During the peak segment of the storm (Hs = 5.8 m), the observations were similar to the previous segment with the motion generally restricted to "rocking-in-place".

During the sixth segment (Hs = 5.2 m) and later segments the motion was considerably reduced compared to that observed with the same wave heights on the "up side" of the storm. When the design storm was repeated the structure was observed to be considerably more stable - in the sense that less motion was observed - than noted in the first storm.

The question of the durability of the stones was given consideration during the development of the design. Clearly the stones used in the model were considerably more durable than those used for construction. Principally this problem was addressed by testing the model with a gradation of reduced stone sizes as an approximation of an in-place gradation with broken stones. However, it was also observed that only a small volume of the placed armour stones actually moved or rocked and it is doubtful that the stability of the breakwater is dependent on the durability of these stones.

The validity of the physical modelling process was also questioned during the development of the design. Because of this concern all tests were completed with a large model (scale of 1:35) in a wide basin with irregular waves. Various Reynolds number criteria published in the literature, were satisfied and it was determined that the physical model would provide conservative results. This conclusion has subsequently been confirmed by undertaking additional tests with identical models at scales of 1:7 and 1:35.

THE DESIGN AND CONSTRUCTION OF A MASS ARMOURED BREAKWATER AT HAY POINT, AUSTRALIA

by

W. Bremner, Dr. B.A. Harper
and
Prof. D.N. Foster

Abstract

A mass armoured breakwater is defined as a rubble-mound structure that is designed and built in an initially unstable form, but with sufficient material provided to allow natural forces to modify its shape to a stable profile. During the whole process, the breakwater continues to perform its design function.

In this paper the design and construction of a prototype breakwater of this type is briefly described. The paper also explores the practicality of allowing the progressive interaction of design, physical model testing of design and construction to overcome the difficulties often encountered in the construction of rubble-mound structures when the rock source is undeveloped before construction is commenced. The results of the model testing, although somewhat limited to the solution of the specific problem, are further analysed to explore the relativity of high permeability in the design of structures of this type. This has resulted in the development of a computer model (HARBREM) which may be used as an initial design tool in the selection of armour sizes for physical testing. The development of this model is briefly described in the paper together with an example of model input and output.

Résumé

Un brise-lames à carapace en vrac est défini comme étant un ouvrage en enrochement conçu et construit suivant une forme initialement instable, mais avec suffisamment de matériaux pour permettre aux forces naturelles d'en modifier la forme jusqu'à l'obtention d'un profil stable. Pendant tout le processus le brise-lames continue de jouer le rôle pour lequel il a été conçu.

Dans cette étude, la conception et la construction d'un prototype de brise-lames de ce genre sont brièvement décrites. L'étude explore également les possibilités d'interaction progressive de la conception, des essais sur modèle physique et de la construction pour surmonter les difficultés souvent renontrées lors de la construction d'ouvrages en enrochements lorsque la source de roche n'est pas mise en valeur avant le début de la construction. Les résultats des essais sur modèle, quoique quelque peu limités à la solution du problème spécifique, sont davantage analysés afin d'explorer le caractère relatif d'une perméabilité élevée lors de la conception d'ouvrages de ce type. Cela a permis la mise au point d'un modèle informatique (HARBREM) qui peut être utilisé comme instrument initial de conception pour le choix des dimensions des blocs de carapace pour les essais physiques. La mise au point de ce modèle est brièvement décrite dans l'étude qui fournit également un exemple d'entrées et de sorties du modèle.

THE DESIGN AND CONSTRUCTION OF A MASS ARMOURED BREAKWATER AT HAY POINT, AUSTRALIA

ABSTRACT

A mass armoured breakwater is defined as a rubble-mound structure that is designed and built in an initially unstable form, but with sufficient material provided to allow natural forces to modify its shape to a stable profile. During the whole process, the breakwater continues to perform its design function.

In this paper the design and construction of a prototype breakwater of this type is briefly described. The paper also explores the practicality of allowing the progressive interaction of design, physical model testing of design and construction to overcome the difficulties often encountered in the construction of rubble-mound structures when the rock source is undeveloped before construction is commenced. The results of the model testing, although somewhat limited to the solution of the specific problem, are further analysed to explore the relativity of high permeability in the design of structures of this type. This has resulted in the development of a computer model (HARBREM) which may be used as an initial design tool in the selection of armour sizes for physical testing. The development of this model is briefly described in the paper together with an example of model input and output.

1.00 DESIGN ORIGIN

In cyclone "David" in January 1976, a conventional rubble mound breakwater was severely damaged at Rosslyn Bay. This damage was closely observed by the author of this design. Blain Bremner & Williams Pty Ltd (BBW) were commissioned to re-design this breakwater and supervise its construction. From experienced observation at Rosslyn Bay and subsequent model testing of the redesigned breakwater for Rosslyn Bay, it became clear why the damaged breakwater continued to protect the harbour. It was the first time (as far as is known) that a physical model was tuned to fail in the same way as the prototype (ICCE Hamburg 1978). Engineers have known for a long time that the empirical Hudson equation used in the design of breakwaters takes no account of wave period, wave grouping, wave direction, wave reflection and the real effects of permeability of the armour layers. The importance of wave grouping, in terms of damage to gravity type breakwaters, was first explored in a paper by J Ploeg (ICCE Hamburg 1978).

Consequently, designers of rubble mound type breakwaters rely heavily on their experience. Because of the influence of the Hudson equation and its use in the last twenty years or so, there is an increasing number of artificial units of complex and distorted shapes that are now commonly in use as armour units. These are propagating at the rate of a new unit every year or so. The design philosophy of these units is that because of their geometric shapes they have increased mechanical interlock forces as well as gravity forces giving higher stability factors for any given mass. Physical model tests frequently bear this out. It should be emphasised that the remarkable work by Hudson was never meant to be applied to armour units that rely for their stability on a large measure of interlock with one another. It is well known from the study of many breakwater failures around the world that the effects of mechanical interlock are very much influenced by wave period, wave grouping, wave direction and wave reflection. Failure modes of these units in armour layers are both unpredictable and frequently catastrophic. There is a body of experienced opinion that if suitable natural rock is not available, the only acceptable artificial armour material is in the form of concrete cubes or prisms placed in a random fashion so that the permeability of the armour layers is maximised. These armour units behave predictably under all conditions.

It seems that it is nature's way that artificial harbours more often than not are backed by low level coastal plains that are usually remote from good sources of natural rock. A notable recent exception to this is at Sines where good natural rock was adjacent to the site, but where artificial armour units were used and suffered catastrophic failure.

The use of any artificial concrete armour unit is usually very much more expensive than natural rock. The problem in Queensland and many other areas of the world for that matter, is that natural rock greater than 5 to 6t is difficult and costly to obtain. Natural rock of larger size usually has a very low yield from rock formations and from normal quarrying procedures.

This has been recognised for some years and the experience at Rosslyn Bay introduced the idea of using commonly available rock sizes, intermixed with modified concrete cubes with a grading that has the highest possible permeability, placed in a cross-sectional shape that eliminates or reduces the use of a crane and maximises the construction by end tipping the rock with a minimum amount of trimming by dozer and backhoe. The construction cross-section is designed so that natural wave action will reshape the seaward slope to the stable "S" shape found in nature.

This is also the basis of the design of the extension of the eastern breakwater at Townsville. This breakwater is designed to protect a reclaimed area including a container terminal, bulk gas terminal and similar port facilities. In this case, the breakwater is seawards of a revetted reclaimed area and separated from it by a stretch of open water. When the breakwater is reshaped to a stable configuration by nature, it becomes a partially submerged structure and as such, is capable of attenuating the incident wave height by a factor of at least 0.5. The revetment is designed to withstand forces due to this transmitted wave. This structure has a very high tolerance to waves very much greater than the design wave height.

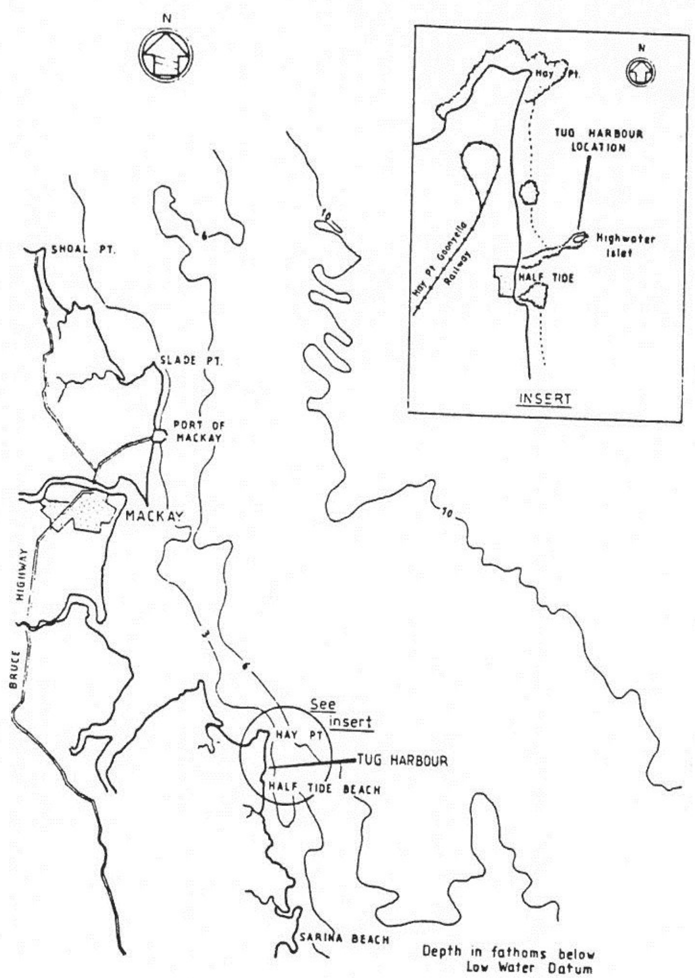
2.00 TUG HARBOUR AT HALF TIDE

At present, ships mooring at the Hay Point Coal Loader are assisted by tugs which operate from Mackay which is nearly 20km away from the coal loader terminal. To improve the efficiency of ship berthing at the Hay Point Coal Loader it was decided to construct a tug harbour at the High Water Islet near the town of Half Tide. This will enable the tugs to operate from a shelter which is only 3km away from the coal loading facilities at Hay Point (Figure 1).

In the past, several design proposals for the construction of a tug harbour at Hay Point have been considered. In 1977 the Department of Harbours and Marine, Queensland proposed a tug harbour design consisting of two breakwaters each approximately 1km long. This design was model tested at the Water Research Laboratory, but the construction of the breakwater was not carried out. More recently a second design which consisted of a single breakwater approximately 350m long was proposed by the Department of Harbours and Marine, Queensland. This design consisted of a conventional breakwater having two layers of 12 tonne dolosse as primary armour on the seaward and leeward faces and 40 tonne concrete blocks on the crest. This design was model tested at the Department of Harbours and Marine's Laboratory at Deagon Queensland.

Preliminary investigations and trial blasts at a nearby quarry site at Mount Griffiths, which is 2.7 km from the harbour site, had shown that the maximum rock size available was of the order of 2 to 3 tonnes. Rocks of this size were then judged to be completely inadequate as primary armour in a conventional breakwater design.

Hence the Department of Harbours and Marine selected concrete dolosse and concrete cube units as the primary armour in the design.



LOCALITY PLAN

FIGURE 1

However, following further investigations of the abandoned quarry at Mount Griffiths by a joint venture of the port users, Utah Development Company, Dalrymple Bay Coal Terminals Pty Ltd, in consultation with the Department of Harbours and Marine, the presence of more massive rock in andersitic dyke swarms was discovered. This led to the possibility of a redesign of a breakwater which could maximise the use of local material including large quantities of sound but heavily fractured rock mined in the process of extracting armour rock.

Blain Bremner & Williams Pty Ltd, (BBW) were engaged and developed a solution to meet this criterion which indicated a significant cost saving over all previous proposals and involved the design of a new breakwater section consisting mainly of mass armour stone. This is different from the conventional breakwater sections which normally uses only two layers of armour. It was also thought that by careful selection and blasting, the quarry may possibly produce some larger armour units. Hence the initial design of the mass armour breakwater section was based on rock units ranging between 3 to 7 tonnes.

The Half Tide breakwater completed in March 1987, embodies the collective experience of Grassy Island, BBW at Rosslyn Bay and Townsville and is designed on the basis that it will reshape under natural forces to a stable "S" shape. Physical model tests confirmed that this design results in a stable structure. This design also allows easy future maintenance of sections of the breakwater where reshaping by nature may be greater than in other sections. This usually occurs in local areas where there is a change of direction in the breakwater. This is often encountered in natural rock and shingle beaches for the same reason.

To test the performance of the proposed new mass armour breakwater and its stability under the design wave conditions, BBW requested Unisearch through the Water Research Laboratory to carry out hydraulic model studies to test the design.

From the earliest stages of models testing of Rosslyn Bay and Townsville breakwaters, the outstanding feature of this type of design was its extraordinary tolerance to wave forces very much higher than the design forces. It is this feature that has excited the interest of the Engineers who are the Authors of this paper.

The Half Tide breakwater during the model testing has been subjected to wave heights much greater than those depth-limited waves that can occur naturally on the site at Half Tide.

This breakwater design therefore, has the following basic characteristics:

1. It proposes the use of natural rock in commonly occurring quarried sizes.
2. It reaches stability by reshaping by natural forces.
3. It has a very high tolerance of forces very much larger than the design forces.

4. It is capable of very easy and relatively inexpensive maintenance by end tipping the rock. It also can be readily increased in height and length by the same method.
5. It results in large savings in capital cost against any other known design solution.

The variation in stability between a conventional two layered structure and a mass armour structure can be attributed mainly to the difference in the permeability of the two structures and the volume of material used. This results in:

i) Lower wave run up
ii) Lower wave reflection
iii) Increased absorption of wave energy within the body of the structure
iv) Higher transmission of wave energy to the leeward side
v) Reduction of breakwater slope following damage.

All these above factors were clearly evident during the testing of the mass armour breakwater.

It is well known that as the water level fluctuates on the face of the structure between run up and run down levels, a higher run up level causes steeper pressure gradients. This leads to higher seepage and draw down forces on the armour units. Further, for a given wave period, the velocity of water rushing down the face of the structure increases with increasing run up. As the drag force on the armour units is a function of the square of the fluid velocity, increasing run up will also increase the drag forces. Hence by reducing the run up, the stability of the armour units is improved by reducing both seepage and drag forces. Another influence not yet investigated is the influence of the compression of entrapped air in the interstices of the armour rock.

The wave height resulting at the structure is a combination of both the incident and the reflected waves. As the reflected wave height is reduced, the wave height acting on the structure is also reduced leading to a more stable structure.

The reflection from a conventional two layered breakwater with a similar sloping seaward face may is of the order of 40% whilst the reflection from a mass breakwater may be as low as 10%. Thus for a given incident wave height H, the wave height acting on the structure for a mass breakwater section is of the order of 1.1 H, whilst for a conventional breakwater, the wave height on the structure will be of the order of 1.4 H. If as stated by Hudson the stability of an armour unit varies as H^3, this influence alone may double the mass of the armour unit.

Unlike the conventional breakwaters where only two primary armour layers overlay a secondary layer and an impervious core, in a porous mass armour breakwater, energy can penetrate into the body of the structure. Hence the concentration of wave energy on the top layers of the breakwater is significantly reduced and the stability of the exposed armour layers is increased.

When viewed from the point of structure stability as against stability of individual armour units, a structure having many layers of armour units is capable of absorbing wave energy for a longer period when compared with a two layered structure which will be quickly destroyed once the two outer layers are removed and the filter and core becomes exposed. This was recognised by Hudson in the values given for K_D for 2, 3 and 4 layers of armour.

In a structure with an impervious core, wave energy is not transmitted through the body of the structure to the leeward face. The high permeability of the mass armour breakwater results in transmission of wave energy through the structure to the leeward side of the breakwater. For the breakwater sections tested during this investigation, the transmitted wave heights for incident wave heights of 3.6m at MSL and 6m at RL 4.5m AHD, transmitted wave heights were of the order of 0.1m and 0.6m respectively.

The transmission of wave energy, although resulting in some minor wave action on the leeward face, reduces the wave energy which otherwise would have been dissipated on the outer face. As the amount of energy which needs to be dissipated on the outer face of the structure is reduced, it will also reduce the forces acting on the exposed armour layers, thus increasing the stability of the structure.

The stability of the leeward face is not significantly affected by the transmitted wave as these wave heights are relatively small compared to the incident wave for which the whole structure is designed.

At wave heights in excess of that required to initiate damage, the seaward face tends to develop an "S" shape with relative flat slopes at the still water level (SWL) with steeper slopes both above and below. For a conventional breakwater, maximum damage occurs in the vicinity of SWL where wave forces are a maximum and it is the slope at this location which is the critical parameter in Hudson's equation. Reducing the breakwater slope in this area is known to increase breakwater stability and several investigators have suggested that for a conventional two layer breakwater increased stability can be achieved by using an "S" shaped seaward profile rather than one of constant slope. In the mass armour breakwater this shape develops naturally during reshaping by nature without damage to the core or secondary armour which would occur in a conventional two layer breakwater.

The influence of permeability on breakwater stability is presently poorly defined mainly because of the difficulty of modelling flow through porous media in a Froude model. For coarse materials where head losses in the prototype are proportional to velocity squared (i.e. independent of viscosity), a Froude model will give useful answers provided a sufficiently high Reynolds number is used.

At the scales chosen for the model studies, the scale effects are considered to be very small and are lower than model testing of a conventional breakwater, where it is impossible to model correctly the permeability in the core and secondary armour.

The main thrust of this design is its higher factor of safety against wave forces greatly in excess of the design waves and its ability to protect the area in its lee, even when severely damaged. Repairs and maintenance are readily and economically carried out. Every maintenance repair also increases the stability of this type of structure.

BBW were granted a research grant in 1985 by The Australian Marine Sciences and Technologies Grants Scheme to further research the Stability of Highly Permeable Breakwaters. Mr W Bremner and Dr B A Harper of BBW were the participants. The research work under this grant is also briefly outlined in Section 8.

3.00 NUMERICAL WAVE MODELLING

Although waverider recording systems had been operated in the area over a number of years, data on severe storm and tropical cyclone events was not extensive, with the highest recorded significant wave heights only of order 2.5m. However, even the relatively low intensity storms of decayed tropical cyclone Otto (990 mb) in March, 1977 and tropical cyclone Kerry (994 mb) in February-March 1979 generated waves of sufficient height to cause minor damage at the Utah Berths at nearby Hay Point. Because of the much higher waves likely to occur, numerical wave height prediction methods were used in determining suitable design wave parameters for the breakwater.

The wind wave predictions were performed using SPECT, a numerical spectral wave model originally devised by James Cook University (Young and Sobey) but also extensively developed by BBW.

For this project, a numerical model was formulated which extended from Cape Townshend north to the Whitsunday Islands and seawards to the Great Barrier Reef covering an area of over 50,000 square kilometres (Figure 2). Within this area model tropical cyclones were directed at Half Tide in an attempt to determine the highest possible wave heights for various tropical cyclone intensity, sizes and speeds of approach. Two basic types of storm were considered as shown in the model grid - Figure 2.

i) Classical coast-crossing tropical cyclones which move directly onshore. Four separate approach directions of N, NE, E and SE were used.

ii) Low intensity slow moving or offshore low pressure systems-such as decayed tropical cyclone Otto.

