land is reclaimed immediately behind this structure (termed herein as the *Northern Breakwater*);
- a new breakwater along the north-west perimeter of the Breakwater Cove waterways (termed herein as the *Strand Breakwater*);
- revetments within the Breakwater Cove waterways.

The requirements of the Environmental Protection Agency's operational policy "*Building and engineering standards for tidal works - Version 1.2*" have been incorporated into the design of these various rock structures.

The determination of structural concepts for rock armour works are preliminary at this stage and will be confirmed by a more rigorous detailed engineering design and documentation phase prior to construction. The intent of the subsequent detailed design phase is to utilise physical modelling techniques in a random wave flume to determine the most appropriate structural arrangements.

As a step towards this later stage, conceptual designs for the rock armouring works have been prepared using mathematical calculations. It is expected that the subsequent physical modelling of these preliminary designs will result in optimisation of their various components (eg. rock sizes, layer thicknesses, crest armouring, etc.) but the overall structural concepts will remain unchanged.

The ocean entrance to the Breakwater Cove precinct is configured so as to provide wave protection to internal waterways which is in accordance with the wave climate requirements of "*AS3962 - Guidelines for design of marinas*".

Compliance with EPA Guidelines Regarding Storm Tide

The Queensland EPA has issued a Guideline document titled "*Mitigating the Adverse Impacts of Storm Tide Inundation - vers 1.2*" which provides advice and information on interpreting Coastal Hazards' Policy 2.2.4 of the *State Coastal Management Plan - Queensland's Coastal Policy* (State Coastal Plan). The Guideline aims to ensure that storm tide inundation is adequately considered when decisions are being made about coastal developments.

For the Townsville Ocean Terminal Project, the 100 year Average Recurrence Interval (ARI) storm tide and associated wave effects constitutes the Designated Storm Tide Event (DSTE) under the State Coastal Plan policy 2.2.4.

The proposed Townsville Ocean Terminal Project complies with the requirements of State Coastal Plan policy 2.2.4 with regard to:

- Dwellings within the development are sited so that the floors on all habitable rooms are above the DSTE level; and by being set back from the perimeter seawalls they are not located within the high hazard zone. Access roads for emergency evacuation purposes are also above the DSTE level.

- The proposed works do not adversely increase the storm tide or the associated waves on adjacent foreshores or properties.
- Being set back from areas potentially prone to wave overtopping, the proposed building work is not sited within the high storm tide hazard zone. The dwellings are also sited with the floors of all habitable rooms located above the DSTE level.
- There are anticipated to be some changes to the wave climate on the southern foreshores of The Strand beaches. This will result in some gradual re-distribution of sand on the foreshore as the local beaches in this area stabilise naturally to new plan orientations. These changes will not adversely affect the natural coastal processes that are currently shaping the southern shores of The Strand; nor will they significantly interfere with tidal flows; alter existing coastal hydrological flows; or create adverse conditions for adjoining coastal vegetation.

1 INTRODUCTION

Coastal Engineering Solutions Pty Ltd has been commissioned by City Pacific Limited to undertake the *Coastal Engineering Studies* to support the preparation of an Environmental Impact Statement (EIS) and to provide technical input to the detailed engineering design / documentation of maritime elements of the Townsville Ocean Terminal Project.

As illustrated on Figure 1.1, the proposed development will be located immediately adjacent to the Townsville Port precinct. Figure 1.2 shows the intended layout; and Figure 1.3 illustrates the proposed development concept in relation to the existing physical environment of the site.

The Townsville Ocean Terminal Project will comprise a cruise ship terminal, and an integrated residential waterfront development (titled Breakwater Cove) located within the site identified as the "Future Development Area" in the *Breakwater Island Casino Agreement Act 1984*.

In accordance with the TOR for the proposed Townsville Ocean Terminal Project, the *Coastal Engineering Studies* address the following aspects with regard to the existing conditions and those that will occur subsequent to the completion of the proposed development:

- the local inshore wave climate (under both cyclonic and ambient conditions); and
- sediment transport regime on adjacent foreshores.

Preliminary structural designs for the rock armoured breakwaters, seawalls and revetments around the perimeter of the development have also been prepared using the findings of investigations into the extreme wave climate and storm tides. These preliminary designs will be refined using physical modelling techniques during the detailed engineering design phase of project implementation.

Figure 1.1 : Location of the Project

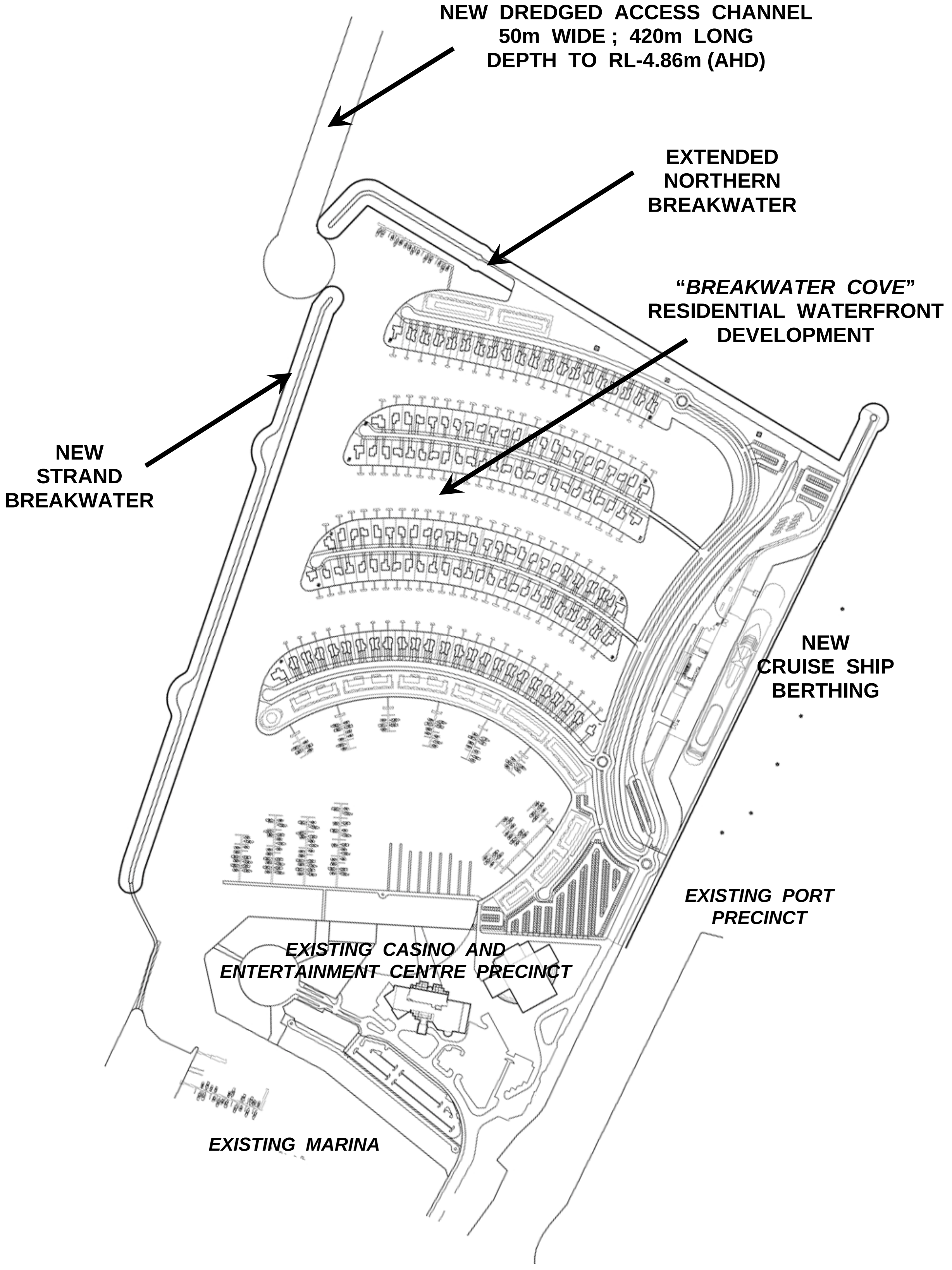


Figure 1.2 : Layout of the Proposed Townsville Ocean Terminal Project

Figure 1.3 : Proposed Townsville Ocean Terminal Project

2 EXISTING CHARACTERISTICS OF THE COASTAL ENVIRONMENT

This section offers a brief outline of the existing coastal environment, whilst later sections provide details as to how the bathymetry and ocean levels have been specifically incorporated into the *Coastal Engineering Studies* for the Townsville Ocean Terminal Project.

Given that it is the depth of water over the seabed approaches which significantly controls the wave energy reaching local foreshores, it is important to have a sound appreciation of the bathymetry and prevailing ocean water levels when assessing the influences of waves on littoral processes.

2.1 Bathymetry

As illustrated in Figure 2.1, the project site is located on the shores of Cleveland Bay - which is an approximately 15km wide, 15km long embayment facing north-east. Cape Cleveland forms its eastern boundary and Magnetic Island forms its western boundary. Both of these topographical features play an important role in defining the wave climate, tidal hydrodynamics and ocean water levels on the foreshores and nearshore regions of Cleveland Bay.

The Great Barrier Reef (GBR) lies some 70kms offshore. However the central section of the GBR just north of the Townsville region is the most porous section of the entire reef system with respect to ocean wave penetration from the Coral Sea and Pacific Ocean (Hardy et al. 2003). This is because it does not form a completely continuous barrier to wave energy which is generated in the Coral Sea to its east. Nevertheless, this extensive offshore reef system still considerably attenuates the passage of ocean swell wave energy.

Cleveland Bay is a somewhat shallow embayment fronting north-east onto the broad open waters between the mainland and the GBR. At its seaward limit, Cleveland Bay is only some 12 metres deep (below the level of the Lowest Astronomical Tide). The seabed approach slopes through the Bay to local foreshores are therefore very flat. These flat shallow approach slopes, in conjunction with the surrounding land features of Magnetic Island and Cape Cleveland, provide natural protection and wave energy attenuation for the project site - particularly during cyclones.

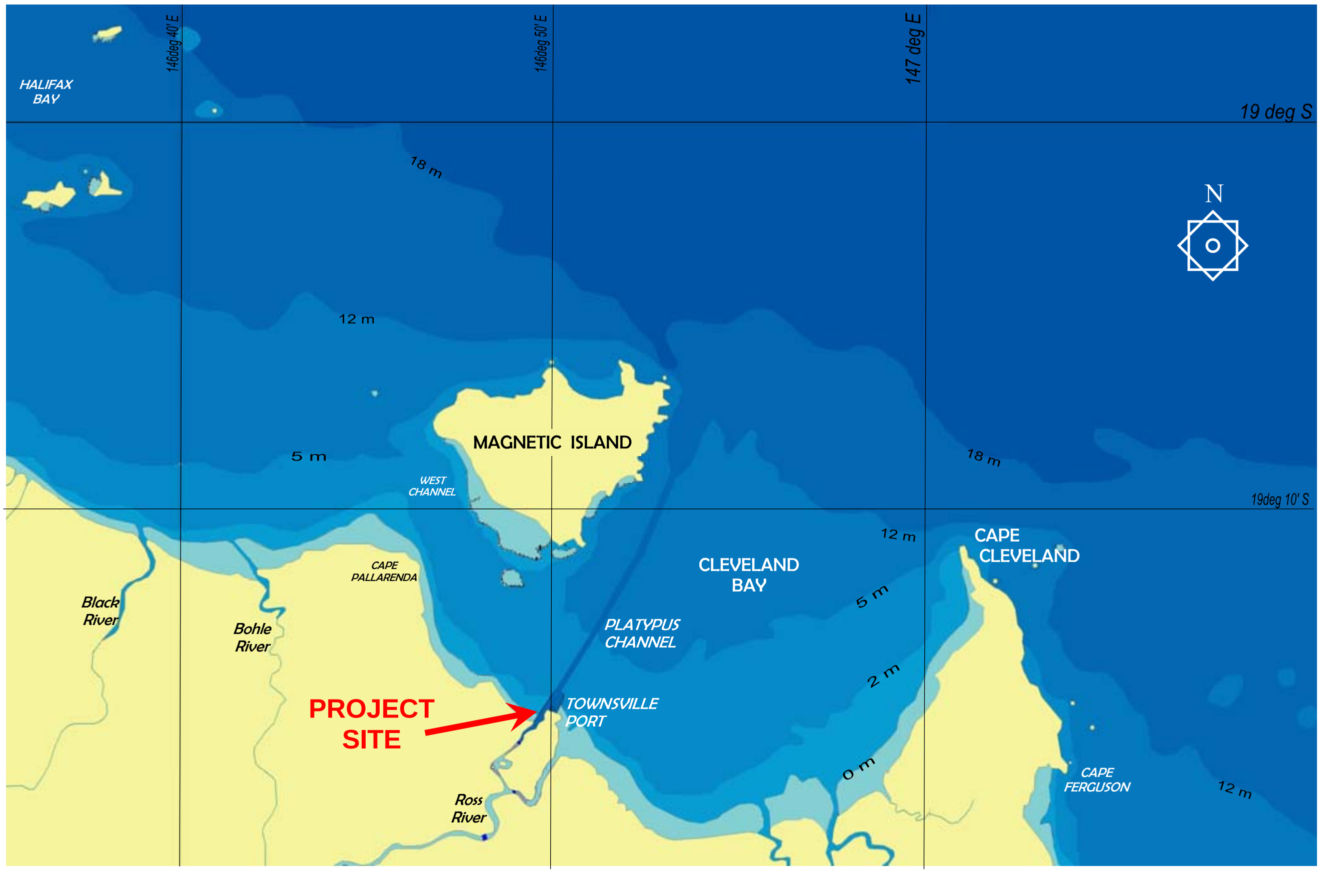


Figure 2.1 : Regional Bathymetry

(depths are in metres below Chart Datum)

Nevertheless, the fetches to the north-east and east of Cleveland Bay are quite long, with the main Great Barrier Reef system being some 70kms offshore. It is from across these open north-east and east fetches that the largest waves can propagate into Cleveland Bay.

The 13km long main shipping channel on the western side of Cleveland Bay that leads into Townsville Port (ie. Platypus Channel) also influences the nearshore wave climate of locations to the west of the channel, and the main port precinct. Platypus Channel is typically 92 metres wide and has a depth of approximately 11.7 metres below Lowest Astronomical Tide. Maintenance dredging is undertaken periodically by the Port of Townsville so that these channel dimensions are sustained for safe navigation of vessels using the Port.

The shallow West Channel between Magnetic Island and Cape Pallarenda allows some waves that are generated in Halifax Bay (across north-western fetches) to propagate to the project site.

2.2 Ocean Water Levels

When planning and designing the broad range of marine facilities proposed for the Townsville Ocean Terminal Project it is necessary to consider the ocean water levels that prevail from time to time. This appreciation not only relates to the day-to-day tidal influences, but also to the storm surges which occur as a result of extreme weather conditions. The expected impacts of climate change on sea level also need to be accommodated in any new coastal development.

As well as ensuring the optimum positioning of proposed infrastructure to the varying ocean water level itself, the effect that ocean levels can have on the local wave climate is also important. As ocean waves propagate shoreward into shallower water, they begin to “feel” the seabed. The decreasing depths cause the waves to change direction so as to become aligned to the seabed contours and to also shoal up in height until such time as they may break - dissipating their energy as they do so. Just how much wave energy reaches the shoreline is therefore determined largely by the depth of water over the seabed approaches. Ocean water levels and the seabed bathymetry are important aspects in this wave energy transmission process.

Consequently it is necessary to have a thorough understanding of the following ocean level phenomenon at the Townsville Ocean Terminal project site:

- *Astronomical Tide* - this is the “normal” rising and falling of the oceans in response to the gravitational influences of the moon, sun and other astronomical bodies. These effects are predictable and consequently the astronomical tide levels can be forecast with confidence.

- *Storm Tide* - this is the combined action of the astronomical tide and any storm surge that also happens to be prevailing at the time. Surge is the rise above normal water level as a consequence of surface wind stress and atmospheric pressure fluctuations induced by severe synoptic events (eg. cyclones).

- *Climate change* - this is the combined effect of environmental changes as a consequence of “Greenhouse” gas emissions into the atmosphere. One of these possible effects is an increase in sea levels.

2.2.1 Astronomical Tides

The tidal planes at Townsville are presented in Table 2.1 below. This information has been derived from widely published information (Maritime Safety Queensland, 2006/07).

Tidal Plane	*to AHD*	*to Chart Datum*
Highest Astronomical Tide	2.15	4.01
Mean High Water Springs	1.21	3.07
Mean High Water Neaps	0.36	2.22
Mean Sea Level	0.01	1.96
Mean Low Water Neaps	-0.27	1.59
Mean Low Water Springs	-1.13	0.73
Lowest Astronomical Tide	-1.86	0.00

Table 2.1 : Tidal Planes at Townsville

In a lunar month the highest tides occur at the time of the new moon and the full moon (when the gravitational forces of sun and moon are in line). These are called "*spring*" tides and they occur approximately every 14 days. Conversely "*neap*" tides occur when the gravitational influences of the sun and moon are not aligned, resulting in high and low tides that are not as extreme as those during spring tides.

As can be seen in Table 2.1, the maximum possible astronomical tidal range at Townsville is 4.01 metres, with an average range during spring tides of 2.34 metres and 0.63 metres during neap tides. Spring tides tend to be higher than normal around the time of the Christmas / New Year period (ie. December - February) and also in mid-year (ie. around May - July). The various occurrences of particularly high spring tides are often referred to in lay terms as "*king tides*" - in popular terminology meaning any high tide well above average height.

The widespread notion is that king tides are the very high tides which occur around Christmas / New Year. However, equally high tides occur in the winter months, but these are typically at night and therefore are not as apparent as those during the Christmas holiday period - which generally occur during daylight hours.

The "normal" rising and falling of the oceans as tides is in response to the gravitational influences of the moon, sun and other astronomical bodies. Whilst being complex, these effects are nevertheless predictable, and consequently past and future astronomical tide levels can be forecast with confidence. However since predictions are computed on the basis of astronomical influences only, they inherently discount any meteorological effects that can also influence ocean water levels from time to time.

When meteorological conditions vary from the average, they can cause a difference between the predicted tide and the actual tide. This occurs at Townsville to varying degrees. The deviations from predicted astronomical tidal heights are primarily caused by strong or prolonged winds, and/or by uncharacteristically high or low barometric pressures.

Differences between the predicted and actual times of low and high water are primarily caused by wind. A strong wind blowing directly onshore will "pile up" the water and cause tides to be higher than predicted, while winds blowing off the land will have the reverse effect.

Clearly the occurrence of storm surges associated with tropical cyclones can also strongly influence ocean water levels, however the effects of cyclones are discussed in the following Section 2.2.2.

Table 2.2 presents a summary of the occurrence of different ocean levels recorded at Townsville Port. These are based on an analysis of observed water levels during the approximate forty-four year period of 04th January 1959 to 31st May 2003. They will include a component of surge during storms and cyclone occurrences in those years, nevertheless they provide very useful information when designing marine infrastructure associated with the Townsville Ocean Terminal Project.

Figure 2.2 presents a plot showing the frequency that various observed high tide levels are equalled or exceeded. The exceedance in the figure is expressed as a percentage of high tide occurrences. For example, reference to Figure 2.2 suggests that in the vicinity of the Townsville Port:

- a water level of RL+1.54m AHD will be equalled or exceeded by 10% of high tides;
- a water level of RL+1.71m AHD will be equalled or exceeded by 5% of high tides;
- a level of RL+1.97m AHD will be equalled or exceeded by only 1% of high tides; and
- a level of RL+2.22m AHD will be equalled or exceeded by 0.1% of high tide occurrences.

Range (to AHD)			no. of hrly observations	% of total	% exceed
-1.96	to	-1.87	6	0.002	100.000
-1.86	to	-1.77	119	0.033	99.998
-1.76	to	-1.67	378	0.103	99.966
-1.66	to	-1.57	719	0.197	99.862
-1.56	to	-1.47	1,362	0.372	99.666
-1.46	to	-1.37	2,236	0.612	99.293
-1.36	to	-1.27	3,188	0.872	98.682
-1.26	to	-1.17	4,401	1.204	97.810
-1.16	to	-1.07	5,861	1.603	96.606
-1.06	to	-0.97	8,398	2.297	95.003
-0.96	to	-0.87	9,191	2.514	92.706
-0.86	to	-0.77	10,536	2.882	90.193
-0.76	to	-0.67	13,868	3.793	87.311
-0.66	to	-0.57	14,653	4.008	83.518
-0.56	to	-0.47	15,740	4.305	79.511
-0.46	to	-0.37	17,412	4.762	75.206
-0.36	to	-0.27	20,338	5.562	70.444
-0.26	to	-0.17	18,413	5.036	64.882
-0.16	to	-0.07	19,166	5.242	59.846
-0.06	to	0.03	18,037	4.933	54.604
0.04	to	0.13	19,435	5.315	49.671
0.14	to	0.23	18,042	4.934	44.356
0.24	to	0.33	17,228	4.712	39.421
0.34	to	0.43	17,074	4.670	34.709
0.44	to	0.53	15,112	4.133	30.040
0.54	to	0.63	14,712	4.024	25.907
0.64	to	0.73	13,144	3.595	21.883
0.74	to	0.83	12,947	3.541	18.288
0.84	to	0.93	10,887	2.978	14.747
0.94	to	1.03	9,589	2.623	11.770
1.04	to	1.13	8,573	2.345	9.147
1.14	to	1.23	6,530	1.786	6.803
1.24	to	1.33	5,337	1.460	5.017
1.34	to	1.43	4,120	1.127	3.557
1.44	to	1.53	3,152	0.862	2.430
1.54	to	1.63	2,105	0.576	1.568
1.64	to	1.73	1,489	0.407	0.993
1.74	to	1.83	1,031	0.282	0.585
1.84	to	1.93	551	0.151	0.303
1.94	to	2.03	321	0.088	0.153
2.04	to	2.13	138	0.038	0.065
2.14	to	2.23	72	0.020	0.027
2.24	to	2.33	16	0.004	0.007
2.34	to	2.43	10	0.003	0.003
2.44	to	2.53	1	0.000	0.000
		TOTAL	365,638		

Based of observations Jan. 1959 to May 2003 (levels are in metres to AHD)

Table 2.2 : Occurrences of Ocean Water Levels at Townsville Port

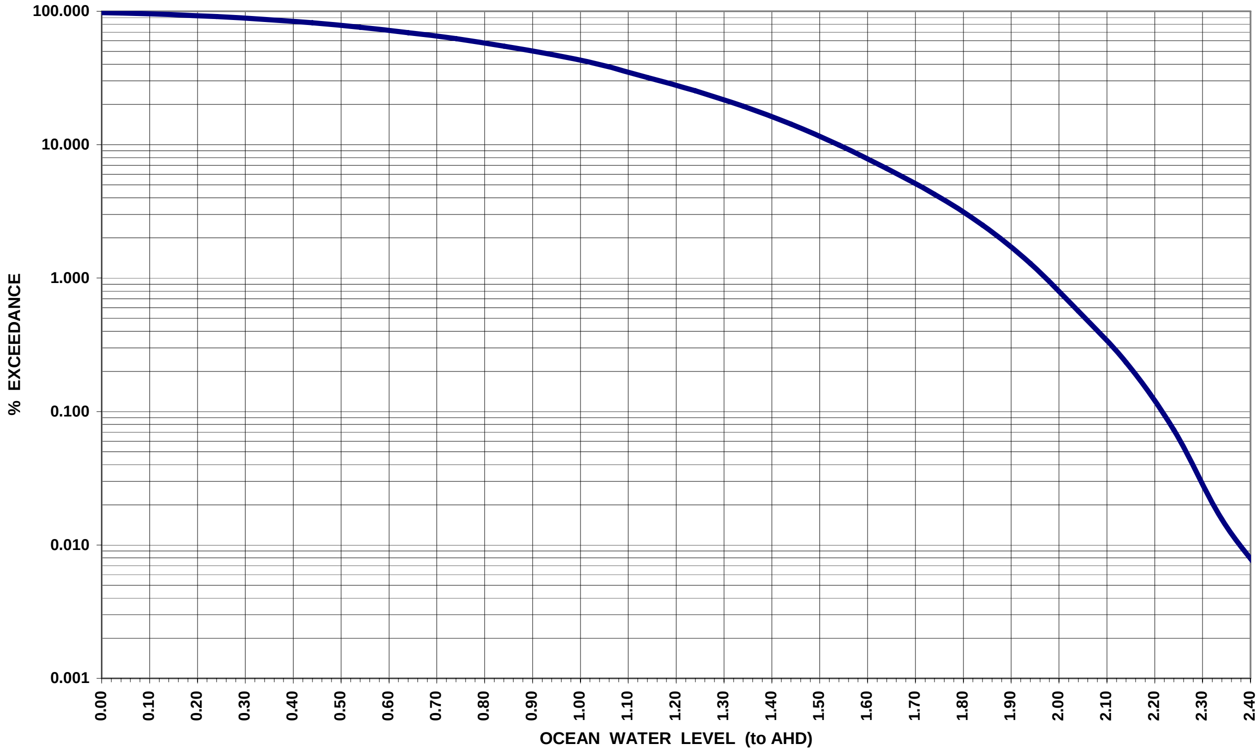


Figure 2.2 : Exceedance Plot of High Tide Levels

Based of observations Jan. 1959 to May 2003 (levels are in metres to AHD)

The frequency of occurrence of tide levels presented in Table 2.2 and Figure 2.2 enable physical infrastructure associated with the Townsville Ocean Terminal Project to be designed for maximum utilisation under most operating conditions. For example the designs for boat / vessel access within the Breakwater Cove waterways and at the Cruise Ship berth can be optimised for the expected range and occurrences of different tide levels.

