Ballina Accropode Breakwater
Flume Testing

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BALLINA ACCROPODE BREAKWATER
FLUME TESTING

Department of Public Works and Services
Manly Hydraulics Laboratory
Foreword

The report was prepared by the Department of Public Works and Services' Manly Hydraulics Laboratory on behalf of the Coastal and Riverine Management Directorate of the Department of Land and Water Conservation. The study was undertaken and report prepared by I. Jayewardene.
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1. Introduction

The Minor Ports Section of the Department of Land and Water Conservation (DLWC) requested Manly Hydraulics Laboratory (MHL) to undertake a model study in the random wave flume to test a cross-section of the Ballina South breakwater round head Accropode design. The Accropode is a precast concrete armour unit developed by Sogreah. The design was prepared by DLWC consultants in conjunction with Sogreah. Model testing is being undertaken to confirm the final design.

The Accropode breakwater cross-section design drawings for Ballina South breakwater were supplied by DLWC. Design drawing No. 1743-106 (Figure 1.1) shows two typical cross-sections through the breakwater. The first cross-section is typical of sections of the head and is from chainages 575 to 585. The second cross-section is typical of chainages 540 to 570. The axis of the breakwater is offset approximately 30° to the depth contours. As the south-east side of the breakwater is more exposed to wave attack, this side formed the seaward face in the model testing. The round head cross-section which was tested in the flume is typical for chainage 575 to 585 and represented by profile 135° (being 135° to the north).

Inspections (Reference 1) on the Ballina breakwater head indicated that the head extends some 70m seaward below the water level. The field inspection also reported extensive crest failure due to wave overtopping. The wave climate is such that the larger waves break offshore due to depth-limiting conditions but are still large enough to damage the crest due to overtopping. These conditions were simulated in the flume during the testing. Design wave heights at the structure were based on an analysis of the major storms on the NSW coastline carried out for previous studies (References 2 and 3).

Model testing was undertaken with a simulated scour hole in front of the breakwater face to ensure that the design waves reached the structure. Since the Accropode is a single layer placed unit its performance was compared with single layer placed rock primary armour performance obtained from previous laboratory tests carried out at MHL (Reference 4). The stability of the rock toe armour was also assessed.

The results of these studies are outlined in this report.
RICHMOND RIVER

PLAN BREAKWATER HEAD

NOTES:
1. Level and contour information provided by Department of Lands is a
   CONSTRUCTION FROM A DETAILED SURVEY OBTAINED FEBRUARY 1994
2. VALUES NOTED ON SURFACE ONLY DETERMINED IN SURVEY.
   ALL LEVELS ARE
   REFERENCED TO 0.86 AHD AND ARE SUBJECT TO CHANGE DUE TO TAMING/FOSSILIZATION
3. THE TIME AND LEVEL DETAILS ARE SUBJECT TO CONFIRMATION.
4. AN UNDERWATER INSPECTION OF THE STRUCTURE INDICATES THE EXISTING TIE TO BE
   APPARENTLY STRONG.
5. IT IS RECOMMENDED TO SELECT A STRUCTURE IN THE SHAPE OF THE
   DESIGNED RIVER WITH A SHAPE FACTOR OF 12:1 (x 2) FOR STUDY.
6. SOURCE: DETAILED IN THE SURVEY OF THE
   DESIGNED STRUCTURE AMONG OTHERS.

LEGEND:
- EXTERNAL CONTINGENCY
- AREAS OF LARGE ROCKS OR COALS THAT MAY PROVE DIFFICULT TO WORK
- ACROPOLIC MOUND
- ROCK/BEACH INTERFACE
- SOURCE:

TYPICAL DESIGN CROSS SECTION THROUGH ROUNDHEAD
CHAINAGE 570 TO 580

NOTE: DESIGN CROSS SECTION SHEET
ACROSS BREAKWATER AT CHAINAGE 570.
2. The Physical Model

2.1 Model Scales

A length scale of 40 was chosen on the basis of availability of Accropode armour units, the size of the random wave flume and the need to minimise scale effects. The mass scale was determined from the length scale. The time scale was derived from the length scale using Froudian similitude. The mass scale allows for the differences in density between sea water in the prototype and fresh water used in the model study as well. The model sections were constructed using Accropode units of mass 222 grams and specific gravity 2.33 borrowed from Sogreah, the original developers of the unit.