Storm intensity was based on previous research carried out by BBW, into the probability of occurrence of severe storm events along the Queensland coast.

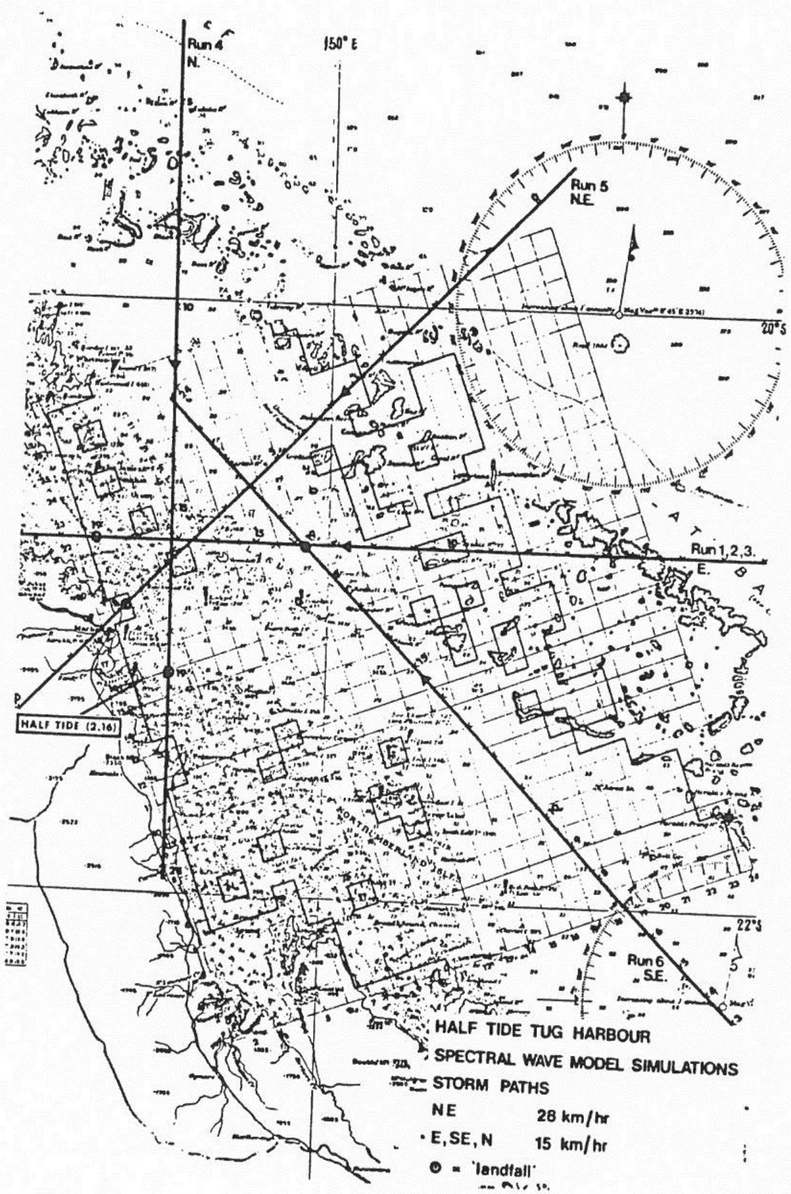


FIGURE 2.

Results for the Mackay region, which has the highest frequency of severe events, were representative at Half Tide and a 100 year design storm of 950 mb was determined for an area within 100 km of the breakwater site.

The east approach storm produced the highest significant wave at Half Tide for the 100 year storm of 4.7 m followed by 4.0 m for N, 3.9m for NE and 3.4 m for SE approaches. Very complex patterns of wave height contour and direction were forced by the near circular wind field of the storm. Together with the storm movement effect the areas of wind exposure constantly change throughout the simulation because of the numerous islands, shoals and the Great Barrier Reef. In most cases the area of highest waves was approximately due east of Mackay in the vicinity of Bailey Island.

The model highlighted the directional variation of the wave pattern but also showed energy shifts within the spectrum, as shown in Figure 3, from one wave length to another as the sea-state becomes more fully developed. In particular the NE approach indicated the influence of locally generated seas as the region of maximum winds neared the coast while the SE approach produced offshore wind for much of its approach with onshore waves only occurring after the storm passed to the north of Half Tide.

The worst approach (E) storm was re-run with a slightly higher mean water level of MSL +3m. This resulted in a higher significant wave at Half Tide of 5.0m as shown in Figure 4. The increased depth, being representative of a surge and/or tide combination, modified the wave paths approaching the shore and also reduced wave attenuation particularly in the extensive shoal areas SE of Half Tide.

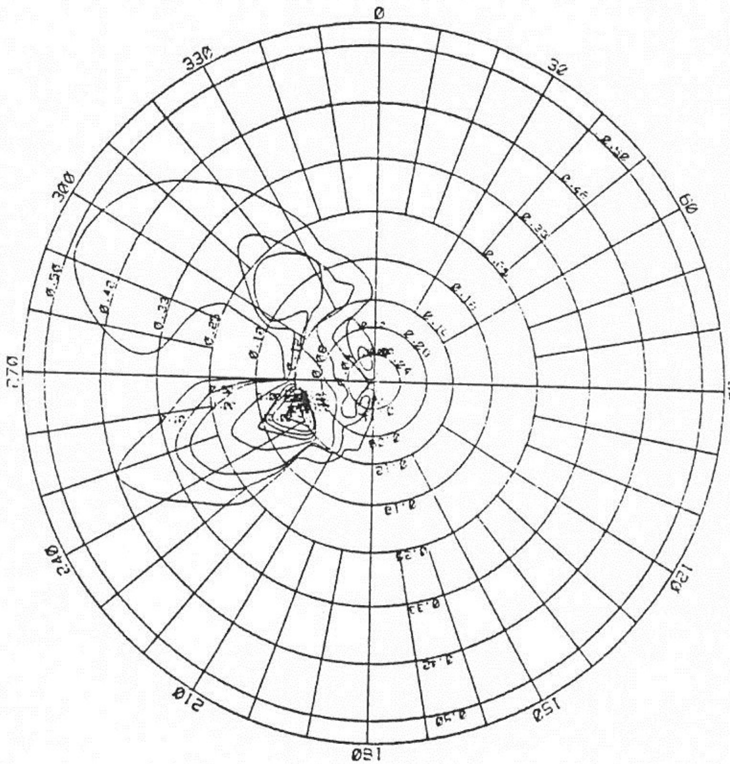
The E approach for a 500 year return period storm of 925 mb resulted in a peak significant wave at Half Tide of 5.8m. This can be compared to a 10 year return period value of 990 mb which produced a 2.7m significant wave. For a stationary storm the direction of wave approach was similar to the E approach storm but the constant wind speed and direction produced a complex pattern of wave heights throughout the area. Of particular interest was the energy shift experienced at Half Tide resulting in a peaking of the significant wave height and then subsequent decrease while offshore waves continue to build. The observed variation of wave height with central pressure was an essentially linear response over the range of central pressures tested.

The results of the spectral wind-wave modelling in the vicinity of the proposed Half Tide Tug Harbour can be summarised as follows:-

1) Highest waves at Half Tide are indicated for storms approaching from the east and making landfall north of the site near Cape Hillsborough
2) The peak 100 year event at Half Tide indicates a peak significant wave of order 5m with peak period of order 7 sec and bearing 254°.

HALF TIDE TUG HARBOUR 950MB STORM FROM E (HALF TIDE TUG HBR

SIMULATION TIME : 1-JAN-84 21:00



DIRECTIONAL WAVE ENERGY SPECTRUM

FIGURE 3

[26-SEP-83 16:23] [trace] [hte950e3] [Half Tide Tug Harbour 950mb storm from E (high tid] [B.A.Harper] [2-Jan-84 0:00]

Time history trace at site : Half Tide Tug Hbr
Grid co-ordinates : 2.00 16.00

Simulation time	Hs	Fm	Fav	Dir.
[1-Jan-84 0:00]	0.22	0.06	0.04	28.
[1-Jan-84 0:30]	0.37	0.08	0.29	24.
[1-Jan-84 1:00]	0.79	0.25	0.33	25.
[1-Jan-84 1:30]	0.95	0.25	0.29	24.
[1-Jan-84 2:00]	1.09	0.25	0.25	23.
[1-Jan-84 2:30]	1.25	0.24	0.25	22.
[1-Jan-84 3:00]	1.37	0.24	0.25	21.
[1-Jan-84 3:30]	1.41	0.24	0.25	20.
[1-Jan-84 4:00]	1.44	0.24	0.25	19.
[1-Jan-84 4:30]	1.48	0.23	0.25	18.
[1-Jan-84 5:00]	1.54	0.23	0.25	17.
[1-Jan-84 5:30]	1.61	0.22	0.25	17.
[1-Jan-84 6:00]	1.67	0.22	0.25	17.
[1-Jan-84 6:30]	1.74	0.22	0.25	16.
[1-Jan-84 7:00]	1.84	0.21	0.24	14.
[1-Jan-84 7:30]	1.92	0.21	0.21	13.
[1-Jan-84 8:00]	2.02	0.21	0.18	11.
[1-Jan-84 8:30]	2.11	0.20	0.18	10.
[1-Jan-84 9:00]	2.20	0.20	0.18	9.
[1-Jan-84 9:30]	2.30	0.20	0.18	8.
[1-Jan-84 10:00]	2.41	0.20	0.18	6.
[1-Jan-84 10:30]	2.52	0.19	0.18	4.
[1-Jan-84 11:00]	2.62	0.19	0.18	2.
[1-Jan-84 11:30]	2.72	0.19	0.18	1.
[1-Jan-84 12:00]	2.81	0.19	0.18	1.
[1-Jan-84 12:30]	2.95	0.19	0.18	360.
[1-Jan-84 13:00]	3.13	0.18	0.18	358.
[1-Jan-84 13:30]	3.27	0.18	0.18	357.
[1-Jan-84 14:00]	3.39	0.17	0.18	356.
[1-Jan-84 14:30]	3.52	0.17	0.18	354.
[1-Jan-84 15:00]	3.60	0.17	0.18	354.
[1-Jan-84 15:30]	3.74	0.17	0.17	352.
[1-Jan-84 16:00]	3.94	0.16	0.16	349.
[1-Jan-84 16:30]	4.08	0.16	0.15	348.
[1-Jan-84 17:00]	4.13	0.16	0.14	343.
[1-Jan-84 17:30]	4.20	0.16	0.14	337.
[1-Jan-84 18:00]	4.24	0.16	0.16	333.
[1-Jan-84 18:30]	4.20	0.16	0.17	323.
[1-Jan-84 19:00]	4.42	0.15	0.12	299.
[1-Jan-84 19:30]	4.74	0.15	0.12	279.
[1-Jan-84 20:00]	4.91	0.14	0.12	267.
[1-Jan-84 20:30]	4.97	0.14	0.12	259.
[1-Jan-84 21:00]	5.01	0.14	0.12	254.
[1-Jan-84 21:30]	4.99	0.14	0.12	251.
[1-Jan-84 22:00]	4.85	0.15	0.12	248.
[1-Jan-84 22:30]	4.72	0.15	0.13	245.
[1-Jan-84 23:00]	4.61	0.15	0.14	242.
[1-Jan-84 23:30]	4.51	0.15	0.14	238.
[2-Jan-84 0:00]	4.40	0.15	0.14	235.

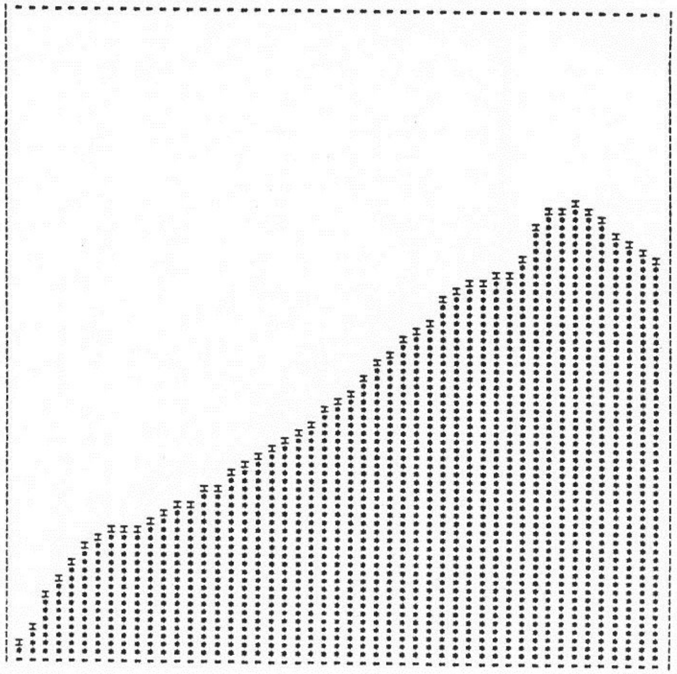
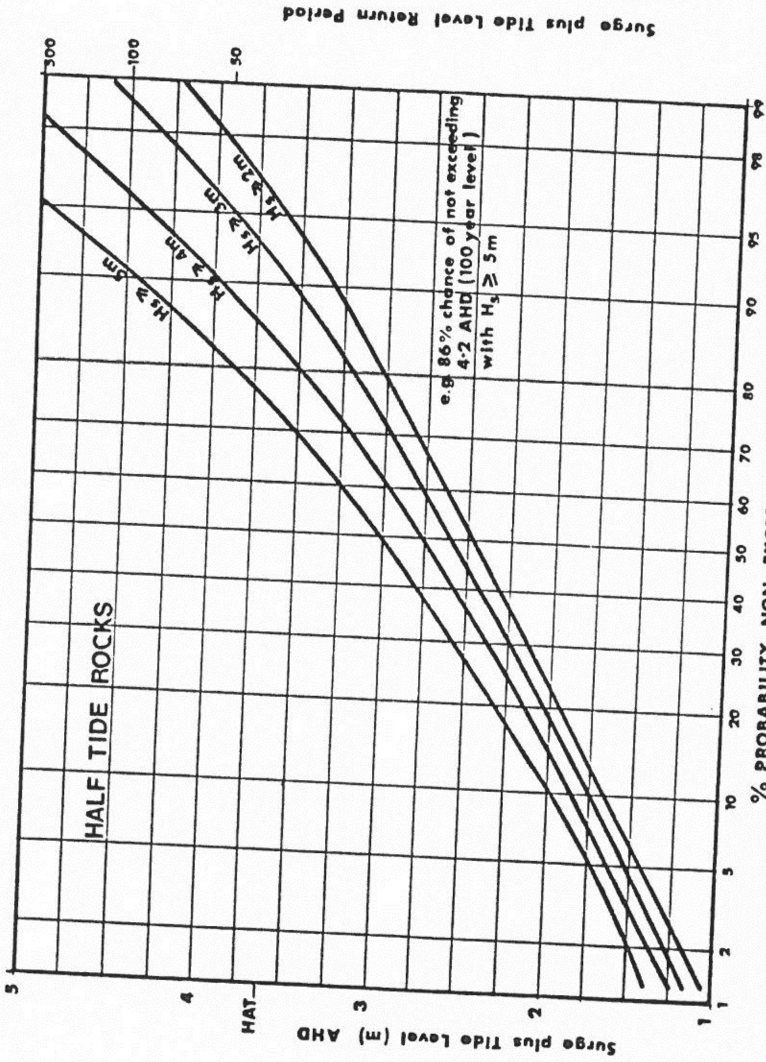


FIGURE 4



PREDICTED PROBABILITY OF SURGE PLUS TIDE LEVELS OCCURRING WITH A GIVEN PEAK SIGNIFICANT WAVE HEIGHT

FIGURE 5.

3) In all simulation runs peak waves at Mackay are predicted as somewhat higher than Half Tide (order 1m) with longer peak periods (0.5 to 1.5 s). In particular, for the relatively rare SE approach storm, Half Tide is considerably more protected than Mackay. At various stages of storm approach, differences in significant wave height between these two sites of up to 2m are often experienced.

Storm surge levels at the breakwater site were also estimated using numerical modelling techniques in a two stage process:-

1) The result of Beach Protection Authority storm surge levels due to particular tropical cyclones were compiled to produce a set of surge response factors for the breakwater site in terms of storm speed, intensity, distance from site etc.
2) The BBW statistical simulation model SATSIM was then used to simulate the occurrence of tropical cyclones in the area and to estimate their resulting storm surge for a period of 15,000 years. This was achieved by projecting forward the historical record of storm parameters and randomly "generating" events which, over the long simulation period, closely resembled the statistical makeup of previous storms.

The 100 year event design water level for surge effects alone at Half Tide was determined to be 4.2m AHD.

The storm surge results were also combined with the predicted wave heights to assess the influence of wave setup levels at the breakwater site and also to determine the probability of waves of certain heights attacking the breakwater coincidentally with a storm surge, as shown in Figure 5.

4.00 HYDRAULIC MODEL TESTS

The following model tests were carried out by Unisearch through the University of New South Wales Water Research Laboratory at Manly Vale Sydney under the direction of Professor D N Foster. The authors initiated the designs to be tested and assisted in the supervision of the testing. Full reports of the tests are contained in the Technical Reports of the University nominated under.

Test Series 1. Technical Report No. 83/15 January, 1984
Test Series 2. Technical Report No. 86/02 May, 1986
Test Series 3. Technical Report No. 86/08 August, 1986.

MODEL TEST SERIES 1

Test series 1 was conducted using a design by BBW after an independent geological investigation of rock resource at Mt Griffiths indicated that the yield armour from 4t to 7t would be of the order of 10% to 15%. Estimated costs of this design indicated that a mass armour breakwater was significantly lower than that of previous designs using conventional two layer armour design and artificial armour units.

This design outline is shown in Figure 6 with a typical cross-section of the trunk of the breakwater in Figure 7.

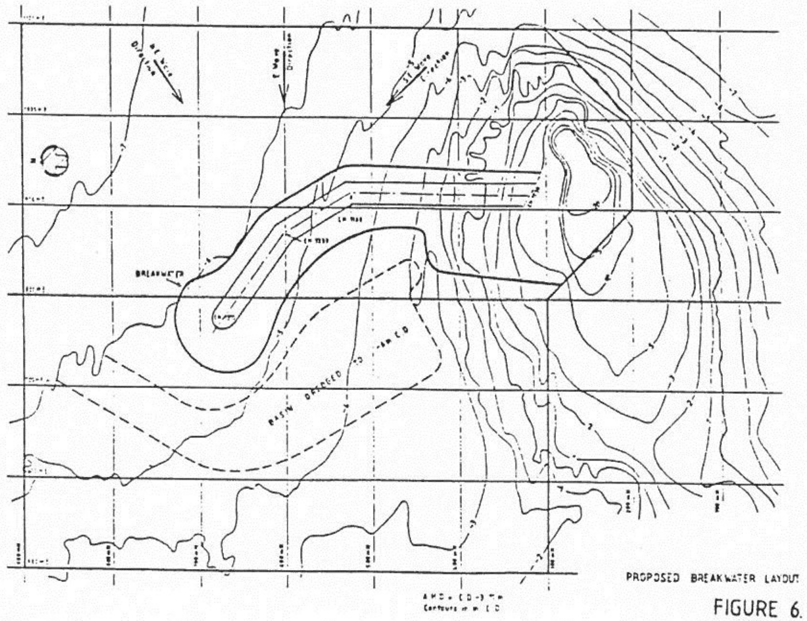


FIGURE 6.