2.2.2 Storm Tide

The level to which ocean water can rise on a foreshore during the passage of a cyclone or an extreme storm event is typically a result of a number of different effects. The combination of these various effects is known as *storm tide*. Figure 2.3 illustrates the primary water level components of a storm tide event.

A brief discussion of each of these various components that contribute to the height of storm tides in the Townsville region is offered below.

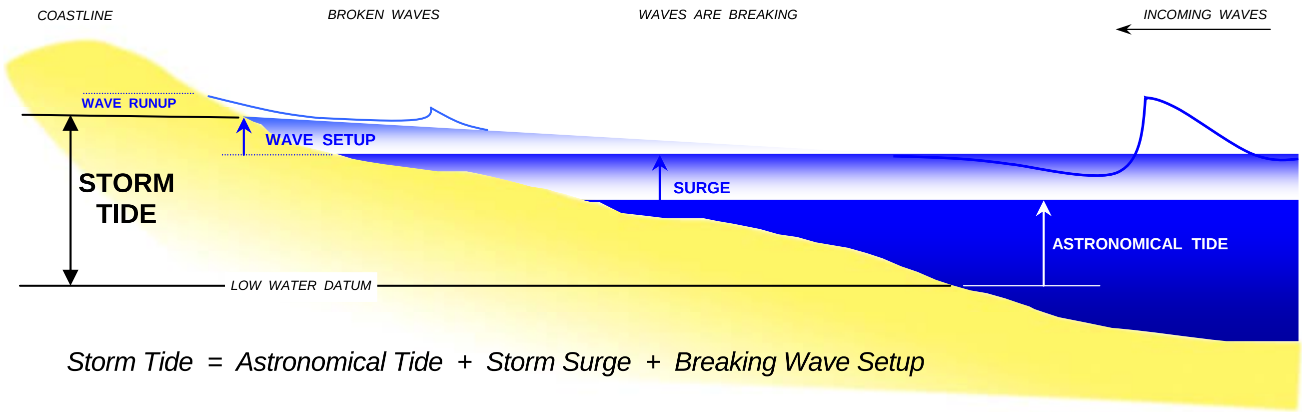


Figure 2.3 : Components of a Storm Tide Event

Astronomical Tide

As discussed in Section 2.2.1, the astronomical tide is the normal day-to-day rising and falling of ocean waters in response to the gravitational influences of the sun and the moon. The astronomical tide at the Townsville Ocean Terminal site can be predicted with considerable accuracy.

Astronomical tide is an important component of the overall storm tide because if the peak of the storm/cyclone were to coincide with a high spring tide for instance, severe flooding of low lying coastal areas can occur and the upper sections of coastal structures can be subjected to severe wave action. The quite high spring tides that typically occur in summer are of particular interest since they occur during the local cyclone season.

Storm Surge

This increase in the ocean water level is caused by the severe atmospheric pressure gradients and the high wind shear induced on the surface of the ocean by a tropical cyclone. The magnitude of the surge is dependent upon a number of factors such as the intensity of the cyclone, its overall physical size, the speed at which it moves, the direction of its approach to the coast, as well as the specific bathymetry of the coastal regions affected.

In order to predict the height of storm surges, these various influences and their complex interaction are typically replicated by numerical modelling techniques using computers.

Breaking Wave Setup

The strong winds associated with cyclones or severe storms generate waves which themselves can be quite severe. As these waves propagate into shallower coastal waters, they begin to shoal and will break as they encounter the nearshore region. The dissipation of wave energy during the wave breaking process induces a localised increase in the ocean water level shoreward of the breaking point which is called *breaking wave setup*. Through the continued action of many breaking waves, the setup experienced on a foreshore during a severe wave event can be sustained for significant timeframe and needs to be considered as an important component of the overall storm tide on a foreshore.

However it is important to appreciate that (as the name implies) breaking wave setup only occurs at the land/sea interface where waves are actually breaking immediately offshore. It does not exist in the deeper areas offshore of the breaking waves (ie. seaward of the surf zone). Nor is it necessarily a significant component of the ocean water levels against coastal structures such as breakwaters or seawalls that are located in deep water where waves don't break offshore before encountering the structure.

Wave Runup

Wave runup is the vertical height above the local water level to which incoming waves will rush up to when they encounter the land/sea interface. The level to which waves will run up a structure or natural foreshore depends significantly on the nature, slope and extent of the land boundary, as well as the characteristics of the incident waves. For example, the wave runup on a gently sloping beach is quite different to that of say a near-vertical impermeable seawall.

Consequently because this component is very dependent upon the local foreshore type, it is not normally incorporated into the determination of the storm tide height. Nevertheless it needs to be considered separately during the assessment of the storm tide vulnerability of each element of the proposed maritime works.

Storm Tide Events at Townsville

A number of studies have previously been undertaken with regard to storm tides that may occur in the Townsville region. The most recently published being the *Townsville - Thuringowa Storm Tide Study* (GHD Pty Ltd, 2007). That study also addresses the effect of enhanced Greenhouse conditions on sea level rise and tropical cyclone occurrences. The storm tides reported by that regional study have been used in the *Coastal Engineering Studies* for the

Townsville Ocean Terminal Project. The storm tide return period relationships determined for the Townsville Harbour precinct indicate the following ocean water levels associated with tropical cyclone occurrences:

Average Recurrence Interval	*RL to AHD with Breaking Wave Setup*	*RL to AHD without Breaking Wave Setup*
50 years	2.64	2.25
100 years	2.89	2.41
500 years	3.52	2.98

Table 2.3 : Storm Tide Levels at Townsville (to AHD datum)

The storm tide levels in Table 2.3 are presented both with and without the effects of breaking wave setup. This is because both values are required for consideration by the *Coastal Engineering Studies*.

As discussed in the preceding pages, breaking wave setup only occurs on the land/sea interface which is shoreward of breaking waves in the surf zone. The ocean water level which includes breaking wave setup is therefore required when considering issues such as foreshore inundation and sediment transport processes on foreshores. The level that excludes breaking wave setup is required when considering how waves change through refraction, diffraction and shoaling processes as they propagate through the deeper waters offshore of where they break near the land/sea interface.

The duration of the storm tide is also a critical consideration when determining effects on sandy shorelines and foreshore structures in Cleveland Bay. The elevated ocean water levels which occur during a storm tide event enable larger waves to impinge directly onto the upper regions of natural foreshores and maritime structures, potentially resulting in their overtopping by incident waves. The surge component of the storm tide typically builds to a peak over several hours, then drops away over a similar or even shorter timeframe as the cyclone influences pass.

2.2.3 Climate Change

Climate change as a consequence of greenhouse gas emissions will cause environmental changes to ocean temperatures, rainfall, sea levels, wind speeds and storm systems. If climate changes develop as predicted, the foreshores of the Townsville region could be subjected to potentially greater storm and cyclone activity, higher waves, stronger winds and increased water levels.

Of all potential impacts, only sea level has been quantified to the extent that some policy statements quote actual values. There are still significant uncertainties regarding predictions of the impact of climate change on sea level rise.

At the present time, the best analytical data seems to suggest that the global mean sea-level could rise by about 48cm (plus or minus approximately 40cm) between 1990 and 2100 (IPCC; 2001 and CSIRO; 2001 and NCCOE; 2004). The policy adopted by the Queensland Government - with respect to building and engineering standards for maritime works - requires an allowance for Greenhouse induced sea level rise of 0.3 metres.

The likely effects of a Greenhouse-induced sea level rise are also presented in the *Townsville - Thuringowa Storm Tide Study*. That study concluded that by the year 2050, the estimated storm tide levels throughout the study region will have increased on average by 0.4m at the 50 year ARI and 0.5m at the 100 year and 500 year ARI. Consequently these values have been adopted in the *Coastal Engineering Studies* for the Townsville Ocean Terminal Project when considering sea level rise as a consequence of future climate change.

These expected increases in average sea levels will therefore result in the predicted storm tide levels shown below in Table 2.4.

Average Recurrence Interval	*RL to AHD <u>with</u> Breaking Wave Setup*	*RL to AHD <u>without</u> Breaking Wave Setup*
50 years	3.04	2.65
100 years	3.39	2.91
500 years	4.02	3.48

Table 2.4 : Storm Tide Levels at Townsville including Greenhouse-induced Sea Level Rise

2.3 Wave Climate

Waves arrive in the nearshore waters around the Townsville Ocean Terminal Project as a consequence of several phenomena, namely;

- Swell waves - generated by weather systems in the distant waters of the Coral Sea and Pacific Ocean beyond the Great Barrier Reef. In order to propagate to the mainland foreshores in the vicinity of Townsville, these waves must pass thorough and over the extensive reefs and shoals that constitute the Barrier Reef. There is extensive attenuation of wave energy during this propagation process.

- Distant Sea waves - generated by winds blowing across the open water fetches between the mainland and the outer Great Barrier Reef system (some 70 kms offshore). This includes those fetches to the north of Magnetic Island and south-east of Cape Cleveland (from which waves are then refracted as they propagate shoreward to the project site).
- Local Sea waves - generated by winds blowing across the open waters of Cleveland Bay and the West Channel between Magnetic Island and The Strand / Rowes Bay foreshores.

Waves from these various sources can occur simultaneously.

A significant focus of the *Coastal Engineering Studies* undertaken for this project has been the determination of the ambient (ie. the “day-to-day”) wave climate - as well as the extreme wave climate (ie. due to cyclones and severe storms). Because of the complex nature of the wave transformation processes, the studies have utilised mathematical modelling techniques. The following Section 3 of this report provides details as to the methodology applied to this determination, along with the results. However some comment is warranted with respect to the various types of waves that can affect the project.

2.3.1 Swell Waves

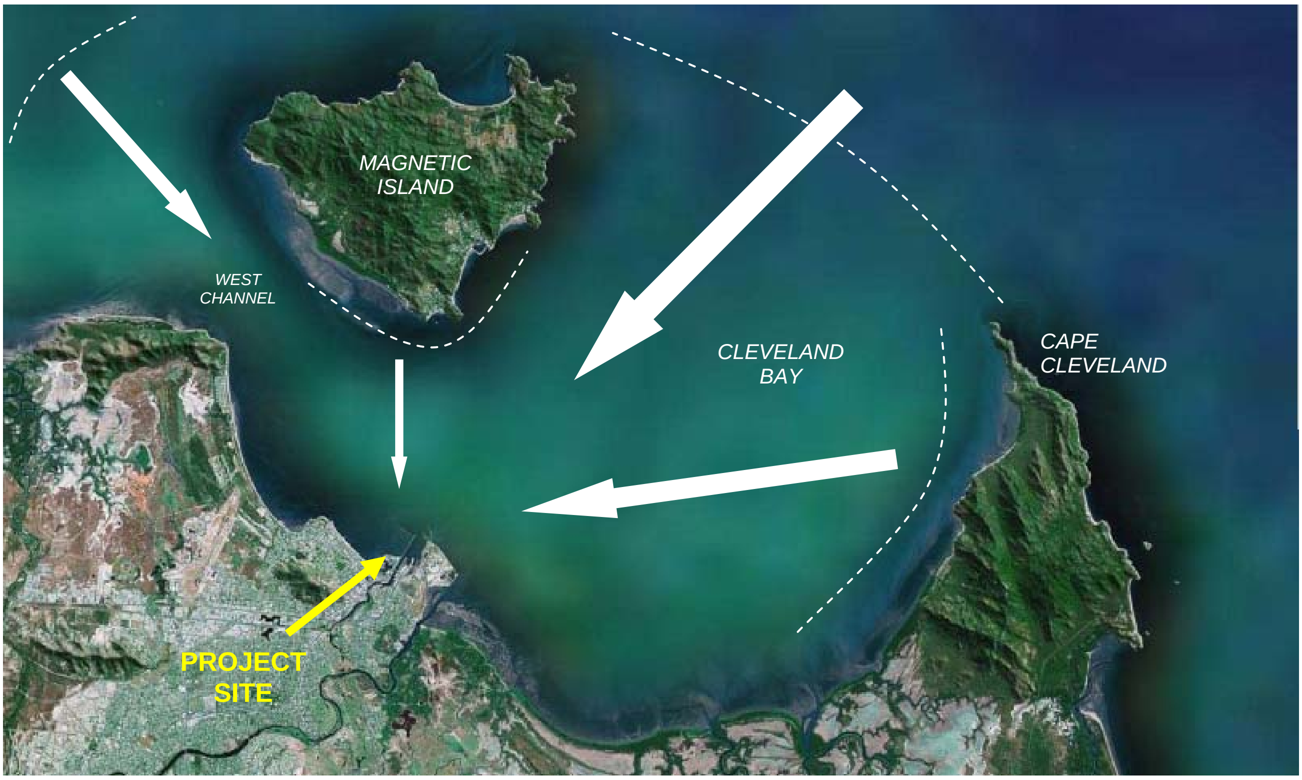
The Great Barrier Reef inhibits the open ocean swell generated by weather systems out in the Coral Sea as this swell propagates to the mainland. Nevertheless, whilst inshore swell wave heights are quite low, because of their relatively long wave periods (typically in excess of around 10 seconds) they can contribute somewhat to local sediment transport processes.

2.3.2 Distant Seas

The significant distances between the mainland and the Great Barrier Reef means that quite sizeable waves can be generated by winds blowing across these fetches - particularly during cyclones which are a common synoptic event in these waters. Figure 2.4(a) shows the different fetches across and from which Distant Seas can propagate to the proposed Townsville Ocean Terminal development.

To the north-east and east of Townsville there are very long open water fetches across which winds can generate significant wave energy. It is from this sector that the largest waves approach the entrance to Cleveland Bay.

(a) Wave Fetches for Distant Seas



(b) Wave Fetches for Local Seas

Figure 2.4 : Wave Fetches Affecting Project Site

Whilst the project site is sheltered by Cape Cleveland (and to a lesser extent by the more distant Cape Bowling Green) from the direct effects of waves generated out of the south-east quadrant, these waves can diffract and refract around the northern tip of Cape Cleveland and propagate to the project site. The attenuating effects of diffraction and refraction mean that the energy of these waves is diminished.

Nevertheless, because they are driven by the predominant seasonal weather systems, these waves from the south-east and east sectors represent an important component of the ambient wave climate in Cleveland Bay. Their persistent nature means that they can strongly influence beach processes in the region.

2.3.3 Local Seas

Whilst the northerly and north-westerly fetches (from the Townsville Port precinct across towards Magnetic Island and the West Channel) are relatively short and shallow, they still enable substantial wave energy to be generated and propagate to the project site - particularly during local cyclone events. Figure 2.4(b) illustrates the different fetches across and from which Local Seas can propagate to the proposed Townsville Ocean Terminal development.

These waves will be those critical to the design of the proposed new Strand Breakwater since it will be aligned almost perpendicular to these local sea waves as they approach and impinge on the structure. They will also play an important role in determining operational aspects of the entrance into the Breakwater Cove waterways, since this entrance also faces into these local seas.

2.4 Littoral Processes & Historical Changes to Foreshores

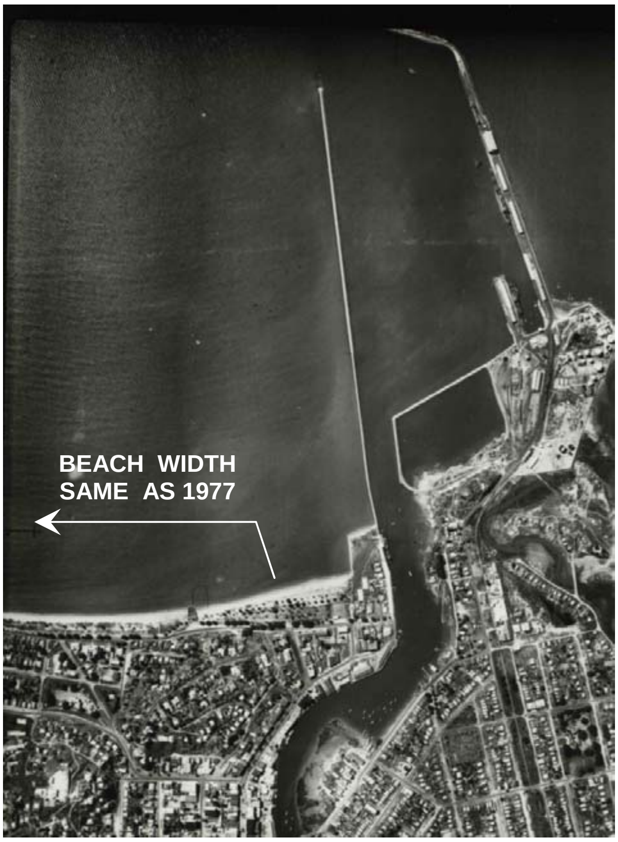
There have been many changes to the physical environment of Townsville since European settlement which, whilst benefiting the community, have initiated changes to the local littoral transport regime.

Prior to the building of dams and flood mitigation works on the Ross River; the construction of the Port of Townsville; and the extraction of sand to for the building industry to provide for the expansion of Townsville, a greater volume of sediments was supplied to local foreshores - both to the east and to the west of the present port site. Over the years several estimates have been prepared regarding what the original sediment yield from the Ross River might have been. These estimated sediment discharges range from 68,000 tonnes/year (Sinclair Knight Merz, 1996) to 330,000 tonnes/year (Belpario, 1978).

The subsequent development of the Townsville region resulted in this natural supply of sand onto local foreshores being severely restricted. This was due to both a reduction in the volume being delivered to the coast by the Ross River, and by the sediment pathways from the river entrance to adjacent foreshores being impeded by development.

Prior to European settlement, local foreshores to the north-west of the Ross River entrance would have been gradually accreting seaward as the prevailing wave and current conditions dispersed the sediments brought down the river to the coast. Gradually the coastal reach up to Kissing Point would have filled with sand. Some of the finer sand fractions would have been transported onshore by winds to supplement dune formation. It is likely that, up until recent times, there would have been some sand being swept northwards from The Strand precinct around Kissing Point into Rowes Bay.

Following the restrictions to sand supply as a consequence of Townsville's development, sand that was already within the active beach system began to redistribute itself, creating a pocket beach adjacent to the port's Strand Breakwater. It is clear from measurements taken off historical aerial photographs that this beach had reached a state of equilibrium by 1941 - as evidenced by no further widening or extension of that beach in the duration between the photographs of 1941 and 1977 (refer to Figure 2.5).



1941

1977

Figure 2.5 : Historical Changes to The Strand Beaches

The sand to create this pocket beach would have come from the central part of The Strand foreshore. Waves arriving from the north-easterly sector would have transported the sand alongshore to the southern end of The Strand beach compartment. However the new port breakwaters created a "wave shadow" during south-easterly waves that prevented this sand being returned northwards to maintain a dynamic equilibrium in the planform of the beach. Other losses of sand from The Strand beach would have occurred due to onshore wind and waves washing some sand around Kissing Point into Rowes Bay.

The subsequent construction of Kissing Point Pool effectively created a barrier to longshore sand transport around Kissing Point. A small amount of sand had accumulated to the south-east of the pool as a consequence of ambient wave conditions. However, the wave shadow cast by the Kissing Point Pool reclamation prevented sand being transported from this northern end of The Strand precinct back towards the central and southern areas.

In other words, seasonal wave processes were transporting sand from the central portion of The Strand foreshore towards either end, from where it could not be transported back during subsequent seasons. This feed of sand into “sinks” at either end of the coastal reach, in conjunction with the diminished supply from the Ross River entrance, resulted in the gradual erosion of the central sections of The Strand foreshore. A seawall was intermittently constructed along the entire length of The Strand in response to this gradual erosion.

Nevertheless, the natural processes causing the erosion of sand from the central portion of The Strand coastal reach continued to the extent that the seawall became increasingly vulnerable as its foundations became more and more exposed by the lowering sand levels on the eroding beach. In March 1997, the persistent but relatively moderate waves generated by Tropical Cyclone Justin were sufficient to initiate undermining and structural failure along several sections of the seawall. This prompted a major foreshore protection project involving seawall upgrade, the creation of beach compartments between artificial headlands, and significant sand nourishment. These works were completed in late 1999.

The coastal engineering design of The Strand foreshore protection works achieved the objective of creating a 2km long coastal reach between the port/marina reclamations and Kissing Point that has minimal sand loss. Nevertheless this “self contained” beach compartment does experience seasonal variations to shoreline orientation which manifests itself as local erosion of sand in some locations that are balanced by local accretion at other locations.

These local erosion / accretion cycles are the natural response to the varying seasonal wave conditions. The beach compartments and the artificial headlands have been designed to accommodate such natural processes for ambient and cyclonic wave conditions (Coastal Engineering Solutions, 1998) and are consequently stable beaches in dynamic equilibrium with the prevailing coastal conditions.

An important focus of the *Coastal Engineering Studies* undertaken for the Townsville Ocean Terminal Project has been to identify any adverse changes that might occur to The Strand beaches. Section 5 of this report provides details as to the methodology applied to this determination, along with the results.

3 MATHEMATICAL MODELLING OF WAVES

3.1 Methodology

There are currently no long-term reliable wave measurements available for the nearshore waters in the immediate vicinity of the Townsville Port. Consequently it is necessary to undertake mathematical modelling to generate an appropriate understanding of the local wave climate - both in terms of the ambient waves (ie. those which prevail on a “day-to-day” basis), as well as the extreme waves associated with cyclones and severe storms.

