

The First Berm Breakwater in Hong Kong

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Abstract

An alternative berm type breakwater was proposed at the South Breakwater of the Typhoon Shelter at Hei Ling Chau, Hong Kong. The required berm width and the stone sizes were initially determined using a numerical model. The proposed berm breakwater was then tested using physical modelling approach. The testing programme made use of both two dimensional and three dimensional hydraulic model studies to assess the stability of the trunk and head sections respectively. Both tests confirmed that the proposed berm type breakwater would perform satisfactorily and the reshaped profiles would not encroach on the core material.

Introduction

To meet the forecast demand for typhoon shelter space in Hong Kong, it is recommended that a typhoon shelter at Hei Ling Chau be constructed to provide about 50 hectares of effective anchorage area. As the typhoon shelter would be located at the southwest of Hei Ling Chau, three breakwaters (see Figure 1) with a total length of about 2.6 km would be required in order to provide a safe and calm anchorage area. Among these three breakwaters, the South Breakwater will be exposed to the severe waves approaching from the southeast direction. Various types of breakwater have been considered for the design of the South Breakwater. However, if the South Breakwater was to be designed as a conventional rock armoured breakwater, it was estimated that an armour rock size of 17t was required. This is generally not commercially available in large quantity in Hong Kong. Precast concrete units could be used instead of amour rocks, but in this case 4t to 9t of cubes,

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tetrapods or accropods would be required, which will require large works area for construction and thus increase the construction cost.

As an alternative option, the dynamically stable berm type breakwater was considered. With this type of breakwater, the size of armour rocks required would be smaller than that for a conventional breakwater. The construction cost would be lowered. The profile of the breakwater will, however, adjust to severe storm events. The dynamically stable berm type breakwater is thus considered as the preferred form of construction for the South Breakwater. In this paper, development of the berm type breakwater from the outline design to the verification using a physical model is presented.

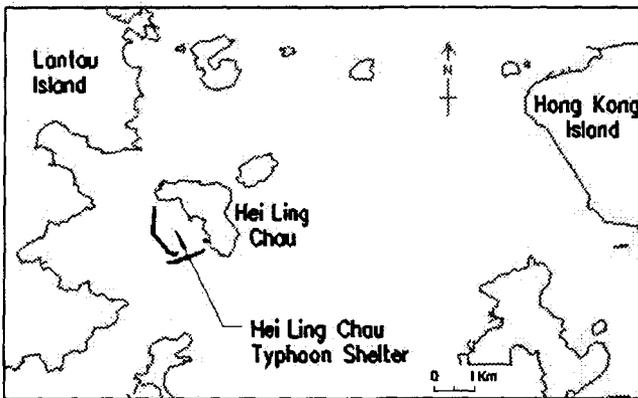


Figure 1 Typhoon Shelter at Hei Ling Chau, Hong Kong

Berm Breakwater Concept

The berm breakwater design consists of a porous berm of armour stones which dissipates wave energy. The stones are expected to move under wave action leading to a stabilised and consolidated equilibrium profile. As a result, the shear strength of the stone mass and the resulting overall stability of the structure increases as the stones settle, nest and interlock. The geometry of the breakwater is a function of the prototype stone gradation and the design wave conditions.

The initial width of the berm determines the extent to which the profile reshapes. Berm breakwaters may be designed so that:

- (i) full reshaping of the berm occurs (dynamic)
- (ii) only a slight rounding of the outer edge of the berm occurs (static)

The dynamic design is a more common form of the berm breakwater and is the concept which has undergone the most extensive investigation. This type of structure is very cost effective and allows for maximum utilization of the quarry material. By knowing the quarry yield and the relative volumes of core to armour, the quarry yield can simply be split such that a production balance between core and armour is maintained for full utilization of the quarry. The dynamic berm concept has been implemented at a number of prototype locations in Australia, Canada, Iceland, the South Pacific and the USA with great success.

The static design approach requires approximately 70 - 80 % more material and is therefore more costly. However the factor of safety is increased. Static berms have been utilized in Australia, Iceland and the South Pacific.

Implementation of the berm concept at Hei Ling Chau also presents several advantages to the contractor involved with construction of the breakwater. The berm design is simple and thus an easier structure to build. With the berm alternative, it is only important to ensure that the full berm width is placed according to the design. Minimum underwater inspection is required due to the fact that the seaslope will change with time and thus may be built at the angle of repose of the stones. Furthermore, the armour layer is built up out from the core until the required berm width is achieved. Feasibility of the berm breakwater applications in Hong Kong can be found in Chow and Sayao (1991).