Breakwater Geometry

The breakwater is connected to the land at the High Water Islet and extends approximately 350m in a northerly direction towards the -6m Chart Datum (CD) depth contour. The headland at the High Water Islet shields the breakwater from the South Easterly waves.

Inside the harbour the bed is dredged to RL -6m CD (-9.11m AHD) to provide sufficient draft for the tug boats. Along the seaward face of the breakwater the water depth varies between 10 to 14m for the 1:100 year storm tide level of RL 4.5m AHD.

The breakwater section is made of a core consisting of quarry stone smaller than 2 tonnes and primary armour ranging between 3 to 7 tonnes. (Figure 7). The same material is used in the trunk and the head sections of the breakwater. The core is 16m wide at the top with side slopes of 1V:1.35H. The crest of the core is set at RL 0m AHD. The crest of the breakwater is set at RL 6.39m AHD and is 25.5m wide. In the trunk section of the breakwater the ocean face is sloped at 1V:1.35H and the leeward face is flatter at a slope of 1V:2H.

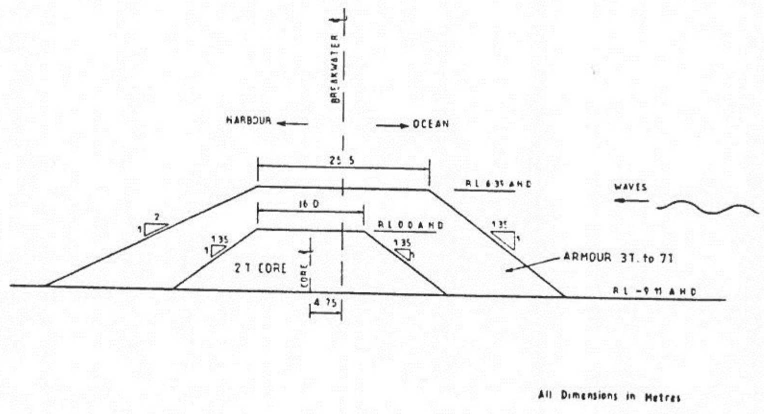
The head section is sloped at 1V:3H and has a semi-circular shape in plan form. In the transition section between the head and the main trunk section to the side slopes vary between 1:3 to 1:1.35 in the seaward face and 1:3 to 1:2 on the leeward face.

The design intention was to construct the core as shown in Figure 3 with two layers of armour on the outer face. This stage 1 construction was to be completed at the fastest practicable rate to give protection to dredging and harbour infrastructure and for this construction to commence at the earliest date.

Design Data

At the time of commissioning the hydraulic model study, BBW was undertaking a tropical cyclone wave modelling study (para 3.00) to arrive at the design wave conditions at the breakwater site.

As the cyclone wave modelling investigation was still continuing at the start of the hydraulic model study, the following preliminary design data was provided by BBW for testing the hydraulic model:-



CROSS SECTIONAL GEOMETRY OF PROPOSED BREAKWATER

FIGURE 7.

Design Wave Height H_s for 100 year return period = 6m
Wave Periods 8 and 11 sec
Wave Directions SE, E and NE
20 Year Return Storm Tide Level RL 3.75m AHD
100 Year Return Storm Tide Level RL 4.50m AHD
Tidal Range 6.5m

Initial model testing was undertaken using these design conditions.

After completion of the cyclone wave model study the following revised design data was provided and used in the tests of the final design:

Significant Wave Height = H_s = 5m
Peak Spectral Wave Period = 7 sec
Dominant Wave Direction N 74 E

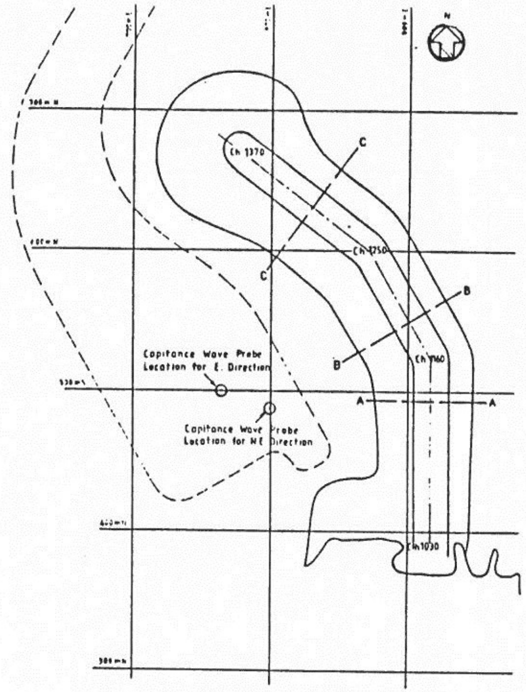
Significant wave heights in excess of 7m are virtually unobtainable at the site.

During the model study both 3D wave basin tests of the complete breakwater as well as 2D wave flume tests on several breakwater sectional geometries were undertaken. In all 12 series of tests were completed.

All model tests except the final flume tests, were carried out using monochromatic waves. The water level corresponding to various surge levels was simulated by increasing the water level in the model by discrete steps.

Testing was carried out to investigate the stability of breakwater under various storm conditions for waves approaching from SE, E and NE directions. Figure 9 is a typical cross-section. For each wave direction two wave periods 8 sec and 11 sec were tested. It is possible for the prototype to experience several minor storms that can reshape the breakwater to an extent which may affect the stability when exposed to the design wave. Hence it was decided to simulate these conditions by testing the breakwater at lower wave height and storm tide levels before testing at the 100 year design storm condition. Since the stability tests were carried out using monochromatic waves, the higher waves in the group were simulated by testing the breakwater with 9m waves. For each wave direction and wave period the breakwater was tested for a period corresponding to a 7.5 hour storm.

For all wave and storm tide conditions tested the breakwater was stable and the damage was limited to the seaward side of the breakwater centre line. Even under the worst test conditions the damage did not extend beyond the crest centre line at any point along the length of the breakwater. The maximum damage occurred when the 6m wave was plunging on the structure.



CROSS-SECTIONAL PROFILE LOCATIONS
AND LOCATIONS WHERE HARBOUR WAVES MEASURED

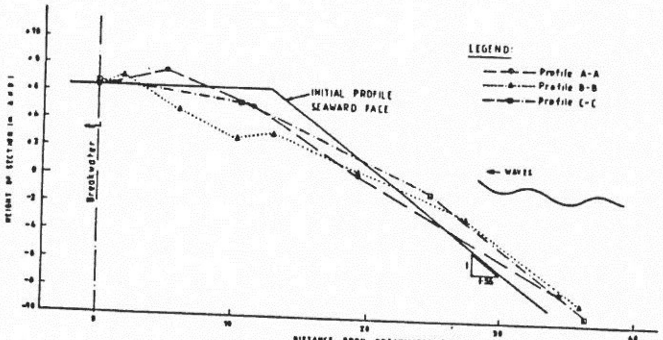
FIGURE 8.

The 8 sec waves from the easterly direction caused the most damage on the structure. At this wave period the higher waves were plunging directly on the structure and reshaped the seaward face of the structure between Chainages 1160 and 1250. The crest on the seaward side was cut back by nearly 8m.

The breakwater head section was extremely stable and there was no measurable change in the geometry for any of the wave directions tested. Similarly, there was no change in the leeward face or the leeward section of the crest.

The highest waves in the group were breaking before reaching the structure, even at 1:100 year storm tide level tested. Due to this depth limited conditions, any significant overtopping and/or damage due to the large waves in the group is unlikely.

Observations during the test revealed the mass armour breakwater to be an efficient dissipator of wave energy. Waves which travelled along the structure dissipated without becoming steeper and breaking on the structure and there was minimum reflection from waves which came directly onto the structure. Waves which ran on to the crest were rapidly absorbed without causing any significant overtopping.



BREAKWATER CROSS SECTIONS – 3D WAVE BASIN TEST
INITIAL AND FINAL PROFILES
Crest Width = 25.5m at RL 6.39m A.H.D.
6.3 T Armour, Period 8 sec.
Wave Direction: East

FIGURE 9

Due to the extremely high permeability of the structure there was only minimal overtopping even at the highest storm tide level of RL 4.5m.

Wave heights within the harbour were measured for 8 and 11 sec period waves from the NE and 7 sec period waves from the E.

Waves within the harbour were caused by wave diffraction around the head of the breakwater, wave transmission through the permeable breakwater and wave overtopping. As the offshore wave height increased the wave height due to each of the above factors also increased thus leading to a higher wave within the harbour. However, in the absence of any significant overtopping the main factor which caused waves within the harbour was wave diffraction. Wave diffraction is a function of the wave length which in turn is dependent on the wave period. The effect of wave diffraction inside the harbour was clearly evident from the higher waves which result at 11 sec period compared to the 8 sec period.

All 2D tests were carried out in a 1m wide laboratory flume. The test sections were built on the 1:100 sloping floor section of the flume which represents the approximate bed slope seaward of the structure. For these tests the bed level at the seaward toe of the structure represented approximately -9m AHD.

The wall level and wave height combinations for these tests are the same as those described for the 3D tests. The model material was the same size and grading as that used for the 3D tests. For the 6.3T armour test the linear scale was 60 which is the same linear scale as for the 3D test series. For the 5T armour tests, the linear scale was 55.7. For the 4T armour tests the linear scale was 51.6.

Under these conditions the maximum wave height which reached the structure was of the order of 7 to 8m. Even under the worst conditions there was only minor overtopping and some splashing on the leeward side. This did not cause any significant damage on the leeward section of the crest or the sloping face. There was small but measurable damage on the seaward face and the seaward section of the crest.

The damaged breakwater profiles at three sections were taken at the end of nearly 8 hours of testing. The results compared well with those from the 3D wave basin tests and confirms the greater stability and high resilience of the structure compared to that of a conventional two layered structure. The significant hydraulic characteristics, namely the lower run up, extremely high permeability and absorption and little or no wave reflection from the structure were also noted during these 2D tests.

The results from the 25.5 wide crest showed that only about 5m of the seaward crest width was damaged and that there was no significant damage on the leeward face. As the extent of damage was relatively small compared to the total area of the breakwater it was decided to undertake tests to optimise the sectional geometry of the breakwater. Hence tests were carried out on a 17m wide crest.

Except for the reduced crest width all other dimensions including crest elevation, armour size and model scale and test conditions are the same.

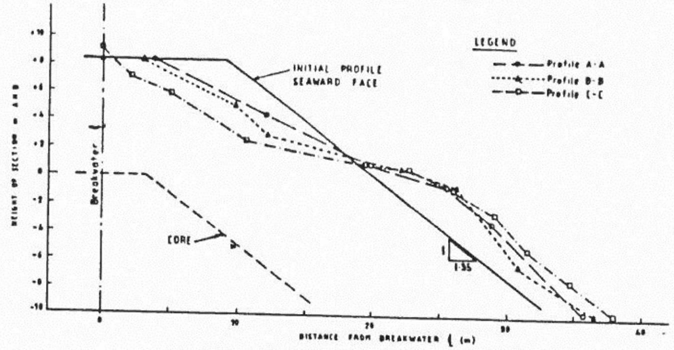
The damage in the seaward crest section for the same distance of 5m and this still left nearly 12m of undamaged crest width.

Due to the narrower crest width there was slightly more overtopping and splashing on the leeward crest but this was not sufficient to cause any significant damage on this section of the breakwater. The increased splashing also resulted in slightly higher wave heights in the harbour side which reached to nearly 0.6m.

The final sectional profiles after 8 hours of testing are shown in Figures 9 and 10. These profiles show a marginally greater damage than for the equivalent 6.3T armour section. This was partly due to the test being carried out for a longer duration with plunging wave conditions which caused the most damage. The seaward crest was cut back by 6.5m to reach within 2m of the crest centre line leaving an undamaged crest width of approximately 10m.

The reduced crest width also resulted in greater overtopping of the structure but this overtopping was still not sufficient to cause significant damage on the leeward section of the breakwater.

Except for marginally greater damage, the damaged profiles for the 5T and 6.3T armour breakwater sections were similar. The stability of the two armour sizes also did not appear to be significantly different. It is likely that in a breakwater consisting of mass armour the weight of the armour units plays a lesser role in the stability of the breakwater section compared with that of a conventional breakwater.



BREAKWATER CROSS SECTIONS – 2D FLUME TESTS
INITIAL AND FINAL PROFILES – MONOCHROMATIC WAVES
Crest Width = 18.0 m at RL 8.39 m A.H.D.
4T Armour, Period 7 sec.

FIGURE 10

Previous tests carried out at the Water Research Laboratory on mass armour breakwater sections using both 2T and 4T armour units also have shown that the final equilibrium profiles are relatively independent of the mass of the armour units.

From the previous tests it was clear that there was significant overtopping of the structure although this did not result in any significant damage on the leeward face. To minimise the risk of damage to the leeward face by overtopping of the large waves in the group, the crest of the breakwater was raised by 2m to RL 8.39m AHD. To reduce the proportion of armour material in the breakwater, the core area was also extended.

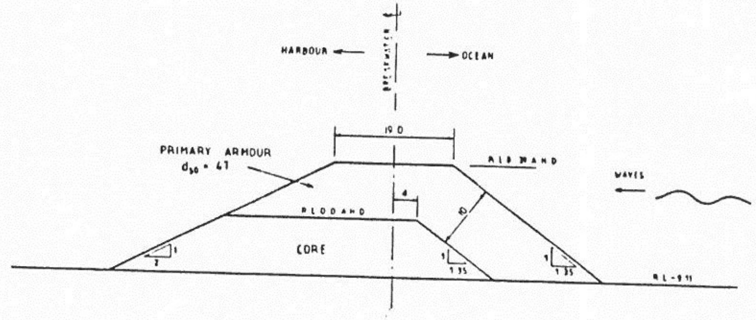
Testing showed that the higher crest level reduced overtopping and splashing of water to the leeward face.

In order to investigate the feasibility of using 4T rock, tests were carried out on the same breakwater geometry. The linear scale for this test was 51.6. The final profile after 7 hours of testing and the stability of the structure and the equilibrium were similar to that for the 5T armour. The section as tested is shown in Figure 10.

To study the stability of the structure under random wave conditions, the same breakwater section was tested with random waves.

The tests were carried out for a duration of 7 hours using Pearson-Muskowitch spectra having a peak spectral period of 8 sec.

The breakwater was tested at three water levels, namely MSL, RL 3.75m AHD and RL 4.5m AHD which were the same as for the monochromatic wave tests. The characteristic significant wave heights of the spectra at the MSL and RL 3.75m AHD were 6m and 6.3m respectively. At the highest water level of RL 4.5m two spectra corresponding to H_s = 6 and 7.1 were tested.



SECTIONAL GEOMETRY OF BREAKWATER
FINAL DESIGN

FIGURE 11.

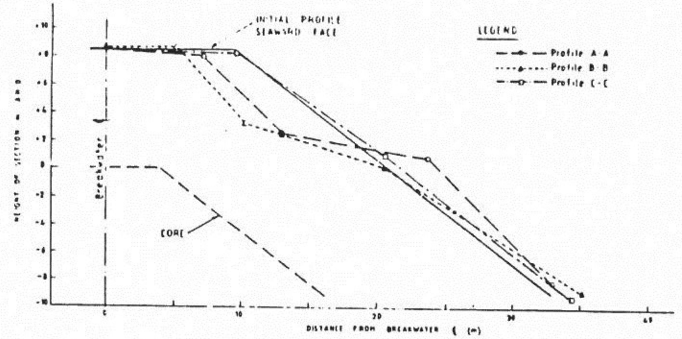
Due to the depth limited conditions the larger waves in the group were breaking before reaching the structure.

The sectional geometry is nearly the same as that used in previous tests using 4T armour and 19m crest at RL 8.39m AHD. The layout of the breakwater is similar to the initial design except for realignment and extension of the breakwater head to reduce wave penetration into the harbour as shown in Figure 13.

The final design was tested in the 3D wave basin for waves approaching from the easterly direction. For stability tests on the breakwater a linear scale of 51.6 was used. At this scale the model armour material represents a d_{50} = 4T rock in the prototype. The topography of the ocean bed was not remodelled and was the same as that for the 60 scale model. The length of the breakwater was also based on a 60 scale. The sectional geometry of the breakwater, model water level, wave height and wave period were all based on 51.6 scale.

The breakwater (Figure 11) was tested at three water levels, MSL, RL 3.75m AHD and RL 4.5m AHD for waves up to 9m. For these tests the wave period used was 7 sec.

The final equilibrium profiles and the breakwater outline after 8 hours of testing clearly show that the extent of damage is limited to the seaward face. The variation of damage along the length of the breakwater was similar to that of the initial design which is described previously. Even at the section where worst damage took place, there was nearly 14m of crest which was unaffected by wave action. There was no significant overtopping and the leeward face was completely unaffected. The core was not exposed at any location along the breakwater. The tests clearly showed the structure to be stable even under the worst storm attack which it is likely to experience. (Figures 12 and 13).



3D WAVE BASIN TESTS FINAL DESIGN
INITIAL AND FINAL PROFILES
Crest Width = 19.0 m at RL 8.39 m AHD
4T Armour, Period 7 sec
Wave Direction: East

FIGURE 12.

These model tests clearly showed that the mass armour breakwater was highly resilient and extremely stable when compared with a conventional breakwater using equivalent size primary armour. This increase in stability is attributed mainly to the high permeability of the structure which reduced drag and seepage forces as well as markedly reducing wave reflection. Wave heights inside the harbour for varying water levels are shown in Figure 14.

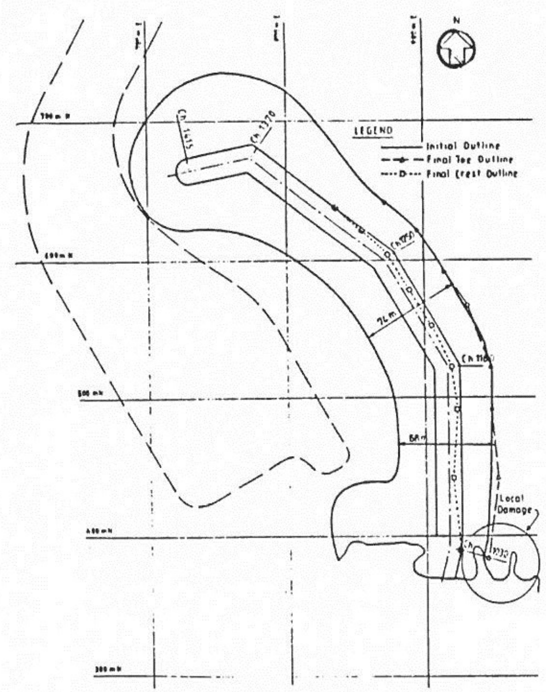
5.00 CONSTRUCTION

In 1983 investigations of the quarry at Mount Griffiths indicated the presence of massive rock in andesitic dykes. This led to a design of the breakwater by BBW, using a rubble mound mass armour rock structure which maximised the use of local material and which included large quantities of sound but heavily fractured rock mined in the process of extracting the required 4 tonne rock armour. The reject material from the quarry was to be used in the construction of a haul road 2.7km in length and a causeway 0.8km long.

