

THE EFFECTS OF CONSTRUCTION TECHNIQUES & BULK TERMINAL OPERATIONAL REQUIREMENTS ON THE DESIGN CRITERIA FOR ISLAND BREAKWATERS

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SYNOPSIS

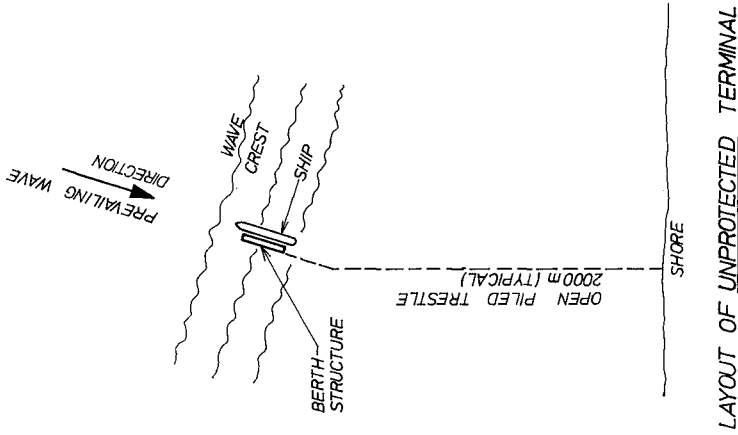
The title of the paper contains two important key words - CONSTRUCTION and ISLAND. The techniques for safe and economical construction of an island breakwater located some distance from the shore in exposed waters are radically different from the techniques required for conventional breakwaters with one end attached to the land. The differences in construction technique are so profound as to change the preferred basic design selection from rubble mound to caissons.

The paper discusses:-

1. The circumstances in which island breakwaters will become increasingly necessary for unloading of bulk ships.
2. Reasons why prefabricated caisson breakwaters are preferable for construction in an offshore situation.
3. The economy to be gained by using the caissons as the structural support for ship unloading machinery.
4. A consequent necessity to develop a caisson breakwater configuration with adequate design criteria to ensure total safety of the equipment supported on it.
5. A research program aimed at deriving these criteria.

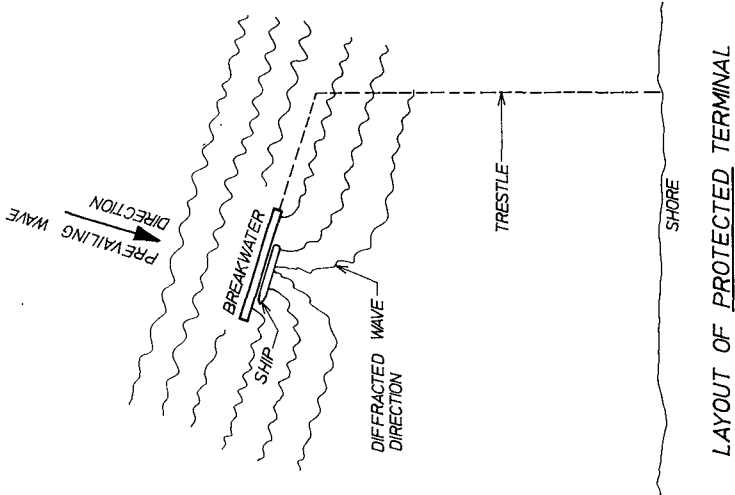
INTRODUCTION

The concepts discussed in this paper are based on the experience of Connell Eddie & Associates in the planning of two bulk unloading terminals, one in the Mediterranean and one in Western Australia. Both require deep water for large ships, and located 1500-2000 metres offshore outside existing sheltered ports.



LAYOUT OF UNPROTECTED TERMINAL

FIG. 1a



LAYOUT OF PROTECTED TERMINAL

FIG. 1

There is adequate experience world wide, and particularly in Australia to confirm that large bulkships can be loaded at exposed offshore terminals without requiring the protection of a breakwater.

There is no experience of unloading large bulk carriers other than in sheltered harbours. However the consensus of opinion is that unloading operations at an unprotected offshore berth would be rendered impossible in sea conditions much less severe than those which cause the cessation of loading operations.

The differences arise because loading equipment need never be in contact with the ship or its cargo, the bulk material being loaded through a telescopic chute suspended above the ship's hold. Therefore pitching, rolling and heaving of the ship have negligible effect on the loading operation. However, unloading equipment (grabs, screws, bucket chains etc) are immersed in the bulk cargo and hence movement of the ship can endanger the unloading equipment.

In the current world energy crisis situation, a rapidly escalating world trade in bulk coal can be expected, using ever larger vessels. Unloading of these vessels at existing ports causes high capital dredging costs and environmental problems, such as urban aesthetics involving coal stockpiles and power stations.

#### LIMITING OPERATIONAL CONDITIONS FOR BULK TERMINALS

In planning an offshore bulk terminal (i.e. one which is not in a sheltered harbour), the most important issue is whether a breakwater is necessary.

FIGS 1 & 1a show typical arrangements of protected and unprotected terminals respectively. In typical circumstances, such terminals are about 2km offshore.