Wave Climate and Design Storm

The extreme waves attack at the south of Hei Ling Chau are mainly generated by typhoons. There were no measured wave data available at the study site, the design wave conditions were derived using wave hindcasting techniques. The extreme offshore wave climate was hindcast using a parametric wind wave generation model developed by the Hong Kong Polytechnic University (Li et al 1992). For waves approaching from the south and southeast of Hei Ling Chau, various numerical models considering the effects of refraction, shoaling, bottom friction and wave breaking were applied to derive the nearshore wave climates. The derived design wave conditions can be found in Table 1. The Storm No. roughly represents the return period of the design storm.

Table 1. Design Wave Condition

Storm No.	Water Level (mCD)	Significant Wave Height H_s (m)	Significant Wave Period T_m (s)
2	2.8	2.4	8.5
10	3.2	3.6	11.6
100	3.7	4.0	13.9
250	3.8	4.2	14.7

Conceptual Design

Based on the above design storm, a quarry stone size of 17t was computed to be required for the conventional breakwater. However, this stone size is generally not locally available in large quantity commercially. Using the software BREAKWAT acquired from Delft Hydraulics, a conceptual design of the berm breakwater was carried out. The software showed that a width of berm of 12 m and a median rock size of 2 t and 4.5 t were found to be appropriate for the trunk and head sections respectively under the design storm conditions. Typical cross-section of the proposed berm breakwater profile can be found in Figure 2.

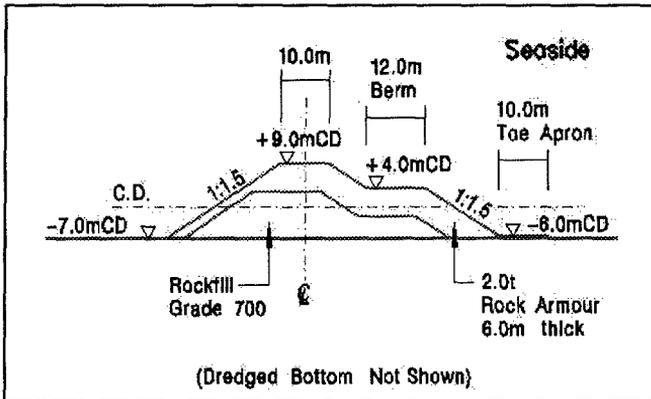


Figure 2. Typical Trunk Section of the Berm Breakwater

Physical Model Tests

To confirm the conceptual design, Scott Wilson was engaged by the Port Works Division of the Civil Engineering Department to undertake physical hydraulic model tests at the Hydraulics Laboratory of the National Research Council (NRC) in Canada. The testing programme made use of both two dimensional (2D) and three dimensional (3D) hydraulic model studies to assess the stability of the trunk and head sections respectively.

Testing Facilities and Model Set-up

The 2D berm breakwater stability tests and the 3D physical hydraulic model tests were carried out in the 14 m coastal wave flume and the multi-directional wave basin at NRC respectively.

Figure 3 illustrates the layout of the 2D model test structures in the 14 m flume (dimensions in model units), the irregular wave-generator and the location of the wave probes used to measure both the wave climate and reflection coefficients. Two model berm breakwater cross sections were tested (side by side) in the 2D model; one with a 12 m berm width and a second with a 18 m berm. As a consequence of this arrangement, both model structures were subjected to the identical wave climate.

Figure 4 illustrates the layout of the 3D model test structure in the Multidirectional Wave Basin, the location of the wave probes used to measure the wave climate and the location of the profile lines selected to record profile reshaping. The 3D model berm breakwater structure consisted of a 90 m trunk section, the full 75 m transition and the complete head. The wave basin is an indoor wave tank which is approximately 30 m by 20 m. The maximum depth of water is approximately 2.5 m. The segmented wave machine (60 individual paddle segments, total paddle length of 30 m) utilized in this basin is capable of producing model significant wave heights of 1 m, and can reproduce the interaction of irregular waves from different directions, i.e. truly three dimensional sea states. The basin is sufficiently large such that a 90° change in wave direction can occur at any point during the design storm without moving either the model structure or the wave machine. This means that many directions of wave attack may be modelled in this facility.