Within the first eight weeks of the specified intensive development and operation of the quarry a much higher than predicted yield of fine material indicated that a considerable amount of 4 tonne nominal armour may need to be imported to construct the breakwater to the original design. From the first series of model test, it was believed there existed considerable scope to reduce the nominal size of the core material and possibly increase the volume of the core and still maintain a dynamically stable structure at the design wave and surge levels.

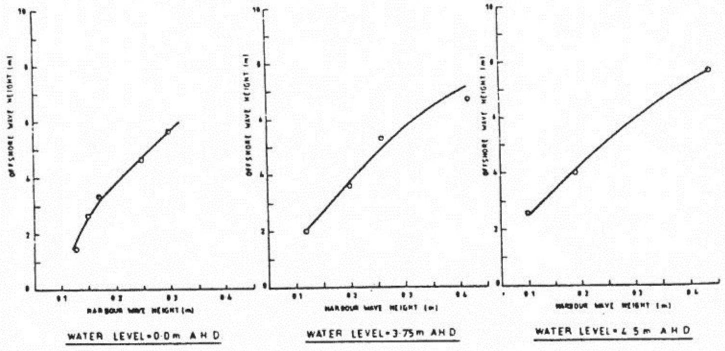
6.00 MODEL SERIES 2 DURING CONSTRUCTION

The purpose of this new series of tests was to examine options for including as much of the finer quarry run material as possible hence minimising, or even negating, the importation of extra material for the armouring of the breakwater.



CREST AND TOE OUTLINE BEFORE AND AFTER TESTING
Wave Direction: East - Wave Period: 7 secs.
FINAL DESIGN

FIGURE 13.



WAVE HEIGHT INSIDE THE HARBOUR FOR 7 SEC. PERIOD WAVES FROM EAST

FIGURE 14.

The report describes the two dimensional model flume testing of the following breakwater sections:

a) Breakwater trunk made entirely from the readily available quarry run material.

b) Core as per original design except that the nominal size is to be reduced from 2T to 1T.

c) Quarry Run Core. Core made completely out of quarry run material.

d) Composite Core. A core made up by substituting part of the nominal 1T core structure with quarry run material.

e) Composite Core Structure. The testing of the Composite Core covered by the nominal 4 tonne armour as in the original design.

Each of the breakwater sections was constructed in the test flume and the flume flooded to the appropriate depth. A one metre wave was applied and the water level varied over the normal tidal range (-2m to +2m) until all visible settlement of the model had occurred. The design storms were then applied to the structure. During this time a video film was taken at startup and shutdown of each stage of the test to record the model's performance. At the end of each stage of the design storms the average profile of the structure was recorded.

For the initial model test the water level was stepped up and down in discrete quantities to simulate the given water level for that design period of the storm. This method was abandoned for subsequent tests sections in favour of a more realistic linearly varying rise and fall of water levels to simulate storm surge.

The wave spectra as designed was input via the random wave flume computer. The test was run and periodic sampling of the spectra was made. A three probe analysis was made which divided the waves into incident and reflected. From this analysis the significant wave height and period as designed could be cross checked against that as recorded and minor adjustments made to bring the designed and recorded spectra into line.

The structure was tested against a 1 in 100 year storm based on a wave spectra emanating from a 950mb tropical cyclone approaching from an easterly direction and making landfall just north of Mackay.

The storm selected was designed to create the highest levels of damage to the structure by maximising the water level persistence at the 100 year level of 4.5m AHD.

6.01 Test A - Quarry Run Breakwater

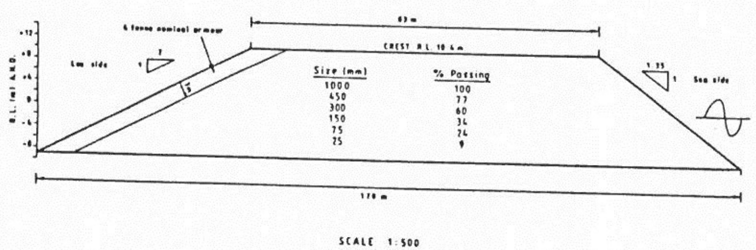
This structure uses all available material (quarry run) with no armour on the seaward face and represents the investigation of a design extreme. The lee face is to be stabilised against overtopping using the originally proposed 4T nominal armour and the head to consist entirely of 4T armour.

This test sequence was run for general interest. The section shown in Figure 15 was tested in a 2D flume with random wave spectra of the same properties as used in the final tests in Series 1.

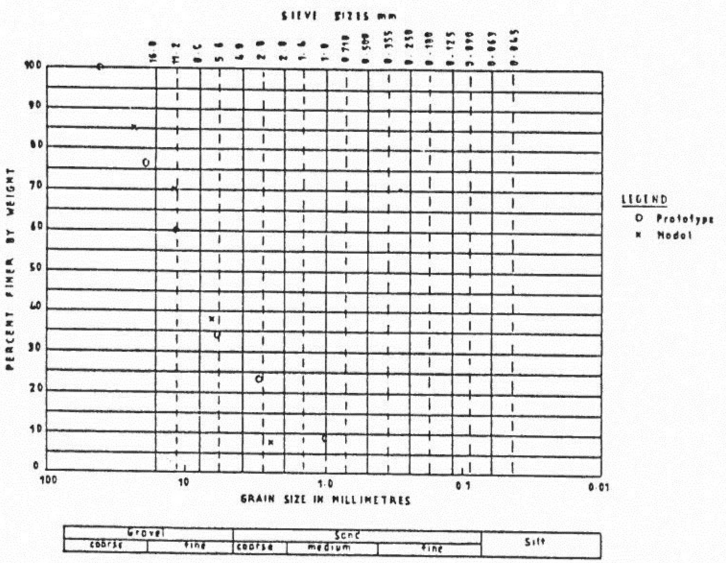
Grading tests on the quarry output at that date indicated the following size distribution.

Nominal Size (mm)	Percentage Passing	Nominal Weight (kg)
1000	100	2600
450	77	240
300	60	70
150	34	9
75	24	1
25	9	0

A comparison of the gradings prototype versus model are shown in Figure 16.



QUARRY RUN BREAKWATER DESIGN-TEST A FIGURE 15.



GRADING OF QUARRY RUN BREAKWATER (Model units)

Prototype (mm) = Model (mm) x 25

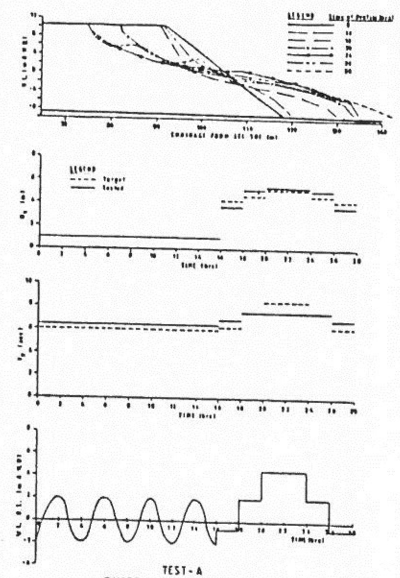
FIGURE 16

The model was subjected to the following sequence of storms:

a) Settling in - 1 metre waves over a number of tidal cycles.
b) First 100 year storm event.
c) Second 100 year storm event.
d) Summer storm - 2 metre waves over a number of extreme tidal cycles.
e) Estimated extreme 500 year storm event.

A time history of the profiles taken throughout the tests is shown in Figures 17 and 18. As expected massive reshaping of the structure occurred, however the long term profile demonstrates that the breakwater forms into a dynamically stable structure. The structure behaves like a beach where the profile acts to cause breaking of the waves before reaching the main body of the breakwater.

It is noted that the stepping of the storm water levels in the model has artificially built up berms. In the prototype, having a dynamic change in the water level throughout the storm, a more even profile would have resulted. It is concluded however that the berms have not affected the final profiles and that those given are considered representative.



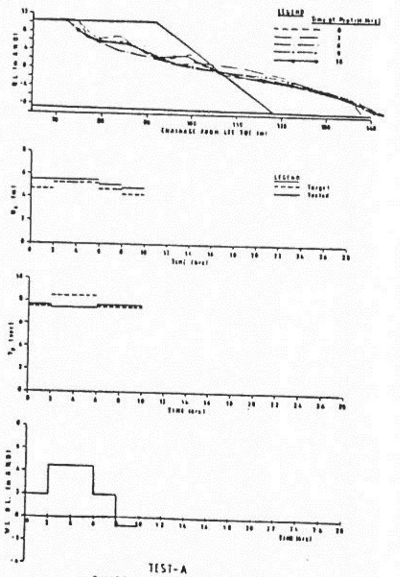


FIGURE 17

Tests B and C were carried out of core materials only with the core shape the same as the original design. Due to the shortage of armour and the possible time lapse involved in the obtaining or developing of other sources of armour rock, it was elected to test the stability of an unprotected breakwater core.

6.02 Test B: Core Reduced to Nominal 1 Tonne

The core was as per original design except using finer material, the nominal weight being reduced from 2 tonnes to 1 tonne. These tests were run to assess the risk of core construction being progressed without any armour.

The grading of the core is as follows:

Size Tonnes	Percentage Passing
2.0	100
1.0	50
0.25	0

The model was subjected to the following sequence of storms:

a) Settling in - 1 metre waves over a number of mean tidal cycles.
b) Winter storm - 2 metre waves over a number of extreme tidal cycles.
c) Summer storm - 3 metre waves over a number of extreme tidal cycles.
d) 100 year storm event.

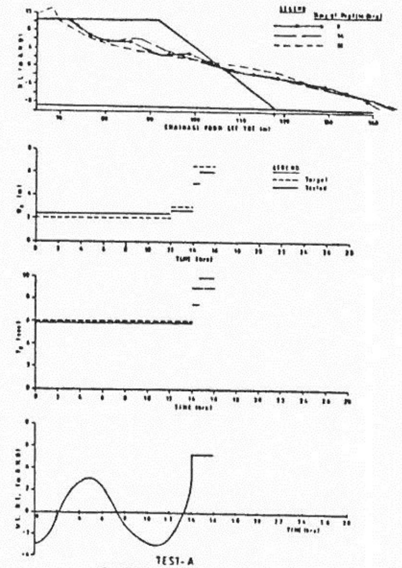
A time history of the profiles taken throughout the tests is shown in Figure 18. It can be seen from the profile that during a normal winter storm there is minimal change to the core's profile. The larger waves progressively continue to flatten out the leading face until it becomes dynamically stable half way through the 100 year storm event.

6.03 Test C: Quarry Run Core

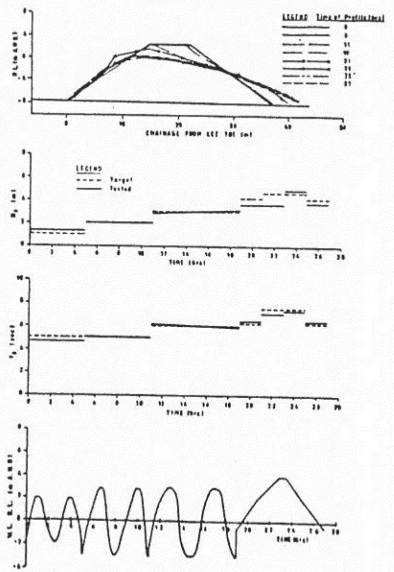
This design investigates the stability of a modified core design using only the quarry run material.

The grading size of this material is tabulated below. It was found that the quarrying methods in use would economically allow all larger rock sizes to be separately stockpiled and all fines rejected.

Size (mm)	Percentage Passing
450	100
150	0



TEST-A
QUARRY RUN BREAKWATER
500yr. Storm



TEST-B
CORE - 0.25/2.0 TONNE

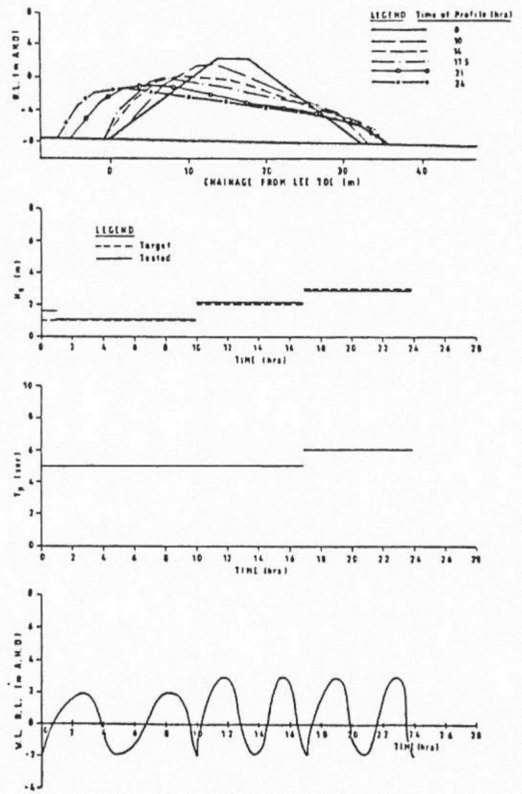
FIGURE 18.

The model was subjected to the following sequence of storms.

a) Settling in - 1 metre waves over a number of mean tidal cycles.
b) Winter storm - 2 metre waves over a number of extreme tidal cycles.
c) Summer storm - 3 metre waves over a number of extreme tidal cycles.

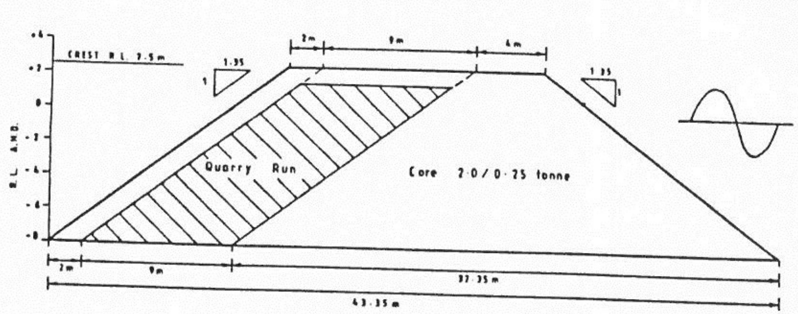
A time history of the profile is given in Figure 19.

As expected, massive reshaping to the structure occurred - even under settling conditions. As in the quarry run breakwater, the structure formed a leading protective beach.



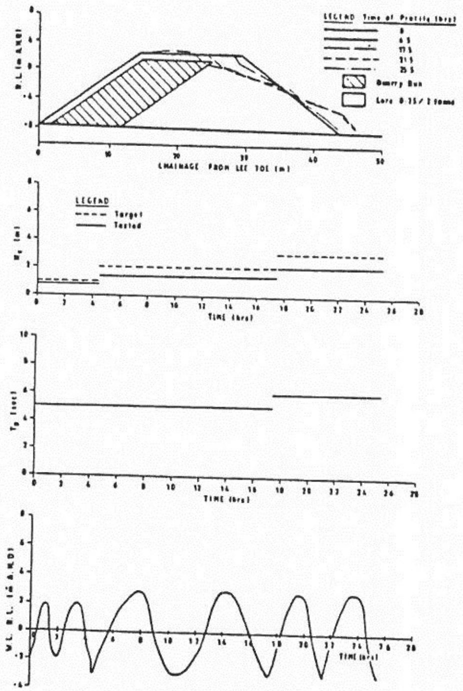
TEST-C
QUARRY RUN CORE

FIGURE 19.



SCALE 1:200

COMPOSITE CORE DESIGN TEST-D



TEST-D
COMPOSITE CORE

FIGURE 20.

6.04 <u>Test D: Composite Core</u>

The core design and material is the same as that for Test B except that a section of the core material in the lee of the structure is replaced by quarry run material as shown in Figure 20.

The model was subjected to the following sequence of storms.

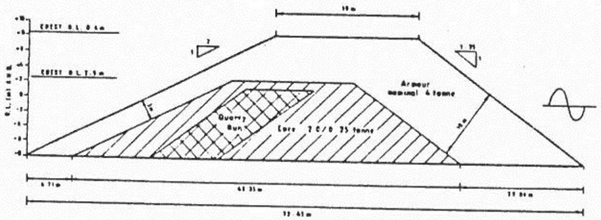
a) Settling in - 1 metre waves over a number of mean tidal cycles.
b) Winter storm - 2 metre waves over a number of extreme tidal cycles.
c) Summer storm - 3 metre waves over a number of extreme tidal cycles.

A time history of the profiles and storm conditions are given in Figure 20.

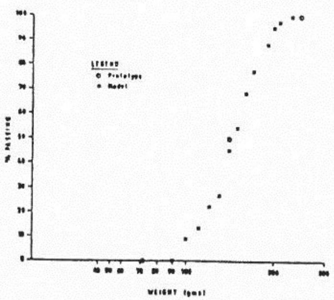
This structure performed in a very similar way to the structure containing no quarry run material.

6.05 <u>Test E: Composite Core Breakwater</u>

The composite core material was as described in Test D. The section tested is shown in Figure 21. A comparison of the model/prototype weight gradings of the armour are given in Figure 22.



DESIGN OF COMPOSITE CORE BREAKWATER TEST-E FIGURE 21



GRADING OF ARMOUR UNITS FOR COMPOSITE BREAKWATER
(Model units)
Prototype (g) = Model (g) = 28230

FIGURE 22

The model was subjected to the following sequence of extreme storm wave attack.

a) Settling in - 1 metre wave over a number of tidal cycles.
b) 100 year storm.
c) First 500 year storm waves with extreme water levels up to about a 2000 year return period.
d) Second 500 year storm waves with extreme water levels up to about a 2000 year return period.
e) Maximum waves. Largest monochromatic waves that will break continually on the structure for a wide range of water levels up to about a 2000 year period. To achieve maximum possible damage, wave heights and periods were adjusted so that maximum waves were made to plunge on the structure at all water levels.

A time history of the profiles and storm conditions are given in Figures 23 and 24.

The model tests clearly showed that this structure is extremely stable against a wide range of wave attack.

The model tests demonstrated that the potential existed for inclusion of the finer quarry material in the final design of the prototype and confirmed that large savings in construction costs are possible when this type of design and modelling approach is applied.

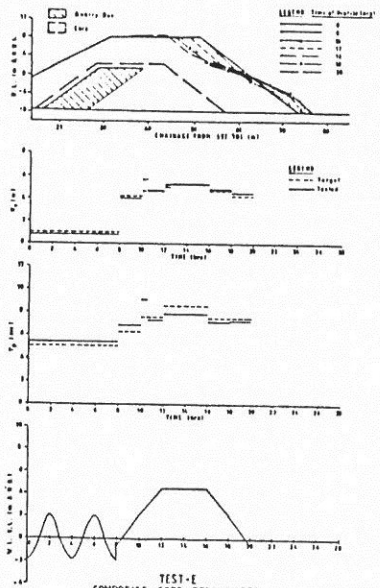
7.00 <u>MODEL SERIES 3 - BREAKWATER HEAD DURING CONSTRUCTION</u>

Following the results of test series 2 it was decided to further explore the stability of the breakwater head with a view to further reducing the armour quantity.