Ambient wave conditions influence operational and useability aspects of the proposed marine facilities. They also affect beach processes on adjacent foreshores. Whilst ambient waves are not as severe as those which occur during cyclones, their persistent and on-going nature means that they strongly influence stable beach orientations and the seasonal variations that occur to such orientations.

Cyclone wave characteristics at nearshore locations determine such important structural parameters as the crest levels of breakwaters/seawalls; the sizes and gradings of armour rocks; the front face slopes of breakwaters/seawalls; and quantities of any overtopping water during severe wave events.

A suite of computer programs has been used to mathematically model the waves affecting the Townsville Ocean Terminal Project. In order to establish the specific wave characteristics at inshore locations, it is first necessary to determine the characteristics of the waves where they are generated in the deep waters offshore. Subsequent consideration of local effects as these deep water waves approach the coastline provides the necessary understanding of the inshore wave climate.

The mathematical modelling of waves has therefore been undertaken in two parts, namely:

1. *Wave Generation* : the determination of the offshore (deep water) wave characteristics through consideration of meteorological and oceanographic influences. Wave conditions as a consequence of cyclone events have been considered along with those which occur on a “day-to-day” basis.

2. *Wave Transformation* : the transformation of these offshore waves as they propagate towards the shoreline and across the nearshore bathymetry of Cleveland Bay – taking into account wave refraction, diffraction, shoaling, breaking and attenuation due to seabed friction as necessary.

As discussed in the preceding Section 2.3, the wave climate at the project site consists of several distinct but potentially co-existing phenomena - namely Swell, Distant Seas and Local Seas. So the two stage process of determining wave generation and wave transformation for each of these wave types needs to be considered.

When undertaking mathematical modelling of waves, the model predictions need to be verified by comparing them to actual measurements. The absence of any suitable nearshore wave measurements in the Townsville region for verification purposes does not necessarily represent a problem in this regard. The wave study is also being undertaken as input to a subsequent Sediment Transport module of the modelling process to determine potential implications to the orientation of existing beaches adjacent to the proposed development.

The process by which the modelling work has been verified has therefore been to use the *Wave Generation / Wave Transformation Modules* in conjunction with the *Sediment Transport Module* to predict the stable orientation of existing beach systems - and to then compare that prediction with the actual orientation. The various beach compartments within The Strand coastal reach have been used to verify the model. The outcome of this verification process is presented and discussed in Section 5.2.

3.2 The Design Storm Event

The Queensland Environmental Protection Agency's guideline document "*Mitigating the Adverse Impacts of Storm Tide Inundation - vers 1.2*" provides advice and information with regard to interpreting and implementing the "Coastal hazards" policy (policy 2.2.4) of the *State Coastal Management Plan - Queensland's Coastal Policy* (State Coastal Plan).

We note that for recent projects proposed for Townsville's foreshores, the Townsville City Council has required that all such developments must accommodate the 100 year Average Recurrence Interval (ARI) storm tide and associated wave effects.

This is the storm tide level adopted by Council for managing development in this location, and therefore constitutes the Designated Storm Tide Event (DSTE) under the State Coastal Plan policy 2.2.4. Consequently these *Coastal Engineering Studies* have focused on determining the extent to which the site may be affected by this 100 year ARI Defined Storm Tide Event.

Furthermore (and as discussed in the later Section 6 of this report), the requirements of the Environmental Protection Agency's operational policy "*Building and engineering standards for tidal works - Version 1.2*" have been incorporated into the design of marine structures for the Townsville Ocean Terminal Project.

Of particular relevance are the minimum acceptable standards for seawalls presented under Clause G of the policy, which states that seawalls must be designed to withstand wave and water level conditions associated with the 50 year Average Recurrence Interval (ARI) event. A further requirement is that damage to structures as a consequence of such an event does not result in more than 5% of the armour units being dislodged.

However, marine infrastructure associated with the Townsville Ocean Terminal Project will be designed to accommodate the more severe 100 year ARI design storm event - which as discussed above is also the storm event to be used for storm tide hazard mitigation under State Coastal Plan policy 2.2.4.

The selection of the 100 year ARI event is not a straight forward or simple process, as it consists of a combination of severe waves and extreme water levels. When designing coastal defences it is necessary to consider the likelihood of both conditions occurring simultaneously.

The assumption of complete dependence between waves and water levels in an analysis of joint occurrence would lead to a very conservative design - since the 100 year Average Recurrence Interval event would have to comprise a 100 year ARI storm tide level and a 100 year ARI wave height.

Conversely the assumption of independence between waves and water levels could lead to under-design, since any increase in the probability of high waves at times of very high water levels would have been ignored. The actual correlation between waves and storm tide levels will lie between these two extremes of complete dependence and complete independence.

Wave characteristics and the storm surge can generally be estimated for a cyclone of any given intensity and size, however the storm tide level depends upon when the peak surge generated by the cyclone occurs in relation to the astronomical tide. For example, a severe cyclone which produces high waves and a high storm surge will not produce a high storm tide if it occurs around the time of low tide. The large surge and severe waves occurring at low tide might result in less wave energy reaching the foreshore (due to the waves breaking in the shallower seabed approaches) than a moderate surge and moderate wave conditions occurring at high tide.

The approach adopted when developing structural concepts for the various perimeter seawalls and breakwaters of the Townsville Ocean Terminal Project has been to consider the following scenarios as potentially constituting the 100 year ARI event, and to then select the one having the most adverse effect on structural performance:

- Scenario 1 : 100 year ARI storm tide level occurring simultaneously with the 50 year ARI wave characteristics; or
- Scenario 2 : 50 year ARI storm tide level occurring simultaneously with the 100 year ARI wave characteristics.

This “rule of thumb” approach to the determination of cyclone wave conditions affecting the Townsville Ocean Terminal Project is appropriate at this initial stage of the engineering design process. The resulting wave and storm tide conditions are used to formulate initial structural concepts for breakwaters and seawalls which will be subsequently refined during detailed engineering design. The design phase will utilise physical modelling of the various structures in a Random Wave Flume. This aspect is discussed in greater detail in Section 6.1.

3.3 Offshore Wave Climate

As stated previously, the first step in the wave modelling process is the determination of waves that exist in the deep waters offshore of Cleveland Bay. A discussion of the methodology and findings in relation to the offshore wave climate is offered in this Section 3.3. In doing so, it is necessary to discuss the methodology adopted for separately determining the offshore waves associated with each of the main components of the wave climate at Townsville - namely Swell, Distant Seas and Local Seas - as well as for the ambient and extreme conditions associated with each of these wave types.

The assessment of swell waves requires special consideration. Consequently when presenting the following discussion of methodologies, it is considered more appropriate to discuss swell waves after Distant and Local Seas.

3.3.1 Distant Seas

Distant Seas are those waves generated by winds blowing across the open water fetches between the mainland and the Great Barrier Reef (refer to earlier Figure 2.4(a)).

3.3.1.1 Ambient Conditions

In July 1975, the (then) Beach Protection Authority installed a wave recording station in the waters offshore of the Townsville region. This station has been in operation since that time and is currently maintained and operated by the *Coastal Services Unit* of Queensland's Environmental Protection Agency. It consists of a Waverider buoy moored offshore of Cape Cleveland. Individual buoys have been replaced as part of their on-going maintenance, therefore the actual position of the offshore measuring site has varied slightly over the years. The instrument has typically been moored 4km to 5km north-east of Cape Cleveland, in approximately 20 metres depth of water (refer to Figure 3.2).

Whilst there have been some gaps in the records over the three decades of its operation, these measurements nevertheless represents a considerable database with respect to the important parameters of wave height and period offshore of Cleveland Bay.

The various Waverider buoy deployments over the years did not include wave direction as part of the record - apart from an approximate four year period from 12th October 2000 to 28th September 2004 when the station was converted temporarily to a directional recording site. During this time, wave measurements were typically recorded at half-hourly intervals. Some 53,777 wave records were collected during the four year deployment of the directional Waverider buoy.

Given the importance of wave direction to the subsequent transformation of offshore waves to inshore sites in the vicinity of the Townsville Ocean Terminal Project, the data recorded during the deployment of the directional Waverider buoy has been used by the mathematical modelling process to investigate the ambient Distant Sea waves. During the four year dataset there were a number of significant weather events that resulted in offshore significant wave heights of 1.5m to 2m, indicating that the Waverider records included the effects of a number of storms and distant cyclones.

3.3.1.2 Extreme / Cyclone Conditions

Cyclone wave information in the deep waters offshore of Townsville has been extracted from the data generated for the *Atlas of Tropical Cyclone Waves in the Great Barrier Reef*. The

Marine Modelling Unit (MMU) of the School of Engineering at James Cook University has compiled the wave atlas from the results of previously completed simulations of 6,000 tropical cyclones representing the tropical cyclone population that threatens the Great Barrier Reef region. The modelling represents approximately 2,500 years of simulated tropical cyclone occurrences.

The analyses include Average Recurrence Interval and encounter probability curves for significant wave height (H_s) as well as peak period (T_p) and mean direction (θ_m) information. A detailed description of the methodology applied for this earlier study is presented in Hardy *et al.* (2003).

The resolution of the computational grid used for the simulation of tropical cyclones was 1,500m and a location in the deep water offshore of Cleveland Bay was investigated to provide the extreme offshore wave climate for this project. This offshore site is some 24km north-north-east of Magnetic Island and was selected because it is directly opposite Magnetic Passage (which is a significant gap in the reefs and shoals of the outer Barrier Reef). It is therefore located so as to be particularly exposed to cyclone-generated waves that may pass through this gap in the Great Barrier Reef.

A summary of the cyclone wave climate for this site is presented in Figure 3.1. It would be appropriate to provide some comment regarding interpretation of the data provided in this presentation format. Consequently a discussion is offered by way of reference to the results summarised in Figure 3.1.

Figure 3.1 (a)

Waves with a significant wave height H_s of approximately 7.1 metres have an ARI (or return period) of 50 years. This means that significant wave heights of this magnitude (or greater) occur on a long-term average once every 50 years.

Figure 3.1 (b)

If a project life of 50 years were assumed, then the probability of encountering offshore waves with Hs $\geq$ 6.0 metres is PE = 0.85. In other words, there is an 85% chance of encountering waves in the waters offshore of Cleveland Bay which are equal to or greater than 6.0 metres at least once during any 50 year period.

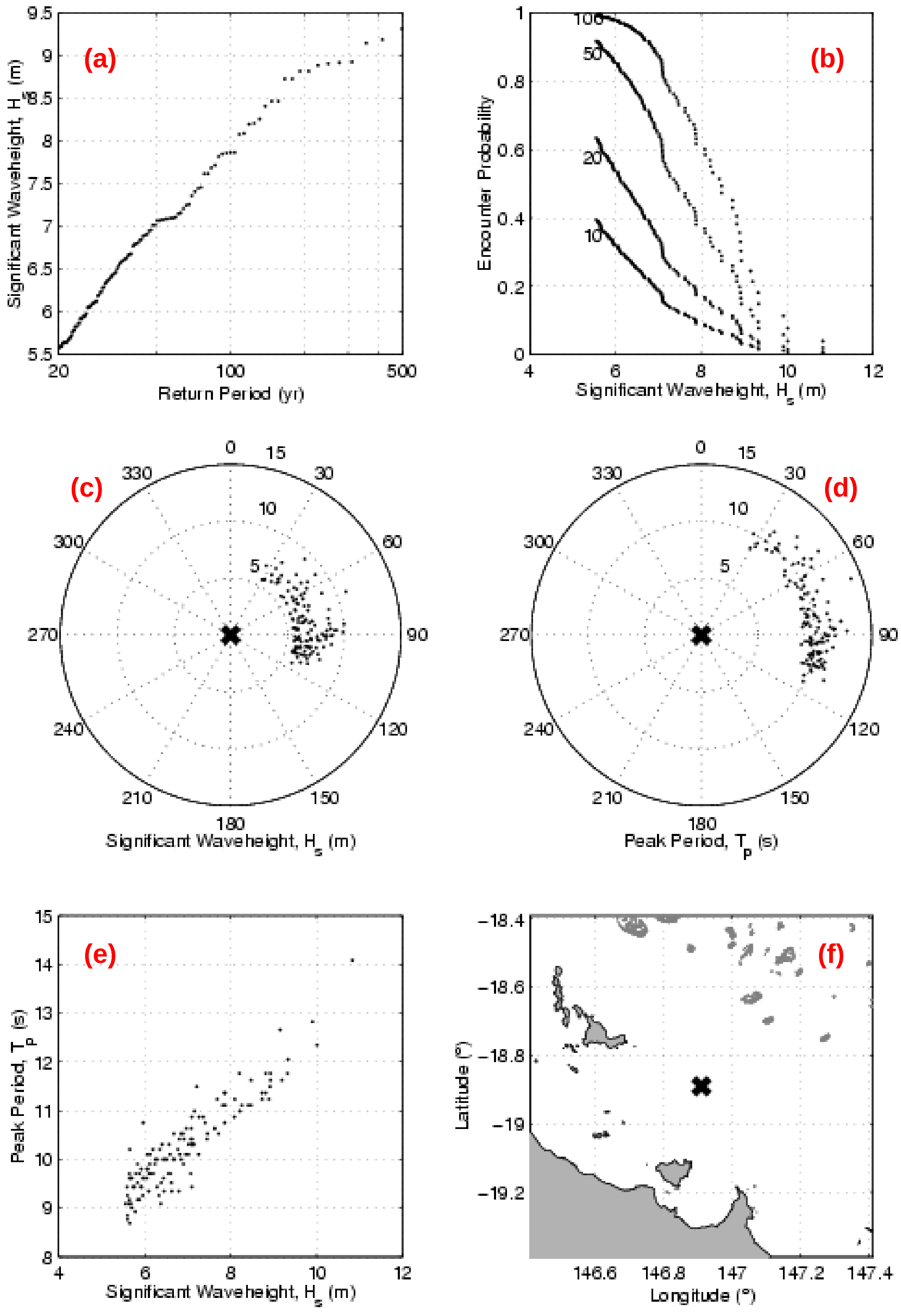


Figure 3.1 : Data Summary for Extreme Waves Offshore of Cleveland Bay

(a) return period curve; (b) encounter probability curves for 10, 20, 50 and 100 year lives;
(c) polar plot of Hs vs. θm ; (d) polar plot of Tp vs. θm ; (e) scatter plot of Hs vs. Tp; (f) location plot

Figure 3.1 (c) and Figure 3.1 (d)

These two polar plots contain the results of the simulations that produced the largest waves (ARI greater than 20 years) at the offshore site. The plots show waves coming from a range of directions, from 30° to 120° (to True North).

Figure 3.1 (e)

This part of the figure shows the relationship between significant wave height H_s and peak period T_p - with the tendency (as would be expected) for longer periods to be associated with larger waves.

Figure 3.1 (f)

This part of the figure simply shows the approximate location of the offshore site with respect to surrounding topographic features.

The largest record in the 2,500 year ensemble comes from a bearing of approximately 70° and has a H_s value of approximately 11m and a T_p of 14 seconds. The extreme wave climate in the deep waters offshore of Cleveland Bay can be summarised as follows:

Wave Parameter	50 year ARI	100 year ARI
Significant Wave Height (H_s)	7.1 metres	8.0 metres
Peak Period (T_p)	9.5 secs - 11.5 secs	10.5 secs - 12 secs
Mean Wave Direction (θ_m)	………… all between bearings of 30° to 120° …………..	

Table 3.1 : *Extreme Wave Conditions for Distant Seas Offshore of Cleveland Bay*

3.3.2 Local Seas

Local Seas are those waves generated by winds blowing across the open water fetches of Cleveland Bay itself (refer to Figure 2.4(b)). These fetches extend from the location of the Townsville Port precinct out across to Magnetic Island and towards Cape Cleveland.

3.3.2.1 Ambient Conditions

Hindcasts for waves generated by winds blowing across local Cleveland Bay fetches have been produced using standard mathematical techniques. This requires the use of directional wind data - as measured by the Bureau of Meteorology at anemometer sites. Some discussion is warranted as to the most appropriate location of these wind measurements.

The most obvious site to use for hindcasting waves across Cleveland Bay would at first appear to be the Bureau's weather station at Townsville Airport since it has long, quite complete records from an AWS (Automatic Weather Station). However, this anemometer location has a number of limitations with regard to using recorded winds for wave hindcasting. These relate to the proximity of significant topographical features - such as Many Peaks Range (adjacent to Cape Pallarenda), Magnetic Island and Castle Hill.

Many Peaks Range (and Mt. Marlow in particular) deflects those winds blowing from the north and north-west, consequently the actual wind fields across Cleveland Bay from these directions are not correctly represented by the data recorded by the airport anemometer. Similarly Castle Hill distorts the wind field from the predominant eastern sector.
As there is no reliable and systematic way of "correcting" the data for these shielding effects it is not considered appropriate to utilise the recorded winds from Townsville Airport for hindcasting purposes.

The Bureau's weather station located at Cape Cleveland is perhaps the next most obvious alternative, however data collection ceased in 1987. More significantly however, the station was located such that it experienced a wind amplification effect. This phenomena increased the wind speed from some directions by as much as a factor of two when compared to the wind velocity over open water. Consequently, the data from this anemometer was not used for wave hindcasting purposes.

The wind data measured by the Townsville Port Authority has previously been investigated regarding it's suitability for local wave hindcasting (Coastal Engineering Solutions, 1998) and found to not adequately represent the winds that blow over most of the wave generating areas in Cleveland Bay. It appears that nearby port structures or possibly Castle Hill are causing local disturbances to the wind fields. Other Port Authority wind records taken by anemometer stations on beacons marking Platypus Channel are inappropriate for wave hindcasting in Cleveland Bay because Magnetic Island shields the instrument locations from the important winds from the north to north-east.

The wind data selected as being most representative of the conditions for wave hindcasting across Cleveland Bay fetches was that recorded at the Bureau's Lucinda AWS station.

The Lucinda instrument is located at the offshore end of the 5.76km long sugar loading jetty; at a height of 10 metres above the jetty's deck. No significant shielding occurs from any of the offshore fetches. It is acknowledged that the location is some 90kms north-west of Townsville, however the shoreline is similarly orientated and a reasonable representation of sea breeze effects is expected. The scale of the synoptic systems that generate the winds that arrive at the Lucinda site and at Cleveland Bay is such that the wind fields are expected to be similar at both coastal locations.

These recorded winds were used to design The Strand beach system (Coastal Engineering Solutions, 1998) using mathematical modelling of waves and coastal processes and the results were verified by beach performance.

The fetches across which Local Seas are generated are presented in Table 3.2.

The mathematical techniques of *Sverdrup - Monk - Bretschneider* (SMB) as presented in the 1977 edition of the Shore Protection Manual (CERC, 1977) have been applied to the recorded winds to determine the wave heights resulting from winds blowing across the open water fetches in the region.

It is universally acknowledged that the empirical steady-state wave prediction methods given in the subsequent 1984 edition of the Shore Protection Manual (CERC, 1984) which uses an adjusted wind speed factor based on friction velocity tends to over-predict wave height. Nevertheless, the SMB techniques presented in this later edition are appropriate for determining wave period, and have therefore been applied for that purpose.

Because the Local Seas generated by ambient winds can occur in conjunction with Distant Seas, the wind speeds and directions recorded three-hourly by the anemometer at Lucinda over the same timeframe as the Waverider measurements of Distant Sea have been used to establish a wave database over the this same four year timeframe.

Some 10,730 sequential wave records (each consisting of the offshore wave height, wave period and wave direction) therefore make up the complete time series of hindcast Local Sea wave events.

Direction (deg True North)	Length (metres)	Depth (m below CD)
330	16,000	5.0
340	16,000	5.0
350	8,500	4.0
360	7,500	4.0
10	6,500	4.0
20	9,500	4.0
30	25,000	6.0
40	25,000	10.0
50	27,000	10.0
60	30,000	7.0
70	25,000	7.0
80	21,000	5.0
90	19,000	3.5
100	18,000	2.5
110	12,000	2.0

Table 3.2 : Fetch Characteristics Used to Hindcast Local Sea Waves

3.3.2.2 Extreme / Cyclone Conditions

When determining the extreme wave conditions associated with Local Seas, a similar approach to that of hindcasting ambient waves has been adopted. However, in this case measured winds have not been used since the extent and directional resolution of the records is such that the extrapolation required to obtain the extreme 50 year and 100 year ARI winds for hindcasting purposes is considered somewhat tenuous.

An alternative method has been adopted; namely the determination of the winds associated with cyclone events having 50 year and 100 year ARI occurrences. This relies on previous studies undertaken with regard to cyclone occurrences in the Townsville region. The most relevant study currently available concerning the frequency of particular cyclone intensities is that prepared by Blain Bremner & Williams (1984). That numerical modelling study considered likely cyclone influences in the Townsville region - specifically storm surge and its occurrence in conjunction with the astronomical tide (ie. the storm tide, refer to discussions in Section 2.2.2).

In developing storm tide effects, the study determined a relationship between Average Recurrence Interval and cyclone central pressure - these being on average 938mb and 927mb for the 50 year and 100 year ARI respectively. The corresponding maximum sustained wind over a ten minute period for each such cyclone is estimated to be 42 m/sec and 45 m/sec respectively.

These 10 minute-averaged wind speeds have therefore been used to hindcast the extreme waves for Local Seas in the vicinity of the Townsville Ocean Terminal Project by applying them across each of the local fetches listed in Table 3.2.

It is important to appreciate that this technique is conservative. This is because it not only requires such 50 year and 100 year ARI cyclones to occur in the Townsville region (defined as a length of coastline 5º of latitude in length), but that they each should also occur with the very specific track which results in the cyclone's position and forward speed being such as to align the maximum winds across those Cleveland Bay fetches which produce the largest waves.

Having this additional and very specific requirement with regard to cyclone track means that the selected 50 year and 100 year ARI cyclone characteristics are in fact indicative of cyclones having greater Average Recurrence Intervals. In other words, the estimated 10 minute-averaged wind speeds of 42 m/sec and 45 m/sec across the local fetches of Cleveland Bay are over-estimates of 50 year and 100 year ARI winds and are therefore conservative. The resulting extreme waves associated with Local Seas are summarised in Table 3.3 overleaf.

Direction (deg True North)	50 Year ARI H_s (metres)	50 Year ARI T_p (secs)	100 Year ARI H_s (metres)	100 Year ARI T_p (secs)
330	1.78	6	1.86	6
340	1.78	6	1.86	6
350	1.49	5	1.56	5
360	1.48	5	1.55	5
10	1.46	5	1.54	5
20	1.50	5	1.57	5
30	2.05	7	2.14	7
40	2.83	7	2.98	7
50	2.85	7	3.00	7
60	2.29	7	2.40	7
70	2.28	7	2.38	7
80	1.80	6	1.88	6
90	1.24	6	1.29	6
100	0.92	5	0.95	5
110	0.92	5	0.95	5

Table 3.3 : Extreme Wave Conditions for Local Seas Offshore of Cleveland Bay

3.3.3 Swell Waves

Swell waves are generated by distant weather systems in the waters of the Coral Sea and the Pacific Ocean beyond the Great Barrier Reef. The techniques applied to the Wave Generation aspects of the mathematical modelling of waves for the *Townsville Ocean Terminal Project* is such that the effects of swell are inherently incorporated into those for Distant Sea.

Consequently no separate and distinct database for swell waves needs to be established. This is the case for both ambient and extreme / cyclone conditions. The following sections provide clarification on this issue.

3.3.3.1 Ambient Conditions

As discussed in Section 3.3.1.1, the ambient wave climate established for Distant Seas utilises the half-hourly records from the Waverider station off Cape Cleveland for the approximate four year period from 12th October 2000 to 28th September 2004.

