# Rosslyn Bay Breakwater, Queensland, Australia

by

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## Introduction

The Queensland Harbours and Marine Department is charged with the responsibility of providing small craft facilities throughout the State. In 1964, the Department engaged a firm of consulting engineers, Blain, Bremner and Williams Pty. Ltd. to prepare a preliminary report on possible boat harbour sites between Yeppoon and Port Alma on the Central Queensland Coast.

Five sites were evaluated according to the following criteria:-

(i) Degree of protection afforded

(ii) Tidal access

(iii) Degree of maintenance dredging anticipated

(iv) Capital cost necessary to establish the harbour and the ability to construct the harbour in stages

(v) Availability of foreshore area for development

(vi) Accessibility by road transport and to established amenities

(vii) Availability of suitable quarry material

The recommendation that the most suitable site would be at Rosslyn Bay (Figures 1 & 2) was accepted and approval to commence construction was obtained. A 105m rubble mound rock breakwater was constructed in 1968. The breakwater was extended to 210m in 1970 and was further extended to 300m in 1972.

In 1976 tropical cyclone 'David' extensively damaged the breakwater and harbour facilities. Subsequently the breakwater has been redesigned following model studies and reconstruction was completed in May 1978.

This paper discusses damage to the breakwater from wave and surge action, model studies and repair of the breakwater.

## Original Design

The area on the Central Queensland Coast between Port Alma and

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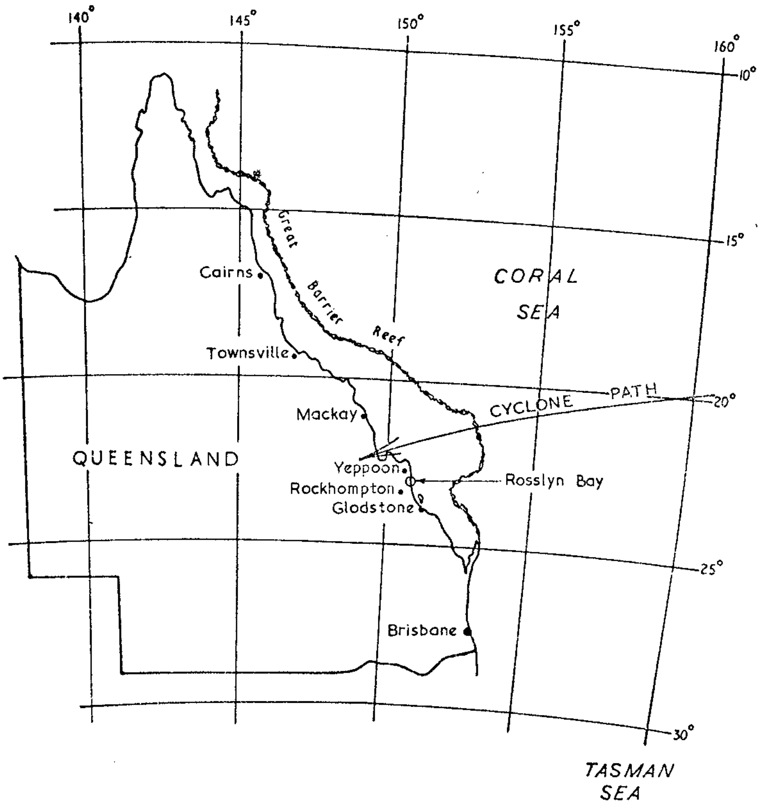



FIGURE 1: LOCALITY SKETCH AND PATH OF CYCLONE DAVID.