The final design of the breakwater shoreward of chainage 1330m was handed to the contractor on 2 June, 1986. The remaining outer length of the breakwater between chainage 1330m and the head at chainage 1413m was tested for the following modified designs:

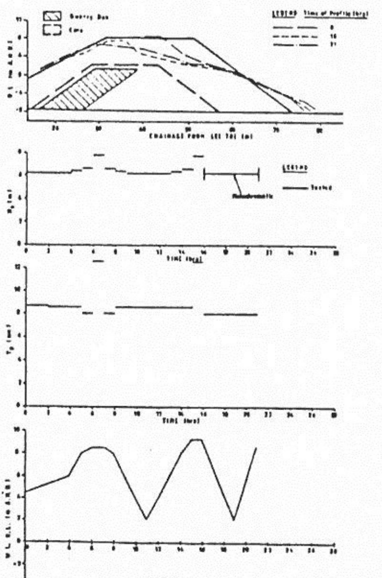
i) Core Re-design
a) Core cover to filter crest increased from 1.0m to 2.0m (i.e. RL 2.5m to RL 3.5m AHD).
b) Core crest width increased from 17.0m to 24.0m.
c) Seaward core slope steepened from 1:3 to 1:2.

ii) Armour Re-design
a) Armour material grading on seaward slope reduced from 4-7t to 2-7t (4t nominal).
b) Armour material grading on lee slope reduced from 4-7t to 0.25-2t (1t nominal), i.e. to the equivalent of the core material.
c) Armour thickness over core crest reduced from 5.9m to 4.9m.
d) Seaward armour slope steepened to 1:2.5 and 1:2 with the original 19.0m armour crest width maintained. Note that the original 1:3 sloped breakwater, although accepted from previous model work, comprises a thicker armour cover with a smaller core and therefore has also been tested for the modified design.



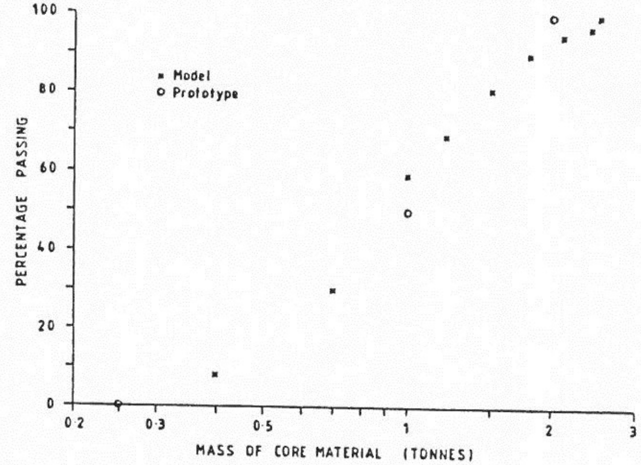
TEST-E
COMPOSITE CORE BREAKWATER
100 YEAR STORM

FIGURE 23.



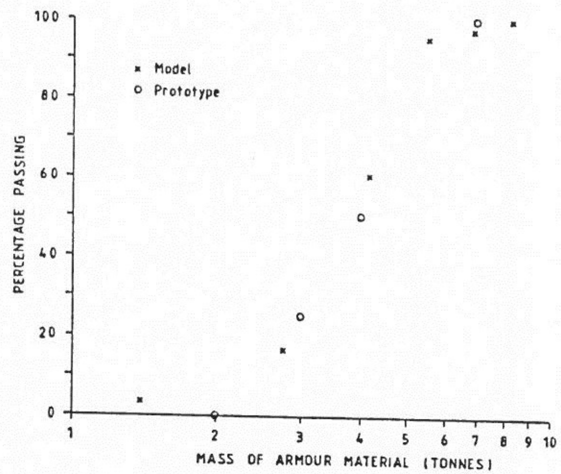
TEST-E
COMPOSITE CORE BREAKWATER
TWO 500 YEAR STORMS AND
MAXIMUM WAVE TESTING

FIGURE 24.



HAY POINT TUG HARBOUR
GRADING OF 0·25 - 2·0 TONNE CORE MATERIAL

FIGURE 25.



HAY POINT TUG HARBOUR
GRADING OF 2·0 - 7·0 TONNE ARMOUR MATERIAL

FIGURE 26.

The re-designed breakwater head defined above is presented in Figures 27 and 28.

In this testing programme of the re-design of the breakwater head, regular waves were generated such that wave breaking is initiated on the breakwater itself; the most severe wave attack scenario. The design water levels and associated incident wave heights used for testing were as follows:

Water levels: 0.0m to 4.4m AHD
Wave heights: 5.0m to 8.1m
Wave periods: 7.3s to 9.7s
Wave direction: N E

Breakwater stability tests were conducted in WRL's 30m long x 3m wide x 1.6m deep regular wave flume under fresh water conditions. Flume layout and survey codes of the model are shown on Figure 22.

Grading curves for the armour and core materials are given in Figures 25 and 26. The difference in rock densities together with the fresh-salt water buoyancy discrepancy is taken into account when establishing the breakwater material mass scale M_R.

Prior to testing the wave generator was calibrated. This involved establishing period and amplitude settings for the range of desired test water levels such that the most severe wave conditions were reproduced, i.e. wave breaking directly on the breakwater. The as constructed breakwater was then "bedded-in" under typical wave (H_s = 1 to 2m) and tidal (SWL = -2m to 2m AHD) conditions for an equivalent duration of 10 hours. Under such pre-test conditions, the breakwater is allowed to attain a denser, more compact structure which better represents the developed situation in the prototype.

Three Test Series comprising 15 tests were conducted altogether (i.e. 5 tests for each of the 3 breakwater configurations).

Test Series I - Seaward head slope 1:2 - Test Nos. 1 to 5
Test Series II - Seaward head slope 1:2.5 - Test Nos. 6 to 10
Test Series III - Seaward head slope 1:3 - Test Nos. 11 to 15

For each Test Series the still water level was varied from approximately 0m to 4.5m and back down to 0m AHD for maximum breaking waves at the structure lasting 30 hours, i.e. 6 hours for each water level.

Table 1 sets out in summary form the equivalent prototype test condition for each test series.

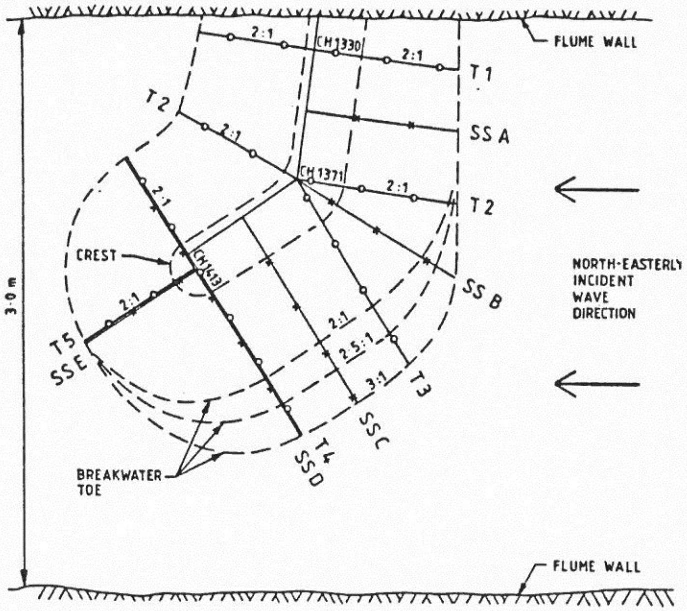
Table 1 - Test Conditions

		Equivalent Prototype Test Conditions				
Test Series	Test No	SWL (m AHD)	H_s (m)	T (s)	Storm Duration (Hrs)	Total Test Time (Hrs)
I Seaward Slope 1:2	1	0.5	5.3	7.5	6.0	6.0
	2	1.5	6.6	9.7	6.0	12.0
	3	4.1	7.8	9.7	6.0	18.0
	4	1.6	6.6	9.5	6.0	24.0
	5	0.0	5.0	7.1	6.0	30.0
II Seaward Slope 1:2.5	6	0.0	5.4	7.3	6.0	6.0
	7	1.0	6.4	9.7	6.0	12.0
	8	4.4	7.4	9.7	6.0	18.0
	9	1.4	6.8	9.9	6.0	24.0
	10	0.2	5.9	7.4	6.0	30.0
III Seaward Slope 1:3	11	0.0	5.5	7.3	6.0	6.0
	12	1.5	6.5	9.7	6.0	12.0
	13	4.4	8.1	9.7	6.0	18.0
	14	1.7	6.6	9.7	6.0	24.0
	15	0.0	5.5	7.3	6.0	30.0

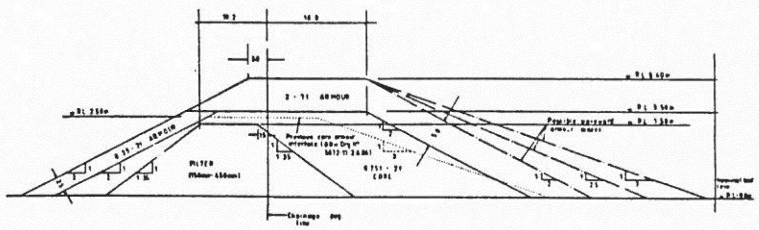
After the first 3 tests and following the final test within each test series, a survey of the breakwater was undertaken. Survey section locations are identified in Figure 27. In addition, after the third test within each test series (i.e. after testing at the maximum water level condition), the breakwater was photo-surveyed from above with the water level varied to identify the -2.2m, -0.2m, +1.9m, +4.5m and +6.5m AHD breakwater contours.

This structure is similar to the original design tested in test series 1 in both 2 dimensional flume and 3 dimensional basin model studies. The nominal weight (4T), crest level (8.4M AHD), crest width (19m), leeward slope (1:2), seaward slope (1:1.35) and thickness (10m) of armour are the same as the original design. Differences are present in the size of materials, crest level and leeward extent of the core.

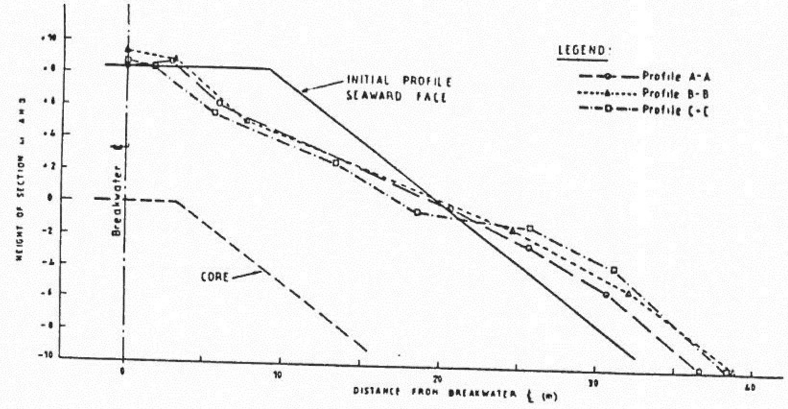
A comparison of the profiles obtained after a 100 year event from this and series 1 tests showed that there are minimal differences between the profiles.



BREAKWATER MODEL LAYOUT FIGURE 27.



RE-DESIGNED BREAKWATER HEAD FIGURE 28.



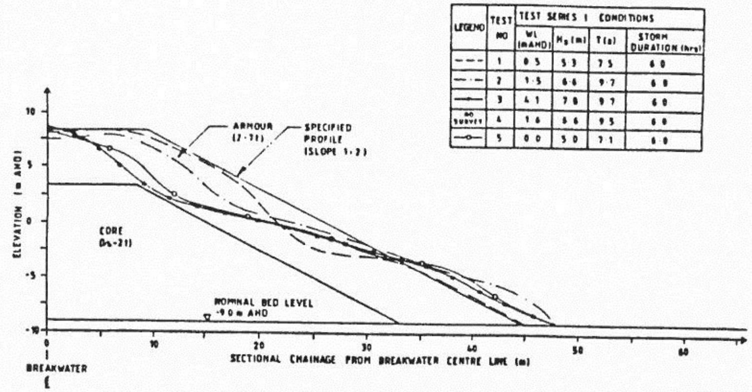
BREAKWATER CROSS SECTIONS — 2D FLUME TESTS
INITIAL AND FINAL PROFILES — RANDOM WAVES
Crest Width = 18·0 m at RL 8·39 m A.H.D.
4T Armour, Period 7 sec. FIGURE 29

A summarised video record of the 15 independent model tests has also been prepared.

Throughout all testing, no armour reshaping was observed to the lee slope of the breakwater. All reported reshaping or damage to the breakwater is confined seaward of the centre line. Representative surveyed results of the reshaped breakwater at section C for this test series are presented in Figures 30 to 32. All quantitative discussion pertaining to the reshaped structure must be deemed approximate and both chainage and elevation description taken as accurate to the nearest metre.

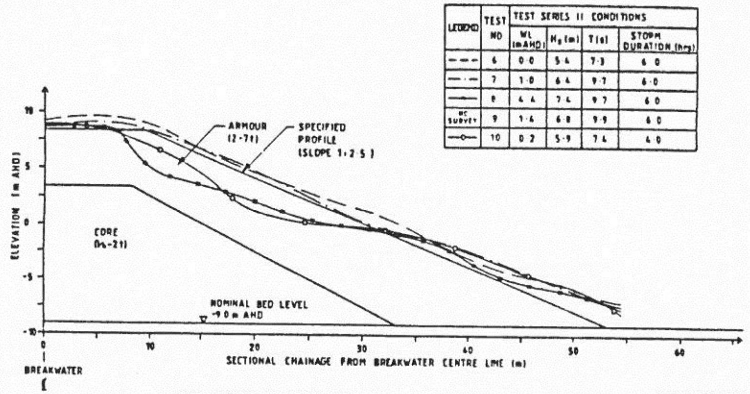
The results from Test Series III on the 1:3 slope head are in general agreement with previous extreme wave testing reported for waves from the East. The degree of reshaping and demonstrated stability of the overall structure were reproduced.

In all tests, irrespective of the degree of reshaping, or resultant armour over the core, a stable and resilient breakwater structure remained which effectively withstood the extreme conditions of breaking waves at high water levels imposed during testing.



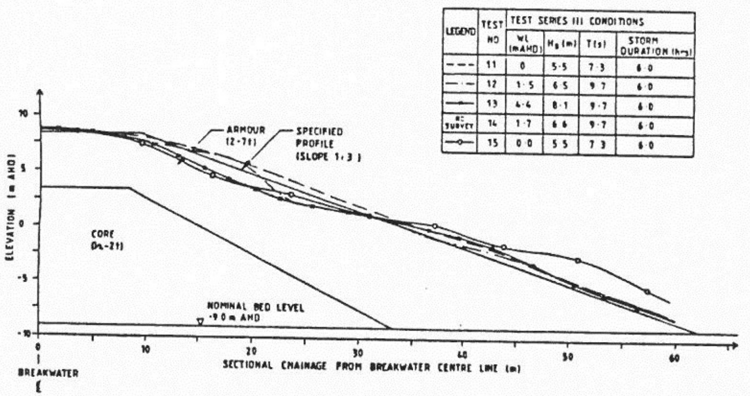
LEGEND	TEST NO	TEST SERIES I CONDITIONS			
		WL (mAHD)	H_s (m)	T (s)	STORM DURATION (hrs)
	1	0.5	5.3	7.5	6.0
	2	1.5	6.6	9.7	6.0
	3	4.1	7.8	9.7	6.0
NO SURVEY	4	1.6	6.6	9.5	6.0
	5	0.0	5.0	7.1	6.0

TEST SERIES I WAVEWARD HEAD SLOPE 1 : 2 SECTION C FIGURE 30



LEGEND	TEST NO	TEST SERIES II CONDITIONS			
		WL (mAHD)	H_s (m)	T (s)	STORM DURATION (hrs)
	6	0.0	5.4	7.3	6.0
	7	1.0	6.4	9.7	6.0
	8	4.4	7.4	9.7	6.0
NO SURVEY	9	1.4	6.8	9.9	6.0
	10	0.2	5.9	7.4	4.0

TEST SERIES II WAVEWARD HEAD SLOPE 1 : 2.5 SECTION C FIGURE 31.



LEGEND	TEST NO	TEST SERIES III CONDITIONS			
		WL (mAHD)	H_s (m)	T (s)	STORM DURATION (hrs)
	11	0	5.5	7.3	6.0
	12	1.5	6.5	9.7	6.0
	13	4.4	8.1	9.7	6.0
NO SURVEY	14	1.7	6.6	9.7	6.0
	15	0.0	5.5	7.3	6.0

TEST SERIES III WAVEWARD HEAD SLOPE 1 : 3 SECTION C FIGURE 32.

Wave protection afforded to the harbour by the final reshaped structure is equal to that of the breakwater before reshaping occurred.

Because of the possibility that Mt Griffiths quarry may not produce sufficient armour to complete the job the schedule for tendering contained an item of 75,000t Breakwater Armour from a nominated source at Mt Scrubby which is about 36km from the site. Hence there was room in the contract to negotiate prices for armour rock from other sources. The contractor and the Principal conjointly developed an excellent quarry at Mt Chelona which is 32 km from the site and the breakwater was successfully completed using this source of armour rock.

The only other two existing quarries within economic range in the Mackay area in addition to being 42 km distant from the site also had a very low yield of 4t + armour rock.

The focus for redesign of the breakwater during construction concentrated on maximising the use of the local Mt Griffiths quarry output knowing that there would be a shortage of 4t nominal armour rock by the time this resource was exhausted.

Following the series 2 and 3 model tests drawings (Figures 33 and 34) for the final construction of the breakwater were issued in July 1986. Mt Griffiths quarry was closed on 17 September, 1986. The average rate of production was 3000 t.p.d. of all materials. Mt Chelona production averaged 700 t.p.d. of 4t to 7t and 1t to 2t armour stone of high quality. The lead from this quarry to the job is 36km.

The following tabulation of quantities used in this project are included to indicate its relative size.

Armour	4t to 7t	270,000 t
	1t to 2t	85,000 t
Filter		147,000 t
Core	1/4t to 2t	129,000 t
Causeway	Quarry Run Armoured with core material	850,000 t
	TOTAL	1,481,000 t

The breakwater, causeway and car park were completed on 18 March, 1987.

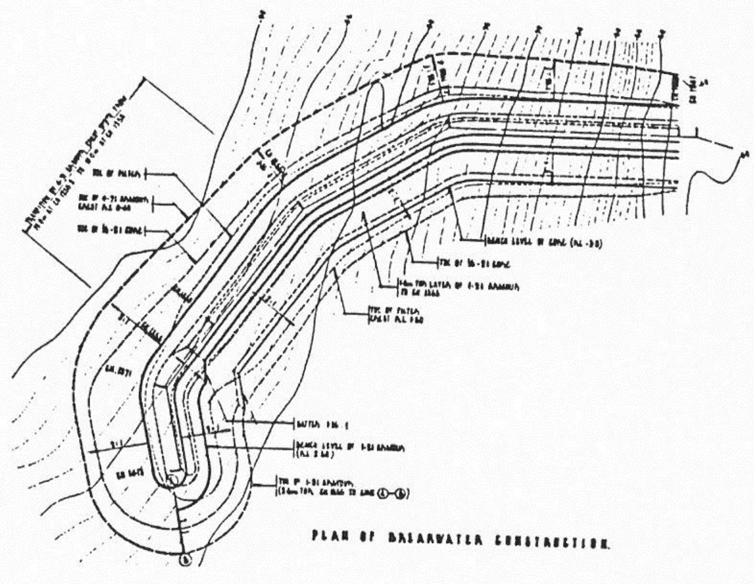


FIGURE 33.