The model scales selected for the study were:

<table>
<thead>
<tr>
<th>Scale</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Length scale</td>
<td>( L_r = 40 )</td>
</tr>
<tr>
<td>Time scale</td>
<td>( T_r = 6.325 )</td>
</tr>
<tr>
<td>Mass scale</td>
<td>( M_r = 67,567 )</td>
</tr>
</tbody>
</table>

(15 tonne prototype = 222 g in model)

2.2 The Accropode Unit

The Accropode unit was designed for application as a single layer primary armour. The rationale for the unit shape (Figure 2.1), apart from the conditions of fabrication, handling and individual strength, was that there was no need to specify a particular attitude for placement (Reference 5). Some relevant placement parameters (Reference 6) for the Accropode unit are listed below. Figure 2.2 illustrates the idealised placement of these units in a flume. Design guidelines for unit placement are also included in the abovementioned reference.

- **M1** - Horizontal mesh = \( 1.24 H_b \)
- **M2** - Mesh on the slope - distance between two horizontal rows = \( 0.6 H_b \)
- **H_b** - Height of Accropode unit
- **D_e** - Effective diameter of unit = \( 0.7 H_b \)
- **NB_\text{a}** - Placement density of Accropode units = \( 134.4/H_b^2 \) (blocks/100 m²)

*For a 15 tonne unit \( H_b = 2.64 \) m*
2.3 Model Description

The study was undertaken in the random wave flume at MHL. The flume layout is shown in Figure 2.3. Wave generation is accomplished by a sliding wedge wave paddle driven by a servo-hydraulic system. The paddle is controlled by an input signal provided by a personal computer (PC). The user defines the peak frequency of a Pierson-Moskowitz spectrum to be generated by the PC. The PC controls the data acquisition from the wave recording probes as well as the probes recording water level. A three probe reflection measurement using a method suggested by Funke and Mansard (Reference 7) was used during the tests to differentiate between the incoming wave and the reflected wave.

The floor of the flume is adjustable in panels of 1.5 m length. The floor was adjusted to a slope of 1 in 200 to represent the bathymetry of the seabed in front of the structure as in the bathymetric survey dated February 1996 (Figure 1.1). In order to accurately reproduce the profile directly in front of the structure for a distance of approximately 150 m, a compromise had to be made with the slopes of the flume further away from the structure. The slopes had to be fairly steep to have adequate depth of water to generate the larger waves required when testing the structure with simulated scour. The deeper section near the paddle rises at a slope of 1 in 11 and 1 in 15 to meet the breakwater profile discussed in Section 1. The high slope in front of the generator caused some of the extreme waves in a time series to break prematurely on this slope before being subject to depth-limited breaking at the structure. The breakwater cross-section (Figure 2.4) was built in the 10 m glass section of the flume to permit viewing from the side, photography and video filming. The test section was constructed by marking the cross-section on both sides of the flume and then placing the materials in each layer.

Extensive testing carried out by Van der Meer (Reference 8) indicates that the permeability of the core has an influence on the stability of the breakwater. Within limits discussed by Van der Meer, the more permeable the core, the higher the stability. Hence, it is necessary to simulate the actual core on which the Accropode units are to be placed. The Ballina breakwater is fairly unique in that it has a concrete cap. This would result in a relatively lower permeability. With this end result in mind, filter material was also used in the core with the equivalent of 20 tonne and 30 tonne rock mentioned in the breakwater asset study (Reference 1). Hence the core built for the testing would result in slightly more conservative criteria (or criteria for lower stability) of the Accropode units. The Accropode units were placed as instructed by Sogreah (Reference 6). The placement density is given by equation 1.

\[ NB_n = 134.4/H_n^2 \text{(blocks/100 m}^2\text{)} \]  

(1)

The height of a 15 tonne Accropode unit is 2.646 m. This results in a placement density of 19.2 blocks/100 m². The cross-section modelled in the flume had an equivalent cross-section area of 680 m². Hence, the number of Accropode units required was 130. The initial density of Accropodes was increased for subsequent tests due to concerns raised by Sogreah that the breakwater section 'looked sparse'.