The protected terminal is at right angles to the dominant wave direction, whilst the unprotected terminal is oriented in the same direction as the dominant waves. In both cases the waves arriving at the ship are travelling in the direction of the longitudinal axis of the ship and hence cause minimum disturbance to the ship.

In the case of the unprotected terminal, waves varying from the dominant direction will cause significantly increased ship disturbance. In the breakwater case, a change in wave direction has little effect because, (subject to adequate breakwater length) the diffracted waves are still travelling parallel to the breakwater when they arrive at the ship position.

Thus a breakwater has two effects which reduce the disturbance of the ship

- . Attenuation of wave height;
- . Virtual elimination of increased ship disturbance due to variation in wave direction.

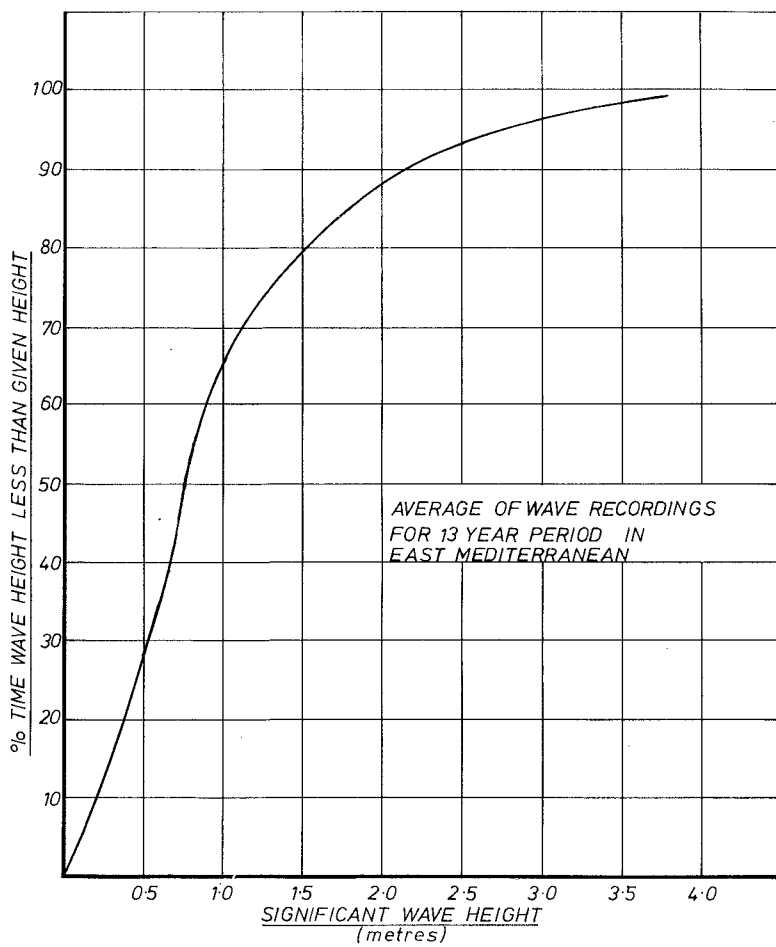


FIG. 2.

Experience at a number of Australian loading terminals has provided good information on the limiting wave conditions which cause loading operations to cease. For unloading terminals there is no information available world-wide, but enquiries made of a number of operators and equipment manufacturers suggest the figures in TABLE 1.

TABLE 1

Wave direction relative to ship heading	Limiting Wave Height (metres)	
	Loading	Unloading
0°	2.5	1
45°	1	0.5

There are of course many other factors affecting the limiting conditions for safe operation (e.g. wave period, wind, ship size and inertia characteristics).

A full analysis would include all these factors, but the authors believe that a good first approximation to the berth availability equates with the amount of time each year that the waves fall into the categories shown in TABLE 1.

At offshore loading terminals so far constructed in Australia (1980) sufficient berth availability has been obtained without using a breakwater. Availability is normally required in the range 90-95%, but in some circumstances (e.g. low annual throughput tonnage) much less may be quite satisfactory. In periods of unavailability, the berth is shut down and if there is a ship in port it is taken off and anchored in deep water awaiting favourable weather. This type of off-shore terminal operation differs from the usual concept of all-weather port operations and must be understood in the context of this paper.

The planner of an unloading terminal is much less likely to find adequate berth availability if the berth is unprotected by a breakwater. FIGS 2, 3 & 3a show a typical wave climate such as the authors found at a proposed terminal site in the Mediterranean. If no breakwater is provided, the availability of an unloading terminal would correspond to the wave conditions shown in the last column of TABLE 1. Thus FIG 2 suggests an availability of 65%. However, FIG 2 represents the average of 13 years wave recordings and the authors wish to draw attention to the dangers of basing terminal planning on average statistics.