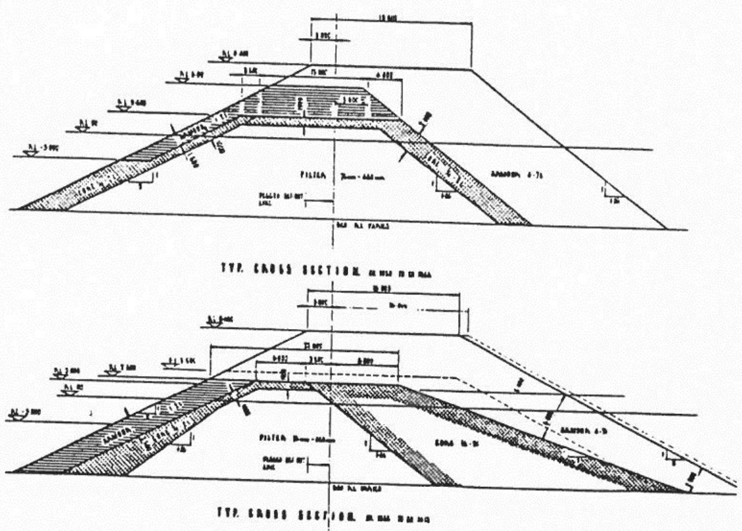


FIGURE 34.

8.0 <u>DEVELOPMENT OF THE COMPUTER MODEL HARBREM</u>

<u>Previous Studies of Breakwater Stability</u>

The vast majority of studies of breakwater behaviour have been based on experiment, due to the large number of variables affecting the problem and the lack of real knowledge surrounding the fluid behaviour. Generally, much of this work has been on an "as needs" basis for particular structures and as a result, the scientific yield has been small due to the absence of standardised procedures. A large bulk of "evidence" has therefore been collected over the years and correlated with the only available soundly based scientific study of the time - the Hudson Equation.

$$M = \frac{\rho_a}{K_D} \frac{H^3}{\Delta^3} \cot\alpha \qquad (8.1)$$

where

M	is the mass of individual armour units:
ρ_a	the density of armour;
H	the height of the characteristic design wave;
Δ	the relative submerged weight of armour $(\rho_a-\rho_w)/\rho_w$
$\cot\alpha$	is the slope of the armour face

K_D is an empirical coefficient designed to account for the effects of all other (unknown) variables such as:

- armour type and shape
- number of layers
- armour placement method
- friction and interlocking
- water depth
- breakwater geometry
- size and porosity of underlayers
- wave spectra

Traditionally, results of breakwater tests have been expressed in terms of K_D, the so-called "damage coefficient", which has been reported to vary as a result from 1 to 150.

A major drawback of the Hudson Equation has been shown to be the omission of the effect of wave period in the basic equation. It is now recognised that wave period does influence breakwater behaviour because it controls, together with other factors, whether or not a wave of given height will plunge or simply surge against the structure. The difference in resulting wave forces is considerable, the breaking wave force being an order of magnitude greater than the non-breaking wave.

In search of an alternative approach which might be better suited to the mass armour design the work by Meer was examined and found to offer certain advantages.

Basis Of The Meer Empirical Model

Development of the Meer Model progressed from the reanalysis of other researcher's results such as Ahrens, Losada et al and Thompson and Shuttler. In particular, a cornerstone of the Meer model is the correlation of stability as a function of the Surf Similarity Parameter ξ_z after Battjes. This single parameter embodied the essential interaction between wave period and armour slope needed to correlate the previous test data and allowed Meer to design an extensive series of physical model tests to further support the concept. The other major contribution was in establishing a clear correlation of damage against duration of wave attack.

The following variables have been identified as significant to the breakwater design problem.

Nominal Armour Diameter

$$D_{n50} = \left(\frac{W_{50}}{\rho_a}\right)^{1/3} \quad (8.2)$$

where D_{n50} = nominal armour diameter

W_{50} = actual 50% value of armour mass distribution

ρ_a = mass density of armour

Relative Mass Density of Armour

$$\Delta = \rho_a/\rho_w - 1 \quad (8.3)$$

where ρ_w = mass density of water (salt or fresh)

Armour Slope

Implicitly this variable (like armour mass) is a large factor in design, with flat slopes ($\cot\alpha > 2$) generally exhibiting much increased stability over steeper slopes. However, total structure cost is also highly dependent on the choice of this parameter.

Wave Height

The incident wave height is the most commonly used indicator of wave energy, although it is used in various forms. The Hudson Equation uses a "characteristic height" H which normally includes the reflected wave superimposed on the incoming wave and thereby also contains information perhaps more correctly attributed to structure "permeability". In the present study, following Meer, random waves derived from a Pierson - Moscowitch (P-M) spectrum are used and the incident wave is characterised by the "significant wave height" H_s.

Wave Period

$$T_z = 0.71\ T_p \quad (8.4)$$

where T_p = period of peak spectral energy

Hudson Stability Number

$$N_s = \frac{H_s}{\Delta\ D_{n50}} \quad (8.5)$$

where N_s = the "stability number" or "normalised wave height".

The Hudson Equation can then be rearranged such that

$$N_s = (K_D \cot\alpha)^{1/3} \quad (8.6)$$

where K_D = the "damage coefficient"

Surf Similarity Parameter

This single dimensionless parameter provides a valuable measure of the combined effects of wave height, wave period and armour slope acting externally on the structure;

$$\xi_z = \frac{\tan\alpha}{\sqrt{2 * H_s/(gT_z{}^2)}} \quad (8.7)$$

This allows classification of the incident wave regime, based on experiment, such that for

$\xi_z < 2.5 - 3.5$; waves will tend to plunge on the structure
$\xi_z > 2.5 - 3.5$; waves will tend to surge against the structure
$2.5 < \xi_z < 3.5$; an intermediate condition exists.

The Meer model predictions are best summarised in terms of N_s versus ξ_z.

Unit Damage Level

The basic correlation in the Meer model is related to the eroded area of armour A_e, below the original starting profile, but this is extended to a non-dimensional form as the unit damage level S, viz

$$S = \frac{A_e}{D^2{}_{n50}} \quad (8.8)$$

Physically, S is then the number of cubical stones of size D_{n50} eroded over a unit width D_{n50}.

Structure Permeability

Definition of this parameter has been universally elusive. Meer was forced to carry it through as a dimensionless coefficient P, similar in effect to the Hudson K_D value. The present study aimed to establish a more rational basis for this parameter based on the actual armour layer thickness D_a.

Duration of Wave Attack

This simple concept and its obvious application to the problem of breakwater stability was fully exploited by Meer through a systematic approach to model testing.

The duration of attack is directly measured as the number of waves N, on the basis of the average zero crossing period T_z. Profiles of A_e were taken at regular intervals of N waves.

Other Factors

Meer also touched on other aspects such as spectral shape and wave groupiness, both thought to embody significant although secondary effects. Armour mass distribution (e.g. uniform stones versus "riprap") is another factor which affects void ratio and hence "permeability" which was partly investigated by Meer and will need more attention by future researchers.

Still water level (SWL) is another factor which obviously limits the height of the incident wave but is also known to influence wave runup. This was kept constant in the Meer investigation.

The range of permeability values P was derived by curve fitting such that:

$$0.1 < P < 0.6$$

was determined by Meer for the structures used.

Unfortunately Meer did not state the thickness of the uniform armour structure used in the type A structure tests. Overall, the tests were modelled on the "conventional" designs where the bulk of experience and test results are available.

Meer also concluded that spectral shape and groupiness of the wave train had no measurable effect on stability for the range of tests conducted. Likewise the differences between uniform stones and riprap were reported as immeasureable.

The influence of duration of attack to stability was found to be strongly correlated such that $S \alpha \sqrt{N}$.

Meer Stability Formulae

Two distinct types of structure behaviour were isolated and categorized according to ξ_z;

a) for plunging waves ($\xi_z < 2.5$ to 3.5)

$$N_s = 6.2\ P^{0.18}\ (S/\sqrt{N})^{0.2}\ \xi_z^{-0.5} \qquad (8.9)$$

b) for surging waves ($\xi_z > 2.5$ to 3.5)

$$N_s = 1.0\ P^{-0.13}\ (S/\sqrt{N})^{0.2}\ \xi^{P}_{z}\sqrt{\cot\alpha} \qquad (8.10)$$

Within the transitional zone ($2.5 < \xi_z < 3.5$) the results from both Equations must be compared to determine which solution prevails.

General conclusions embodied in the above equations are that minimum stability occurs around the transition from plunging to surging. For plunging conditions ξ_z describes the influence of both wave period and slope angle whereas for surging conditions different results are found for each slope angle.

In addition, for impermeable cores, wave period effects for surging waves are small but are more evident in the case of permeable cores, comparable to the effect of plunging waves.

Difficulties in Applying the Meer Model

The first major difficulty with this model is its complexity, at least when compared with the Hudson Equation. This is an unavoidable by-product of increased sophistication but the equations are nevertheless somewhat daunting at first encounter, especially since ξ_z is itself a function of other variables. Also, both equations must be solved in the transitional zone which leads to further possibility of error.

Incorporation of the various equations into a computer program is almost an essential adjunct to its general application.

Another problem relates to the choice of S as the indicator of damage which, even as Meer acknowledges the extent of damage depends on the slope angle. More stones have to be displaced for gentler slopes before the "failure" criterion is reached. On this basis, and for the tests conducted, S values corresponding to "failure" ranged from 8 to 17. The present study addresses this problem by extending the unit damage level to a layer damage level which accounts for the slope angle.

The structure types tested are also difficult to interpolate and/or extrapolate in terms of deciding on a representative P value for configurations where more than two layers may be desired. The type A (uniform, no core, no filter) structure is not usefully applicable because the thickness of the armour layers is not given. The present study worked towards establishing a more rational estimator for P by exploring the relative influences of different numbers of armour layers.

Finally, the Meer results address the question of so-called "static" stability of the structure where it is assumed little change in profile shape occurs. Of more interest in the present study was the added effect of quite large changes in profile shape, or "dynamic" stability, where reshaping of the structure is not only expected but in many cases desirable. All of the above aspects were considered when designing the series of model tests described in the following sections.

PHYSICAL MODEL TESTING

The primary aim of the tests was to investigate the changes in structure behaviour as a function of the thickness of the armour layer. Incident wave conditions were allowed to vary only over a limited range such that wave steepness was near constant and plunging waves predominated (near the most critical stability condition). Other parameters such as water depth and armour slope were kept constant throughout the tests to simplify understanding and interpretation of the results.

TABLE 2 - SUMMARY TEST PARAMETERS

				Armour Thickness			Incident Wave Conditions					
				Test Series			Wave A		Wave B		Wave C	
	D	W_{50}	D_{n50}	1	2	3	H_s	T_p	H_s	T_p	H_s	T_p
Prototype Units	15.0	4500	1.2	11.7	7.8	3.9	4.0	6.0	5.0	7.0	6.0	8.0
Model Units	.50	.160	.0385	.39	.26	.13	.13	1.10	.17	1.28	.20	1.46

(All units kg, m, s)

The selected experimental breakwater structure is shown in Figure 35. Thickness of the armour layer was varied as three discrete values, referred to as Series 1, 2 and 3. Still water level was maintained constant at 0.5m at the toe of the structure. A series of three wave conditions (A, B and C) were applied to each armour layer thickness configuration for a total of nine separate tests.

Test Procedures

A nominal length scale (L_R) of 30 was used in formulating the model parameters, based on Froude scaling criteria. Table 2 summarises the model/prototype conditions tested.

Testing was performed in the programmable random wave flume of the N.S.W. Department of Public Works Hydraulics Laboratory at Manly Vale in Sydney. The facility is outlined in Figure 36 as being 1m in width, 1.5m deep and with overall length 30m. The flume paddle motions were computer controlled and water levels were logged from three capacitance wave probes to enable discrimination between incident and reflected wave energy.

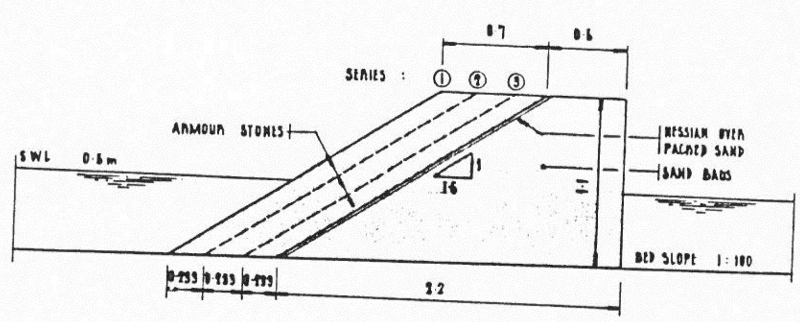
Series 1 tests were performed first, subjected to each of the three sets of wave conditions but rebuilt and raked over prior to each wave change. Some armour stone was then removed to conduct Series 2 and Series 3 tests.

Prior to the commencement of each test a reference profile was recorded, against which to measure subsequent armour damage. Pierson-Moskowitch (P-M) spectral forms were used to characterise the random sea conditions.

The duration of each test was 5,000 waves, based on the average zero crossing period T_z (in line with Meers). After each 1,000 waves the tests were paused and a single centreline armour profile was taken. Maximum levels of wave runup (R_u) and rundown (R_d) were taken visually, measured against the glass sidewall of the flume.

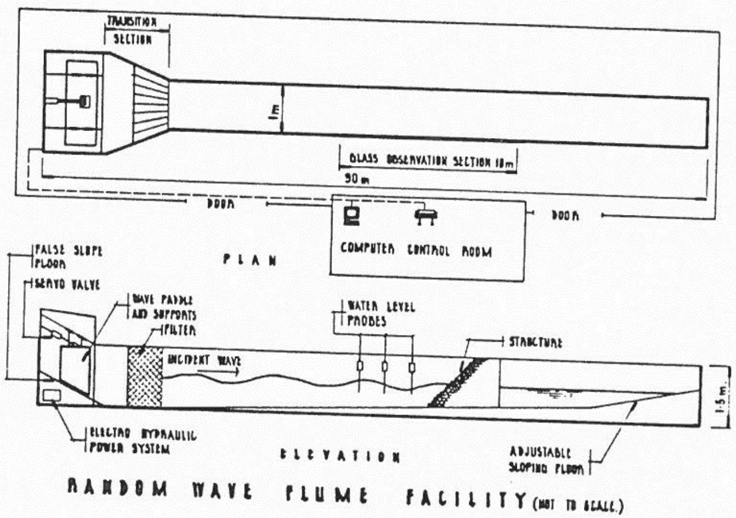
Discussion of Results

A typical set of progressive armour erosion profiles is given in Figure 37 showing gradual development of the characteristic "S-shaped" curves, with the formation of a flattened berm area extending out to around the limit of wave rundown. Use of identical wave paddle control sequences ensured good repeatability of H_s and T_p across all nine tests. Runup and rundown, somewhat surprisingly, was found to vary only slightly with armour thickness and appeared unaffected by the progressive profile changes throughout an individual test. The reflection coefficient behaved in a likewise manner, being predominantly a function of the incident wave alone.



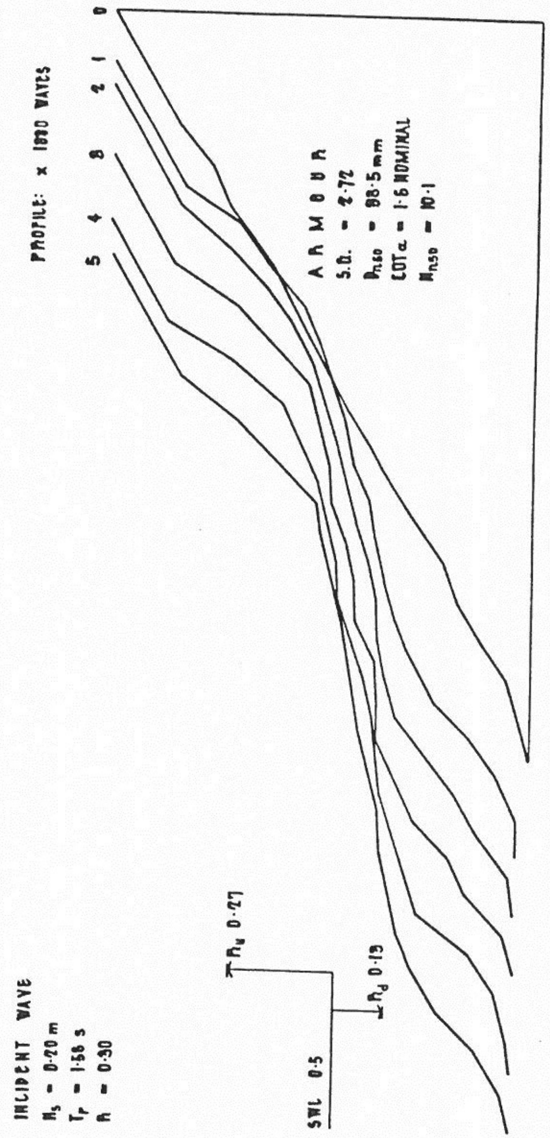
EXPERIMENTAL BREAKWATER STRUCTURE

FIGURE 35.



RANDOM WAVE FLUME FACILITY (NOT TO SCALE)

FIGURE 36.



PROGRESSIVE PROFILES - SERIES 1 WAVE C

FIGURE 37.

Damage Criteria Definitions

Figure 38 details the major variables used in describing the structure behaviour. Following Meer, the eroded area A_e below the original profile $\cot\alpha$ is the primary damage indicator, found to be well represented by a sine curve over an eroded base length L_e and with maximum eroded depth D_e; viz

$$A_e = \frac{2}{\pi} D_e L_e \tag{8.11}$$

New dimensionless parameters are also defined, being the relative armour layer thickness;

$$N_{n50} = D_a/D_{n50} \tag{8.12}$$

and the maximum relative eroded depth;

$$\delta\Delta_{n50} = D_e/D_{n50} \tag{8.13}$$

The eroded profiles were analysed to determine A_e and D_e in each case. From these, the damage level S was calculated, together with L_e based on Equation 8.11. This method of calculating L_e from D_e rather than as a direct measurement was found to be a practical way of disregarding relatively minute surface armour layer shifts at the coarse level of profile definition.

Although only a small number of tests were performed, there were clear responses for the increase in damage as a function of wave type, duration N and relative armour thickness N_{n50}. In general, lower levels of damage were sustained by the thicker armour layer (Series 1), as would be expected.

Figure 39 shows the increase in layer damage δ_{n50} versus duration N for each test.