Following are some of the criteria set out by Sogreah for the idealised placement of Accropode armour units. The idealised placement of Accropode units is indicated in Figure (2.2).
• Single layer placement.
• No contact between two blocks on the same horizontal row.
• Each block is interlocked between two blocks of the horizontal row below.
• Placing density between 95% and 105% of the theoretical value.

2.4 Model Design and Layout

The position of the breakwater cross-section in the flume and the bottom slopes in front of the breakwater are represented by the design diagram given in Figure 2.3. An almost flat bed (slope 1:200) was adopted in front of the breakwater cross-section. Damage was defined as a single Accropode dislodged and moved from its original location by a distance greater than its effective diameter ($D_e$). The following are the construction details of the breakwater (prototype) cross-section tested. The cross-section is shown in Figure 2.4. The distributions of the 16 tonne and 10 tonne toe armour which were tested are shown in Figure 2.5.

<table>
<thead>
<tr>
<th>Breakwater slope</th>
<th>A representative slope of 1:1.3 (V:H) was adopted.</th>
</tr>
</thead>
<tbody>
<tr>
<td>Accropode unit</td>
<td>15 tonne unit ($W_{50}$) at linear scale of 40</td>
</tr>
<tr>
<td>Rock for 16 tonne toe armour</td>
<td>$W_{50}$ size equivalent to 16.8 tonne rock at a linear scale of 40</td>
</tr>
<tr>
<td>Rock for 10 tonne toe armour</td>
<td>$W_{50}$ size equivalent to 10.3 tonne rock at linear scale of 40</td>
</tr>
<tr>
<td>Rock for 10 tonne toe armour</td>
<td>$W_{50}$ size equivalent to 10.3 tonne rock at linear scale of 40</td>
</tr>
<tr>
<td>Rock for 10 tonne toe armour</td>
<td>$W_{50}$ size equivalent to 10.3 tonne rock at linear scale of 40</td>
</tr>
<tr>
<td>Rock for 10 tonne toe armour</td>
<td>$W_{50}$ size equivalent to 10.3 tonne rock at linear scale of 40</td>
</tr>
<tr>
<td>Filter material</td>
<td>Rock of size .75-1.5 tonne at a linear scale of 40</td>
</tr>
</tbody>
</table>

2.5 Selection of Design Wave Parameters

The Ballina breakwater is located between the wave stations offshore of Byron Bay and Coffs Harbour. The offshore wave data at Byron Bay and Coffs Harbour were analysed by considering the storm duration effects (Reference 3). For this analysis a storm event is defined as a period of at least one hour when the significant wave height exceeded 3 metres. The analysis adopted the following relationship to evaluate storm wave return periods:

$$ R = \frac{1}{NP(D)} $$

where

- $R$ = return period in years
- $N$ = number of storms per year which exceed design wave height
- $P(D)$ = probability that storm duration $D$ is exceeded.
Storm return periods evaluated using the above relationship are shown in Table 2.1. As the Ballina breakwater is closer to the Cape Byron station the design 100-year wave would approximate the one obtained offshore from Cape Byron. Table 2.1 indicates that the 100-year design wave height for storms of one-hour duration is approximately 7.6 m and for storms lasting sixteen hours (equivalent of 5,000 waves in the model) is approximately 6.2 m. The effective recording period lengths for the Byron Bay and Coffs Harbour stations are 12.6 years and 15.4 years respectively.