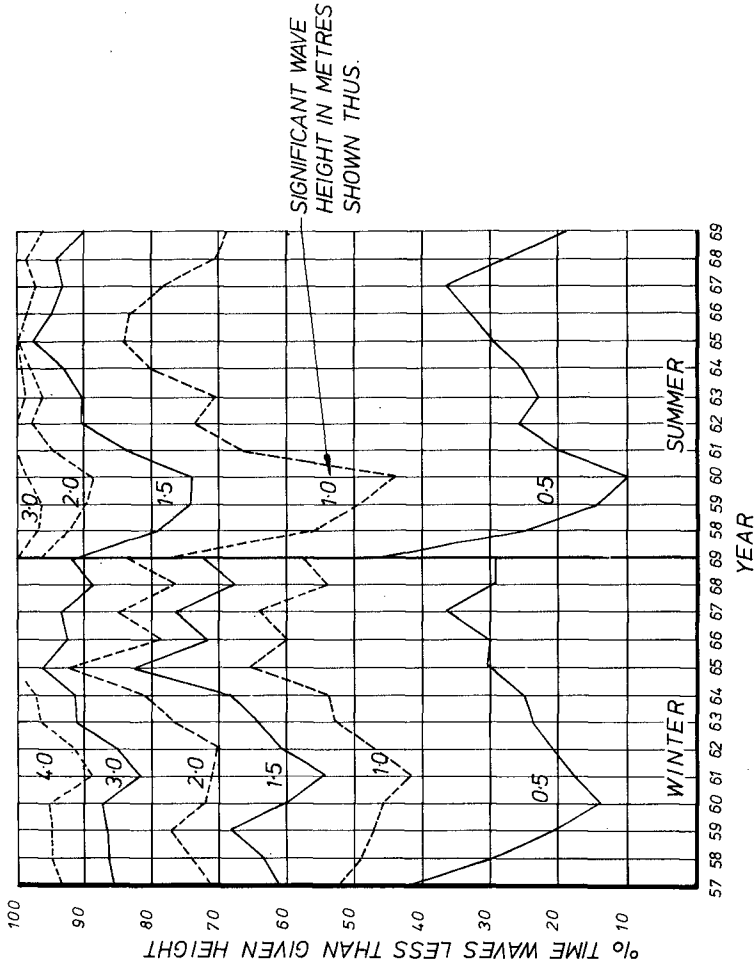


FIG. 3

FIG 3 shows the same figures broken down into winter and summer for each of the 13 years separately. This shows that in a bad winter (such as 1961) availability would be only 42%. FIG 3a shows the 1961 winter figures broken down into wave directions, and again using the criteria of TABLE 1, it can be seen that the availability is reduced to 30%.

In these circumstances, a high annual throughput installation would not be feasible with a single unprotected berth. Even a two-berth installation, whilst providing in theory sufficient unloading time, would be subject to long shut-down periods of both berths simultaneously. Accordingly, the authors believe that a single berth protected by a breakwater is the preferred solution.

The layout shown in FIG 1 is consequently the subject of this paper. It will be seen from FIG 3 that if the breakwater is capable of attenuating 4 metre incident waves to 1 metre at the berth, there will be 90% availability even in the worst recorded winter period. (As stated earlier, the variation of incident wave direction does not significantly affect a breakwater protected terminal).

Such an installation would be operated in the same way as the existing unprotected loading terminals in Australia - i.e. when the sea conditions prevent operations, a ship in berth would be taken off and anchored in deep water to await favourable weather.

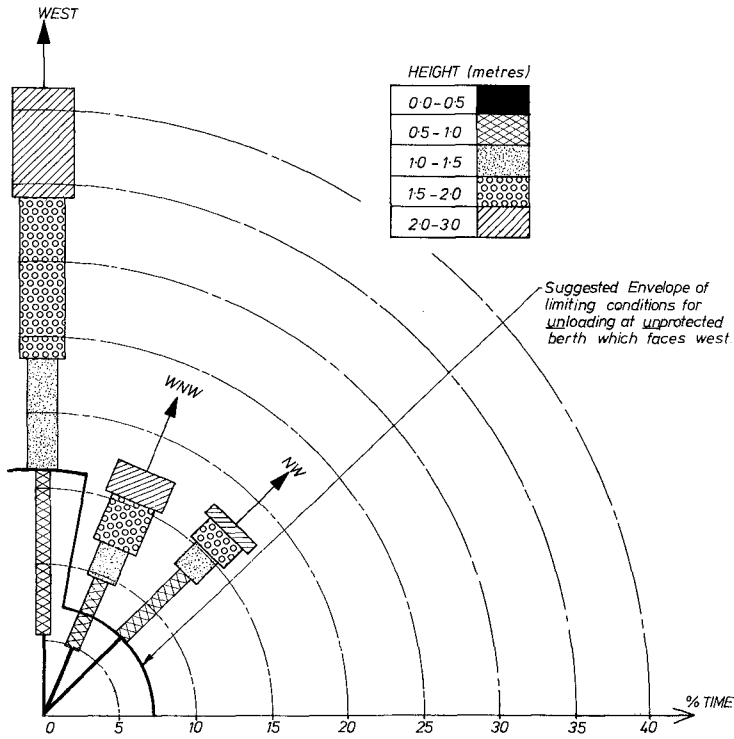
#### DESIGN & CONSTRUCTION CONSIDERATIONS

New style offshore terminals satisfying the aforementioned availability requirements give rise to design and construction problems which necessitate further research.