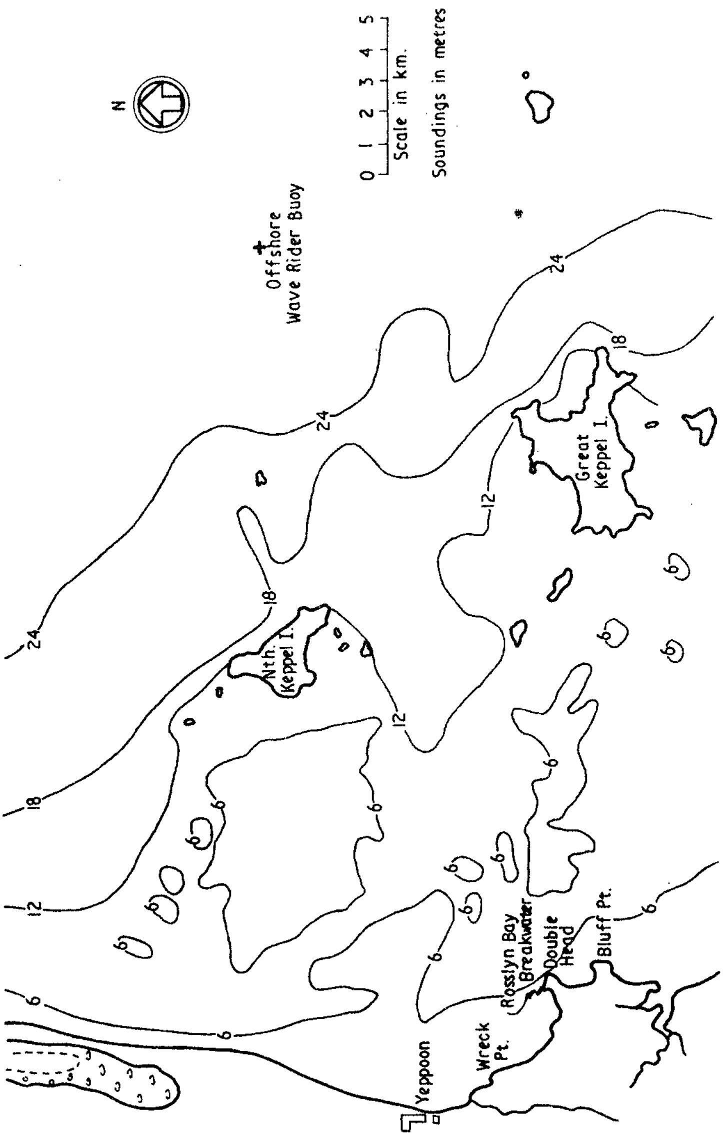



FIGURE 2: OFFSHORE TOPOGRAPHY AND LOCATION OF WAVE RIDER BUOY

Yeppoon, although sheltered from deep ocean swell by the Great Barrier Reef, has no natural boat havens (except for tidal estuaries) and is subject to cyclonic storms. The fetch length from the Great Barrier Reef to the coast is approximately 80 km. As any structure is subjected to large wave forces, construction of small craft harbours in the region is relatively costly. Tidal estuaries in the area (indeed on the whole of the eastern coast of Queensland) are generally avoided for boat harbour development because of the high frequency of major floods with accompanying high water levels and flow velocities, debris and siltation problems. However, they are used by small craft as shelter during cyclones.

Due to the limitation of available funds, economic considerations were a major factor in fixing the design of the harbour.

Double Head (Figure 2) provides a degree of natural shelter against the prevailing south-easterly winds. To provide protection against north-easterly weather and south-easterly waves diffracted around Double Head, a breakwater was constructed running parallel to the natural contours in a north-westerly direction. In order to reduce costs, the breakwater was designed to the minimum standards necessary to provide shelter against the majority of anticipated weather conditions. To construct a breakwater designed to provide protection during severe cyclonic conditions proved too costly for the available funds. This factor, together with the availability close by, of effective cyclone shelter to small craft in mangrove lined estuaries led to the decision to accept a higher than normal degree of risk of severe storm damage to the breakwater.

The following design criteria were adopted for the breakwater design:

| | |
|---|---|
| High tidal level | 4.27m above L.W.D. |
| Storm surge | 1.22m |
| Wave height | 2.13m |
| Crest level | 7.32m above L.W.D. |

A typical section through the breakwater and armour specification is shown in Figure 3. The seaward face was armoured with a nominal 3 ton rock. The crest was armoured with nominal $\frac{1}{2}$ ton rock blinded with fine material to provide an access road and was not designed to resist overtopping.

As there were no wave records for the area, the design storm surge and wave heights were estimated by hindcasting using available wind records. In Jan. 1967 the Consulting Engineer estimated from the available cyclone records that damage to the breakwater may occur once in every 10 years and such damage was likely to be severe. As it happened, the breakwater was severely damaged by overtopping some 8 years after construction of its first section.

MODEL MATERIAL USED

| | | |
|---|---|---|
| CLASS A | 75% | 120 to 140 grams |
| | 20% | 60 to 120 grams |
| | 5% | 20 to 60 grams |
| CLASS B | 35% | 20 to 40 grams |
| | 35% | 10 to 20 grams |
| | 25% | 2 to 10 grams |
| | 5% | 0·5 to 2 grams |
| FILTER | 30% | 20 to 60 grams |
| | 55% | 10 to 40 grams |
| | 15% | 6 to 3 mm |
| CLASS C | 60% | 6 to 3 mm |
| | 25% | 3 to 1 mm |
| | 15% | < 1mm |

PROTOTYPE MATERIAL

| | | |
|---|---|---|
| CLASS A | 75% | 3 ton |
| | 20% | 1½ to 3 ton |
| | 5% | ½ to 1½ ton |
| CLASS B | 35% | ½ to 1 ton |
| | 35% | 500 lbs. to 1000 lbs. |
| | 25% | 100 lbs. to 500 lbs. |
| | 5% | 15 lbs. to 100 lbs. |
| FILTER | 30% | ½ to 1 ton |
| | 55% | 100 lbs. to 1000 lbs. |
| | 15% | Coarse Class C |
| CLASS C | 60% | 6" to 3" |
| | 25% | 3" to 1" |
| | 15% | Minus 1" |