<table>
<thead>
<tr>
<th>Duration</th>
<th>1 Year</th>
<th>20 Year</th>
<th>50 Year</th>
<th>100 Year</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Coffs</td>
<td>Byron</td>
<td>Coffs</td>
<td>Byron</td>
</tr>
<tr>
<td>1 Hour</td>
<td>5.0</td>
<td>4.9</td>
<td>7.3</td>
<td>6.7</td>
</tr>
<tr>
<td>6 Hours</td>
<td>4.5</td>
<td>4.4</td>
<td>6.5</td>
<td>5.9</td>
</tr>
<tr>
<td>24 Hours</td>
<td>3.8</td>
<td>3.7</td>
<td>5.4</td>
<td>4.9</td>
</tr>
<tr>
<td>48 Hours</td>
<td>3.2</td>
<td>3.1</td>
<td>4.4</td>
<td>4.0</td>
</tr>
</tbody>
</table>

As the offshore waves travel towards the breakwater, shoaling and refraction effects would cause changes to the wave conditions at the breakwater head as well as along the channel section. This would lead to a different statistical distribution for the wave conditions at the breakwater site from that for the offshore wave measuring site.

The inshore distribution would depend on many factors including offshore wave height, direction and period, water level and prevailing nearshore bathymetry during a storm. A detailed statistical analysis encompassing all these factors is beyond the scope of this study.

The wave climate used for these tests was representative of that produced by the major storms on the New South Wales coastline (References 2 and 3). When estimating design wave conditions the directions of hindcasts were incorporated. The analysis showed a dominant 12-second wave period (Tₚ) for the directions east and south-east. Hence, nearly all the testing was done using spectra having a peak period of 12 s. As described in Section 2.3 the increased slope in front of the generator caused some of the extreme waves in a time series to break on the slope before being subject to depth-limited breaking by the structure. This wave break on the slope may be attributed to the fact that some of the wave heights are a little lower than those measured under purely depth-limited conditions. Adequate wave height was achieved at the structure. A maximum significant wave height of 6.3 m was generated close to the wave paddle (offshore) when testing the structure with 5000 waves. This was equivalent to a 16.5-hour storm acting on the structure and represented a 100-year event acting on the structure.

2.6 Wave Facility Operation

Random waves were generated according to a Pierson-Moskowitz type spectrum. This spectrum was defined by the significant wave height (Hₛ) and the peak period (Tₚ). As indicated in Section 2.5 nearly all the testing was done using spectra having a peak period of 12 s. This peak period largely gave rise to surging breakers on the structure. The structure, with no scour hole and under the 100-year water level conditions, was also tested with an 8 s peak wave period as this results in plunging breakers for the wave height and slope tested.
2.7 Simulation of Wave Groups

The importance of examining the group nature of a wave regime has long been established in problems involving coastal structures. However in the past, coastal engineering design in NSW has generally been based on wave data without specific consideration for groupiness effects. To overcome this shortcoming, which could be significant for some designs, NSW Public Works as part of its coastal program initiated a study to investigate and compute the groupiness of wave conditions measured along the NSW coast. A groupiness factor was used to measure the grouping effect of waves measured offshore (Reference 10). The brief indicated that a qualitative study had to be carried out on the effect of grouping on the structure.

The computer controlling the wave paddle contains a file with 2048 time series values produced from the spectrum. The wave paddle is fed this signal at a time step of 50 Hz. Each time series generated has a characteristic wave grouping. This groupiness factor (GF) is difficult to quantify when waves are breaking near a structure. As the brief required only a qualitative insight into the effect of grouping, a number of time series having identical spectral characteristics (i.e. $H_s$, $T_p$) were generated and tested on the structure so as to test it under a range of groupiness factors. The grouping had little effect on the stability of the structure, but had considerable effect on the overtopping. The overtopping increased from negligible to over 10% due to the grouping effect.

2.8 Cross-Section Damage Definition

Two damage definitions have been used in breakwater research. One uses the erosion area ($S$) and requires the use of a cross-section profiler. The other definition which is used in this report relates damage ($N_o$) to the movement of an Accropode unit after testing from its position prior to testing. If the unit has moved more than the distance of one nominal diameter ($D_o$) it was considered to be damaged. For the Accropode unit the nominal diameter is equal to $0.7H_b$, where $H_b$ is the height of the Accropode. Percent damage was calculated as the number of Accropode units that had been displaced from their placed position divided by the number of Accropode units in the area that the unit was displaced from. The total number of units were considered in this report because during the testing it was evident that the wave forces (runup and drawdown) affected the total Accropode layer. Van der Meer (Reference 8) defined $N_o$ to be the fraction obtained by dividing the number of units displaced from a coloured band of width $2M$, by the number of effective diameters ($D_o$) in a width of flume. This definition is used when a large percentage of the Accropode layer is under water and hence unlikely to be damaged during testing.
Volume of block = 0.43H
Effective diameter ($D_o = .7H$)