The proposed offshore terminal differs from traditional concepts in two important ways.

1. The breakwater is not required to provide a safe anchorage on its lee side for 100% of the time, but only when the incident waves are 4 metres high or less. Operation of the terminal is not required when waves greater than 4 metres are allowed to overtop. This concept is termed a "limited availability breakwater".
2. The breakwater is an island structure, and this raises construction problems which have a profound effect on the design concept.

The required island breakwater is in fact connected to the shore by a piled trestle. For operational purposes, this trestle carries one or more conveyor belts and a roadway. Thus it may be argued that this trestle can be built first and used for access by trucks to construct a conventional rubble mound breakwater by end-over-end tipping.



FIGURES SHOW PERCENTAGE OF TOTAL TIME WAVES ARE WITHIN STATED HEIGHT AND DIRECTION RANGE WORST RECORDED WINTER PERIOD.

FIG. 3a



However, in practice this is un-economic for two reasons. Firstly it requires an unacceptably long time to construct the trestle and breakwater sequentially. Secondly the roadway on the trestle would have to be strengthened significantly above the standard required for operation of the berth.

The fundamental economics of construction dictate that the break-water must be constructed as an offshore operation, without use of the trestle for construction access. An offshore operation is defined by the authors as an operation in exposed waters with no access during construction other than by boats or helicopters. This requires the use of floating construction equipment and daily use of barges, tugs and other vessels for the transfer of men and equipment. These activities cannot generally be carried out safely when the waves exceed about 1 metre. Thus it can be seen that in a typical situation such as shown in FIGS 2 and 3, there may be an average of 35% downtime, with a maximum of 58% in a bad winter and a minimum of 16% in a good summer. These figures are not untypical of previous experience in the construction of offshore loading terminals (Ref.1).

Weather down-time in an offshore operation is very costly because it shuts down the entire construction establishment, which after each shut-down requires some time to restart. Furthermore, the wide variability of shut-down time from 16% to 58% makes effective planning of the operation very difficult.

In offshore operations such as construction of the North Sea oil platforms, these problems are so obvious that there is no economic alternative to total onshore pre-fabrication of the structure followed by very rapid offshore installation. The whole purpose of this concept is to reduce the required offshore working time to a minimum by making maximum use of onshore pre-fabrication.

The type of offshore terminal shown in FIG 1 shares the same problems. Instead of being 100km or more from the shore like an oil platform, it is only 2km. Therefore whilst it becomes feasible to adopt "conventional" construction such as would be used for the same structure if it were in sheltered waters, the authors believe that maximum onshore pre-fabrication is economically essential. In the context of a break-water this means caissons. The "conventional" alternative of a rubble mound requires the piecemeal offshore construction of both the break-water and a complete separate berth structure.

A number of eminent authorities have pointed out the disadvantages of caisson breakwaters (Ref. 2). It is the primary purpose of this paper to draw attention to the construction advantages of pre-fabricated caissons for the particular case of a detached bulk unloading terminal, and the consequent necessity to develop design criteria which adequately overcome the various objections to caissons. In the authors' opinion, the disadvantages of caisson breakwaters apply almost entirely to vertical wall fully reflecting structures. Caissons do not need to be fully reflecting or to have vertical walls.

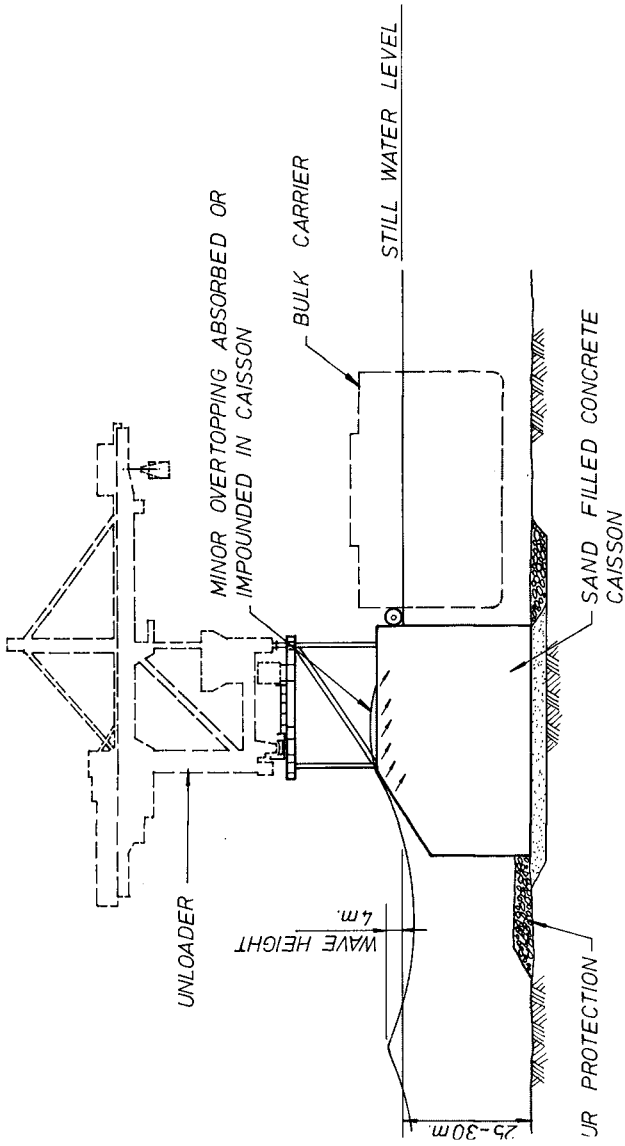


FIG 4 (a)