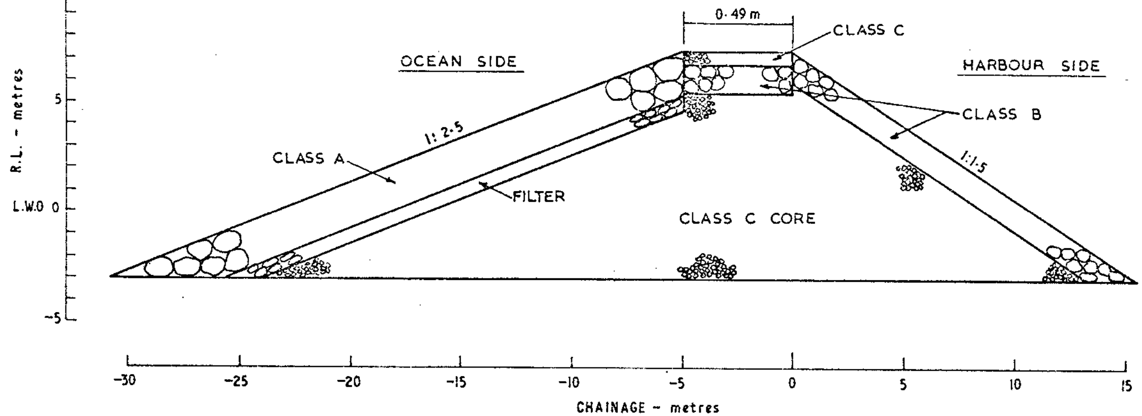



FIGURE 3: TYPICAL CROSS SECTION OF BREAKWATER

Cyclone "David"

Cyclone "David" originated in the Coral Sea on 15.1.76, moving first in a south-south-westerly direction and then west to cross the coast approximately 50 km north of the harbour at a speed of 28 km/hr. The path of the cyclone is shown in Figure 1. Central pressure as it crossed the coast was 961 mb and the radius to maximum wind was estimated by the Bureau of Meteorology to be 35 km using radar interpretation methods.

Associated with the cyclone, there was an intense high pressure system (central pressure 1010 to 1024 mb) located in the Tasman Sea some 1500 km to the south-south-east. The consequential steep pressure gradient resulted in a general strengthening of the south-east winds which had persisted since 15th January. This climatic situation is assessed as having had a significant influence on the abnormally long duration of the storm surge which accompanied the cyclone.

Deep water wave conditions during the cyclone were recorded at 12 hr. intervals on wave rider buoys at Double Island Point, 390 km to the south, Mackay 270 km to the north and Yeppoon immediately offshore of the harbour. Recorded wave conditions are given in Table 1.

Water levels during the cyclone were recorded on a network of tide gauges along the Queensland coastline. Storm tide levels at Rosslyn Bay and Gladstone are given in Table 2. Unfortunately the tide gauge at Rosslyn Bay was inoperative over part of the storm but a close estimate of water levels can be obtained from the Gladstone records.

A feature of the storm was the long duration of the surge which reached a maximum of 1.1m during the p.m. tide on 19.1.76. The peak water level of 5.3m occurred during the a.m. tide on 19.1.76.

Wave and storm tide conditions during the cyclone are summarised in Figure 4. Wave conditions at the breakwater are complicated by partial protection provided by two offshore islands approximately 15 km to the east and north-east of the harbour and refraction over a rather complex bed topography (see Figure 2).

From energy considerations wave heights at the breakwater have been estimated to be between 0.8 and 0.9 of the deepwater conditions. Inshore visual observations by two independent observers, one by Bremner and one by COPE,* during the peak of the storm estimated wave heights of 2.5 and 3.3m against a measured deep water significant height of 3.8m.

Wave conditions following the cyclone and prior to a detailed survey of

*COPE Coastal Observation Programme Engineering - A Beach Protection Authority, Department of Harbours and Marine, Visual Data Acquisition Programme

Table 1: Wave Data 'Cyclone David'

| | | Double Is. Point | | | Yeppoon | | | Mackay | | |
|---|---|---|---|---|---|---|---|---|---|---|
| Date | Time | H. sig. | Tz. | Tp. | H. sig. | Tz. | Tp. | H. sig. | Tz. | Tp. |
| 16 | 0300 | 0.90 | 7.0 | - | 0.50 | 4.75 | - | 0.52 | 2.80 | - |
| | 1500 | 1.75 | 4.65 | - | 1.29 | 4.60 | - | 1.23 | 3.50 | - |
| 17 | 0300 | 4.00 | 6.90 | - | 2.63 | 6.18 | - | 2.15 | 5.18 | - |
| | 1500 | 4.19 | 7.03 | 9.10 | 3.00 | 6.60 | - | 2.20 | 4.57 | - |
| 18 | 0300 | 4.60 | 7.60 | 9.90 | 3.15 | 6.56 | - | 1.90 | 4.87 | - |
| | 1500 | 5.80 | 8.32 | 10.70 | 3.56 | 7.25 | - | 1.87 | 5.10 | - |
| 19 | 0300 | 5.76 | 8.46 | 11.40 | 3.79 | 7.59 | - | 2.50 | 5.60 | - |
| | 1500 | 5.36 | 7.80 | 10.70 | 3.79 | 6.61 | 8.70 | 2.88 | 6.02 | 11.90 |
| 20 | 0300 | 4.82 | 8.24 | 10.60 | 3.84 | 7.74 | 9.60 | 2.41 | 5.32 | 6.40 |
| | 1500 | 3.14 | 7.19 | 9.80 | 2.03 | 5.67 | 7.60 | 1.66 | 4.57 | 5.60 |
| 21 | 0300 | 2.52 | 6.17 | 8.60 | 1.25 | 6.10 | - | 1.12 | 4.00 | 4.30 |
| | 1500 | 2.45 | 6.00 | 9.10 | 1.09 | 5.63 | - | 0.70 | 4.18 | 9.40 |
| 22 | 0300 | 2.11 | 5.69 | 8.50 | 1.38 | 5.39 | - | 0.80 | 3.90 | 9.50 |
| | 1500 | 2.15 | 5.69 | 8.00 | 1.90 | 5.27 | - | - | - | - |