Wave periods of up to 16 seconds were recorded during this timeframe, albeit with quite low wave heights associated with these long period waves. Typically waves events having periods greater than 10 seconds were less than about 0.3m in height and were recorded by the Waverider buoy as having approach directions between 40º and 60º (relative to True North).

Such long wave periods and their corresponding wave directions indicate that the Waverider buoy was also measuring swell waves that had propagated through and over the Barrier Reef. Consequently the database of offshore waves for ambient Distant Seas inherently incorporates swell waves.

3.3.3.2 Extreme / Cyclone Conditions

When determining the offshore wave characteristics associated with extreme events, the cyclone wave information in the deep waters offshore of Townsville has been extracted from the data generated for the *Atlas of Tropical Cyclone Waves in the Great Barrier Reef*. This data was prepared by the Marine Modelling Unit (MMU) of the School of Engineering at James Cook University (refer to discussions in Section 3.3.1.2).

The model regime established by the MMU incorporates waves that are generated by cyclonic influences in the waters beyond the Great Barrier Reef. Therefore these cyclone-generated waves that have been categorised as Distant Sea could also be considered as swell waves. Typically they have long periods that are commensurate with swell. Reference to Figure 3.1 indicates that wave periods are typically greater than 8 seconds. Indeed the extreme wave climate used for the mathematical modelling of Distant Sea (refer to Table 3.1) has wave periods greater than 9.5 seconds.

Consequently the database of offshore waves for extreme / cyclone conditions of Distant Seas inherently incorporates swell waves.

3.4 Nearshore Wave Climate

The preceding Section 3.3 presented a brief discussion and results of the investigations to determine ambient and extreme wave characteristics in the deeper waters offshore of Cleveland Bay. When considering how these waves are affected as they propagate shoreward, it is necessary to consider the processes of wave refraction, diffraction, seabed friction, shoaling and wave breaking.

These complex wave transformation processes are replicated by application of the various mathematical modelling techniques discussed in the following sections. When applied together, these various techniques constitute the *Wave Transformation Module* of the complete wave modelling procedure.

The following discussions relate to the determination of the wave climate in the nearshore waters around the Townsville Ocean Terminal Project. This has been done for the pre-development and post-development scenarios for two reasons, namely:

- to identify any impacts that the proposed development may have on the wave climate of adjacent foreshores; and
- to provide wave information to assist in the conceptual design of marine / foreshore infrastructure associated with the Townsville Ocean Terminal Project.

The same methodology and procedures were undertaken for the modelling of each condition. Comparisons and identification of impacts are presented and discussed in Section 4 of this report.

3.4.1 Methodology for Modelling Wave Transformation to Nearshore Areas

The program *CES350* is used to determine the wave refraction and wave shoaling effects as waves propagate shoreward from the deep waters beyond Cleveland Bay into the nearshore waters surrounding the Townsville Ocean Terminal Project.

In order to undertake the wave transformation computations, it is necessary to define the seabed bathymetry over which all such waves can propagate. An extensive computational grid arrangement has therefore been established to schematise the form of the seabed. This is achieved by assigning a depth to each intersecting point across the entire grid system.

This system actually consists of a series of adjoining grids (termed "*zones*"). The spacings within these zones can be made variable - with a fine spacing chosen to better define the seabed in areas of complex or rapidly changing bathymetry, and a wider spacing where the seabed is relatively flat and unchanging.

Figure 3.2 shows the extent of this computational grid arrangement. As discussed in the preceding Section 3.3.3, the effects of offshore swell are inherently incorporated into those for Distant Seas and no further distinction is made in this report when discussing the methodology and results of such waves.

To investigate the propagation of Distant Seas and Local Seas into nearshore waters it has been necessary to establish four computational grids - one for each combination of these two main wave types, and the pre-development and post-development scenarios.

The parallelograms shown on Figure 3.2 represent the various zones - within which different grid spacings are assigned. The grid systems established for this study cover the entire nearshore waters of Cleveland Bay from north-west of Magnetic Island, to near Cape Ferguson in Bowling Green Bay to the south-east. They extended out into the deep waters between the mainland and the Great Barrier Reef.

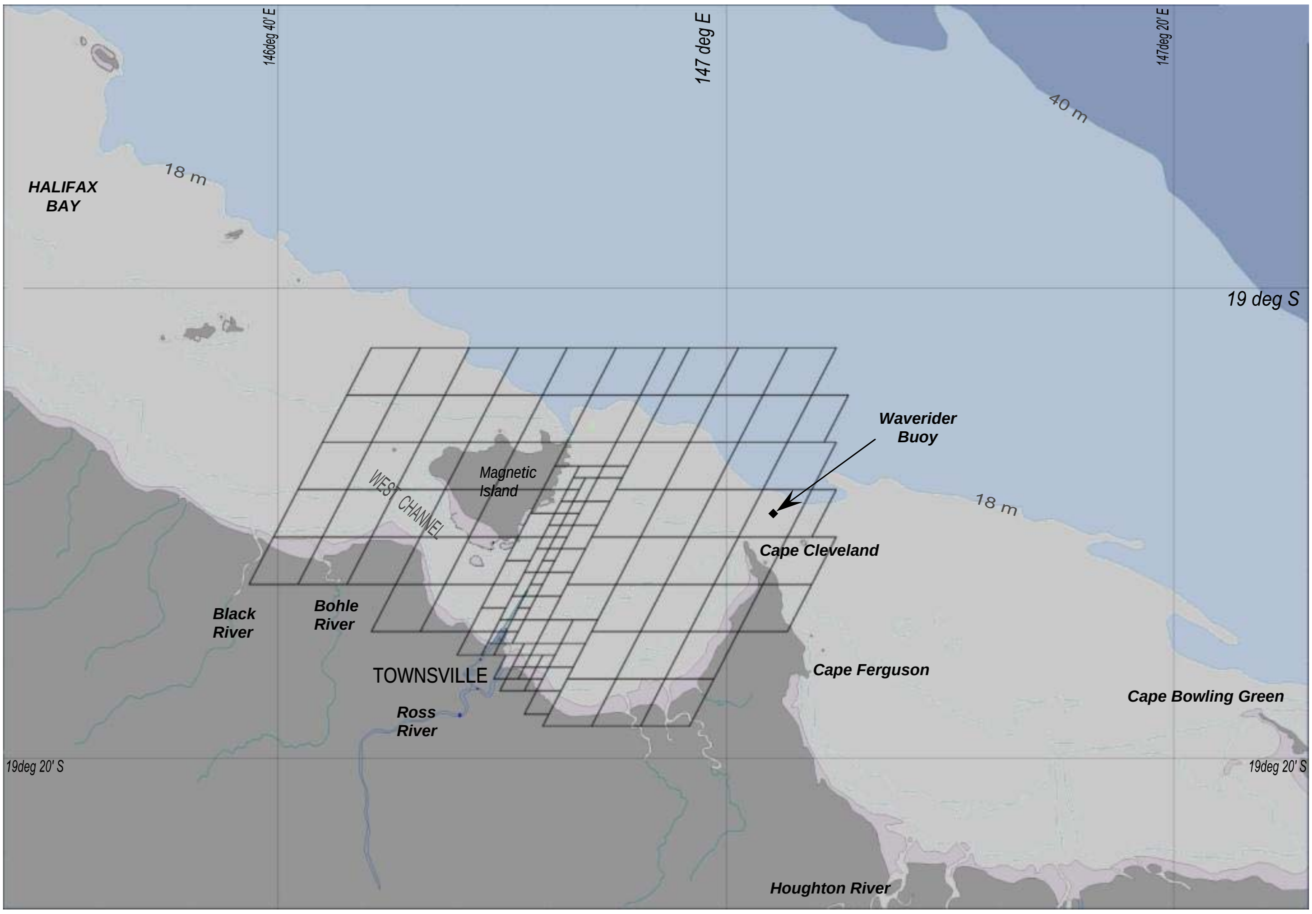


Figure 3.2 : Computational Grid for Wave Transformation Modelling

A total of 124 zones containing over 760,000 grid points were used to represent depths to the seabed across the extent of the grid system for Distant Seas. The grid utilises maximum spacings of 50 metres in areas where the seabed is relatively flat - typically this is in the deep waters east of Cleveland Bay itself. In the shallower inshore waters (and around steep sloped seabed features such as Platypus Channel), the grid spacing is reduced to 12.5 metres.

Similarly a total of 35 zones containing over 200,000 grid points has been used to represent depths across the extent of the grid system for Local Seas.

The wave transformation program CES350 is a "reverse ray refraction" algorithm. This model tracks wave orthogonals across the computational grid from selected nearshore locations out into deep water. A wave orthogonal is basically a "ray" drawn perpendicular to the alignment of the wave crests and therefore defines the approaching path of the wave. Each ray depicts the reverse path that a wave orthogonal takes between deepwater (the generation area) into each selected nearshore site.

The output from this component of the Wave Transformation Module is typically about 2,000 reverse ray orthogonals covering all likely wave periods and directions which could conceivably occur for any particular site near the shoreline - including those for swell waves that may have propagated across or through the Barrier Reef. Consequently the reverse rays from sites in the vicinity of the Townsville Ocean Terminal Project tracked out across the approaches to the port precinct; through Cleveland Bay; across the open water fetches of the north-east and the east/south-east sectors; into the deep water where the offshore wave climate was determined.

For the purposes of defining the inshore wave characteristics in the vicinity of the Townsville Ocean Terminal, a number of specific locations have been selected for investigation in nearshore waters. Those sites which are in the immediate vicinity of the project are shown on Figure 3.3. Additional sites were also chosen in the much deeper offshore areas of Cleveland Bay, so that wave refraction processes could be determined for subsequent use by other algorithms in the suite of wave transformation programs - such as for the determination of seabed friction effects.

Basically the nearshore sites selected for investigation are as follows:

- Site 01 - *opposite the northern end of The Strand coastal reach.*
- Site 02 - *opposite the central region of The Strand coastal reach.*
- Site 03 - *opposite the southern end of The Strand coastal reach.*
- Site 04 - *offshore of the northern end of the new Northern Breakwater.*
- Site 05 - *offshore of the central portion of the new Northern Breakwater.*
- Site 06 - *at the centre of the harbour entrance into the Townsville Port.*
- Site 07 - *at the centre of the entrance into the new Breakwater Cove Marina & waterways.*
- Site 08 - *at the offshore end of the new entrance channel to the Breakwater Cove marina.*

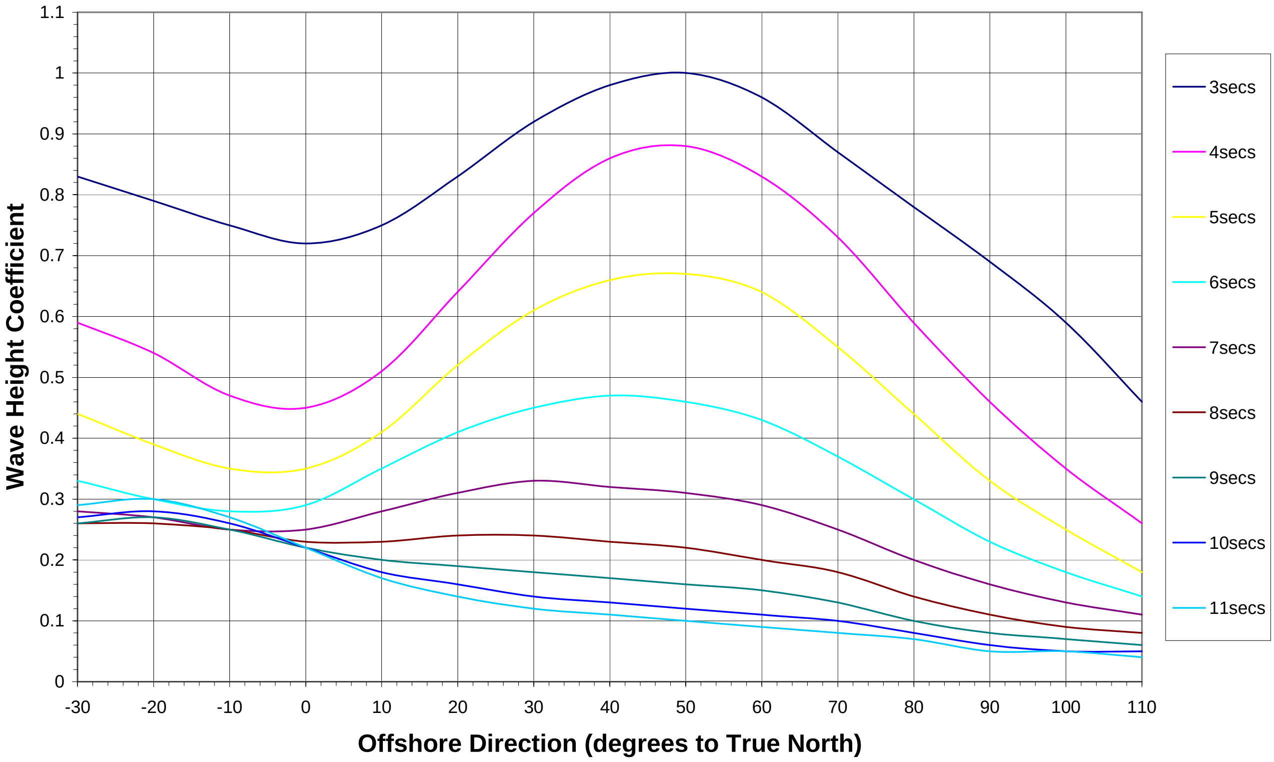
The Wave Transformation Module uses a computational routine to determine the wave coefficients (ie. the ratio of inshore to offshore wave heights), and the relationship between the direction of the waves out in deep water and the corresponding wave directions inshore.

Figure 3.3 : Locations of Inshore Sites Used for Wave Transformation Modelling

These calculations are completed on a spectral basis for the complete range of possible offshore wave directions and periods; and for each of the nearshore sites listed above. Different ocean water levels are also investigated for these various scenarios, so as to account for the variability in wave transformation which occurs during the astronomical tidal cycle and during storm tides.

As well as including the effects of wave refraction, the resulting wave coefficients also account for wave shoaling - but not seabed friction or wave breaking, as these more complex processes are replicated by subsequent modelling routines where required.

Figure 3.4 shows a typical example of the output from this phase of the wave transformation modelling. Wave coefficients and inshore wave directions are shown in this figure for Distant Seas at a location just offshore of the main breakwater (ie. Site 04) and for an ocean water level at the 100 year ARI storm tide level. The results are for the post-development scenario.



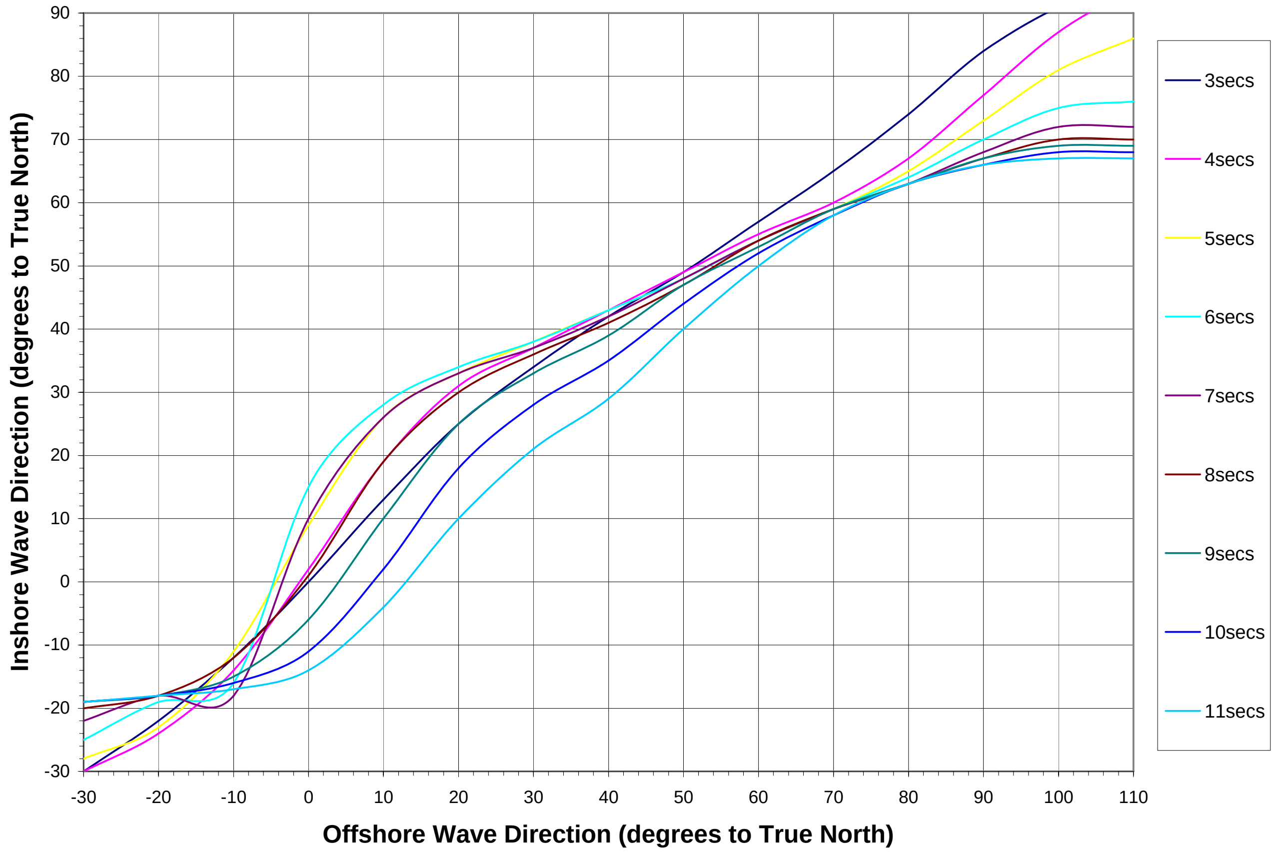


Figure 3.4 : Wave Height Coefficients and Inshore Wave Directions for SITE 04

As an example of how to interpret Figure 3.4 the results presented in, for an offshore wave of 1.0 metres height having a period of 6 seconds and approaching Cleveland Bay from a bearing of 040°, the corresponding wave height inshore at Site 04 is 0.47 metres. In other words, the wave height has been reduced by a factor of 0.47.

Reference to the lower half of the figure indicates that as a consequence of the transformation processes, the same 6 second wave will be arriving offshore of the main breakwater at Site 04 on a bearing of approximately 043°.

Similar relationships between the offshore Distant and Local Sea waves at all of the nearshore sites, at various ocean water levels, and for both existing and post-development scenarios have been determined by the program CES350.

Seabed friction effects play only a very minor role in the attenuation of waves as they propagate through Cleveland Bay onto local foreshores. In general terms, the "roughness" of a seabed with respect to its effect on waves as they pass over it depends on whether or not there are ripples present.

The roughness assigned to a rippled bed by the wave transformation modelling is determined to be 4 times the calculated ripple height (Hsiao & Shemdin, 1978). However, where calculations indicate that the wave height is such that ripples do not occur, then the roughness assigned to the sea bed is 4 times the median diameter of the seabed material (Riedel, 1972). Therefore the roughness of a rippled seabed tends to be several orders of magnitude greater than that for a flat sandy bed. This can have a significant influence on wave attenuation.

The seabed sediments throughout Cleveland Bay are very fine, consequently any ripples that do form on the seabed were found by the modelling to be readily "wiped out" to form a flat sandy bed - even under quite moderate wave conditions. Certainly during cyclone events, the large long period waves cause the seabed to become a relatively flat featureless plane bed, with negligible seabed ripples. The resulting flat seabed does not significantly attenuate wave energy. Consequently, seabed friction does not significantly attenuate wave energy propagating through Cleveland Bay approaches to the Townsville Ocean Terminal Project.

In summary then, the wave transformation modelling for ambient conditions basically assembles the wave hindcasts for Distant and Local Seas (undertaken in deep offshore waters) then applies the effects of wave refraction / shoaling and breaking to determine the corresponding ambient wave climates at the selected nearshore sites.

This has been done for two reasons:

- to provide wave information to assist in the design of marine / foreshore infrastructure - and therefore has typically been undertaken for the post-development scenario only; and
- to identify any impacts that the proposed development may have on the wave climate of adjacent foreshores. The wave transformation modelling has therefore been undertaken for the pre-development and post-development scenarios, and the results compared. This aspect is discussed in more detail in Section 4.

3.4.2 Ambient Conditions

The ambient wave climate in the Townsville region consists of Distant Seas and Local Seas, which can co-exist at any particular time.

Distant Seas

As discussed in the preceding Section 3.3.1.1, the Distant Sea hindcast database for modelling ambient wave conditions consists of a time series of offshore wave conditions over the approximate four year period from 12th October 2000 to 28th September 2004. These were recorded by the EPA's directional Waverider buoy off Cape Cleveland at half-hourly intervals throughout that four year period. The offshore wave conditions for each of these 53,777 Waverider records is modified by the Wave Transformation Model in accordance to the relationships determined by the reverse ray modelling. This has been undertaken for various nearshore sites under investigation.

The outputs for each nearshore site consist of wave data files for Distant Seas, each containing the four year long wave data set (of 53,777 records).

Local Seas

As discussed in the preceding Section 3.3.2.1, the Local Sea hindcast database for modelling ambient wave conditions also consists of a time series of offshore wave conditions over the same period from 12th October 2000 to 28th September 2004 as that determined for the Distant Sea database. These Local Seas however are at three-hourly intervals - calculated by standard wave hindcasting techniques using local fetch characteristics and measured wind data.

The offshore wave conditions for each of these 10,730 sequential records have been modified by the Wave Transformation Model in accordance to the relationships determined by the reverse ray modelling. This has been undertaken for various nearshore sites under investigation.

The outputs for each nearshore site therefore consist of wave data files for Local Sea, each containing the approximate four year long wave data set (of 10,730 records).

By way of example, Figure 3.5 presents a summary of results that can be obtained from an analysis of the datasets. The figure shows the frequency of occurrence of significant wave heights generated by Distant and Local Seas for Site 07 (this site is at the entrance to the new marina and waterways of the Breakwater Cove precinct of the Townsville Ocean Terminal Project). It relates to waves arriving at the site from all possible directions.