Because the bathymetry of the site at Hei Ling Chau is characterized by a very flat slope, the seabed topography for both 2D and 3D models was not modelled. That is, all tests were conducted with a fixed horizontal bottom located at elevation -7.0 mCD. The wave flume had a series of capacitance-type wave gauges located at strategic locations to measure the wave characteristics. The measured wave records were analyzed using zero crossing analysis and variance spectral density analysis to produce wave parameters and spectral plots respectively.

Both 2D and 3D physical model tests followed the Froude model law (U.S. Army, 1979). It consisted of an undistorted, two-dimensional, fixed bed model, with a geometric scale of 1:30. The model was sufficiently large to avoid Reynold's effects (U.S. Army, 1984).

Required Stone Sizes

Armouring requirements for the trunk section of the model structure was specified in the Project Brief as Type 13 with the following parameters:

Parameter	Specified	Acceptable Range
M_{15}	1000 Kg	1000 - 1500 Kg
M_{50}	2000 Kg	1900 - 2100 Kg
M_{85}	3000 Kg	2600 - 3800 Kg
M_{85}/M_{15}	3.0	2.5 - 3.3

Armouring requirements for the transition and head of the model structure was specified in the Project Brief as Type 14 with the following parameters:

Parameter	Specified	Acceptable Range
M_{15}	3200 Kg	3200 - 4000 Kg
M_{50}	4500 Kg	4300 - 4600 Kg
M_{85}	6400 Kg	5000 - 6500 Kg
M_{85}/M_{15}	2.0	1.8 - 2.1

The Type 13 stone used in the 2D tests was also used for the trunk section in the 3D tests. Model armour stone was sorted in the laboratory to yield a gradation which closely represents the above specified values for the Type 13 and Type 14 materials.

Waves, Water Levels and Wave Measurements

Irregular waves were synthesized by specifying a JONSWAP wave energy density spectrum with γ of 3.3 for both models. Wave measurement probes were positioned at various locations to measure water level fluctuations. These probes are representative of wave conditions generated by the wave machine, and the wave condition in front of the test structures. The specified wave climates and water levels for the 2D and 3D tests can be found in Tables 2 and 3 respectively.

Table 2 Specified Wave Climate and Water Level Conditions for 2D Tests

Run Segment	Significant Wave Height H_s (m)	Peak Wave Period T_p (s)	Water Level (mCD)	Number of Waves
Run 1	2.4	11	2.8	1000
Run 2	3.6	15	3.2	1500
Run 3	4.0	18	3.7	2000
Run 4	4.0	20	3.7	2000
Run 5	4.2	19	3.8	2000

Table 3 Specified Wave Climate and Water Level Conditions for 3D Tests

Run Segment	Significant Wave Height Hs (m)	Peak Wave Period Tp (s)	Direction of Wave Attack	Water Level (mCD)	No. of Waves
Run 1	2.4	11	30°	2.8	1000
Run 2	3.6	15	30°	3.2	1500
Run 3	4.0	18	30°	3.7	2000
Run 4	2.4	11	60°	2.8	1000
Run 5	3.6	15	60°	3.2	1500
Run 6	4.0	18	60°	3.7	2000
Run 7	4.0	20	60°	3.7	2000
Run 8	4.2	19	60°	3.8	2000

Prior to testing the stability of the model structures, the waves were calibrated to the specified conditions. That is, for each Run, the (initially) synthesized wave train was generated in the model basin for a limited period of time. Data samples were recorded and analyzed. As a consequence, adjustments of the synthesized wave trains were required to "tweak" the waves to match those specified wave climate (see Tables 2 and 3). However, when the generated wave heights (at the wave machine) were increased for both 2D and 3D tests, more waves broke offshore of the berm structure resulting in a reduced wave climate and a significantly transformed spectral shape. By studying the spectral plots collected at the laboratory, it is believed that the longer period waves underwent transformation to another spectral form due to finite depth conditions.

Berm Breakwater Profiling Device

An automated berm breakwater profiling device was installed and used in the basin to record profile development. The device is an extremely simple mechanism used to measure any 2D (X,Z) surface profile be it a berm breakwater profile or a beach profile. The device operates on the concept of rolling a wheel over a surface and tracking the X,Z location of that wheel with a real time recording system. To accomplish this, a post and beam assembly was installed over the berm breakwater as shown in Figure 5. On the top surface of the beam, a mouse is moved from one end of the beam to the other by means of a hand crank, which pulls the mouse by a cable in tension. The horizontal location of the mouse is measured by a linear potentiometer at a frequency of 20 Hz. This measurement is the mouse location X₁ (see Figure 5) relative to some fixed reference point.