Jan. 1976

H-sig - Significant wave height in metres = $H^{1/3}$
Tz. - Zero crossing period in seconds
Tp. - Peak energy period in seconds (where available)

Table 2: Tide Records during Cyclone 'David'

| Station | | | Rosslyn Bay | | | Gladstone | | |
|---|---|---|---|---|---|---|---|---|
| Date | Time | Tide | Ht. | Pre-dicted | Resid-ual | Ht. | Pre-dicted | Resid-ual |
| 16.1.76 | a.m. | L | | | | 0.7 | 0.8 | -0.1 |
| | a.m. | H | | | | 4.4 | 4.4 | 0.0 |
| | p.m. | L | | | | 0.9 | 0.8 | +0.1 |
| | p.m. | H | | | | 3.8 | 3.7 | +0.1 |
| 17.1.76 | a.m. | L | | | | 0.9 | 1.2 | +0.3 |
| | a.m. | H | | | | 4.6 | 4.5 | +0.1 |
| | p.m. | L | | | | 0.9 | 0.7 | +0.2 |
| | p.m. | H | | | | 3.9 | 3.6 | +0.3 |
| 18.1.76 | a.m. | L | | | | 0.9 | 0.5 | +0.4 |
| | a.m. | H | | | | 4.7 | 4.5 | +0.2 |
| | p.m. | L | | | | 1.0 | 0.5 | +0.5 |
| | p.m. | H | | | | 4.1 | 3.8 | +0.3 |
| 19.1.76 | a.m. | L | | | | 1.1 | 0.6 | +0.5 |
| | a.m. | H | | | | 5.1 | 4.5 | +0.6 |
| | p.m. | L | 1.5 | 0.5 | +1.0 | 1.7 | 0.6 | +1.1 |
| | p.m. | H | 4.9 | 3.9 | +1.0 | 4.6 | 3.6 | +1.0 |
| 20.1.76 | a.m. | L | 1.0 | 0.6 | +0.4 | 1.3 | 0.7 | +0.6 |
| | a.m. | H | 4.9 | 4.6 | +0.3 | 4.7 | 4.3 | +0.4 |
| | p.m. | L | 0.8 | 0.6 | +0.2 | 0.9 | 0.9 | 0.0 |
| | p.m. | H | 3.9 | 3.9 | 0.0 | 4.0 | 3.8 | +0.2 |

Notes:

1. Tide levels at Rosslyn Bay are based on Low Water Datum
2. Tide levels at Gladstone are based on Chart Datum
3. All units expressed in metres