ELEVATION A

ELEVATION B

PLAN VIEW

DIMENSIONS OF ACCROPODE BLOCK
AS FUNCTION OF HEIGHT

Source: Sogreah 1994
TOE MOUND OF NATURAL ROCK

Figure 2.2

h = height of ACCROPODE (R) block
w = weight of ACCROPODE (R) block
WAVE FLUME LAYOUT

Wave paddle, and supports

False sloping floor

Filter

Probes 1, 2 and 3

For details refer to fig 2.4

Return pipe

ELEVATION

Not to scale

30 m

1.5m

1.5m

1.5m

1.5m

1.5m

level
Figure 2.5
EQUIVALENT PROTOTYPE TOE ARMOUR SIZE DISTRIBUTION
3. Design Water Levels and Water Depths

3.1 General

The water depth in the vicinity of the breakwater has a significant influence upon the amount of offshore wave energy reaching the structure. It also influences runup, drawdown and the overtopping forces acting on a structure. It is not only the extreme high water level that affects the design but also the extreme low water level that can affect aspects such as the toe design.

The water depths to be used in this design are dependent on the Still Water Level (SWL), Mean Water Level (MWL) including wave setup and the bed levels surrounding the structure. These components were analysed to determine the effective water depth at the structure.

3.2 Still Water Level

Water level analyses conducted for Sydney using nearly 80 years of data at Fort Denison produced a design SWL curve for various return periods (Reference 11). This design curve was obtained by separately determining the frequency distribution of tidal and surge levels.

The data at Fort Denison has been checked and correlated to the limited data available at several other sites along the NSW coast, including Coffs Harbour. This analysis has shown that there is very good correlation for sites such as Crowdy Head to the north of Sydney and Batemans Bay to the south of Sydney. Further north and south of these two sites along the coast of NSW the correlation was found to be good for all practical estimates and hence Fort Denison results could be used with confidence at the Ballina breakwater site. The design SWL plot for the various return periods is shown in Figure 3.1.

The design Still Water Levels for the 1-year, 10-year, 20-year, 50-year and 100-year return intervals are summarised in Table 3.1. The extreme low water level was taken to be 0.39 m ISLW as indicated in the drawing (Figure 2.4).
Table 3.1 Design Still Water Levels

<table>
<thead>
<tr>
<th>Return Period (Years)</th>
<th>ISLW (m)</th>
<th>AHD (m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>2.20</td>
<td>1.28</td>
</tr>
<tr>
<td>10</td>
<td>2.31</td>
<td>1.39</td>
</tr>
<tr>
<td>20</td>
<td>2.35</td>
<td>1.43</td>
</tr>
<tr>
<td>50</td>
<td>2.39</td>
<td>1.47</td>
</tr>
<tr>
<td>100</td>
<td>2.43</td>
<td>1.50</td>
</tr>
</tbody>
</table>

3.3 Design Water Levels

Due to wave action, the water level along the shoreline is further increased and the resulting mean water level is a combination of the wave setup and SWL. Whilst the water level during a storm will fluctuate considerably, the mean water level is a useful parameter for assessing the stability of armour units.

The wave setup is dependent on many hydraulic and geometric factors, but most importantly on the wave height. The wave analysis outlined in studies carried out on the Ballina South and Clarence breakwater designs (References 2 and 3) indicated the one-year return design significant wave to be around 4.6 m (6-hour storm duration) and the 100-year design wave to be in excess of 7 m (6-hour storm duration). Considering the wave setup to be 10%-15% of the significant wave height, the one-year design wave setup is likely to be of the order of 0.5 m and the wave setup for the 100-year event is likely to be around 1 m.