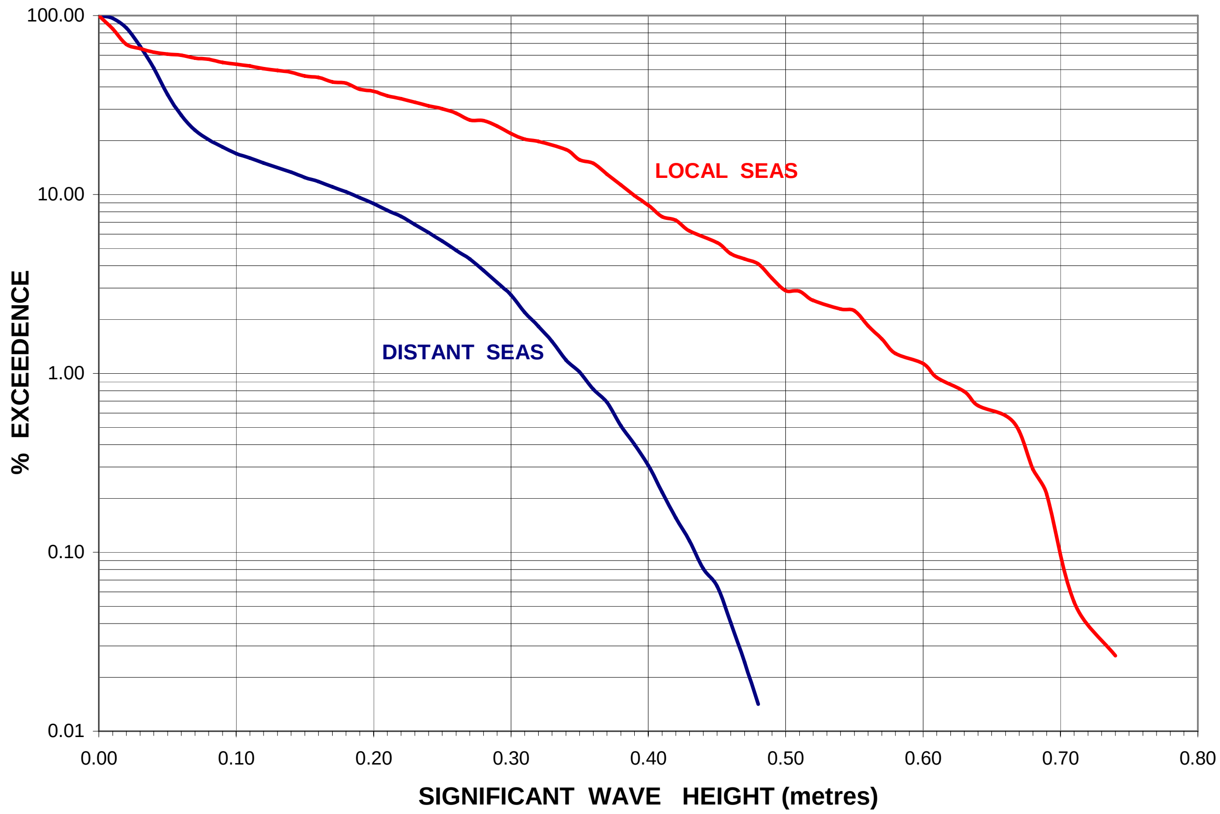


Figure 3.5 : Significant Wave Height H_s (*in metres*) occurrence at SITE 07

It can be clearly seen that whilst the wave climate is quite mild (with H_s values only ever reaching or exceeding 0.6m around 1% of the time), it is the Local Seas which dominate at this particular site. This is to be expected given that the site is at the centre of the proposed entrance into the Breakwater Cove marina which because of its orientation is sheltered from Distant Seas, but faces out across local fetches towards Magnetic Island and is therefore susceptible to Local Seas.

3.4.3 Extreme / Cyclone Conditions

The extreme wave climate in the Townsville region also consists of waves generated as either Distant Seas or Local Seas. These large waves typically occur as a consequence of an extreme synoptic event such as a cyclone. Therefore, unlike ambient conditions, extreme wave conditions will occur as either Distant Seas or as Local Seas - their most adverse wave conditions will not co-exist. This is because the maximum winds associated with the particular cyclone event will be such that they will affect open water fetches that result in either Distant Seas or Local Seas being the dominant wave source at any particular time.

Discussions in the preceding Sections 3.3.1.2 and 3.3.2.2 (for Distant and Local Seas respectively) summarised the findings of the investigations into the cyclone waves which could occur in the deep waters offshore of Cleveland Bay. The relationship between these offshore waves and those in nearshore waters has been determined by the Wave Transformation Model for various storm tide levels in a similar manner as that for the ambient wave climate.

When these relationships are considered in association with the deep water extreme wave climate, the corresponding cyclone waves can be determined at appropriate nearshore sites. These then become the wave characteristics used for the conceptual design of structures in their immediate vicinity.

As discussed in 3.2 the approach adopted when developing structural concepts for the various perimeter seawalls and breakwaters of the Townsville Ocean Terminal Project has been to consider the following scenarios as potentially constituting the 100 year ARI Designated Storm Tide Event, and to then select the one having the most adverse effect on structural performance:

- Scenario 1 : 100 year ARI storm tide level occurring simultaneously with the 50 year ARI wave characteristics; or
- Scenario 2 : 50 year ARI storm tide level occurring simultaneously with the 100 year ARI wave characteristics.

Table 3.4 presents the extreme wave characteristics at locations around the perimeter of the Townsville Ocean Terminal site. They represent the DSTE and are therefore also the design wave conditions for the project elements indicated in the table.

Structure	*Design Parameter*	*Scenario 1*	*Scenario 2*
Northern Breakwater	Significant Wave Height : H_s	2.35 metres	1.84 metres
	Peak Period : T_p	10 to 11 secs	11 to 12secs
	Storm Tide Level (to AHD)	RL+2.91m	RL+2.65m
Strand Breakwater	Significant Wave Height : H_s	1.7 metres	1.8 metres
	Peak Period : T_p	6 secs	6 secs
	Storm Tide Level (to AHD)	RL+2.91m	RL+2.65m

Table 3.4 : 100 year ARI Design Wave Conditions for Project Infrastructure

Note : The two scenarios potentially represent the DSTE - during conceptual design of the above structures, the most adverse loading and overtopping condition will be adopted (refer Section 6).

The storm tide levels reported in Table 3.4 include the expected Greenhouse-induced sea level rise - but since they have been used to assess wave transformation through deep water approaches to the site, exclude the allowance for breaking wave setup and wave runup on the structures. These later aspects are considered during the structural assessment of each specific structure.

4 IMPACTS OF PROPOSED DEVELOPMENT ON WAVE CLIMATE

4.1 The Strand Beaches

In order to identify any impacts of the proposed development on the ambient wave climate along The Strand beaches, the wave transformation modelling was undertaken for the three nearshore sites 01, 02 and 03 - representing the northern, central and southern sections of The Strand foreshore. Figure 4.1 shows the location of these three sites. Both the existing and post-development scenarios were modeled at each location for comparison.

Figure 4.1 : Location of Nearshore Investigation Sites Along The Strand

Given the reasonably flat and shallow seabed approach slopes, the effect that the prevailing tide level has on wave transformation to The Strand foreshore can be significant, particularly since the tide range in the study area is of the order of 2.3 metres during spring tides and up to a maximum of approximately 4.0 metres. Consequently wave transformation modelling for existing and post-development was undertaken at three tide levels, namely; at an "average" high tide of RL+0.85m AHD; at approximately Mean Sea Level of RL0.0m AHD; and at an "average" low tide of RL-0.8m AHD.

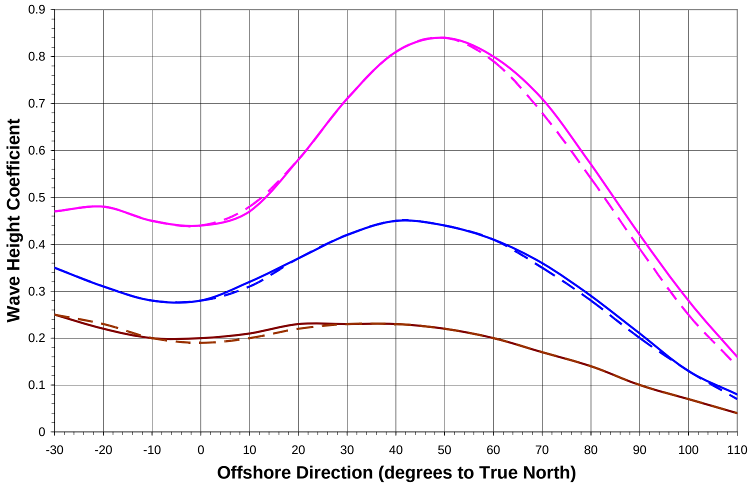
Figure 4.2 shows a comparison of existing and post-development conditions at locations along The Strand beaches for several wave periods. Figure 4.3 shows the comparison of inshore wave directions.

The results indicate that the substantial reclamations associated with the Breakwater Casino and the Townsville Port that currently exist to the immediate south of The Strand foreshore provide varying protection to The Strand Beaches.

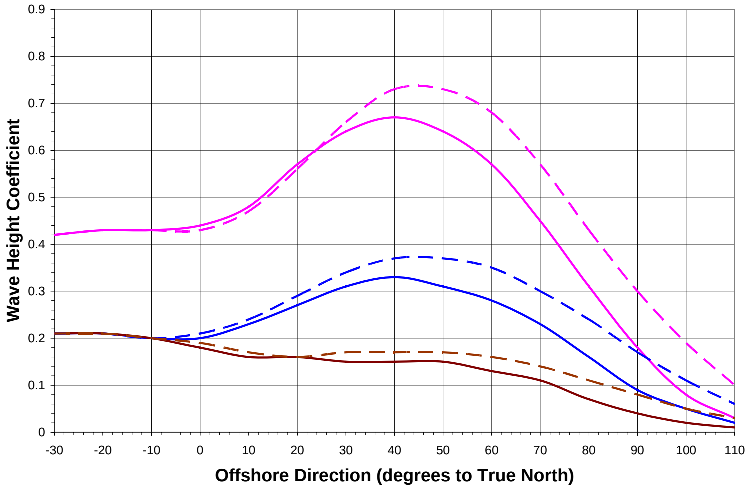
In particular, the 635m long existing Northern Breakwater that extends to the west from the Port entrance (and which will form part of the new Northern Breakwater for the Townsville Ocean Terminal Project) creates a "wave shadow" on The Strand foreshore. The wave shadow is an area of reduced wave energy. This existing breakwater therefore strongly influences the wave climate on the sandy beaches of The Strand. It consequently also affects the performance of these beaches since it is the prevailing waves which shape this foreshore.

Given that the proposed Townsville Ocean Terminal Project will be extending the existing Northern Breakwater by some 225m in a west-north-west direction, the wave shadow from the lengthened structure will reach further along The Strand foreshore than it does currently. This will have an effect on the beaches. The proposed dredging of an access channel and turning area into the new Breakwater Cove waterways will also affect the way in which incoming waves will sweep around this lengthened structure.

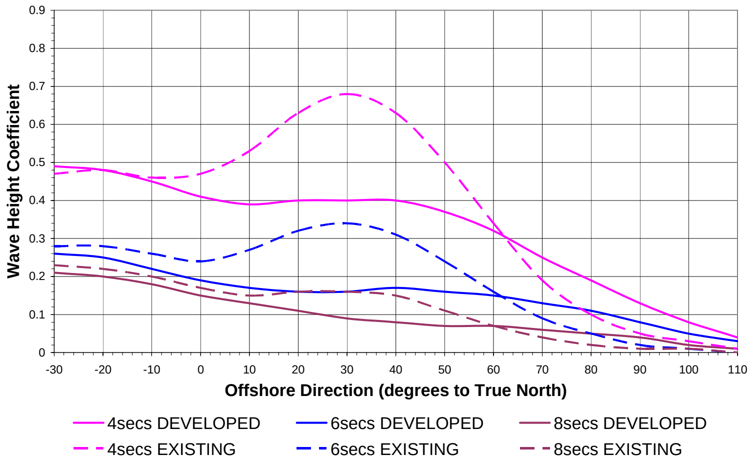
Under existing conditions, the wave shadow significantly affects the foreshore south of about the Gregory Street headland. Further northward toward Kissing Point, the effect diminishes gradually. However because the existing Northern Breakwater is to be lengthened to form the new Northern Breakwater, the wave shadow on The Strand Beaches will extend further north. The location at which its effects will gradually diminish migrates northward to a location around the Burke Street headland.



(a) SITE 01 - Northern End

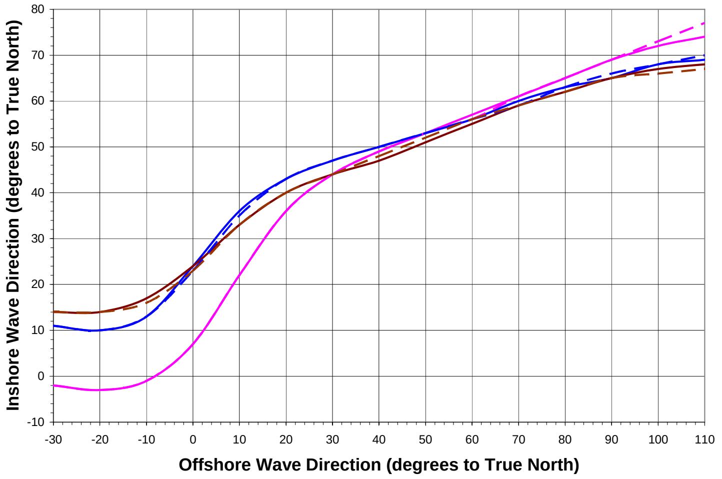


(b) SITE 02 - Central Section

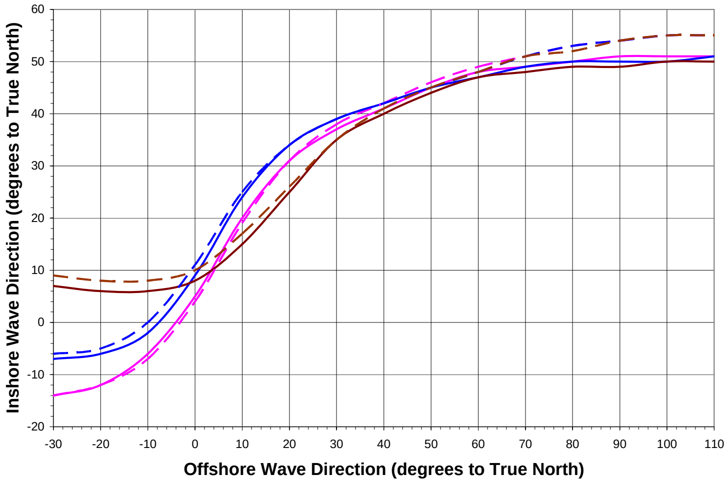


(c) SITE 03 - Southern End

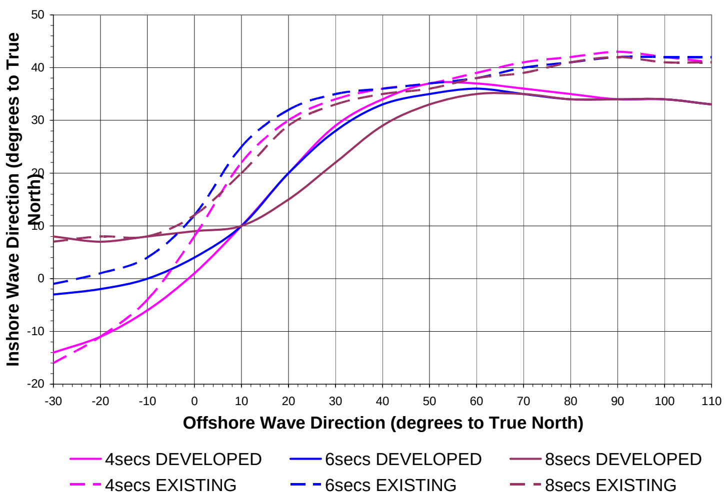
Figure 4.2 : Comparison of Wave Coefficients for Distant Sea on The Strand Beaches



(a) SITE 01 - Northern End



(b) SITE 02 - Central Section



(c) SITE 03 - Southern End

Figure 4.3 : Comparison of Inshore Wave Directions (Distant Sea) on The Strand Beaches

In addition to the above, the results of the wave modelling for pre- and post-development conditions indicate that the incoming waves within the wave shadow south of around Burke Street headland tend to be refracted such that they arrive at The Strand foreshore on a more northerly approach. This is will have implications to the orientation of the beaches along these southern reaches, with a subtle change in the plan alignment of the foreshore in response to the changed wave directions. This aspect will be discussed further in Section 5.

There are negligible changes to the height and direction of waves arriving at the northern end of The Strand Beaches as a consequence of the proposed development.

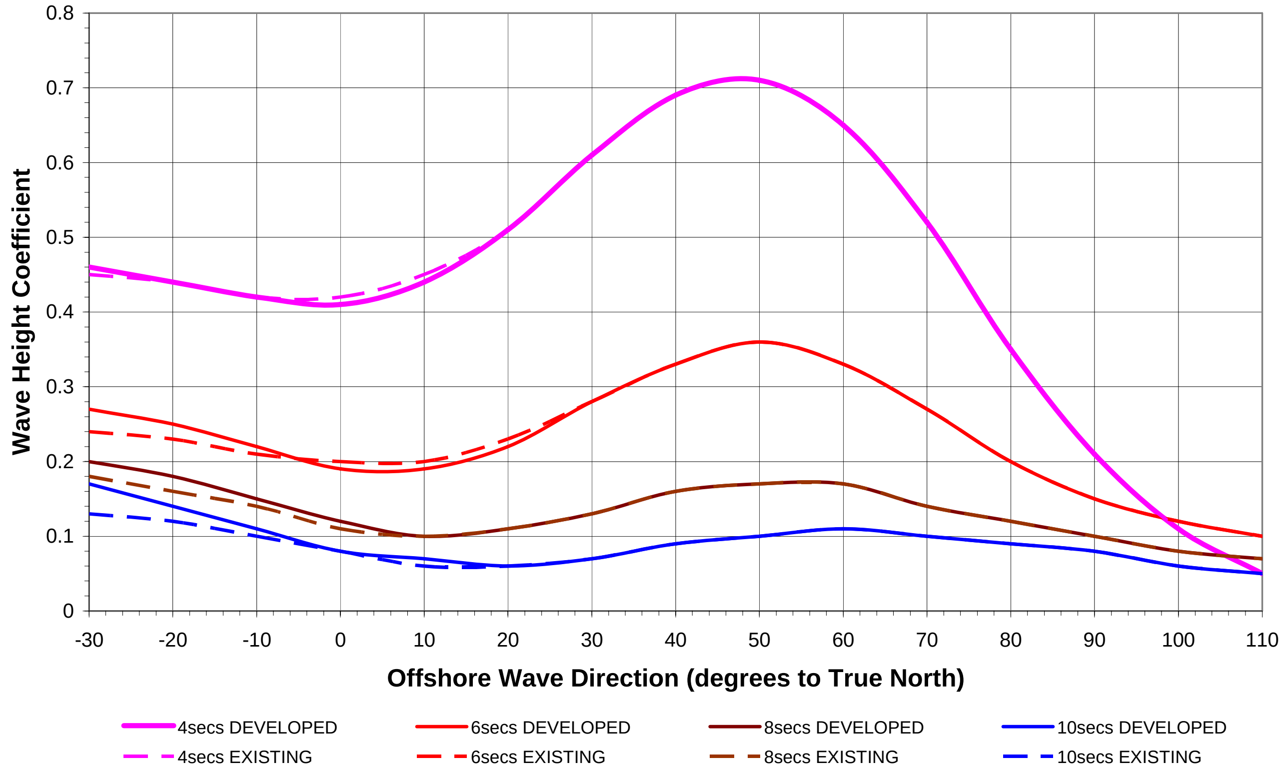
4.2 Existing Townsville Port

Given that the main entrance into the main Townsville Port harbour is in the vicinity of the proposed dredged entrance channel into the Breakwater Cove precinct, there is the possibility of altered wave conditions at the Port entrance. This was therefore investigated by consideration of the wave coefficients and inshore wave directions at a location in the centre of the Port entrance (nominated as Site 6 - refer Figure 3.3). Both Distant and Local Sea waves were considered for the pre-development and the post development scenarios. The results are shown summarised on Figure 4.4 and Figure 4.5 for a range of wave periods.

These various figures show that there is negligible change to the height and direction of incoming Distant Seas and Local Seas at the existing Townsville Port entrance as a consequence of the proposed development.

To further appreciate the possible impacts on the wave climate at the entrance to the Port, the frequency of occurrence of wave heights for both the pre-development and the post-development conditions has been determined. The results are summarised on Figure 4.6 and indicate no appreciable change at the port entrance for Distant Seas. The greatest impact from Local Seas being less than a 0.1metre increase in wave height at the 0.02% exceedance level. This represents negligible change to the wave energy at the port entrance as a consequence of the proposed lengthening of the Northern Breakwater and the dredging of the access channel into the Breakwater Cove precinct.

The proposed dredging of the berth for the Cruise Ship Terminal is within the confines of the main harbour of Townsville Port and will not affect the wave climate outside of the harbour, nor will there be any adverse impact within the harbour itself.



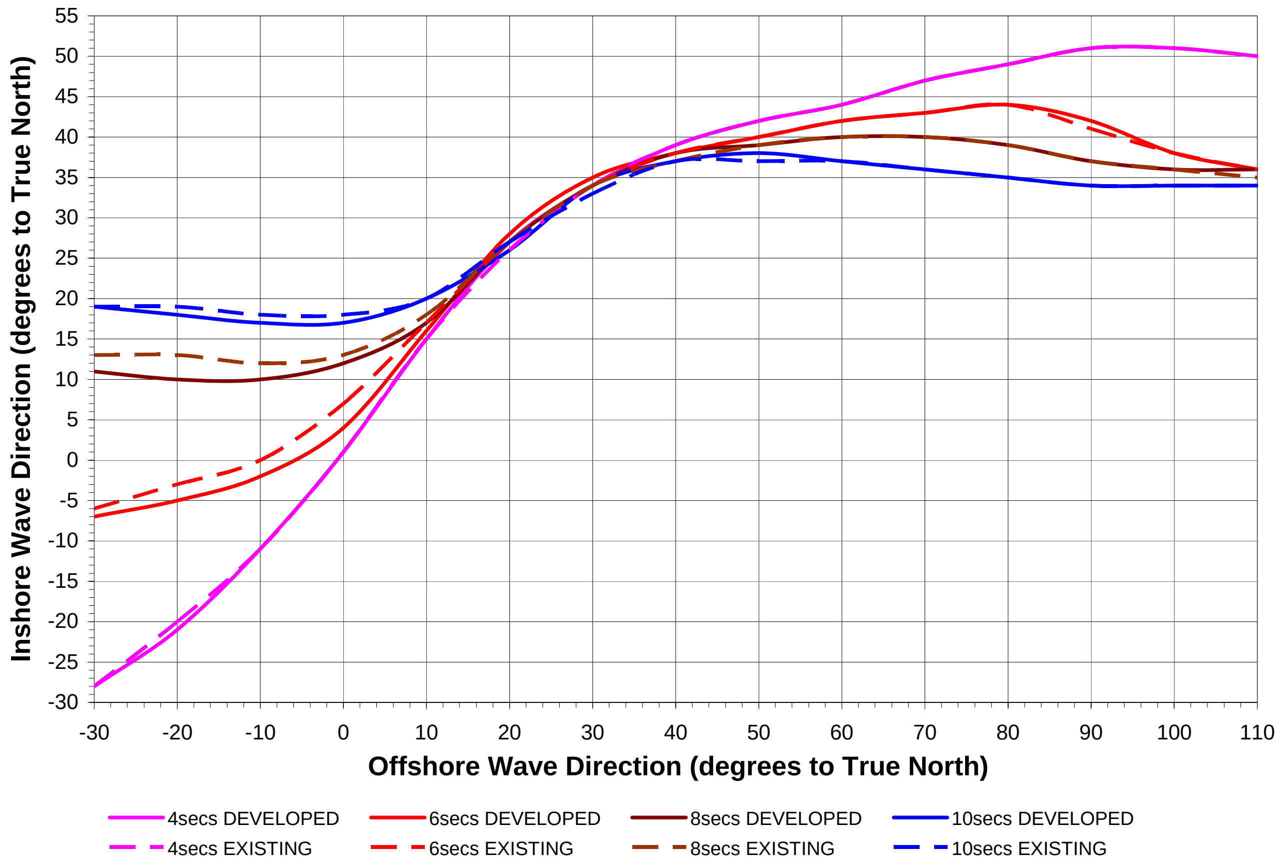
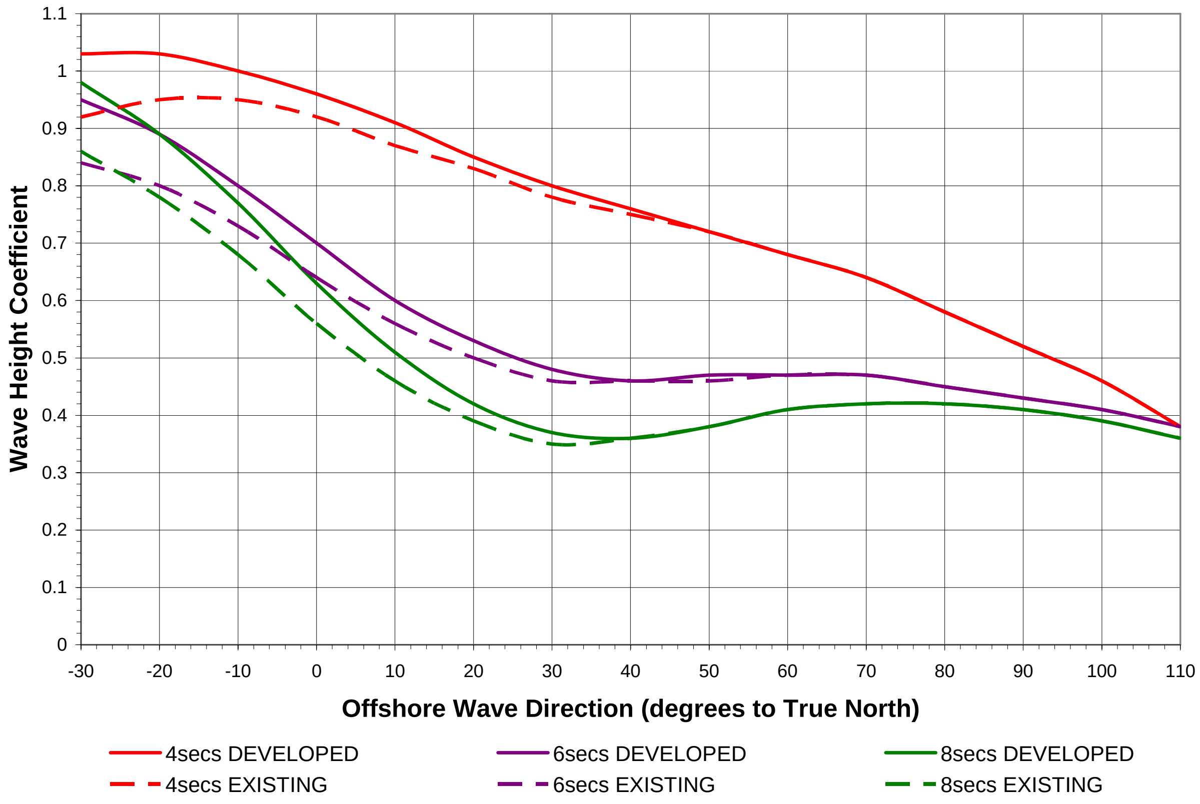