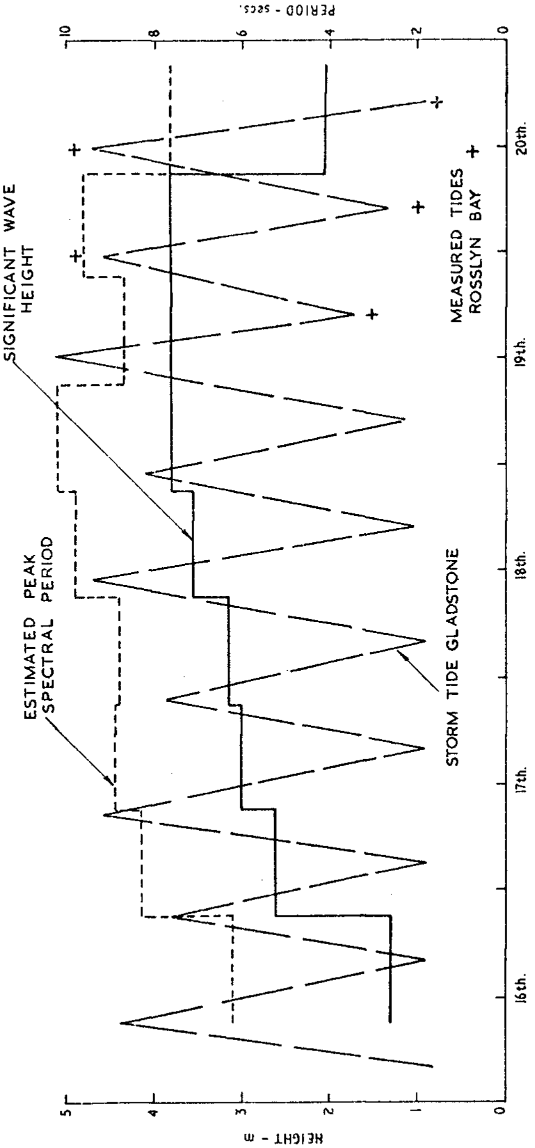



FIGURE 4: WAVE AND STORM TIDE CONDITIONS

the damage were mild with significant wave heights of less than 2m and zero crossing period of between 4.0 and 5.0 seconds.

Breakwater Damage

Inspection of Figure 4 shows that the breakwater was subjected to wave and surge conditions approaching or exceeding the design conditions from 17th to 20th January. Overtopping of the breakwater occurred at high tide with very heavy overtopping being observed during 19th and 20th. The observed history of damage is given in Table 3.

Table 3: History of Damage

| Day Jan. 1976 | Time (hours) | Damage |
|---|---|---|
| 16th | 2230 | No observable damage |
| 17th | 0900 | No observable damage |
| 17th | 2100 | No observable damage |
| 18th | 0950 | Slight damage |
| 18th | 1630 | Slight damage |
| 19th | 1130 | Heavy overtopping - slight damage |
| 19th | 2400 | Heavy overtopping - minor damage |
| 20th | Early hours | Major failure |

Major damage to the breakwater occurred at or soon after the evening high tide on 19th. At this time the breakwater failed catastrophically with the crest being destroyed and lowered within a few hours by some 4m over most of its length. The majority of the rock was displaced landwards coming to rest immediately on the harbour side. There was little damage to the seaward face.

A survey of damage to the breakwater was undertaken 20 days following the failure as shown in Figure 5.

Some comments on the damage are worthy of note. The damage resulted from wave overtopping, the majority of the rock being displaced landwards. When damage occurred it was catastrophic, taking place over a few hours. Despite high combinations of wave and storm tide levels and heavy overtopping little damage was noted prior to final collapse. It is difficult to believe that the class C or B material at the crest (Figure 3) would withstand any significant overtopping as was observed to be the case (see section on model simulation of failure). It is possible that as a result of compaction under road traffic the crest acted as an impervious scour blanket giving protection until incipient failure occurred, after which total failure followed quickly. Damage was very uniform over the entire length of the breakwater except at one section where a wharf abutment on the harbour side acted as a buttress. At this location damage was slight. Little damage occurred to the seaward face for deep water wave heights of up to