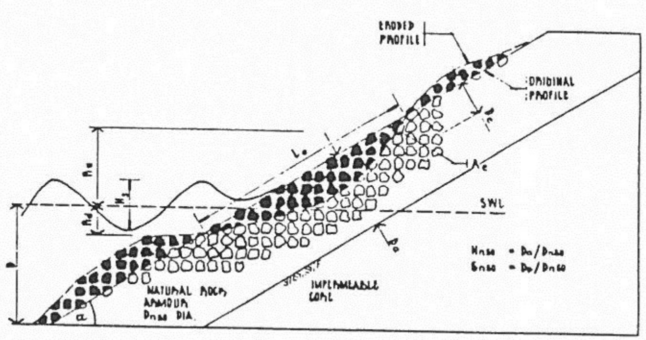
Values of Meer's "permeability" coefficient P were then derived for each test result, based on the plunging regime formula (Equation 8.9), i.e.

$$P_{best} = \left[\frac{6.2}{N_s}\frac{(S/\sqrt{N})^{0.2}}{\sqrt{\xi_z}}\right]^{-1/0.18} \tag{8.14}$$

and based on these values, Figure 40 shows P_{best} as a function of N_{n50} and the adopted line of best fit as

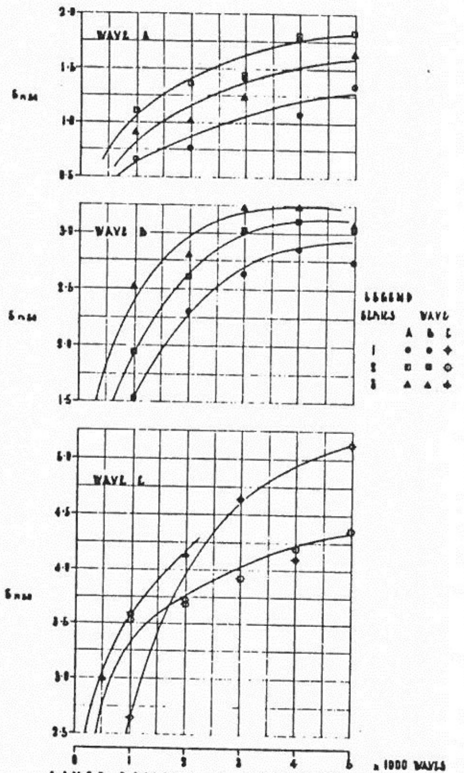
$$P = 0.017\, N_{n50} + 0.044 \tag{8.15}$$

derived using a least squares approach, but excluding outlier values for test 2A, assumed to be the result of some localised slippage of armour during early stages of the test. The only independent Meer reference point is at (2.2, 0.1) which lies above the line in this case, the other Meer values (P = 0.5, 0.6) being less easily located on this plot due to lack of interpretation and information respectively.



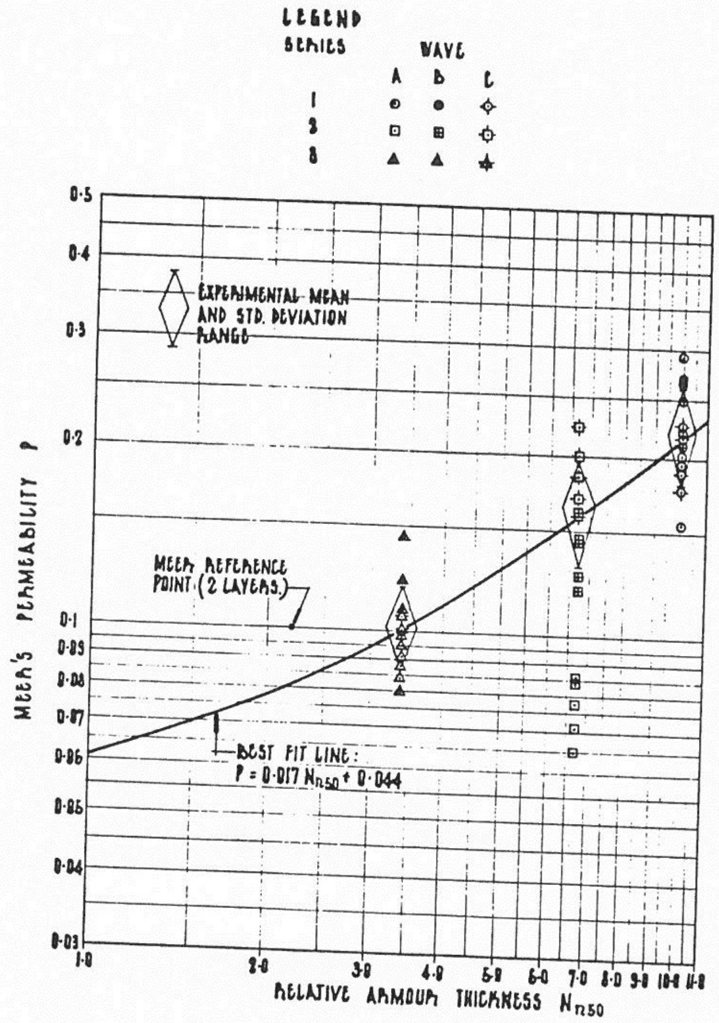
DAMAGE CRITERIA DEFINITIONS

FIGURE 38.



LAYER DAMAGE Vs DURATION

FIGURE 39



ADOPTED $P - N_{n50}$ RELATIONSHIP

FIGURE 40

Using these values, back substitution into the plunging regime formula yields values of the independent variables shown plotted on Figure 41 with dependent variable $S/\sqrt{N}$ and overlaid by the Meer best fit relationship. On this basis the plunging regime fit remains very reasonable.

Estimates of Length of Eroded Slope

The suitability of a "layer damage" model extension to the Meer "unit damage" model relies on an independent estimate for the eroded slope length L_e. A seemingly rational basis for establishing the scale of this parameter is the swash zone length, i.e.

$$L_e = f\ (R_u + R_d) = f\ (H_s,\ \xi_z,\ P) \text{ intuitively} \qquad (8.16)$$

Assuming the extent of A_e is relatively small and the effect of changes in $\cot\alpha$ is also small, then $(R_u + R_d)$ might be expected to be near constant for a given geometry. In fact, not only was $(R_u + R_d)$ insensitive to A_e throughout any given test, it was also largely insensitive to P (or N_{n50}). A fit to the relationship between L_e and $(R_u + R_d)$ was based on equating the parallel-to-slope swash zone to the eroded area base, viz

$$L_e = k\ (R_u + R_d)\ \sqrt{1 + \cot^2\alpha} \qquad (8.17)$$

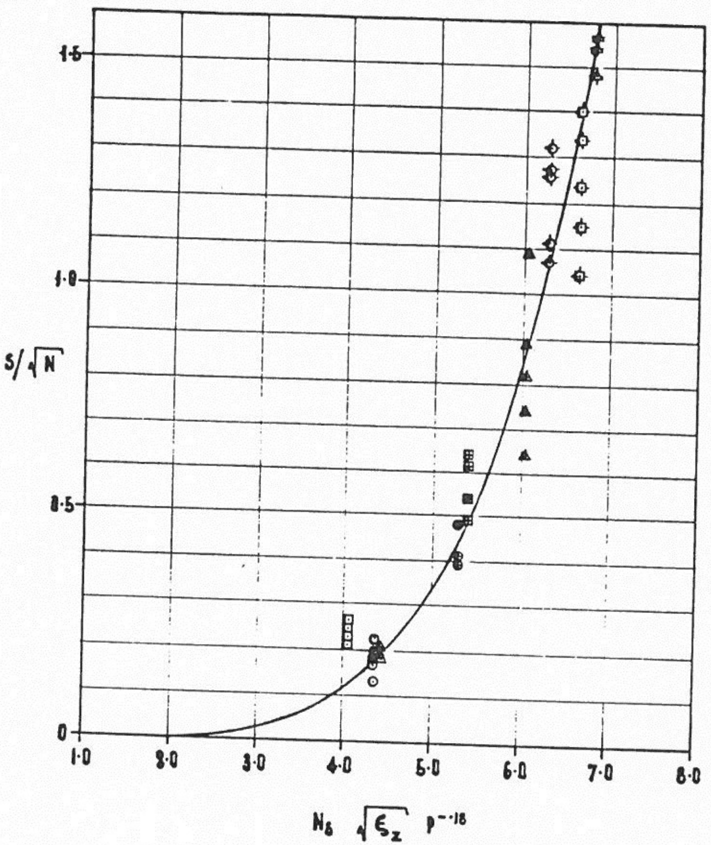
where k = 1.2. This model then estimates L_e as 20% longer than the swash zone length.

THE PREDICTIVE MODEL

The model (HARBREM) has been formulated to simplify the design process by presenting the engineer with a broad view of the problem and at the same time using parameters which are easily understood and assimilated. By showing a range of predicted structure behaviours the designer can immediately gauge the areas of sensitivity and avoid them, opting for a more "plastic" response region.

The present model reflects the traditional "design" approach, i.e. the design storm is chosen "a priori", being f(environment). Then a particular geometry ($\cot\alpha$) is chosen and the design problem reduces to armour selection and the questions of "How heavy?" and "How many layers?" are answered on the basis of the predicted degree of erosion. Also, because rubble breakwater design depends heavily on economic considerations, the model aids the designer in choosing the overall least capital cost solution.

The extension of the Meer formulae to address armour layers, rather than unit damage alone, relies on a good independent estimate of the eroded area base length L_e. With a correlation established relating L_e to (R_u+R_d), it leaves then an estimate for the latter to be obtained. Perhaps one of the most comprehensive summaries of runup and rundown has been presented by Losada et al where six sets of R_u and four sets of R_d are correlated against the surf similarly parameter ξ_z. Each experimental data set was fitted to an equation of the form.



SERIES	WAVE A	B	C	N_{n50}	P
1				10·1	0·32
2				6·7	0·16
3				3·4	0·10

——— $y = \left(\frac{x}{6\cdot2}\right)^5$

CURVE FIT TO PLUNGING REGIME STABILITY FORMULA (ξ_z < 2·5-3·5)

FIGURE 41.

$$Ru/_H = A\ (1 - e^{B\xi_z}) \tag{8.18}$$

where H is the characteristic wave height and A and B are curve parameters. Figure 42 shows the experimental results from the present study and the Losada best fit curve for a quarrystone slope tested by Kamel and Dai. The resulting approximate parameters are:

	A	B
R_u	1.4	-0.4
R_d	-0.7	-0.4

The following development then steps through all the necessary calculations to arrive at the minimum stable armour size for a given "design" condition.

Given Variables

a) Design storm = f(environment):

H_s;	significant wave height	(m)
T_z;	zero crossing period	(s)
t_d;	storm duration	(hr)

b) Geometry details:

$\cot\alpha$;	slope of breakwater face
N_{n50};	relative armour layer thickness
δ_{n50};	maximum relative eroded layer depth

c) Physical details:

ρ_a;	armour density (tonnes/m^3)
ρ_w;	water density (tonnes/m^3)

Calculated Variables

$\Delta = f\ (\rho_a, \rho_w)$
$\xi = f\ (H_s, T_z, \cot\alpha)$
$P = f\ (N_{n50})$
$L_e = f\ (R_u, R_d, \cot\alpha, k)$
$\quad = f\ (H_{max}, \xi_z, A, B)$

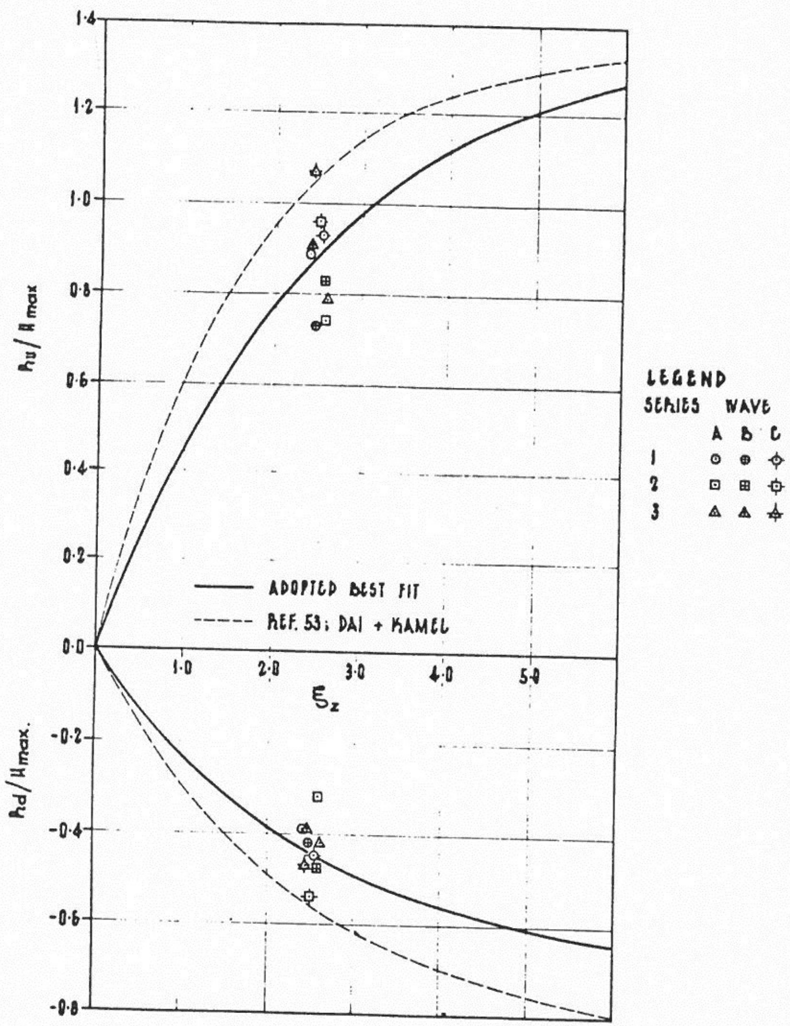
where $H_{max} = H_{ratio}\ H_s$ with normally H_{ratio} = 1.86 for 0.1% exceedance of Rayleigh distribution.

The minimum stable armour nominal diameter D_{n50} is obtained from:

$A_e = f\ (D_e, L_e)$
$S = f\ (A_e, D_{n50})$
$\delta_{n50} = f\ (D_e, D_{n50})$

with simultaneous solution of the above;

$$D_{n50} = \frac{2}{\pi} \frac{\delta_{n50}\ L_e}{S} \tag{8.19}$$



COMPARISON OF RELATIVE WAVE RUN-UP AND RUN-DOWN VALUES

FIGURE 42.

where S is derived from the Meer formulation, but firstly by considering the stability equations in their (generic) damage and duration independent forms, i.e.

$$D_{n50}\left(\frac{S}{\sqrt{N}}\right)^{0.2} = f(H_s, \xi_z, \Delta, P, \cot\alpha) = D'_{n50}$$

so that

$$S = \sqrt{N}\left(\frac{(D'_{n50})}{D_{n50}}\right)^5 \qquad (8.20)$$

Substituting into Eq 8.19 gives,

$$D_{n50} = \left[\frac{\pi}{2}\ \frac{\sqrt{N}}{\delta_{n50}}\ \frac{D'^{\,5}_{n50}}{L_e}\right]^{1/4} \qquad (8.21)$$

where D'_{n50} is given by either, for plunging wave conditions:

$$(D'_{n50})_p = \frac{H_s \sqrt{\xi_z}}{6.2\ P^{0.18}\ \Delta} \qquad \text{for } \xi_z < 2.5 - 3.5 \qquad (8.22)$$

or, for surging wave conditions:

$$(D'_{n50})_s = \frac{H_s}{1.0\ P^{-0.13}\ \sqrt{\cot\alpha}\ \xi_z^P\ \Delta} \qquad \text{for } \xi_z > 2.5 - 3.5 \qquad (8.23)$$

The model calculates D_{n50} on the basis of both the above equations and chooses the minimum of the two solutions as the correct value. Other derived parameters may then be calculated, such as

$W_{50} = f(\rho_a, D_{n50})$
$N_s = f(H_s, \Delta, D_{n50})$
$K_D = f(N_s, \cot\alpha)$

An estimate of design capital cost is then achieved by considering the armour mass per unit surface area normal to the armour slope.

$$M_A = N_{n50}\ D_{n50}\ \rho_a\ (1 - \phi) \qquad (8.24)$$

where ϕ is the average armour porosity (assumed constant 0.4 by default) and the cost per unit area, similarly

$$C_A = M_A\ C_{n50} \qquad (8.25)$$

where C_{n50} is the estimated \$ cost per tonne of armour mass W_{50} quarried, transported and placed on the structure. Given a range of possible N_{n50} and $\cot\alpha$ values the model selects the lowest cost solution from the alternatives.

Finally, the estimated total breakwater armour cost for the lowest cost (N_{n50}, W_{50}) solution is

$$B_C = B_L \ B_H \ \sqrt{1 + \cot^2\alpha} \ C_A \qquad (8.26)$$

where B_L and B_H are length and crest height of structure respectively.

A calibration check of the model was performed by back-substituting into the equations with the aim of deriving calculated values of S, δ_{n50} and L_e as a function of H_s, T_z and t_d; given the geometric properties of the structure. In this way the approximations implicit in the definition of the P-N_{n50} relationship and the (R_u+R_d) - L_e relationship would become evident.

Also to provide an independent check of the model performance a similarly designed and conducted set of test results was needed. Unfortunately this was difficult to obtain because of the wide variety of tests undertaken during the earlier Half Tide investigations. These were targeted at specific design conditions and in particular had quite high changes in SWL and/or considerable overtopping of the model structures occurred. Only one test (Test E, Figures 21 and 23) was a close approximation to the structure of Figure 35.

The comparisons between unit damage measurements and predictions is given in Figure 43. It should be noted the model S value prediction is not a function of the L_e prediction. These show a reasonably good agreement, generally giving a prediction within 30% of the measured value for both calibration and verification data. The exceptions to this rule are at the low damage end where the model tends to underpredict damage. Generally however the result is considered a good one, taking into account the limited scope of the testing.