Previously published work has suggested the concept shown in FIG 5 for the type of terminal shown in FIG 1. The Danish Hydraulic Institute (Ref 5) have recognised the advantages of an overtopping breakwater but suggested that a high reflecting wall, as shown in FIG 5, is necessary in cases where equipment is mounted on the caisson.

However, the maximum design wave load on the caisson depicted in FIG 5 can be shown to be 2 or 3 times that on a sloping face overtopping caisson, and furthermore it suffers all the other disadvantages of vertical wall structures, such as scour at the base.

Accordingly, the authors have developed the concept shown in FIGS 4a & 4b. This meets all the operational availability requirements discussed earlier. Incident waves up to 4 metres high are diffracted round the ends but do not overtop, creating safe conditions for unloading. Waves higher than 4 metres pass over the caissons but under the elevated plant deck. Under these circumstances the berth is shut down and ships taken off to await favourable weather.

The concept is based on the type of breakwater developed by the Danish Hydraulic Institute (Ref 5) but goes a step further by mounting the berth structure on an elevated deck. This retains all the advantages of a sloping face overtopping caisson without endangering the equipment.

The Danish Hydraulic Institute's published work (Ref 5) also draws attention to the fact that much of the advantage of a sloping face breakwater is lost when the tide level falls below the bottom of the sloping section.

The authors consider that from the construction point of view it is not essential to extend the vertical wall up to still water level. Accordingly, it is proposed to include in the research program the concept shown in FIG 6, with the slope extended below low tide level. Such a caisson would require temporary buoyancy tanks to assist flotation and installation. Its potential advantages are further reduction of the maximum wave load on the caisson, and reduced scour.

The paper so far has discussed operational and constructional requirements and demonstrated that these factors combine to create a need for research to establish suitable design criteria for the concept shown in FIGS 4 & 6.

The next part of the paper discusses the requirements of the research program proposed by the authors.

#### PROPOSED RESEARCH AND DEVELOPMENT

The proposed island breakwater concept (see FIGS 4 & 6) has to meet three principal design criteria:

- (i) its geometry must ensure that the wave climate at the berth behind the breakwater provides the required percentage time availability for unloading, i.e. it must have the specified attenuation factor.

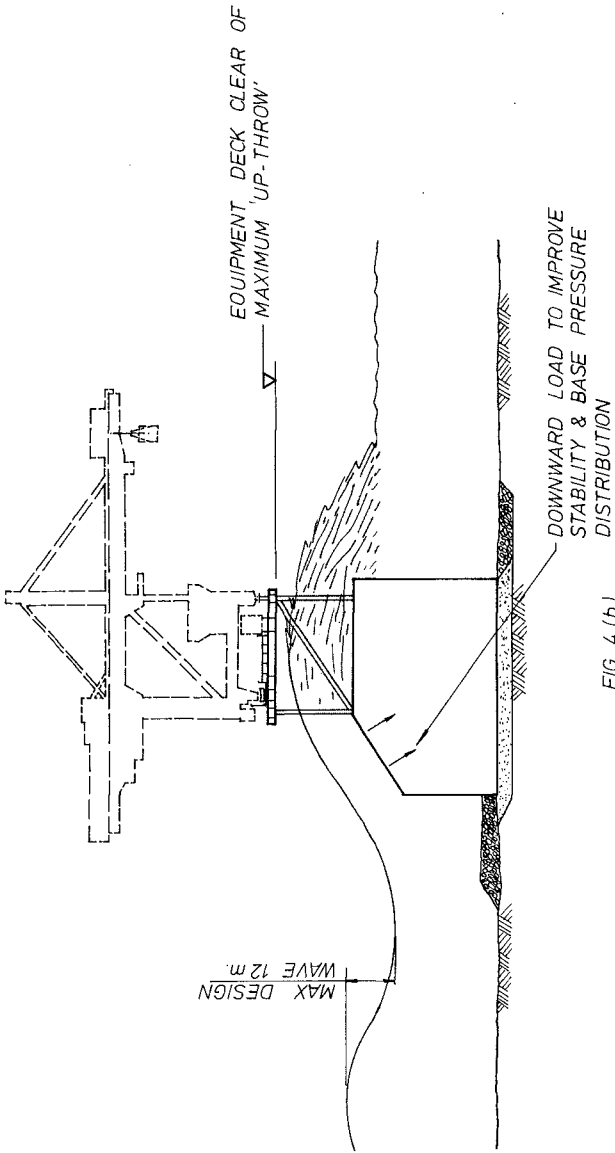


FIG 4 (b)

- (ii) the breakwater must have an adequate factor of safety against a stability failure commensurate with the risk to personnel and unloading equipment.
- (iii) the breakwater geometry should minimise the "up throw" of overtopping waves in order to limit the require deck elevation.