To determine the MWL, it is necessary to combine the SWL and the wave setup. Wave setup is simulated in the flume by the generated design wave. If the above wave setup figures are assumed then the MWL for the 100-year design wave could lie between 2.0 and 2.5 AHD. It should be noted that the MWL is the average over a period of time and the actual water level at any given instance could be higher than the MWL. This is important as for a depth limited wave condition, larger waves than those computed based at MWL could occur during instances when the actual water level near the breakwater is above the MWL condition.

3.4 Water Depth

The water depth at the structure, which is the most important parameter for the determination of limiting wave conditions, is dependent on the bed level conditions.

Bed levels near the breakwater were determined from the 1996 hydrosurvey information provided by DLWC (Figure 1.1).

If the 100-year design SWL of 2.43 m (ISLW) together with 0.5 m of setup is applied to the 5.0 m bed level immediately seaward of the existing breakwater then the 100-year design water depth at the breakwater head is 8 m. Under storm conditions it is highly probable that scouring action due to wave activity could further lower the bed level in the vicinity of the breakwater.
As indicated, the bed level near the structure would vary considerably under storm conditions and could fluctuate by 2 to 3 m on the ocean side. For the physical model a maximum scour of 5 m was also simulated. Hence lower bed levels under the less frequent events would lead to higher water depths. Compared to these bed level fluctuations the differences in the MWL between the one-year and 100-year design conditions are likely to be relatively insignificant.

Due to the variability of bed levels the likely water depths for the 10-, 20-, 50- and 100-year recurrence events could not be determined to a high degree of accuracy. Hence 2 m, 3 m and 5 m scour were also simulated in the flume in addition to the extreme low water level of 0.39 m ISLW and the 100-year high water level of 2.43 m ISLW. Table 3.2 indicates the water levels tested.

Table 3.2 Water Levels Tested

<table>
<thead>
<tr>
<th>Test Series</th>
<th>Water Level (ISLW)</th>
<th>Comment</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>0.39</td>
<td>Testing toe armour</td>
</tr>
<tr>
<td>2</td>
<td>0.39</td>
<td>Testing Accropode unit stability, no scour</td>
</tr>
<tr>
<td>3</td>
<td>2.43</td>
<td>Testing Accropode unit stability, 2 m scour</td>
</tr>
<tr>
<td>4</td>
<td>2.43</td>
<td>Testing Accropode unit stability, 3 m scour</td>
</tr>
<tr>
<td>5</td>
<td>2.43</td>
<td>Testing Accropode unit stability, 5 m scour</td>
</tr>
</tbody>
</table>
Source: MHL Report No MHL621

Manly Hydraulics Laboratory
Report 795
Department of Public Works and Services

DESIGN STILL WATER LEVEL

Figure 3.1
4. Testing Procedure

4.1 Introduction

The following tests were conducted in the flume:

- stability tests on toe armour - $M_{so} = 16.8$ tonnes (quarry stone)
- stability tests on toe armour - $M_{so} = 10.3$ tonnes (quarry stone)
- stability tests on Accropode armour
- stability tests on Accropode armour - 2 m scour
- stability tests on Accropode armour - 3 m scour
- stability tests on Accropode armour - 5 m scour.

The extreme low water level for testing the toe armour was maintained at 0.39 ISLW. The 100-year water level at 2.43 ISLW was used to test the structure. Scour was simulated by building the structure up. This resulted in larger waves breaking on the structure as required by the brief.

4.2 Stability Tests

4.2.1 General

Damage measurements were made on a 1 m model width. A bedding-in test of 500 waves was performed prior to testing the cross-section without scour. The wave height was increased so as to give rise to the worst breaking conditions on the breakwater. Damage was assessed after every 1000 waves. Hence cumulative damage was assessed. Work done by Van der Meer (Reference 8) indicates damage to a section built with Accropode units is fairly independent of duration of the storm. However testing was carried out for 5000 waves to investigate this assertion. The test waves were constituted from a number of time series so as to investigate grouping effects on the structure. These 5000 waves represented a sixteen-hour storm in prototype. Hudson's equation was used to obtain the relationship between dimensionless wave height ($H_s/\Delta D_n$) and damage coefficient ($K_D$) as indicated below

$$H_s/\Delta D_n = (K_D \cot \alpha)^{1/3} \quad (3)$$

where

- $H_s = \text{significant wave height}$
- $\Delta = \text{relative mass density}$
- $D_n = \text{equivalent diameter of } M_{so} \text{ armour or } 0.7H_s \text{ for Accropode unit}$
- $\alpha = \text{angle of breakwater slope}$
5. Test Results