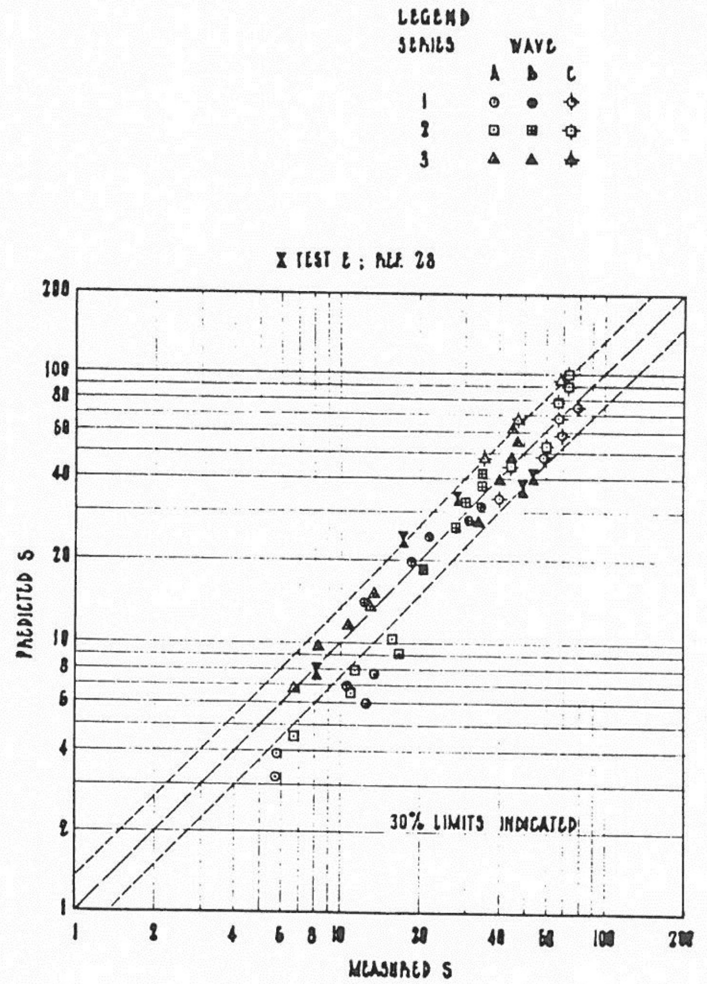
Finally, Figure 44 shows the results of the δ_{n50} layer damage predictions versus measured, with the values now being a function of L_e. In spite of the errors in L_e, again the prediction is of the order of 30% of measured values but with a more pronounced tendency to under-predict at low damage levels and over-predict at high damage levels. The over-prediction of eroded layer depth at high levels can be directly related to the inability of the present model to allow an increase in L_e with time.

EXAMPLE MODEL USAGE

The model is easy to use, fast and only requires a printer for output. Input can be either in interactive mode or via a prepared input file. Any number of separate design cases can be examined in a single model run.

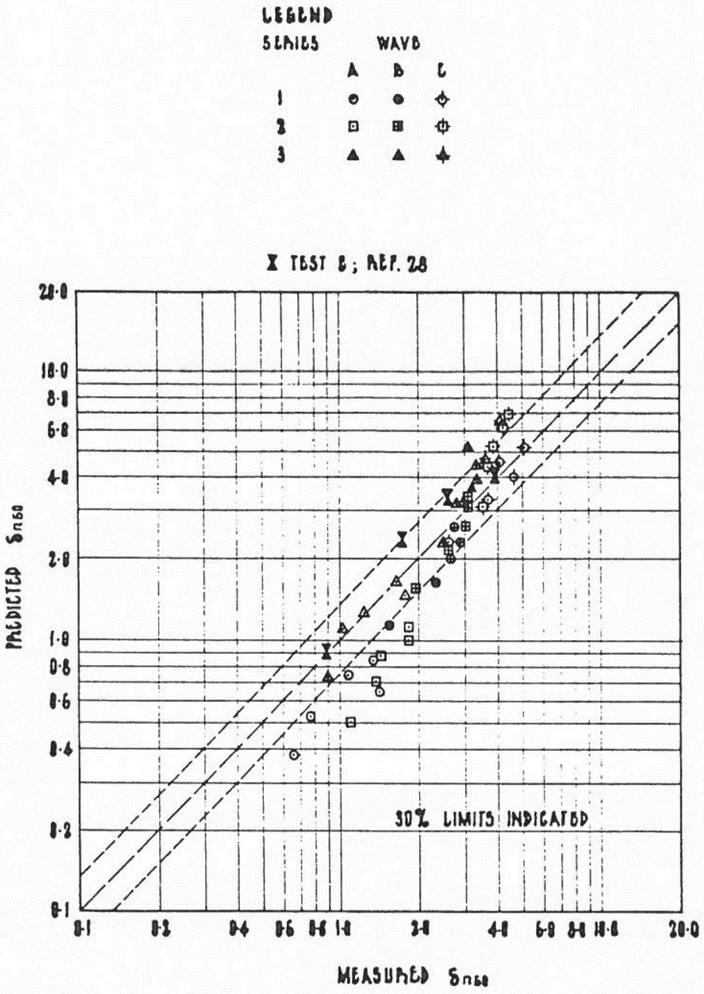
Input

The example treated here is broadly based on the design of the Half Tide harbour breakwater.



MODEL CALIBRATION AND VERIFICATION - S VALUES

FIGURE 43.



MODEL CALIBRATION AND VERIFICATION - δ_{n50} VALUES

FIGURE 44.


```
/DESIGN_CASE
Tug Harbour Breakwater - Trunk 100 yr

/DENSITIES
     2.72     1.025
```

being ρ_a and ρ_w (tonnes/m^3) respectively.

```
/GEOMETRY
   1.35    18.    300.
```

being Cotα, B_H (m) and B_L (m) respectively.

```
/STORM
100 yr Tropical Cyclone "Alpha"
5.0    5.0    8.0
```

being H_s (m), T_z (s) and t_d (hrs) respectively.

Because the economics of construction are also to be considered, estimates of the cost per tonne of placing armour on the structure is represented to the model as four pairs of tonne and $/tonne figures forming a table as follows:

```
/ARMOUR_COSTS
 4
 2.0   15.0
 5.0   20.0
10.0   45.0
20.0   60.0
```

such that the model will interpolate as necessary between these values when calculating costs.

The preceding information is all that is required to run the model (with the exception of some additional job and user identification) for this particular design case.

If no specific type of output is requested, the model produces only the matrix of W_{50} values which would be "stable" under the design conditions, or rather, with a given thickness of N_{n50} layers would result in a given number of δ_{n50} eroded layers. The present output options are:

```
/OUTPUT_W50    -  as just described (the default)
/OUTPUT_DN50   -  the corresponding armour sizes
/OUTPUT_NS     -  Hudson's Stability Number Matrix
/OUTPUT_KD     -  Hudson's Damage Coefficient Matrix
/OUTPUT_MPA    -  Armour mass per unit surface area of structure
/OUTPUT_MTOT   -  Total armour mass matrix
/OUTPUT_$PA    -  Placed cost per unit surface area
/OUTPUT_$TOT   -  Total armour placed cost
/OUTPUT_$OPT   -  The least capital cost alternative selected from the
                  above matrices.
```

Figure 45 shows the range in W_{50} to survive the 100yr storm to varying degrees of damage, given the specified geometry. For example, with an armour mass of 4 tonnes and N_{n50} around 9.0, the model indicates a δ_{n50} value of approximately 2.5 eroded layers for this option. If larger armour units are used, but with the same number of layers then the damage is reduced. Alternatively, the damage is reduced by increasing the number of layers for a given armour mass. The contours of W_{50} indicate where the stability changes most rapidly. In this example, it becomes difficult to limit the number of eroded layers to less than one without incurring increasingly more severe unit mass penalty. Between two and five eroded layers, the variation is much more gradual and smaller increases in unit mass yield reasonable decreases in eroded depths. Areas of the matrix shown as "-" indicate technically the failure region, here taken to be less than two layers remaining. Areas shown as "+" indicate the armour mass was either in excess of the upper mass limit to be considered (here 20 tonnes) or was below the lower limit (0.1 tonnes).

Note that the model has issued warnings to the effect that both the values of Cotα and Meer's P (from N_{n50}) which have been used in forming the matrix, are outside the experimental ranges so far tested and therefore further caution should be used in interpretation.

Figure 46 shows the corresponding matrix of armour cost per unit surface area by also considering the cost per tonne of the particular W_{50} value for each option. The solution surface in this case shows there are some cost advantages as a function of the number of layers placed if two eroded layers can be tolerated, i.e. from $4.1/m^2 to $6.3/m^2. With a different cost structure, the result could have been more pronounced and may have clearly indicated a particular course of action. In any case, there may be other reasons for selecting a particular option, e.g. the costs for 3.5 layers with 1.0 eroded layers are comparable to the costs for 11.0 layers with 1.5 eroded. Overall construction times for the two options could however be quite different with one more prone to disruptions by weather or industrial dispute etc.

Figure 47 shows the model's selection of the lowest capital cost design for the given design ranges. In this case it is 6.0 layers of 3.3 tonne armour which could be eroded a further 4.0 layers after the 100 year return period storm. The model does not yet balance this capital cost against the likely maintenance costs in restoring the structure to its former level of stability, but this could be included based on design estimates.

The functional form of the empirical Meer model has been shown to extend reasonably well for the case of the highly permeable (multi-layered) design. Through a systematic approach to model testing, a relationship has been proposed which relates the Meer permeability coefficient to the relative thickness of armour layers overlaying an impermeable core. Several practical deficiencies in the application of the Meer model have been overcome or improved by reformulation.

[HARBREM V1.0] [Example Breakwater Design] [B.A.Harper] [In:example.inp] [Out:EXAMPLE.OUT] [/OUTPUT_W50] [7-MAY-87 12:39]

/DESIGN_CASE Tug Harbour Breakwater - **Trunk 100yr** [WARNING: COTalpha beyond experimental range]
[WARNING: Meer's P beyond experimental range]

/STORM 100yr Tropical Cyclone "Alpha"
Hs = 5.00 m Tz = 5.00 s td = 8.00 hr

/DENSITIES ROHa = 2.720 tonnes/m**3 ROHw = 1.025 tonnes/m**3 [Surface Area = .91E+04 m*m]

/GEOMETRY COTalpha = 1.35 BH = 18.00 m BL = 300. m [Surf Similarity = 2.07 (plunging)]

MINIMUM ARMOUR MASS W50 (tonnes)

DelnS0 Max. No. Eroded Layers	Nn50 - No. Layers of Armour																		
	2.0	2.5	3.0	3.5	4.0	4.5	5.0	5.5	6.0	6.5	7.0	7.5	8.0	8.5	9.0	9.5	10.0	10.5	11.0
0.2	-	+	+	+	+	+	+	+	+	+	+	+	+	+	+	+	+	+	+
0.4	-	+	+	+	+	+	+	19.3	18.5	17.8	17.2	16.6	16.1	15.6	15.1	14.7	14.3	13.9	13.6
0.6	-	-	18.3	17.2	16.3	15.6	14.9	14.2	13.7	13.2	12.7	12.3	11.9	11.5	11.2	10.9	10.6	10.3	10.0
0.8	-	-	14.7	13.9	13.2	12.5	12.0	11.5	11.0	10.6	10.2	9.9	9.6	9.3	9.0	8.7	8.5	8.3	8.1
1.0	-	-	12.5	11.8	11.1	10.6	10.1	9.7	9.3	9.0	8.7	8.4	8.1	7.8	7.6	7.4	7.2	7.0	6.8
1.5	-	-	-	8.7	8.2	7.8	7.5	7.2	6.9	6.6	6.4	6.2	6.0	5.8	5.6	5.5	5.3	5.2	5.0
2.0	-	-	-	-	6.6	6.3	6.0	5.8	5.5	5.3	5.1	5.0	4.8	4.7	4.5	4.4	4.3	4.2	4.1
2.5	-	-	-	-	-	5.3	5.1	4.9	4.7	4.5	4.4	4.2	4.1	3.9	3.8	3.7	3.6	3.5	3.4
3.0	-	-	-	-	-	-	4.4	4.3	4.1	3.9	3.8	3.7	3.5	3.4	3.3	3.2	3.2	3.1	3.0
3.5	-	-	-	-	-	-	-	3.8	3.6	3.5	3.4	3.3	3.2	3.1	3.0	2.9	2.8	2.7	2.7
4.0	-	-	-	-	-	-	-	-	3.3	3.2	3.1	3.0	2.9	2.8	2.7	2.6	2.5	2.5	2.4
5.0	-	-	-	-	-	-	-	-	-	-	2.6	2.5	2.4	2.3	2.3	2.2	2.2	2.1	2.0

FIGURE 45.

```
[HARBREM V1.0] [Example Breakwater Design    ] [B.A.Harper  ] [In:example.inp ] [Out:EXAMPLE.OUT ] [/OUTPUT_$PA ] [ 7-MAY-87 12:39]

/DESIGN_CASE    Tug Harbour Breakwater -  Trunk 100yr        [WARNING: COTalpha beyond experimental range]
                                                             [WARNING: Meer's P beyond experimental range]
/STORM          100yr Tropical Cyclone "Alpha"
                Hs =  5.00 m   Tz =  5.00 s   td =  8.00 hr

/DENSITIES      ROHa = 2.720 tonnes/m**3   ROHw = 1.025 tonnes/m**3     [ Surface Area    = .91E+04 m*m      ]

/GEOMETRY       COTalpha =  1.35   BH = 18.00 m   BL =  300. m          [ Surf Similarity =  2.07 (plunging) ]
```

ARMOUR COST PER UNIT SURFACE AREA ($100's/m*m)

DelnSo Max. No. Eroded Layers	NnSO - No. Layers of Armour																		
	2.0	2.5	3.0	3.5	4.0	4.5	5.0	5.5	6.0	6.5	7.0	7.5	8.0	8.5	9.0	9.5	10.0	10.5	11.0
0.2	-	+	+	+	+	+	+	+	+	+	+	+	+	+	+	+	+	+	+
0.4	-	+	+	+	+	+	+	10.2	10.7	11.3	11.8	12.3	12.8	13.3	13.7	14.2	14.6	15.0	15.5
0.6	-	-	5.3	5.9	6.5	7.0	7.5	8.0	8.5	8.9	9.4	9.6	10.2	10.6	11.0	11.4	11.8	12.1	12.5
0.8	-	-	4.5	5.0	5.5	6.0	6.4	6.8	7.3	7.7	8.1	8.4	8.5	8.6	8.8	8.9	9.0	9.1	9.1
1.0	-	-	4.0	4.4	4.9	5.3	5.7	6.0	6.1	6.3	6.4	6.5	6.7	6.8	6.8	6.9	7.0	7.1	7.1
1.5	-	-	-	3.2	3.4	3.6	3.7	3.8	3.9	4.0	4.1	4.2	4.2	4.3	4.3	4.4	4.4	4.4	4.5
2.0	-	-	-	-	2.5	2.6	2.7	2.8	2.8	2.9	2.9	3.0	3.1	3.2	3.3	3.5	3.6	3.7	3.8
2.5	-	-	-	-	-	2.0	2.1	2.2	2.3	2.4	2.5	2.6	2.8	2.9	3.0	3.1	3.2	3.3	3.4
3.0	-	-	-	-	-	-	1.8	2.0	2.1	2.2	2.3	2.4	2.5	2.6	2.7	2.8	2.9	3.0	3.1
3.5	-	-	-	-	-	-	-	1.8	1.9	2.0	2.1	2.2	2.3	2.4	2.5	2.6	2.7	2.8	2.9
4.0	-	-	-	-	-	-	-	-	1.8	1.9	2.0	2.1	2.2	2.3	2.4	2.5	2.5	2.6	2.7
5.0	-	-	-	-	-	-	-	-	-	-	1.8	1.9	2.0	2.1	2.1	2.2	2.3	2.4	2.5

FIGURE 46


```
[HARBREM V1.0] [Example Breakwater Design    ] [B.A.Harper  ] [In:example.inp ] [Out:EXAMPLE.OUT ] [/OUTPUT_$OPT] [ 7-MAY-87 12:39]

/DESIGN_CASE    Tug Harbour Breakwater -  Trunk 100yr        [WARNING: COTalpha beyond experimental range]
                                                             [WARNING: Meer's P beyond experimental range]
/STORM          100yr Tropical Cyclone "Alpha"
                Hs =  5.00 m   Tz =  5.00 s   td =  8.00 hr

/DENSITIES      ROHa = 2.720 tonnes/m**3   ROHw = 1.025 tonnes/m**3     [ Surface Area    = .91E+04 m*m      ]

/GEOMETRY       COTalpha =  1.35   BH = 18.00 m   BL =  300. m          [ Surf Similarity =  2.07 (plunging) ]
```

MINIMUM CAPITAL COST ARMOUR DESIGN

```
Unit Armour Mass :     W50    =  3.3 tonnes
Nom. Armour Dia. :     Dn50   =  1.1 m
Nom. Armour Vol. :     Vn50   =  1.2 m*m*m
Unit Armour Cost :     Cn50   = $  56.54
Placed Thickness :     Nn50   =  6.0 layers
Erodable Thickness:    DelnS0 =  4.0 layers

Per Unit Surface Area of Armour Slope:

Mass   =     10.44 tonnes/m*m
Volume =      3.84 m
Cost   =  $ 179.10 /m*m

TOTALS:

Mass   =   .95E+05 tonnes
Volume =   .35E+05 m*m*m
Cost   = M$  1.62
```

FIGURE 47

A FORTRAN 77 computer program has been developed which not only simplifies the design process, but provides new insights to the structure behaviour and assists in optimising capital costs. Overall accuracy of the model developed here is of the order of 30 % when compared with that data, which is a good result for the prediction of rubble mound breakwater stability. The model should be used as an initial design tool in selecting appropriate armour sizes for physical model testing only, to ensure other possible effects (e.g. 3-D) are adequately addressed by the designer.

CONCLUSION

This project introduced into Australia the potential for using hydraulic models to maximise the output of quarried materials in phase with the construction of rubble mound breakwaters. This can result in very significant savings in cost and the minimising of risk to the principal and the contractor. In this way the efficiency of all parties to the project may be markedly increased.

For contractors, weather factors and rock materials source/s in the construction of rubble mound breakwaters, usually represent very high risk ventures. Where risk reduction is achieved, price reduction is its natural corollary. In this contract a deliberate attempt was made in the documentation and subsequent construction supervision to minimise construction risks.

This project demonstrated that when a knowledgeable principal is allied with competent engineering and an experienced contractor, a satisfactory completion of a high risk project is the end result.

ACKNOWLEDGEMENT

The Authors acknowledge with thanks, the co-operation and assistance given to this project by the Principals - "DBCT - UDCL Joint Venture"; Department of Harbours and Marine, Queensland; University of New South Wales Hydraulics Research Laboratory and the Contractors - Roche Bros. Australia Pty Ltd.

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PERFORMANCE OF A BERM ROUNDHEAD IN THE ST. GEORGE BREAKWATER SYSTEM

by

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Abstract

A new harbor under construction on St. George Island in Alaska's Bering Sea is using the berm breakwater concept for protection from wave attack. Three breakwaters are included in the system, two outer breakwaters to protect an entrance channel, and an inner breakwater to protect an 8-acre moorage basin. In late 1986 the designers were faced with a shutdown in construction with the North Breakwater roundhead only half finished. There was a question as to the capacity of the structure to withstand wave attack. The purpose of this paper is to demonstrate that during the winter of 1986-87 storms occurred which approached the design storm in intensity and that, even with the half-complete nature of the structure, the berm roundhead performed very well and suffered only minor berm profile modification.

Résumé

Un nouveau port en construction sur l'île St. George dans la mer de Béring en Alaska sera protégé des vagues par des brise-lames de type à risberme. Le système prévoit trois brise-lames, deux brise-lames extérieurs protégeant un chenal d'entrée et un brise-lames intérieur abritant un bassin d'amarrage de 8 acres. Vers la fin de 1986 les concepteurs ont vu les travaux interrompus alors que le musoir du brise-lames nord n'était qu'à demi complété. On s'interrogeait quant à la possibilité que l'ouvrage résiste à l'assaut des vagues. Le but de la présente étude est de démontrer que même si des tempêtes d'une intensité approchant celle de la tempête nominale se sont abattues sur l'ouvrage à demi achevé pendant l'hiver de 1986-1987, le musoir à risberme s'est très bien comporté et qu'il n'y a eu qu'une modification mineure du profil de la risberme.