Two Dimensional Wave Flume Tests

A series of 2D berm breakwater stability tests were carried out in the 14 m coastal wave flume. Under the synthesized wave climate, both the 12 m and 18 m model berm breakwaters performed well. The reshaped profiles did not encroach on the core material, and the crest and backslope model armour material were stable under all wave conditions specified. The toe protection armour stone was also determined to be statically stable under the tested conditions.

Van der Meer (1987) has developed the parameter "S" which relates to the "damage" measure of a conventional breakwater structure. In the context of berm breakwater, the parameter "S" does not relate to damage, but rather to the degree of reshaping of the profile. Van der Meer defines "S" as

$$S = \frac{A}{(D_{n50})^2}$$

where A is the eroded area and D_{n50} is the nominal diameter of the armour stone associated with the 50% value of the mass distribution curve.

The actual value of S is not important, but rather the rate of change of S with respect to the number of waves. Since S really represents the degree of reshaping (in the context of berm breakwaters), one is interested in determining at what locations in the storm profile the rate of change of S is large, i.e. the profile is reshaping significantly, and subsequently, when the rate of change of S becomes small, i.e. reshaping has stabilized.

Figure 6 shows the values of S as a function of cumulative waves for each berm structure. It is apparent from these curves, that the rate of change of S is greatest in Run 3, which indicates that the greatest reshaping of the berm profiles occurred during this storm segment.

On a previous project, it was shown that a 9 m berm in 2D tests reshaped in a dynamically similar manner as an 11 m berm in 3D (all other parameters held constant) (Fournier et al, 1990). Therefore, the 3D tests will be more conservative estimates of breakwater stability than the 2D counterparts. To confirm the stability of the test section, 3D model tests were conducted for the 12 m berm breakwater design.

Three Dimensional Wave Basin Tests

Under the synthesized wave climate, the model berm breakwater performed well. The reshaped profiles did not encroach on the core material, and the crest and backslope model armour material were stable under all wave conditions specified. The toe protection armour stone was also determined to be statically stable under the tested conditions.

During Runs 4 to 8, longshore transport of the armour material occurred due to the 60° oblique angle of wave attack. The material was transported along the trunk towards the transition. Because of this longshore transport, the trunk profiles in close proximity to the transition actually built up due to deposition of Type 13 stone. The source of the transported material was the trunk section closest to the wave machine. As a result, a scour hole was developed in Run 5. To maintain integrity of the test and to simulate the supply of armour stone from a structure much longer than that which was modelled, "artificial feeding" of Type 13 armour stone was performed at the scour hole location. The rate of stone feeding was roughly in accordance with the rate of scouring.

Conclusions

With the aid of the software BREAKWAT, a berm breakwater with 12 m berm width was adopted for the South Breakwater of the Hei Ling Chau Typhoon Shelter. The proposed berm breakwater was tested in a 2D wave flume and a 3D wave basin. In the 2D and 3D physical modelling tests, the proposed structure would reshape, but without excessive damage, under the design storm conditions. Both 2D and 3D tests confirmed that the proposed 12 m berm breakwater performed satisfactorily and the reshaped profiles did not encroach on the core material..