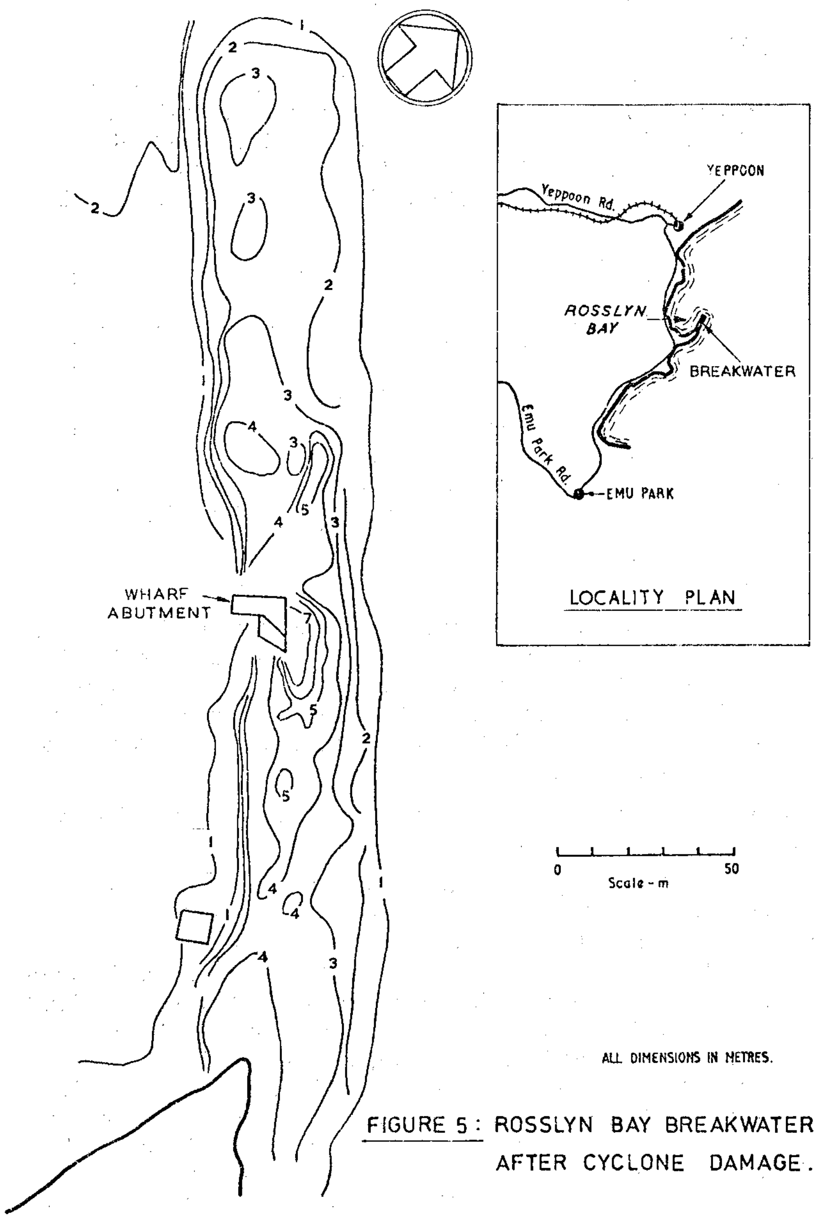



FIGURE 5: ROSSLYN BAY BREAKWATER AFTER CYCLONE DAMAGE.

3.8m and estimated wave heights at the structure of between 3.0 and 3.4m. Equivalent damage coefficients in Hudson's equation are approximately 2.7 to 3.9 and wave conditions varied between breaking at low tide and non-breaking at high tide.

After failure the structure continued to give substantial protection by acting as a partially submerged breakwater, significantly reducing damage to the harbour infrastructure during the storm and enabling the harbour to be used for its design function under the more common weather conditions that followed. The action of prudent yachtsmen in removing their vessels from the harbour to nearby natural shelter at the onset of the cyclone resulted in the damage to moored vessels being not too severe.

Model Simulation of Failure

As part of the model studies to investigate methods of repairing the breakwater, tests were undertaken to simulate the failure. The test layout for the model is shown in Figure 6. The studies were carried out in a monochromatic wave flume using Froudian scaling. The model scale was 1:27.6. Test conditions reproduced were, offshore significant wave height (approximately $H_{10}$ at the structure), peak spectral period and storm tide assuming a linear variation between high and low water. Typical test results are shown in Figure 7.

The test began at 2230 hours on 16th (low tide) and was continued until 0700 hours on 20th (low tide). Initial damage to the breakwater occurred at 0900 hours on 17th when the crest was destroyed in a landward direction.

Damage continued during each period of high tide when the crest was overtopped. The main damage was the result of overtopping and only a small proportion of the armour was displaced seaward.

At the completion of the test series the structure was subjected to continuous wave attack under stationary conditions corresponding to a wave height of 3.8m, wave period of 1.8s and a storm tide level of 5.1m. Little additional damage occurred indicating that near equilibrium conditions had been reached.

Repeating the test gave almost identical results both as to incidence of damage and the final profile.

The mechanics of damage in the model and prototype were the same with the majority of damage resulting from rock being displaced from the crest and settling on the immediate harbour side. No significant damage occurred to the seaward face. However, the model differed from the prototype in that initial failure happened much earlier. As soon as the structure was significantly overtopped class C and B material was removed and the crest destroyed. (See comment on prototype damage). Damage in the model continued over five high tide

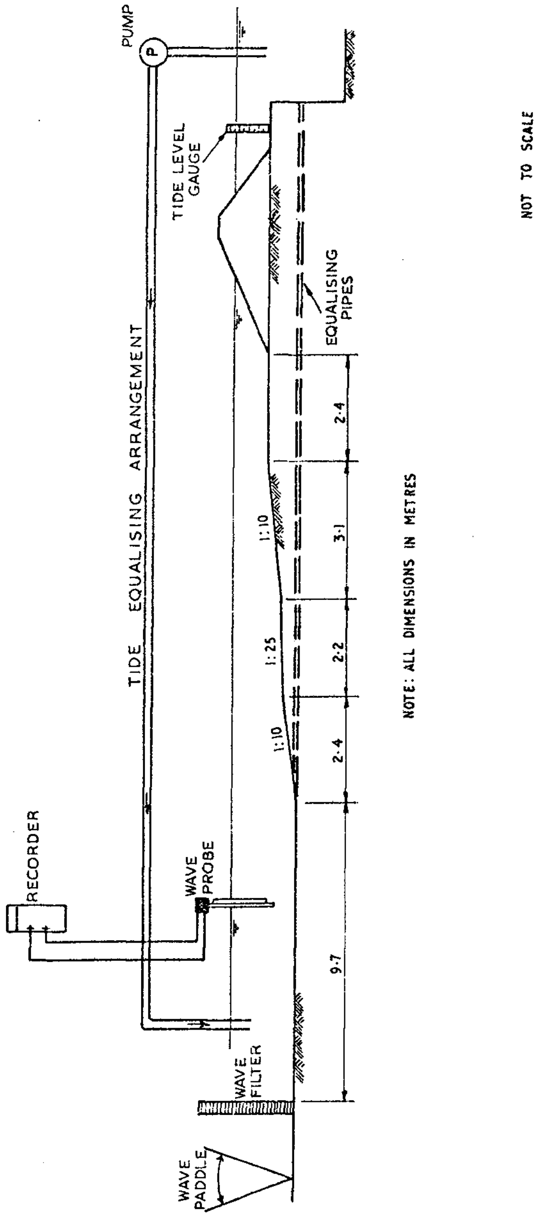
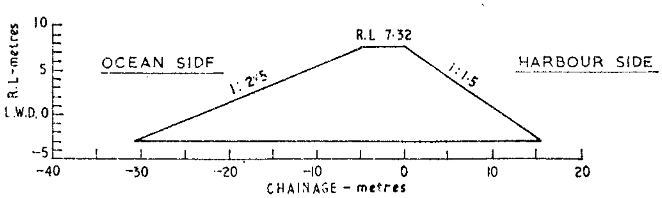
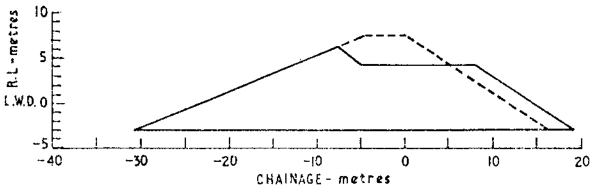