These three criteria require opposing solutions with respect to the breakwater's height and seaward wall geometry, and hence a compromise design must be selected. In general terms a profile which minimises the loads on the structure will also create more overtopping.

#### Breakwater Stability

The basic requirement of the breakwater stability design comprise:

- (a) provision of the required factors of safety against sliding and overturning of the breakwater when subjected to the maximum design wave.
- (b) prevention of scour at the base such that the foundations are not progressively undermined.

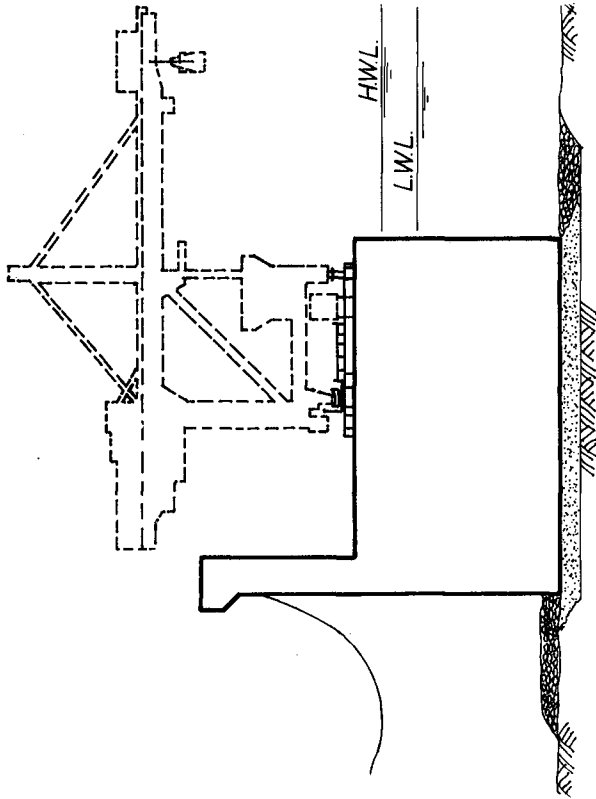
The authors consider that ideally the factor of safety against an overturning failure or base failure, which would have catastrophic consequences for the installation, should be significantly higher than that against sliding. A slide of a few centimetres need not be classed as irreparable damage as it can be rectified by adjustments to the rails carrying the ship unloading equipment. It is of interest to note that most of the documented caisson breakwater failures have in fact involved slides (Ref 3).

#### Effect of Breakwater Profile with Respect to Stability

If a vertical wall caisson is first considered the wave forces can, as expected, be significantly decreased by allowing the higher waves to overtop instead of totally reflecting them. The graphs in FIG 7 were derived using the MICHE-RUNDGREN theory with the approximations discussed in Refs. 3 & 4. Both the horizontal force and the overturning moment are lower for the overtopping structure. More importantly, the rate of increase of moment with respect to wave height, is significantly reduced. Hence, the overtopping breakwater is considerably less sensitive to an unforeseen increase in the design wave height.

These points are also demonstrated by previous model test results (Ref 5) particularly as the wave conditions approach breaking.

It is possible to further reduce the wave forces and scour erosion of the overtopping breakwater by providing an inclined face to the seaward wall.



FULLY REFLECTING VERTICAL WALL  
BREAKWATER

FIG. 5

Model tests (Ref 6) show that the pressure distribution acting on an overtopping inclined face breakwater are of the form shown in FIG 8. This diagram demonstrates another advantage of allowing waves to overtop, namely the stabilising effect of the downward component of the water pressure acting on the inclined face. Besides increasing the friction resistance to sliding this downward component can also be used to counteract some of the overall overturning moment.

Furthermore, model tests have shown that the shock pressures as well as the overall horizontal wave force can be reduced by inclining the seaward wall (Ref 5).

The model test results in Ref 6 indicate that by inclining the front face, the uplift force is also reduced, with further stability benefit.