Acknowledgements

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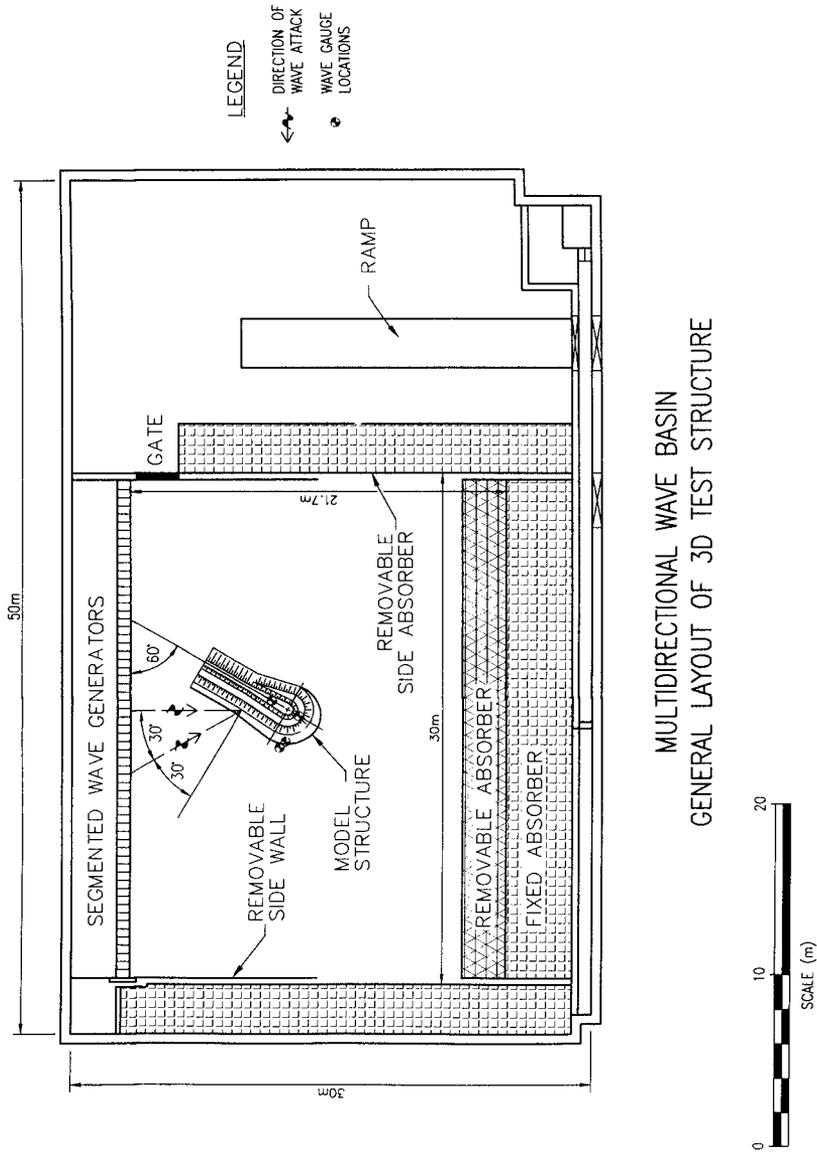