Figure 4.4 : Refraction Coefficients (Distant Sea) at Townsville Port Entrance



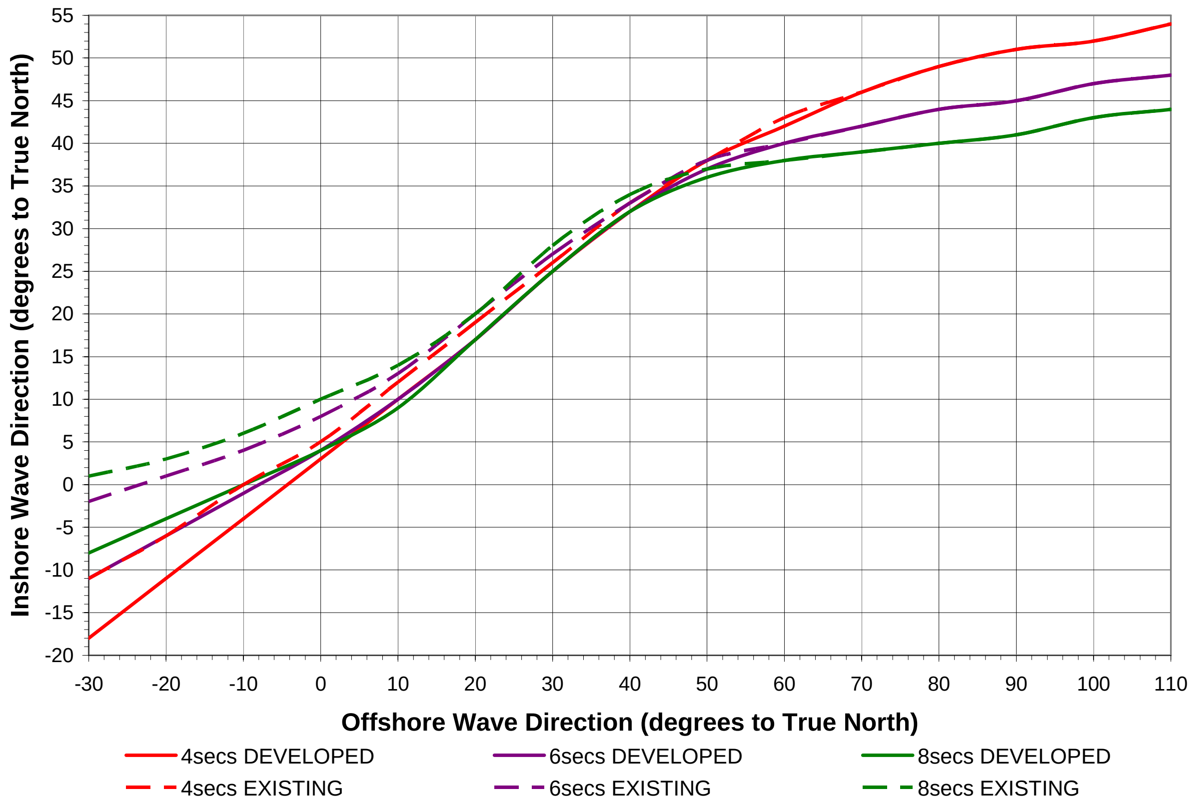


Figure 4.5 : Refraction Coefficients (Local Sea) at Townsville Port Entrance

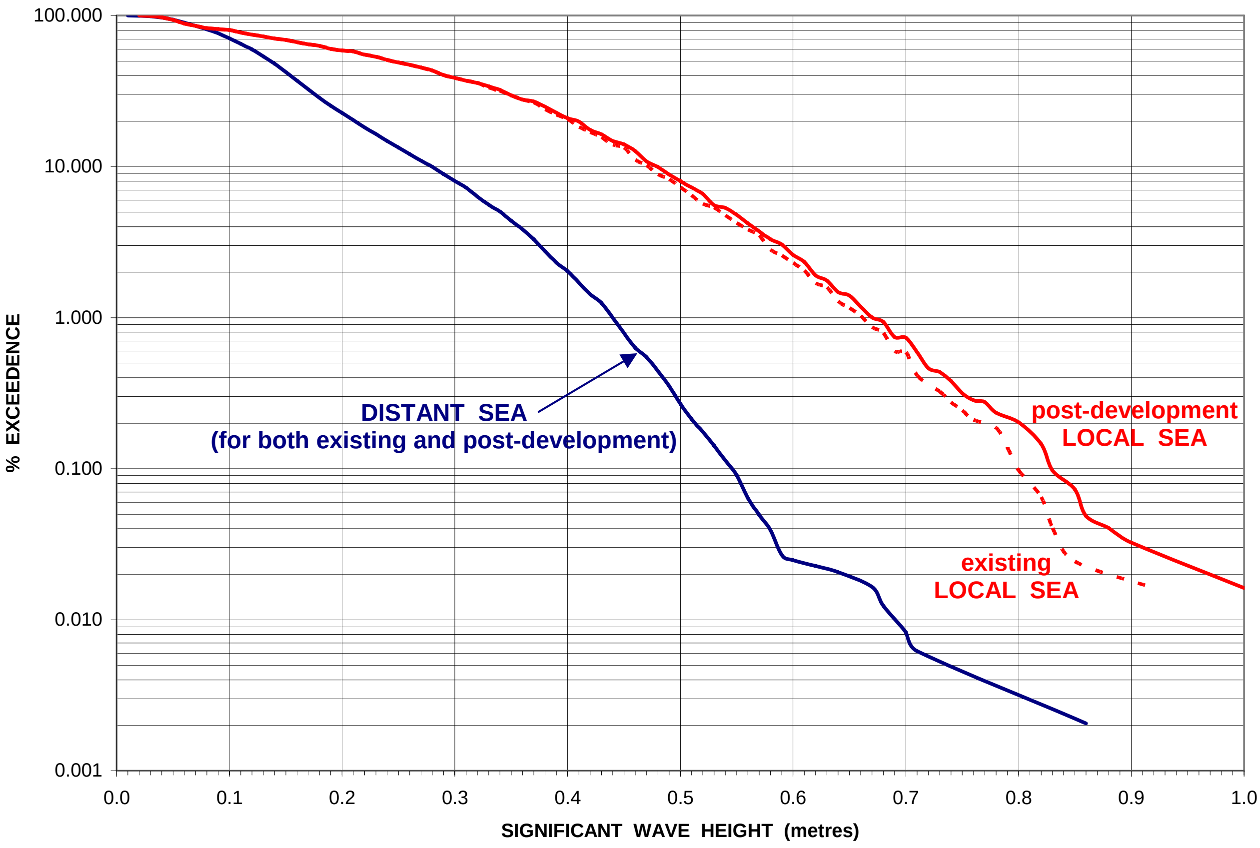


Figure 4.6 : Significant Wave Height H_s occurrence at Townsville Port Entrance

4.3 Summary of Impacts on Local Wave Climate

In summary, the proposed Townsville Ocean Terminal Project will have the following impacts to the local wave climate:

- The southern reaches of The Strand foreshore will experience lower waves and therefore have a more sheltered wave energy environment than at present. This effect will diminish further northward along these beaches - with negligible impact at their northern end.

- Whilst the incoming waves on the central and southern reaches will be less, they will tend to arrive with a slightly more northerly alignment, resulting in realignment of local beaches. This subtle readjustment of the beach will likely manifest itself as erosion of sand from one area and deposition in another. This aspect has been investigated and is discussed in the following Section 5 of this report.

* The sheltering of waves experienced along the central and southern sections of The Strand foreshore is caused primarily by the new Northern Breakwater being longer than the existing breakwater, with its offshore end being some 225 metres further to the west-north-west.

* The wave energy currently arriving at the existing entrance into the Townsville Port is not significantly altered by the proposed development.

5 MATHEMATICAL MODELLING OF SEDIMENT TRANSPORT

5.1 Methodology

Waves arriving with their crests at an angle to the plan alignment of the shoreline create an alongshore current which initiates and maintains sand transport along sandy foreshores, such as those adjacent to the proposed Townsville Ocean Terminal Project.

It is important when designing any nearshore works to identify and predict any induced changes to these processes. The various orientations of adjacent foreshores are in response to the prevailing wave climate, subtle shifts in the approach angle of the waves can result in changes to the alignment of adjacent beaches - which can result in local erosion, typically accompanied by accretion elsewhere. Given that the foreshores immediately adjacent to the proposed Townsville Ocean Terminal Project are the sandy beaches of The Strand, it is important to understand the implications of any changes to these beaches.

The central component of Coastal Engineering Solutions' Sediment Transport Module is the QUEENSED mathematical model for longshore sediment transport. This model uses the algorithm developed at Queens University in Canada – ie. the "*QUEENS formula*". It is used to assess the longshore sediment transport potential, the equilibrium beach-plan alignments and the seasonal rotations of beach-plan alignments along particular coastal reaches.

Longshore sediment transport rates are computed for local foreshores as a time series, over the entire four years of the ambient wave database - for both Distant and Local Seas (refer Section 3.4.2).

QUEENSED uses the output from the Wave Transformation Module (namely the inshore directional wave climate) to determine sediment transport rates. Nevertheless, there is a further wave transformation computation which is required to be undertaken as part of the calculations for longshore sediment transport. Typically the nearshore sites are selected some distance offshore of the beach and seaward of the breaking wave zone.

However it is the characteristics of the breaking wave that must be used in the longshore sediment transport calculations - not those waves that are seaward of the breaking zone.

Therefore the QUEENSED program determines the propagation of waves between each of the nearshore sites chosen for investigation by the Wave Transformation Module to the point where wave breaking actually occurs. The characteristics of height, period and orientation (with respect to the plan alignment of the beach) of this breaking wave are then adopted by QUEENSED for longshore sediment transport calculations.

The QUEENS formula for determining longshore sediment transport rates includes several parameters which define the physical characteristics of the nearshore sediments as well as the wave conditions prevailing at the time. Some parameters have a greater effect on longshore transport rates than others. For instance the transport rate is strongly dependent on the incident wave period; the wave height; the average sand grain size; the slope of the seabed approach onto the beach and the orientation at which the wave breaks relative to the seabed contours. The other parameters in the formula have a lesser influence on sediment transport rates.

The QUEENS formula is as follows:

$$S = \frac{1.3\text{x}10^{-3}}{(1-p)\rho_s}\frac{\rho H_b^3}{T}\left(\frac{H_b}{L_o}\right)^{-1.25}\tan^{0.75}(\alpha)\left(\frac{H_b}{D_{50}}\right)^{0.25}\sin^{0.6}2\phi_b$$

in which:

- S = longshore sediment transport *(m^3/s)*
- ρ = density of sea water *(kg/m^3)*
- ρ_s = density of the sand *(kg/m^3)*
- p = porosity of the sediment
- H_b = significant wave height at breaking point *(metres)*
- T = peak period of the spectrum *(sec)*
- D_{50} = median sand grain size *(metres)*
- ϕ_b = wave angle at breaking to the seabed contours
- α = beach slope, and
- L_0 = deep water wave length *(metres)*

The sediment transport output can be presented by the QUEENSED program in a number of ways. The basic raw output consists of a lengthy data file for each site and for each scenario, where for every time step the direction and quantity of sand moved by waves is presented. For Distant Seas the time step is half-hourly over the four year long database, and three-hourly for the Local Seas. This raw output data can then be assembled to produce either the longshore sediment transport which occurred specifically during a storm event, or on a monthly, or on an annual basis.

The model is also used to compute the extent of likely seasonal rotations within contained beach compartments/reaches on The Strand - for both existing and post-development conditions. It is also used to determine the equilibrium beach angle – that is the local plan-alignment of the beach for which there would be no net longshore transport. This later parameter is vital when determining the influence of the proposed works on stable plan alignment of these adjacent foreshores.

5.2 Verification of the Model

In order to verify the predictions of the model, the Wave Transformation and Sediment Transport modules were applied to three actual beaches within The Strand coastal reach. The model predictions of the preferred alignment (ie. that with zero net longshore transport) for each location were then compared to the actual alignments.

The locations selected are shown on Figure 5.1, Figure 5.2 and Figure 5.3, these being the beach alignments respectively within the following compartments:

- Site 1 - between the headland at Stuart Street and Kissing Point;
- Site 2 - between the headlands at Burke and Gregory Streets; and
- Site 3 - between the headland at Gregory Street and the marina.

Also shown on these figures are the model's predicted plan alignments for the beach immediately inshore of each location. These orientations have zero net longshore transport in response to the four years of the wave climate which was hindcast for the period of 12th October 2000 to 28th September 2004. As can be seen, the model is predicting the actual beach alignments very well - to within 0.1º in fact.

Consequently it can be used to predict the impacts on the alignment and stability of these beaches with confidence. It is also a verification of the Wave Transformation Modelling since beach alignments are determined by the wave climate. The strong correlation between the predicted and actual beach alignments could not have eventuated if the model was not closely replicating the actual wave conditions.

5.3 Impacts on The Strand Beaches

As discussed in the preceding Section 4.1, one of the expected impacts of the proposed development is the reduction and realignment of the wave energy propagating onto the southern reaches of The Strand foreshore. This will manifest itself as some localised realignment of the beach in this area. The extent of this induced change to the plan orientation has been investigated using mathematical modelling techniques.

The results of the Wave Transformation process (reported in Section 4.1) indicated that the proposed lengthening of the existing breakwater to create the new Northern Breakwater, in conjunction with the proposed dredging of the new access channel and navigable entrance into Breakwater Cove, will result in some changes to the directional wave climate along the central and southern sections of The Strand foreshore. This will induce changes to the orientation of the beaches in this location.

As discussed previously, Wave Transformation and Sediment Transport modelling has been applied to three locations along the length of The Strand coastal reach. These being Sites 01, 02, and 03 as shown on Figure 5.1, Figure 5.2 and Figure 5.3. The following discussions relate to the various beach precincts along The Strand inshore of these locations.

Figure 5.1 : Predicted Beach Alignment Inshore of Site 01

Figure 5.2 : Predicted Beach Alignment Inshore of Site 02

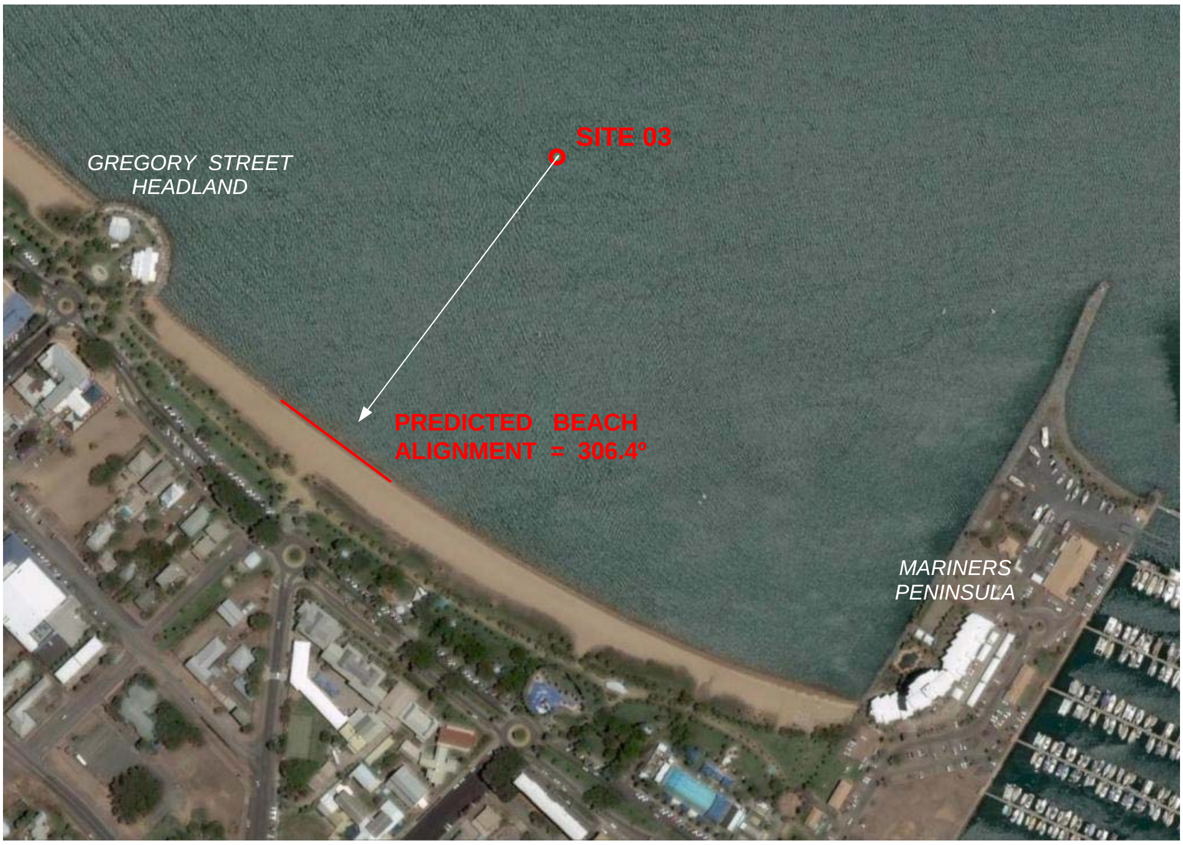


Figure 5.3 : Predicted Beach Alignment Inshore of Site 03

Mariners Peninsula - Gregory Street Headland

Being immediately adjacent to the significant reclamations that have been undertaken to create the Breakwater Casino and the Port precincts, this coastal reach is already somewhat protected from the Distant Sea waves that are entering Cleveland Bay and propagating to shore. The curved plan shape of the beach reflects the influence that the southern reclamation has on providing this shelter. The main exposure is to the Local Seas that propagate to this foreshore across the open water fetches to Magnetic Island.

The longshore sediment transport modelling undertaken for this reach of foreshore uses the wave climate at Site 3 as representative of that which approaches through nearshore waters. The results predict that the stable plan alignment along the section of this foreshore nearest the Gregory Street headland will realign itself so as to face some 0.5º more northwards (ie. a very slight "anti-clockwise" rotation in its plan alignment).

This very minor realignment of the naturally preferred plan form of the beach will occur through the transport of sand from this area by the changed wave conditions towards the southern end adjacent to Mariners Peninsula. Once this realignment occurs, the new beach plan shape will then become the preferred orientation. This readjustment and movement of sand will manifest itself as a slight narrowing of the beach width in the area south of the Gregory Street headland and the widening of the beach at its southern end.

Consequently it may be perceived by the community as erosion of the beach south of the Gregory Street headland (which indeed it is) but will be accompanied by accretion of the same beach further south. There will be no net loss of sand from the beach precinct.

It is anticipated that the beach against the southern side of the Gregory Street headland will gradually migrate to a stable position that is only some 2m inland from its present location. This will taper back to its approximate current position some 80 to 100m further southward towards Mariners Peninsula. The predicted response of this beach precinct is shown in Figure 5.4. This recession will not threaten the pathway that currently provides access to the beach from the southern side of the Gregory Street headland.

The impacts on this beach compartment are therefore minor. They will not threaten any foreshore infrastructure and will not adversely affect the beach amenity.