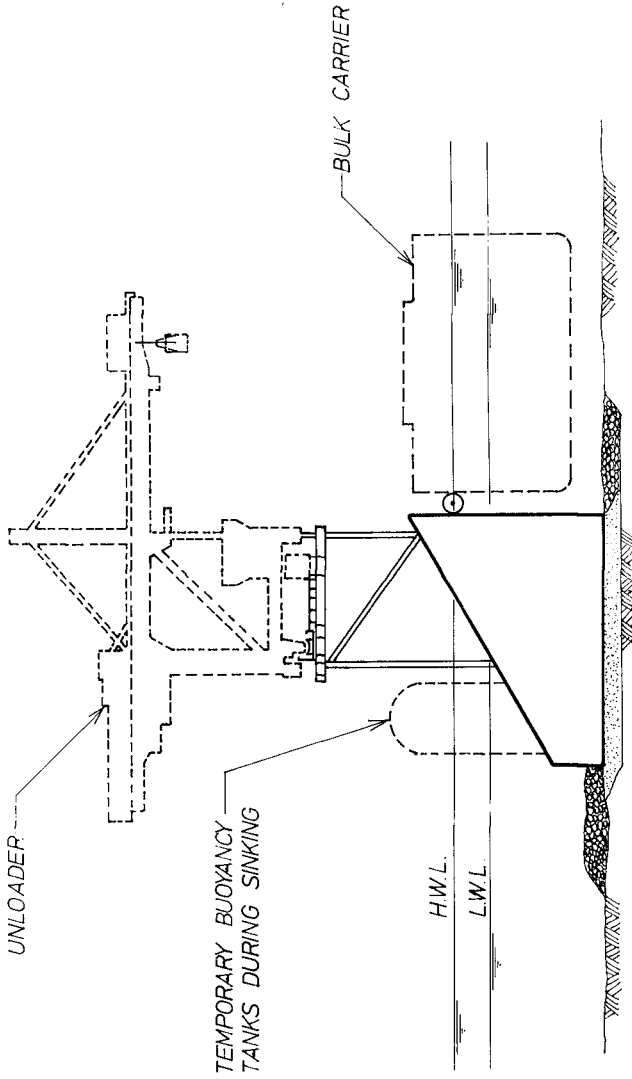
The inclined wall breakwater has a further advantage in relation to scour effects. Waves breaking against a vertical wall are deflected both upward and downward and the latter effect results in high water velocities at the seabed which can lead to foundation erosion (Ref 4). The downward effect is substantially reduced by an inclined seaward face.

#### PROPOSED MODEL TEST PROGRAM

It can be seen that the attenuation and stability criteria require opposing solutions with respect to the breakwater's profile. Hence a model test programme is proposed to investigate optimum profiles.

A summary of recent relevant literature (Ref 5) suggests the following criteria may be pertinent to the test program:

- (i) physical models are valuable in deriving design criteria for stability of structures and wave forces on structures.
- (ii) the design of a shock-sensitive structure requires that the inertia of the structure and the stiffness of the foundation be taken into consideration in a dynamic analysis. Further, it is common that the natural period of a caisson-type breakwater on its soil foundation is about an order of magnitude smaller than the mean wave period, thus suggesting that the characteristics of the individual wave are all-important.
- (iii) acceptable values of shock forces may be expected on physical models of caissons providing air entrainment is unimportant in both model and prototype. Further, the shock forces are sensitive phenomena such that a large scatter in results is likely unless repetitive tests are carried out - 1000 waves minimum and preferably 3000-5000 waves have been suggested. Even with a random or model-reproduced natural wave train generation, due care should be taken in accepting one single random field record due to the large scatter in force reproductibility.



PROPOSED ALTERNATIVE  
BREAKWATER

FIG. 6



- (iv) the maximum force on a vertical-faced structure increases with wave height up to the stage where the wave breaks on the face and begins to overtop. For higher waves the maximum force remains approximately constant or reduces.
- (v) sloping face structures cause less reflection than vertical faces because of energy loss and likely overspill.
- (vi) there are problems with model tests of reflecting structures due to re-reflections within the wave tank.
- (vii) wave height and water level with respect to the structure are the most important independent variables for structural stability analysis.
- (viii) for the same wave height, the longer the period the larger is the force.
- (ix) it is commonly considered erroneous to design structures on the basis of tests with regular waves of design height.

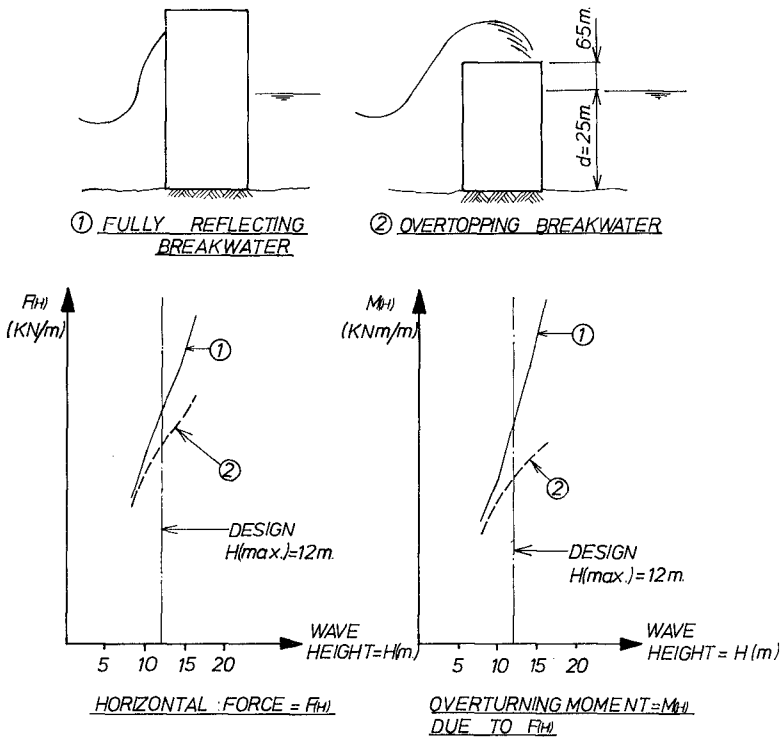
A wave tank 60 metres long and 1.83 m x 1.83 m in section will be used to contain physical models of scale 1:50 to study waves of height up to 5m and periods up to 30 sec. in water up to 50m deep. A larger model (scale 1:20) will be used for confirmatory pressure distribution studies, with waves up to 2m high. In view of the criterion (ii) above applying, and despite criterion (ix), regular waves are considered satisfactory for these tests, recognising re-reflections as a problem.