Figure 4a General Layout of 3D Test Structure

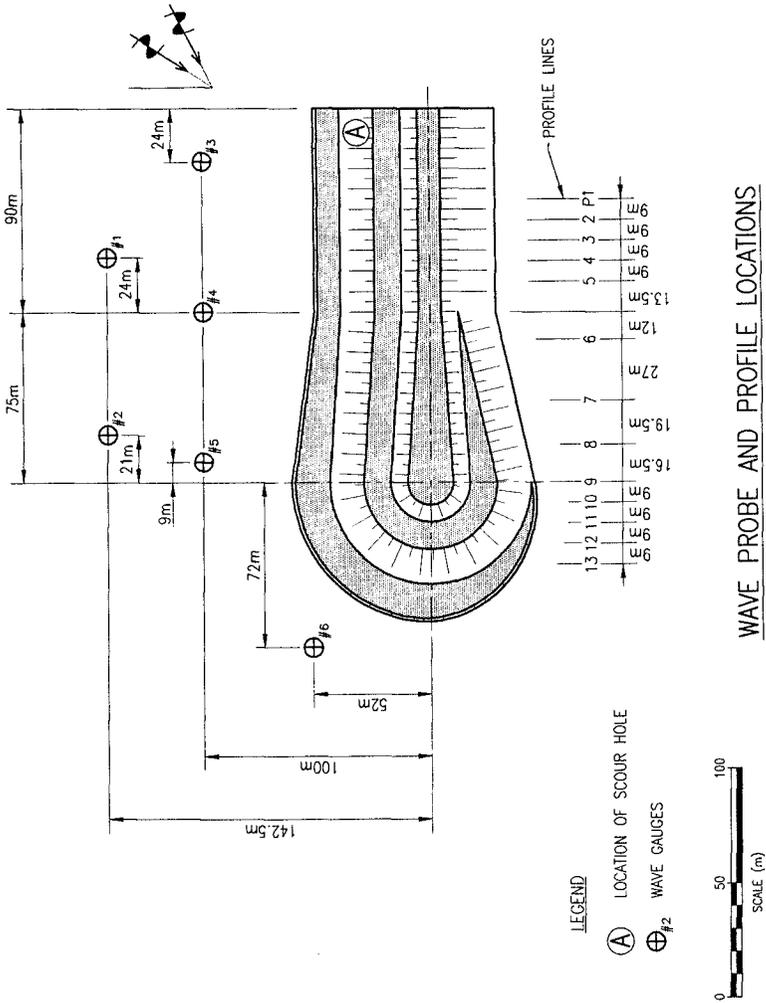


Figure 4b Wave Probe and Profile Locations

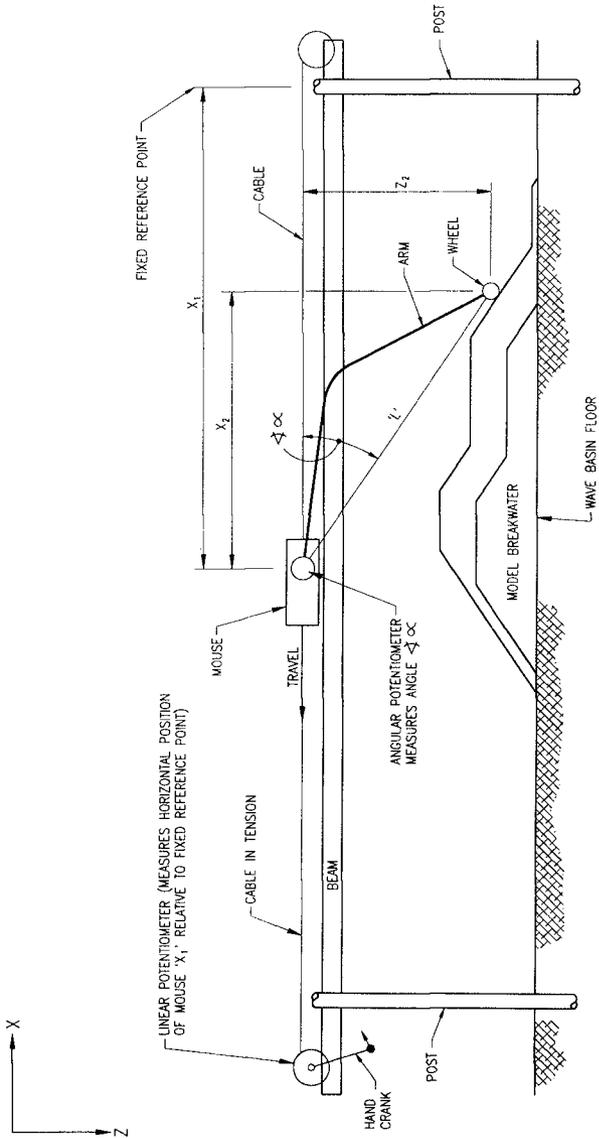


Figure 5 Schematic of Automated Berm Breakwater Profiling Device

Not to Scale
Concept Diagram Only

Physical Modelling for the South Breakwater of the
Hei Ling Chau Typhoon Shelter

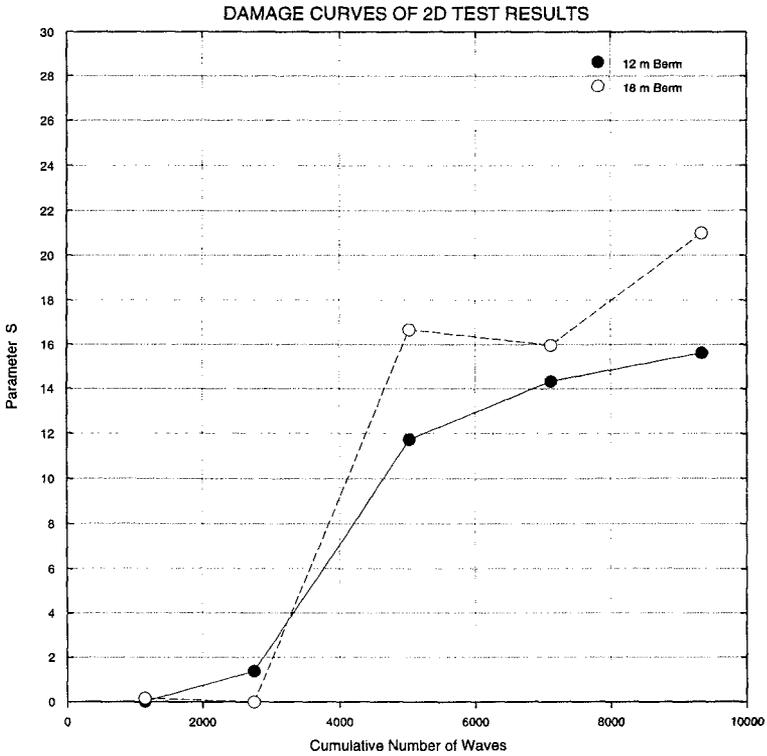


Figure 6 **Damage "S" Value Curves**