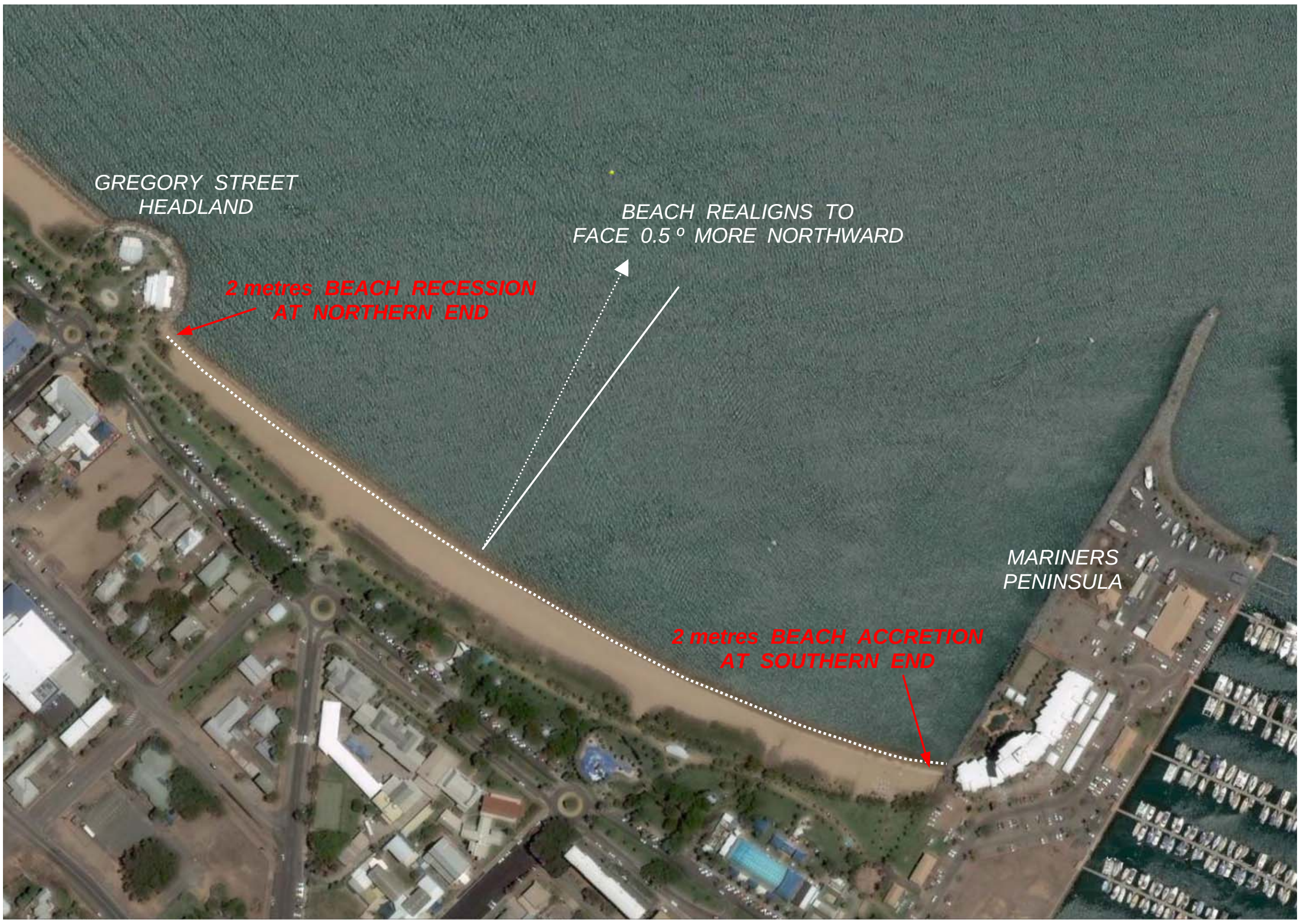


Figure 5.4 : Predicted Impact on the Beach between Mariners Peninsula & Gregory Street
(Beach re-alignment is drawn conceptually and rotation is not necessarily to scale)

Gregory Street Headland - Burke Street Headland

This 365m long beach compartment is contained between the two headlands opposite Gregory Street and Burke Street. Of all the beach compartments along The Strand foreshore, it is this particular one which experiences the greatest changes to the naturally preferred stable plan alignment. Nevertheless this is only entails a minor realignment of the stable plan position.

The impact on the existing beach alignment occurs because under the pre-development condition the wave shadow caused by the existing Northern Breakwater tends to diminish near the southern end of this compartment - but under the developed scenario, the shadow extends further northward to encompass most of this beach.

In addition to having reduced heights, the incoming waves also arrive at this foreshore from a slightly more northerly direction. Despite the reduction in incoming wave energy, its slight directional rotation will induce a corresponding realignment of the beach between these two headlands as it rotates in plan to align itself with the changed wave energy regime.

The longshore sediment transport modelling undertaken for this beach compartment uses the wave climate calculated at Site 2 as representative of that which approaches through nearshore waters. Calculations of the longshore sediment transport indicate that to achieve its stable alignment (of zero net annual longshore transport) the beach will realign itself by rotating some 0.75º more northwards (ie. an "anti-clockwise" rotation in its plan alignment). The predicted response of this beach precinct is shown in Figure 5.5.

Figure 5.5 : Predicted Impact on the Beach between Gregory Street & Burke Street
(Beach re-alignment is drawn conceptually and rotation is not necessarily to scale)

To accomplish this natural realignment, sand will gradually be moved by the prevailing waves from the northern end of the beach (ie. the section of beach south of the Burke Street headland) to its southern end (against the northern side of the Gregory Street headland). The beach at the northern end of this compartment will recede by approximately 2.5m to attain its stable orientation with regard to incoming wave energy. A compensating accretion will occur against the Gregory Street headland - there will be no net loss of sand from the beach compartment.

The impacts on this beach compartment are therefore minor and will not threaten any foreshore infrastructure, nor adversely affect beach amenity.

Burke Street Headland - Stuart Street Headland

This is a 170m long beach compartment contained between the two headlands opposite Burke Street and Stuart Street. The wave shadow created by the existing breakwater near the Port entrance does not have a strong influence on this length of The Strand foreshore, as its effect is focussed on the foreshore to the south. The proposed extension of the breakwater will cause the influence of the shadow to migrate towards this compartment, nevertheless its effect will still be reasonably diminished this far north along The Strand foreshore.

The calculations of longshore sediment transport show that a new stable beach alignment will be some 0.5º further northward. Figure 5.6 illustrates the predicted response of this beach compartment. This reorientation of the beach plan form will manifest itself as removal of sand from the northern end of the compartment (ie. from the area south of the Stuart Street headland) and placement at its southern end (ie. against the northern side of the Burke Street headland). The recession of the northern end is predicted to be approximately 1m.

However it is unlikely that the changed beach alignment will be visually detectable given the natural seasonal fluctuations to the beach alignment that presently occur.

Stuart Street Headland - Kissing Point

This is a long beach compartment, stretching some 575m along The Strand foreshore. A small headland exists at the back of the beach opposite Landsborough Street, however this feature is too small and located too high in the beach profile to have any significant effect on containing longshore sand transport. The longshore sediment transport modelling undertaken for this reach of foreshore uses the wave climate at Site 1 as representative of that which approaches through nearshore waters.

Calculations of the wave climate and the sediment transport regime along this beach precinct indicates no detectable effect on the existing foreshore alignment.

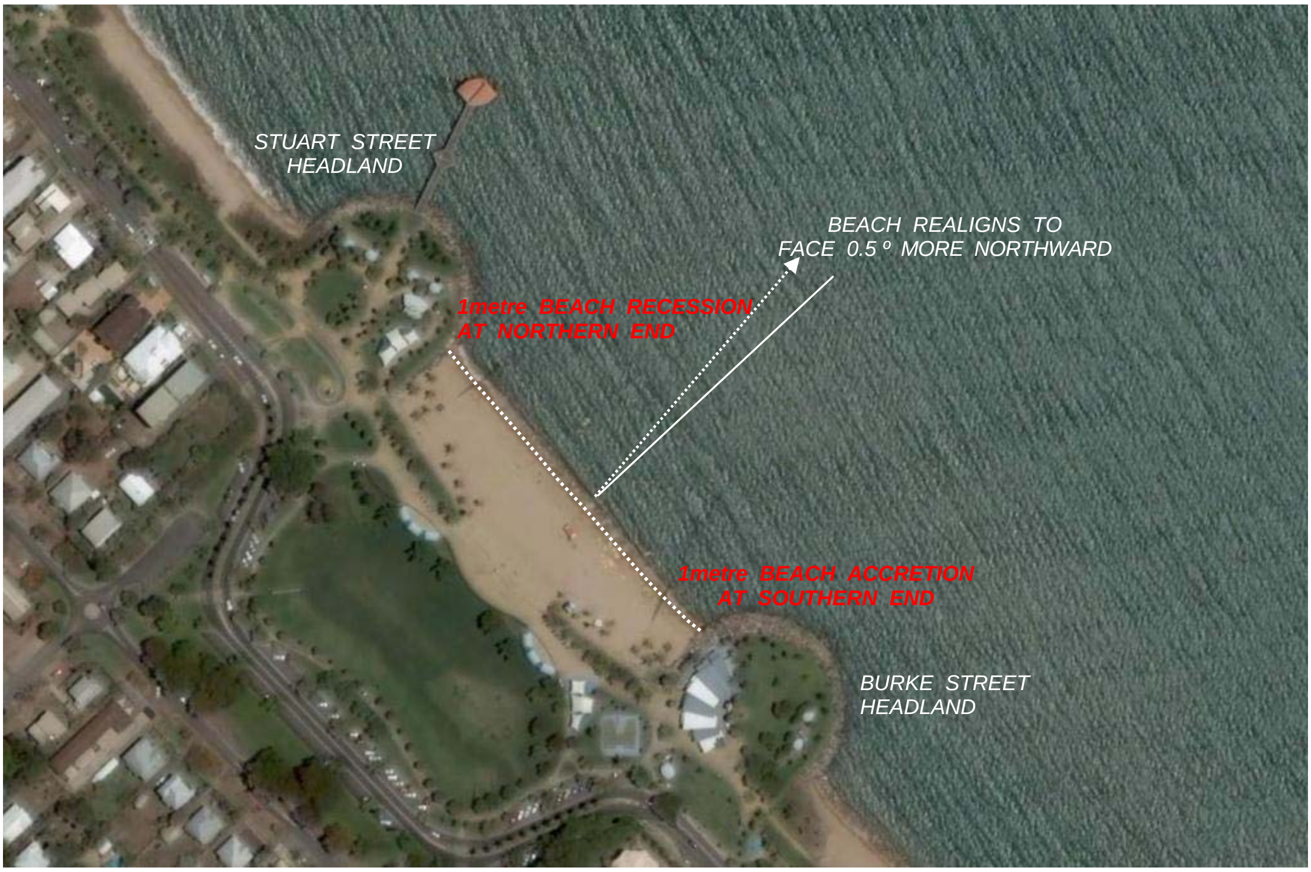


Figure 5.6 : Predicted Impact on the Beach between Burke Street & Stuart Street
(Beach re-alignment is drawn conceptually and rotation is not necessarily to scale)

Timescales for Changes

The timescale of the adjustment to the various beach alignments will depend significantly upon the severity of future ambient wave conditions. However based on “average” conditions this is expected to take some 1 to 3 years to eventuate once the Northern Breakwater is extended and the access channel and entrance into the Breakwater Cove precinct are dredged.

6 PRELIMINARY DESIGN OF ROCK ARMOUR WORKS

6.1 Structural Design Methodology

There are a number of permanent marine structures within the proposed Townsville Ocean Terminal Project which rely on rock armouring to maintain their structural integrity. These being:

- the existing breakwater facing north-east directly out to Cleveland Bay, which will serve as a seawall when land is reclaimed immediately behind this structure (termed herein as the *Northern Breakwater*);
- a new breakwater along the north-west perimeter of the Breakwater Cove waterways (termed herein as the *Strand Breakwater*);
- revetments within the Breakwater Cove waterways.

Figure 6.1 shows their respective locations within the proposed development.

It is important to appreciate that the following discussions and determination of structural concepts for rock armour works are preliminary and therefore will be confirmed by a more rigorous detailed engineering design and documentation phase prior to construction. The intent of the subsequent detailed design phase is to utilise physical modelling techniques to determine the most appropriate structural arrangements.

Physical modelling is often used by the coastal engineering profession to design maritime structures. It involves the construction of scaled models of coastal structures to precise scaling laws of fluid dynamics, which are then subjected to scaled waves and/or currents to test and improve their structural performance.

Physical model testing in the subsequent detailed engineering phase offers the opportunity to replicate the specific conditions associated with each individual component of the Townsville Ocean Terminal Project. The performance of each can be investigated and its design confidently optimised by tailoring it to the specific conditions of the site. This avoids the otherwise high costs that would be the consequence of an over-designed structure, or an under-designed structure.

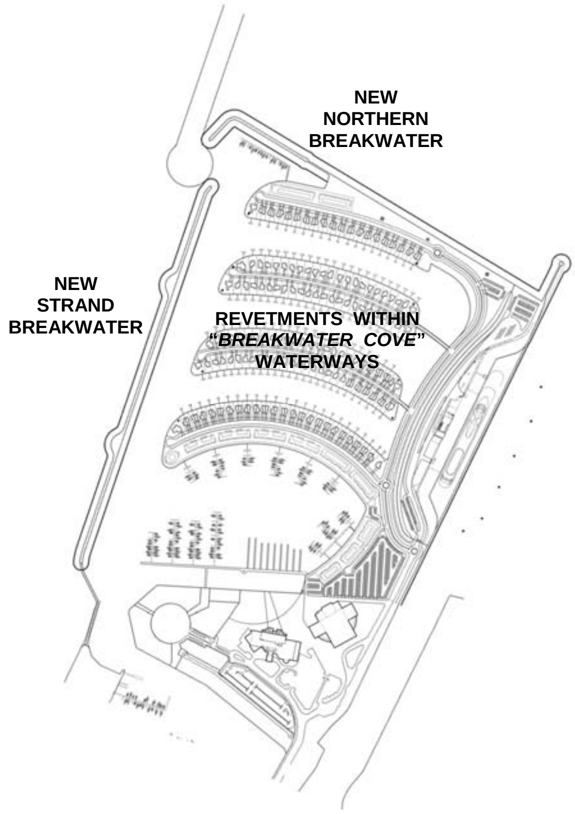


Figure 6.1 : Locations of Rock Armour Works

Important parameters such as the local wave spectrum (wave heights, periods, etc.), storm tide levels, breaking wave setup, wave runup levels, cyclone/storm durations, seabed approaches, rock sizes and densities, etc. can all be incorporated into the physical model. Under such a testing regime, the wave and storm tide parameters associated with the 100 year ARI design event can be varied within a wide range of all possible scenarios. Typically structures are tested to failure to identify the failure mechanisms and to thereby devise means of improving the performance at minimal additional cost.

Other aspects can be explored by the physical modelling process - for instance experience has shown that it may not necessarily be the highest storm tide levels that inflict the greatest damage to breakwater structures. Lower water levels may cause incoming waves to break and "plunge" directly onto the front face of the structure rather than surge unbroken up the slope as might occur during higher storm tide levels.

It is for this reason that the design parameters associated with the 100 year ARI storm tide event nominated in Section 2.2.2 serve as a guide to preliminary structural design.

The complexity and cost of physical modelling is only warranted during the detailed engineering design phase of project implementation. As a step towards this later stage, preliminary designs for the rock armouring works have been prepared using mathematical calculations. It is expected that the subsequent physical modelling of these preliminary designs will result in optimisation of their various components (eg. rock sizes, layer thicknesses, crest armouring, etc.) but the overall structural concepts will remain unchanged.

Various methods for calculating the size of rock armour under wave attack have been proposed by coastal engineers in the past few decades. The decision as to which mathematical technique is the most appropriate has been the subject of much deliberation, however most practitioners are now generally agreed that the formulae developed by Van der Meer (1988) are the most appropriate. They are based upon an extensive series of physical model tests conducted at Delft Hydraulics, which included a wide range of core / underlayer permeabilities and incident wave conditions.

These *Coastal Engineering Studies* for the Townsville Ocean Terminal Project have utilised design methodologies based on the formulae of Van der Meer when determining the structural concepts and the performance of rock armoured works.

The requirements of the Environmental Protection Agency's operational policy "*Building and engineering standards for tidal works - Version 1.2*" have been incorporated into the design of these various rock structures. Of particular relevance are the minimum acceptable standards for seawalls presented under Clause G of that policy. Some comment with respect to the application of the standards in Clause G to the proposed Townsville Ocean Terminal structures is therefore appropriate

Design Storm Event

The EPA's policy states that seawalls must be designed to withstand wave and water level conditions associated with the 50 year Average Recurrence Interval event. A further requirement is that damage to structures as a consequence of such an event does not result in more than 5% of the armour units (in this case, individual rocks) being dislodged.

However, marine infrastructure associated with the Townsville Ocean Terminal Project will be designed to accommodate the more severe 100 year ARI design storm event whilst retaining the same acceptable damage criteria. The rationale being that the Designated Storm Tide Event (DSTE) required at this location under the State Coastal Plan policy 2.2.4 for storm tide hazard mitigation is the 100 year ARI event.

The selection of the 100 year Average Recurrence Interval event is not a straight forward process, as it consists of a combination of severe waves and extreme water levels. When designing coastal defences it is necessary to consider the likelihood of both conditions occurring simultaneously.

The assumption of complete dependence between waves and water levels in an analysis of joint occurrence would lead to a very conservative design - since the 100 year Average Recurrence Interval event would have to comprise a 100 year ARI storm tide level and a 100 year ARI wave height.

Conversely the assumption of independence between waves and water levels could lead to under-design, since any increase in the probability of high waves at times of very high water levels would have been ignored. The actual correlation between waves and storm tide levels will lie between these two extremes of complete dependence and complete independence.

As discussed in the earlier Section 3.2 of this report, the approach adopted when determining structural concepts for the marine works of the Townsville Ocean Terminal Project has been to consider the following scenarios as potentially constituting the 100 year Average Recurrence Interval event, and to then select the one having the most adverse effect on structural performance:

- Scenario 1 : 100 year ARI storm tide level occurring simultaneously with the 50 year ARI wave characteristics; or
- Scenario 2 : 50 year ARI storm tide level occurring simultaneously with the 100 year ARI wave characteristics.

Calculations of rock armour requirements and structural performance of all exposed rock armour indicates that it is the former scenario which results in the greater impact on the structures. This is because the higher ocean water level allows the larger waves in the sea state to propagate onto the structures. Whereas the lower ocean water levels adopted under the second scenario tend to cause these larger waves to break and expend their energy before reaching the structures. Likewise overtopping of foreshore structures was found to be more sensitive to storm tide level than to the height of the offshore waves generated by cyclones.

Consequently Scenario 1 for the 100 year ARI Designated Storm Tide Event has been used to prepare preliminary designs for the rock armour works. However, as discussed in the preceding pages, this will be further examined by physical modelling during the subsequent detailed engineering design phase.

Overtopping

The EPA policy states that overtopping by waves is permitted but the design must be such that the structural stability of the wall is unaffected. The *Coastal Engineering Studies* have specifically investigated overtopping performance of all armoured works - under both the 50 year ARI and 100 year ARI scenarios.

As will be discussed later, this has resulted in providing some additional armouring in the crest of the main Northern Breakwater. Elsewhere the crest levels on structures have been designed to either be high enough to prevent any adverse overtopping, or an armouring arrangement has been implemented to accommodate overtopping. When determining the overtopping performance of the various structures, the techniques presented in HR Wallingford (1999) have been applied.

Toe of the structure

The EPA policy requires that the toe of the structure be designed to accommodate potential long term erosion for at least 50 years. This has been incorporated into the designs by application of appropriate toe scour protection where necessary. All rock armoured structures are founded below the level of the Lowest Astronomical Tide - another requirement of the EPA's policy.

Water Levels

The policy requires that all design water levels include an allowance of 0.3 metres for the influence of Greenhouse Effects on sea level rise. As discussed in Section 2.2.3, these *Coastal Engineering Studies* utilises an increase of 0.4m at the 50 year ARI and a 0.5m increase at the 100 year ARI when devising structural concepts for the rock armoured coastal defences for the Townsville Ocean Terminal Project.

Other considerations

The EPA policy states that the armoured slopes are to be designed to minimise wave reflection and any "end effects" on the adjacent foreshores. A two layered armouring arrangement (with an additional two layers of underlying filter rock) of slopes that are typically 1 vertical to 1.5 horizontal is widely acknowledged to provide acceptable wave dissipation performance.

This is the structural concept adopted for the Townsville Ocean Terminal structures. As discussed in Section 5.3, the proposed works will not adversely impact on any adjacent foreshores or structures.

6.2 Northern Breakwater

The primary purpose of this seawall is to protect the development along its exposed northern boundary. The Northern Breakwater faces directly towards the exposed fetches out across Cleveland Bay.

No building infrastructure is planned near the crest of the seawall and this offers the opportunity to have the crest level of the seawall (and the level of the reclamation behind it) reasonably low so as to improve the visual amenity of the parkland intended for this area. Nevertheless the crest area needs to accommodate any overtopping which might occur during extreme wave and storm tide events.

As stated in the preceding Section 6.1, the design philosophy applied to the rock armouring works for the Townsville Ocean Terminal Project has been to limit any damage to less than 5% of armour being dislodged under a 100 year ARI Designated Storm Tide Event. The design wave parameters for such scenarios have been determined from the mathematical modelling of waves undertaken in these *Coastal Engineering Studies*.

As discussed in Section 3.4.3, there were two potential scenarios that constituted the Designated Storm Tide Event. However the scenario that had the greatest structural requirement was that associated with the 100 year ARI storm tide in association with the 50 year ARI wave event (ie. Scenario 1 discussed in Section 3.4.3). The design parameters associated with that occurrence are as follows:

Design Parameter	*100 year ARI*
Significant Wave Height : H_s	2.35 metres
Peak Period : T_p	10 - 11 secs
Storm Tide Level (excluding wave setup)	RL+2.91m AHD

Table 6.1 : Wave Parameters for Conceptual Design of the Northern Breakwater

When preparing the conceptual design of the armour on the front slope of the seawall, the use of armour from the existing Northern Breakwater (which lies approximately on the proposed alignment) as a structural solution has been adopted. The original design drawings for this structure indicate rock armour of 2 tonne to 5 tonne size, placed to a crest level of RL+4.4m AHD and with a seaward slope of 1 vertical to 1.35 horizontal.

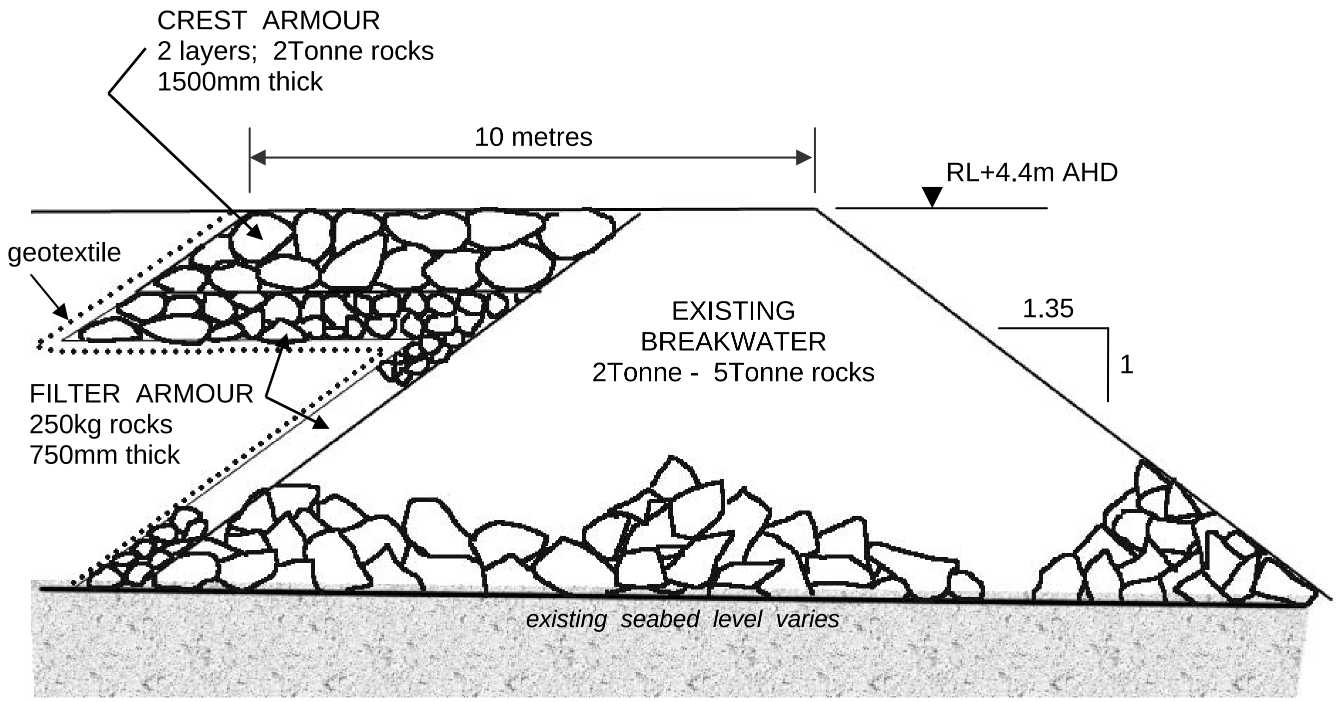
Calculations of structural integrity using the analytical techniques of Van der Meer (1988) indicate that if the armour rocks that currently constitute the existing breakwater were to be used within the new Northern Breakwater, then they could accommodate the loadings from the design conditions presented in Table 6.1 with acceptable levels of damage. However consideration of overtopping performance of this existing structure indicates that average overtopping rates during the 100 year ARI event will exceed the thresholds for crest damage offered for guidance by HR Wallingford Ltd (1999).