5.1 Stability Tests

5.1.1 Test Results - No Scour

Preliminary tests were run simulating the existing bed bathymetry seaward of the breakwater without a scour hole. Each test was run for 5000 random waves with a peak spectral period of 12 seconds. This equates to a 16.5 hour prototype equivalent storm (approximately 2.6 hours in the model). The larger waves in the spectrum broke before reaching the structure. A range of wave heights ($H_w$) was therefore used in each test to observe different breaking conditions on the structure. The number of Accropode units on the modelled design was 130 (or 105% of design value). The cross-section after testing is shown in Figure 5.1a. Significant overtopping was observed (Figure 5.1), depending on the grouping of the waves as seen in Figure 5.1.b. The test results are summarised in Table 5.1

Table 5.1 Test Results - No Scour

<table>
<thead>
<tr>
<th>Test No.</th>
<th>Water Level ISLW (m)</th>
<th>Wave Period $T_c$ (s)</th>
<th>Wave Height $H_w$ (m)</th>
<th>Comment</th>
<th>Damage*</th>
</tr>
</thead>
<tbody>
<tr>
<td>1a</td>
<td>0.39</td>
<td>12</td>
<td>2.0-3.0</td>
<td>No overtopping</td>
<td>15 t rock on toe - no damage</td>
</tr>
<tr>
<td>1b</td>
<td>0.39</td>
<td>12</td>
<td>2.0-3.0</td>
<td>No overtopping</td>
<td>10 t rock on toe - no damage</td>
</tr>
<tr>
<td>2a</td>
<td>2.43</td>
<td>12</td>
<td>3.7-4.4</td>
<td>Considerable overtopping, 10 t rock on toe (10% of waves)</td>
<td>Less than 1% damage to Accropode units</td>
</tr>
<tr>
<td>2b</td>
<td>2.43</td>
<td>8</td>
<td>4.1</td>
<td>Structure overtopped, 10 t rock on toe (&lt;10% of waves)</td>
<td>Less than 1% damage to Accropode units</td>
</tr>
</tbody>
</table>

* 0.74% damage corresponds to one unit moving

5.1.2 Test Results - With Scour

In order to have larger waves breaking on the structure, it was tested again initially simulating 2 m of scour, and then 3 m of scour and finally 5 m of scour adjacent to the structure. Once again a range of wave heights was used to simulate different breaking conditions on the structure. For this series of tests the specified number of Accropode units was increased to 136 due to concerns raised by Sogreah that the breakwater section ‘looked sparse’. The results for these tests are outlined in Table 5.2. It was observed that some of the oblique wave
forces and forces on a round head due to wave overtopping were not adequately simulated during the 2D flume testing and would be better represented in 3D basin tests.

Table 5.2 Test Results - With Scour

<table>
<thead>
<tr>
<th>Test No.</th>
<th>Water Level ISLW (m)</th>
<th>Wave Period T (s)</th>
<th>Wave Height H (m)</th>
<th>Comment</th>
<th>Damage</th>
</tr>
</thead>
<tbody>
<tr>
<td>3</td>
<td>2.43</td>
<td>12</td>
<td>4.4-4.7</td>
<td>2 m scour - considerable overtopping, damage to units at the crest (10% of waves)</td>
<td>Less than 1% damage to Accropode units</td>
</tr>
<tr>
<td>4</td>
<td>2.43</td>
<td>12</td>
<td>5.1-5.6</td>
<td>3 m scour - considerable overtopping, damage to units at the crest (10% of waves)</td>
<td>Less than 1% damage to Accropode units</td>
</tr>
<tr>
<td>5</td>
<td>2.43</td>
<td>12</td>
<td>6.3</td>
<td>5 m scour - considerable overtopping, damage to units at the crest (10% of waves)</td>
<td>Less than 2% damage to Accropode units</td>
</tr>
</tbody>
</table>

Test 5 resulted in a $K_o$ value of 7.25 (at less than 2% damage). A higher $K_o$ value may have been achieved if larger unbroken waves impinged on the structure. However as discussed earlier in Section 2.3, some of the more extreme waves were breaking on the slopes in the flume. Hence to test for higher $K_o$ values, a greater water depth or tests at larger scour depths would have to be carried out. Sogreah (Reference 1) uses a $K_o$ factor of up to 12, depending on the nature of the seabed approaching the structure.