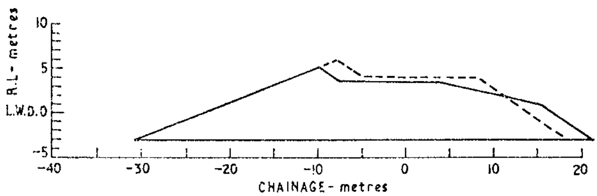

FIGURE 6: MODEL DETAILS



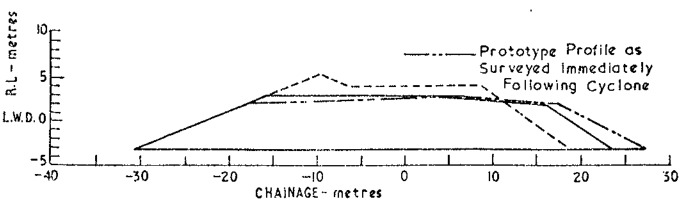

(1a) BREAKWATER AS CONSTRUCTED
2230 hr. 16th. Jan.




(1b) BREAKWATER PROFILE
0900 hr. 17th. Jan.




(1c) BREAKWATER PROFILE
1030 hr. 18th. Jan.




(1d) BREAKWATER PROFILE
2300 hr. 19th. Jan.

FIGURE 7: MODEL SIMULATION OF FAILURE

periods whilst the prototype damage occurred predominantly under one tide. The rate of damage (Froudian scaling) was somewhat slower than the prototype. This was somewhat surprising (especially as the $H_{10}$ height was used in the tests) as comparison between random and monochromatic wave tests on non-overtopped structures tend to indicate a faster rate of damage under monochromatic waves. It is possible that the higher waves in the spectra and wave period play a more significant role in the stability of overtopped structures.

The final profile reached in the model showed close agreement with the prototype. However, it is not known whether further damage to the prototype would have occurred if the storm had continued, whilst the final model profile was in equilibrium with the wave and surge conditions.

## Modified Design

A section through the modified design is shown in Figure 8. The repair made as much use of the failed breakwater as possible. The design wave height was increased to 4.6m and the maximum storm tide level to 8.5m. The seaward face was flattened to 1 on 4 and the landward face to 1 on 2. Armour mass was increased to 5 tonne nominal and was carried over the crest and down the landward face to allow for overtopping.

Model tests indicated that the proposed design would be satisfactory although some damage could be expected, particularly at the crest under critical combination of wave period and storm tide conditions. A typical set of test results is shown in Figure 9. The tests were undertaken in a monochromatic wave flume using the design offshore significant wave height. Storm tide levels were increased in steps of 1m from RL 3m up to the maximum design value of 8.5m and then decreased in a similar manner. At each water level the structure was subjected to waves for an equivalent prototype duration of 82 minutes. The test was repeated over a range of wave periods of between 9 and 16 seconds. Rock displaced was counted as damage and expressed in terms of a percentage of the total number of armour rock. It will be noted that the rate of damage (particularly to the crest) is dependent upon both wave period and storm tide level. Maximum damage to the seaward face occurred for a wave period of 12 seconds and to the crest for a wave period of 14 seconds. At very high surge levels the crest is submerged and wave forces are reduced by a water cushion between the wave and the armour and damage is reduced. For the same reason it may be that a steeper landward face may have been more stable than the 1 in 2 slope used. However, this was not investigated in detail. Test results shown in Figure 9 are for random placed stone. By careful placement of the crest rock a substantial increase in stability could be obtained.