In other words, the front face of the new structure is unlikely to be significantly damaged, however it is predicted that (because the crest level is too low), green water overtopping would significantly scour any unprotected fill material immediately behind the armour layers at the top of the seawall.

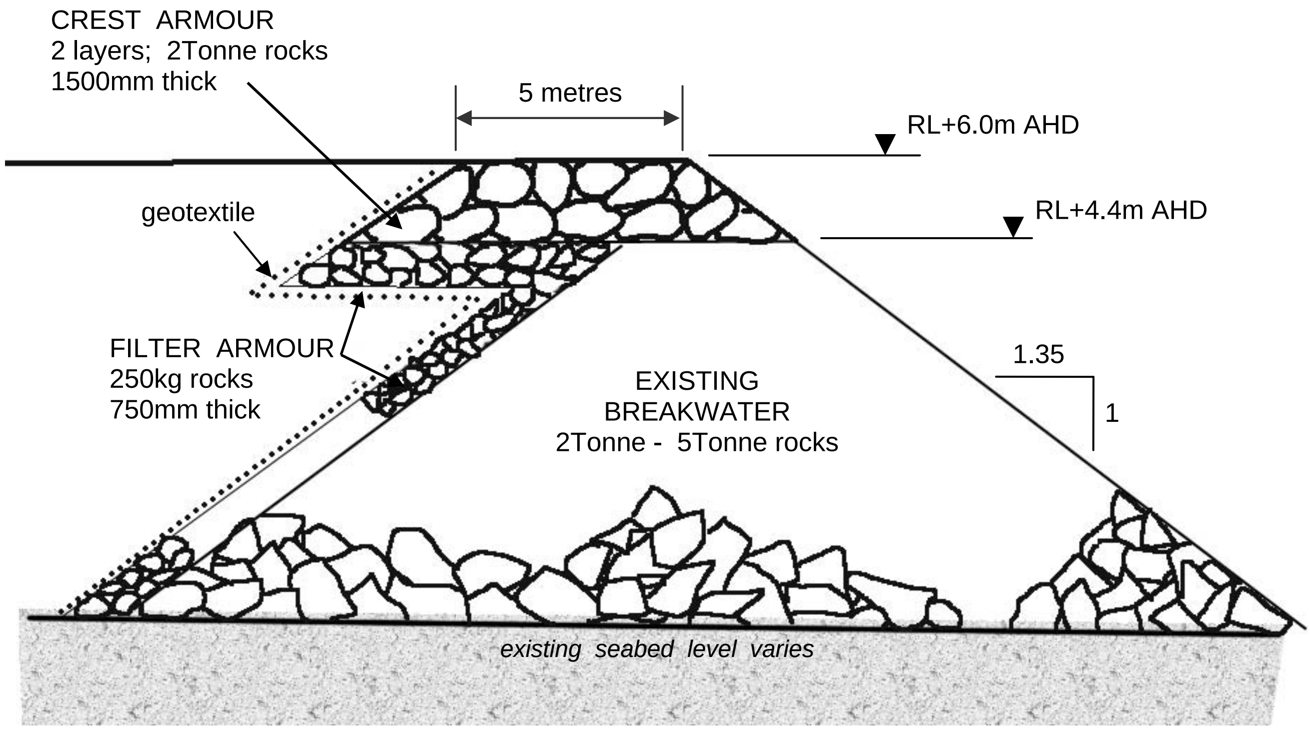
Consequently, a scour blanket needs to be provided as crest armouring in this “at risk” region of the Northern Breakwater to ensure that its structural integrity is not compromised by the 100 year ARI event. Figure 6.2(a) shows the crest armouring arrangement which extends some 10 metres back from the top of the armoured slope.

An alternative to providing a wide scour blanket at the crest of the seawall is to raise the level of the crest to such an extent that the amount of green water overtopping coming over the crest is reduced to levels that will not instigate significant scour. Calculation of overtopping rates using techniques outlined in HR Wallingford Ltd (1999) indicate that the crest level would need to be raised to RL+6.0m AHD to achieve the necessary reduction. Figure 6.2(b) shows this alternative structural concept.

Scour protection is also required at the toe of the Northern Breakwater. The design procedures presented by McConnell (1998) have been used to determine two options for protecting the toe of the seawall, depending upon the extent to which the existing breakwater structure is utilised in the new works.. These are presented in Figure 6.3 as a “buried toe” arrangement and a “toe berm”.

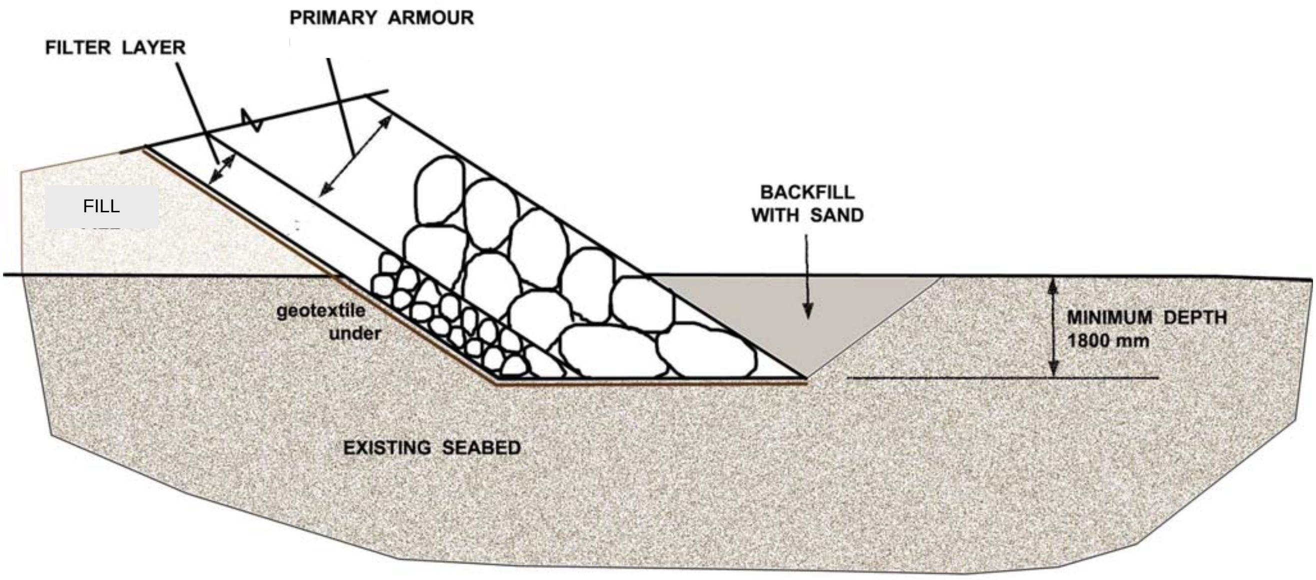


(a) Option with Crest Level at RL+4.4m AHD

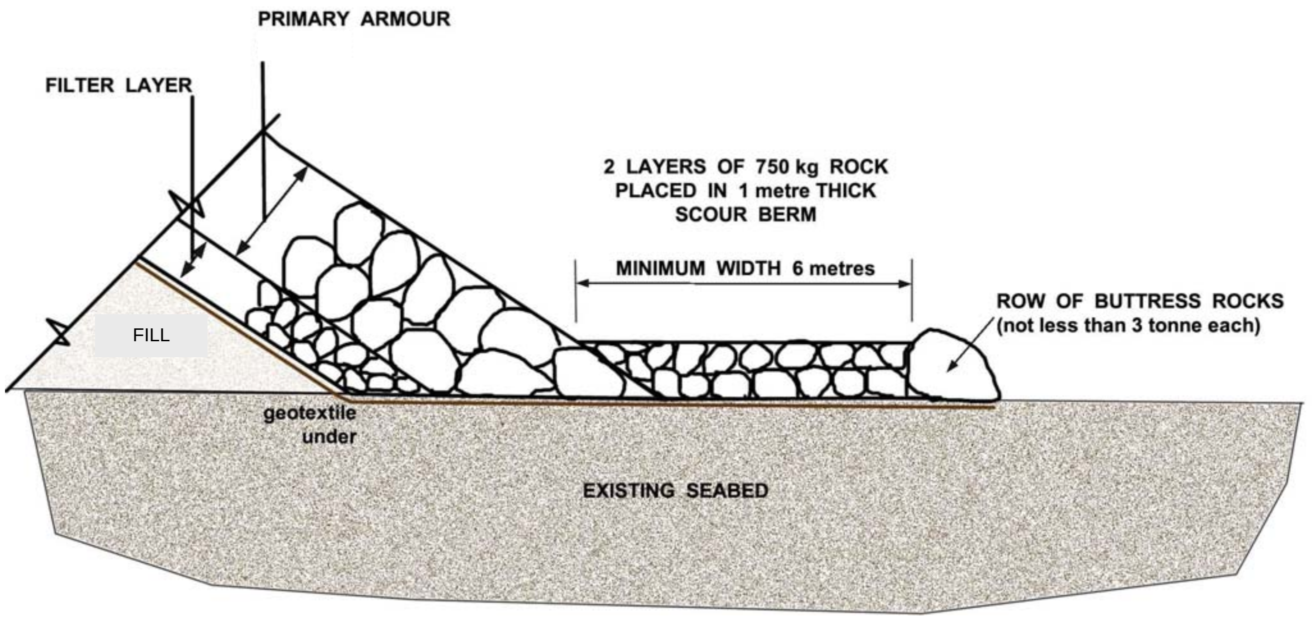


(b) Option with Crest Level at RL+6.0m AHD

Figure 6.2 : Viable Structural Concepts - Northern Breakwater



a) "Buried Toe" Arrangement



b) Toe Berm Arrangement

Figure 6.3 : Northern Breakwater - Toe Protection Options

The selection of the most appropriate will be based upon geotechnical and constructability issues, however it is likely that the toe berm will be adopted as it does not require excavating and maintaining a trench in the submerged seabed when placing rocks. It would also be the option used where the existing structure is incorporated into the new Northern Breakwater.

6.3 Strand Breakwater

This breakwater provides wave protection along the western perimeter of the proposed Townsville Ocean Terminal development. It faces directly out across the local fetches of Cleveland Bay towards Magnetic Island. Nevertheless, it is exposed to any extreme / cyclone waves that can be generated as Local Seas across these fetches.

Some overtopping of this structure can be tolerated provided such overtopping does not result in failure of the breakwater crest.

As discussed in Section 3.4.3, there were two potential scenarios that constituted the Designated Storm Tide Event. However the scenario that had the greatest structural requirement was that associated with the 100 year ARI storm tide in association with the 50 year ARI wave event (ie. Scenario 1 discussed in Section 3.4.3). The design parameters associated with that occurrence are as follows:

Design Parameter	*100 year ARI*
Significant Wave Height : H_s	1.7 metres
Peak Period : T_p	6 secs
Storm Tide Level	RL+2.91m AHD

Table 6.2 : Wave Parameters for Conceptual Design of the Strand Breakwater

Again the structural design techniques of Van der Meer have been used to determine armour size for this Strand Breakwater. The structural concept is shown on Figure 6.4. A significant structural feature of this breakwater is the placement of larger rocks along the rear (shoreward) edge of the crest. This is to ensure that any green water overtopping of the breakwater that occurs during the design cyclone event does not initiate failure of the crest.

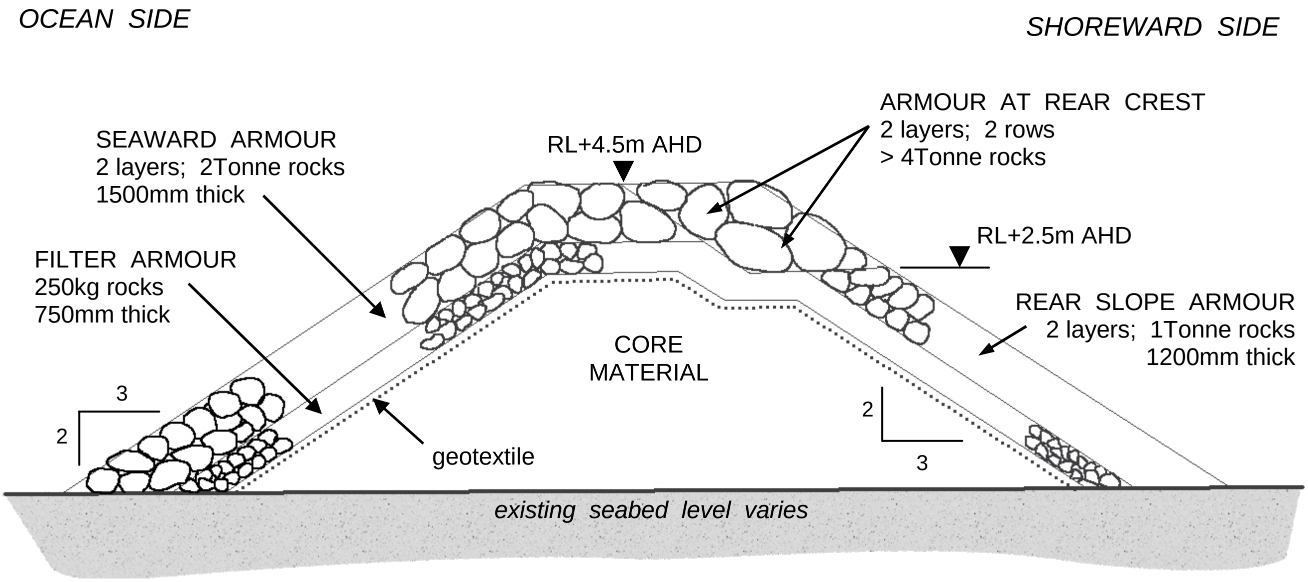


Figure 6.4 : Viable Structural Concept - Strand Breakwater

6.4 Internal Revetments

Rock revetments are proposed as the internal walls of the marina basin and waterways of Breakwater Cove. A two-layered rock armoured slope provides a very effective way of absorbing any waves which diffract into the marina, as well as dissipating any wash generated by vessel traffic in and around the project waterways.

The rock armour has been determined to be two layers of 300 kg nominal size (acceptable range in rock size being 100kg to 500kg). An appropriate geotextile is to be placed on the bank slope beneath the two layers of rock armour.

6.5 Wave Protection for Breakwater Cove Waterways

Unless the layout and design of breakwaters are properly considered, wave energy can enter and potentially compromise the protected waterways and internal infrastructure of harbours and marinas. This can occur as a consequence of two primary mechanisms:

- wave overtopping; and/or
- wave penetration through the ocean entrance.

The specific layout of the Breakwater Cove development (and the characteristics of the perimeter structures which provide wave protection) have been determined through consideration of the local wave climate and the need to provide appropriate wave protection to those marina berths nearest the entrance.

6.5.1 Mitigation of Wave Overtopping

The preceding discussions regarding the performance of the new Northern Breakwater and Strand Breakwater have highlighted that a primary design criteria for these wave protection structures is that they limit the extent of wave overtopping during the Design Storm Event to acceptable levels. The acceptance criteria being:

- For the Northern Breakwater - overtopping is less than the threshold that would instigate scour of the reclamation material behind the crest of the structure. At its offshore end, this breakwater provides protection to several marina berths immediately inside the ocean entrance to the Breakwater Cove waterways. The acceptance criteria for wave overtopping along this length of the structure would be the attainment of the wave criteria at these berths as specified in "*AS3962 - Guidelines for design of marinas*".
- For the Strand Breakwater - the acceptance criteria for wave overtopping along this structure would be the attainment of the wave criteria in the protected waterways behind it as specified in "*AS3962 - Guidelines for design of marinas*".

The potentially adverse effects of wave overtopping on the waterways and reclamations of the Breakwater Cove development will be mitigated by the appropriate selection of crest height and width for both the Northern Breakwater and the Strand Breakwater. As discussed previously, the detailed design phase will utilise physical modelling techniques to ensure that wave overtopping effects are properly considered.

6.5.2 Mitigation of Wave Penetration Through Entrance

As shown in Figure 6.5, the ocean entrance into the Breakwater Cove waterways is configured so as to face directly towards East. Rather than face directly out across the open water fetches towards Magnetic Island and other exposed areas of Cleveland Bay, the entrance has been orientated so as to face towards The Strand foreshore and Kissing Point (which is less than 2km away).

When undertaking the Wave Modelling for these *Coastal Engineering Studies,* an inshore site was selected midway between the entrance into the Breakwater Cove development (nominated as Site 07, refer to Figure 3.3). This enabled the entrance arrangement to be evaluated and optimised with respect to the ambient and extreme / cyclone wave climates.

Figure 6.5 : Wave Approaches to the Breakwater Cove Entrance

Note : the angle θ represents the maximum range of possible wave approaches to the entrance

Also shown conceptually on Figure 6.5 is the “directional window” through which waves approach the entrance into the Breakwater Cove waterways. One of the main outcomes of the wave modelling at the entrance (Site 07) was the determination of the extent of directions for waves approaching the entrance - that is, the maximum value of the angle θ in Figure 6.5.

This result is summarised in Table 6.3 - which shows the maximum range of approach directions for each wave period (represented in the Table by the angle θ_{max}), along with the maximum significant wave height associated with each particular wave period and direction (over the approximate four years from 12th October 2000 to 28th September 2004).

Of particular relevance is that waves having periods of around 10 seconds and greater (typically Distant Seas and swell waves) have very little influence at the entrance. This is because such waves have been significantly refracted by the relatively shallow seabed approach slopes as they propagate through Cleveland Bay towards shore. Not only are they considerably attenuated in height by this refraction process, but they also have been aligned so that their advancing wave crests are parallel to the seabed contours. Therefore they tend to sweep passed the entrance rather than propagate directly through the entrance opening.

Wave Period	*Direction Range* θ_{max}	*maximum* H_s
2 secs	31°	0.4 metres
3 secs	29°	0.7 metres
4 secs	28°	0.6 metres
5 secs	25°	0.3 metres
6 secs	21°	0.1 metres
7 secs	18°	0.1 metres
8 secs	15°	< 0.1 metres
9 secs	12°	< 0.1 metres
10 secs	7°	*negligible*
> 10 secs	< 2°	*negligible*

Table 6.3 : Range of Wave Approaches to The Breakwater Cove Entrance

Nevertheless some wave energy from these long period waves can still enter the entrance by way of *wave diffraction*. This phenomenon occurs whenever waves encounter an obstruction (such as the breakwaters forming the entrance into the Breakwater Cove waterways). As they pass by the end of the obstacle they disperse some wave energy around its end into the shadow zone.

Reference to Table 6.3, in conjunction with the layout of the entrance shown in Figure 6.5 indicates that the waves which approach the entrance do so at relatively acute angles. The location of the marina berths adjacent to the entrance are such as to be protected from direct wave action. However as discussed above, wave diffraction effects can result in these waves dispersing some energy into the lee of the protective Northern Breakwater where some marina berths are located.

Consequently wave diffraction calculations were undertaken for both ambient and extreme/ cyclone wave characteristics. The universally applied computational techniques outlined by Goda (2000) have been applied to determine wave conditions at the marina berths within the entrance as a consequence of wave diffraction. Ambient conditions were represented by the four years of half-hourly wave records from 12th October 2000 to 28th September 2004;

whereas the extreme / cyclone conditions associated with the Design Storm Event for both Distant and Local Seas were used to investigate the scenario of cyclone occurrences.

The Australian Standard "*AS3962 - Guidelines for design of marinas*" presents wave height criteria for small craft harbours (such as that proposed within the Breakwater Cove waterways). The results of the diffraction analyses indicate that the proposed entrance configuration complies with the requirements nominated in AS3962 for a "good" wave climate at the most exposed marina berths within the waterways; namely

- for ambient conditions : H_s < 0.3m for head seas and H_s < 0.15m for beam seas;
- for 50 year ARI conditions : H_s < 0.6m for head seas and H_s < 0.25m for beam seas.

In other words, the ocean entrance to the Breakwater Cove development is configured so as to provide wave protection to internal waterways which is in accordance with the requirements of "*AS3962 - Guidelines for design of marinas*".

7 COMPLIANCE WITH EPA GUIDELINES REGARDING STORM TIDE

7.1 Requirements

The Queensland EPA has issued a Guideline document titled "*Mitigating the Adverse Impacts of Storm Tide Inundation - vers 1.2*" which provides advice and information on interpreting Coastal Hazards' Policy 2.2.4 of the *State Coastal Management Plan - Queensland's Coastal Policy* (State Coastal Plan). The Guideline aims to ensure that storm tide inundation is adequately considered when decisions are being made about coastal developments.

Some comment is offered in respect to the EPA's Guideline document and the findings of this assessment of the possible effects of storm tide and wave inundation on the Townsville Ocean Terminal Project.

When development applications are assessed against Coastal Plan Policy 2.2.4, consideration is given to whether the development minimises as far as practical the adverse impacts of storm tide inundation, and that it does not result in an unacceptable risk to people or property. Appendix 4 of the EPA Guideline offers assistance in devising appropriate measures for achieving those outcomes. A table is presented in that appendix which lists specific outcomes, solutions and comments relating to assessment of coastal developments.

Consequently the comments presented below are structured so as to correspond to the outcomes required (and the solutions offered) in the table of the EPA's Appendix 4. As discussed in Section 3.2, the Designated Storm Tide Event (DSTE) is the 100 year ARI storm tide event.

7.2 Compliance Assessment

<u>*Specific Outcome 1*</u> *: Development maintains the safety of people on the development site from all storm tide inundation up to and including the DSTE.*

- Complies. Dwellings within the development are sited so that the floors on all habitable rooms are above the DSTE level; and by being set back from the perimeter seawalls they are not located within high hazard zone. Access roads for emergency evacuation purposes are also above the DSTE level.

Specific Outcome 2 : Development does not increase the severity of the storm tide hazard on adjacent properties.

- Complies. The proposed works do not adversely increase the storm tide or the associated waves on adjacent foreshores or properties.

Specific Outcome 3 : Development minimises the potential damage from storm tide inundation to property on the development site.

- Complies. Being set back from areas potentially prone to wave overtopping, the proposed building work is not sited within the high storm tide hazard zone. The dwellings are also sited with the floors of all habitable rooms located above the DSTE level.

Specific Outcome 4 : (relates to manufacture and storage of hazardous materials)

- The issue regarding the deployment or otherwise of hazardous materials throughout the development site is beyond the scope of this coastal engineering assessment.

Specific Outcome 5 : (relates to the operation of essential services during the DSTE)

- The issue regarding the operation and location of essential services' infrastructure throughout the development site is beyond the scope of this coastal engineering assessment.

Specific Outcome 6 : Physical coastal processes are protected from development impacts and are generally allowed to occur naturally.

Essentially complies. There are anticipated to be some changes to the wave climate on the southern foreshores of The Strand beaches. This will result in some gradual re-distribution of sand on the foreshore as the local beaches in this area stabilise to new alignments. These changes are minor and will not adversely affect the natural coastal processes that are currently shaping the southern shores of The Strand; nor will they significantly interfere with tidal flows; alter existing coastal hydrological flows; or create adverse conditions for adjoining coastal vegetation.

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