The value for $K_o$ obtained for single layer placed rock for similar damage obtained from a previous study (Reference 4) carried out at MHL was 1.5.

5.2 Effect of Wave Grouping

Testing the structure with spectrally identical time series having different grouping characteristics had little or no effect on the stability of the structure. However, a previous study has shown that grouping effects could cause an increase of overtopping volume by two spectrally identical time series by a factor of 100 (Reference 9). In the present test series grouping effects caused an increase in overtopping in test 2a from 0 to 10% of the waves in the test series.
(a) Cross-section with no scour after completion of test
   Photo shows $T_p = 8\text{s}$ $H_s$ = depth limited

(b) Wave overtopping the structure
   Photo shows $T_p = 12\text{s}$ $H_s$ = depth limited
Cross-section with 5m scour after completion of test

Photo shows $T_p = 12s$ $H_s = \text{depth limited}$
6. Conclusions and Recommendations

Previous field inspections on the Ballina breakwater head indicate that the head extends some 70 m seaward below the water level (References 2 and 3). The field report detailed extensive crest failure. A wave climate where the larger waves break offshore but are still large enough to damage the crest by overtopping the structure was simulated in the flume during the testing.

The Ballina Accropode breakwater cross-section was subject to model waves reaching a significant wave height of 6.3 m. Whilst this resulted in some of the more extreme waves breaking offshore of the breakwater some of the larger waves reaching the breakwater broke on the structure and overtopped it.

The wave spectrum used for these tests was based on the major storms recorded on the New South Wales coastline (References 2 and 3). When estimating design wave conditions the directions of hindcast investigation were incorporated, resulting largely in a dominant 12-second wave climate for the directions east and south-east. Hence, the testing was mainly done using spectra having a spectral peak period \( T_p \) of 12 s. This resulted in surging waves breaking on the structure. An 8 s spectrum was also used to simulate the effect of plunging waves on the structure.

The damage caused to the structure was insignificant \(<2\%\) and was due to the displacement of two Accropodes on the crest of the section tested. However due to depth limitation in wave height, the estimate for the coefficient of damage \( K_D = 7.25 \) may be considered to be low. Sogreah uses a \( K_D \) factor of 12. It compares very well with that obtained in a previous study (Reference 4) for single layer placed rock \( (K_D = 1.5) \). Storm duration had little influence on damage for the wave climates tested.

Due to the significant overtopping of the structure and the fact that the 2D testing does not adequately represent the forces acting on a round head it is recommended that the round head be tested in the basin under 3D conditions.
7. References


Appendix A
List of Symbols
**List of Symbols**

\( D_n = \) Equivalent diameter of \( M_{so} \) rock armour  
\( = \) Equivalent diameter of Accropode unit = 0.7\( H_b \)  
\( GF = \) Groupiness factor  
\( H_b = \) Height of Accropode unit  
\( H_s = \) Significant wave height of time series  
\( K_d = \) Coefficient of damage  
\( L = \) Wave length  
\( M_1 = \) Horizontal mesh  
\( M_2 = \) Mesh on the slope - distance between two horizontal rows  
\( N_{B_m} = \) Placement density of Accropode units  
\( N = \) Number of storms per year which exceed wave height in design  
\( P(D) = \) Probability that storm duration \( D \) is exceeded.  
\( R = \) Return period in years  
\( T_p = \) Peak period of spectrum  
\( W_{so} = \) Mass of 50\% percentile in rock armour distribution  

\( \alpha = \) Angle of breakwater slope  
\( \Delta = \) Relative mass density