Additional to the physical tests, it is planned to examine the fully overtopping phenomenon by means of numerical techniques for wave force determination. This will provide valuable guidance as to the type of profile likely to produce the best results in the physical models.

The cross-section configuration of the caisson will likely be similar to that in FIGURES 4 & 6. However the length and angle of sloping front face, level and length of top face of caisson, freeboard height above caisson to plant deck are all variables to be examined. A free-draining wave-absorbing stilling basing set into the top face of the caisson may also be examined, as a possible way of lowering the caisson profile whilst still preventing overtopping.

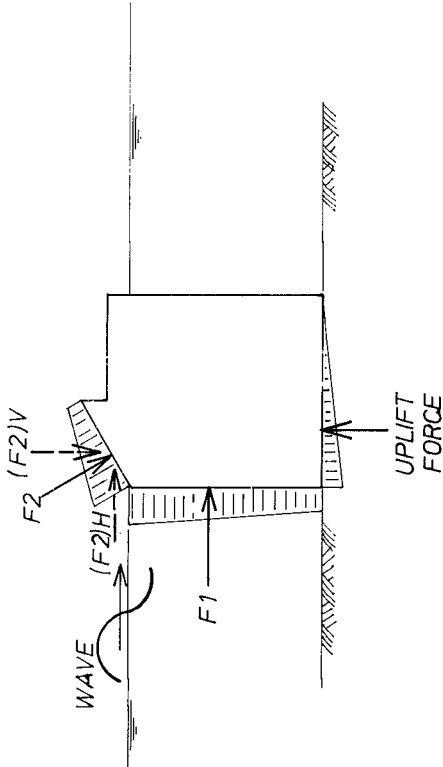
It is too early to report the findings of these various studies in this paper. Instead, comment is invited on, especially, the usefulness and pitfalls associated with the numerical modelling envisaged.

This paper does not consider the very important issue of foundation strength and stability. Apart from considerations of sliding, shearing, overturning, and uplift pressure distribution, the foundation will be subject to erosion forces, the severity of which will depend on wave climate, water depth, and foundation configuration and composition.



NON-BREAKING WAVE FORCES USING MICHE - RUNDGREN FORMULAE (12 SECOND PERIOD WAVE)

FIG. 7



TYPICAL WAVE PRESSURE  
DISTRIBUTION FOR OVERTOPPING  
BREAKWATER

NOTE:

The forces shown are those additional to the hydrostatic and gravity loads.

FIG. 8.

CONCLUSIONS

- . The construction of bulk unloading terminals in deep water will require the use of detached island breakwaters.
- . For construction reasons related to the delay risks when working in an offshore situation, pre-fabricated caisson breakwaters are preferable in this situation.
- . Significant cost savings can be achieved if the caissons forming the breakwater are also used as the structural support for the ship unloading equipment.
- . Significant further cost savings can be achieved if the highest 5% to 10% of waves are allowed to overtop the caisson.
- . Sloping face caissons supporting an elevated plant deck above the crest of the highest overtopping waves offer the most attractive solution.
- . Further research is required to define wave loads and other parameters with an adequate degree of assurance for safety of the equipment. The paper makes recommendations for the required research.

REFERENCES

1. EDDIE, A.G.F., GERWICK, Prof. Ben C., LEE, J.O.T. - 24th International Navigation Congress, Published by PIANC, Leningrad 1977. Design and Construction Aspects of Exposed Offshore Bulk Mineral Export Terminals in Australia.
2. PIANC - Final Report of the International Commission for the Study of Waves
3. GODA, Yoshimi - New Wave Pressure Formulae for Composite Breakwaters Proc 14th Coastal Engineering Conference, Copenhagen, 1974.
4. U.S. ARMY COASTAL ENGINEERING RESEARCH CENTRE - Shore Protection Manual, Volume II, 1975.
5. GRAVESEN, H. - Published by Danish Hydraulic Institute, January 1979, Design of Caisson Breakwaters
6. BRAGADIN, G.L. - Published by International Association for Hydraulic Research XVII Congress - Baden-Baden, 1977, Hydraulic Behaviour of Overflow Breakwaters.