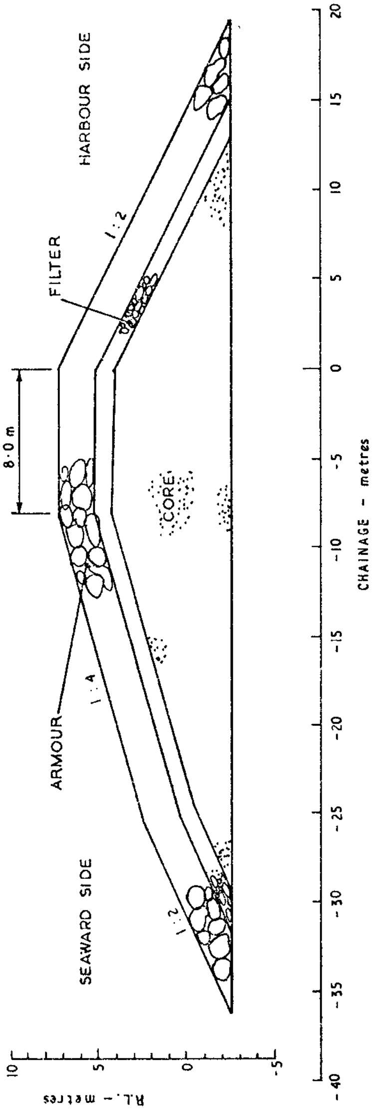



FIGURE 8: REPAIRED SECTION AS MODELLED

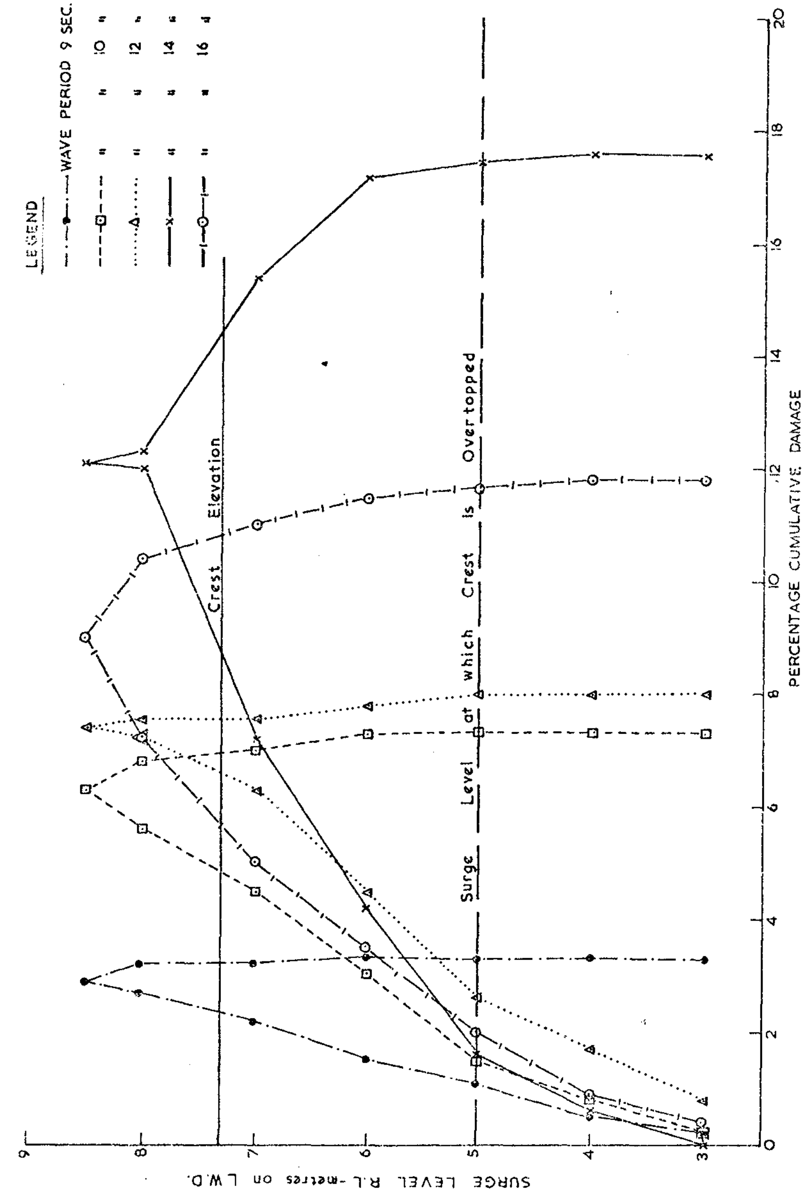



FIGURE 9 : DAMAGE AS A FUNCTION OF SURGE LEVEL FOR WAVE PERIODS FROM 9 - 16 SECONDS

## Breakwater as Constructed

After the quarry was opened up the yield of 5 tonne rock was less than anticipated. For this reason the crest was actually armoured with 3.2 tonne modified cubes and the front and back slopes with a random mixture of 5 tonne rock and 3.2 tonne modified cubes on an approximately fifty-fifty basis. Repairs to the breakwater have been completed but as yet the structure has not been subjected to significant storm action.

## Acknowledgement

The permission of the Director, Department of Harbours and Marine, Queensland, to publish this paper is gratefully acknowledged.